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Subject: MnDOT LRFD Bridge Design Manual Update

The MnDOT Bridge Office LRFD Bridge Design Manual (BDM) is available for download in Adobe PDF at:
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Instructions (for two-sided printing):

1. Remove from the BDM:
    - Title Page
    - Table of Contents pages i through xii
    - Entire Section 2
    - Section 5: Pages 5-3 & 5-4, 5-7 & 5-8, 5-29 through 5-32, 5-43 & 5-44, 5-93 & 5-94, 5-111 & 5-112, 5-119 through 5-136
    - Entire Section 9
    - Section 11: Pages 11-5 &11-6, 11-9 through 11-14, 11-19 through 11-22, 11-27 through 11-30, 11-35 through 11-38, 11-81 & 11-82
    - Entire Section 14
2. Print and insert in the BDM:
    - Title Page
    - Table of Contents pages i through xii
    - Entire Section 2
    - Section 5: Pages 5-3 &5-4, 5-7 & 5-8, 5-29a through 5-32, 5-43 & 5-44, 5-93 & 5-94, 5-111 & 5-112, 5-119 through 5-175
    - Entire Section 9
    - Section 11: Pages 11-5 &11-6, 11-9 through 11-14, 11-19 through 11-22, 11-27 through 11-30, 11-35 through 11-38, 11-81 & 11-82
    - Entire Section 14
    - Memo to Designers #2023-01

BDM Update Summary

Revisions in the "NOVEMBER 2023" update consist of the following changes:

Section 2 (THE ENTIRE SECTION HAS BEEN REPUBLISHED, SO LOOK FOR **1** VERTICAL BAR IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 2.1.3: Under **General Criteria for Lateral Clearance** and **Lateral Clearance for Roadways**, updated Technical Memorandum reference.
- Article 2.3.2: Under **Abutment and Pier Locations**, updated Technical Memorandum reference.
- Article 2.4.1.4: Updated MnDOT Standards Plans Manual figure reference for bridge approach panels.
- Article 2.4.2: Added sentence to clarify which informational sheets should be included in final bridge plans.
- Article 2.4.2.6: Removed paragraph which excludes box culverts from typical bridge standard detail procedure.
- Article 2.5.2.1: Updated Figure 2.5.2.1.1 "Repair Paving Bracket" to depict more concrete removal beyond first layer of reinforcement.
- Appendix 2-C and 2-D:
  - Added construction notes designating bar mark suffixes for epoxy coated 4% chromium and GFRP reinforcement bars.
  - Added mass concrete and deck pour sequence notes.
  - Updated notes from passive to active voice.
  - Several other miscellaneous updates to existing notes.

Section 5 (LOOK FOR **3** VERTICAL BARS IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 5.1.1: Added **Mass Concrete Elements**, which describes how to identify and label mass concrete in bridge plans. Updates an existing table number.
- Article 5.2.2: In the first paragraph, added language that standardizes the suffix for epoxy coated 4% chromium and GFRP reinforcement bars.
- Article 5.4.2: Updated the pretensioned beam maximum tension stress limit at transfer. This included adding Table 5.4.2.1 which lists the minimum bonded reinforcement requirement for new tension limit.
- Article 5.4.3: In second and third bullet points, updated guidance on use of draped strands.
- Article 5.7: Added design example for pretensioned debonded concrete I-beam. Updated title and article numbers of existing design examples.
- Article 5.7.2:
  - Added "Draped" to article name to differentiate this example from the new debonded strand example.
  - Under ***E. Design Beam Pretensioning With Draped Strands for Control of End Stresses***, corrected calculation error in **Check Stresses at Midspan After Losses**.
  - Under ***F. Design Reinforcement for Shear***, corrected equation error in **2. Interface Shear Transfer**.
  - Under ***G. Design Pretensioned Anchorage Zone Reinforcement***, updated design splitting force and subsequent calculations in **Splitting Reinforcement**.
  - In Figure 5.7.2.5, updated to current beam standard and fixed errors.

- Article 5.7.3: Added entirely new prestressed beam design example with debonded strands. This example closely mirrors the conditions of Article 5.7.2 to highlight the difference in design between draped and debonded strands.
- Article 5.7.4: Renumbered article, intended for future manual content, to accommodate the new prestressed beam design example with debonded strands.

Section 9 (THE ENTIRE SECTION HAS BEEN REPUBLISHED, SO LOOK FOR **1** VERTICAL BAR IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 9.2.1: Under **Detailing**, updated guidance on deck and raised sidewalk detailing over piers with prestressed concrete beams. Updated Figure 9.2.1.10.

Section 11 (LOOK FOR **2** VERTICAL BARS IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 11.1: Under **Detailing/Reinforcement**, revised dimension line for note ② to top of abutment stem (previously bottom of beam).
- Article 11.1.1: In multiple locations, updated the shape of the approach panel to bridge dowel bar.
- Article 11.1.2: Under **Geometry**, updated guidance for minimum abutment stem thickness when rustication is required. In multiple locations, updated the shape of the approach panel to bridge dowel bar.
- Article 11.1.3.1: In Figure 11.1.3.1.1, revised figure based on updated Service Life Design Guide and mass concrete requirements. Updated the shape of the approach panel to bridge dowel bar.
- Article 11.1.3.2: In Figure 11.1.3.2.1, revised figure based on updated Service Life Design Guide and mass concrete requirements. Updated the shape of the approach panel to bridge dowel bar.
- Article 11.1.5: Revised approach panel design guidance based on updated standards. Updated the shape of the approach panel to bridge dowel bar. Added guidance for cantilevered approach panels.
- Article 11.4.1: Under ***H. Check Shear in Footing***, corrected equation error in **Check Two-Way Shear in Footing**.

Section 14 (THE ENTIRE SECTION HAS BEEN REPUBLISHED, SO LOOK FOR **1** VERTICAL BAR IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 14.2.3: Added language that describes length requirements for modular expansion joints without a field splice.
- Article 14.2.4: Entire article updated to clarify the required geometry and detailing practices of expansion joint devices for various railing types and skew magnitudes.
- Article 14.3.4: Added last paragraph which requires designer to calculate coordinates for disc bearing blockouts and include them with the final calculations.
- Article 14.7: Added columns and/or notes to Table 14.7.1, 14.7.2, 14.7.4, and 14.7.5 that contain anchor rod and pintle shear capacity. Anchor rod and pintle shear capacities were added to the BDM due to the values being removed from the B-Details.
- Article 14.8.1: Under ***D. Anchor Rods/Pintles***, clarified the design procedure for calculating anchor rod and pintle capacity. Updated Figures 14.8.1.4 and 14.8.1.5 to current B-Details.

Memos

- Addition of new Memo to Designers #2023-01 regarding the design of *Debonded Strands in Prestressed Concrete Beams*

For ***technical questions*** regarding this transmittal contact Karl Johnson, Bridge Design Manual Engineer, at karl.johnson@state.mn.us.

Edward A. Lutgen

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State Bridge Engineer

MANUAL

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MINNESOTA DEPARTMENT OF TRANSPORTATION

Bridge Office

LRFD Bridge Design Manual

MnDOT BRIDGE OFFICE

LRFD Bridge Design Manual

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APPENDIX A. Memos to Designers

2. GENERAL DESIGN AND LOCATION FEATURES

The design of a bridge typically takes place in two major phases of work: preliminary design and final design. During preliminary design, the structure type, the foundation type, the aesthetics, and the primary geometry for the bridge are determined. During final design, specific details for all elements of the bridge are developed and presented in the plan set. These details include material descriptions, quantities, and geometric information. Final plan sets are typically assembled in an order that roughly follows the order of construction, from the ground up.

This section of the manual contains a large amount of information useful for the preparation and assembly of plans for a project. To facilitate the production of plans and standardize the content of bridge plan sets, the Bridge Office has developed special provisions, standard bridge details, standard plans, standard plan notes, and standard pay items.

Guidance for the design of specific structural elements (e.g. beams, abutments, piers, etc.) is provided elsewhere in the manual.

2.1 Geometrics

Definitions

For discussion of bridge geometrics in this section, roadways are classified as Mainline Highways, Ramps, Local Roads, and Local Streets. Each of these four groups is further classified under either Urban or Rural Design.

The following definitions apply:

- Mainline Highways – Roadways that carry through traffic lanes for freeways, expressways, and primary and secondary highways.
- Local Roads – Rural roads off the state trunk highway system.
- Local Streets – Urban roads off the state trunk highway system.
- Ramps – Segments of roadway connecting two or more legs at an interchange.
- Urban Design – Roadways with curbs on the right and/or left sides.
- Rural Design – Roadways without curbs.
- Median Width – The distance between the inside edges of opposing through traffic lanes.
- Auxiliary Lane – A lane adjoining a through traffic lane for a purpose supplementary to through traffic movement such as truck climbing, weaving, speed change or turning.

2.1.1 Bridge Geometrics

General Criteria

The width of the bridge deck and the typical section at the bridge undercrossing are determined by the classification and geometrics of the approaching roadway, together with appropriate design considerations for shoulder needs. The geometrics of the approaching roadway are to be carried over and under the bridge to the maximum extent practicable.

Bridge width requirements are a function of the lane and shoulder widths of the approaching roadway, together with assessment of pedestrian and bicycle needs, multimodal requirements, user safety requirements, drainage requirements, staging, and other project specific considerations such as snow storage and emergency vehicle access. The determination of the appropriate width for each project requires study of specific project needs. Detailed decision documentation is required by the Roadway Designer during the preliminary design phase and must be coordinated with the Preliminary Bridge Plans Engineer. Bridge shoulder and lane widths should be included with project design element documentation in the District project design memo, including informal design exceptions as necessary.

The discussion of geometric details included in this section describes bridge deck geometrics separately from bridge undercrossing geometrics.

Application of Standards

Unless stated otherwise, the geometrics discussed in the following articles apply specifically to new work. However, use of these geometrics is also highly desirable when upgrading or widening existing facilities and should be incorporated in those situations also. For bridge repair projects, see the *Bridge Preservation and Improvement Guidelines*, found on the MnDOT Bridge Office web site, for more information. Bridge deck geometrics on the local road system must comply with *State Aid for Local Transportation Operations Rules*, Chapter 8820. Note that an exception to this applies to local road system bridges at an interchange with a trunk highway, which must be designed to trunk highway standards between the ramp terminals on the local road.

Responsibility

The Preliminary Bridge Plans Engineer will be responsible for assuring that the geometric standards in this section are followed. Where a deviation from the standard is necessary, a written description of the deviation shall be prepared by the Preliminary Bridge Plans Engineer and submitted to the State Bridge Engineer for approval prior to submitting the Preliminary Bridge Plan for signature.

2.1.2 Bridge Deck Requirements

Bridge Width Criteria

Roadway cross sections that approach bridges will normally provide a clear zone outside the travel lanes as a recovery area for out-of-control vehicles. It is typically not economical or practical to carry these clear zones across bridges. Therefore, crash-tested bridge railing is required across the bridge, and guardrail protection is typically required in the approach area.

Roadway shoulder and bridge shoulder width standards have been developed to allow project designers more flexibility, providing them greater latitude to address specific project requirements. For the majority of bridges, the bridge width will match the approach roadway width. For longer and/or more complex bridges, a risk assessment of non-standard width options will be performed to determine the appropriate bridge width. Refer to the document *Performance-Based Practical Design Process and Design Guidance*, found as an attachment to Technical Memorandum No. 18-09-TS-07. (http://www.dot.state.mn.us/pbpd/design-guidance.html)

Detailed design decision documentation should include a checklist leading to the selected bridge width over, and portal width under, for the project and must include consideration of the following functions of the shoulder:

- Recovery area to regain control of a vehicle.
- Emergency parking area for stalled vehicles and escape route for stranded motorists.
- Passageway for bicycles and pedestrians.
- Passageway for emergency vehicles.
- Parking area for bridge maintenance and inspection vehicles (working area for under-bridge inspection vehicle and lane closure requirements).
- Temporary traffic lane during deck repairs or overlay construction.
- Area for deck drainage and snow storage.
- Accommodation for passing of wide oversize loads, especially farm machinery.
- Escape area to avoid a head-on collision with an oncoming passing vehicle on a two-lane highway.
- Designated bus shoulder lane.
- Staging needs during construction.

For local roads and streets, bridge widths are given in the *State Aid Manual*, Section 5-892.210 and the *State Aid Operations Rules*, Chapter 8820. Note that local road system bridges at an interchange with a trunk highway must be designed to trunk highway standards between the ramp terminals on the local road.

Cross Slopes on Bridges

1) Use a cross slope on the bridge traffic lanes that is the same as the approaching roadway lanes, normally 0.02 ft/ft. The shoulder on a bridge may continue at the adjacent lane cross slope or, if better drainage is desired, may be 0.005 ft/ft greater than the adjacent lane. If a shoulder functions as a pedestrian access route, cross slopes must not exceed 0.02 ft/ft to be ADA-compliant. When the bridge deck is superelevated, provide the same slopes for the shoulders as the adjacent bridge traffic lanes. The 0.005 ft/ft maximum cross slope change between adjacent lanes and shoulders is determined for constructability by limiting the need for atypical detailing such as special bar bends in the deck. Also note that the greater the change in cross slope, the more difficult it is to remove snow to bare pavement. Changes in cross slope between adjacent lanes and shoulders that are greater than 0.005 ft/ft will be considered where steeper slopes will reduce the number of deck drains on the bridge, but must be approved by the Preliminary Bridge Plans Engineer. Note that the effects of a changing cross slope are magnified on curved alignments and require additional consideration and adjustment of stools, seat elevations, and resulting encroachment on vertical clearances.

   Keep superelevation transitions off bridges. In instances where they are unavoidable, it is preferable for ease of deck placement to maintain a straight line across the deck at all locations, because it allows a straight screed between paving rails placed at both sides of the deck. Locate begin and end points of transition breaks at piers.

2) Provide ramp cross slopes that are uniform between the bridge curbs.

Bridge Median

On divided highways with a separate bridge for each roadway, the openings between bridges must be a minimum of 8'-0" wide if access for bridge inspection vehicles is required.

Use longitudinal joints along the median of bridges only on bridge roadways wider than about 100 feet or for other special cases. By eliminating this joint on bridges with medians, simpler detailing and simpler construction can be used.

Sidepaths (Shared-Use Paths) and Sidewalks on Bridges

Sidepaths are shared-use paths parallel to the roadway that are physically separated from motor vehicle traffic. Providing a sidepath on a bridge is an option to accommodate both pedestrian and bicycle traffic. Sidewalks are similar but are intended to accommodate pedestrian traffic only.

The width of bridge sidepaths and sidewalks are highly dependent on their context (i.e., factors such as land use, user type, expected volume, state and local non-motorized plans, network connections, trip attractions, overlooks, future growth, and bridge length).

When including pedestrian and/or bicycle traffic on a bridge, note that safety, accommodation, and cost must be balanced for all users throughout the roadway cross section. This includes balancing the widths of lanes, shoulders, sidepaths, and sidewalks, particularly in constrained cross-sections.

The AASHTO *Guide for the Development of Bicycle Facilities* (GDBF) recommends a minimum two-way shared-use path paved width of 10 feet.

The *Proposed Guidelines for Pedestrian Facilities in the Public Right of Way* (PROWAG) requires a continuous minimum clear public access route (PAR) width of 4 feet and a minimum clear PAR width of 5 feet at intervals of 200 feet to allow for passing.

On bridges, MnDOT also includes a buffer width added on each side of the sidepath/sidewalk in order to protect users from vertical barriers and edge of sidepath/sidewalk drop-offs.

MnDOT also has additional width criteria based on the maintenance accessible route (MAR). The MAR width requirements allow maintenance forces access for snow and ice maintenance of the sidepath/sidewalk. The MAR width is dependent on the agency responsible for snow and ice maintenance and the equipment they use. The minimum width for the MAR is 6 feet between obstructions (sometimes more if achievable).

Use the following guidance for determination of bridge sidepath/sidewalk widths. For local bridges, also refer to *State Aid Operation Rules*, Chapter 8820.

1) New vehicular bridges
   Best practice is to provide continuity by matching the measured width of the approach sidepath/sidewalk and adding a 1 foot buffer width on each side. See Figure 2.1.2.1. For approach sidepaths/sidewalks that are located immediately behind a curb, the approach width is measured to the back side of the curb. Integral brush curbs (maximum of 2 inches wide x 6 inches high) may be included in the clear width dimension where the total width is greater than 10 feet. For approach sidepaths not meeting the AASHTO GDBF recommended minimum width of 10 feet, consult state or local plans and/or the appropriate trail

authority to identify the future intent and feasibility of providing a greater approach sidepath width. Consult with functional group experts as necessary.

Total bridge sidepath/sidewalk widths greater than best practice or greater than 12 feet require consultation with the state or local authority and/or the appropriate trail authority to identify the need for additional width. The District and/or local authority must document the need for and feasibility of providing this width (plan, cross section, letter, user volume, etc.). Total widths beyond 12 feet require concurrence from functional group experts and discussion to determine whether municipal cost participation is necessary.

The minimum total bridge sidepath width for new vehicular bridges is 10 feet, which is based on an 8 foot approach sidepath (two times the 4 foot PAR width) plus a 1 foot buffer width on each side. Consideration may be given to a minimum total bridge sidepath width less than 10 feet when the approach sidepath width is less than 8 feet and/or there is concurrence from functional group experts.

For new vehicular bridges that accommodate pedestrian traffic only, the minimum total bridge sidewalk width is 7 feet, which is based on the 5 foot PAR width for passing plus a buffer width of 1 foot on each side.

The total bridge sidepath/sidewalk width is defined as the minimum clear width measured from the sidepath/sidewalk side of the curb/barrier/parapet/railing to the sidepath/sidewalk side of the opposite curb/barrier/parapet/railing. For situations where there is no barrier/parapet on the traffic side of the sidepath/sidewalk (raised sidepath/sidewalk), the measurement is to the top outside edge of the sidepath/sidewalk. There, the location of the top outside edge of the sidepath/sidewalk is defined as 1 inch from the gutter line (based on 6 inch curb height x 0.125 slope = 0.75 inches, rounded up to 1 inch). Integral brush curbs (maximum of 2 inches wide x 6 inches high) may be included in the clear width dimension where the total sidepath/sidewalk width exceeds 10 feet. See Figure 2.1.2.1.

2) New pedestrian bridges
For new pedestrian bridges that carry both pedestrians and bicycle traffic, follow the guidance given in 1) above.

For new pedestrian bridges carrying pedestrians only (note that this is a rare occurrence), the minimum total bridge walkway width is 8 feet per the requirements of AASHTO's *A Policy on Geometric Design of Highways and Streets.*

3) Bridge repair projects
   Where possible, follow the guidance given in 1) above for bridge repair projects.

   On bridge repair projects with constrained cross-sections, the minimum total bridge sidepath width is 8 feet.

   On bridge repair projects with constrained cross-sections that accommodate pedestrian traffic only, the minimum total bridge sidewalk width is 5 feet. Consideration may be given to a minimum total width of no less than 4 feet where constrained bridge cross-sections are less than 200 feet long and there is concurrence from functional group experts. See Figure 2.1.2.1. Note that these minimums are less than the MAR minimum width of 6 feet, which would then require a special plan for snow and ice maintenance.

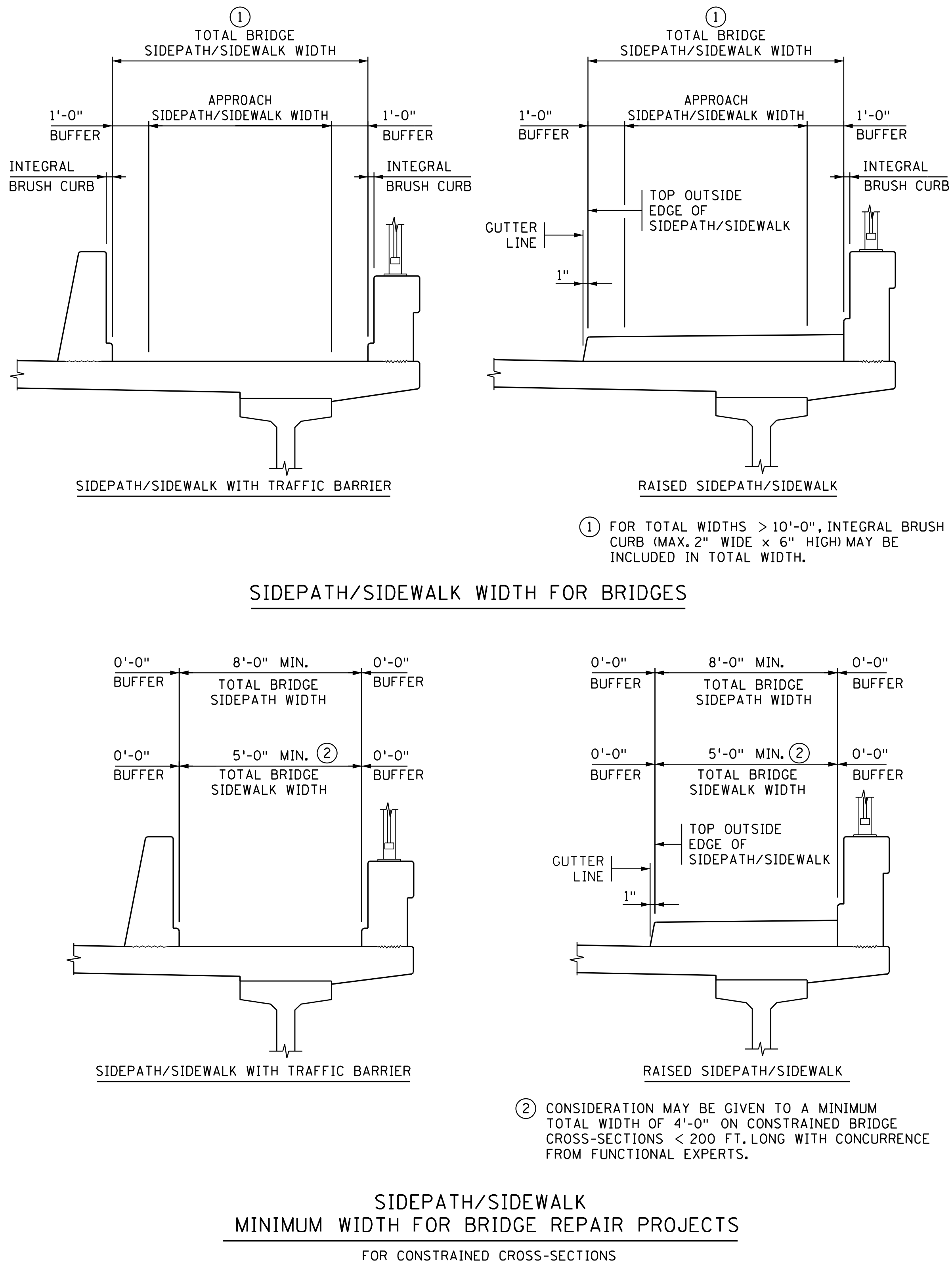
AASHTO defines high speed as design speeds 50 mph and greater, and low speed as design speeds 45 mph and less. Safety for both vehicles and pedestrians/bicycles must be considered when determining separation requirements on a bridge. A separation railing between the roadway and sidepath/sidewalk can provide protection to users of the sidepath/sidewalk, but also can reduce sight distance and present a possible hazard to vehicle traffic. Functional group experts within MnDOT have agreed on the following guidance for determining when to use a separation railing:

- For design speeds of 45 mph and greater, a separation railing is always required.
- For a design speed of 40 mph, consider the following when determining whether separation is needed:
  - Adjacent land uses, such as the pedestrian destinations described in [Minnesota Walks].
  - Amount of available space approaching the bridge (length and width) for the appropriate barrier end treatment.
  - Proximity of intersections to the bridge and whether intersection sight distance will be affected by inclusion of barrier and guardrail.
  - Volume of pedestrians, bicyclists, passenger cars, and heavy vehicles that use the bridge.
  - Actual motor vehicle operating speed compared to the design speed.
  - Horizontal alignment and location of the sidepath/sidewalk relative to a curve (e.g. – inside or outside of a curve).

  - Regional significance of sidepath/sidewalk connection.
- For design speeds of 35 mph or less, use a raised sidepath/sidewalk without a separation railing.

Separation railings must meet the MASH crash test level and traffic railing height requirements found in BDM Article 13.2.1. In addition, a pedestrian/bicycle railing is required on the outside edge of the sidepath/sidewalk. When a separation railing is provided, use the bridge slab for the sidepath/sidewalk. When a separation railing is not used, provide a raised sidepath/sidewalk with a 6 inch curb height measured at the gutter line. Advise the road plans designer to provide for any necessary sidepath/sidewalk ramping off the bridge.

The minimum cross slope for sidepaths/sidewalks is 0.01 ft./ft.


1
TOTAL BRIDGE
SIDEPATH/SIDEWALK WIDTH
APPROACH
SIDEPATH/SIDEWALK WIDTH
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BRUSH CURB
INTEGRAL
BRUSH CURB
SIDEPATH/SIDEWALK WITH TRAFFIC BARRIER
1
TOTAL BRIDGE
SIDEPATH/SIDEWALK WIDTH
APPROACH
SIDEPATH/SIDEWALK WIDTH
1'-0"
BUFFER
1'-0"
BUFFER
INTEGRAL
BRUSH CURB
TOP OUTSIDE
EDGE OF
SIDEPATH/SIDEWALK
GUTTER
LINE
1"
RAISED SIDEPATH/SIDEWALK
1 FOR TOTAL WIDTHS > 10'-0", INTEGRAL BRUSH CURB (MAX. 2" WIDE x 6" HIGH) MAY BE INCLUDED IN TOTAL WIDTH.
SIDEPATH/SIDEWALK WIDTH FOR BRIDGES
0'-0"
BUFFER
8'-0" MIN.
TOTAL BRIDGE
SIDEPATH WIDTH
0'-0"
BUFFER
0'-0"
BUFFER
5'-0" MIN. 2
TOTAL BRIDGE
SIDEWALK WIDTH
0'-0"
BUFFER
SIDEPATH/SIDEWALK WITH TRAFFIC BARRIER
0'-0"
BUFFER
8'-0" MIN.
TOTAL BRIDGE
SIDEPATH WIDTH
0'-0"
BUFFER
0'-0"
BUFFER
5'-0" MIN. 2
TOTAL BRIDGE
SIDEWALK WIDTH
0'-0"
BUFFER
TOP OUTSIDE
EDGE OF
SIDEPATH/SIDEWALK
GUTTER
LINE
1"
RAISED SIDEPATH/SIDEWALK
2 CONSIDERATION MAY BE GIVEN TO A MINIMUM TOTAL WIDTH OF 4'-0" ON CONSTRAINED BRIDGE CROSS-SECTIONS < 200 FT. LONG WITH CONCURRENCE FROM FUNCTIONAL EXPERTS.
SIDEPATH/SIDEWALK
MINIMUM WIDTH FOR BRIDGE REPAIR PROJECTS
FOR CONSTRAINED CROSS-SECTIONS

Figure 2.1.2.1

Protective Barriers at Bridge Approaches

The ends of bridge barriers must be protected from being impacted (except on low speed roads such as city streets). For design speeds over 40 mph, a crash tested guardrail transition is required.

Refer to *State Aid Operation Rules*, Chapter 8820 for guardrail requirements on local bridges. Note that local road system bridges at an interchange with a trunk highway must be designed to trunk highway standards between the ramp terminals on the local road.

2.1.3 Bridge Undercrossing Geometrics

General Criteria for Lateral Clearance

Bridge undercrossing geometrics must rationalize safety requirements with costs and physical controls such as span length and permissible depth of structure. The following guidelines apply in establishing these geometrics:

1) Safety

   Piers, abutments, inslopes and unpaved back slopes steeper than 1:3, and guardrails can all be crash hazards to an out-of-control vehicle.

   Historically at bridge undercrossings, it was considered best practice to provide a clear zone recovery area beside the roadway, free from these hazards. The clear zone was determined using the MnDOT *Road Design Manual*, Section 4-6.04 and was based on the roadway curvature, design speed, ADT, and ground slope. For the area under bridges a practical maximum clear zone of 30 feet was used as permitted in the *2011 AASHTO Roadside Design Guide*, Table 3-1, based on consistent use and satisfactory performance.

   Acceptance of the Performance-Based Practical Design (PBPD) philosophy led to reevaluation of the risk of not meeting the clear zone requirements at bridge undercrossings. This evaluation resulted in new guidance for determination of the appropriate clear zone, which is found in the document, *Performance-Based Practical Design Process and Design Guidance*:

   http://www.dot.state.mn.us/pbpd/design-guidance.html

   In addition, Technical Memorandum No. 21-05-TS-04 was created, which contains the current PBPD guidance for determining the bridge undercrossing width for MnDOT projects:

   https://techmemos.dot.state.mn.us/techmemo.aspx

   Where drainage must be carried adjacent to the roadway passing under a bridge, either a culvert must be provided at the approach fill or the ditch section must be carried through at the toe of the bridge approach fill. The use of a culvert will often permit more desirable bridge geometrics, but the culvert openings can also introduce a roadside hazard, requiring guardrail. A determination regarding drainage (need for culverts, size of a culvert, and assessment of possible hazard) will be a controlling factor in deciding geometrics of the bridge for the site.

2) Economics
   Prestressed concrete beam spans are normally the most economical type of construction for grade separations. In addition, there will usually be greater economy in constructing grade separations using two long spans rather than constructing four shorter spans.

   The span lengths and overall bridge length affect the abutment heights, which in turn affect the overall cost of the bridge. See BDM Article 2.3.2 for discussion of bridge types and their economical/typical span ranges as well as a discussion on abutment and pier type selection.

3) Appearance
   The use of longer spans will necessitate a deeper superstructure and higher approach fills. Consideration should be given to the effect of the depth of structure on the overall appearance and design of the undercrossing.

   For rough calculations during preliminary planning, the depth of highway bridge superstructures can be assumed to be about 1/20 of the length of the longest span. (Depth of superstructure refers to the dimension from top of slab to bottom of beam.) Contact the Preliminary Bridge Plans Engineer for the exact dimensions to be used in final planning and for depth of structure on railroad bridges.

Lateral Clearance for Roadways
As noted above, acceptance of the Performance-Based Practical Design (PBPD) philosophy has led to revised lateral clearance requirements at bridge undercrossings. Refer to Technical Memorandum No. 21-05-TS-04 when determining the width requirements for the under-bridge typical section. This technical memorandum can be found at:

https://techmemos.dot.state.mn.us/techmemo.aspx

Eliminate side piers wherever possible.

Lateral Clearance for Railroads
Lateral clearances at railroads are to be determined as follows:
1) The statutory clearances diagram shown on Figure 2.1.3.1 represents the absolute minimums that must be adhered to. For design, use a

minimum horizontal clearance of 9'-0" to a pier or abutment (8'-6" is the legal minimum).

2) MnDOT and FHWA historically agreed to the horizontal clearances shown in Figure 2.1.3.1 (25'-0" minimum clearance to pier, 30'-6" to “back slope control point”) for mainline BNSF RR tracks at sites meeting the following conditions:
   a) When the standard will not increase the cost of the structure by more than $50,000.
   b) When sufficient vertical clearance exists between the tracks and in-place or proposed roadway profile to accommodate the structure depth necessary for the longer spans typically required by the standard.

3) Back slopes shall be 1V:2H and pass through the “back slope control point” shown on Figure 2.1.3.1 for the applicable case. The dimension to the “back slope control point” indicates the maximum extent of federal participation in the construction and must not be exceeded.

4) The Preliminary Bridge Plans Engineer will contact the MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO), to negotiate with the railroad the need for provisions for a maintenance road for track maintenance equipment and future track requirements.

   The publication *UPRR/BNSF Railway Guidelines for Railroad Grade Separation Projects* includes railway owner design expectations for roadway bridges over railroads, as well as bridges carrying railroads. While this document may include guidance that is unattainable by the MnDOT project at hand, the OVCO has directed the Preliminary Bridge Plans Unit to use this document during plan development and negotiations with the railroads.

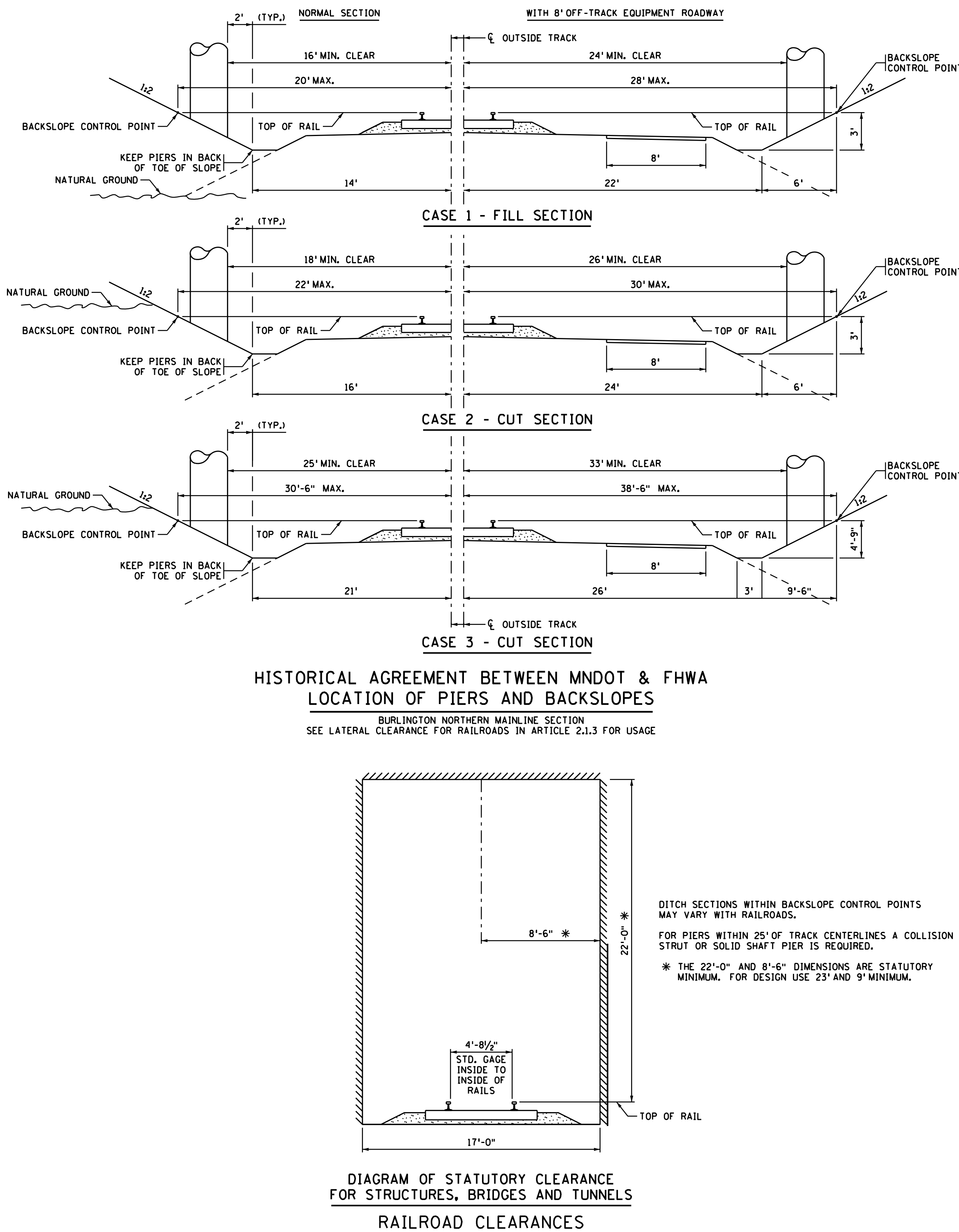


Figure 2.1.3.1

General Railroad Clearances

(Note that railroad approval is required for specific project clearances)

Waterway Sections Under Bridges

The Waterway Analysis (hydraulics report) gives information on the required stream cross section under the bridge including waterway area and low member elevation. Potential flood damage, both upstream and downstream, and permitting agencies' requirements must be considered.

The Preliminary Bridge Plans Engineer will coordinate with the Waterway Unit to determine when a wildlife passage bench is required under bridges over waterways.

For bridges on the local system, go to the State Aid Bridge Web Site at http://www.dot.state.mn.us/stateaid/bridge/resources.html and refer to the guidance found under Hydraulics.

Vertical Clearance for Underpasses

Vertical clearance requirements are aligned with the 2018 edition of the AASHTO *A Policy on Geometric Design of Highways and Streets* (AASHTO GDHS). For highway bridge structures, AASHTO GDHS Article 10.8.4.2 recommends a minimum vertical clearance of 1'-0" above the legal vehicle height, plus an allowance for future pavement resurfacing and other considerations. The legal height of a truck in Minnesota is 13'-6". For pedestrian bridges and sign bridges, AASHTO GDHS Article 5.2.2.7.2 recommends a minimum vertical clearance 1'-0" greater than the highway bridge structure clearance. Table 2.1.3.1 lists the minimum vertical clearance requirements for Minnesota trunk highway underpasses, pedestrian bridges, sign bridges, railroad underpasses, and truss/arch portals. The clearance over highways applies to the traffic lanes and full usable width of shoulders.

Per Minnesota Rules, Chapter 8820, Local State-Aid Route Standards, the minimum vertical clearance for highway underpasses (including construction tolerance) is 16'-4" for rural-suburban designs and 14'-6" for urban designs. For trunk highways crossing local roads or streets at a freeway interchange, the minimum vertical clearance with construction tolerance is 16'-4". A complete list of vertical clearances for local roads and streets is found in the *State-Aid Operations Rules*, Chapter 8820. Note that local road system bridges at an interchange with a trunk highway must be designed to trunk highway standards between the ramp terminals on the local road.

Where bikeways pass under a bridge or through a tunnel, a 10'-0" vertical clearance is desirable for adequate vertical shy distance. (See AASHTO *Guide for the Development of Bicycle Facilities*, 4th Edition, pages 5-6 and 5-26.) Where this is impractical, a lesser clearance down to a minimum of

8'-0" is acceptable. Clearances below 10′-0″ on the local road system will require a variance to the *State-Aid Operations Rules*, Chapter 8.

Table 2.1.3.1 Vertical Clearance Requirements for Bridges

TYPE OF STRUCTURE	MINIMUM VERTICAL CLEARANCE FOR NEW BRIDGES ①②	MINIMUM VERTICAL CLEARANCE UNDER EXISTING BRIDGES FOR ROADWAY PAVEMENT PROJECTS ③
Trunk Highway Under a Roadway or Railroad Bridge on a Super Load OSOW Corridor ④	16'-6"	16'-6"
Trunk Highway Under a Roadway or Railroad Bridge on a Non-Super Load OSOW Corridor	16'-4"	16'-0"
Trunk Highway Under a Pedestrian Bridge on a Super Load OSOW Corridor ⑤	17'-6"	17′-6″
Trunk Highway Under a Pedestrian Bridge on a Non-Super Load OSOW Corridor ⑤	17'-4"	17′-0″
Trunk Highway Under a Sign Bridge on all corridors ⑥	17′–6″	17′-0″
Railroad Under a Trunk Highway Bridge ⑦	23'-0"	NA
Portal Clearance on a Trunk Highway Bridge Through-Truss or Through-Arch ⑧	20'-4"	20′-0″
Portal Clearance on a Railroad Bridge Through-Truss Over a Trunk Highway ⑧	21'-4"	21′-0″

① Provide additional clearance for future resurfacing where practical. Traditional bituminous overlay allowances range from 3″ to 6″. Un-bonded concrete overlay are greater than the 6″ tolerance and can be as high as 12″. The appropriate design value will depend on the pavement type, its initial structure type, and lifecycle strategy, and should be coordinated with the Pavement Design Engineer.

② A clearance height that includes a future resurfacing allowance may be used in place of the listed minimums, provided the resulting clearance is at least as much as the listed minimums in this column. Construction tolerance requirements for these values have been reviewed and deemed adequate for new bridge construction. Adjust table values upward as required for overlay requirements exceeding 4”.

③ The minimum vertical clearances shown are the absolute minimum clearances to be achieved after pavement preservation, pavement rehabilitation, or pavement reconstruction under an existing bridge or structure. These minimums are only acceptable due to the known spatial location of the existing structure, thus eliminating the construction tolerance risk of that existing element.

These minimums do not apply to existing bridge repair projects. Refer to MnDOT *Bridge Preservation and Improvement Guidelines* for the minimum vertical clearances for bridge preservation and bridge rehabilitation projects.

④ A minimum vertical clearance of 16’-6” is required under highway and railroad bridges on designated Super Load OSOW Corridors. Super Load OSOW Corridors are designed to accommodate an envelope size of 16’ wide x 16’ high x 130’ long, traveling along the corridor. For specific locations, the clearance for existing bridges for pavement projects may be reduced with approval from the MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO). Contact the OFCVO for specific corridor locations and requirements: http://www.dot.state.mn.us/cvo/index.html

⑤ Additional vertical clearance is required under pedestrian bridges because they are much less substantial than highway and railroad bridges and could collapse in the event of a hit. The 1’-0” of additional vertical clearance meets AASHTO GDHS Article 5.2.2.7.2 recommendations.

⑥ Additional vertical clearance is required under sign bridges because they are much less substantial than highway and railroad bridges and could collapse in the event of a hit. For new sign bridges on all corridors and for existing sign bridges on non-Super Load OSOW Corridors, the minimum vertical clearances listed meet or exceed AASHTO GDHS Article 5.2.2.7.2 recommendations. The minimum vertical clearance for existing sign bridges on Super Load OSOW Corridors is less than AASHTO GDHS Article 5.2.2.7.2 recommendations. This allows room for pavement mill & overlay projects before a sign bridge must be raised or replaced. Based on consideration of many factors (OSOW 16’-0” height limit, a strict permitting process for OSOW vehicles, cause and frequency of hits, low risk of sign bridge occupancy when hit, cost of replacement, sign structure service life), 17’-0” is deemed adequate for existing bridges.

⑦ Vertical clearance over railroad infrastructure requires approval of the railroad. The 23'-0" clearance above top of rails is the minimum clearance required by the American Railway Engineering and Maintenance of Way Association (AREMA) manual. The maximum vertical clearance for Federal Cost Participation is 7.1 meters (23’-4”) per the Code of Federal Regulations (see CFR 646 Appendix to Subpart B of Part 646). This is allowed where the railroad’s standard practice is to accommodate future ballasting of the tracks. The Minnesota statutory minimum vertical clearance is 22'-0". See Figure 2.1.3.1. For clearances below 22'-0", approval from the MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO) and the railroad is required. Contact the Rail Safety and Coordination Project Manager from the OFCVO for assistance with railroad coordination, agreements, and approvals.

⑧ Minimum portal clearance values were set based on historical portal heights. Railroad through-truss bridges require an additional 1'-0" of portal clearance to meet railroad stipulations.

Vertical Clearance over Waterways

The location and project description for all bridges over Minnesota waterways are to be reviewed by the U.S. Coast Guard (USCG) for potential permitting requirements.

1) Non-Navigable Waterways

   A 3'-0" minimum clearance between the 50-year flood stage and low point on the bridge superstructure is recommended. This amount of clearance is desired to provide for larger floods and for the passage of ice and/or debris. If this amount of clearance is not attainable due to constraints relating to structure depth, roadway grades or other factors, reduced clearance may be allowed. The Preliminary Bridge Plans Engineer, after consultation with the Waterway Unit and the MnDOT District Office, will determine the required clearance.

2) Navigable Waterways

   a) Examples of waterways that require a construction permit (generally considered to be waterways for commercial shipping) from the USCG include:

      - The Mississippi River downstream from the railroad bridge that crosses the river south of 42nd Avenue North in Minneapolis (River Mile Point 857.6)
      - The Minnesota River downstream from location just west of T.H. 101 river crossing in Shakopee (River Mile Point 25.6)
      - The St. Croix River downstream from Taylors Falls
      - The St. Louis River downstream from Oliver, Wisconsin.

      Guide vertical clearances published by the USCG are:

      - Mississippi River:
        - 52.0' above 2% flowline elevation or 60.0' above normal pool elevation, whichever is greater, for the portion downstream of the Burlington Northern Railroad Bridge near the University of Minnesota (River Mile Point 853.0).
        - 21.4' above river stage of 40,000 c.f.s. for the river portion upstream (River Mile Point 853.0 to 857.6).
      - Minnesota River:
        - 55.0' above normal pool elevation from the river mouth to I-35W bridge (River Mile Point 10.8).
        - 30.8’ above 1881 high water elevation from I-35W bridge (River Mile Point 10.8) to Shakopee (River Mile Point 25.6).
      - St. Croix River:
        - 52.0' above 2% flowline elevation or 60.0' above normal pool elevation, whichever is greater, from the river mouth to Stillwater.

- Lake Superior Watershed:
  - Navigation clearances are determined by USCG on a case-by-case basis.

Consult the Preliminary Bridge Plans Engineer when establishing navigation clearances.

b) All Other Navigable Waterways
Bridges that cross waterways in other portions of the state may be required to provide for local pleasure boat traffic. Vertical clearance for these bridges will be determined on an individual basis, based on local needs. The Preliminary Bridge Plans Engineer, in consultation/concurrence with the Waterway Unit, the MnDOT District Office, and the MnDNR, validate the design based on specific conditions of the waterway.

Vertical and Horizontal Alignment

Information governing vertical curves, horizontal curves, and sight distance may be found in the *Facilities Design Manual* and *Technical Manual*.

Note that in MnDOT plans the term "azimuth" refers to the angle of horizontal deviation, measured clockwise, from true north to the direction of increasing stationing.

Example: Consider a straight line horizontal alignment that runs exactly east-west. If stationing increases from west to east, azimuth is 90 degrees. If stationing increases from east to west, azimuth is 270 degrees.

When preparing preliminary bridge plans for the local road system, refer to the *State Aid Manual* for vertical and horizontal alignment requirements.

2.1.4 Bridge Barriers and Railings

See Section 13 of this manual for the policy on design of bridge barriers and railings for MnDOT projects.

2.2 Bridge Aesthetics

The aesthetic design process is initiated early in the bridge planning phase.

The Preliminary Bridge Plans Engineer, the Preliminary Bridge Architectural Specialist, the District, and the financial stakeholders determine the aesthetic design level with an eye on constructability and cost. Other people, offices, agencies, etc. may also be involved. The extent of this involvement may vary depending on the individual project. This process leads to the development of an Aesthetic Plan for the bridge. Once the project reaches the final stage, the Bridge Design Unit Leader implements the Aesthetic Plan to completion with assistance from the Preliminary Bridge Architectural Specialist as needed.

Note that constructability of aesthetic components and complexity of the aesthetic details may affect the project schedule, and therefore must be considered during the development process.

Section 3 of the *Aesthetic Guidelines for Bridge Design Manual* describes the process of aesthetic design in more detail.

Maximum levels of MnDOT participation in aesthetic costs are given in the *Cost Participation and Maintenance Responsibilities with Local Units of Government Manual*.

2.3 Preliminary Bridge Plans

2.3.1 General

Purpose

The Preliminary Bridge Plan serves to document the main features of the bridge (type, size, location, aesthetics, etc.) and is used to obtain approvals and coordination before final design begins. By doing this, the time and expense of revising a completed plan will hopefully be avoided. The plan coordinates the work between Road Design and the Bridge Office and enables the cost and scope of the work to be estimated.

Specific users of the plan include:

- Road Designers to verify the grade, alignment and roadway widths and to obtain the approximate limits of grading, paving and guardrail at the bridge ends.
- FHWA to review and approve unusual or complex bridge projects.
- Bridge Office Consultant Agreements Unit to select and negotiate contracts with consultants.
- Final Bridge Design Units and Consultants to prepare final plans.
- Bridge Scoping Engineer and Bridge Estimates Unit to prepare a preliminary estimate of the bridge costs.
- MnDNR, U.S. Coast Guard, U.S. Corps of Engineers, and Watershed Districts to review and issue required permits for stream crossings.
- Cities, Planning Agencies, and citizen groups to review and approve projects.
- District Traffic Engineer and Regional Transportation Management Center (RTMC) to convey their needs on the new bridge.
- MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO) for use in negotiating railroad agreements.

In preparing preliminary bridge plans, the plan users should always be kept in mind, particularly those without bridge technical experience.

Requirements for Preliminary Bridge Plans

Preliminary bridge plans are required for all new trunk highway bridges (including MnDOT precast concrete arch and three-sided structures and pedestrian underpass box culverts) and all bridge widening projects where substructure widening is required. In addition, preliminary plans signed by the State-Aid Bridge Engineer are required for all county and local bridges that cross a trunk highway. Preliminary bridge plans are not required for culverts (except those used for pedestrian access), overlays, deck replacements, and other projects where substructures are not widened.

The Bridge Preliminary Plans Unit normally prepares preliminary plans for new trunk highway bridges, although consultants may also develop plans. Preliminary plans for bridge widening projects are normally prepared by the Bridge Design Units since significant design work is required to evaluate the existing structure and schemes for widening and handling traffic.

Preliminary plans prepared by Consultants or Design Units are submitted to the Bridge Preliminary Unit for review, acceptance, submittal to the State Bridge Engineer for signature, and distribution of signed copies.

Contents

The Preliminary Bridge Plan consists of a general plan and elevation sheet, survey sheet, and borings sheet. For complex urban structures additional road design sheets giving alignment, superelevation diagrams, utilities, contours, traffic staging, intersection layout, and aesthetics may be included. The Preliminary Bridge Plan contains: plan and elevation views, a transverse section, design data, data on the type of structure, foundation requirements, and aesthetic treatment. When aesthetics are of special importance, architectural type drawings showing the proposed treatment or type of construction may also be included. For bridge widening projects, the survey sheet may be eliminated or a copy of the survey sheet from the existing bridge may be included.

Preparation of Preliminary Bridge Plans

The steps involved, although not necessarily in order, in preparing a typical preliminary plan set for a new trunk highway bridge by the Preliminary Unit are as follows:

1) *Request for Bridge Scoping and Cost Estimating Assessment – Bridge Replacement* (Form A) is completed, which provides the initial information for the project. Form A is found at:

http://www.dot.state.mn.us/bridge/scoping.html

Consideration is given to the use of Accelerated Bridge Construction (ABC) methods at this step in the process. Results of the ABC Stage 1 assessment is reported on Form A and, if applicable, the ABC Stage 2 assessment is completed and included as an attachment. In addition, a bridge number is requested by the MnDOT District Project Manager or their designee by completing the online *New bridge number request form,* found at:

http://www.dot.state.mn.us/bridge/bridgereports/index.html

A new bridge number is then assigned.

2) Approved geometric layouts are received from the District.
3) Bridge survey sheets are received from the District Surveys Section. Copies are sent to the Foundations Unit of the Office of Materials and Road Research requesting soil borings. For stream crossings, a copy is sent to the Bridge Office Waterway Unit requesting a waterway analysis.
4) A depth of structure and span arrangement are determined using the approved geometric layout and waterway analysis (if applicable) and are given to Road Design. This typically involves communication between the Bridge Office, Road Design, and Hydraulics to arrive at a structure depth and span arrangement that produces the best overall solution. Note that when calculating structure depth for preliminary plan purposes, assume that the beam stool plus the beam camber equals a depth of 4 inches.
5) If a railroad is involved, negotiations are held with the railroad to determine what features should be incorporated into the plan to satisfy the railroad's needs and also meet MnDOT standards.
6) Final grades and alignment are developed and officially received from Road Design.
7) A CADD technician is assigned the project and drafting of the plan begins. Clearances are checked and more exact span lengths determined.
8) Traffic data is requested and received from the District Traffic Engineer.
9) The extent of aesthetic treatment is determined following the process described in BDM Article 2.2.
10) Deep foundation borings are received electronically from the Foundations Unit along with the *Foundation Analysis and Design Recommendations* (FADR). The borings are plotted on the survey sheets.
11) The Preliminary Bridge Plans Unit checks the completed preliminary package, except for finalizing the foundation type.
12) The preliminary package is given to the Regional Bridge Construction Engineer along with the FADR for determining foundation type; this

includes pile type, lengths, and resistances. When received, the final foundation information is added to the preliminary plan.

13) The completed Preliminary Bridge Plan is reviewed with the Bridge Planning and Hydraulics Engineer and taken to the State Bridge Engineer for signature.

Time Schedule for Preliminary Plan Preparation

The time schedule for receiving information and completing preliminary bridge plans for normal bridges, as given in Primavera P6, is shown in Table 2.3.1.1.

Table 2.3.1.1 Preliminary Plan Time Schedule

WORK ITEM	TIME PRIOR TO SCHEDULED LETTING DATE
Bridge Survey	21 months
Hydraulics	18 ½ months
Grades and Alignment	18 ½ months
Foundations	17 months
Preliminary Plan Completed	16 months (typical bridges) 20 months (major bridges)

Additional lead-time beyond that given in the table above is required for major bridges, bridges involving agreements with cities or railroads, and bridges with extensive aesthetic requirements.

In addition to the work items listed above, time must be allotted for a formal type selection study for major bridges.

Use of Preliminary Bridge Plans

The completed and signed Preliminary Bridge Plan becomes the department's official proposal for that structure. The following steps are then taken:

1) The Bridge Estimating Unit in the Bridge Office prepares an estimated contract construction cost for the structure.

2) Copies of the Preliminary Bridge Plan are distributed to the various offices of MnDOT and outside agencies for information, review, and approval, as the case may be. (See Table 2.3.1.2.)

   Approval by all concerned of the proposed structure dimensions, type of construction, and geometrics before the start of final design is one of the most important functions of the Preliminary Bridge Plan. This is particularly true of stream crossings, railroad crossings (over and under), and structures requiring special aesthetic treatment.

The Federal-Aid Highway Program (FAHP) provides federal-aid to State-selected projects. The Federal Highway Administration (FHWA) administers the FAHP on behalf of the U.S. Secretary of Transportation under Title 23 and therefore is one of the outside agencies that reviews bridge projects. The *FHWA Minnesota Division and Minnesota Department of Transportation Stewardship & Oversight Agreement* documents the roles and responsibilities of the FHWA and MnDOT regarding project approvals and review:

- For most bridge projects, MnDOT assumes the FHWA's Title 23 responsibilities and only a courtesy copy of the Preliminary Bridge Plan transmittal letter is sent to FHWA (without the plans) for informational purposes.

- For unusual or complex bridges and structures, the FHWA Minnesota Division is responsible for the approval of the Preliminary Bridge Plan. For the purpose of this guidance, unusual or complex bridges and structures are defined as those that the FHWA Minnesota Division determines to have unique foundation problems, new or complex designs, exceptionally long spans, exceptionally large foundations, complex hydrologic aspects, complex hydraulic elements or scour related elements, or that are designed with procedures that depart from currently recognized acceptable practices. Examples of unusual or complex bridges and structures include cable-stayed bridges, suspension bridges, arch bridges, segmental concrete bridges, movable bridges, truss bridges, tunnels, complex geotechnical wall systems, and complex ground improvement systems.

  When submitting preliminary documents to the FHWA, include the Preliminary Bridge Plan and supporting information. Supporting information includes all bridge/structures related environmental concerns and suggested mitigation measures, studies of bridge types and span arrangements, approach bridge span layout plans and profile sheets, controlling vertical and horizontal clearance requirements, roadway geometry, design specifications used, special design criteria, special provisions (if available), and cost estimates. In addition, submit hydraulic and scour design studies/reports which show scour predictions and related mitigation measures. Also submit geotechnical studies/reports along with information on substructure and foundation types.

For unusual or complex bridge projects, the State Bridge Engineer will submit one copy of the Preliminary Bridge Plan along with a transmittal letter requesting approval directly to the FHWA Division Bridge Engineer. The transmittal letter also includes the estimated contract construction cost of the structure. The FHWA is the only outside agency to which the Bridge Office sends a direct request for approval. All other outside agencies are contacted through other offices of MnDOT.

Note that the FHWA Headquarters Bridge Division is available for technical assistance on other Federal-aid and non-Federal-aid highways when requested.

3) The Preliminary Bridge Plan is used as a basis for preparing permit drawings to accompany applications to construct structures and approaches over navigable waters of the United States within or bordering our state. Such drawings are prepared in the Preliminary Plans Unit in accordance with detailed instructions issued by the U.S. Coast Guard. The Coast Guard is charged with the responsibility of issuing permits for bridges over navigable waters of the United States within or bordering our state. This includes all bridge spans (including land spans) from abutment to abutment. The Corps of Engineers is responsible for issuing permits for any other miscellaneous structures or work to be performed in navigable waters of the United States.

There are two Coast Guard districts that have jurisdiction within the State of Minnesota; the 9th Coast Guard District based in Cleveland has jurisdiction over the Duluth harbor and navigable portion of the St. Louis River, and the 8th Coast Guard District based in St. Louis has jurisdiction over the navigable portions of the Mississippi, Minnesota, and St. Croix Rivers.

After receiving a permit application, the Coast Guard issues a public notice of application with prints of the permit drawings. These are sent to shipping interests, other agencies, displayed in post offices, etc. Generally, if no comments are received from others within 30 days of the notice of application, and if environmental statements have been submitted and a certification given by the Minnesota Pollution Control Agency, a permit will be issued.

Correspondence to the Coast Guard is generally prepared for the signature of the State Bridge Engineer.

4) When all approvals have been obtained, the Preliminary Bridge Plan is used as the basis for the bridge design and for the preparation of final detailed plans. If the design is to be by a consulting engineer, the Preliminary Bridge Plan is typically used as the basis for negotiation of the consultant fee.

Table 2.3.1.2 General Distribution of Preliminary Bridge Plans

DISTRIBUTION TO	PURPOSE		REMARKS
	INFO. & REVIEW	PRE-APPROVAL REQUIRED	
MnDOT District Project Manager	x		
District Pre-Design	x		
District Final Design	x		
District Construction	x		
District Environmental Coordinator	x		
District Hydraulics Engineer	x		For bridges that cross waterways.
District Maintenance	x		
District Bridge Engineer	x		
District Traffic Engineer	x		Send with request for determination of need for lights, signals, conduit, and bridge mounted signs.
Office of Materials & Road Research – Foundations Unit	x		
Regional Transportation Management Center	x		Send with request for determination of need for conduit and mounting devices for surveillance system.
Environmental Stewardship Office	x		
MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO)		x	For railroad crossings only.
Federal Highway Administration (FHWA)	x	x	Approval required for unusual or complex bridge projects only. For all other bridges, a courtesy copy is provided.
Bridge Final Design Unit	x		
Bridge Estimating Unit	x		
Bridge Waterway Unit	x		For bridges that cross waterways.
Bridge Consultant Agreements Unit	x		For bridge projects with consultant involvement.
Bridge Consultant	x		For bridge projects with consultant involvement.
Other Stakeholders	x		As needed.

Preliminary Plans for Local Bridges

Consult the State Aid Bridge Web site at: http://www.dot.state.mn.us/stateaid/bridge/resources.html for the submittal and acceptance process of State Aid Preliminary Bridge Plans.

2.3.2 Bridge Type Selection

General

The type of structure and span arrangement selected will depend on cost, depth available, geometrics, site conditions, and aesthetics. For some bridges this may be an obvious choice. For others it may involve a great deal of study, especially if aesthetics is a main concern. The section that follows gives some general guidelines on the selection process.

Aesthetic Design Process

See Section 2.2 of this manual for a general discussion of the aesthetic design process.

Structure Type

The most commonly used structure types and their characteristics are as follows:

1) Precast Pretensioned Concrete Beam

   This is the most common structure type in Minnesota. Advantages include: low initial and future maintenance costs, high quality factory produced product, a stiff deck, and simple spans that accommodate tapers. Beams are limited to standard depths and straight segments, and a maximum length of about 200 feet. Beams in excess of 150 feet may require special shipping considerations.

2) Welded or Rolled Steel Beam

   This type of structure is well suited to complex urban freeways with limited depth, long spans, and complex geometrics. Steel beam bridges are also well suited for areas with bad soils, such as the Red River Valley, as steel allows the flexibility of modifying the bearing location and adding or reducing span lengths to accommodate shifting abutments and piers. Advantages include: a shallower depth of structure than prestressed concrete, beams with the ability to be field spliced to produce long span lengths, web plates that can be cut to any depth or to a haunched shape, beams that can be curved horizontally, and beams that can be painted a color which contrasts with the slab to make the structure appear thinner. Disadvantages include: a typically higher cost than other structure types, more difficult fabrication and inspection, a longer fabrication time, the possible need for initial painting and future maintenance painting, weathering steel staining of supports, and rusting of weathering steel when under salt exposure.

3) Cast-In-Place Concrete Slab Span
This type of structure is used for shorter span bridges where depth is a major consideration. For simple spans conventionally reinforced, spans range up to 40 feet. Continuous spans are limited to about 60 feet. (See table in Section 5.3.1 of this manual for limits.) Advantages include: a minimum depth superstructure, ease of design and detailing, pleasing aesthetics, and economy for short span bridges. Disadvantages include: span lengths are limited, falsework is required, concrete delivery rate requirements may be a problem, a wearing course may be required to achieve a smooth ride, and the maximum skew angle is 45°.

4) Post-tensioned Concrete Box Girder
Concrete box girders provide an attractive structure with high torsional resistance making them especially well suited for curved structures. The ability to accommodate an integral pier cap is an advantage since horizontal clearance is only required to the column top and not the cap top. Limitations and drawbacks may include the need for falsework, the inability to redeck or widen, and the higher construction cost.

5) Timber
This bridge structure is used only on the local road system, for 1 or 3 spans with a maximum span length of about 25 feet. Advantages include: timber has a natural and aesthetically pleasing appearance, special equipment is not required for installation, and construction can be done in virtually any weather conditions. Disadvantages include: timber is not an economical structure type, it is limited to low-volume roads (roads with an AADT under 750), and the asphalt wearing surface tends to crack due to differential deck deflections.

6) Precast Concrete Box Culvert
Box culverts provide a quickly constructed and economical structure for stream crossings and pedestrian tunnels. Precast concrete box culvert standards are available for culverts up to 16 ft. x 12 ft. in size. Use of up to three large barrel boxes may be economical compared with a bridge. Advantages include: standardized plans, quick installation and low maintenance. Disadvantages include: span limitations, possible debris build-up when multiple barrels are used, and lack of a natural stream for fish unless the invert is lowered and riprapped.

7) Three-Sided Bridge Structure
Three-sided precast concrete structures offer an alternative for short span structures up to 42 feet. Advantages include: quick installation, and a natural stream bottom if scour protection is not required.

Disadvantages include: a higher cost than cast-in-place structures, and pile foundations are typically required for stream crossings unless founded on rock.

Not all bridge sites lend themselves to the use of the more common bridge types listed above. For these situations, specialized bridge types may be required, such as post-tensioned I-girder bridges, tied arch bridges, cable-stayed bridges, or extradosed bridges.

Abutment and Pier Locations

The following guidelines aid in setting abutment and pier locations:

1) Water Crossings

   For water crossings, keep the number of substructures located in the water to the minimum practical. Piers in rivers and streams block the natural flow of the waterway, trap ice and debris, impede navigation, and are subject to scour. In addition, construction of a pier in the water is expensive (especially if cofferdams are needed), and environmentally disturbs the stream/river/lake bottom and water quality. Ideally, set piers and abutments on shore to minimize dewatering and allow easy access for the Contractor. Set substructures to avoid interference with in-place substructures, including piling, wherever practical. Setting spans and structure depth involves balancing the hydraulic requirements of the low member elevation and waterway area with the constraints of approach grades, structure depth, and cost.

2) Grade Separations

   For grade separations, fewer piers are also desirable wherever practical. When determining pier locations adjacent to roadways, refer to Technical Memorandum No. 21-05-TS-04, which can be found at:

   https://techmemos.dot.state.mn.us/techmemo.aspx

   In locations where ramps enter or exit a highway under a bridge, avoid piers between the mainline and ramp, if possible, as they restrict visibility.

   Piers to be placed in the median must be located with consideration of the project parameters, including: safety, current and future roadways, available buffer distance between traffic and piers in both directions, cost, etc. For a typical divided highway median pier, place the pier equidistant to current lanes in both directions. Where location of future lanes is markedly different from the current configuration, discuss placement of median pier with the District and the Preliminary Bridge Plans Engineer to determine the best pier location for the current and future configurations.

Abutment Types

Abutments can generally be classified into 3 categories: stub, semi-high, and high abutments. A further breakdown of abutments can be made according to the way expansion is handled – integral, semi-integral, or parapet type.

1) Stub Abutment: This is the shortest category of abutment, located at the top of the fill slope with generally 2 to 4 feet of stem exposure.

   Integral type stub abutments are the preferred type of abutment due to their jointless nature and simplified construction. Integral type stub abutments have the lowest initial construction cost, are the fastest abutment type to construct, and eliminate the future maintenance and repair required for strip seal expansion joints. Refer to BDM Article 11.1 for length, skew, and exposure limits for integral type stub abutments.

   Semi-integral type stub abutments are the preferred type of abutment when the requirements for integral abutments cannot be met. Semi-integral abutments have a lower initial construction cost than parapet type abutments, and eliminate the future maintenance and repair required for strip seal expansion joints. Refer to BDM Article 11.1 for length and skew limits for semi-integral type abutments.

   Parapet type stub abutments use a strip seal or modular expansion device to accommodate movement. They have the highest initial construction cost, and will require future maintenance and repair for the strip seal expansion joints. The move toward jointless abutments has diminished the use of parapet type stub abutments, but this type is still used where appropriate.

2) Semi-high abutment: This abutment type is located part way up the fill slope and became more popular as two-span overpasses came into use. A slightly higher abutment and elimination of the berm reduces the span length and depth of beam. This allows a lesser profile grade increase, resulting in lower grading costs. Limit the exposed height of abutment face to approximately 8 feet, if possible. Undertake a cost evaluation of longer spans vs. taller abutments when considering a semi-high abutment.

   This category includes semi-integral and parapet type abutments only (integral abutment height restrictions limit them to the stub abutment category). Semi-integral type abutments are the preferred type because of their lower initial construction cost and lower maintenance requirements.

3) High abutment: This abutment type is located at the bottom of the fill slope and is used primarily in congested urban design where structure depth is difficult to obtain. Their use is discouraged since they are more difficult to construct, expensive, require lengthy retaining walls and approach panels, and give a closed-in feel to the highway. Again, this category includes semi-integral and parapet type abutments only, with the semi-integral type preferred due to its lower initial construction cost and lower maintenance requirements.

   In locations where a high abutment would be required and use of a mechanically stabilized earth (MSE) retaining wall is economical, another option is a parapet type abutment supported by a pile foundation behind an MSE retaining wall.

Things to consider when deciding on what height of abutment will best serve a specific project include:

- Advantages of choosing a shorter abutment over a taller abutment:
    - Lower abutment cost.
    - Longer bridge length results in reduced grading and pavement cost.
    - Shorter wingwall and approach panel lengths.
    - Construction of abutments farther from roadway underneath allows for construction staging, possible future expansion underneath for roadway widening or addition of sidewalks, shared-use trails, or other facilities.
- Disadvantages of choosing a shorter abutment over a taller abutment:
    - Requires a longer bridge length, resulting in a higher superstructure cost and increased future maintenance cost.
    - May require a grade raise, resulting in a higher roadway grading and pavement cost.
    - Higher cost for slope protection.

Wingwalls parallel to the bridge roadway are used most often for aesthetic reasons. Flared wingwalls, typically with a flare angle of 45 degrees for bridges with no skew, will result in shorter wingwall lengths and less length of railing. Straight wingwalls, an extension of the abutment parapet, are the simplest to construct but are appropriate only for shallow beams where aesthetics is not a concern.

See additional limitations and guidance for integral, semi-integral, and parapet abutments in Section 11 of this manual.

Pier Types

There is a wide variety of pier types used in bridges, with the most typical consisting of a pier cap supported on multiple columns.

Consider the following general guidelines in order to achieve cost effective piers:

- Minimize the number and size of the columns.
- Minimize the pier width.
- Minimize the number of construction joints in the columns and the pier cap.
- For ease of reinforcement detailing, avoid use of inside corners in the pier column/shaft cross-section.
- For column and cap type piers that require a crash strut, consider use of a solid shaft type pier to reduce construction time and cost.

The feature being crossed is an important consideration when choosing the pier type. Discussions on pier type are provided below for water crossings and grade separations.

1) Water Crossings

   Pile Bent Piers: These piers consist of a row of piles with a concrete cap encasing the pile top, and are the simplest and most economical type of pier. They are used for water crossings where a general maximum height from the top of pier to stream/river/lake bed is under 20'-0" and there is no ice or debris problem. Note that it is important to confirm by analysis that the pile unbraced length under a scour condition does not create instability in the pile. Spans must also be short enough to allow a single row of piles to support the deck at reasonable spacing. The piles act as columns, and bending strength to resist side impacts from ice or debris is important. For cast-in-place piles (the most widely used), a 16" minimum diameter is required. If H-piles are used, the upper portion is encased by a cast-in-place pile shell filled with concrete. Timber piles are not permitted. Concerns with pile bent piers include the potential to trap debris, pile stability, and appearance.

   Wall Type Piers: These piers consist of a single row of piles (usually H-piles) encased with concrete to form a wall. They provide more resistance to ice and debris and allow debris to pass through without becoming entangled on the piles. This type of pier is used where more resistance to ice and debris than afforded by the pile bent is needed, and yet the size and expense of a solid shaft pier can be avoided. This type of pier can be constructed by driving the piling, supporting the wall forms on the stream/river/lake bed, placing a seal with a tremie, dewatering, adding reinforcement, and pouring the wall. Pile stability can be a concern and must be evaluated.

Solid Shaft or Multiple Column Piers: These piers are used for major water crossings where tall piers are required or where heavy loads or sizable ice and debris loads may occur. This type of pier includes a footing with the bottom of footing located a minimum of 6'-0" below the stream/river/lake bed. Construction of this type of pier involves driving sheeting to form a cofferdam, excavating inside the cofferdam, driving piles, pouring a seal, dewatering, and placing concrete.

2) Grade Separations
Piers at grade separations are typically multiple column type with a cap. Piers are visible to passing motorists and the emphasis on aesthetics has led to more use of rectangular shaped column type piers, often with form liner treatments or rustication grooves. For narrow ramp bridges, a single shaft pier may be considered. Where aesthetics is not a concern, a round column pier will usually provide the lowest cost.

For the majority of bridges over roadways, piers located within 30 feet of the roadway edge (defined as the edge of the lane nearest to the pier) must be designed to withstand a 600 kip load unless they are protected as specified in LRFD 3.6.5.1. This may impact the aesthetics by requiring inclusion of a crash strut. The alternative is to provide columns with a substantial cross-section designed to resist the crash load or protect them with a TL-5 barrier. See Article 11.2.3 of this manual for complete pier protection policy and requirements.

For bridges over railroads, piers located within 25 feet of the centerline of railroad tracks must either be of "heavy construction" or have crash walls. Refer to Article 11.2.3.2.2 of this manual for complete requirements.

See Section 11 of this manual for additional guidance on piers.

2.4 Final Bridge Plans and Special Provisions

The primary purpose for preparing the Final Bridge Plan and special provisions is to communicate the geometric, material, and procedural requirements for the construction of a bridge. Several audiences will use the final bridge plan or contract documents during the life of the bridge. Initially, contractors use the documents to prepare their bids. A clear, accurate, and complete set of documents will result in competitive bidding.

Well-communicated information reduces contractor uncertainty regarding what is required for different elements of construction.

During construction, many parties will use the contract documents. For example, surveyors will locate and mark the position of working points, fabricators and construction engineers will prepare shop drawings and other submittals/drawings, inspectors and suppliers will use the documents for their work, and the contractor's forces will use the documents.

After construction of the bridge the detailed plans will be referenced when modifying the bridge (e.g., adding signage), performing load rating of the bridge, or rehabilitating/replacing the bridge.

The Final Bridge Plan contains geometric information, a schedule of quantities and pay items for the bridge, traffic phasing (if applicable), limits of removal of existing structures and foundation items (if applicable), foundation details, substructure details, superstructure details, typical sections, utilities (if applicable), survey information, and other miscellaneous items.

Specifications are also required for each project. They describe procedures for award and execution of the contract, how work will be measured and paid, procedures to be followed during execution of the work, and material and testing requirements for items incorporated into the project.

Bridge projects use specifications from four different sources:

1) Most of the specifications used for a project are provided in MnDOT's *Standard Specifications for Construction*. They are necessarily general in nature and are intended to cover all types of MnDOT projects.

2) The Bridge Office has assembled additional specifications. Because they are not included in the standard specifications they are called special provisions. A list of available standard bridge special provisions (*2018 "SB" Bridge Special Provisions*) is provided on the Bridge Office web site at: http://www.dot.state.mn.us/bridge/construction.html. Special provisions address a variety of work items, ranging from concrete placement to the fabrication and installation of expansion joint devices. Not all of the special provisions are intended to be used on every project; use only those applicable to the project.

3) The Bridge State Aid Unit has additional standard bridge special provisions that apply to local road bridge projects.

4) Custom special provisions. If a work item is of such unique character that the standard specifications and the standard bridge special provisions don't describe or address the work, a custom special provision will need to be prepared. Custom special provisions may be generated for any number of items. Items may include schedules (e.g., dates the contractor will have access to certain portions of the project) or lists of required submittals, etc.

In general, information that is highly graphical or geometric in nature should be presented on plan sheets. Large amounts of information conveyed with text should be assembled in special provisions.

A specification or special provision usually contains the following five sections:
1) Description of work
2) List of the materials used (and their specifications)
3) Construction requirements for the work
4) Description of how the work will be measured
5) Basis of payment (pay item for the work)

2.4.1 Final Design Instructions

Unless specified otherwise within this manual, design all structures in accordance with the current AASHTO *LRFD Bridge Design Specifications*. For those few cases where LRFD specifications have not been created or adopted, discuss options with the State Bridge Design Engineer prior to beginning final design.

Design railroad bridges according to the current AREMA specifications for the live load specified by the railroad. Additional notes concerning the design of railroad bridges:

1) Railroad bridges will usually be designed with simple spans to avoid uplift from the live load.
2) Bridges for the Duluth Mesabe & Iron Range Railway require a special live load.

Plans and documents prepared during the preliminary design phase should be reviewed prior to beginning final design. These documents include:
1) Preliminary Bridge Plan
2) Bridge Construction Unit Foundation Recommendation Report
3) Design Study Report (if completed)
4) Preliminary Design Folder (found in ProjectWise)

When reviewing preliminary plans, pay particular attention to geometry and utilities. Check the layout. This includes reviewing grades, stationing, end slopes, beams, railings, roadways, shoulders, and the median (if applicable).

2.4.1.1 Superstructure

Space beams so moments in fascia beams will not be larger than moments in interior beams.

2.4.1.1.1 Framing Plan

For steel beams and pretensioned I-beams, deck projections beyond the centerline of the fascia beam should generally not exceed the smallest of:

1) Depth of beam: During construction, overhang support brackets that support deck forms, safety walkway, etc., contain a diagonal member that is supported off the beam bottom flange. If the overhang exceeds the beam depth, analyze to check if bracing of the beam is required. Include a note in the bridge plan if bracing is required.
2) 40% of the beam spacing: This limit keeps the deck overhang moment and the exterior beam dead load within a reasonable range. If exceeded, Section 9 deck tables cannot be used and a special design is required.
3) Deck coping width + barrier width + 1′-0″ + ½ flange width: This keeps the design truck wheel within the limits of the exterior beam top flange, thereby ensuring that the live load will not govern the deck overhang design.

For rectangular pretensioned beams, deck projections beyond the centerline of the fascia beam should generally not exceed the smaller of:

1) 40% of the beam spacing: This limit keeps the deck overhang moment and the exterior beam dead load within a reasonable range. If exceeded, Section 9 deck tables cannot be used and a special design is required.
2) Deck coping width + barrier width + 1′-0″ + ½ beam width: This keeps the design truck wheel within the limits of the exterior beam top flange, thereby ensuring that the live load will not govern the deck overhang design.

Provide a minimum slab projection beyond the tip of the flange of 6 inches.

2.4.1.1.2 Bridge Decks and Slabs

For bridges with reinforced concrete decks or slabs, the deck or slab may be cast in one lift (monolithic) or two lifts (deck/slab plus low slump wearing course). Note that the wearing course and the future wearing course are separate and distinct items.

Bridge Deck Protection Policy

For new bridge decks and slab span superstructures, utilize:

- High Performance Concrete (3YHPC). In remote areas of the state where ready mix suppliers cannot produce 3YHPC, use Low Cracking High Performance Concrete (3YLCHPC).
- Monolithic Deck or Slab (no separate wearing course).
- Synthetic Fibers (a combination of micro and macro synthetic fibers).

A list of common conditions that may warrant exceptions to the above is provided in Table 2.4.1.1.2.1. Note that these exceptions apply only to bridges with AADT greater than 2,000.

For new bridges in remote areas and bridges that meet any of the conditions found in Table 2.4.1.1.2.1, the Preliminary Bridge Plans Engineer will consult with the Regional Bridge Construction Engineer and the District to determine the appropriate concrete mix and whether a wearing course is required. The concrete mix type and type of deck or slab (monolithic deck/slab or deck/slab with wearing course) will be included in the Preliminary Bridge Plan.

Table 2.4.1.1.2.1
Bridge Conditions That Require Consideration of a Low Slump Wearing Course ①

Condition	Commentary
1) Project locations where HPC concrete is not available.	Not all MN concrete plants have successful history with HPC concrete production and delivery.
2) Bridge is located on a constant grade < 0.83%.	Variations in superstructure deflections and finishing tolerances can make positive drainage difficult.
3) Bridge has a continuous steel superstructure with degree of curvature > 10 degrees.	Behavior of steel superstructure deflections and rotation during sequential pouring can be difficult to adequately predict to achieve ride tolerance.
4) Skew > 30 degrees on 2 spans or more with an aspect ratio (deck width/span length) > 0.5.	Finishing rails must deflect uniformly to produce the most uniform cross-section. Finishing of skewed bridges is best accomplished by placing wet concrete uniformly on all beams within the span by setting the finishing machine on a similar skew to substructures.
5) Bridge is located on a vertical curve with approach grades > 3% and support skews > 20 degrees.	Where a vertical profile and skew exists, the difference in elevation at either rail may produce a warped superelevation if finished on skew.
6) Superelevation transition occurs on the bridge.	Finishing machines cannot easily accommodate variable superelevation breaks during a pour.
7) Bridge deck or slab has a longitudinal construction joint due to traffic staging or large deck width.	Multiple pour placements with longitudinal construction joints are more prone to cracking during deflections incurred during the casting sequence. A concrete wearing course placed after major deflections have occurred results in better crack size control.
8) Variable width bridges such as single-point interchanges that are difficult to finish with a finishing machine.	Finishing machines have limited ability to expand width, and finishing outside of the screed rail locations requires finishing equipment with higher risk of placement irregularities.

① Applies only to bridges with AADT greater than 2,000.

2.4.1.1.3 Diaphragms and Cross Frames

For most bridges, the orientation of the primary superstructure elements is parallel to the centerline of the bridge. Aside from slab bridges, most bridges in Minnesota are supported on multiple beam lines. The beam lines are typically spaced on 5 to 15 foot centers. These bridges usually have diaphragms or cross frames, which serve a number of purposes:

1) They provide compression flange bracing during erection and construction of the bridge.

2) They increase lateral load distribution (more beams or girders participate in carrying live loads).
3) They provide a load path for lateral loads to be carried from the deck to the bearings.

During final plan assembly, specify the type of diaphragm on the framing plan, the deck cross section, and the longitudinal section.

For bridges with integral or semi-integral abutments, the end diaphragm also functions as an abutment element.

2.4.1.2 Pedestrian Bridges

Pedestrian bridges shall be designed in accordance with the *Guide Specifications for Design of Pedestrian Bridges*. Several additional constraints are placed on pedestrian bridges to ensure they are accessible, safe, and durable:

1) For guidance regarding determination of pedestrian bridge width, refer to Article 2.1.2 of this manual under **Shared-Use Paths and Pedestrian Walkways on Bridges**.
2) The maximum grade permitted on a pedestrian bridge is 8.33%. A grade flatter than the maximum is preferable. When the grade equals or exceeds 5%, provide a 5'-0" platform for each change in elevation of 2'-6". Also, a handrail is required when the grade equals or exceeds 5% per ADA requirements.
3) Protective screening, preferably a chain link fence system or a railing system, must be placed on both sides of the bridge where the bridge crosses roadway traffic. The height of the fence or railing system must be 8'-0" above the top of the sidewalk. For sites with special aesthetic treatments involving ornamental railings, a minimum height of 6'-0" will be allowed. Where a fence or railing system is also required on retaining walls that are connected to the bridge, it is recommended that the same system (chain link fence or railing) be utilized throughout; although heights may vary.
4) Provide a 6'-0" clear platform at the bottom of each ramp.
5) Provide a platform at each abrupt change in a horizontal direction. The minimum plan dimension for a platform is 5'-0" by 5'-0".
6) Lay out the profile grade such that there are no abrupt grade breaks at expansion joint devices. This will ensure that the cover plate does not bind on the deck or stick up above the deck. If a grade break is required, locate the expansion joint device completely to one side of break or the other.
7) Only in the rare case where handicap accessibility need not be provided can stairs be incorporated into a design. When stairs are provided, use the following guidelines:

   a) Provide stairs with a 1'-0" tread and a 6" rise.
   b) Adjust the sidewalk or superstructure elevations to make all risers 6" tall.
   c) The preferred number of risers in a flight of stairs is 14 to 16. The maximum number is 19.
8) Detail the rails in accordance with the following:
   a) Refer to Section 13 of this manual for metal railing height and spindle spacing requirements.
   b) When required, place handrails 2'-8" above the top of the deck.
9) Provide an electrical ground for continuous chain link fences, ornamental railings, and metal handrails. If appropriate, provide bicycle ramps on pedestrian bridges that contain stairs.

Materials

Use steel, prestressed concrete, reinforced concrete, or timber for the superstructure of pedestrian bridges. Aluminum is not an acceptable material for use in any portion of the superstructure.

The minimum structural steel thickness is $^{1}/_{4}$ inch for pipe or tube sections and $^{5}/_{16}$ inch for all other sections. The minimum thickness requirements do not apply to railings. Provide structural tubing details that are watertight or designed such that moisture cannot be trapped in or on the member to accelerate corrosion.

Use a high performance concrete mix for the deck of a pedestrian bridge. Contact the Regional Bridge Construction Engineer to determine which mix is most appropriate for the site: 3YHPC-M or 3YLCHPC-M.

The Brazilian hardwood known as IPE, though very durable, is not an accepted decking material on state or federally funded projects. If the use of IPE wood is desired by the owner, local funds are the only option for payment. In addition, a maintenance agreement is required to identify repair and replacement thresholds.

Stay-in-place deck forms for prefabricated steel truss spans are acceptable with District/Metro maintenance concurrence.

Bridge Substructure

Use reinforced concrete supported on piling, drilled shafts, or spread footings for bridge substructures as recommended in the Bridge Construction Unit Foundation Recommendations report. Incorporate drainage systems (Detail B910) into the abutments as needed.

Bridge Superstructure
To limit transverse deck cracking due to negative flexure, provide additional longitudinal bars in the top of the deck over the piers. Stagger the ends of the additional longitudinal bars to transition the capacity of the section. (See Figures 9.2.1.8 and 9.2.1.9.)

Detail anchorages for the piers and abutments to resist uplift and overturning forces associated with wind loads.

Provide a cover plate over all pedestrian bridge expansion joint openings to protect pedestrians from a tripping hazard.

Type 5.0 strip seals with expansion joint openings up to 5.0 inches are allowed on pedestrian bridges since the joint is concealed by a cover plate.

Highway Geometrics
Meet MnDOT design standards for horizontal and vertical clearances for a pedestrian bridge over a roadway.

2.4.1.3 Temporary Bridges and Widening

Temporary Bridges
Temporary bridges are used to detour traffic while removal of an existing bridge and construction of a new bridge occur on the mainline of the roadway.

Design temporary bridges in accordance with the LRFD Specifications using the HL-93 live load with an associated load factor of 1.75.

For work zones with posted speeds of 45 mph or less, the bridge railing, bridge railing-to-deck connection, and deck overhang must meet the requirements of Test Level 2 or higher. For speeds greater than 45 mph, Test Level 3 is the minimum standard.

Use of glued-laminated (glulam) wood panels with a bituminous wearing course, where the glulam panels are placed transversely across beam supports, is an option for temporary bridge decks that has been highly successful. Follow the guidance below for this bridge deck type:

- Design glulam panels to meet AASHTO LRFD requirements. See BDM Article 8.2 for guidance on timber bridge decks and BDM Article 8.7.4 for a glulam panel design example.

[Table 8.4.1.2.3-2]

- Use laminates made of visually graded Southern Pine (Identification Number 48) for glulam panels. The minimum net finished panel depth allowed is 5 inches.

- For the bridge railing, use temporary portable precast concrete barriers (Standard Plate 8337) anchored to the glulam panels per Detail B919. The bridge railing, bridge railing-to-deck connection, and glulam panel overhang will meet the requirements of MASH Test Level 3 when anchored in accordance with Detail B919 and the following bullets. Overhangs that do not meet the limits below require a special design.
  - The maximum overhang allowed is 7 feet, measured from the centerline of beam to the edge of the glulam panel deck. In addition, the maximum distance from the centerline of fascia beam to the back bottom edge of the portable barrier is 3 feet.
  - For the bituminous wearing course, the minimum thickness allowed at the gutterline is 2 inches and the maximum is 4 inches. The portable barriers must be supported by wood plank risers with a thickness that matches the bituminous wearing course thickness at the gutterline. This ensures the barrier is at the correct height from the roadway to meet the test level requirements.
  - Loading is limited to the self-weight of the overhang components shown in B919, plus a maximum construction load of 0.020 ksf applied to the area between the back bottom edge of the portable barrier and the edge of deck.
  - Follow guidance provided in MnDOT *Memo to Designers #2019-01* and the MnDOT *Temporary Barrier Guidance Manual* regarding the required deflection distance behind the portable barriers to the edge of deck.

Temporary Widening

Temporary widening occurs when staging requires widening of an existing bridge while construction of an adjacent new bridge occurs.

Design structural components of the temporary widening to meet or exceed the capacity of the existing bridge components. For a temporary widening, match the deck material of the existing bridge.

For temporary widening projects, design the barriers, the barrier/deck connection, and the deck overhang to meet the barrier test level required for the roadway.

2.4.1.4 Bridge Approaches

In most cases, the bridge approach panel will be included with the roadway grading plans for a project. Guidance for the treatment and details of approach panels can be found in the following:

Bridge Approach Treatment

The approach treatment standard sheets describe the limits and treatment of excavation and backfill near the abutments. These sheets are found in the *MnDOT Standard Plans Manual*, Figures 5-297.233 and 5-297.234. The Preliminary Bridge Plan contains a note indicating which approach treatment sheet to use.

Bridge Approach Panel

The standard sheets covering bridge approach panels are found in the *MnDOT Standard Plans Manual*, Figures 5-297.223 through 5-297.228. These figures cover standard approach panels for abutments with joints, abutments without joints, abutments with different amounts of skew, different mainline pavement types, and miscellaneous details. The Bridge Preliminary Plan contains a note indicating which approach panel sheets to use.

Specify a concrete wearing course on approach panels when the bridge deck has a concrete wearing course. The wearing course on the approach panels will be placed at the same time as the wearing course on the bridge. Include the approach panel wearing course quantity in the summary of quantities for the superstructure.

2.4.1.5 Survey

When assembling the survey sheets for final plans, verify that the most current grading plans are being used.

Include the centerlines and object lines for the abutment and pier footings on the final design survey sheets. Also identify and locate all test piles.

2.4.1.6 Utilities

The Bridge Office Preliminary Plans Unit in coordination with the District Traffic Engineer determines if structural provisions must be made for safety lighting (roadway, navigation, inspection, etc.), signing, or signals. Coordination is also done with the MnDOT District Project Manager regarding the need for other types of utilities.

2.4.1.6.1 Suspended Utilities & Utilities Embedded in Bridges

The conduit for utilities is to be suspended below the deck on hanger systems between the beams. Locate the entire conduit and hanger system above the bottom of the beams and generally below the diaphragms or in the lower openings of a cross frame diaphragm. To minimize the impact to the structure in the future, avoid casting conduits for utility companies in the deck, sidewalk, or barriers/parapets.

Use polyvinyl chloride (PVC) coated hot dipped galvanized rigid steel conduit (RSC) for utilities requiring conduit. Use galvanized steel hangers and supports.
Roadway lighting conduit ($1^1/_2$ inch diameter maximum) will be allowed in barriers/parapets (maximum of 2 per barrier/parapet) and sidewalks.

Suspended water, sewer, communications, and electrical power (less than 35kV) utility systems are allowed on bridges. However, natural gas pipelines are considered a safety risk and will not be allowed.

For hanging conduit systems on bridges with parapet or semi-integral type abutments, and when conduit is embedded in concrete barrier, deck, or sidewalk, use a combination expansion/deflection fitting at the abutments. This will accommodate horizontal movements (due to temperature change, creep, shrinkage, etc.) and vertical movements (due to jacking operations for bearing replacement, etc.).

For hanging systems on bridges with integral abutments, only an expansion fitting is required at the abutments.

The temperature movements of RSC approximate those of concrete. Consequently, lateral bracing is not needed. Choose a transverse spacing for the conduits that permits proper placement of concrete between embedded anchors.

Typical conduit and utility details are available in the Bridge Office final design cell library, available at:

http://www.dot.state.mn.us/bridge/drafting-aids.html

2.4.1.6.2 Buried Utilities

To protect structures, restrictions on the location of new or existing buried utilities and drainage pipes must be considered near new and existing bridge substructures, box culverts, and wall structures (e.g.- conventional concrete cantilever retaining walls, sheet pile walls, mechanically stabilized earth (MSE) walls, etc.).

Location restrictions, installation techniques, protection measures, and plan review are required for utilities in the utility-critical region immediately adjacent to and below the structure, defined below for the type of structure. Allowance of new or existing utilities in the utility-critical region of a new or existing structure requires review and approval from the MnDOT Bridge Office. Additional restrictions on the location of utilities may be specified in other documents relevant to the project.

For purposes of this section, utilities are defined as any utility requiring a permit as well as State owned utilities and stormwater structures. Dry utilities are defined as facilities that do not carry fluid or gasses, such as power and communications. Wet utilities are defined as those facilities that carry fluid or gasses, except they do not include roadway edge drains or subsurface drains associated with the bridge or wall structure.

Certain types of utilities may pose a significant risk to foundations when placed in the utility-critical region. If these types of utilities were to fail, the foundation could be at risk of failure due to the loss of material from localized scour or erosion. The determination of high-risk utilities will be made on a case-by-case basis by the Bridge Office and will be based on many factors including, but not limited to utility location, flow pressure, flow rate, structure size, and utility size. Additional restrictions to those contained in this document could be applied to utilities that pose a significant risk to the foundations.

Utilities Near Bridge Substructures and Conventional Concrete Cantilever Retaining Walls Supported on Spread Footings, Prefabricated Modular Block Walls (PMBWs) Without Earth Reinforcement, and Box Culverts

The limits of the utility-critical region are defined as 50 feet lateral to, 50 feet below, and the distance from the bottom of footing elevation or leveling pad to the highest point of ground (up to 15 feet maximum).

Within the utility-critical region, three zones have been identified to provide general guidance for MnDOT approval. See Figure 2.4.1.6.2.1 for the definition of the zones.

The following restrictions on utilities are dependent on their position relative to the structure:

Zone ①

For new substructures/walls/box culverts, new utilities are to be placed and existing utilities relocated outside of Zone ①. For existing substructures/walls/box culverts, new utilities are to be placed outside of Zone ①. If this is impractical or impossible, the requirements for utilities in Zone ① are:

- General
  - All new pipes and conduits must be designed for any surcharge loading due to structure bearing pressures and possible resulting deformations. All existing pipes and must be checked for structural adequacy.

  - Future open trench excavation for utility access is prohibited to protect the substructure, wall, or box culvert from potential undermining. Other forms of excavation may be permitted in this zone with Bridge Office approval.
- New or Existing Dry Utilities
  - Dry utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the substructure or wall) in Zone ①.
  - Utility owners may choose to case dry utilities to allow for future maintenance or access. However, casing is not required.
- New or Existing Wet Utilities
  - Wet utilities may be located transversely (defined as perpendicular to the substructure or wall, or up to 30 degrees from perpendicular) in Zone ①.
  - Wet utilities may <u>not</u> be located longitudinally (defined as parallel to the substructure or wall, or up to 60 degrees from parallel) in Zone ①.
  - All wet utilities in Zone ① require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.
  - All wet utilities in Zone ① must be cased for additional protection against risk of leakage and soil loss. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements. If facilities are too large or cannot be cased effectively, a risk analysis approved by the Regional Bridge Construction Engineer and a site-specific design is required.

<u>Zone ②</u>

The requirements for utilities in Zone ② are:

- General
  - Future excavation for maintenance or replacement will be permitted with proper sheeting and shoring. No unbraced open cuts will be allowed.
- New or Existing Dry Utilities
  - Dry utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the substructure or wall) in Zone ②.
  - Utility owners may choose to case dry utilities to allow for future maintenance or access. However, casing is not required.
- New or Existing Wet Utilities

- Wet utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the substructure or wall) in Zone ②.
- All wet utilities in Zone ② require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.
- All wet utilities in Zone ② must be cased for additional protection against risk of leakage and soil loss, with the exception of some stormwater facilities. Casing is required for stormwater pipes with any of the following:
  - Flow velocities greater than 10 ft/s, where flow rate is determined for maximum velocity condition.
  - Pipe diameters or spans greater than 48 inches.

  Other stormwater facilities need not be cased in Zone ② unless required by contract specifications or as recommended by the Bridge Office. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements. If facilities are too large or cannot be cased effectively, a risk analysis approved by the Regional Bridge Construction Engineer and a site-specific design is required.

Zone ③

The only requirement for new or existing utilities in Zone ③ is that all wet utilities require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.

If the conditions above cannot be met, options include relocation or replacement of the utility or placing the substructure or wall on deep foundations (piles or drilled shafts). However, note that there are some restrictions for wet utilities placed in Zone ① of deep foundations. Also note that deep foundations are not an option for PMBWs or box culverts.

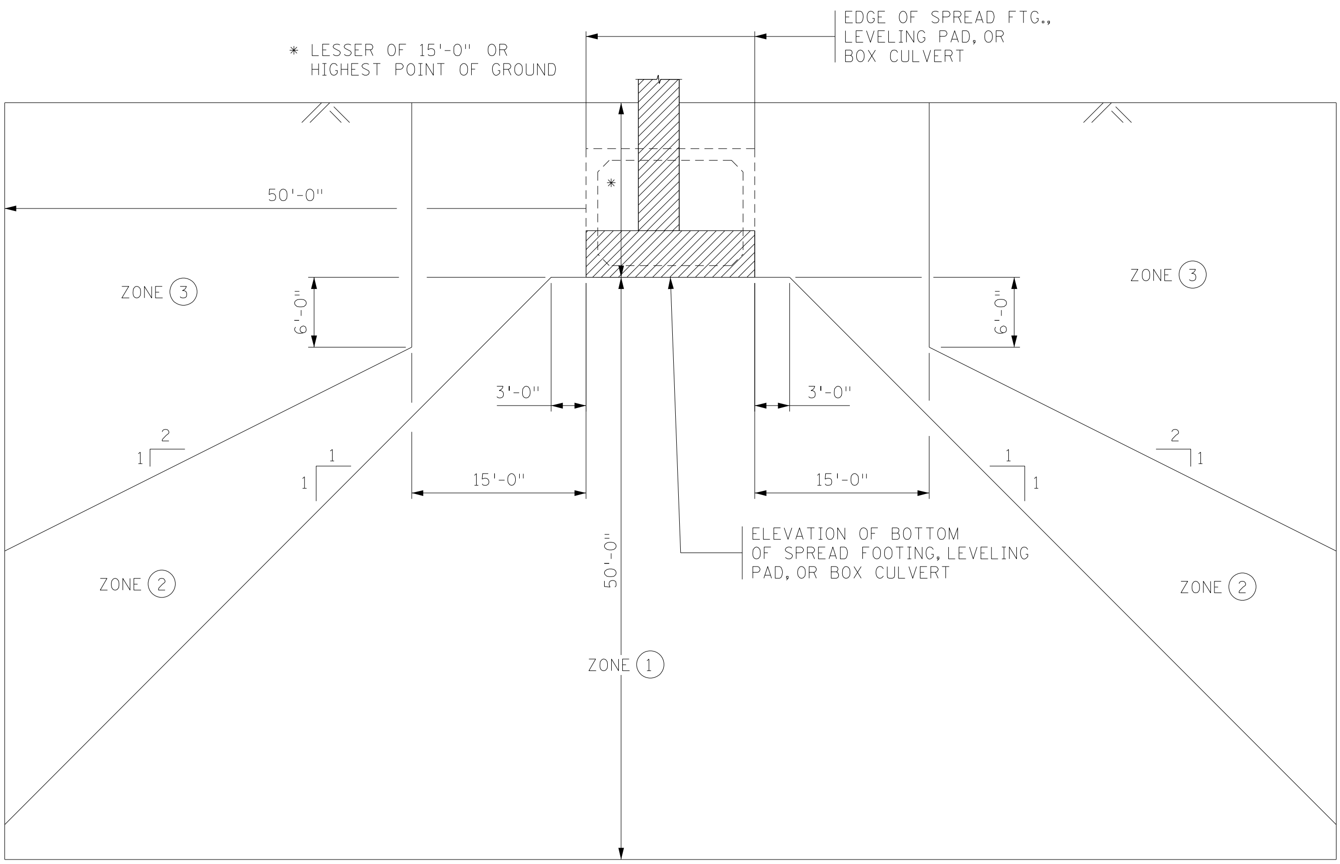


Figure 2.4.1.6.2.1

Utilities Near Bridge Substructures and Conventional Concrete Cantilever Retaining Walls Supported on Spread Footings, Prefabricated Modular Block Walls (PMBWs) Without Earth Reinforcement, and Box Culverts

Utilities Near Bridge Substructures and Conventional Concrete Cantilever Retaining Walls Supported on Deep Foundations (Piles or Drilled Shafts)

Note that there is a risk of interference between the piles/shafts and utilities that is dependent on the pile/shaft locations, size of the utility, and the order of construction. Coordination is required between the designer of the structure, the road designer, and the utility owner to avoid any potential interference.

There are two utility-critical regions, defined in Figure 2.4.1.6.2.2, for substructures and walls on deep foundations. The left side is for end bearing piles and the right side for piles that do not extend to bedrock.

Zone ①

The requirements for utilities in Zone ① are:

- General
  - Future open trench excavation for utility access is prohibited to protect the substructure or wall from potential undermining. Other forms of excavation may be permitted in this zone with Bridge Office approval.
- New or Existing Dry Utilities
  - Dry utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the substructure or wall) in Zone ①.
  - Utility owners may choose to case dry utilities to allow for future maintenance or access. However, casing is not required.
- New or Existing Wet Utilities
  - Wet utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the substructure or wall) in Zone ①.
  - All wet utilities in Zone ① require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.
  - All wet utilities in Zone ① must be cased for additional protection against risk of leakage and soil loss. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements. In lieu of casing, a risk analysis approved by the Regional Bridge Construction Engineer is acceptable for substructures on deep foundations. The risk analysis must include a sensitivity study that examines the deep foundation stability and capacity if soil loss should occur.

Zone ②

The requirements for utilities in Zone ② are the same as for Zone ①, except that for wet utilities, casing is not required.

There are no utility restrictions outside of Zone ②.

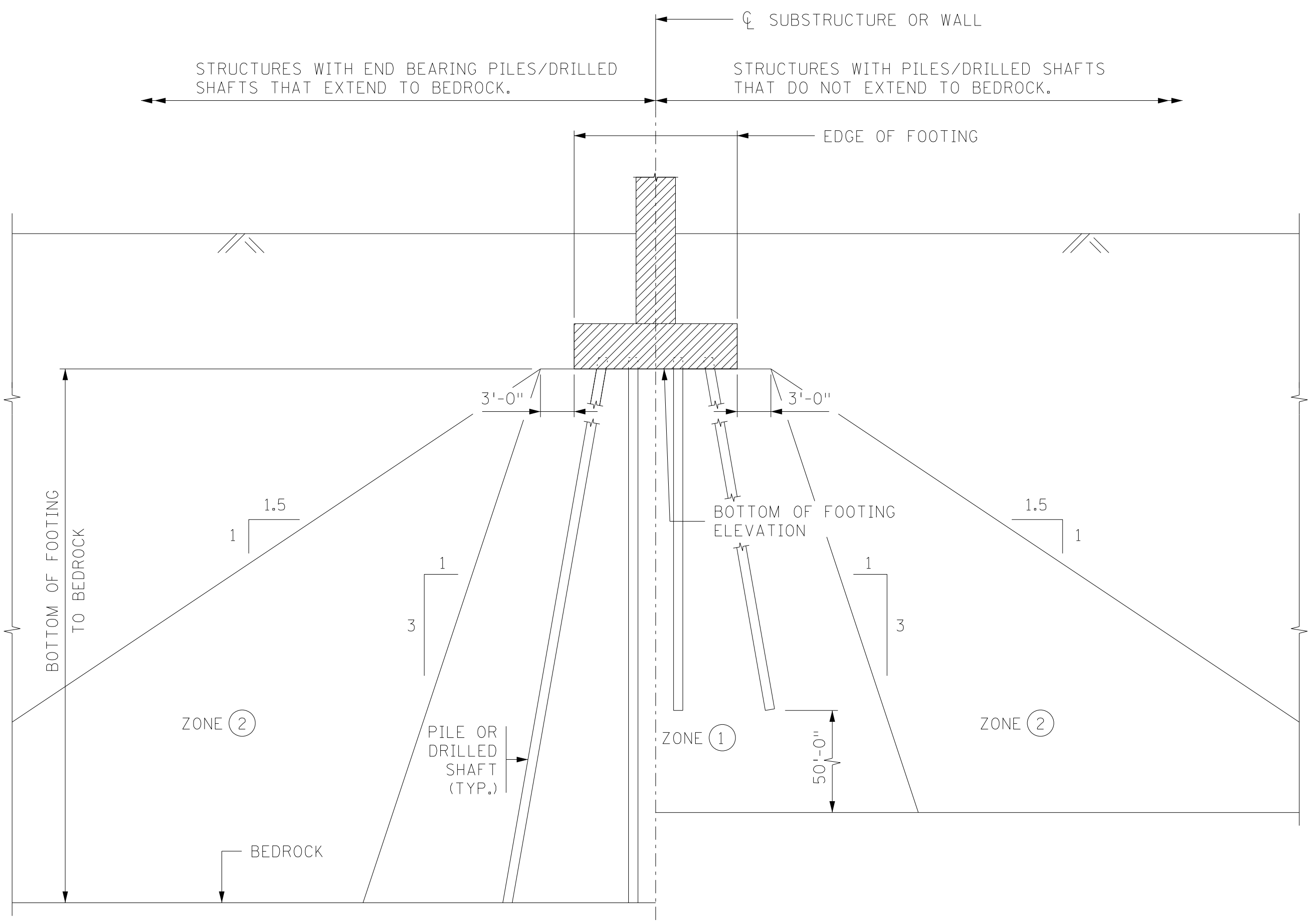


Figure 2.4.1.6.2.2

Utility-Critical Region for Utilities Near Bridge Substructures and Conventional Concrete Cantilever Retaining Walls Supported on Deep Foundations

Utilities Near Mechanically Stabilized Earth (MSE) Walls and Prefabricated Modular Block Walls (PMBWs) With Earth Reinforcement

For new and existing MSE walls and PMBWs, the limits of the utility critical region are defined as 50 feet lateral to the front of the wall, 50 feet measured laterally beyond the end of the earth reinforcement, 50 feet below the bottom of the leveling pad, the top of the fill on the front side of the wall, and the top of the reinforced earth/retained fill in the back of the wall. Refer to Figure 2.4.1.6.2.3.

Five zones have been identified to provide general guidance for MnDOT approval when new or existing utilities are near MSE walls and PMBWs constructed using earth reinforcement. The zones are defined in Figure 2.4.1.6.2.3.

In addition to the requirements below, walls near utilities where stray electrical current could be an issue are subject to the following restrictions. For existing walls with earth reinforcement consisting of metal strips, new buried high voltage lines or other utilities/infrastructure that may cause stray currents in the soil mass must be located a minimum of 20 feet outside the limits of Zone ⑤. For new walls where new or existing buried high voltage lines or other utilities/infrastructure that may cause stray currents in the soil mass are located closer than 20 feet to the limits of Zone ⑤, use of non-metallic geosynthetic earth reinforcement is required.

The restrictions on utilities are dependent on their position relative to the structure:

<u>Zones ①, ②, and ③</u>

Zones ①, ②, and ③ restrictions for MSE walls and PMBWs are the same as for bridge substructures on spread footings. See restrictions given previously for these zones found under **Utilities Near Bridge Substructures and Conventional Concrete Cantilever Retaining Walls Supported on Spread Footings, Prefabricated Modular Block Walls (PMBWs) Without Earth Reinforcement, and Box Culverts**.

<u>Zone ④</u>

The requirements for utilities in Zone ④ are as follows:

- General
  - When excavating within this zone, install shoring or sheet piling (without damage to the geomembrane), to protect the reinforced zone.

    - Future excavation for maintenance or utility replacement will be permitted with proper sheeting and shoring. No unbraced open cuts will be allowed.
- New or Existing Dry Utilities
    - Dry utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the wall) in Zone ④.
    - Casing is not required in Zone ④.
- New or Existing Wet Utilities
    - Wet utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the wall) in Zone ④.
    - All wet utilities in Zone ④ require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.
    - Casing is not required in Zone ④.

<u>Zone ⑤</u>

Existing utilities in Zone ⑤ cannot remain in place during construction of new MSE walls and PMBWs as they will be disturbed. New utilities are to be placed outside of Zone ⑤. If this is impractical or impossible, the requirements for utilities in Zone ⑤ are as follows:

- General
    - Future excavation is prohibited in this zone to protect the structural integrity of the wall.
- New Dry Utilities
    - Dry utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the wall), but only during the original construction of the wall.
    - Utility owners may choose to case dry utilities to allow for future maintenance or access. However, casing is not required for dry utilities.
- New Wet Utilities
    - Wet utilities may be located transversely (defined as perpendicular to the wall, or up to 30 degrees from perpendicular), but only during the original construction of the wall.
    - Wet utilities may <u>not</u> be located longitudinally (defined as parallel to the wall, or up to 60 degrees from parallel) in Zone ⑤.
    - All wet utilities in Zone ⑤ require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or

joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.

- All wet utilities in Zone 5 must be cased for additional protection against risk of leakage and soil loss. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements. If facilities are too large or cannot be cased effectively, a risk analysis approved by the Regional Bridge Construction Engineer and a site-specific design is required.

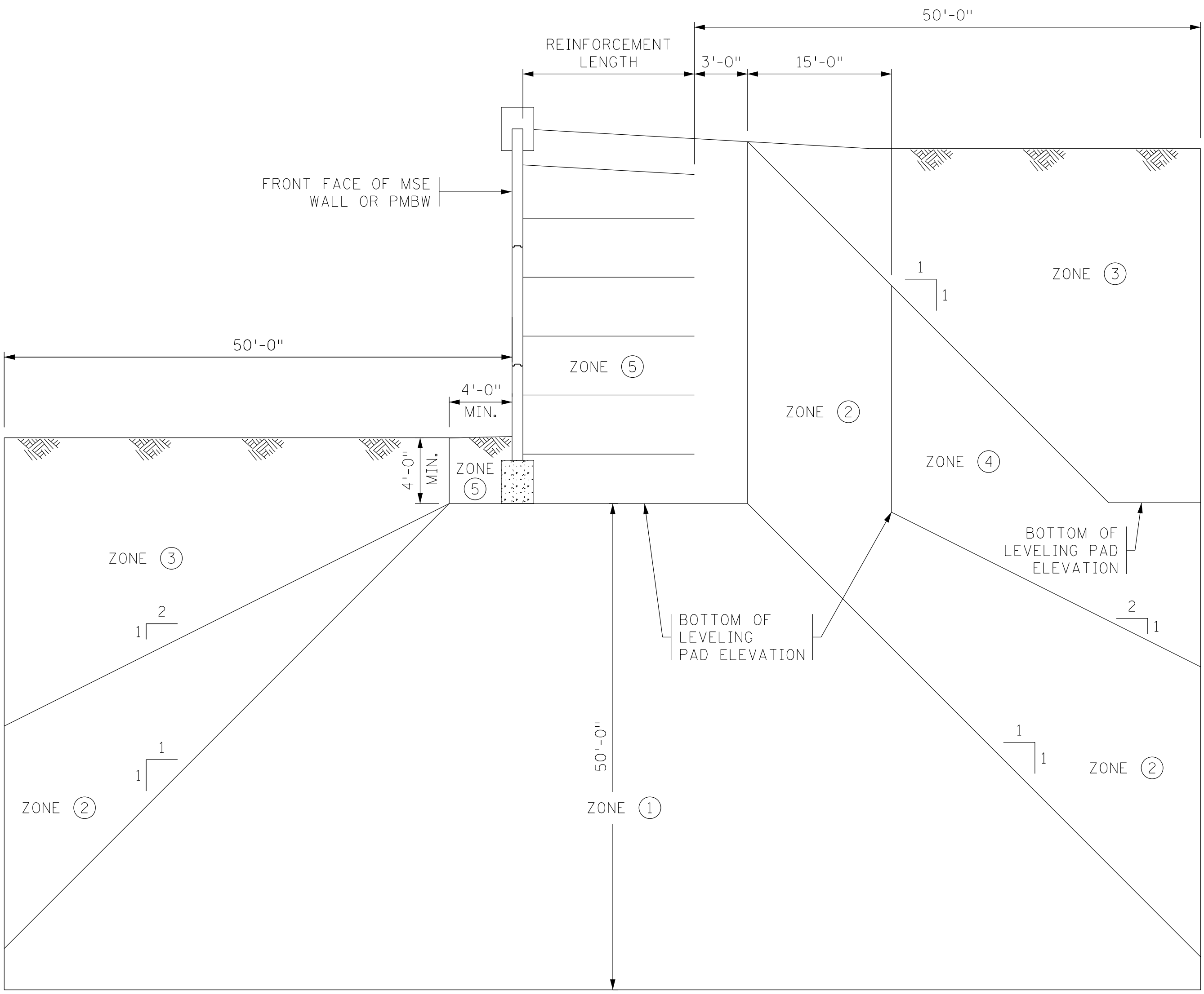


Figure 2.4.1.6.2.3

Utility-Critical Region for Utilities Near Mechanically Stabilized Earth (MSE) Walls and Precast Concrete Block Walls (PMBWs) With Earth Reinforcement

Utilities Near Sheet Pile Walls, Soldier Pile Walls, and Noise Walls
For new and existing sheet pile walls, soldier pile walls, and noise walls, the limits of the utility-critical region are defined as 50 feet lateral to each edge of pile/post, 30 feet below the pile/post tip, and the vertical distance above the pile/post tip to the ground elevation on the each side of the wall.

Within the utility-critical region, three zones have been identified to provide general guidance for MnDOT approval. See Figure 2.4.1.6.2.4 for the definition of the zones.

Zone ①

For new walls, new utilities are to be placed or existing utilities relocated outside of Zone ①. For existing walls, new utilities are to be placed outside of Zone ①. If this is impractical or impossible, the requirements for utilities in Zone ① are as follows:

- General
    - Future excavation is prohibited in this zone for sheet pile and soldier pile walls to protect the structural integrity of the wall. Limited future excavation is allowed for utility access for noise walls with proper consideration of post support and adequate bracing.
- New or Existing Dry Utilities
    - Dry utilities may not be located transversely (i.e., defined as perpendicular to the wall, or up to 30 degrees from perpendicular) to sheet pile walls in Zone ①. Dry utilities may be located transversely to soldier pile and noise walls (defined as perpendicular to the wall, or up to 30 degrees from perpendicular), provided they meet the minimum clearances to the piles/posts shown in the partial wall elevation of Figure 2.4.1.6.2.4.
    - Dry utilities may be located longitudinally (defined as parallel to the wall, or up to 60 degrees from parallel) to sheet pile walls, soldier pile walls, or noise walls, but only before the original construction of the wall.
    - All dry utilities must be cased in Zone ① for sheet pile and soldier pile walls to allow for future maintenance or access. Casing is not required for noise walls. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements.
- New or Existing Wet Utilities
    - Wet utilities may not be located transversely (defined as perpendicular to the wall, or up to 30 degrees from perpendicular) to sheet pile walls in Zone ①. Wet utilities may be located transversely to soldier pile and noise walls (defined

as perpendicular to the wall, or up to 30 degrees from perpendicular), provided they meet the minimum clearances to the piles/posts shown in the partial wall elevation of Figure 2.4.1.6.2.4.

- Wet utilities may <u>not</u> be located longitudinally (defined as parallel to the wall, or up to 60 degrees from parallel) to sheet pile walls, soldier pile walls, or noise walls in Zone ①.
- All wet utilities in Zone ① require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.
- All wet utilities must be cased in Zone ① for soldier pile walls to provide additional protection against risk of leakage and soil loss. Casing is not required for noise walls. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements. If facilities are too large or cannot be cased effectively, a risk analysis approved by the Regional Bridge Construction Engineer and a site-specific design is required.

<u>Zone ②</u>

For new walls, new utilities are to be placed and existing utilities relocated outside of Zone ②. For existing walls, new utilities are to be placed outside of Zone ②. If this is impractical or impossible, the requirements for utilities in Zone ② are:

- General
  - Future open trench excavation for utility access is prohibited to protect the wall from potential undermining. Other forms of excavation may be permitted in this zone with Bridge Office approval.
- New or Existing Dry Utilities
  - Dry utilities may be located transversely or longitudinally (i.e., in any direction between parallel and perpendicular to the wall) in Zone ②.
  - Utility owners may choose to case dry utilities to allow for future maintenance or access. However, casing is not required.
- New or Existing Wet Utilities
  - Wet utilities may be located transversely (defined as perpendicular to the wall, or up to 30 degrees from perpendicular) in Zone ②.
  - Wet utilities may <u>not</u> be located longitudinally (defined as parallel to the wall, or up to 60 degrees from parallel) in Zone ②.

- All wet utilities in Zone ② require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.
- All wet utilities must be cased in Zone ② for sheet pile and soldier pile walls to provide additional protection against risk of leakage and soil loss. Casing is not required for noise walls. Refer to the *MnDOT Utility Accommodation & Coordination Manual*, Article VII.E, for casing requirements. If facilities are too large or cannot be cased effectively, a risk analysis approved by the Regional Bridge Construction Engineer and a site-specific design is required.

Zone ③

The only requirement for utility installations in Zone ③ is that all wet utilities require pipe with gaskets that meet Standard Plate 3006 for concrete culverts and storm drains, or joints designed to prevent leakage due to pressurized flow for other utilities or pipe materials.

An exception to this is for anchored walls, where anchors extend from the front face of the wall into the fill behind the wall. For this case, replace the boundary and requirements for Zone ③ behind the anchored wall with the boundaries and requirements for Zone ⑤ and Zone ② outlined under **Utilities Near Mechanically Stabilized Earth (MSE) Walls and Prefabricated Modular Block Walls (PMBWs) With Earth Reinforcement**.

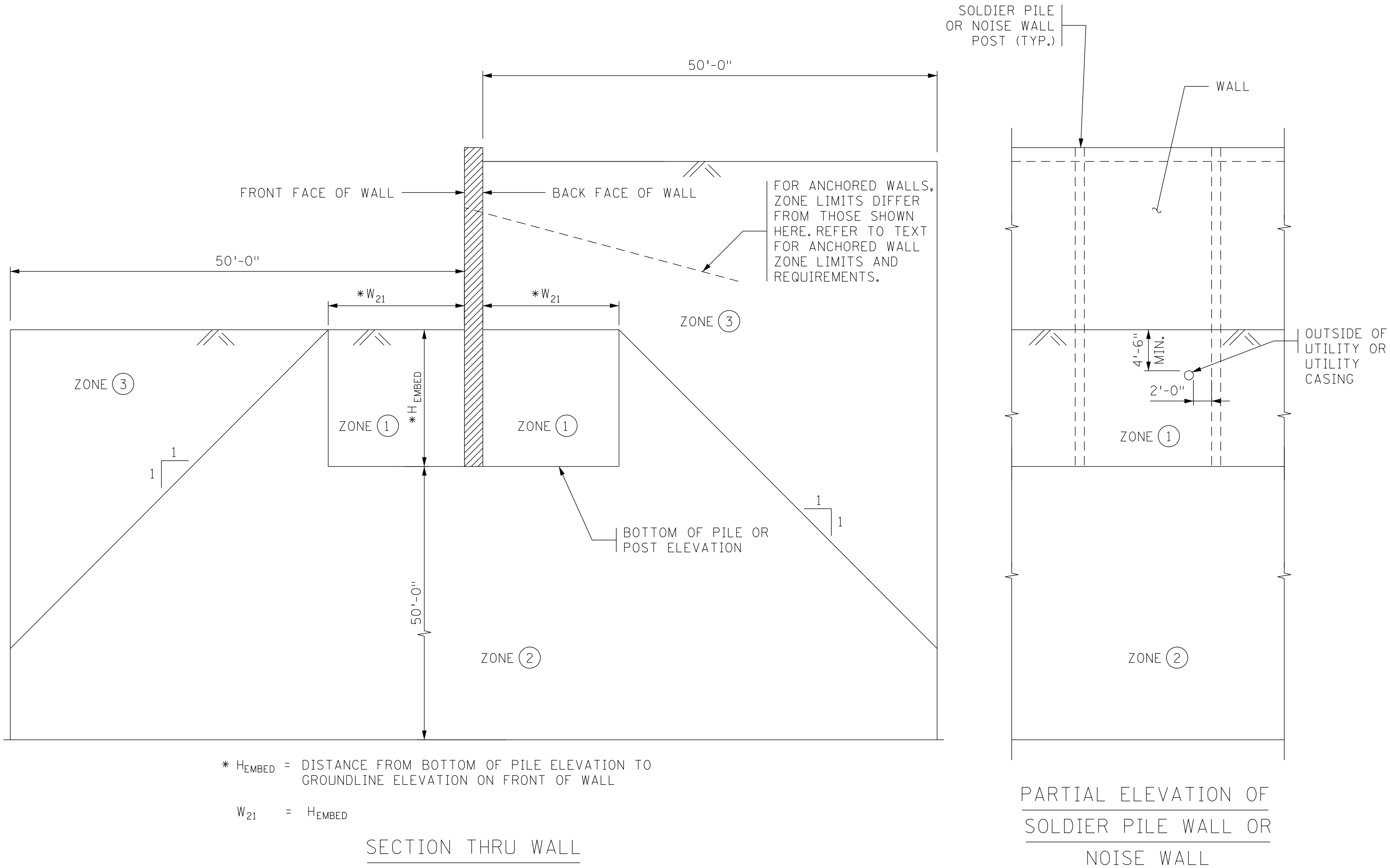


Figure 2.4.1.6.2.4
Utility-Critical Region for Utilities Near Sheet Pile Walls, Soldier Pile Walls, and Noise Walls

2.4.1.7 Precedence of Construction Documents

Designers, while striving to produce accurate error-free construction documents, may at times end up with documents that have conflicting content. A hierarchy has been established to determine which content is governing for a project. In general, the more project specific the document, the higher the document's position in the hierarchy. Section 1504 of the *Standard Specifications for Construction* describes the precedence of construction documents for a project:

If discrepancies exist between the Contract documents, the following order of precedence applies:

1) Addenda
2) Special Provisions
3) Project-Specific Plan Sheets
4) Supplemental Specifications
5) Standard Plan Sheets and Standard Plates
6) Standard Specifications

If discrepancies exist between dimensions in the Contract documents, the following order of precedence applies:

1) Plan dimensions
2) Calculated dimensions
3) Scaled dimensions

2.4.1.8 Design Calculation Requirements

Office practice is to permit the limit states to be exceeded by a maximum of 3%. However, caution should be exercised to ensure that a 3% exceeded limit state at a particular location does not adversely affect the structure load rating.

2.4.2 Final Plans

The plan order shall typically follow this list:

- General Plan and Elevation
- Cross Section and Pay Items
- Staging Plan
- Working Point Layout
- Removal Details
- Abutment Details and Reinforcement
- Pier Details and Reinforcement
- Framing Plan
- Beam Details
- Superstructure Details and Reinforcement
- Plan Details (Railing, Expansion Joint, Slope Paving, Conduit, etc.)
- B-Details
- As-Built Bridge Data

- Surveys
- Borings
- Unrevised Informational Sheets

When presenting geometric information, enough baseline information needs to be provided to permit others to verify the information presented. For example, the top of roadway elevations presented on a bridge layout sheet can be confirmed by others using vertical curve information on the general elevation and the cross slopes provided on the typical transverse section.

In general, do not present the same dimensions several times. Providing dimensions in multiple locations increases the chance that not all dimensions will be updated as changes occur during the design process.

The clarity of the details used in plan sets should be a primary concern of designers. Only the simplest details should combine the presentation of concrete geometry and reinforcement. In most cases there is less confusion if two details are used, one to convey concrete geometry and a second to identify and locate reinforcement.

Show the initials of the individuals responsible for the design, drafting, design check, and drafting check on all plan sheets except the as-built sheet, survey sheets, boring sheets, and unrevised information sheets (such as those showing alignment tabulations, superelevation transitions, or aesthetics) taken from the preliminary bridge plan. Note that these unrevised informational sheets are to be placed at the end of the bridge plan. Only include informational sheets with direct relation to the bridge plan and remove any project wide informational sheets only pertaining to other bridges. For the boring sheets, show the initials of the individuals responsible for the drafting and drafting check. Similarly, all sheets, except the as-built sheet, survey sheets, boring sheets, and unrevised information sheets must be certified by a Professional Engineer licensed in the State of Minnesota.

In most cases, details are presented with stationing increasing as one moves from the left side to the right side of the sheet. Always include a north arrow on plan views. Plan views are typically oriented with north arrows pointing toward the top or to the right of the sheet. Stationing increases for northbound and eastbound traffic.

2.4.2.1 Drafting Standards

The Bridge Office has adopted standards to be used when drafting plan sheets. Download *Summary of Recommended Drafting Standards* from:

http://www.dot.state.mn.us/bridge/drafting-aids.html

2.4.2.2 Drafting Guidelines

Sheet Layout and Continuity

Read plans from a contractor's perspective to check that they contain all information needed to build the bridge. Make sure enough dimensions are given for constructability. Use extra details for uncommon work. Use perspective views when clarity is needed.

Use sheets efficiently. Balance the drawings on sheets to avoid one sheet being empty while another is crowded. Use additional sheets, as needed, to avoid crowding details on sheets. Make sure that details, data, and other information given on more than one sheet agree between sheets. Avoid unnecessary repetition of details and notes.

Large-scale corner details are required for all skewed bridges and for other complex corners.

Round dimensions to the nearest $^1/_8$ of an inch.

Note and dimension bar splices.

Cross-referencing sheets to details is recommended.

Use bill of reinforcement tables for all but very minor reinforced concrete work. Do not enlarge details (such as rebar bends) just to fill up space. Referencing bar bend details by letter to various generic shapes should never be used.

Keep details together for abutments, piers, superstructure, etc.

For abutments, piers, and other complex drawings, use different views and sections to separate dimensions and reinforcement.

Place pile design loads and notes pertaining to a particular substructure on the sheet that contains the footing plan view.

For bridges with numerous footings and curved alignment, a separate foundation layout drawing is recommended.

If the plan contains numerous variable dimensions and other data (especially for framing plans and beams), make use of tables to keep this data in order.

On the Framing Plan, show bearing type beside each bearing point instead of lines and arrows, which tend to clutter the drawing.

For simple beam spans (prestressed beams, etc.), dimension beam spacing at pier cap along centerline of the pier(s). Include supplemental dimensions along centerline of bearing for curved and flared structures.

On projects with staged construction, use enough drawings to clearly indicate how the bridge construction is to be coordinated with the staging. Keep structure units together. Reinforcement and quantity tabulations are to be split between stages.

On repair projects, clearly indicate cut lines and extent of all removals. If there is a saw cut, be sure to use a straight line (WT=5). If elevations are taken off original plans, note as such and require the contractor to verify elevations in the field.

When it appears that plan notes, such as procedure descriptions, specifications, etc., will become excessively wordy, relegate these notes to the special provisions.

List general notes first and specific numbered notes last. Number specific detail notes with circles and reference the detail to which they apply. Place all notes together on the right hand side of the sheet.

Leave extra lines in the Summary of Quantities and Bill of Reinforcement for additions. Also, leave extra space in the list of notes.

Pay Quantities

Make computations neat and readable. Strive for continuity. These computations may be needed for future reference and the reader must be able to interpret them.

Box in or underline computation totals for quicker take off. Initial, date, and put the bridge (or project) number on every computation sheet.

Two sets of independently worked quantity computations are required for each pay item.

Arrange design and quantity computations into a neat and orderly package.

In addition to pay item quantities, compute informational quantities. This applies to final plans for new bridges and bridge repair plans for existing bridges. Do not include these quantities in the bridge plan, but instead submit them to the Bridge Estimating Unit for use in developing the Engineer’s Estimate. Examples of information quantities include: summary quantities for conduit systems, summary quantities for drainage systems, cubic yard quantities for concrete items paid for by the square foot or linear foot, and miscellaneous minor items such as polystyrene and waterproofing membrane. Sample forms for reporting informational quantities are available at: http://www.dot.state.mn.us/bridge/design.html

Additional Drafting Guidance

Additional drafting resources can be found at:

http://www.dot.state.mn.us/bridge/drafting-aids.html

These resources include:

- *Suggested Reinforcement Detailing Practices*
- *Guidelines for Checking Final Design Bridge Plans*
- Bridge Preliminary and Final Design Seed Files
- Bridge Preliminary and Final Design Cell Libraries

2.4.2.3 General Plan and Elevation

The General Plan and Elevation sheet is intended to summarize the primary features and horizontal geometry of the bridge. Figure 2.4.2.3.1 shows an example General Plan and Elevation sheet and Figure 2.4.2.3.2 shows a Typical Cross Section sheet with pay items.

Plan

On the plan view identify the following: working points, working line, centerlines, utilities, location of in-place bridges or substructures, ditch drains, deck drains, lights, and nameplate. Label the following: span lengths, deck width, size of angles between the working line and centerlines, horizontal curves, minimum horizontal clearance to substructure units, point of minimum vertical clearance for each roadway under the bridge, extent of slope protection, roadway stationing and elevations, and distance between twin bridges. Provide a north arrow. Tie bridge dimensions to working points. Show the direction of traffic for each design lane.

Elevation

Present the primary vertical geometry of the bridge on the elevation view. This consists of vertical curve data, end slopes, existing ground lines, footing elevations, limits of excavation, grading notes, ditch clean out along railroad tracks, and scale. Label bearings as fixed, expansion, or integral. Also label piers, spans, abutments, and slope protection.

For bridges over waterways, provide hydraulic information. Required information includes: channel bottom width, low member elevation, design high water elevation, and assumed flowline elevation.

For grade separation bridges, provide the minimum vertical and horizontal clearances. In addition, provide the dimension from centerline of pier to toe of slope protection. If there is no side pier, give the dimension from toe of slope to centerline of roadway. Dimension the pier, lane, and shoulder widths on the roadway under. Lane slopes on the roadway under are typically omitted, but can be provided if space permits.

When illustrating slope protection use a straight slope line; do not follow the ditch radius curve. To reduce confusion concerning slopes, do not show slopes as 1:2. Many individuals are unsure of whether the first or second number is the horizontal part of the slope. Show the slopes graphically. Where slopes need to be provided in text, explicitly call out the slopes (e.g., 1V: 2H).

Typical Cross Section

The typical cross section is the third general view of the structure. Combined with the general plan and elevation views, the primary geometry of the bridge is conveyed. On the typical cross section show transverse bridge dimensions, lane widths and slopes, beam depth and spacing for all spans, roadway slab and concrete wearing course thicknesses, type of barrier, medians, sidewalks, profile grade location, working line, and all centerlines.

For staged construction projects, provide the in-place, interim, and final cross sections, including temporary anchored or unanchored safety barrier locations.

For complex projects, consider creating a separate plan sheet for pay items and notes for clarity.

Utilities

Show all utilities that may affect bridge construction. Note what is to be done with them (will they be moved, will they no longer be used or do they need to be protected during construction).

Miscellaneous

Provide a Design Data block on the General Plan and Elevation Sheet of the bridge plan set. The information given in the block provides a summary of the primary parameters used for the design. Information in the Design Data block includes: design specifications, design live load, design material properties, future wearing course load assumed in the design, deck area, traffic data, and the operating rating for the new structure. See Appendix 2-C.

Also on this sheet, identify the governing standard specifications for construction. Show a north arrow on the plan view and include a block for engineering certification. Present applicable project numbers on the first sheet; project numbers depend on specific funding sources, so there may be both state and federal project numbers.

Review the title block to ensure it accurately describes the bridge. Within the title block provide span lengths to the nearest foot and the bridge type identification number. The three-character identification number should follow the numbering scheme provided in Appendix 2-A of this manual.

Include any additional standard construction notes and the sheet list for the plan set on the first sheet of the plan set. Provide the schedule of quantities for the entire bridge in tabular form on the second or third sheet of the plan.

Standard practice for placement of bridge nameplates and bench marks is as follows:

- Generally, include a single nameplate on each bridge with its own bridge number, attaching it to the traffic barrier/parapet in the right hand corner as one approaches the bridge. Specifically, for two-way bridges with a roadway running north and south, place the nameplate in the southeast corner. For a roadway running east and west, provide the nameplate at the northeast corner. On twin bridges (two one-directional bridges that are side-by-side, but each with their own bridge number), place a nameplate on each right hand corner approaching each bridge. Exceptions to the above include railroad, timber, pedestrian, and boardwalk bridges, where the nameplate is to be placed on a substructure unit. On bridges that are widened, redecked, or that receive rail modifications that result in additional roadway width, install a new nameplate with the original year completed and the year renovated.

- Do not show a bench mark disk in the bridge plan. Placement of bench mark disks on bridges is no longer needed.

Check if ditch drainage pipe is necessary for the project. If drainage pipe is necessary and the contract has multiple portions (grading, bridge, etc.), identify which portion of the contract contains the pipe. Label ditch drainage pipe on plan and elevation views.

Concrete or aggregate slope protection is used along a highway or railway (grade separation structures). Aggregate slope protection is used more frequently when pedestrian traffic below the bridge is limited. Stream crossings use riprap slope protection supported on a granular or geotextile filter. The Preliminary Bridge Plan will indicate the type of slope protection to be used.

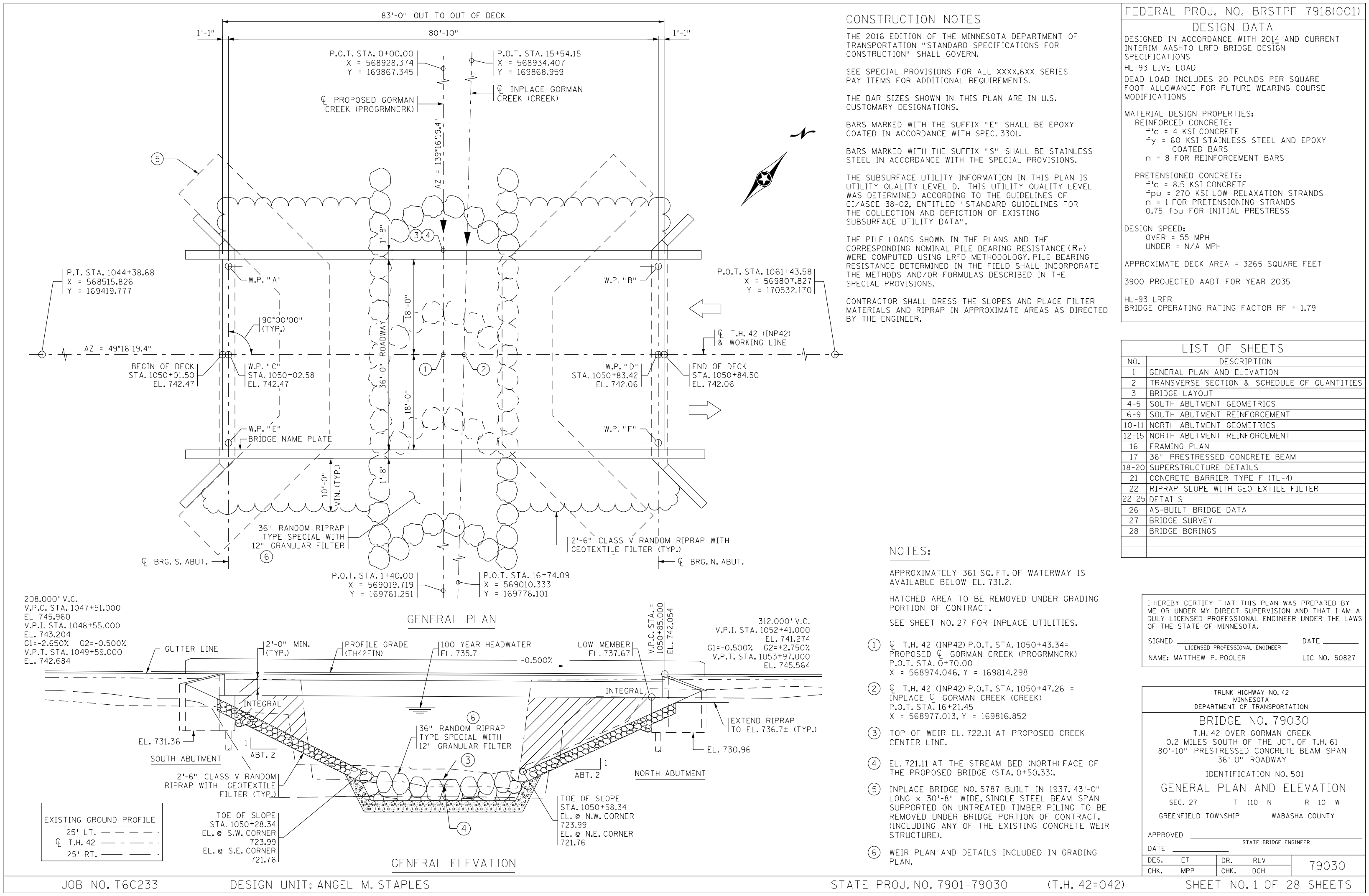
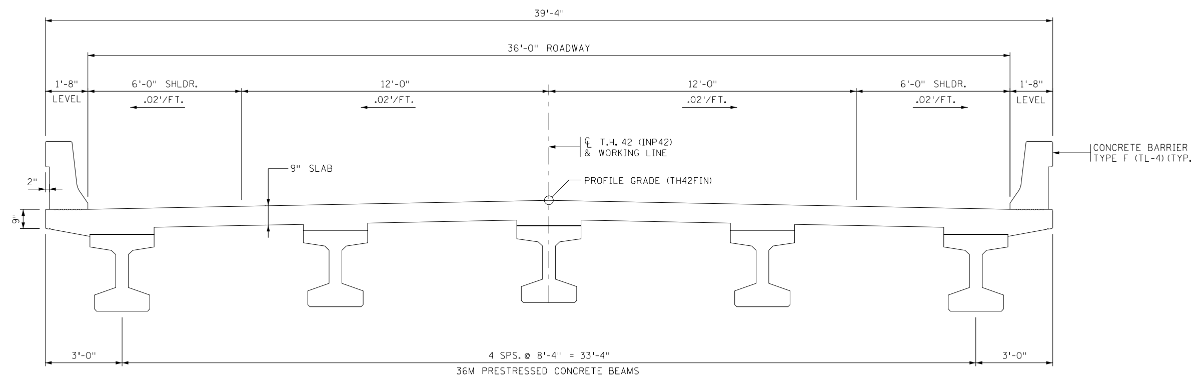


Figure 2.4.2.3.1
General Plan and Elevation



WEST SIDE

TRANSVERSE SECTION

EAST SIDE

	SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE				
	ITEM NO.	ITEM	UNIT	QUANTITY	
	2104.601	REMOVE REGULATED WASTE MATERIAL (BRIDGE)	LUMP SUM	1	
	2401.501	STRUCTURAL CONCRETE (3B52)	CU. YD.	82	(P)
1	2401.513	TYPE F (TL-4) BARRIER CONCRETE (3S52)	LIN. FT.	194	(P)
2	2401.541	REINFORCEMENT BARS (EPOXY COATED)	POUND	33500	(P)
	2401.541	REINFORCEMENT BARS (STAINLESS-60KSI)	POUND	460	(P)
	2401.601	STRUCTURE EXCAVATION	LUMP SUM	1	
	2401.618	BRIDGE SLAB CONCRETE (3YHPC-M)	SQ. FT.	3265	(P)
	2402.590	ELASTOMERIC BEARING PAD	EACH	10	
	2405.502	PRESTRESSED CONCRETE BEAMS 36M	LIN. FT.	411	(P)
	2405.511	DIAPHRAGMS FOR TYPE 36M PREST BEAMS	LIN. FT.	34	(P)
3	2442.501	REMOVE EXISTING BRIDGE	LUMP SUM	1	
	2452.519	C-I-P CONCRETE TEST PILE 85 FT LONG 12"	EACH	1	
	2452.519	C-I-P CONCRETE TEST PILE 100 FT LONG 12"	EACH	1	
	2452.527	PILE REDRIVING	EACH	2	
	2452.603	C-I-P CONCRETE PILING 12"	LIN. FT.	1020	
	2502.502	DRAINAGE SYSTEM TYPE (B910)	LUMP SUM	1	
	2511.501	RANDOM RIPRAP CLASS V	CU. YD.	407	(P)
	2511.515	GEOTEXTILE FILTER TYPE VII	SQ. YD.	734	(P)

NOTES:

1. QUANTITY INCLUDES THAT PORTION FOR THE APPROACH PANELS.
2. QUANTITY INCLUDES THAT PORTION FOR BARRIER REINFORCEMENT ON THE APPROACH PANEL.
3. REMOVE INPLACE BRIDGE NO. 5787 (INCLUDES THE CONCRETE SPILLWAY).

CERTIFIED BY ______ LICENSED PROFESSIONAL ENGINEER ______ DATE	TITLE: TRANSVERSE SECTION AND SCHEDULE OF QUANTITIES	DES: ET	DR: RLV	APPROVED:	BRIDGE NO. 79030
NAME: MATTHEW P. POOLER LIC. NO. 50827		CHK: MPP	CHK: DCH		
		SHEET NO. 2 OF 28 SHEETS			

Figure 2.4.2.3.2
Typical Cross Section

2.4.2.4 Bridge Layout and Staking Plan

The Bridge Layout Sheet is used by surveyors to locate the bridge in space with its primary geometry. The primary geometry consists of centerline of roadway(s) and centerline of substructure bearings. Working points are located on substructure bearing centerlines where they are intersected by fascia beam lines and working lines. By providing stationing, X-coordinates, and Y-coordinates for each of the working points, the position of the bridge can be fixed. Figure 2.4.2.4.1 contains an example.

In Figure 2.4.2.4.1, the working line and its azimuth are labeled. Also shown is the angle of intersection between the working line and each of the substructure units and roadways under the bridge. As a primary geometry line, the working line should be labeled throughout the plan set.

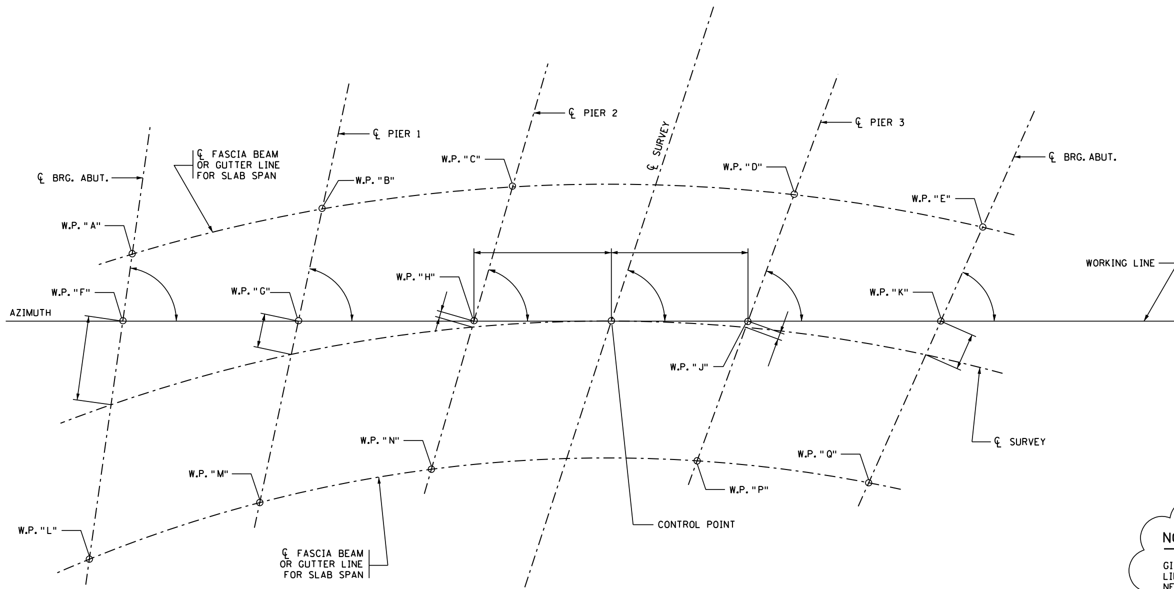
Place the control point at the intersection of the survey line and centerline of cross road, track etc. For river crossings, place the control point at an abutment centerline of bearing. Label the control point with its coordinates. Coordinates of the control point and the working points should be given to three decimals of a foot. Tie the working point layout to the control point. Present dimensions in feet (a note on the sheet should say the same).

List the coordinates for all working points in a table labeled "DIMENSIONS BETWEEN WORKING POINTS". Stations and the distances between working points should be presented to the nearest 0.01 foot. Coordinates are assumed to be given in the Minnesota State Plane Coordinate System. If another system is used, place a note on the sheet identifying the system used.

In addition to horizontal geometry, a limited amount of vertical geometry is provided on the Bridge Layout Sheet. The vertical geometry consists of elevations and drops. The elevation at the top of roadway and the bridge seat is provided for all working points located on beam lines and is appended to the "DIMENSIONS BETWEEN WORKING POINTS" table.

Drop or elevation difference information is provided for each substructure unit. Drop information is summarized in the "TOP OF ROADWAY TO BRIDGE SEAT" table. The table should contain the following items:
1) Deck Thickness
2) Stool Height
3) Beam Height
4) Bearing Height
5) Total Height

If the drop dimension is the same for all beam lines, provide a single value for each substructure unit. If the drop dimensions vary at substructure locations, provide a value for each beam line. Total values should be given in both inches and decimals of a foot to two places.



WORKING POINT LAYOUT

NOTES TO DESIGNER:

GIVE ALL DISTANCES IN TABLE ALONG STRAIGHT LINES BETWEEN WORKING POINTS TO THE NEAREST .01.

SHOW NORTH ARROW DIRECTION.

LOCATE WORKING LINE TANGENT TO ℄ SURVEY AT THE CONTROL POINT.

LOCATE CONTROL POINT AT INTERSECTION OF SURVEY LINES WHENEVER PRACTICAL. IF OVER RIVER, LOCATE CONTROL POINT AT INTERSECTION WITH ℄ BEARING OF ABUTMENT. GIVE STATIONING ON BOTH SURVEY LINES AND X & Y COORDINATES AT CONTROL POINT.

GIVE STATIONING TO NEAREST .01 AND X & Y COORDINATES TO NEAREST .001.

	DIMENSIONS BETWEEN WORKING POINTS																		ELEVATIONS			
POINT	STATION	X-COORDINATE	Y-COORDINATE	A	B	C	D	E	F	G	H	J	K	L	M	N	P	Q	TOP OF ROADWAY	TOP OF RDWY TO BR. SEAT	BRIDGE SEAT	POINT
A																						A
B																						B
C																						C
D																						D
E																						E
F																						F
G																						G
H																						H
J																						J
K																						K
L																						L
M																						M
N																						N
P																						P
Q																						Q

TOP OF ROADWAY TO BRIDGE SEAT						
	DECK THICKNESS	STOOL HEIGHT	BEAM HEIGHT	BEARING HEIGHT	TOTAL	
					INCHES	FEET
ABUT.						
PIER 1						
PIER 2						
PIER 3						
ABUT.						

CERTIFIED BY LICENSED PROFESSIONAL ENGINEER DATE NAME: LIC. NO.	TITLE: BRIDGE WORKING POINT LAYOUT EXAMPLE	DES: DR: CHK: CHK: SHEET NO. OF SHEETS	APPROVED:	BRIDGE NO.

Figure 2.4.2.4.1
Bridge Layout

2.4.2.5 Standard Abbreviations

Use standard abbreviations to clarify information on plan sets and reduce the clutter on a crowded plan sheet. Appendix 2-B presents a list of standard abbreviations that can be utilized in a plan. Define abbreviations used in a plan set on the sheet where they are used or as part of a General Notes sheet.

2.4.2.6 Inclusion of Standard Bridge Details and Bridge Standard Plans in Plan Sets

There are two parts to the Standard Bridge Details Manual: Part I and Part II. They are published on the Bridge Office Web site at:
http://www.dot.state.mn.us/bridge/standards.html

Bridge details are intended, where applicable, to be incorporated into a set of bridge plans.

Bridge Details Manual Part I, typically referred to as the B-Details, are presented in a "portrait" orientation on an 8 1/2" x 11" sheet. Included are details for nameplates, pile splices, bearings, diaphragms, steel superstructures, floor drains, and other miscellaneous details.

Bridge Details Manual Part II consists of details that occupy an entire plan sheet. The majority of these details are for barriers, parapets, medians, prestressed concrete beams, and expansion joints.

Similar to Bridge Details Manual Part II, details from the Bridge Standard Plans Manual are intended to be incorporated into bridge plan sets and occupy an entire plan sheet. The information presented may be much more in-depth as the information for multiple designs is presented on a single sheet. Bridge Standard Plans are only available for precast concrete box culverts.

How standard details are incorporated into a bridge plan will depend on the amount of revisions needed to the details, falling into 3 categories:

1) Standard is unmodified: This is when the standard is used as drawn with no changes, or with all blanks filled in where expected. Box out and place hatching across all specific details that do not apply. Do not remove them. It is not necessary to cross out alternative sizes in tables or alternate dimensions that are not used, or circled notes that are only referred to in the details that do not apply.
2) Standard is modified: This is when details, dimensions, or notes on the standard have been specifically modified from what is shown in the standard. In this case, place the word "MODIFIED" under the B-Detail or after the Figure Number. Also add a box containing a note stating what was modified to help plan readers quickly locate them. Do not "cloud" the changes.

3) Standard is substantially modified: This is when much of the standard is changed by extensive and/or numerous modifications to details, dimensions, and notes to the extent that it ceases to reflect the standard. It will require a judgment call on the part of the engineer as to when this category applies. In this case, remove the State Bridge Engineer approval block and the figure number. Note that for this case, the engineer takes full responsibility for the details shown on the sheet.

2.4.2.7 Standard Plan Notes

Similar to other plan elements, standard plan notes have been prepared to increase the consistency of information presented on final design plans. Plan notes serve a variety of purposes; they communicate design criteria, specific construction requirements, and a variety of notes pertaining to the construction or fabrication of specific bridge elements. Appendix 2-C contains the Standard Plan Notes. These notes have been grouped into the following categories:

- Design Data and Projected Traffic Volumes
- Construction Notes
- Signature Block
- Drainage and Erosion Control
- Excavation and Earthwork
- Reinforcement
- Foundations
- Steel Materials, Fabrication, and Erection
- Concrete Placements
- Welded Steel Bearing Assemblies
- Cutting and Removal of Old Concrete
- Joints and Joint Sealer
- Timber Bridges
- Miscellaneous

Designers unfamiliar with MnDOT's Standard Plan Notes should review the list prior to beginning final design. Reviewing the notes prior to design will familiarize designers with the material properties to be used, and other constraints typically placed on construction. Perform a second review of the notes at the end of design to ensure that all applicable notes were incorporated into the plan set.

2.4.2.8 Quantity Notes and Pay Items

Standard Summary of Quantities Notes

During construction, contractors are compensated according to the work they complete. The value of the work item is identified when the contractor submits their bid. For each work item or pay item the contractor must supply a price. The pay items are coordinated with the *MnDOT Standard Specifications for Construction* and the project special provisions. To clarify what is included in a specific pay item, the Bridge Office has assembled a Standard Summary of Quantity Notes. Like other plan elements, these notes help ensure uniformity across plan sets and permit MnDOT to generate a historical price database that can be used to estimate the cost of future bridges. The Standard Summary of Quantities Notes for bridge projects is listed in Appendix 2-D.

Pay Items

The current MnDOT list of pay items (commonly referred to as the Trns*port List) is located at the following link:

https://transport.dot.state.mn.us/Reference/refItem.aspx

When populating the "SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE" in the bridge plan, it is important to note the following:

1) The Trns*port List shows a 12 digit number under the "ITEM NUMBER" column. Provide only the first seven numbers including the decimal point (all the numbers before the /) for each pay item in the bridge plan.
2) The Trns*port List shows two descriptions for each pay item. Historically, the "ITEM SHORT DESCRIPTION" column was used for pay item descriptions in the bridge plan. Beginning with bridge projects governed by the 2020 Edition of the *MnDOT Standard Specifications for Construction*, the "ITEM LONG DESCRIPTION" column is now used as the pay item description in all bridge plans.
3) Use the "PLAN UNIT NAME" column from the Trns*port List for listing the units in the bridge plan.
4) The item number, item description, and units must read <u>exactly</u> as shown in the Trns*port List, including spaces, dashes, parentheses, etc. In order to avoid errors, it is suggested to copy and paste them directly from the list into the bridge plan.

For each pay item shown in the plan, provide a reconciled quantity estimate. Some pay items are to be designated as "plan quantity pay items", for which payment to the contractor will be based on the quantity given in the plan rather than measuring in the field. For these items, include a "(P)" as an appendix to the item label. For example:

2401.507 STRUCTURAL CONCRETE (3B52) 699 CU YD (P)

Pay items are to be designated as "plan quantity pay items" when the quantity for payment can be calculated using the dimensions given in the plan and the dimensions are not expected to change in the field. Some examples include:

- structural concrete paid for by the cubic yard
- bridge slab concrete paid for by the square foot
- reinforcement bars paid for by the pound
- prestressed concrete beams paid for by the linear foot
- ornamental metal railing paid for by the linear foot

Some examples of pay items that are **not** "plan quantity pay items" include:

- piling paid for by the linear foot (because plan quantity is an estimate only and final pile lengths are determined in the field)
- random riprap paid for by the cubic yard (because plan dimensions are approximate and actual volume may differ)
- removal and patching of concrete slabs paid for by the square foot (because actual area will be determined in the field)
- structural excavation paid for by lump sum (because it is not a calculated value based on plan dimensions)
- bearing assemblies paid for by the each (because it is not calculated using dimensions from the plan)

For minor work items (membrane waterproofing system, bit. felt, joint filler, etc.) that are not official "pay items", avoid using the term "incidental" in plan notes. Instead, tie minor work items to a specific pay item using the term "included in", which will help those in the field to know who is responsible for it. For example: PAYMENT FOR MEMBRANE WATERPROOFING SYSTEM INCLUDED IN ITEM "BRIDGE SLAB CONCRETE (3YHPC-M)".

Miscellaneous

Round off quantities to the nearest pay item unit except for the following:

- Earth excavation to nearest 10 cubic yards.
- Reinforcement bars and structural steel to nearest 10 pounds.
- Piling lengths to nearest 5 feet.

When computing concrete quantities, consider the following:

- For CIP piles that are embedded in a crash strut or an infill wall or a pile bent pier encasement wall, deduct the pile volume when determining the concrete quantity. For these cases, the embedded pile volume is considered substantial enough to affect the concrete quantity. For all other cases where piles are embedded in concrete (e.g. – H-piles embedded in a crash strut, CIP or H-piles embedded in a footing, etc.), do not deduct the embedded pile volume when determining the concrete quantity. The embedded pile volume is minimal and considered unnecessary.

- Do not deduct the rebar volume when determining the concrete quantity.

When computing small bituminous quantities use the following:
Wearing course = 110 pounds / square yard / inch thickness

Shoulder or Wearing Course (6.5%)
0.065 (thickness in inches) (110 pounds) = ___ pounds / sq. yard

Tack Coat = 0.03 gallons / square yard

Binder or Base Course (5.3%)
0.053 (thickness in inches) (110 pounds) = ____ pounds / sq. yard

Compute deck area (rounded to the nearest square foot) by multiplying the transverse out-to-out bridge width by the longitudinal end-of-deck to end-of-deck distance. (Do not include bridge approach panels or paving brackets.)

When computing structural steel quantities, increase the calculated weight by 1.5% to account for the weight of steel for welds and bolt stick-through.

2.4.3 Revised Sheets

Sometimes, revisions to the plan are required after the letting due to an error found in the plan or other issues that arise during construction. When this occurs, use the following procedure:

1) Revise the sheet as follows (See Figure 2.4.3.1):
   i. Make the necessary revisions to the sheet.
   ii. In the revision block, provide the revision number within a triangle border, the revision date, a description of the revision, and the initials of the engineer who approved the revision. When sheets have been revised multiple times, add new entries to the revision block for each sheet revision. Do not remove the revision block entries for the previous revisions.
   iii. "Cloud" the actual revisions to the sheet and include the revision number within a triangle border next to the "clouded" change.
   iv. When sheets have been revised multiple times, remove revision "clouds", revision numbers, and triangles for the previous revisions (except as noted in ii. above). Only "cloud" the current revisions and include the current revision number within a triangle border.

v. Change the sheet number by placing a "-R" and the revision number after the original sheet number. For example, revision 1 to sheet 7 will be designated "SHEET NO. 7-R1", revision 2 will be designated "SHEET NO. 7-R2", etc. For situations where an additional plan sheet must be inserted as part of the revision, repeat the preceding sheet number with an "A" after it. For example, as part of revision 1 where a sheet needs to be added between sheet 5 and 6, designate the new revised sheet as "SHEET NO. 5A-R1".

2) Plot and certify the revised sheet.
3) Draft a transmittal letter from the State Bridge Design Engineer to the Resident Engineer in the District construction office. Submit the letter and the revised sheet to the State Bridge Design Engineer for signature and distribution. Memo templates are available on the Bridge Office network drive. Consultants should contact MnDOT Bridge Office Project Manager to obtain the file.

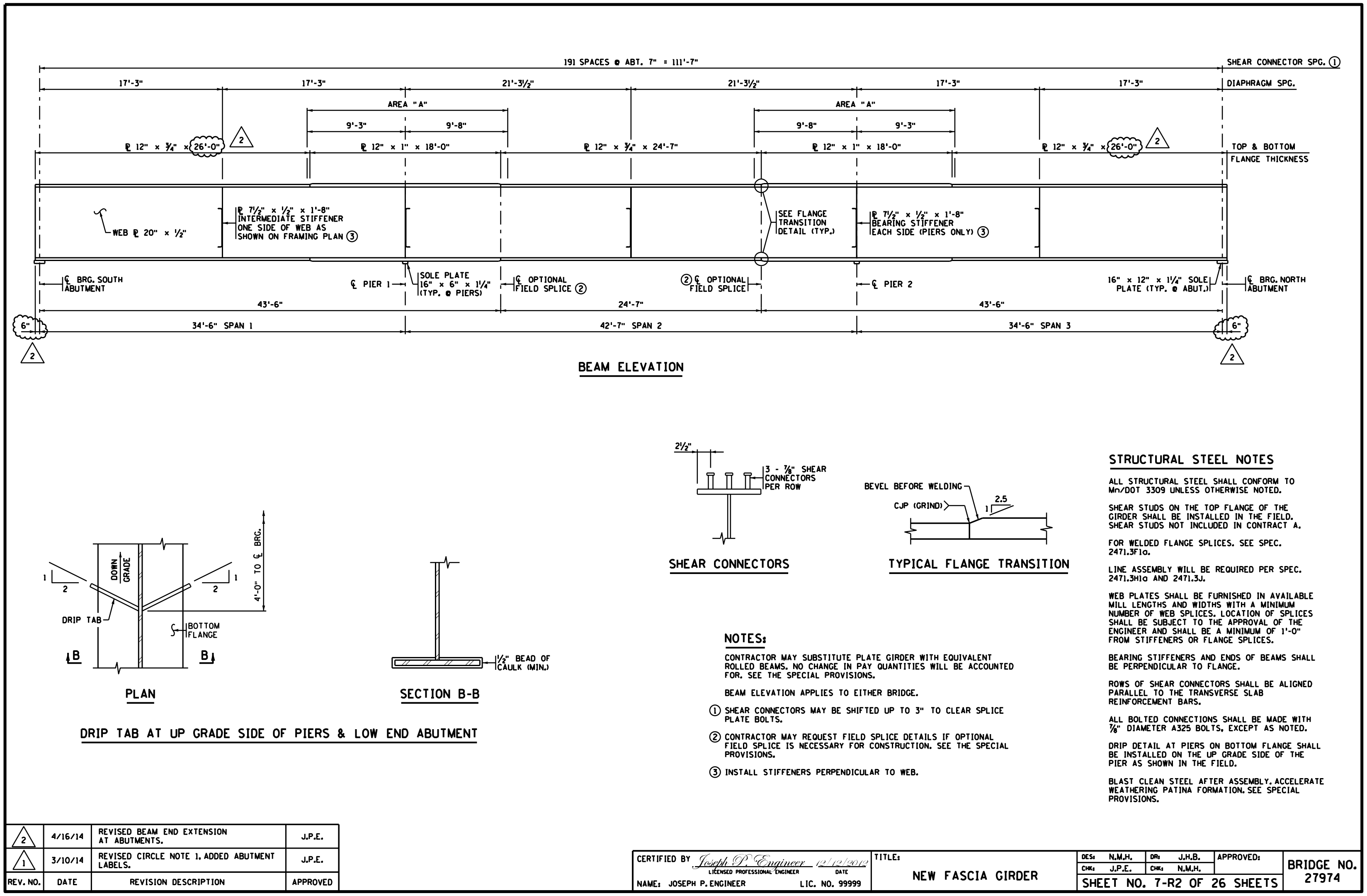

191 SPACES @ ABT. 7" = 111'-7"
SHEAR CONNECTOR SPG. 1
17'-3"
17'-3"
21'-3½"
21'-3½"
17'-3"
17'-3"
DIAPHRAGM SPG.
AREA "A"
9'-3"
9'-8"
AREA "A"
9'-8"
9'-3"
℘ 12" × ¾" × 26'-0"
℘ 12" × 1" × 18'-0"
℘ 12" × ¾" × 24'-7"
℘ 12" × 1" × 18'-0"
℘ 12" × ¾" × 26'-0"
TOP & BOTTOM FLANGE THICKNESS
WEB ℘ 20" × ½"
℘ 7½" × ½" × 1'-8" INTERMEDIATE STIFFENER ONE SIDE OF WEB AS SHOWN ON FRAMING PLAN 3
SEE FLANGE TRANSITION DETAIL (TYP.)
℘ 7½" × ½" × 1'-8" BEARING STIFFENER EACH SIDE (PIERS ONLY) 3
℄ BRG. SOUTH ABUTMENT
℄ PIER 1
SOLE PLATE 16" × 6" × 1¼" (TYP. @ PIERS)
℄ OPTIONAL FIELD SPLICE 2
2 ℄ OPTIONAL FIELD SPLICE
℄ PIER 2
16" × 12" × 1½" SOLE PLATE (TYP. @ ABUT.)
℄ BRG. NORTH ABUTMENT
43'-6"
24'-7"
43'-6"
6"
34'-6" SPAN 1
42'-7" SPAN 2
34'-6" SPAN 3
6"
BEAM ELEVATION
2½"
3 - ⅞" SHEAR CONNECTORS PER ROW
SHEAR CONNECTORS
BEVEL BEFORE WELDING
CJP (GRIND)
2.5
1
TYPICAL FLANGE TRANSITION
DOWN GRADE
DRIP TAB
BOTTOM FLANGE
4'-0" TO ℄ BRG.
B
B
PLAN
½" BEAD OF CAULK (MIN.)
SECTION B-B
DRIP TAB AT UP GRADE SIDE OF PIERS & LOW END ABUTMENT
NOTES:
CONTRACTOR MAY SUBSTITUTE PLATE GIRDER WITH EQUIVALENT ROLLED BEAMS. NO CHANGE IN PAY QUANTITIES WILL BE ACCOUNTED FOR. SEE THE SPECIAL PROVISIONS.
BEAM ELEVATION APPLIES TO EITHER BRIDGE.
1 SHEAR CONNECTORS MAY BE SHIFTED UP TO 3" TO CLEAR SPLICE PLATE BOLTS.
2 CONTRACTOR MAY REQUEST FIELD SPLICE DETAILS IF OPTIONAL FIELD SPLICE IS NECESSARY FOR CONSTRUCTION. SEE THE SPECIAL PROVISIONS.
3 INSTALL STIFFENERS PERPENDICULAR TO WEB.
STRUCTURAL STEEL NOTES
ALL STRUCTURAL STEEL SHALL CONFORM TO Mn/DOT 3309 UNLESS OTHERWISE NOTED.
SHEAR STUDS ON THE TOP FLANGE OF THE GIRDER SHALL BE INSTALLED IN THE FIELD. SHEAR STUDS NOT INCLUDED IN CONTRACT A.
FOR WELDED FLANGE SPLICES, SEE SPEC. 2471.3F1a.
LINE ASSEMBLY WILL BE REQUIRED PER SPEC. 2471.3H1a AND 2471.3J.
WEB PLATES SHALL BE FURNISHED IN AVAILABLE MILL LENGTHS AND WIDTHS WITH A MINIMUM NUMBER OF WEB SPLICES. LOCATION OF SPLICES SHALL BE SUBJECT TO THE APPROVAL OF THE ENGINEER AND SHALL BE A MINIMUM OF 1'-0" FROM STIFFENERS OR FLANGE SPLICES.
BEARING STIFFENERS AND ENDS OF BEAMS SHALL BE PERPENDICULAR TO FLANGE.
ROWS OF SHEAR CONNECTORS SHALL BE ALIGNED PARALLEL TO THE TRANSVERSE SLAB REINFORCEMENT BARS.
ALL BOLTED CONNECTIONS SHALL BE MADE WITH ⅞" DIAMETER A325 BOLTS, EXCEPT AS NOTED.
DRIP DETAIL AT PIERS ON BOTTOM FLANGE SHALL BE INSTALLED ON THE UP GRADE SIDE OF THE PIER AS SHOWN IN THE FIELD.
BLAST CLEAN STEEL AFTER ASSEMBLY. ACCELERATE WEATHERING PATINA FORMATION. SEE SPECIAL PROVISIONS.
2 4/16/14 REVISED BEAM END EXTENSION AT ABUTMENTS. J.P.E.
1 3/10/14 REVISED CIRCLE NOTE 1, ADDED ABUTMENT LABELS. J.P.E.
REV. NO. DATE REVISION DESCRIPTION APPROVED
CERTIFIED BY Joseph P. Engineer 12/12/2012
LICENSED PROFESSIONAL ENGINEER DATE
NAME: JOSEPH P. ENGINEER LIC. NO. 99999
TITLE: NEW FASCIA GIRDER
DES: N.M.H. DR: J.H.B. APPROVED:
CHK: J.P.E. CHK: N.M.H.
SHEET NO. 7-R2 OF 26 SHEETS
BRIDGE NO. 27974

Figure 2.4.3.1

2.5 Reconstruction Guidelines and Details

Typical details for the reconstruction of barriers, superstructure joints, and pavement joints are presented in this section.

2.5.1 Superstructure

[Future manual content]

2.5.1.1 Barriers

[Future manual content]

2.5.1.2 Wearing Course

[Future manual content]

2.5.1.3 Expansion/Fixed Joints

There are many different existing joint configurations in service. The more prevalent configurations have been given "type" designations for pay item purposes. In order to promote consistency among repair plan details, guidance is provided as follows for each "type":

Reconstruct Expansion Joint Type A
Use when replacing an in-place waterproof joint with a new waterproof joint at a parapet type abutment where the joint is located at the front of the parapet. See Figure 2.5.1.3.1. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE A", LIN. FT.

Reconstruct Expansion Joint Type B
Use when replacing an in-place waterproof joint with a new waterproof joint at a parapet type abutment where the joint is located at the back of the parapet. See Figure 2.5.1.3.2. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE B", LIN. FT.

Reconstruct Expansion Joint Type C
This type is no longer used.

Reconstruct Expansion Joint Type D
Use when replacing an in-place waterproof joint with a new waterproof joint at a pier. See Figure 2.5.1.3.3 for a concrete superstructure and Figure 2.5.1.3.4 for a steel superstructure. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE D", LIN. FT.

Reconstruct Expansion Joint Type E
Use when replacing an in-place waterproof joint with a new waterproof joint at a hinge. See Figure 2.5.1.3.5. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE E", LIN. FT.

Reconstruct Expansion Joint Type F
Use when replacing an in-place finger joint or modular joint with a new finger joint and waterproof trough. See Figure 2.5.1.3.6. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE F", LIN. FT.

Reconstruct Expansion Joint Type Davidson
Use when replacing an in-place waterproof joint located at the front of a parapet abutment with a new waterproof joint that includes an abutment end block which is integral with the approach panel.

See Figure 2.5.1.3.7 for the details. Depending upon the geometry of the end block and the condition of the in-place reinforcement, there may be permutations to the details and notes to include in the repair plan:

- If the abutment back face bars that extend into the end block cannot be salvaged, or if the in-place bars do not consist of at least #4 bars with a maximum spacing of 1'-0", addition of bars with adhesive anchors is required. Use #5 epoxy coated bars at a maximum spacing of 1'-0" for the added reinforcement.
- If the end block thickness is < 9", leave a 4" hook on the in-place back face bars. Alternatively, provide new #5 epoxy coated bars with a 4" hook that have a maximum spacing of 1'-0" and are attached to the abutment parapet wall with adhesive anchors.
- If the end block overhangs the main body of the abutment by more than 6" (not including the edge of deck regions), provide additional bars or support, or confirm that in-place reinforcement is adequate by using strut-and-tie analysis method.

The designer must evaluate the specific situation in conjunction with the Regional Bridge Construction Engineer and provide the appropriate notes and details.

Always include an adhesive anchor pay item in the bridge plan. If the expectation is that no anchors will be needed, include a pay quantity equal to 10% of the existing abutment backwall back face bars.

Also note that the bridge designer must contact the road designer to coordinate the approach panel length and limits.

For payment of all work excluding the adhesive anchors, use item no.:
2433.603 "RECONSTRUCT EXP JOINT TYPE DAVIDSON", LIN. FT.
For payment of the adhesive anchor work, use item no.:
2433.603 "ANCH TYPE REINF BARS (TYPE NT)", EACH

Reconstruct Expansion Joint Type Modular
Use when replacing an in-place finger joint or modular joint with a new modular joint. See Figure 2.5.1.3.8. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE MODULAR", LIN. FT.

Reconstruct Expansion Joint Type Special
Use for joint reconstructions that do not match any of the types given above. For payment, use item no.:
2433.603 "RECONSTRUCT EXPANSION JOINT TYPE SPECIAL", LIN. FT.

Reconstruct Fixed Joint
Use when replacing an in-place waterproof joint at a contraction abutment with a new fixed joint, where the in-place approach panel is also being replaced. For this joint type, the approach panel is removed and replaced in the grading portion of the contract. The joint reconstruction work only includes the work on the bridge side of the joint. See Figure 2.5.1.3.9. For payment of all work excluding the adhesive anchors, use item no.:
2433.603 "RECONSTRUCT FIXED JOINT", LIN. FT.
For payment of the adhesive anchor work, use item no.:
2433.603 "ANCH TYPE REINF BARS (TYPE NT)", EACH

Reconstruct Fixed Joint Type A
Use when replacing an in-place waterproof joint at a contraction abutment with a new fixed joint, where the in-place approach panel is not being replaced. For this joint type, the approach panel is partially removed and replaced in the bridge portion of the contract. The joint reconstruction work includes the work done on both the bridge side and approach panel side of the joint. See Figure 2.5.1.3.10. For payment of all work excluding the adhesive anchors, use item no.:
2433.603 "RECONSTRUCT FIXED JOINT TYPE A", LIN. FT.
For payment of the adhesive anchor work, use item no.:
2433.603 "ANCH TYPE REINF BARS (TYPE NT)", EACH

Reconstruct Fixed Joint Type B

Use when replacing an in-place joint at a pier with a new fixed joint. See Figure 2.5.1.3.11. For payment, use item no.:

2433.603 "RECONSTRUCT FIXED JOINT TYPE B", LIN. FT.

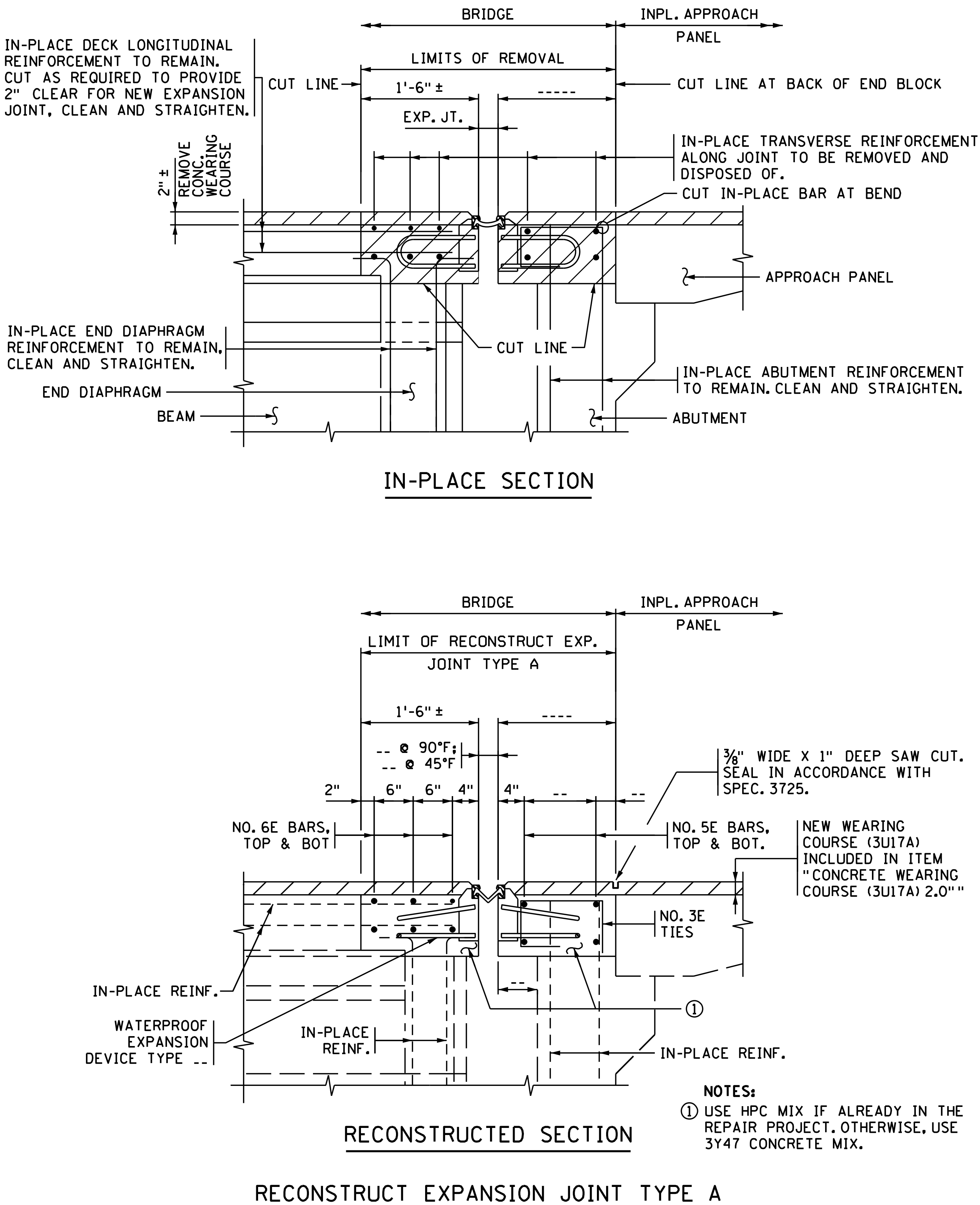
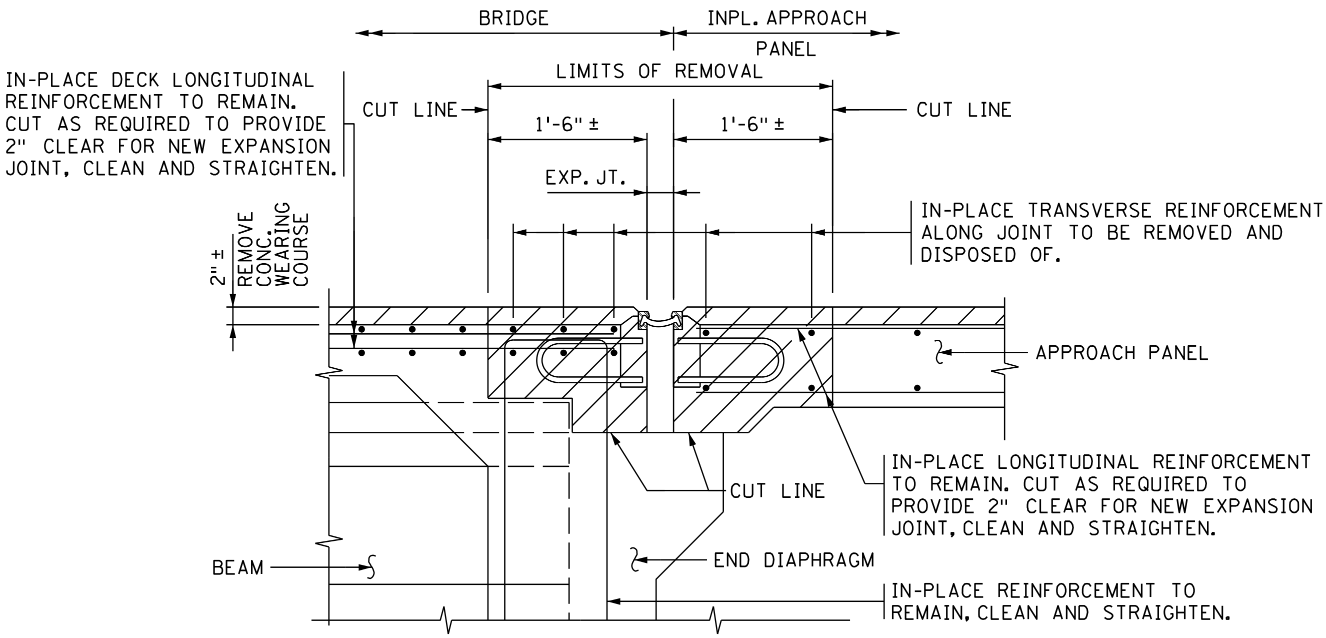
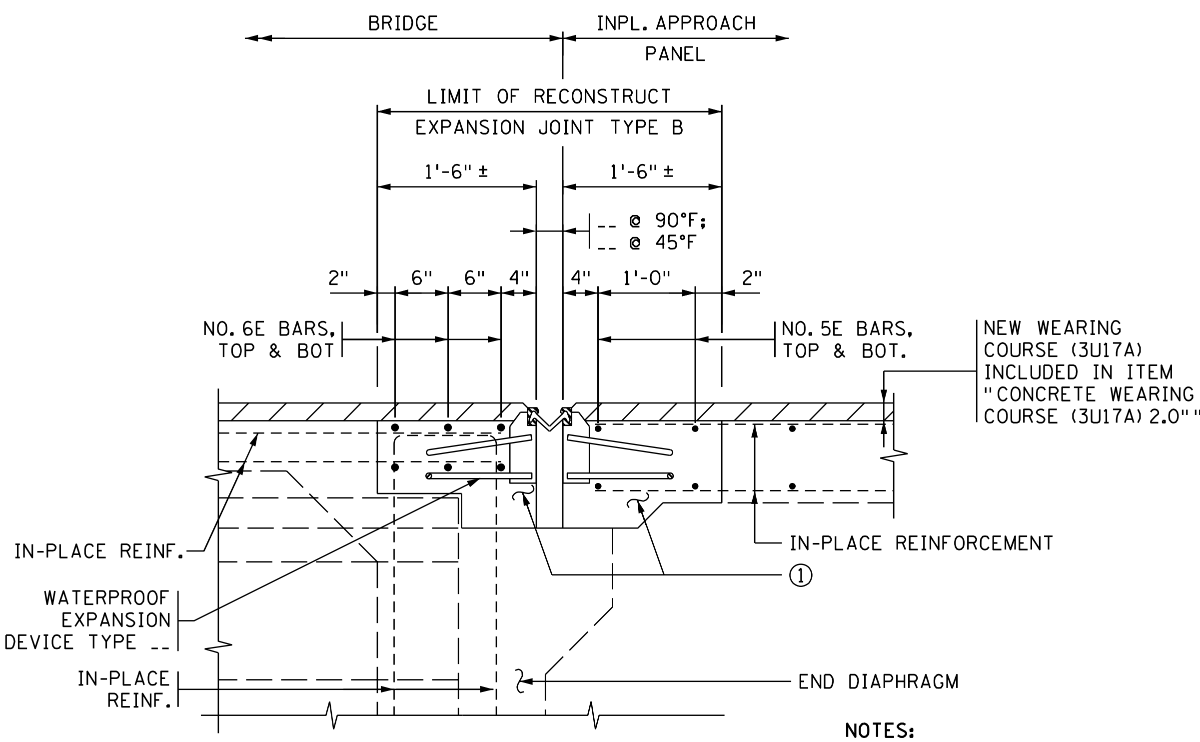


Figure 2.5.1.3.1
Expansion Joints



IN-PLACE SECTION



RECONSTRUCTED SECTION

NOTES:
1. USE HPC MIX IF ALREADY IN THE REPAIR PROJECT. OTHERWISE, USE 3Y47 CONCRETE MIX.

RECONSTRUCT EXPANSION JOINT TYPE B

Figure 2.5.1.3.2
Expansion Joints

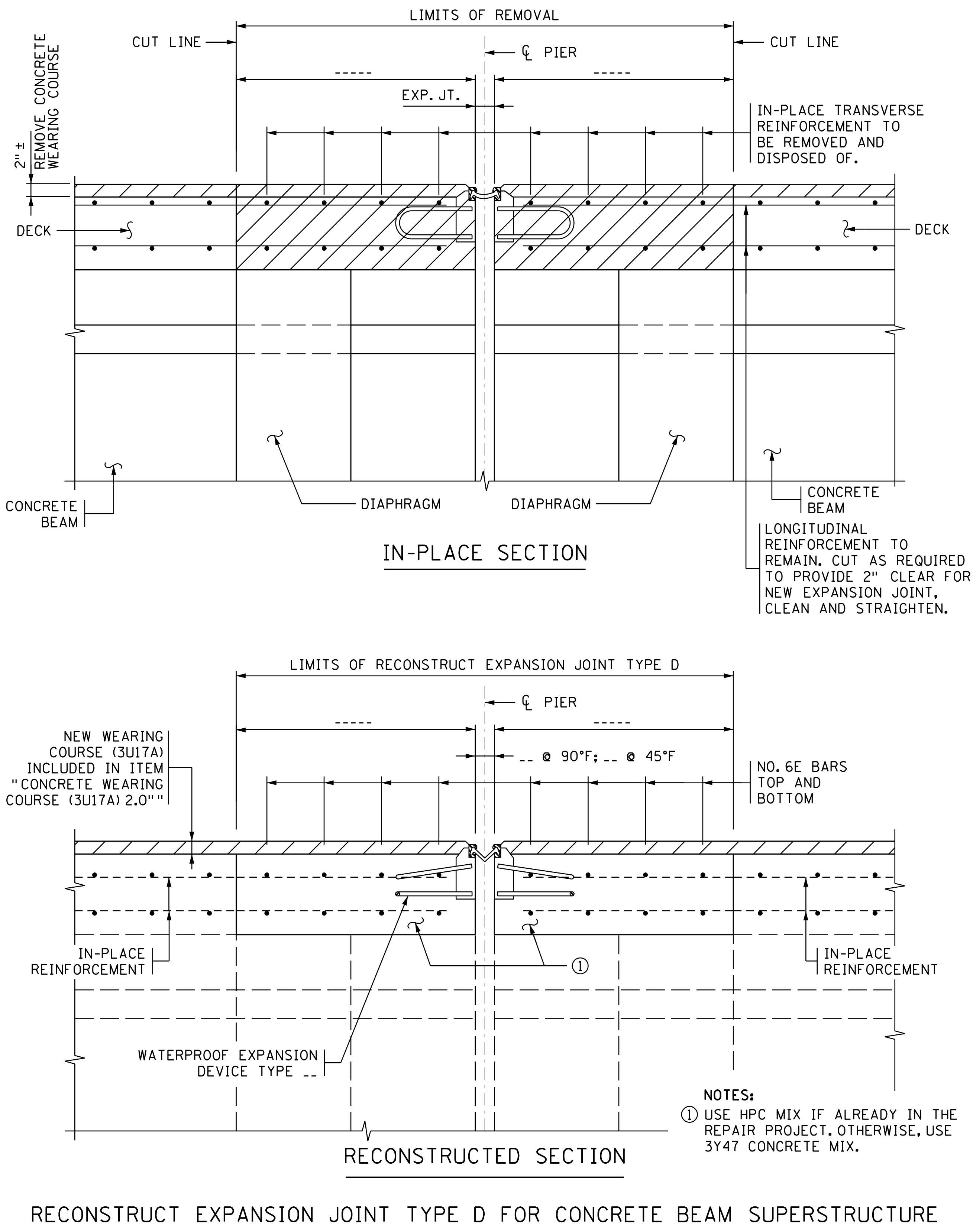


Figure 2.5.1.3.3
Expansion Joints

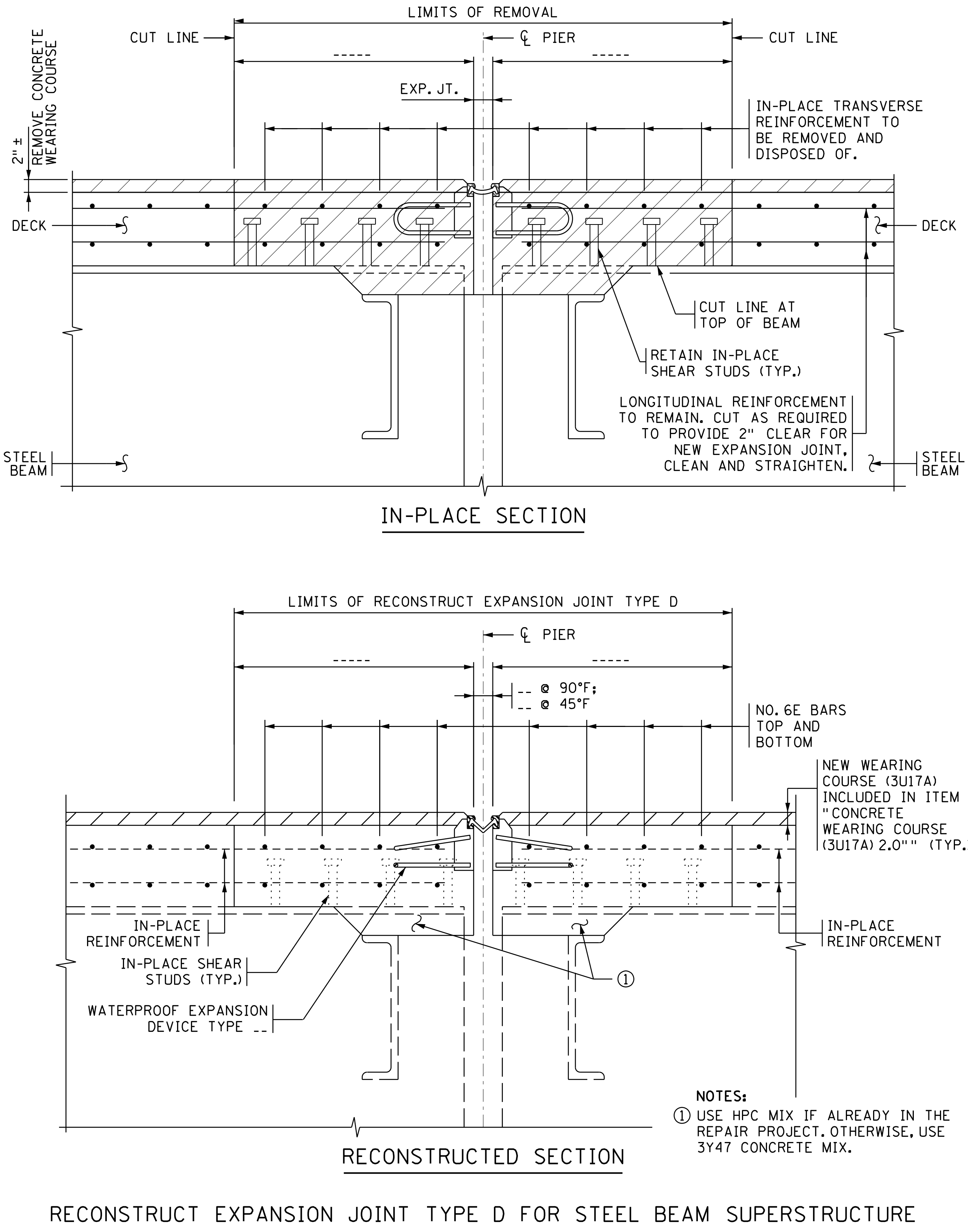
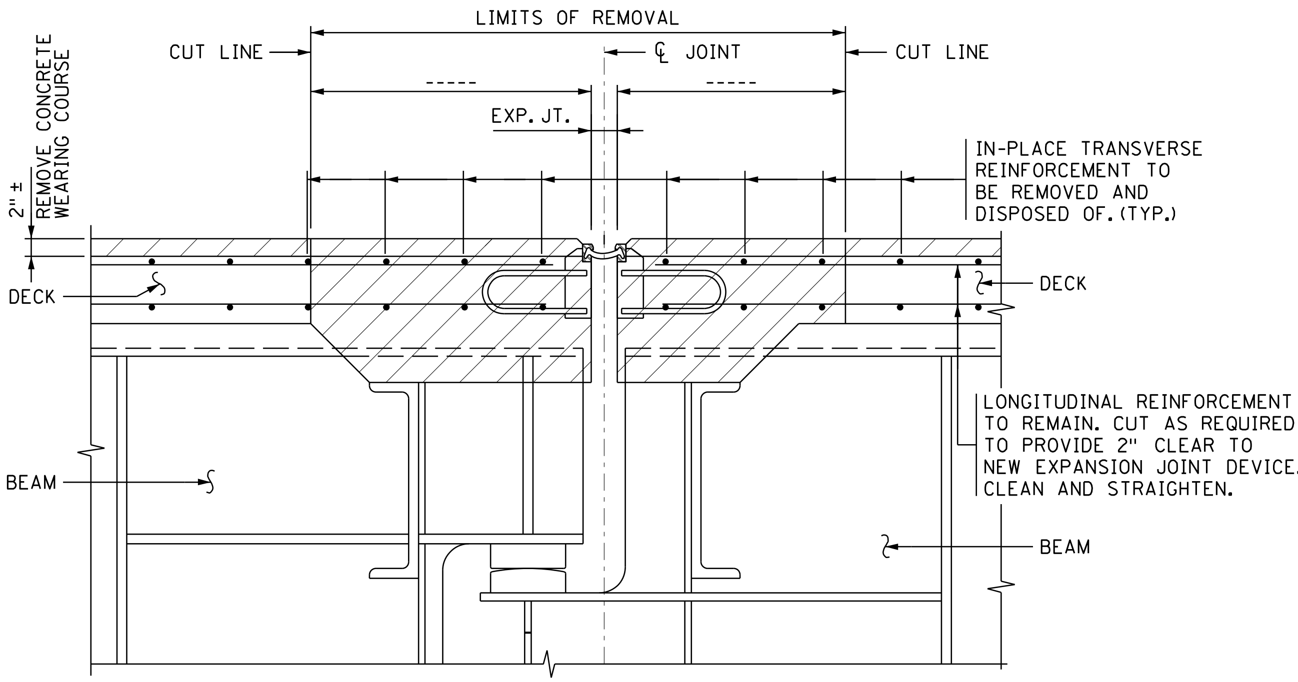
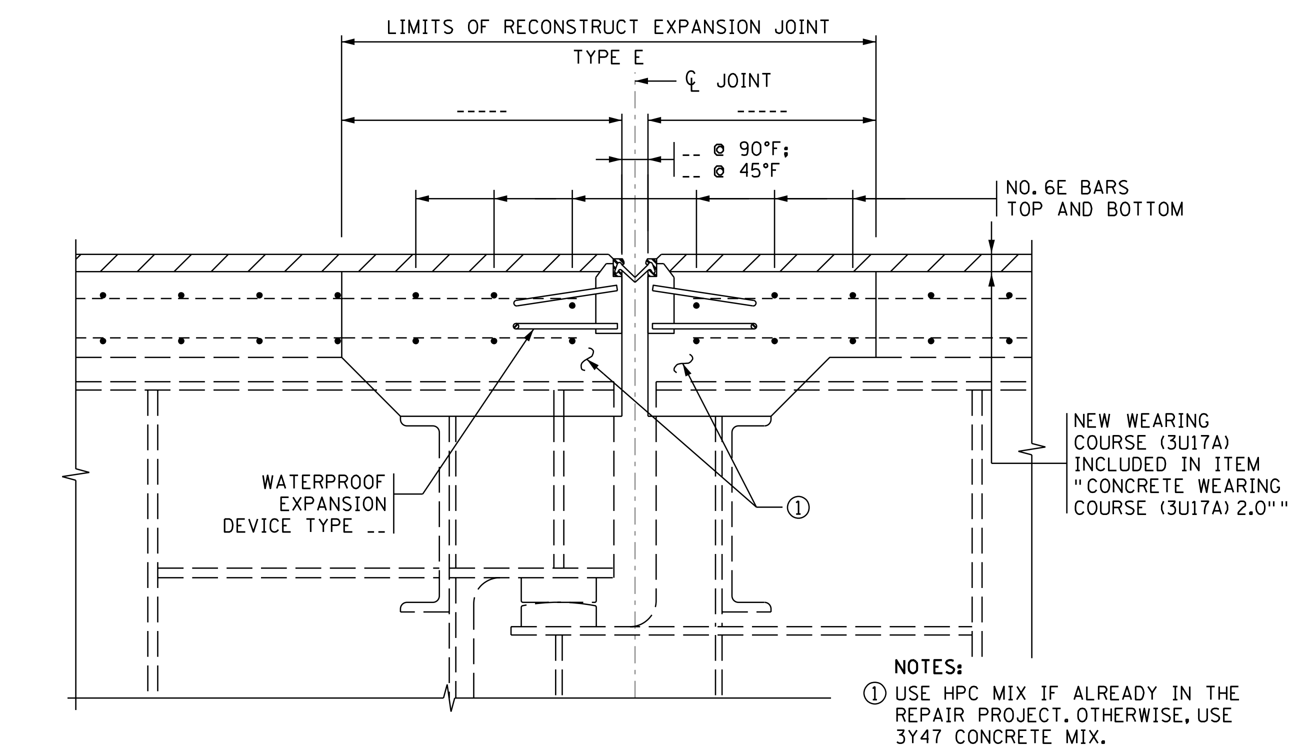


Figure 2.5.1.3.4
Expansion Joints



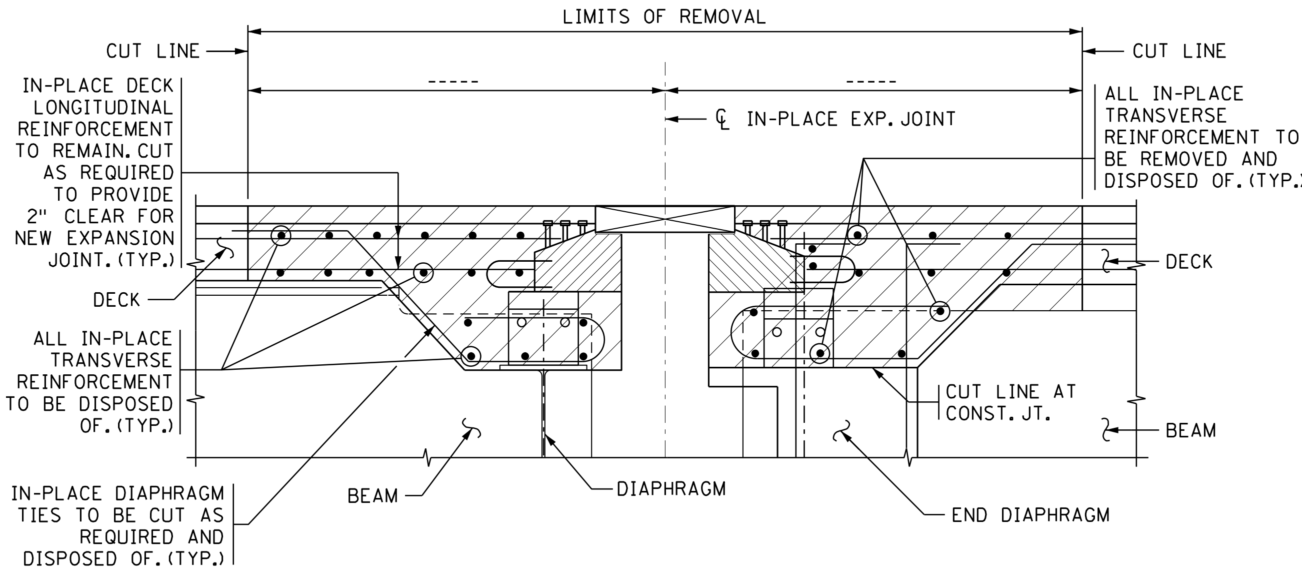
IN-PLACE SECTION



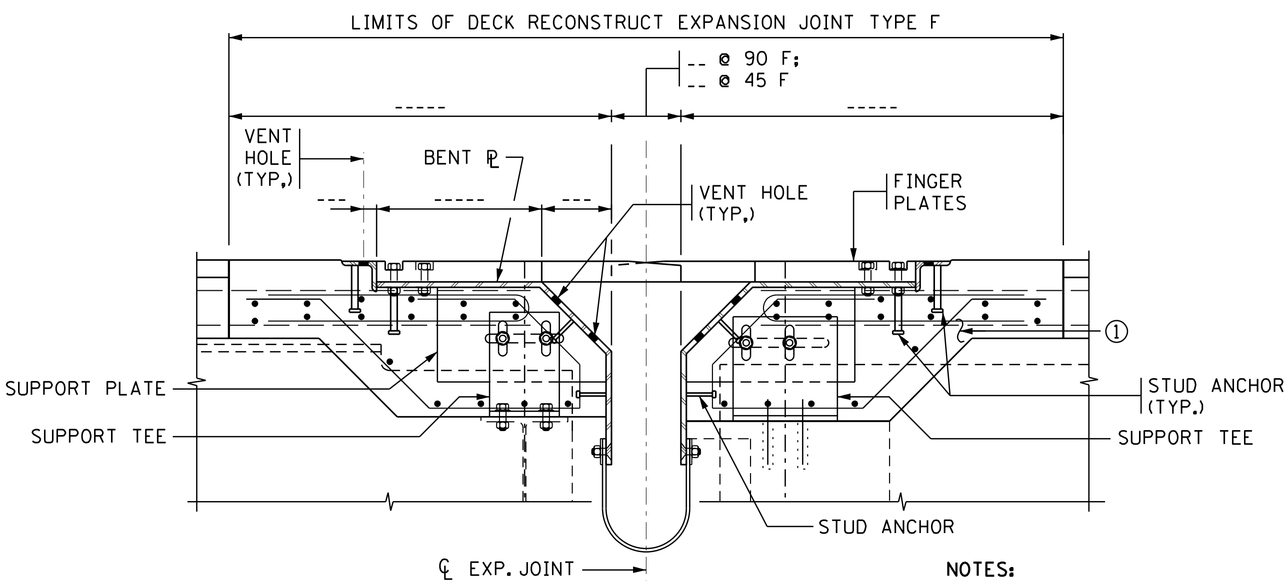
RECONSTRUCTED SECTION

RECONSTRUCT EXPANSION JOINT TYPE E

Figure 2.5.1.3.5
Expansion Joints



IN-PLACE SECTION



NOTES:
1 USE HPC MIX IF ALREADY IN THE REPAIR PROJECT. OTHERWISE, USE 3Y47 CONCRETE MIX.

RECONSTRUCTED SECTION

RECONSTRUCT EXPANSION JOINT TYPE F

Figure 2.5.1.3.6
Expansion Joints

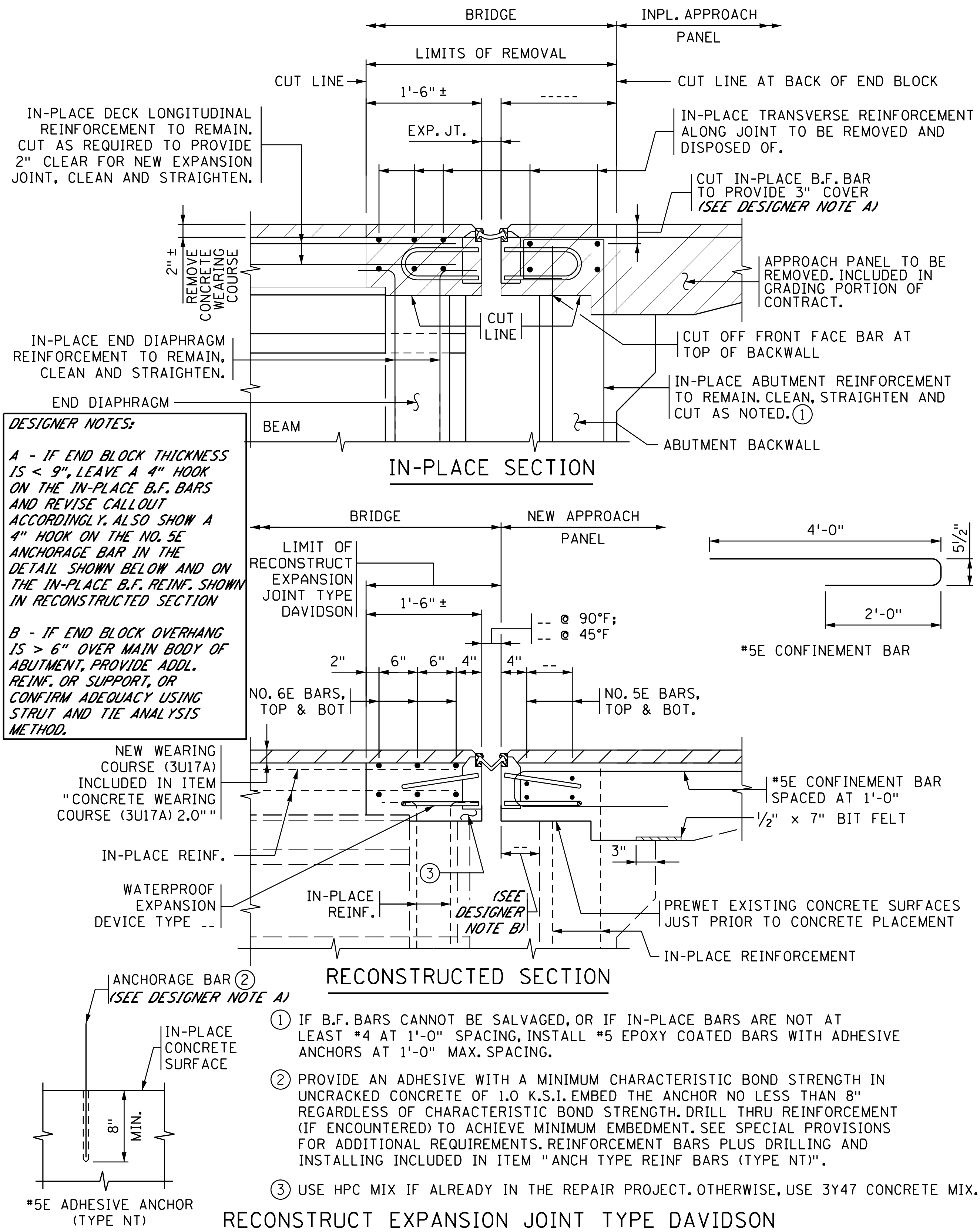
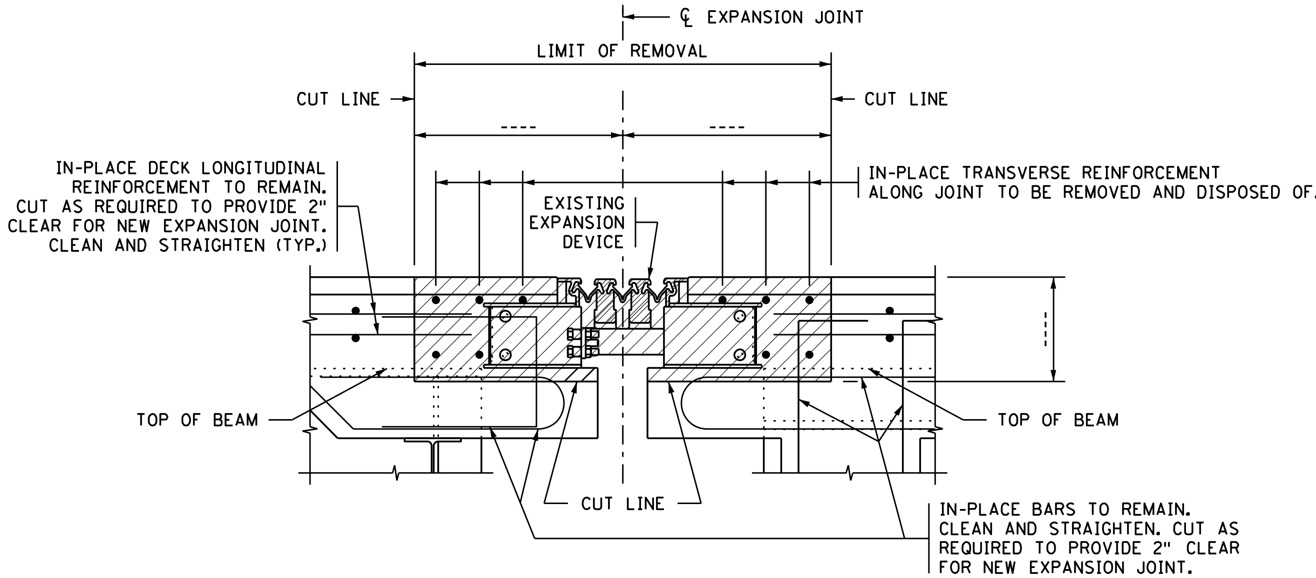
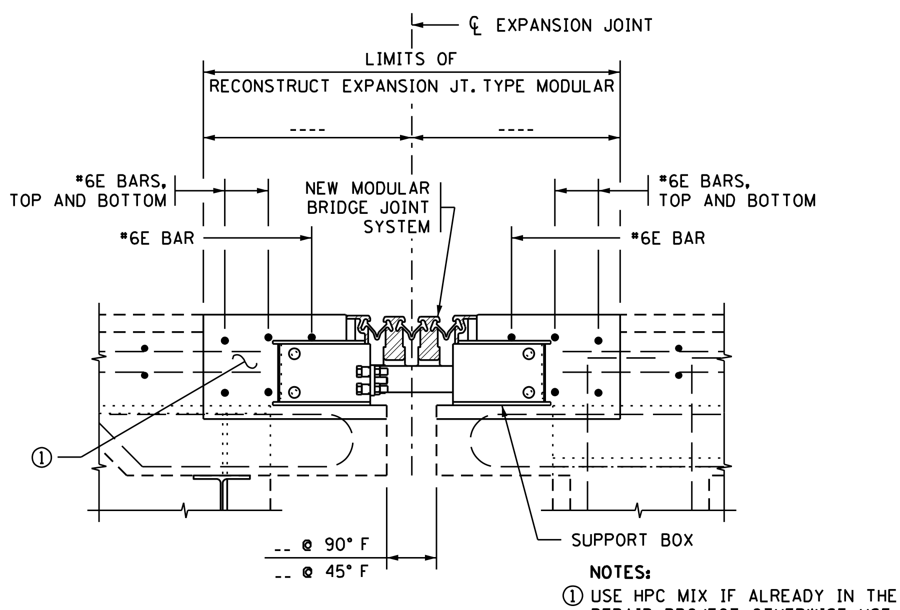


Figure 2.5.1.3.7
Expansion Joints



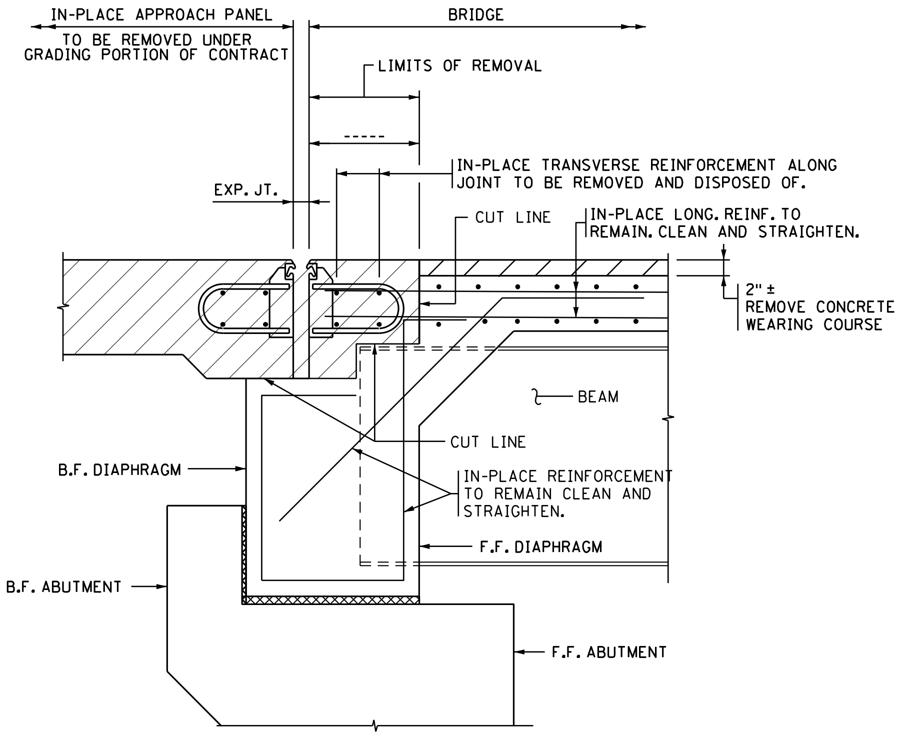
IN-PLACE SECTION



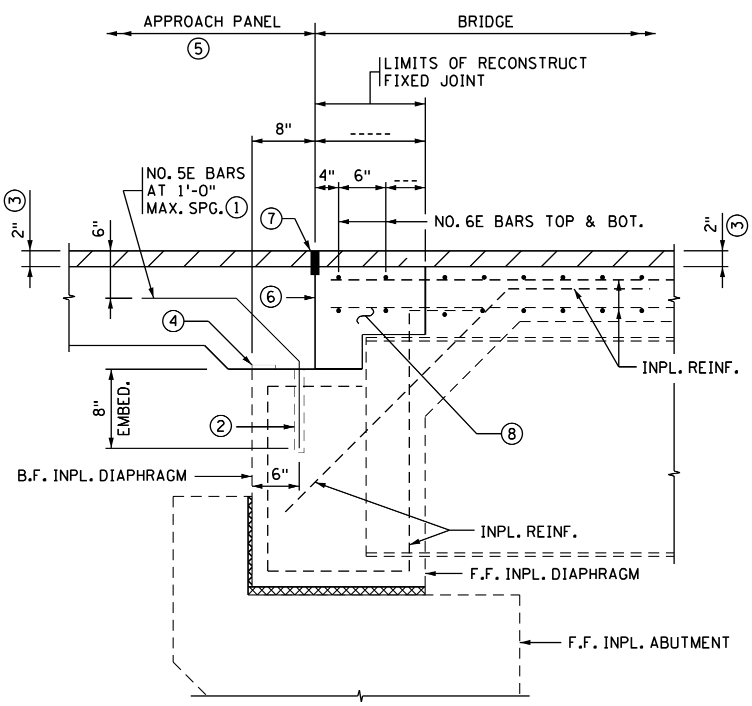
RECONSTRUCTED SECTION

RECONSTRUCT EXPANSION JOINT TYPE MODULAR

Figure 2.5.1.3.8
Expansion Joints



IN-PLACE



RECONSTRUCTED SECTION

NOTES:

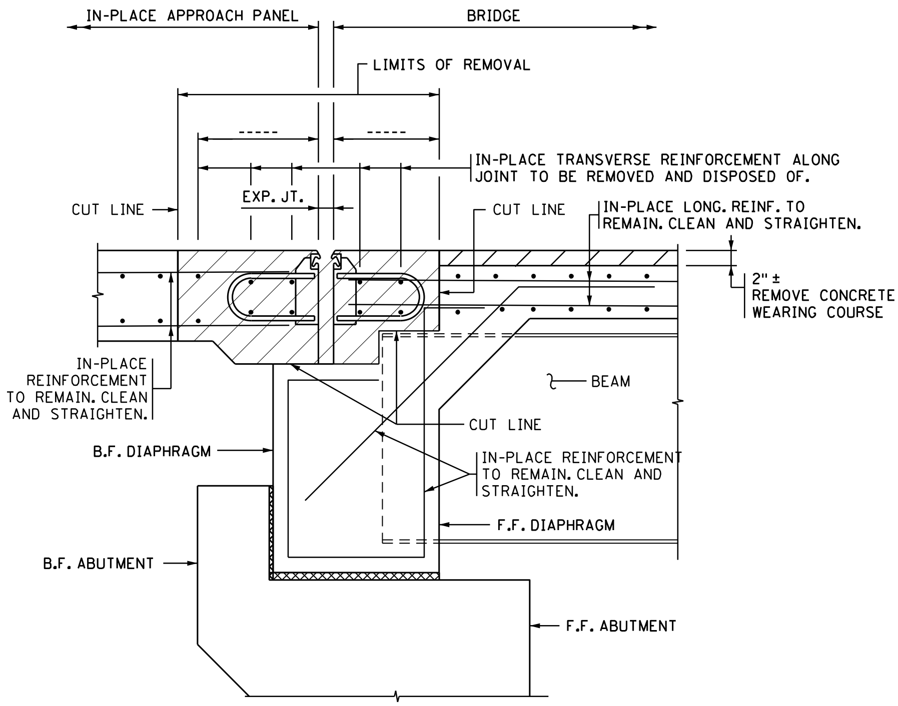
F.F. DENOTES FRONT FACE.

B.F. DENOTES BACK FACE.

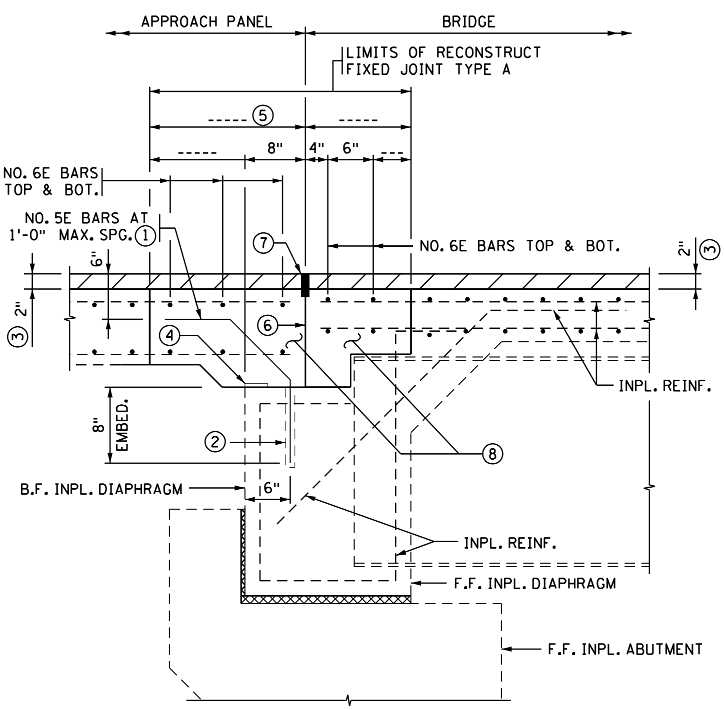
1. TIE NEW APPROACH PANEL TO CONCRETE END DIAPHRAGM. REINFORCEMENT BARS PLUS DRILLING AND INSTALLING INCLUDED IN ITEM "ANCH TYPE REINF BARS (TYPE NT)".
2. PROVIDE AN ADHESIVE WITH A MINIMUM CHARACTERISTIC BOND STRENGTH IN UNCRACKED CONCRETE OF 1.0 K.S.I. EMBED THE ANCHOR NO LESS THAN 8" REGARDLESS OF CHARACTERISTIC BOND STRENGTH. DRILL THROUGH REINFORCEMENT (IF ENCOUNTERED) TO ACHIEVE MINIMUM EMBEDMENT. SEE SPECIAL PROVISIONS FOR ADDITIONAL REQUIREMENTS.
3. NEW CONCRETE WEARING COURSE (3U17A). INCLUDED IN ITEM "CONCRETE WEARING COURSE (3U17A) 2.0""
4. ½"X3" BIT FELT. INCLUDED IN ITEM "RECONSTRUCT FIXED JOINT".
5. APPROACH PANEL TO BE CONSTRUCTED UNDER GRADING PORTION OF CONTRACT. DELAY PLACEMENT OF APPROACH PANEL MINIMUM OF 24 HOURS AFTER PLACING CONCRETE FOR "RECONSTRUCT FIXED JOINT".
6. DO NOT APPLY BONDING AGENT AT THIS CONSTRUCTION JOINT.
7. C2H CONTRACTION JOINT SEE GRADING PLANS.
8. USE HPC MIX IF ALREADY IN THE REPAIR PROJECT. OTHERWISE, USE 3Y47 CONCRETE MIX.

RECONSTRUCT FIXED JOINT

Figure 2.5.1.3.9
Fixed Joints



IN-PLACE



RECONSTRUCTED SECTION

NOTES:

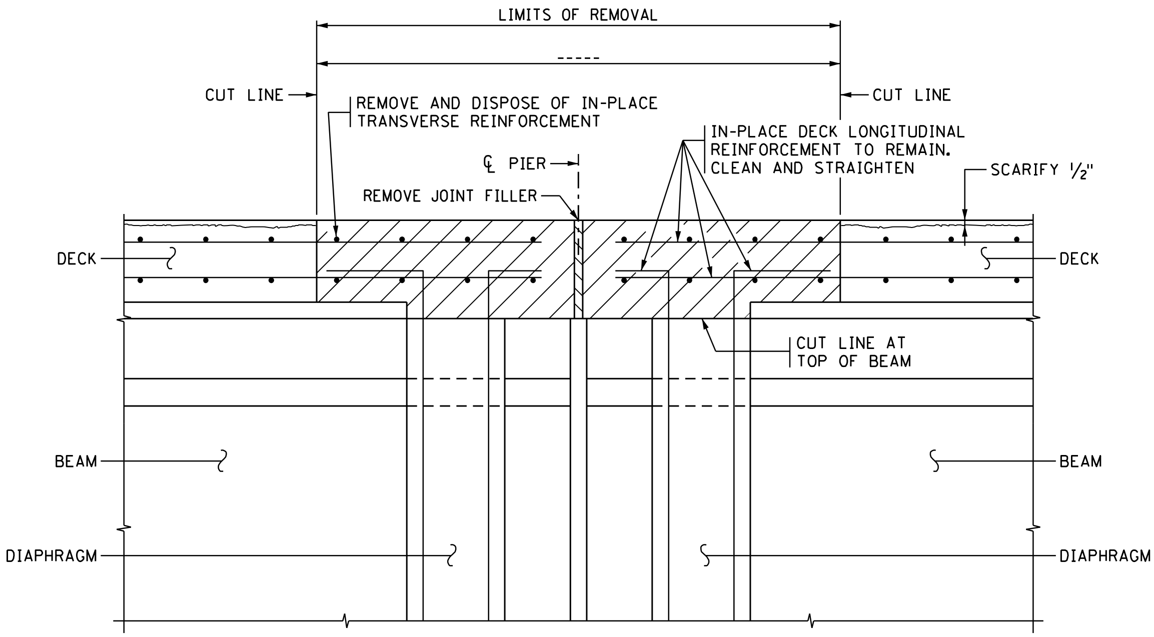
F.F. DENOTES FRONT FACE.

B.F. DENOTES BACK FACE.

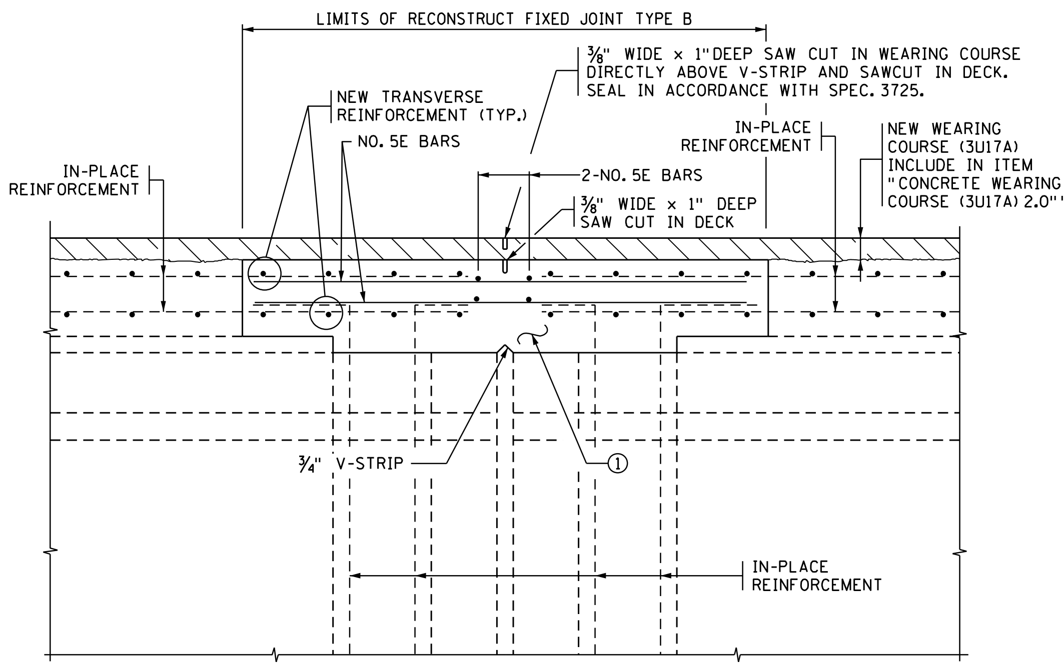
1. TIE APPROACH PANEL TO CONCRETE END DIAPHRAGM. REINFORCEMENT BARS PLUS DRILLING AND INSTALLING INCLUDED IN ITEM "ANCH TYPE REINF BARS (TYPE NT)".
2. PROVIDE AN ADHESIVE WITH A MINIMUM CHARACTERISTIC BOND STRENGTH IN UNCRACKED CONCRETE OF 1.0 K.S.I. EMBED THE ANCHOR NO LESS THAN 8" REGARDLESS OF CHARACTERISTIC BOND STRENGTH. DRILL THROUGH REINFORCEMENT (IF ENCOUNTERED) TO ACHIEVE MINIMUM EMBEDMENT. SEE SPECIAL PROVISIONS FOR ADDITIONAL REQUIREMENTS.
3. NEW CONCRETE WEARING COURSE (3U17A). INCLUDED IN ITEM "CONCRETE WEARING COURSE (3U17A) 2.0""
4. ½" X3" BIT FELT. INCLUDED IN ITEM "RECONSTRUCT FIXED JOINT".
5. DELAY PLACEMENT OF CONCRETE ON APPROACH PANEL SIDE OF FIXED JOINT A MINIMUM OF 24 HOURS AFTER PLACEMENT OF CONCRETE ON BRIDGE SIDE OF FIXED JOINT.
6. DO NOT APPLY BONDING AGENT AT THIS CONSTRUCTION JOINT.
7. C2H CONTRACTION JOINT SEE GRADING PLANS.
8. USE HPC MIX IF ALREADY IN THE REPAIR PROJECT. OTHERWISE, USE 3Y47 CONCRETE MIX.

RECONSTRUCT FIXED JOINT TYPE A

Figure 2.5.1.3.10
Fixed Joints



IN-PLACE SECTION



NOTES:
1. USE HPC MIX IF ALREADY IN THE REPAIR PROJECT. OTHERWISE, USE 3Y47 CONCRETE MIX.

RECONSTRUCTED SECTION

RECONSTRUCT FIXED JOINT TYPE B

Figure 2.5.1.3.11
Fixed Joints

2.5.2 Substructure

[Future manual content]

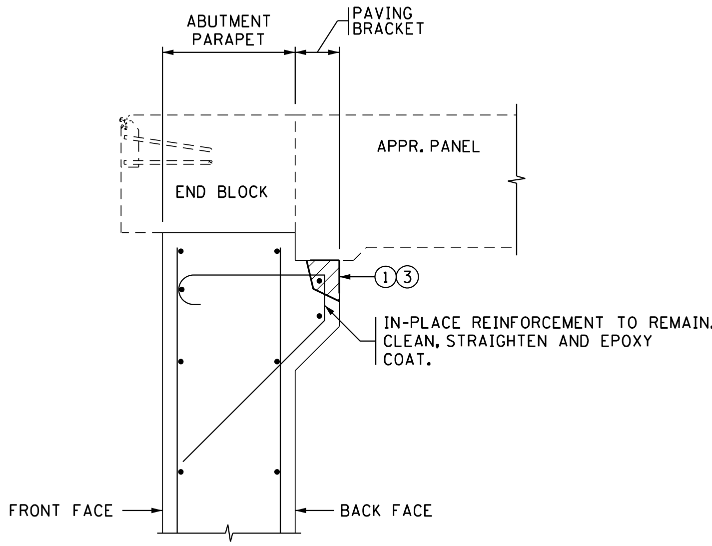
2.5.2.1 Abutments

The following figures show typical details for the repair of abutment paving brackets:

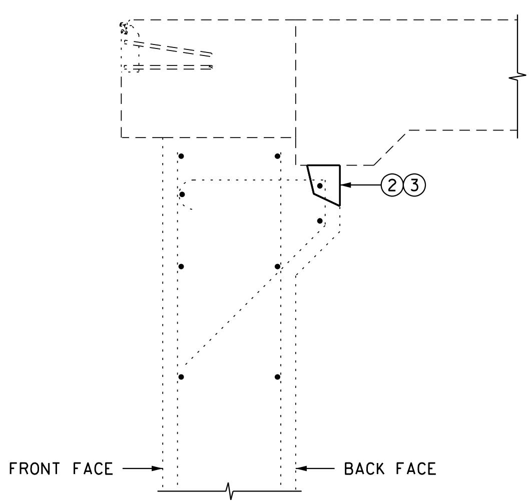
Figure 2.5.2.1.1 Repair Paving Bracket
Figure 2.5.2.1.2 Reconstruct Paving Bracket
Figure 2.5.2.1.3 Reconstruct Paving Bracket and Wall

Paving Bracket Repair Pay Items

- Item No. 2433.603 "Repair Paving Bracket", Lin. Ft.
- Item No. 2433.603 "Reconstruct Paving Bracket", Lin. Ft.
- Item No. 2433.618 "Reconstruct Paving Bracket and Wall", Sq. Ft.



IN-PLACE SECTION REMOVAL



REPAIRED SECTION

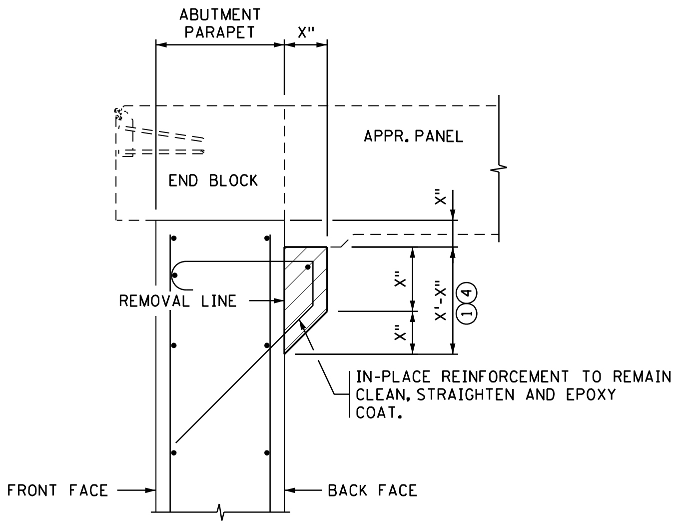
① REMOVE ANY DETERIORATED AREAS OF PAVING BRACKET TO SOUND CONCRETE IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

② REPLACE REMOVED CONCRETE WITH CONCRETE MIX NO. 3B52. COAT SURFACE OF EXISTING CONCRETE WITH BONDING GROUT PER SPECIAL PROVISIONS PRIOR TO CASTING.

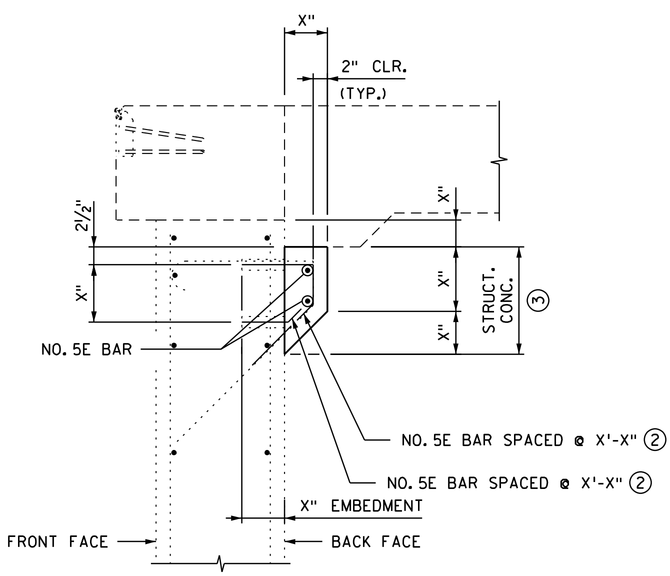
③ INCLUDED IN ITEM "REPAIR PAVING BRACKET".

REPAIR PAVING BRACKET

Figure 2.5.2.1.1
Paving Brackets



IN-PLACE SECTION REMOVAL

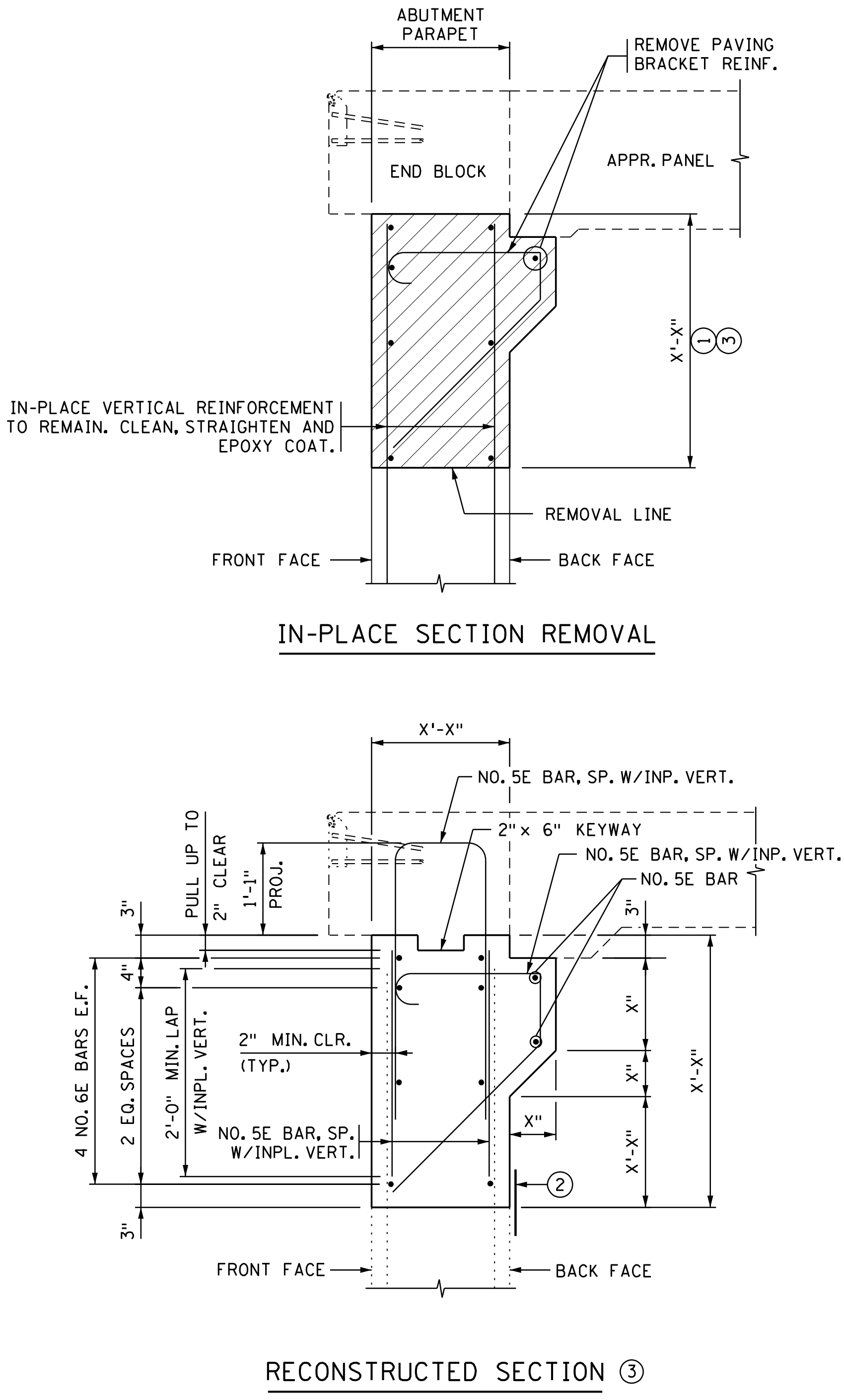


RECONSTRUCTED SECTION 4

1. REMOVE PAVING BRACKET IN ITS ENTIRETY TO SOUND CONCRETE IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
2. SUPPLEMENT IN-PLACE REINFORCEMENT AS NEEDED WITH NEW BARS AND ADHESIVE ANCHORS. PROVIDE AN ADHESIVE ANCHOR WITH A MINIMUM CHARACTERISTIC BOND STRENGTH IN UNCRACKED CONCRETE OF 1.0 K.S.I. EMBED THE ANCHORAGE NO LESS THAN 6" INTO SOLID CONCRETE. INCLUDED IN ITEM "ANCH TYPE REINF BARS (TYPE L)".
3. REPLACE CONCRETE WITH CONCRETE MIX NO. 3B52. COAT SURFACE OF EXISTING CONCRETE WITH BONDING GROUT PER SPECIAL PROVISIONS PRIOR TO CASTING.
4. INCLUDED IN ITEM "RECONSTRUCT PAVING BRACKET" EXCEPT AS NOTED.

RECONSTRUCT PAVING BRACKET

Figure 2.5.2.1.2
Paving Brackets



1. REMOVE PARAPET WALL AND PAVING BRACKET IN ITS ENTIRETY TO SOUND CONCRETE IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
2. MEMBRANE WATERPROOFING SYSTEM.
3. INCLUDED IN ITEM "RECONSTRUCT PAVING BRACKET AND WALL".

RECONSTRUCT PAVING BRACKET AND WALL

Figure 2.5.2.1.3
Paving Brackets

2.5.2.2 Piers

[Future manual content]

2.6 Construction Requirements

MnDOT's general practices and guidelines for the construction of bridges are presented in MnDOT's *Bridge Construction Manual*, which can be accessed from the MnDOT Bridge Office web site.

Provide the required submittals and the qualifications of the individuals responsible for the preparation of falsework and other submittals in the contract documents for the project.

Falsework and forms are to be designed in accordance with the AASHTO *Guide Design Specifications for Bridge Temporary Works*. Falsework submittals must meet the requirements of Bridge Special Provision No. SB2018-2401.2.

Submittals describing proposed temporary shoring for works adjacent to railroad tracks require approval by the railroad.

Provide details of temporary shoring in the plans with consideration of the domestic availability of the sheeting materials. Frequently, showing the location of the sheeting and the minimum required section modulus is sufficient. However, designers should satisfy themselves that adequate clearances have been provided for at least one reasonable shoring scheme for staged construction projects. If more complex details are required, they must be provided in the plans. See Article 11.3.7 of this manual for more guidance.

APPENDIX 2-A
BRIDGE TYPE NUMBERS

MINNESOTA BRIDGE TYPE IDENTIFICATION NUMBER (3 characters)

First Digit (Superstructure Material)

1	Cast-In-Place Concrete
2	Cast-In-Place Concrete Continuous
3	Steel
4	Steel Continuous
5	Prestressed Pretensioned or Precast Concrete
6	Prestressed Pretensioned Continuous Concrete
7	Timber
8	Masonry
9	Wrought or Cast Iron
O	Other
A	Aluminum
P	Prestressed Post-Tensioned

Second & Third Digits (Bridge Type)

01	Beam Span
02	Low Truss
03	High Truss
04	Deck Truss
05	Thru Girder
06	Deck Girder
07	Box Girder
08	Rigid Frame
09	Slab Span
10	Slab Span-Voided
11	Channel Span
12	Arch
13	Box Culvert
14	Pipe Culvert (Round)
15	Pipe Arch
16	Long Span
17	Tunnel
18	Movable
19	Other
20	Double Tee
21	Quad Tee
22	Bulb Tee
23	Suspension
24	Tied Arch
25	Cable Stay
26	Inverted T

EXAMPLES

BRIDGE TYPE	**ID NUMBER**
Precast Concrete Box Culvert	513
Simple Span Cast-In-Place Concrete Slab	109
Tunnel in Rock	017
Pretensioned Concrete Beam Span	501 approach span
Steel Continuous Beam Span	401 main span
Post-Tensioned Concrete Box Girder	P07

Note: A bridge may have different identification numbers for each span. Identify all span types accordingly.

APPENDIX 2-B
STANDARD ABBREVIATIONS

A

AASHTO .... American Association of State Highway and Transportation Officials
ABT. .... About
ABUT. .... Abutment
AADT .... Annual Average Daily Traffic
ADTT .... Average Daily Truck Traffic
ALT. .... Alternate
APPR. .... Approach
APPROX. .... Approximate (or Approximately)
ASSY. .... Assembly
AZ. .... Azimuth
@ .... At

B

B.F. .... Back Face
BIT. .... Bituminous
B.M. .... Bench Mark
BM .... Beam
BOT. .... Bottom
BR. .... Bridge
BRG. .... Bearing
BTWN. .... Between

C

C & G........ Curb and Gutter
C-I-P .... Cast-In-Place
C_L .... Centerline
CL. (or CLR.)........ Clear
C.M.P. .... Corrugated Metal Pipe
COL. .... Column
COMP. .... Composite
CONC. .... Concrete
CONST. .... Construction
CONT. .... Continuous (or Continued)
C.S.A.H. .... County State Aid Highway
CU. .... Cubic
CULV. .... Culvert

D

D.C. .... Degree of Curve
DET. .... Detail
D.H.V. .... Design Hourly Volume
D.H.W. .... Design High Water
DIA. .... Diameter
DIAPH. .... Diaphragm
DL .... Dead Load
DWL. .... Dowel

E

E. .... East
E.B.L. .... East Bound Lane(s)
E.F. .... Each Face
EA. .... Each
ELEV. (or EL.) .... Elevation
EMBED. .... Embedment
ENGR. .... Engineer
EQ. .... Equal
EXP. .... Expansion

F

F. .... Fahrenheit
F.B.M. .... Foot Board Measure
F.F. .... Front Face
F.L. .... Flowline
FIN. .... Finished
FIX .... Fixed
FT. .... Foot (or Feet)
FTG. .... Footing

G

G1 .... Grade One
G2 .... Grade Two
GA. .... Gage

APPENDIX 2-B (Continued)
STANDARD ABBREVIATIONS

H

HCAADT .... Heavy Commercial Annual Average Daily Traffic
H.W. .... High Water
HORIZ. ....Horizontal
HWY. ....Highway

I

INPL. .... In-place
I.D. .... Inside Diameter

J

JCT. .... Junction
JT. .... Joint

K

KWY. ....Keyway

L

L. .... Length of Curve
LL .... Live Load
L.W. ....Low Water
LB. .... Pound
LIN. .... Linear
LT. .... Left
LONG. (or LONGIT.)..................Longitudinal

M

m ....Meter
mm ....Millimeter
M.B.M. .... Thousand Board Feet
M.L. .... Main Line
M.O. ....Maximum Offset
MAX. .... Maximum
MIN. ....Minimum
MISC. ....Miscellaneous

N

N. (or NO.) ....North
N.B.L. ....North Bound Lane(s)
NO. ....Number

O

O.D. ....Outside Diameter

P

P.C. .... Point of curvature
P.C.C. .... Point of compound Curve
P.G. ....Profile Grade
P.I. ....Point of Intersection
P.O.C. .... Point on Curve
P.O.T. .... Point on Tangent
P.S.I. ....Pounds per Square Inch
P.T. .... Point of Tangency
PED. .... Pedestrian
P_L ....Plate
PRESTR. ....Prestressed
PROJ. .... Project (or Projection)
PROV. .... Provision
PT. ....Point

R

R. .... Radius
R.O.W. .... Right of Way
R.R. .... Railroad
R.S.C. .... Rigid Steel Conduit
RDWY. .... Roadway
REINF. ...... Reinforced (or Reinforcing/ment)
REQ'D.. Required
REV. ....Revised
RT. .... Right

S

S. (or SO.) ....South
S.B.L. ....South Bound Lane(s)
SEC. .... Section
SDWK. .... Sidewalk
SHLDR. .... Shoulder

APPENDIX 2-B (Continued)
STANDARD ABBREVIATIONS

S (cont.)

SHT. ....Sheet
SP. (or SPS.) ....Spaces
SPA. .... Spaced
SPEC. .... Special (or Specification)
SPG. ....Spacing
SQ. ....Square
STA. ....Station
STD. ....Standard
STIFF. ....Stiffener
STL. .... Steel
STR. (or STRUC.) ....Structure
SUBGR. .... Subgrade
SUPER. ....Superelevation
SUPERST. ....Superstructure
SYM. .... Symmetrical

T

T & B .... Top and Bottom
T.H. ....Trunk Highway
T.T.C. ....Tangent to Curve
TAN. .... Tangent
TWP. ....Township
TYP. ....Typical

V

V.C. .... Vertical Curve
V.P.C. .... Vertical Point of Curvature
V.P.I. .... Vertical Point of Intersection
V.P.T. ....Vertical Point of Tangency
VAR. .... Varies
VERT. .... Vertical

W

W. ....West
W.B.L. ....West Bound Lane(s)
W.C. ....Wearing Course
W.P. .... Working Point
W.W. .... Wingwall

Y

YD. .... Yard

APPENDIX 2-C
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES

a. NEW BRIDGES OTHER THAN CONCRETE BOX CULVERTS

DESIGNED IN ACCORDANCE WITH 20__ AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS
HL-93 LIVE LOAD
DEAD LOAD INCLUDES 20 POUNDS PER SQUARE FOOT ALLOWANCE FOR FUTURE WEARING COURSE MODIFICATIONS

[Use on all roadway bridge projects. Insert current year of specifications.]

DESIGNED IN ACCORDANCE WITH 20__ AND CURRENT INTERIM AASHTO LRFD GUIDE SPECIFICATIONS FOR THE DESIGN OF PEDESTRIAN BRIDGES
PEDESTRIAN LIVE LOAD = 0.090 KSF
__*__ TRUCK VEHICULAR LIVE LOAD

[Use on all pedestrian bridge projects. Insert current year of specifications.]

[* Insert "H-5" for deck widths between parapet/curb faces ≤ 10 ft. For deck widths between parapet/curb faces > 10 ft., insert "H-10"]

MATERIAL DESIGN PROPERTIES:

REINFORCED CONCRETE:

f'_C = 4 KSI CONCRETE
f_y = 60 KSI PLAIN AND EPOXY COATED BARS
f_y = 75 KSI STAINLESS STEEL BARS
n = 7.3 FOR REINFORCEMENT BARS

[Use on all projects. Delete stainless steel bars when not included in bridge plan.]

PRETENSIONED CONCRETE:

f'_C = __ KSI CONCRETE
f_{pu} = 300 KSI LOW RELAXATION STRANDS
n = 1 FOR PRETENSIONING STRANDS
____ f_{pu} FOR INITIAL PRESTRESS

[Use on bridges with pretensioned beams. Insert f'_C used on beam detail sheet, rounded to 0.1 ksi. Insert fraction of f_{pu} used for initial prestress on beam detail sheet.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES (CONT'D)

a. NEW BRIDGES OTHER THAN CONCRETE BOX CULVERTS (CONT'D)

POST-TENSIONED CONCRETE:

f'_c = __ KSI CONCRETE

f_{pu} = 270 KSI LOW RELAXATION STRANDS

n = 1 FOR POST-TENSIONING STRANDS

0.75 f_{pu} FOR INITIAL PRESTRESS AFTER ANCHOR SET

TOP OF DECK DESIGNED FOR ZERO TENSION UNDER SERVICE LOADS

[Use as required, e.g. post-tensioned slabs or box girders. Insert concrete strength.]

STRUCTURAL STEEL:

F_y = 36 KSI STRUCTURAL STEEL SPEC. 3306 (PAINTED)

F_y = 50 KSI STRUCTURAL STEEL SPEC. 3309 (PAINTED)

F_y = 70 KSI STRUCTURAL STEEL SPEC. 3317 (HIGH PERFORMANCE) (PAINTED)

[Use as required on bridges with steel components. Include description within parentheses as needed.]

WOOD:

F_{bo} = ____ KSI PILE CAPS

F_{bo} = ____ KSI SAWN STRINGERS AND TIMBER RAILS

F_{bo} = ____ KSI GLUED LAMINATED TIMBER RAILS

F_{bo} = ____ KSI GLUED LAMINATED STRINGERS

F_{bo} = ____ KSI GLUED LAMINATED DECK PANELS

F_{bo} = ____ KSI NAIL LAMINATED DECK PANELS

F_{bo} = ____ KSI RAIL POSTS

F_{bo} = ____ KSI ALL OTHER WOOD

[Use as required on bridges with wood components. Insert reference design values.]

DESIGN SPEED:

OVER = ________ MPH

[Use on all projects. Insert speed.]

UNDER = ________ MPH

[Use as required. Insert speed.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES (CONT'D)

a. NEW BRIDGES OTHER THAN CONCRETE BOX CULVERTS (CONT'D)

DECK AREA = ___________ SQUARE FEET

[Use on all projects. Insert area based on dimensions from coping to coping and begin bridge to end bridge.]

20__ PROJECTED TRAFFIC VOLUMES

ROADWAY OVER		ROADWAY UNDER
_______	AADT	________
_______	DHV	________
_______	HCAADT	________

[Use when provided on the Preliminary Plan. Insert appropriate values in vehicles per day. Note that HCAADT is Heavy Commercial Annual Average Daily Traffic, which can be considered equivalent to the Average Daily Truck Traffic (ADTT) used by the AASHTO LRFD Bridge Design Specifications.]

HL-93 LRFR
BRIDGE OPERATING RATING FACTOR RF = _______

[Use on all projects. Insert rating factor.]

b. PRECAST CONCRETE BOX CULVERTS

DESIGNED IN ACCORDANCE WITH ____ AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

[For standard precast concrete box culvert designs, insert date of specifications that standards are based on, which can be found at the top of Bridge Standard Plans Figure 5-395.100(A) Precast Concrete Box Culvert - Basis of Design. For non-standard designs, insert current year of specifications.]

HL-93 LIVE LOAD
BARREL SPAN = _______
BARREL RISE = _______
BARREL LENGTH = ________
EST. MIN. FILL DEPTH A = _______
EST. MAX. FILL DEPTH B = _______
SKEW ANGLE = ___° ___′ ___″

[Use on all precast concrete box culvert projects. Insert appropriate values. For estimated min. and max. fill depths, also show A and B in the box culvert elevation view to depict the min. and max. fill heights at the outside edge of the roadway shoulder.]

APPENDIX 2-C (Continued)

STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES (CONT'D)

b. PRECAST CONCRETE BOX CULVERTS (CONT'D)

DESIGN SPEED = ___ MPH

CURRENT AADT (YEAR) = _____(20__)

PROJECTED AADT (YEAR) = _____(20__)

CURRENT HCAADT (YEAR) = _____(20__)

[Use on all precast concrete box culvert projects. Insert appropriate values.]

MATERIAL DESIGN PROPERTIES:

PRECAST REINFORCED CONCRETE:

f'_c = __ KSI CONCRETE

f_y = 65 KSI WELDED WIRE REINFORCEMENT

f_y = 60 KSI REINFORCEMENT BARS

[Use on all precast concrete box culvert projects. Insert concrete strength.]

HL-93 LRFR

BRIDGE OPERATING RATING FACTOR RF = ______

[Use on all precast concrete box culvert projects where the box culvert has been assigned a bridge number. For standard designs, insert 1.3 rating factor. For non-standard designs, insert calculated rating factor.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

B. CONSTRUCTION NOTES

THE 20__ EDITION OF THE MINNESOTA DEPARTMENT OF TRANSPORTATION "STANDARD SPECIFICATIONS FOR CONSTRUCTION" SHALL GOVERN.

[Use on all projects. Insert current date of specifications.]

SEE SPECIAL PROVISIONS FOR ALL XXXX.6XX SERIES PAY ITEMS FOR ADDITIONAL REQUIREMENTS.

[Use on all projects.]

THE BAR SIZES SHOWN IN THIS PLAN ARE IN U.S. CUSTOMARY DESIGNATIONS.

[Use on all projects.]

PROVIDE EPOXY COATED BARS MARKED WITH THE SUFFIX "E" IN ACCORDANCE WITH SPEC. 3301.

[Use on all projects.]

PROVIDE STAINLESS STEEL BARS MARKED WITH THE SUFFIX "S" IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

[Use as required.]

PROVIDE EPOXY COATED 4% CHROMIUM BARS MARKED WITH THE SUFFIX "M" IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

[Use as required.]

PROVIDE GFRP BARS MARKED WITH THE SUFFIX "F" IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

[Use as required.]

BRIDGE ELEMENTS WITH SYMBOL MC NEXT TO MIX DESIGNATION REQUIRE COMPLIANCE WITH MASS CONCRETE PROVISIONS.

[Use as required. Apply the MC symbol to all concrete mix designation callouts depicted in the Final Bridge Plan that require mass concrete.]

THE SUBSURFACE UTILITY INFORMATION IN THIS PLAN IS UTILITY QUALITY LEVEL ___. THIS UTILITY QUALITY LEVEL WAS DETERMINED ACCORDING TO THE GUIDELINES OF CI/ASCE 38-02, ENTITLED "STANDARD GUIDELINES FOR THE COLLECTION AND DEPICTION OF EXISTING SUBSURFACE UTILITY DATA".

[Use on all projects. Insert quality level shown in Preliminary Bridge Plan. When unknown, the default level is "D", which is the lowest level.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

B. CONSTRUCTION NOTES (CONT'D)

THE GIRDERS HAVE BEEN DESIGNED AND DETAILED WITHOUT DIAPHRAGMS. CONTRACTOR'S ENGINEER TO DESIGN, AND CONTRACTOR TO CONSTRUCT A TEMPORARY BRACING SYSTEM AND/OR A DECK FALSEWORK/FORMWORK SYSTEM. PROVIDE A SYSTEM WITH LATERAL AND ROTATIONAL STABILITY OF THE GIRDERS TO RESIST UNSYMMETRICAL CONCRETE AND CONSTRUCTION LOADS UNTIL THE DECK CONCRETE HAS ATTAINED A MINIMUM STRENGTH OF 2800 PSI.

[Use on all plans with pretensioned RB shapes or 27M, 30MH, or 35MH I-beams where diaphragms are not used.]

INSTALL SETTLEMENT PLATE BEHIND EACH ABUTMENT. SEE GRADING PLANS.

[Use when specified in Foundation Recommendations.]

DO NOT START CONSTRUCTION OF EACH ABUTMENT UNTIL THE APPROACH FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE FULL HEIGHT AND CROSS SECTION (AND ALLOWED TO SETTLE FOR _____ DAYS).

[Use when specified in Foundation Recommendations. Insert waiting period if required.]

C. SIGNATURE BLOCK

APPROVED ____________________________________

STATE BRIDGE ENGINEER

DATE _____________

[Use on all projects. Provide signature line in the title block on the General Plan and Elevation sheet.]

D. DRAINAGE AND EROSION CONTROL

RESTORE SIDE DITCHES AFTER PLACEMENT OF SLOPE PAVING TO PROVIDE DRAINAGE AS DIRECTED BY THE ENGINEER. INCLUDE RESTORATION COSTS IN PRICE BID FOR STRUCTURE EXCAVATION.

[Use this note on railroad underpasses.]

PLACE ________________ PIPE UNDER GRADING PORTION OF CONTRACT.

[Use this note with combined Bridge and Roadway contracts only. Modify the notes to suit job requirements. Insert pipe description.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

E. EXCAVATION AND EARTHWORK

QUANTITY OF STRUCTURE EXCAVATION FOR PAYMENT IS COMPUTED WITH THE ELEVATION SHOWN FOR EACH SUBSTRUCTURE UNIT AS THE UPPER LIMIT. PAYMENT FOR EXCAVATION ABOVE THESE ELEVATIONS IS UNDER THE GRADING PORTION OF THE CONTRACT.

[Use this note when rock and other type excavation will be encountered. Do not use this note when lump sum payment for structure excavation is used. Specify an elevation for top of exposed or buried rock and add the note "Average elevations of top of rock are assumed for estimated plan quantities."]

THE LOWER LIMITS OF STRUCTURE EXCAVATION CLASS E IS THE SAME AS THE UPPER LIMITS OF STRUCTURE EXCAVATION CLASS WE EXCEPT FOR ROCK EXCAVATIONS.

[Use as required.]

ROADWAY (OR CHANNEL) EXCAVATION WILL BE COMPLETED BY OTHERS IN ADVANCE OF BRIDGE CONSTRUCTION.

[Not applicable on combined project.]

KEY FOOTINGS INTO SOUND BEDROCK AS DIRECTED BY THE ENGINEER. PROVIDE 1'-0" MINIMUM COVER TO TOP OF FOOTINGS.

[Use as required.]

DRESS SLOPES AND PLACE FILTER MATERIALS AND RIPRAP IN APPROXIMATE AREAS AS DIRECTED BY THE ENGINEER.

[Use as required.]

SUBCUT FOOTING A MINIMUM OF 2'-0" AND PLACE AGGREGATE BACKFILL (CONFORMING TO SPEC. 3149.2E) IN ACCORDANCE WITH SPEC. 2451 AS SHOWN. COMPACT TO 100% MAXIMUM DENSITY IN ACCORDANCE WITH SPEC. 2106.3G. SEE SPECIAL PROVISIONS.

[Use when a subcut for a spread footing is required.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

F. REINFORCEMENT

SPIRAL DATA
OUTSIDE DIAMETER ________
HEIGHT ________
PITCH _________
SPIRAL ROD SIZE, PLAIN ROUND _________
WEIGHT, EACH ________

[Use for round columns that contain spiral reinforcement. Insert appropriate data.]

OUTSIDE DIAMETER OF DOWEL CIRCLE TO BE 2$^{1}/_{4}$" LESS THAN INSIDE DIAMETER OF SPIRAL.

[Use for round columns that contain spiral reinforcement. Where No. 10 and larger sized column vertical bars are used, increase the 2$^{1}/_{4}$" dimension where required to provide for a proper fit.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

G. FOUNDATIONS

a. SPREAD FOOTINGS ON SOIL

___________ ABUTMENT SPREAD FOOTING LOAD DATA	
* _____†_____ DESIGN BEARING PRESSURE	TONS/SQ FT
EFFECTIVE WIDTH B'	FT
_____†_____ BEARING RESISTANCE $\phi_b \cdot q_n$	TONS/SQ FT

* BASED ON ___________ LOAD COMBINATION.

PIER ________ SPREAD FOOTING LOAD DATA	
* _____†_____ DESIGN BEARING PRESSURE	TONS/SQ FT
EFFECTIVE WIDTH B' (PERPENDICULAR TO PIER)	FT
EFFECTIVE LENGTH L' (PARALLEL TO PIER)	FT
_____†_____ BEARING RESISTANCE $\phi_B \cdot q_n$	TONS/SQ FT

* BASED ON ___________ LOAD COMBINATION.

[Use for spread footing foundations on soil. Insert data. Round loads to nearest 0.1 tons/sq ft.]
[† Insert "SERVICE" or "FACTORED" based on governing limit state.]

b. SPREAD FOOTINGS ON ROCK

___________ ABUTMENT SPREAD FOOTING LOAD DATA	
* MAX FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* MIN FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
FACTORED BEARING RESISTANCE $\phi_b \cdot q_n$	TONS/SQ FT

* BASED ON ___________ LOAD COMBINATION.

PIER ________ SPREAD FOOTING LOAD DATA	
* FTG CORNER 1 MAXIMUM FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* FTG CORNER 2 FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* FTG CORNER 3 FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* FTG CORNER 4 FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
FACTORED BEARING RESISTANCE $\phi_b \cdot q_n$	TONS/SQ FT

* BASED ON ___________ LOAD COMBINATION.

[Use for spread footing foundations on rock. Define footing corners in bridge Plan. Insert data. Round loads to nearest 0.1 tons/sq. ft.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

G. FOUNDATIONS (CONT'D)

c. LOADS FOR PILES

___________ ABUTMENT COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD + EARTH PRESSURE	
FACTORED LIVE LOAD	
* FACTORED DESIGN LOAD	

* BASED ON ___________ LOAD COMBINATION.

PIER ________ COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD	
FACTORED LIVE LOAD	
FACTORED OVERTURNING	
* FACTORED DESIGN LOAD	

* BASED ON ___________ LOAD COMBINATION.

[Use for pile foundations. Insert data. Round loads to nearest 0.1 tons/pile.]

d. ADDITIONAL LOAD TABLE FOR PILES WHERE DOWNDRAG OCCURS

___________ ABUTMENT COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD + EARTH PRESSURE	
FACTORED DOWNDRAG	
*** FACTORED DEAD LOAD + EARTH PRESSURE + DOWNDRAG	

*** BASED ON ___________ LOAD COMBINATION, NOT INCLUDING TRANSIENT LOADS. ONLY USED FOR COMPARISON WITH FACTORED STRUCTURAL RESISTANCE. NOT TO BE USED FOR DRIVING.

PIER ________ COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD	
FACTORED OVERTURNING	
FACTORED DOWNDRAG	
*** FACTORED DEAD LOAD + OVERTURNING + DOWNDRAG	

*** BASED ON ___________ LOAD COMBINATION, NOT INCLUDING TRANSIENT LOADS. ONLY USED FOR COMPARISON WITH FACTORED STRUCTURAL RESISTANCE. NOT TO BE USED FOR DRIVING.

[Use for pile foundations where downdrag occurs. Use this table in addition to the standard load table shown above in **c**. Insert data. Round loads to nearest 0.1 tons/pile.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

G. FOUNDATIONS (CONT'D)

e. H-PILE RESISTANCE

___________ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR H-PILES R_n – TONS/PILE		
FIELD CONTROL METHOD	ϕ_{dyn}	** R_n
MnDOT PILE FORMULA 2012 (MPF12) $R_n=20\sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.60	
PDA	0.65	

** R_n = (FACTORED DESIGN LOAD) / ϕ_{dyn}

PIER ________ REQUIRED NOMINAL PILE BEARING RESISTANCE FOR H-PILES R_n – TONS/PILE		
FIELD CONTROL METHOD	ϕ_{dyn}	** R_n
MnDOT PILE FORMULA 2012 (MPF12) $R_n=20\sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.60	
PDA	0.65	

** R_n = (FACTORED DESIGN LOAD) / ϕ_{dyn}

[Use for H-pile foundations. Insert data. Round loads to nearest 0.1 tons/pile.]

f. CIP CONCRETE PILE RESISTANCE

___________ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES R_n – TONS/PILE		
FIELD CONTROL METHOD	ϕ_{dyn}	** R_n
MnDOT PILE FORMULA 2012 (MPF12) $R_n=20\sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	
PDA	0.65	

** R_n = (FACTORED DESIGN LOAD) / ϕ_{dyn}

PIER ________ REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES R_n – TONS/PILE		
FIELD CONTROL METHOD	ϕ_{dyn}	** R_n
MnDOT PILE FORMULA 2012 (MPF12) $R_n=20\sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	
PDA	0.65	

** R_n = (FACTORED DESIGN LOAD) / ϕ_{dyn}

[Use for cast-in-place concrete pile foundations. Insert data. Round loads to nearest 0.1 tons/pile.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

G. FOUNDATIONS (CONT'D)

g. OTHER

PILE NOTES [Use for substructure with test piles]

____ ______ TEST PILES _______ FT. LONG
____ ______ PILES EST. LENGTH _______ FT.
____ ______ PILES REQ'D FOR __________

[Use for projects with piles. Insert number of piles, pile type, pile length, and substructure name.]

PILE NOTES [Use for substructure without test piles]

____ ______ PILES _______ FT. LONG REQ'D. FOR __________

[Use for projects with piles. Insert number of piles, pile type, pile length, and substructure name.]

PILE NOTES [substructure with special pay items]

____ ______ PILES EST. LENGTH _______ FT. REQ'D FOR __________

[Use for projects with piles. Insert number of piles, pile type, pile length, and substructure name.]

GENERAL PILE NOTES

PILE SPACING SHOWN IS AT BOTTOM OF ____*____.

[Use for projects with piles.]
[* Insert "FOOTING" or "ABUTMENT"]

PILES MARKED THUS (O-> , H->) TO BE BATTERED ____ PER FOOT IN DIRECTION SHOWN.

[Use for all battered piling.]

PILES TO BE HP - ____.

[Use with all steel H piling.]

PILES TO HAVE A NOMINAL DIAMETER OF __________ (AND A THICKNESS OF ____").

[Use with all CIP piles. Insert pile diameter and wall thickness when specified in Foundation Recommendations.]

FOR PILE SPLICE DETAILS SEE DETAIL __*__

[Use for projects with piles.]
[* For CIP piles, insert "B201". For H-piles, insert "B202".]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

G. FOUNDATIONS (CONT'D)

DRIVE TEST PILES TO ESTIMATED FOUNDATION PILE LENGTH. IF REQUIRED NOMINAL PILE BEARING RESISTANCE IS NOT OBTAINED, WAIT 24 HOURS, AND THEN PERFORM REDRIVE IN ACCORDANCE WITH SPEC. 2452.3D.8. IF REQUIRED NOMINAL PILE BEARING RESISTANCE IS NOT OBTAINED AFTER REDRIVE, CONTINUE DRIVING TO FULL TEST PILE LENGTH.

[Use when pile redriving is specified in Foundation Recommendations.]

DENOTES REACTION PILE. INCLUDE IN ITEM "_________ TEST PILE ___ FT. LONG ___ ". WELD ANY PILE SPLICES ON REACTION PILES. SEE SPECIAL PROVISIONS.

[Use for pile load test. Insert pile type, length, and size.]

DENOTES STATIC LOAD TEST PILE. DRIVE TO ___ FT. LONG. INCLUDE IN ITEM "PILE LOAD TEST TYPE __". USE STATIC LOAD TEST PILE AS STRUCTURAL SUPPORT PILE AFTER LOAD TEST IS COMPLETE.

[Use for pile load test. Insert length to drive and pile type.]

DO NOT DRIVE FOUNDATION PILES UNTIL RESULTS OF STATIC LOAD TEST ARE PROVIDED TO THE ENGINEER. THE ENGINEER WILL AUTHORIZE PILE LENGTHS AND CONFIRM PILE DRIVING CRITERIA NO LATER THAN 5 WORKING DAYS AFTER RECEIPT OF LOAD TEST RESULTS.

[Use for pile load test.]

H. STEEL MATERIALS, FABRICATION, AND ERECTION

DIMENSIONS SHOWN ARE BASED ON AN AMBIENT TEMPERATURE OF 45°F. FABRICATOR TO ADJUST FOR TEMPERATURE AT FABRICATION.

[Include this note on the framing plan.]

ALL STRUCTURAL STEEL SHALL CONFORM TO SPEC 3309 UNLESS OTHERWISE NOTED.

[Use as required.]

INSTALL SHEAR CONNECTORS ON THE TOP FLANGE OF THE GIRDER IN THE FIELD.

[Use as required.]

THE MAXIMUM RESIDUAL CAMBER IN SPAN _________ IS ___ INCHES AT THE ______ POINT OF THE SPAN.

[Include this note on camber diagram sheet when residual camber has been included. Insert span number, residual camber, and location.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

H. STEEL MATERIALS, FABRICATION, AND ERECTION (CONT'D)

THERE IS NO RESIDUAL CAMBER IN THESE BEAMS.

[Include this note on camber diagram sheet when residual camber has not been included.]

CHORD LINE IN CAMBER DIAGRAM IS A STRAIGHT LINE FROM END TO END OF BEAM SEGMENT AT _________*_________.

[Include this note on camber diagram sheet of steel girder bridges]

[* For plate girders, insert "TOP OF BEAM WEB". For rolled beams, insert "BOTTOM OF TOP FLANGE".]

CAMBER DIAGRAM SHOWN IS FOR BEAMS IN UNLOADED POSITION AND PROVIDES FOR ALL DEAD LOAD DEFLECTIONS (AND RESIDUAL CAMBER).

[Include this note on camber diagram sheet of steel girder bridges. Include residual camber portion of note when used in the beams.]

LINE ASSEMBLY REQUIRED IN ACCORDANCE WITH SPEC. 2471 FOR BEAM SPLICES. LINE ASSEMBLE THE SECTION FROM ________ TO __________ .

[Use on bridges where girders were designed using line girder analysis. Check with the Structural Metals Unit; abutment to abutment if < 300 ft. Three adjacent points of support if > 300 ft.]

FULL ASSEMBLY REQUIRED IN ACCORDANCE WITH SPEC. 2471.

[The use of full assembly should be considered for extremely complicated curved, superelevated structures (i.e. grid or 3D analysis used for design). Check with the Structural Metals Unit and Fabrication Methods Unit.]

PRIOR TO POURING DECK, SNUG TIGHTEN ONLY BOLTS IN THE END DIAPHRAGMS. AFTER POURING ALL DECK CONCRETE, FULLY TIGHTEN END DIAPHRAGM BOLTS.

[Use on all skewed steel bridges.]

PROVIDE OVERSIZED HOLES IN END DIAPHRAGM CONNECTIONS.

[Use on all skewed steel bridges.]

DETAIL AND FABRICATE STEEL MEMBERS FOR A NO-LOAD FIT.

[Use on all steel girder bridges.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

H. STEEL MATERIALS, FABRICATION, AND ERECTION (CONT'D)

FOR WELDED FLANGE SPLICES, SEE SPEC 2471.3F.1.a.

[Use drawings instead of note only if different than AASHTO/AWS D1.5 Bridge Welding Code 2020 Fig. 2.7 or 2.8.]

FURNISH WEB PLATES IN AVAILABLE MILL LENGTHS AND WIDTHS WITH A MINIMUM NUMBER OF WEB SPLICES. LOCATION OF SPLICES ARE SUBJECT TO THE APPROVAL OF THE ENGINEER. LOCATE SPLICES A MINIMUM OF 1'-0" FROM STIFFENERS OR FLANGE SPLICES.

[Use on all steel plate girder bridges.]

BEARING STIFFENERS AT ABUTMENTS SHALL BE VERTICAL. BEARING STIFFENERS AT PIERS AND DIAPHRAGM CONNECTION STIFFENERS SHALL BE PERPENDICULAR TO FLANGE. ENDS OF BEAMS SHALL BE VERTICAL.

[Use for grades greater than 3% on plate girder bridges or skews greater than 20°.]

BEARING STIFFENERS, DIAPHRAGM CONNECTION STIFFENERS, AND ENDS OF BEAMS SHALL BE PERPENDICULAR TO FLANGE.

[Use for rolled beams or grades ≤ 3%.]

ALIGN ROWS OF SHEAR CONNECTORS PARALLEL TO THE TRANSVERSE SLAB REINFORCEMENT BARS.

[Use on all steel bridges with shear connectors.]

SHEAR CONNECTORS TO PROJECT A MINIMUM OF 2" INTO DECK STRUCTURAL SLAB. IN NO CASE SHALL SHEAR CONNECTORS PROJECT CLOSER THAN __*__" TO TOP OF DECK STRUCTURAL SLAB. ENGINEER TO FIELD VERIFY BEAM ELEVATION AND AUTHORIZE CONNECTOR LENGTH.

[Use on all steel bridges with shear connectors.]

[*Insert "1" for decks with 2" wearing course. Insert "3" for monolithic decks.]

CUT FLANGE PLATES FOR BEAMS TO PROPER CURVATURE.

[Use this note when the minimum radius of curvature requirements of AASHTO LRFD Bridge Construction Specifications, Article 11.4.12.2.2 are not met.]

MAKE ALL BOLTED CONNECTIONS WITH $^7/_8$" DIAMETER A325 BOLTS, EXCEPT AS NOTED.

[Use as required.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

H. STEEL MATERIALS, FABRICATION, AND ERECTION (CONT'D)

ELEVATIONS SHOWN AT FIELD SPLICES ARE THEORETICAL ELEVATIONS FURNISHED AS A GUIDE FOR ERECTION PURPOSES. ELEVATIONS ARE GIVEN AT TOP OF TOP FLANGE SPLICE PLATE AND INCLUDE DEFLECTIONS FROM WEIGHT OF BEAM AND DIAPHRAGM.

[Use as required.]

DEFLECTIONS SHOWN ARE FOR WEIGHT OF SLAB, CONCRETE OVERLAY, ______. NEGATIVE SIGN INDICATES UPLIFT.

[Use as required. Insert barrier, parapet, metal railing, sidewalk, median, etc. as needed. Do not include the weight of steel beams or future wearing course.]

SHEAR CONNECTORS TO CONFORM TO SPEC. 3391.

[Use on all steel bridges with shear connectors.]

I. CONCRETE PLACEMENTS

REMOVE ALL NON-GALVANIZED AND NON-EPOXY COATED FERROUS METAL, EXCLUDING SHEAR CONNECTORS, TO WITHIN ½" OF THE TOP FLANGE PRIOR TO CASTING THE DECK.

[Use on all deck-on-beam type bridges. Locate note on the Transverse Section Deck Reinforcement sheet in the bridge Plan.]

MAKE ⅜" WIDE x 1" DEEP SAW CUT IN STRUCTURAL SLAB (AND CONCRETE WEARING COURSE) OVER CENTERLINE OF PIERS AS SOON AS THE CUTTING CAN BE DONE WITHOUT RAVELING THE CONCRETE. APPLY TYPE B POLYSTYRENE TO TIPS OF FLANGES THAT PROJECT PAST CENTERLINE OF PIER. SEAL JOINT IN ACCORDANCE WITH SPEC. 3725.

[Use on prestressed concrete beam bridges with double diaphragms and slab continuous over piers. Saw cut both structural slab and concrete wearing course. See Figure 9.2.1.10 in this manual for detail.]

TOP OF SLAB UNDER BARRIER IS LEVEL. THE BOTTOM OF SLAB CONTINUES AT THE SAME SLOPE AS THE ROADWAY.

[Use for slab type bridges. Include note on superstructure sheet that contains cross-section.]

CAST COUNTERWEIGHT AT LEAST 48 HOURS IN ADVANCE OF PLACING DECK SLAB.

[Use as required.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

I. CONCRETE PLACEMENTS (CONT'D)

CONSTRUCT BARRIER SUCH THAT ANGLE BETWEEN ROADWAY SURFACE AND BACK FACE OF BARRIER IS NOT GREATER THAN 90°.

[Use for barrier on the high side of bridges with superelevation.]

REQUIRE A MINIMUM OF 72 HOURS BETWEEN ADJACENT DECK POURS.

[Use on all bridges that require a concrete deck pour sequence with two or more separate pours.]

J. WELDED STEEL BEARING ASSEMBLIES

STRUCTURAL STEEL SHALL CONFORM TO SPEC. 3306 EXCEPT AS NOTED.

[Use as required.]

INCLUDE SHIMS IN PRICE BID FOR BEARING ASSEMBLIES.

[Add to B-Detail if shims are used.]

PINS AND ROLLERS SHALL CONFORM TO SPEC. 2471.3D.4.

[Use as required.]

PINS SHALL BE COLD FINISHED ALLOY BAR STEEL IN ACCORDANCE WITH SPEC. 3314 TYPE II.

[For pins 5" or less where pin is not made from a larger diameter stock.]

PINS SHALL BE HOT ROLLED ALLOY BAR STEEL IN ACCORDANCE WITH SPEC. 3313 TYPE II.

[For pins over 5" where pin will be made from a larger diameter stock.]

PINTLES SHALL CONFORM TO SPEC. 3309.

[Use as required.]

LUBRICATED BRONZE BUSHINGS SHALL CONFORM TO SPEC. 3329.

[Use as required.]

ANNEAL ALL WELDED BEARING ASSEMBLIES AFTER WELDING. FINISH PIN HOLES AND TOP AND BOTTOM PLATES AFTER ANNEALING.

[For welded rockers and bolsters.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

J. WELDED STEEL BEARING ASSEMBLIES (CONT'D)

COAT PINS AND PIN HOLES IN THE SHOP WITH AN APPROVED HEAVY PROTECTIVE GREASE. PRIOR TO ERECTION, CLEAN AND COAT THE PINS AND PIN HOLES WITH AN APPROVED GREASE.

[Use as required.]

PROVIDE A TEMPLATE DEMONSTRATING THAT ANCHOR RODS WILL HAVE 2 INCHES CLEAR DISTANCE TO ALL REINFORCEMENT. REFER TO SPEC. 2472.3C.1 FOR MORE DETAILED INSTRUCTION.

[Include this note with anchor rod placement details.]

___ DENOTES ELASTOMERIC BEARING PAD, TYPE __. SEE DETAIL B305.
___ DENOTES FIXED CURVED PLATE BRG. ASSEMBLY, TYPE ___. SEE DETAIL B310.
___ DENOTES EXPANSION CURVED PLATE BRG. ASSEMBLY, TYPE ___. SEE DETAIL B311.

[Include appropriate notes(s) on framing plan when elastomeric bearing pads are used. Insert bearing designation and type number.]

PLAN BRIDGE SEAT ELEVATIONS ARE BASED ON AN ASSUMED DISC BEARING HEIGHT OF ___. DETERMINE FINAL BRIDGE SEAT ELEVATIONS BASED ON ACTUAL HEIGHT OF DISC BEARING ASSEMBLIES. MAKE ANY REQUIRED ADJUSTMENTS TO THE BRIDGE SEAT ELEVATIONS AT NO COST TO THE DEPARTMENT. BRIDGE SEAT PEDESTALS SHALL NOT BE LESS THAN 3" TALL.

[To be used when disc bearings are provided. Include this note on substructure plan sheet where bridge seat elevations are shown. Insert estimated height of disc bearing assembly.]

K. CUTTING AND REMOVAL OF OLD CONCRETE

HATCHED AREAS INDICATE CONCRETE REMOVAL.

[Use as required.]

OUTLINE CUTTING LIMITS AND RECEIVE APPROVAL BY THE ENGINEER PRIOR TO ANY CUTTING. PERFORM REMOVAL AND RECONSTRUCTION IN ACCORDANCE WITH SPEC. 2433.

[Use as required.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

L. JOINTS AND JOINT SEALER

PLACE CONTROL JOINT IN SIDEWALK (AND MEDIAN) OVER CENTERLINE OF PIER. USE A 1" DEEP BY 3/8" WIDE V SHAPED JOINT IN TOP OF SIDEWALK (AND MEDIAN) AND FINISH WITH 1/4" RADIUS EDGER. USE 1/2" V STRIP IN VERTICAL EDGE OF SIDEWALK. BREAK BOND AT JOINT BY APPROVED METHOD. SEAL CONTROL JOINT IN ACCORDANCE WITH SPEC. 3722. NO REINFORCEMENT THROUGH JOINT.

[Use for raised sidewalk and median concrete sections less than 12" in height.]

INCLUDE 1½" TYPE B POLYSTYRENE BETWEEN APPROACH PANEL AND WINGWALL IN GRADING PLAN.

[Use as required.]

SUPERSTRUCTURE DIMENSIONS ARE BASED ON THE EXPANSION JOINT DIMENSION AT 90° F.

[Use when superstructure details show more than one expansion joint dimension for different temperatures.]

M. TIMBER BRIDGES

CONSTRUCTION REQUIREMENTS IN ACCORDANCE WITH SPEC. 2403.

[Use as required.]

FURNISH ALL TIMBER PILING IN ACCORDANCE WITH SPEC. 3471.
GALVANIZE ALL HARDWARE IN ACCORDANCE WITH SPEC. 3392.
UPSET THREADS ON ALL BOLTS AFTER INSTALLATION.

[Use as required.]

FURNISH ALL WOOD AS SHOWN IN THE BILL OF MATERIALS.

[Use as required.]

SHAPE TOP OF WING PILE WHICH PROJECTS OUTSIDE OF WING CAP TO A 45° SLOPE.

[Use as required.]

TREAT TOPS OF WING PILES IN ACCORANCE WITH SPEC. 2403.3E. SEE SPEC. 3491 FOR PRESERVATION REQUIREMENTS.

[Use as required.]

DO NOT PLACE FILL IN BACK OF ABUTMENT UNTIL AFTER SUPERSTRUCTURE IS COMPLETE.

[Use as required.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

M. TIMBER BRIDGES (CONT'D)

FASTEN BACKING TO ABUTMENT PILES WITH TWO 60D NAILS AT EACH INTERSECTION.
[Use as required.]

CUT OFF BOLT PROJECTIONS EXCEEDING 1". REPAIR END OF BOLT BY PAINTING WITH AN APPROVED ZINC-RICH PRIMER.
[Use as required.]

INSTALL TIMBER WINGWALL PILES TO THE LENGTH SHOWN IN THE PLANS.
[Use as required.]

PRESERVATIVE TREAT ALL WOOD IN ACCORDANCE WITH SPEC. 3491.
[Use as required.]

FIELD TREAT ALL WOOD THAT IS CUT OR DRILLED IN THE FIELD IN ACCORDANCE WITH SPEC. 2403.3E.
[Use as required.]

ALL PLANKS FOR PREFAB PANELS SHALL BE DOUGLAS FIR-LARCH GRADE ___ (F_{bo} = ___ KSI).
[Use as required. Insert grade and reference design value.]

FOR GLULAM RAIL ELEMENTS SEE PLAN SHEET ___ .
[Use as required. Insert sheet number.]

RAIL POSTS, CURBS, SCUPPERS, AND RAIL SPACER BLOCKS SHALL BE DOUGLAS FIR-LARCH GRADE ___ (MIN. F_{bo} = ___ KSI).
[Use as required. Insert grade and reference design value.]

ABUTMENT AND PIER CAPS SHALL BE DOUGLAS FIR-LARCH GRADE ___ POSTS AND TIMBERS (F_{bo} = ___ KSI). ALL OTHER LUMBER SHALL HAVE MIN. F_{bo} = ___ KSI.
[Use as required. Insert grade and reference design values.]

SHOP PAINT PILE PLATE ASSEMBLY AFTER FABRICATION IS COMPLETED.
PAINT PIER PILES, PLATES AND RINGS IN ACCORDANCE WITH SPEC. 2452.3J.1
CLEAN AND SPOT COAT AREAS OF PILES, PLATES AND RINGS BURNED DURING WELDING WITH ZINC-RICH PRIMER BEFORE FIELD PAINTING IS STARTED.
[Use as required.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

M. TIMBER BRIDGES (CONT'D)

ALL STRUCTURAL STEEL SHALL CONFORM TO SPEC 3306 UNLESS OTHERWISE NOTED.
[Use as required.]

GALVANIZE ALL STRUCTURAL STEEL IN ACCORDANCE WITH SPEC. 3394.
[Use as required.]

N. MISCELLANEOUS

F.F. DENOTES FRONT FACE.
B.F. DENOTES BACK FACE.
E.F. DENOTES EACH FACE.
[Use as required.]

TAKE FIELD MEASUREMENTS AS NECESSARY PRIOR TO FABRICATION OF THE __________ TO ASSURE PROPER FIT IN THE FINAL WORK.
[Use when not otherwise referenced to Spec. 2433. Insert component to be fabricated.]

BEAM LENGTH DIMENSIONS ARE SLOPED LENGTHS.
[Use where necessary for proper fit for prestressed beams.]

USE GREASE FROM THE "APPROVED/QUALIFIED PRODUCT LIST FOR BRIDGE PRODUCTS, BRIDGE GREASE".
[Use for dowel bar assemblies.]

EPOXY COAT ALL MATERIAL. INCLUDE MATERIAL AND PLACEMENT IN PRICE BID FOR "DOWEL BAR ASSEMBLY (EPOXY COATED)".
[Use for dowel bar assemblies.]

____ PASSAGE BENCH AT EL. _____. SURFACE WITH COARSE FILTER AGGREGATE IN ACCORDANCE WITH SPEC. 3149.2H TO PROVIDE SMOOTH SURFACE. INCLUDED IN ITEM "RANDOM RIPRAP CLASS ___".
[Use when a passage bench is required. Insert appropriate values.]

"X" DENOTES X END OF BEAM.
[Include on framing plan for pretensioned concrete beam bridges.]

APPENDIX 2-C (Continued)
STANDARD PLAN NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

N. MISCELLANEOUS (CONT'D)

PROVIDE 1/8 INCH 60 DUROMETER PLAIN ELASTOMERIC PAD OR PREFORMED FABRIC PAD MEETING AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATION ARTICLE 18.10. WAIVE THE SAMPLING AND TESTING REQUIREMENTS UNDER SPEC 3741, "ELASTOMERIC BEARING PADS," AND AASHTO M251.

[Use as required.]

APPENDIX 2-D
STANDARD SUMMARY OF QUANTITIES NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

PAYMENT FOR ___________ INCLUDED IN ITEM "________________".

[For minor items with small quantities that are not listed as pay items (joint filler, nameplate, etc.). Fill in blanks with description and appropriate pay item (e.g. for joint filler: Payment for joint filler included in item "Structural Concrete (3B52))".]

INCLUDED IN WEIGHT OF "STRUCTURAL STEEL (33__)".

[Miscellaneous steel quantities (protection angle, etc.).]

DOES NOT INCLUDE TEST PILES.

[Use when piling quantities are listed.]

INCLUDES SLAB, END DIAPHRAGM, MEDIAN BARRIER, SIDEWALK, AND RAILING REINFORCEMENT.

[Add to epoxy coated reinforcement bar totals. Remove components that do not apply.]

"CONCRETE WEARING COURSE (3U17A)" INCLUDES ______ SQUARE FEET FOR BRIDGE APPROACH PANELS.

[Use when the item as listed in the Summary of Quantities for Superstructure is paid for on a square foot basis. Insert area.]

PAYMENT FOR BEARINGS INCLUDED IN ITEM "BEARING ASSEMBLY" PER EACH.

[Use as required.]

PAYMENT FOR ANCHORAGES INCLUDED IN ITEM "ANCHORAGES TYPE REINF BARS" PER EACH.

[Use as required.]

PAYMENT FOR ANCHORAGES INCLUDED IN ITEM "GROUTED REINFORCEMENT BARS" PER EACH.

[Use as required.]

PAYMENT FOR THREADED COUPLERS INCLUDED IN ITEM "COUPLERS (REINFORCEMENT BARS) T-___" PER EACH.

[Use as required. Specify bar size.]

MEMBRANE WATERPROOFING SYSTEM IN ACCORDANCE WITH SPEC 2481.3B TO BE INCLUDED IN ITEM "STRUCTURAL CONCRETE (____)".

[Use as required. Insert concrete mix.]

APPENDIX 2-D (Continued)

STANDARD SUMMARY OF QUANTITIES NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the Plan.

PAYMENT FOR SHEAR CONNECTORS INCLUDED IN ITEM "SHEAR STUDS".
[Use as required.]

SEE DRAINAGE SYSTEM TYPE (B910).
[Use as required.]

INCLUDE PILE REDRIVING AND PILE ANALYSIS FOR THE REACTION PILES AND STATIC LOAD TEST PILE IN PRICE BID FOR "PILE LOAD TEST TYPE __".
[Use as required. Insert type number.]

	0"	1"	2"	3"	4"	5"		6"	7"	8"	9"	10"	11"	
0	.0000	.0833	.1667	.2500	.3333	.4167	0	.5000	.5833	.6667	.7500	.8333	.9167	0
1/16	.0052	.0885	.1719	.2552	.3385	.4219	1/16	.5052	.5885	.6719	.7552	.8385	.9219	1/16
1/8	.0104	.0938	.1771	.2604	.3438	.4271	1/8	.5104	.5938	.6771	.7604	.8438	.9271	1/8
3/16	.0156	.0990	.1823	.2656	.3490	.4323	3/16	.5156	.5990	.6823	.7656	.8490	.9323	3/16
1/4	.0208	.1042	.1875	.2708	.3542	.4375	1/4	.5208	.6042	.6875	.7708	.8542	.9375	1/4
5/16	.0260	.1094	.1927	.2760	.3594	.4427	5/16	.5260	.6094	.6927	.7760	.8594	.9427	5/16
3/8	.0313	.1146	.1979	.2813	.3646	.4479	3/8	.5313	.6146	.6979	.7813	.8646	.9479	3/8
7/16	.0365	.1198	.2032	.2865	.3698	.4531	7/16	.5365	.6198	.7032	.7865	.8698	.9531	7/16
1/2	.0417	.1250	.2083	.2917	.3750	.4583	1/2	.5417	.6250	.7083	.7917	.8750	.9583	1/2
9/16	.0469	.1302	.2135	.2969	.3802	.4635	9/16	.5469	.6302	.7135	.7969	.8802	.9635	9/16
5/8	.0521	.1354	.2188	.3021	.3854	.4688	5/8	.5521	.6354	.7188	.8021	.8854	.9688	5/8
11/16	.0573	.1406	.2240	.3073	.3906	.4740	11/16	.5573	.6406	.7240	.8073	.8906	.9740	11/16
3/4	.0625	.1458	.2292	.3125	.3958	.4792	3/4	.5625	.6458	.7292	.8125	.8958	.9792	3/4
13/16	.0677	.1510	.2344	.3177	.4010	.4844	13/16	.5677	.6510	.7344	.8177	.9010	.9844	13/16
7/8	.0729	.1563	.2396	.3229	.4063	.4896	7/8	.5729	.6563	.7396	.8229	.9063	.9896	7/8
15/16	.0781	.1615	.2448	.3281	.4115	.4948	15/16	.5781	.6615	.7448	.8281	.9115	.9948	15/16
	0"	1"	2"	3"	4"	5"		6"	7"	8"	9"	10"	11"	

CONVERSION TABLE
INCHES TO DECIMALS OF A FOOT

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[5.4.2.4] For reinforced concrete elements, use: $E_c = 120{,}000 \cdot K_1 \cdot w_c^2 \cdot f'_c{}^{0.33}$

For checks based on strength (design of reinforcement, maximum reinforcement), use conventional strength methods (reinforcement yielding, Whitney equivalent stress block, etc.).

For checks based on service loads (fatigue, crack control, etc.), use cracked sections with reinforcing steel transformed to an equivalent amount of concrete.

Mass Concrete Elements

Identify mass concrete elements by determining if their geometry meets the criteria listed in Table 5.1.1.2. If the mass concrete designation is required, include any associated standard notes and mass concrete pay item. In addition, identify all required elements as mass concrete anywhere their concrete mix designation is depicted in the Final Bridge Plans.

Table 5.1.1.2 Mass Concrete Dimension Requirements

Concrete Element	Least Dimension*
Buried Footings and Pier Crash Struts	≥ 60 in.
All Other Concrete Elements	> 48 in.

* When the concrete element has variable dimensions on a given axis, use the largest variable dimension for the least dimension determination.

Prestressed Concrete Elements

When computing section properties, use a modular ratio of 1 for the prestressing strands.

For pretensioned beams (M, MH, MN, MW, and RB) fabricated using concrete with a final concrete strength, f'_c, greater than 6.0 ksi, compute the modulus of elasticity using the ACI 363 equations below for the concrete at all stages of strength:

$E_{ci} = 1265 \cdot \sqrt{f'_{ci}} + 1000$ (where f'_{ci} and E_{ci} are in ksi)

$E_c = 1265 \cdot \sqrt{f'_c} + 1000$ (where f'_c and E_c are in ksi)

For all other pretensioned and post-tensioned elements, compute the modulus of elasticity using AASHTO LRFD Equation 5.4.2.4-1, with $K_1 = 1$ and $w_c = 0.150$ kcf.

For both pretensioned and post-tensioned elements, use a unit weight of 0.155 kcf for dead load calculations.

Table 5.1.1.3 summarizes concrete properties for analysis and design:

Table 5.1.1.3
Concrete Properties

Parameter	Equation/Value
Unit Weight	Reinforced Concrete Elements: w_c = 0.145 kcf for calculation of E_c w_c = 0.150 kcf for dead load calculation Pretensioned and Post-tensioned Elements: w_c = 0.150 kcf for calc. of E_c (except pretensioned beams with final concrete strength f'_c > 6 ksi) w_c = 0.155 kcf for dead load calculation
Modulus of Elasticity	Pretensioned Beams: Where $f'_c \le 6$ ksi: E_{ci} (ksi) $= 120{,}000 \cdot K_1 \cdot w_c^2 \cdot f'^{0.33}_{ci}$ E_c (ksi) $= 120{,}000 \cdot K_1 \cdot w_c^2 \cdot f'^{0.33}_c$ Where $f'_c > 6$ ksi E_{ci} (ksi) $= 1265 \cdot \sqrt{f'_{ci}} + 1000$ E_c (ksi) $= 1265 \cdot \sqrt{f'_c} + 1000$ All Other Concrete Elements: E_c (ksi) $= 120{,}000 \cdot K_1 \cdot w_c^2 \cdot f'^{0.33}_c$
Thermal Coefficient	$\alpha_c = 6.0 \times 10^{-6}$ in/in/°F
Shrinkage Strain	Reinf. Conc.: ε_{sh} = 0.0002 @ 28 days and 0.0005 @ 1 year Prestressed Concrete: per LRFD Art. 5.4.2.3
Poisson's ratio	$\nu = 0.2$

5.1.2 Reinforcing Steel

Reinforcing bars shall satisfy MnDOT Spec 3301. ASTM A615 Grade 60 deformed bars (black or epoxy coated) should be used in most circumstances. In some cases, Grade 75 stainless steel bars will be required in the bridge deck and barrier (see Tech. Memo No. 17-02-B-01 *Requirements for the Use of Stainless Steel Reinforcement in Bridge Decks & Barriers*). Use f_y = 75 ksi when designing with stainless steel bars. Always use stainless steel (Grade 75) for the connecting bar between approach panel and end diaphragm at integral and semi-integral abutments.

In specialized situations and with the approval of the State Bridge Design Engineer, welding to reinforcement may be used. ASTM A706 Grade 60 bars must be used for applications involving welding.

Bottom Bars, Pile Cap w/ Pile Embedded 1 foot

- Rest directly on top of trimmed pile.

Bottom Bars, Pile Cap Alone or Where Pile Cap is Cast Against a Concrete Seal, w/ Pile Embedded More Than 1 foot

- Minimum clear cover is 3 inches to bottom of pile cap.

Abutments, Piers, and Pier Crash Struts

- Standard minimum clear cover for all bars is 2 inches (vertical and horizontal).
- At rustications, the minimum horizontal clear cover varies with the size of the recess. For recesses less than or equal to 1 inch in depth and less than or equal to 1 inch in width, the minimum clear cover is 1.5 inches. For all other cases, the minimum clear cover is 2 inches.
- Minimum clear distance between reinforcement and anchor rods is 2 inches.
- In large river piers with #11 bars or larger that require rebar couplers, minimum clear cover to bars is 2.5 inches.

Decks, Slabs, Sidewalks, and Raised Medians

Top Bars, Roadway Bridge Deck or Slab, Sidewalk, and Raised Median

- Minimum clear cover for epoxy coated bars to the top concrete surface is 3 inches.
- Minimum clear cover for stainless steel bars to the top concrete surface is 2½" inches for monolithic decks, 3" for partial depth decks with a wearing course, and 4" for concrete box girder decks.
- Minimum horizontal clear cover is 2 inches. In the bridge plan, detail bars with an edge clear cover of 2 inches, but compute bar length assuming 2½" clear.

Top Bars, Pedestrian Bridge Deck

- Minimum clear cover to the top concrete surface is 2 inches.

Bottom Bars, Deck

- Minimum clear cover to the bottom concrete surface is 1 inch.
- Minimum horizontal clear cover from the end of the bar to the face of the concrete element is 4 inches.
- Minimum horizontal clear cover from the side of a bar to the face of the concrete element is 2 inches.

Bottom Bars, Slab

- Minimum clear cover to the bottom concrete surface is 1.5 inches.
- Minimum horizontal clear cover from the end of the bar to the face of the concrete element is 4 inches.
- Minimum horizontal clear cover from the side of a bar to the face of the concrete element is 2 inches.

5.2.2 Reinforcing Bar Lists

For numbering of reinforcing bars, the first character is a unique alpha character for the given structural element. The first one or two digits of the bar mark indicate the U.S. Customary bar size. The last two digits are the bar's unique sequential number in the bar list for that substructure or superstructure unit. A suffix "E" indicates the bar is epoxy coated, "G" indicates the bar is galvanized, "S" indicates the bar is stainless steel, "M" indicates the bar is epoxy coated 4% chromium, "F" indicates the bar is Glass Fiber Reinforced Polymer (GFRP), "Y" indicates a Grade 75 epoxy coated bar, and "Z" indicates a Grade 75 plain bar.

For example, an A603E bar could be decoded as follows:

A – 6 – 03 – E

Epoxy coated bar
Bar number 3 for this structural unit
Size of bar is #6
Abutment

The cross-sectional areas, diameters, and weights of standard reinforcing bars are provided in Table 5.2.2.1.

Table 5.2.2.1
Reinforcing Steel Sizes and Properties

U.S. Customary Bar Size	Area of Bar (in^2)	Diameter of Bar (in)	Weight of Bar (lb/ft)
#3	0.11	0.375	0.376
#4	0.20	0.500	0.668
#5	0.31	0.625	1.043
#6	0.44	0.750	1.502
#7	0.60	0.875	2.044
#8	0.79	1.000	2.670
#9	1.00	1.128	3.400
#10	1.27	1.270	4.303
#11	1.56	1.410	5.313
#14	2.25	1.693	7.650
#18	4.00	2.257	13.60

Table 5.2.2.2 lists the reinforcing steel area provided (per foot) for different sized bars with different center to center bar spacings.

For bridges with curved decks supported by straight pretensioned beams, use the following guidance when laying out the geometry:

- For exterior beam on outside of curve, provide a minimum overhang of 6 inches between the edge of deck and outside edge of beam flange at the beam ends.
- For exterior beam on inside of curve, provide a minimum overhang of 6 inches between the edge of deck and outside edge of beam flange at the beam midspan.
- When choosing the spacing from the exterior beam to the first interior beam, consider the capacity of the exterior beam. The curvature will likely cause load demands that differ from beams that have a constant overhang.
- Orient each of the first interior beams parallel to the corresponding exterior beam.
- If the angle between the exterior beams is small, consider making them parallel. Note that this may require rechecking the overhang geometry to meet the minimum overhang stated above.
- Space the rest of the beams as evenly as possible. It is preferable to use only one flared space, if possible, to limit the number of different beam lengths and diaphragm lengths.
- Use a minimum beam spacing of 4 feet for RB, M, and MN series beams to provide sufficient space for diaphragms and inspection. Use a minimum beam spacing of 5 feet for MH and MW beams.

5.4.2 Stress Limits [5.9.2.2] [5.9.2.3]

For typical prestressed beams, check tension and compression service load stresses at two stages. The first stage is when the prestress force is transferred to the beams in the fabricator's yard. The second stage is after all losses have occurred and the beam is in the fully constructed bridge.

Design pretensioned beams with a maximum tension at transfer (after initial losses) of:

- Locations not considering bonded reinforcement

  $f_{init\_allow} = 0.0948 \cdot \sqrt{f'_{ci}} \leq 0.2$ ksi (where f'_{ci} is in ksi)

- Locations considering bonded reinforcement

  $f_{init\_allow} = 0.24 \cdot \sqrt{f'_{ci}}$ (where f'_{ci} is in ksi)

  When using the bonded reinforcement tension limit for simply supported pretensioned beams, provide a minimum area of developed longitudinal tension reinforcement in accordance with Table 5.4.2.1 unless calculated per AASHTO Article C5.9.2.3.1b.

Table 5.4.2.1
Minimum Top Flange Longitudinal Bonded Reinforcement at Beam Ends

Beam Shape	Standard Plans A_s ① (in^2)	Minimum Required A_s ② (in^2)
M	3.2	2.6
MH	3.2	3.1
MN	4.7	3.8
MW	6.3	5.8

① Area of top flange mild reinforcement in beam end region included in Bridge Details Part II Fig. 5-397.501 through 5-397.532.

② Minimum required area of top flange mild reinforcement to develop the maximum tensile force permitted by the bonded reinforcement tension limit (f_{init_allow}) with a concrete strength (f'_{ci}) of 8 ksi or less.

Design pretensioned beams with a maximum tension after all losses of:

$$f_{final\_allow} = 0.19 \cdot \sqrt{f'_c} \quad \text{(where } f'_c \text{ is in ksi)}$$

Determine live load distribution using the approximate methods of LRFD Article 4.6.2.2 and check tension stress after all losses using the Service III Limit State.

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5.4.3 Design/Analysis

[5.9.3.2.3a]

[5.9.3.3]

Use gross beam properties for design (i.e - do not transform prestressing strands to get transformed properties). Calculate the instantaneous losses (elastic shortening losses) using LRFD Equation C5.9.3.2.3a-1. Also, do not include any elastic gains caused by application of loads. Use the "approximate method" given in LRFD Article 5.9.3.3 to compute time-dependent prestress losses.

Design all pretensioned beams using uncoated low relaxation 0.6 inch diameter strands (A_s = 0.217 in^2) and epoxy coated mild reinforcement.

At the time of prestress transfer (initial), the minimum required concrete strength (f'_{ci}) is 4.5 ksi and the maximum is limited to 8.0 ksi. At the termination of the curing period (final), the minimum concrete strength (f'_c) is 5 ksi and the maximum strength is 9.5 ksi. Higher initial or final strengths may only be used with approval from the State Bridge Design Engineer. Note that an initial concrete strength greater than 7.5 ksi may add cost to the beam. The fabricator cannot remove the beam from the bed until a cylinder break indicates the concrete has reached its specified initial strength. Strengths higher than 7.5 ksi may require the fabricator to leave the beam in the bed longer than the normal 16-18 hours or require increased amounts of superplastizer and cement, thereby increasing the cost of the beam.

Generally, fabricators have stated that it is most cost effective to design beams with concrete strengths up to 7.5 ksi initial and 9.5 ksi final with as few strands as possible. If the design requires a higher initial concrete strength, the initial concrete strength can be increased up to 8.0 ksi. When a designer is faced with the decision to add strands or increase the concrete strength, it is more economical to increase the concrete strength up to the maximum limits allowed.

Do not use the maximum concrete strengths listed above unless needed. Optimize the design by back-calculating the initial and final concrete strengths needed to meet the allowable stress limits, and then reanalyze the beam with the new values. Reanalysis is needed because changes to the concrete strengths f'_{ci} and f'_c affect the concrete modulus, which affects the prestress losses and the composite beam section modulus. Use the lowest concrete strengths needed for the design. Design the concrete strengths to the nearest 0.1 ksi and report these values on the standard beam sheet.

If possible, design so that the initial concrete strength is 0.5 to 1.0 ksi lower than the final concrete strength. Since concrete naturally gains strength with age, the final strength of the beam will be more efficiently utilized.

In order to minimize the number of fabricator requests for strand pattern changes, use the following rules for placement of prestressing strands:

- Arrange straight strands in a 2 inch grid pattern with the bottom row of straight strands located 2 inches from the bottom of the beam. See standard beam sheets for possible strand locations.
- Use draped strands to reduce the initial required strength f'_{ci} at the end of the beam. Arrange draped strands in a 2 inch grid pattern independent of the straight strands. Locate draped strands starting 4 inches minimum from the bottom of the beam at the hold-downs and 3 inches minimum from the top at the end of the beam. Straight strands should be used in place of draped strands whenever possible. A design with the lowest number of draped strands is often preferred for economic and safety reasons. If using draped strands is necessary, it is desirable to minimize the number of draped strands and corresponding hold-down force. If possible, limit the draped strand angle of inclination to less than 6 degrees.
- For all designs, include a base set of straight strands in the locations shown in Figure 5.4.3.1. These base strands provide the fabricator a stable place to tie the flange confinement reinforcement, which in turn will be used to secure the stirrups in the bottom of the beam. For designs where fully tensioning all the base strands is undesirable, it is acceptable to pull selected pairs of the base strands to a lesser initial tension of 10 kips. If no draped strands are used, include a pair of straight strands in the web columns 6 inches from the bottom of the beam.

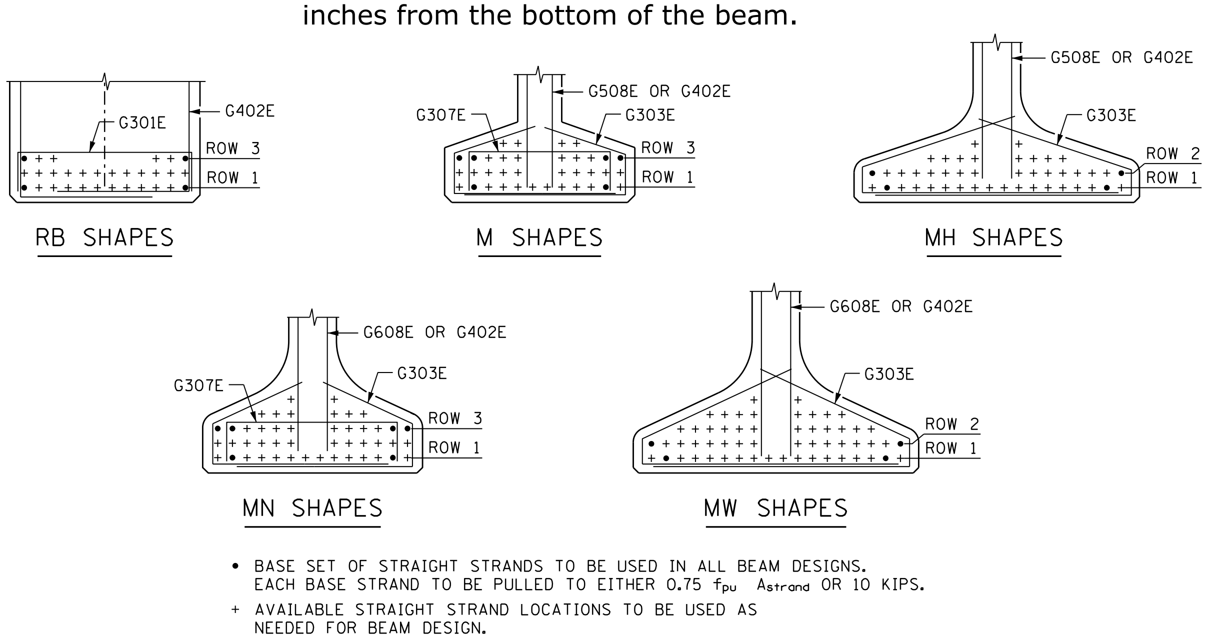


Figure 5.4.3.1

- After inclusion of the base set of strands, typically add other straight strands by starting from the bottom and moving up (i.e. – fill all of Row 1 and then all of Row 2, etc.) to get the largest eccentricity and therefore the most efficient design at midspan. Note that a smaller strand eccentricity is sometimes necessary, which may result in designs where the bottom rows are not entirely filled.
- For rows that do not need to be completely filled, fill rows from the inside out. For a given row, place the first straight strand in the column immediately outside of the stirrup, move two columns outward and place the next strand. Repeat until you reach the end of the row. If more strands are needed in the row, return to the first inside vacant column and again fill the rows from the inside out. See Figure 5.4.3.2 for examples. Place the strands with the goal of providing an approximately uniform prestress force across the width of the bottom flange.

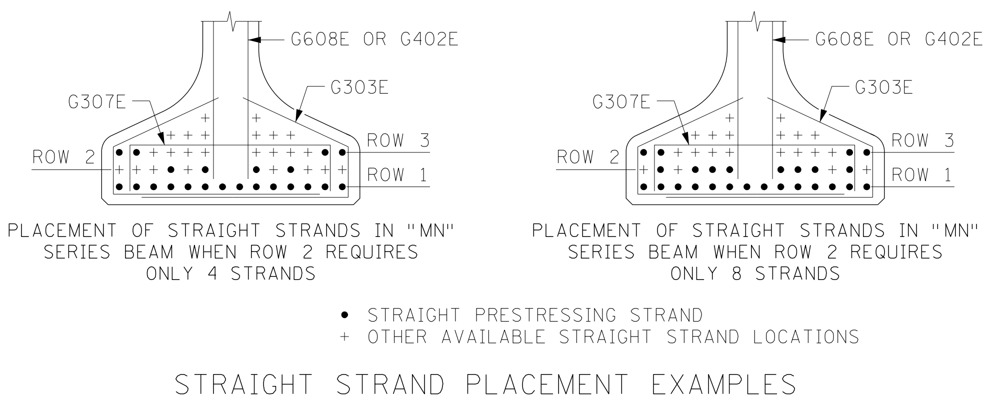


Figure 5.4.3.2

Whenever possible, use a constant strand pattern for all girders on the same project. If the strand pattern varies between beams, the fabricator may be required to tension an entire bed length of strand in order to cast a single girder. This results in a large amount of wasted strand and will increase the cost of the beam.

For pretensioned I-beams with draped strands, the maximum total initial pretensioning force allowed is 3000 kips. This limit is based on the capacity of the fabricator pretensioning beds.

The maximum number of draped strands allowed at each hold-down point varies with the fabricator. Therefore, design and detail beams with one hold-down on each side of midspan, placed at 0.40L to 0.45L from the centerline of bearing. The fabricator will provide additional hold-downs as needed.

5.5.2 PT I-Girders

Post-tensioned spliced I-girder bridges are not commonly used in Minnesota, but the MW series beams were developed with consideration of future use for spliced girder bridges. MnDOT will develop appropriate details as potential projects are identified.

5.5.3 PT Precast or Cast-In-Place Box Girders

The depth of box girders should preferably be a minimum of $^{1}/_{18}$ of the maximum span length.

Place vertical webs of box girders monolithic with the bottom slab.

5.6 Concrete Finishes and Coatings

The finish or coating to be used on concrete elements will usually be determined when the Preliminary Bridge Plan is assembled. In general, provide a finish or coating consistent with the guidance given in the *Aesthetic Guidelines for Bridge Design Manual*.

A wide variety of surface finishes for concrete are used on bridge projects. These range from plain concrete to rubbed concrete to painted surfaces to form liners and stains. Plain concrete and rubbed concrete finishes are described in the MnDOT Spec. 2401. Painted and architectural surfaces must be described in the special provisions.

Specify graffiti protection for concrete elements with a coating system that has more than one color.

5.7 Design Examples

Section 5 concludes with design examples.

The examples are for the following:

- Article 5.7.1: Three-span continuous constant depth reinforced concrete slab superstructure.
- Article 5.7.2: Two-span pretensioned draped concrete I-beam superstructure.
- Article 5.7.3: Two-span pretensioned debonded concrete I-beam superstructure.
- Article 5.7.4: Three-span continuous haunched post-tensioned concrete slab superstructure. (Future content)

5.7.2 Draped Prestressed I-Beam Design Example

This example illustrates the design of a pretensioned I-beam for a single span bridge without skew. The 118'-0" span is supported with MnDOT "40MH" beams on integral abutments. MnDOT standard details and drawings for diaphragms (B403), barriers (Fig. 5-397.138(A)), and beams (Fig. 5-397.503) are to be used with this example. This example contains the design of a typical interior beam at the critical sections in positive flexure, shear, and deflection. The superstructure consists of six beams spaced at 6'-10" centers. A typical transverse superstructure section is provided in Figure 5.7.2.1. A framing plan is provided in Figure 5.7.2.2. The roadway section is composed of two 12' traffic lanes and two 6' shoulders. A Type S barrier is provided on each side of the bridge and a 9" monolithic concrete deck is used. Interior diaphragms are used at the interior third points based on guidance found in BDM Table 5.4.1.1.

This example uses 0.6" diameter, 300 ksi, low relaxation strands for pretensioning. Draped strands are used to control the beam end stresses.

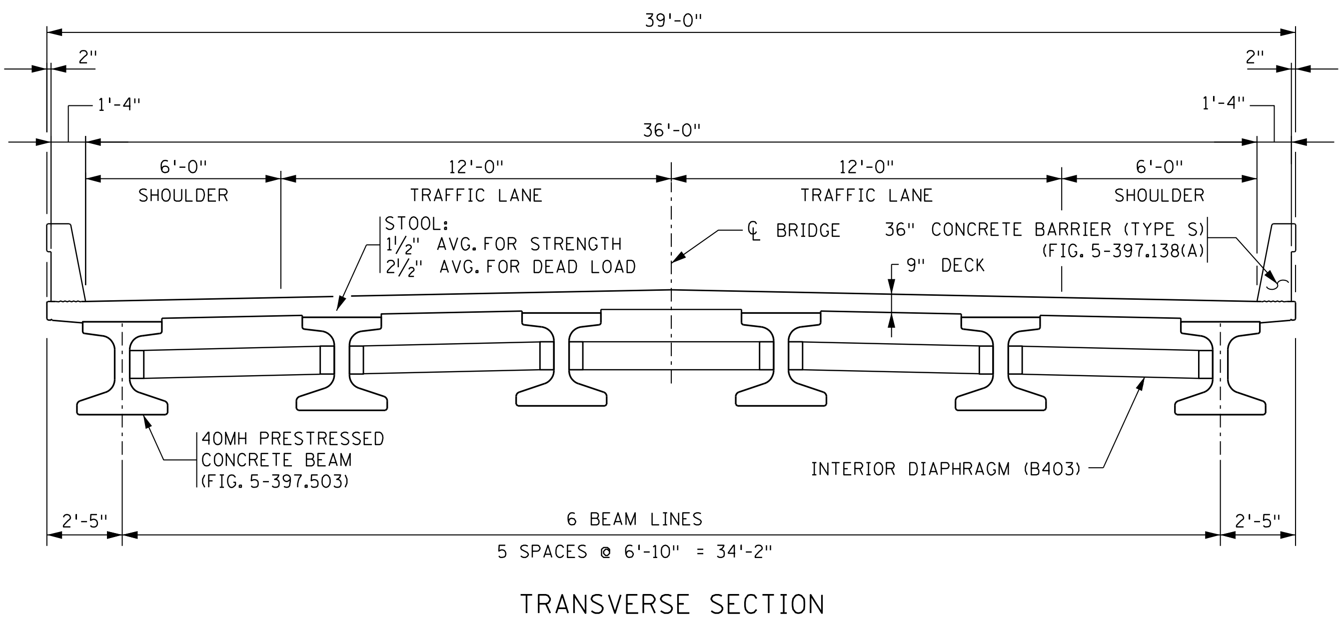
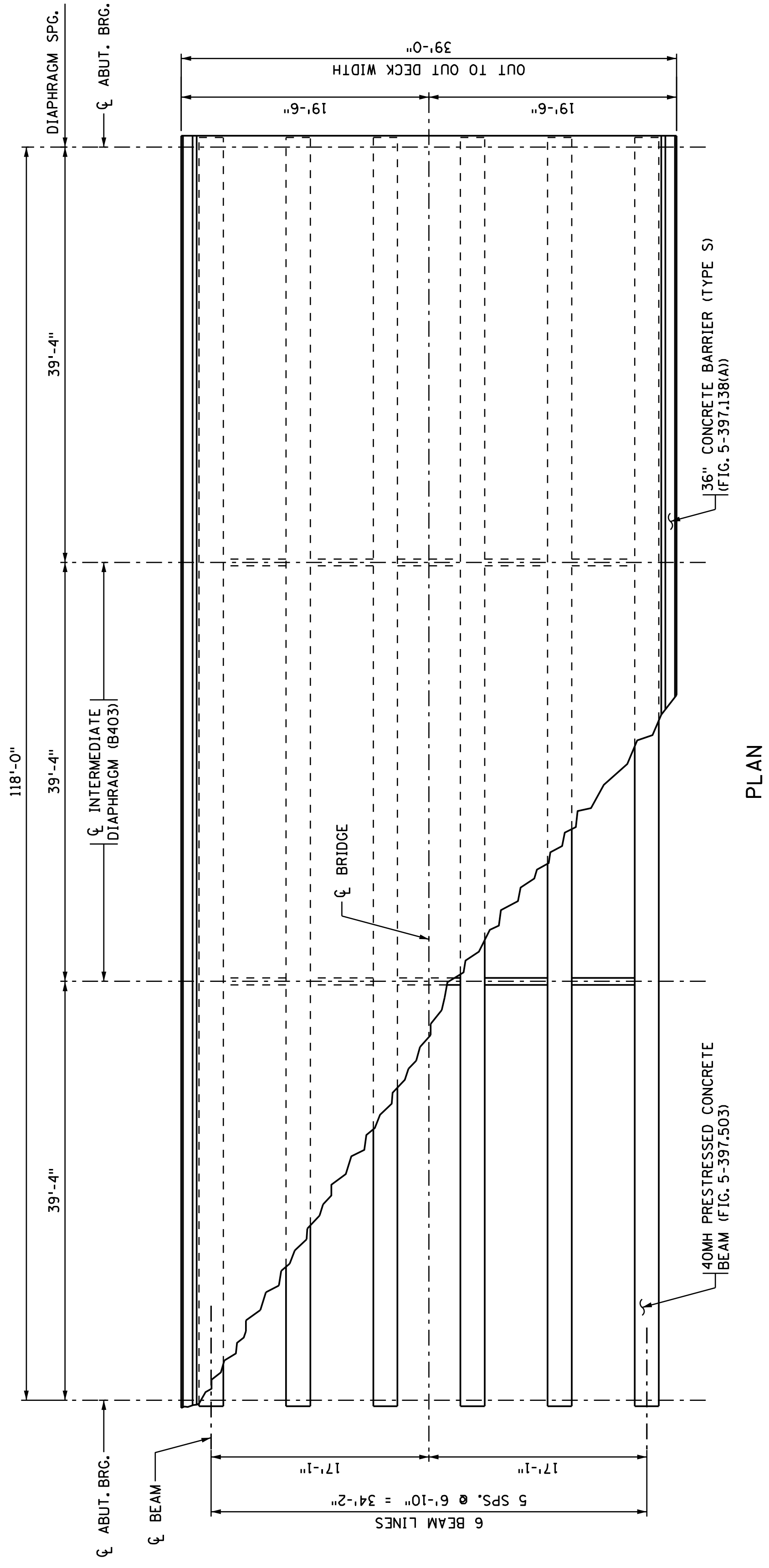


Figure 5.7.2.1



DIAPHRAGM SPG.
℄ ABUT. BRG.
39'-0"
OUT TO OUT DECK WIDTH
19'-6"
19'-6"
36" CONCRETE BARRIER (TYPE S)
(FIG. 5-397.138(A))
118'-0"
39'-4"
39'-4"
39'-4"
℄ INTERMEDIATE
DIAPHRAGM (B403)
℄ BRIDGE
PLAN
40MH PRESTRESSED CONCRETE
BEAM (FIG. 5-397.503)
℄ ABUT. BRG.
℄ BEAM
17'-1"
17'-1"
5 SPS. @ 6'-10" = 34'-2"
6 BEAM LINES

$$= \left(\frac{1773}{704}\right) - \left(\frac{1773 \cdot 13.83}{6794}\right) + \left(\frac{2817 \cdot 12}{6794}\right) + \left(\frac{496 \cdot 12}{42{,}761}\right)$$

$$= 4.02 \text{ ksi} < 4.28 \text{ ksi} \quad \text{OK}$$

Top stress due to fatigue live load plus ½ the sum of prestress and permanent loads

$$= \frac{1}{2}\left(\left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2}}{S_{gtc}}\right)\right) + \left(\frac{M_{LL}}{S_{gtc}}\right)$$

$$= \frac{1}{2}\left(\left(\frac{1773}{704}\right) - \left(\frac{1773 \cdot 13.83}{6794}\right) + \left(\frac{2817 \cdot 12}{6794}\right) + \left(\frac{496 \cdot 12}{42{,}761}\right)\right) + \left(\frac{1008 \cdot 12}{42{,}761}\right)$$

$$= 2.29 \text{ ksi} < 3.80 \text{ ksi} \quad \text{OK}$$

Check the Compression Stresses at End of Beam After Losses

Bottom flange stress at the transfer point due to prestress and permanent loads.

$$= \frac{P_e}{A_g} + \frac{P_e \cdot e_{strand}}{S_{gb}} - \left(\frac{M_{DC1}}{S_{gb}}\right) - \left(\frac{M_{DC2}}{S_{gbc}}\right)$$

$$= \frac{1773}{704} + \frac{1773 \cdot 9.78}{8246} - \left(\frac{222 \cdot 12}{8246}\right) - \left(\frac{39 \cdot 12}{12{,}917}\right)$$

$$= 4.26 \text{ ksi} < 4.28 \text{ ksi} \quad \text{OK}$$

The final concrete strength, f'_c was assumed to be 9.5 ksi. For the most economical beam, the designer should choose the lowest required f'_c for the beam. This is determined by substituting the calculated maximum compression stress for f_{climf1} in the compression limit equation and solving for f'_c.

$$\text{Lowest required } f'_c = \frac{4.26}{0.45} = 9.47 \text{ ksi}$$

The assumed concrete strength cannot be reduced.

Keep $f'_c = 9.5$ ksi

[5.5.4]

5. Flexure – Strength Limit State

Resistance factors at the strength limit state are:

$\phi = 1.00$ for flexure and tension (assumed)

$\phi = 0.90$ for shear and torsion

$\phi = 1.00$ for tension in steel in anchorage zones

Strength I design moment, M_u, is 7498 kip-ft at midspan.

From previous calculations, distance to strand centroid from bottom of the beam at midspan is:

$y_{strand} = 4.24$ in

Similar to Grade 270 strands, the yield strength, f_{py} is taken as $0.9 \cdot f_{pu}$.

$f_{py} = 0.9 \cdot f_{pu} = 0.9 \cdot 300 = 270$ ksi

[5.6.3.1.1]

$$k = 2 \cdot \left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 2 \cdot \left(1.04 - \frac{270}{300}\right) = 0.280$$

d_p = (beam height) + stool + deck - y_{strand}

= 40 + 1.5 + 8.5 − 4.24 = 45.76 in

Begin by assuming the neutral axis lies in the deck.

For $f'_c = 4.0$ ksi, $\beta_1 = 0.85$ and $\alpha_1 = 0.85$.

Then

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_e + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

$$= \frac{10.85 \cdot 300}{0.85 \cdot 4.0 \cdot 0.85 \cdot 82.00 + 0.28 \cdot 10.85 \cdot \left(\frac{300}{45.76}\right)} = 12.67 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 12.67 = 10.77 \text{ in}$$

Compression block depth is greater than the thickness of the slab (8.5 in), so T-section behavior must be considered. The "web width", b_w, of the T-section is the beam flange width, which is 34 in.

Then

$$c = \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_c \cdot (b - b_w) \cdot h_f}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

$$= \frac{10.85 \cdot 300 - 0.85 \cdot 4.0 \cdot (82 - 34) \cdot 8.5}{0.85 \cdot 4.0 \cdot 0.85 \cdot 34 + 0.28 \cdot 10.85 \cdot \left(\frac{300}{45.76}\right)} = 15.81 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 15.81 = 13.44 \text{ in}$$

The revised compression block depth is less than the thickness of the slab plus the flange thickness (15 in), so T-section behavior is confirmed. If the revised compression block depth had been greater than 15 inches, the section would be acting as a stepped T-section and a strain compatibility approach would have been necessary.

$$v_u = \frac{V_u - \phi \cdot V_p}{\phi \cdot b_v \cdot d_v} = \frac{285 - 0.90 \cdot 13.3}{0.90 \cdot 6.5 \cdot 41.7} = 1.12 \text{ ksi}$$

$$v_{ulimit} = 0.125 \cdot f'_c = 0.125 \cdot 9.5 = 1.19 \text{ ksi} > 1.12 \text{ ksi}$$

Then the maximum spacing is the smaller of:

$s_{max} = 0.8 \cdot d_v = 0.8 \cdot 41.7 = 33.0$ in

or $s_{max} = 24$ in GOVERNS

$s_{max} = 24$ in > 12 in OK

Therefore, use double leg #4 stirrups at 12 inch spacing. Other sections are investigated similarly.

[5.7.4]

2. Interface Shear Transfer

The standard beam details require that the outer 6 inches on each edge of the top flange will be smooth with a bond breaker applied, which leaves 22 inches of the top flange to be roughened for engagement of shear transfer.

Then $b_{vi} = 22$ in

The Strength I vertical shear at the critical shear section due to all loads is:

$V_u = 285$ kip

$$v_{ui} = \frac{V_u}{b_{vi} \cdot d_v} = \frac{285}{22 \cdot 41.7} = 0.31 \text{ ksi}$$

Interface shear force is:

$$V_{ui} = v_{ui} \cdot \frac{12 \text{ in}}{\text{ft}} \cdot b_{vi} = 0.31 \cdot 12 \cdot 22 = 81.8 \ \frac{\text{kips}}{\text{ft}}$$

Required nominal interface design shear is:

$$V_{nireg} = \frac{V_{ui}}{\phi_v} = \frac{81.8}{0.90} = 90.9 \ \frac{\text{kips}}{\text{ft}}$$

The interface area per 1 foot length of beam is:

$A_{cv} = 22 \cdot 12 = 264.0$ in^2/ft

[5.7.4.4]

The standard beam details require the top flanges of the beam to be roughened. Then:

$c = 0.28$ ksi $\mu = 1.0$ $K_1 = 0.3$ $K_2 = 1.8$ ksi

The upper limits on nominal interface shear are:

$$K_1 \cdot f'_c \cdot A_{cv} = 0.3 \cdot 4 \cdot 264.0 = 316.8 \text{ kip/ft} > 90.9 \text{ kip/ft} \quad \text{OK}$$

and

$$K_2 \cdot A_{cv} = 1.8 \cdot 264.0 = 475.2 \text{ kip/ft} > 90.9 \text{ kip/ft} \quad \text{OK}$$

The nominal interface shear resistance is:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} \cdot f_y + P_c\right)$$

$$P_c = 0.0 \text{ kip}$$

Substitute and solve for required interface shear steel:

$$A_{vfreq} = \frac{V_{nireq} - c \cdot A_{cv}}{\mu \cdot f_y} = \frac{90.9 - 0.28 \cdot 264.0}{1.0 \cdot 60} = 0.28 \text{ in}^2/\text{ ft}$$

[5.7.4.2] Check minimum interface shear requirements:

The minimum requirement may be waived for girder-slab interfaces with the surface roughened to an amplitude of 0.25 in if the factored interface shear stress is less than 0.210 ksi.

$$v_{ui} = 0.31 \text{ ksi} > 0.210 \text{ ksi}$$

Then the minimum requirement cannot be waived.

The minimum required interface shear reinforcement is the lesser of:

$$A_{vfmin1} = \frac{0.05 \cdot b_v}{f_y} = \frac{0.05 \cdot 22}{60} = 0.018 \text{ in}^2/\text{ in} = 0.22 \text{ in}^2/\text{ ft}$$

or

$$A_{vfmin2} = \frac{1.33 \cdot V_{nireq} - c \cdot A_{cv}}{\mu \cdot f_y} = \frac{1.33 \cdot 90.9 - 0.28 \cdot 264}{1.0 \cdot 60}$$

$$= 0.78 \text{ in}^2/\text{ ft}$$

Then $A_{vfmin} = 0.22 \text{ in}^2/\text{ ft}$

The double leg #4 stirrup at 12” spacing (A_v=0.40 in^2/ft) chosen earlier for vertical shear also meets the requirements for interface shear. Therefore, no additional reinforcement is required for interface shear.

Other sections are investigated similarly.

[5.7.3.5]

3. Minimum Longitudinal Reinforcement Requirement

The longitudinal reinforcement must be checked to ensure it is adequate to carry the tension caused by shear. The amount of strand development must be considered near the end of the beam. There are 2 cases to be checked:

Case 1: From the inside edge of bearing at the end supports out to the critical section for shear, the following must be satisfied, with $A_{ps} \cdot f_{ps}$ modified for development and V_p modified for amount of prestress transfer:

$$A_{ps} \cdot f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

A crack starting at the inside edge of the bearing sole plate will cross the center of gravity of the straight strands at:

$$x_{crack} = L_{soleplate} + y_{sstr} \cdot \cot(\theta) = 15 + 3.71 \cdot \cot(28.0) = 22.0 \text{ in}$$

The transfer length for 0.6" strands is: $\ell_{tr} = 36.0$ in

From the end of the beam to full transfer length, the strand stress increases linearly from zero to f_{pe}. Interpolate to find the tensile capacity of the straight strands at the intersection with the assumed crack:

$$T_{r1} = A_{ps} \cdot f_{pe} \cdot \frac{x_{crack}}{\ell_{tr}} = 42 \cdot 0.217 \cdot 163.4 \cdot \frac{22.0}{36} = 910 \text{ kips}$$

The prestress component in the direction of the shear force must be reduced because $x_{crack} < \ell_{tr}$:

$$V_{pred} = V_p \cdot \frac{x_{crack}}{\ell_{tr}} = 13.3 \cdot \frac{22.0}{36.0} = 8.1 \text{ kips}$$

The tension force to carry is:

$$T_{u1} = \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_{pred}\right) \cdot \cot(\theta)$$

$$= \left(\frac{285}{0.90} - 0.5 \cdot 156.9 - 8.1\right) \cdot \cot(28.0)$$

$$= 432.8 \text{ kips} < 910 \text{ kips} \quad \text{OK}$$

Case 2: At the critical section for shear, the following must be satisfied, with $A_{ps} \cdot f_{ps}$ modified for development:

$$A_{ps} \cdot f_{ps} \geq \frac{M_u}{\phi_f d_v} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

Use values calculated earlier to determine the tensile capacity at the critical section for shear:

f_{ps}=278.7 ksi

A_{ps}= 9.11 in^2

Development fraction, F_{dev}= 0.65

$T_{r2} = A_{ps} \cdot f_{ps} \cdot F_{dev} = 9.11 \cdot 278.7 \cdot 0.65 = 1650$ kips

The factored moment M_u should be the moment concurrent with the factored shear V_u at x_{vcrit}. For simplicity, the maximum M_u at x_{vcrit} is used below.

Modification of V_p is not required because $x_{vcrit} > \ell_{tr}$.

Then the tension force to carry is:

$$T_{u2} = \frac{M_u}{\varphi_f d_v} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

$$= \frac{1025 \cdot 12}{1.0 \cdot 41.7} + \left(\frac{285}{0.9} - 0.5 \cdot 156.9 - 13.3\right) \cdot \cot(28.0)$$

T_{u2} = 718.0 kips < 1650 kips OK

G. Design Pretensioned Anchorage Zone Reinforcement [5.9.4.4.1]

Splitting Reinforcement

To prevent cracking in the beam end due to the transfer of the prestressing force from the strands to the concrete, splitting reinforcement needs to be provided in the anchorage zone.

Use a load factor of 1.0 and lateral force component of 4% to determine the required amount of steel.

The total prestressing force at transfer:

$$P_i = A_{ps} \cdot (f_{pj} - \Delta f_{pES}) = 10.85 \cdot (216.0 - 25.6) = 2066 \text{ kips}$$

The factored design splitting force is:

$$P_{split} = 1.0 \cdot 0.04 \cdot P_i = 1.0 \cdot 0.04 \cdot 2066 = 82.6 \text{ kips}$$

The amount of resisting reinforcement is determined using a steel stress f_s of 20 ksi:

$$A_s = \frac{P_{split}}{f_s} = \frac{82.6}{20} = 4.13 \text{ in}^2$$

This steel should be located at the end of the beam within a distance of:

$$\frac{h}{4} = \frac{40}{4} = 10 \text{ in}$$

The number of #5 double legged stirrups necessary to provide this area is:

$$\frac{A_s}{2 \cdot A_b} = \frac{4.13}{2 \cdot 0.31} = 6.7$$

The first set of stirrups (G505E) is located 2 inches from the end of the beam. See Figure 5.7.2.5.

Provide an additional six sets of #5 stirrups (G508E) spaced at 2 1/2 inch centers.

$x_{splitting} = 2 + 6 \cdot 2.5 = 17.0 \text{ in} > 10 \text{ in}$

Although the splitting reinforcement does not fit within h/4, #5 bars are the largest allowed and 2.5 inches is the tightest spacing allowed. This is OK per MnDOT practice.

[5.9.4.4.2]

Confinement Reinforcement

Reinforcement is required at the ends of the beam to confine the prestressing steel in the bottom flange. G303E bars (see Figure 5.7.2.5) will be placed at a maximum spacing of 6 inches out to 1.5d from the ends of the beam. For simplicity in detailing and ease of tying the reinforcement, space the vertical shear reinforcement with the confinement reinforcement in this area.

$$1.5\ d = 1.5 \cdot 40 = 60.0 \text{ in}$$

H. Determine Camber and Deflection

[2.5.2.6.2]
[3.6.1.3.2]
[5.6.3.5.2]

Camber Due to Prestressing and Dead Load Deflection

Using the PCI handbook (Figure 4.10.13 of the 3rd Edition), the camber due to prestress can be found. The centroid of the prestressing has an eccentricity e_{mid} of 13.83 inches at midspan. At the end of the beams the eccentricity e_e is 9.51 inches. E is the initial concrete modulus (4578 ksi), P_o equals the prestress force just after transfer (2066 kips). The drape points are at 0.4 of the design span, which is 118.0 feet. The span length at release is the end-to-end length of the 119.25 feet since the beam is in the casting bed. Using the equation for the two-point depressed strand pattern:

$$e' = e_{mid} - e_e = 13.83 - 9.51 = 4.32 \text{ in}$$

$$\Delta_{ps} = \frac{P_o \cdot e_e \cdot L^2}{8 \cdot E \cdot I} + \frac{P_o \cdot e'}{E \cdot I}\left(\frac{L^2}{8} - \frac{a^2}{6}\right)$$

$$= \frac{2066 \cdot 9.51 \cdot (119.25 \cdot 12)^2}{8 \cdot 4578 \cdot 149{,}002}$$

$$+ \frac{2066 \cdot 4.32}{4578 \cdot 149{,}002}\left[\frac{(119.25 \cdot 12)^2}{8} - \frac{(0.4 \cdot 118 \cdot 12 + 7.50)^2}{6}\right]$$

$$= 10.00 \text{ in}$$

Downward deflection due to selfweight

$$\Delta_{sw} = \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I} = \frac{5 \cdot \frac{0.758}{12}(119.25 \cdot 12)^4}{384 \cdot 4578 \cdot 149{,}002} = 5.06 \text{ in}$$

Camber at release $\Delta_{rel} = \Delta_{ps} - \Delta_{sw} = 10.00 - 5.06 = 4.94$ in

To estimate camber at the time of erection the deflection components are multiplied by standard MnDOT multipliers. They are:

Release to Erection Multipliers:

Prestress = 1.4

Selfweight = 1.4

Camber and selfweight deflection values at erection are:

Prestress:	$1.4 \cdot 10.00 =$	14.00 in
Selfweight:	$1.4 \cdot (-5.06) =$	-7.08 in
Diaphragm DL:		-0.02 in
Deck and stool DL:		-5.12 in
Barrier:		-0.37 in

Note that the deflection values for diaphragms, deck, stool, and barrier are based on a span length of 118.0 feet.

The values to be placed in the camber diagram on the beam plan sheet are arrived at by combining the values above.

"Erection Camber" = 14.00 – 7.08 – 0.02 = 6.90 in   say 6 7/8 in

"Est. Dead Load Deflection" = 5.12 + 0.37 = 5.49 in   say 5 1/2 in

"Est. Residual Camber" = 6 7/8 – 5 1/2 = 1 3/8 in

Live Load Deflection

The deflection of the bridge is checked when subjected to live load and compared against the limiting values of $^{L}/_{800}$ for vehicle only bridges and $^{L}/_{1000}$ for bridges with bicycle or pedestrian traffic.

Deflection due to lane load is:

$$\Delta_{lane} = \left(\frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I}\right) = \left[\frac{5 \cdot \frac{0.64}{12} \cdot (118 \cdot 12)^4}{384 \cdot 4899 \cdot 396{,}823}\right] = 1.44 \text{ in}$$

Deflection due to a truck with dynamic load allowance is found using hand computations or computer tools to be:

$$\Delta_{truck} = 2.81 \text{ in}$$

Two deflections are computed and compared to the limiting values, that of the truck alone and that of the lane load plus 25% of the truck. Both deflections need to be adjusted with the live load distribution factor for deflection.

$$\Delta_1 = DF_\Delta \cdot \Delta_{truck} = 0.425 \cdot 2.81 = 1.19 \text{ in}$$

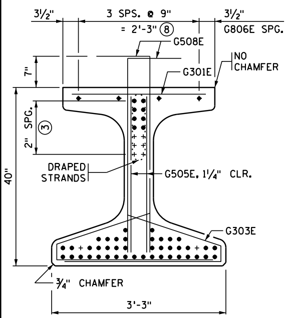
$$\Delta_2 = DF_\Delta \cdot (\Delta_{lane} + 0.25 \cdot \Delta_{truck}) = 0.425 \cdot (1.44 + 0.25 \cdot 2.81) = 0.91 \text{ in}$$

There is no bicycle or pedestrian traffic on the bridge, so the deflection limit is:

$$\frac{L}{800} = \frac{118 \cdot 12}{800} = 1.77 \text{ in} > \text{ than } \Delta_1 \text{ or } \Delta_2 \quad \text{OK}$$

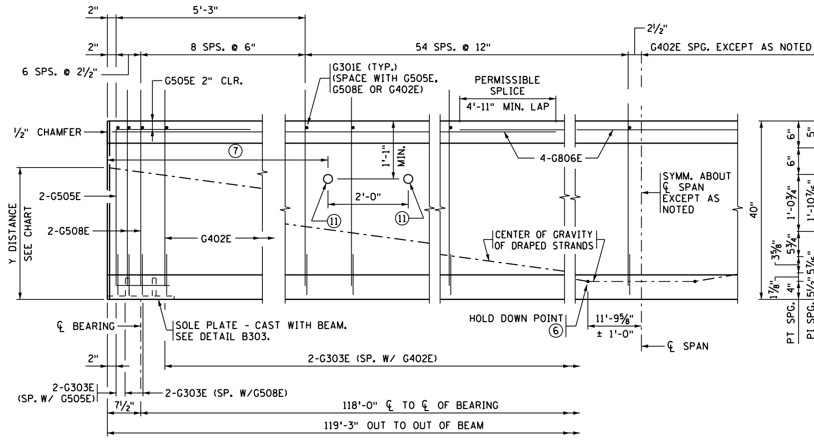
I. Beam Sheet for Bridge Plan

Figure 5.7.2.5 shows the detailed beam sheet for the draped strand configuration that will be included in the bridge plan.

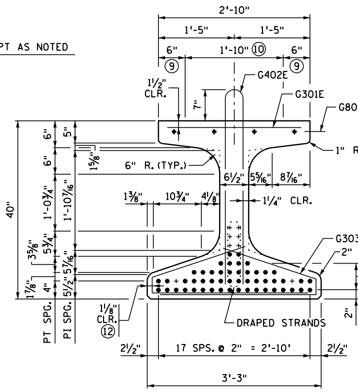


END VIEW

CUT STRANDS FLUSH WITH CONCRETE. COVER ENDS WITH SEALANT PER APPROVED PRODUCTS LIST "BRIDGE-PRESTRESSED BEAMS-CUT STRAND SEALANT."



BEAM ELEVATION



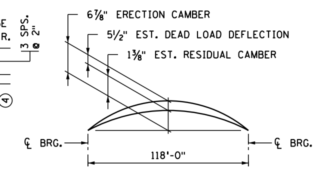
SECTION AT ℄ SPAN

Y DISTANCES (INCHES)	NO.	℄ SPAN	END
STRAIGHT STRANDS	42	3.71	
DRAPED STRANDS	8	7.00	34.00 ¤
⑤ TOTAL STRANDS	50	4.24	

Y = DISTANCE TO CENTER OF GRAVITY OF STRANDS FROM BOTTOM OF BEAM. ALL STRANDS SPACED 2" CENTER TO CENTER, HORIZONTALLY AND VERTICALLY, EXCEPT AS NOTED.

2 OPTIONAL ½" MIN. DIA. STRAIGHT STRANDS ARE NOT INCLUDED IN THIS TABLE.

¤ A TOLERANCE OF ± 1" WILL BE PERMITTED IN THIS DIMENSION.



CAMBER DIAGRAM

ERECTION CAMBER SHOWN IS AFTER DIAPHRAGMS ARE IN PLACE.

DEAD LOAD DEFLECTION SHOWN IS FOR WEIGHT OF SLAB, WEARING COURSE, BARRIER, SIDEWALK AND MEDIAN WHERE APPLICABLE.

CONTRACTOR WILL TAKE ELEVATIONS AT TOP OF BEAMS AFTER ERECTION AND WILL ALLOW FOR DEFLECTION SHOWN TO ENABLE BUILDING FORMS TO CORRECT GRADE AND SPECIFIED SLAB THICKNESS. PROVIDE COPY OF ELEVATIONS TO THE ENGINEER.

INITIAL PRESTRESS
2,343,600 LB.
2 OPTIONAL ½" MIN. DIA. STRAIGHT TOP STRANDS ARE NOT INCLUDED IN THIS FORCE.
☒ 0.72 f_{pu}
☐ 0.75 f_{pu}

CALCULATED PRESTRESS LOSSES	
ELASTIC SHORTENING LOSS	25.6 KSI
LONG TERM LOSSES	27.0 KSI
TOTAL LOSSES	52.6 KSI

MINIMUM CONCRETE STRENGTH - KSI	
① f'ci	② f'c
8.0 KSI	9.5 KSI

GENERAL NOTES

PROVIDE HANDLING HOOKS OR DEVICES AS REQUIRED BY CONTRACTOR.

AT THE CONTRACTORS OPTION, 2 SYMMETRIC ½" MIN. DIA. STRANDS PULLED UP TO 5 KIPS EACH MAY BE ADDED TO THE TOP FLANGE. MAINTAIN THE VERTICAL PROFILE OF THESE STRANDS WITH A TOLERANCE OF ± 1" ALONG THE LENGTH OF THE BEAM AND MAINTAIN A MIN. OF 1¼" CLEAR COVER.

MARK EACH BEAM SHOWING BRIDGE NUMBER, CASTING DATE, AND INDIVIDUAL IDENTIFICATION LETTERS AND NUMBERS ON THE FACE OF THE BEAM, NEAR THE END, SO LOCATED THAT THEY WILL BE EXPOSED AFTER THE END DIAPHRAGMS HAVE BEEN CAST. MARK FASCIA BEAMS ON THE INSIDE FACE. ENSURE ALL MARKINGS ARE STENCILLED AND CLEARLY LEGIBLE. FOR LOCATION OF BEAMS, SEE FRAMING PLAN.

ALL MATERIAL AND WORK SHOWN OR NOTED ON THIS SHEET IS INCLUDED IN UNIT PRICE BID FOR PRESTRESSED CONCRETE BEAMS. SEE SPEC. 2405.

SEE FRAMING PLAN FOR BEAM END MARKED "X" AND DIAPHRAGM SPACING.

REFER TO "PRESTRESSED CONCRETE BEAM DIAPHRAGM CONNECTION DETAILS" SHEET INCLUDED IN THIS PLAN SET FOR CONCRETE END DIAPHRAGM AND STEEL INTERMEDIATE DIAPHRAGM DETAILS.

APPLY AN APPROVED SEALER TO THE SIDES OF THE BEAM NEAR EACH END PER THE SPECIAL PROVISIONS.

① MINIMUM CONCRETE STRENGTH AT TIME OF PRESTRESS TRANSFER.

② MINIMUM CONCRETE STRENGTH WHEN BEAM CAN BE TRANSPORTED AND INSTALLED.

③ DRAPED STRANDS.

④ STRAIGHT STRANDS.

⑤ USE MNDOT GRADE 300, 0.6" DIA. 7-WIRE LOW RELAXATION PRESTRESSING STRAND, IN ACCORDANCE WITH ASTM A416 AS MODIFIED IN THE SPECIAL PROVISIONS.

⑥ CENTER OF GRAVITY OF HOLD DOWNS WHEN MULTIPLE HOLD DOWNS ARE USED.

⑦ DIMENSION DETERMINED BY CONTRACTOR. MAINTAIN 2" MINIMUM CLEAR FROM STRANDS.

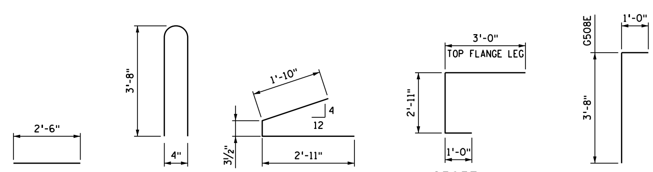
⑧ TWO INSIDE BARS MAY BE PLACED ADJACENT TO VERTICAL STIRRUP FOR TYING CONVENIENCE.

⑨ STEEL TROWEL TO SMOOTH FINISH AND APPLY BOND BREAKER PER APPROVED PRODUCTS LIST.

⑩ ROUGH FLOAT AND BROOM TRANSVERSELY FOR BOND PER SPEC. 2405.3D.

⑪ OPTIONAL: 2" MAX. DIA. SLEEVE FOR HAULING (AFTER INSTALLATION, COAT WITH APPROVED EPOXY BONDING AGENT & FILL WITH APPROVED NON-SHRINK GROUT).

⑫ TYP. CLR. DISTANCE FOR ENTIRE BOTTOM FLANGE.



G301E G402E G303E G505E G508E

REVISED: DECEMBER 21, 2022

APPROVED: DECEMBER 20, 2018

Kevin Western
STATE BRIDGE ENGINEER

REV. NO.	DATE	REVISION DESCRIPTION	BY

I HEREBY CERTIFY THAT THIS PLAN WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA

CERTIFIED BY ______ LICENSED PROFESSIONAL ENGINEER ______ DATE

NAME: ______ LIC. NO. ______

TITLE: 40MH PRESTRESSED CONCRETE BEAM (PRETENSIONED) 40MH-120

BEAMS B1 - B2

FIG. 5-397.503

DES: DR: CHK: CHK: APPROVED:

SHEET NO. OF SHEETS

BRIDGE NO. EXAMPLE

Figure 5.7.2.5

5.7.3 Debonded Prestressed I-Beam Design Example

This example illustrates the design of a pretensioned I-beam for a single span bridge without skew. The 118'-0" span is supported with MnDOT "40MH" beams on integral abutments. MnDOT standard details and drawings for diaphragms (B403), barriers (Fig. 5-397.138(A)), and beams (Fig. 5-397.503) are to be used with this example. This example contains the design of a typical interior beam at the critical sections in positive flexure, shear, and deflection. The superstructure consists of six beams spaced at 6'-10" centers. A typical transverse superstructure section is provided in Figure 5.7.3.1. A framing plan is provided in Figure 5.7.3.2. The roadway section is composed of two 12' traffic lanes and two 6' shoulders. A Type S barrier is provided on each side of the bridge and a 9" monolithic concrete deck is used. Interior diaphragms are used at the interior third points based on guidance found in BDM Table 5.4.1.1.

This example uses 0.6" diameter, 300 ksi, low relaxation strands for pretensioning. Debonded strands are used to control the beam end stresses.

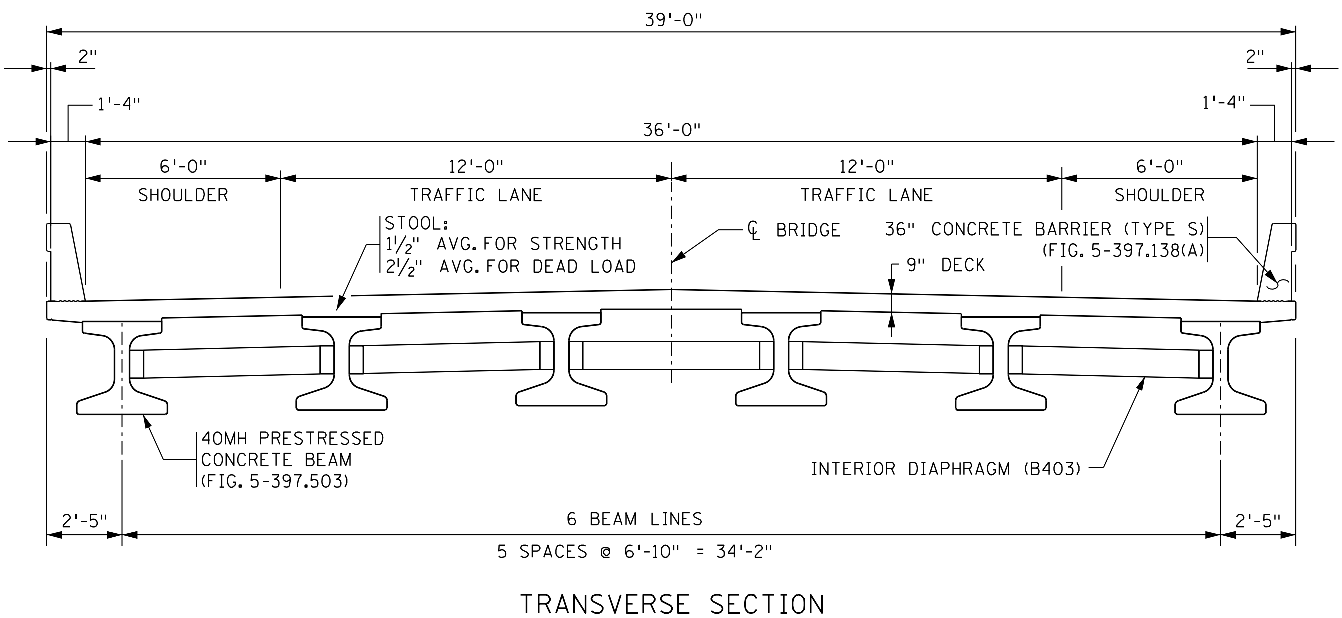
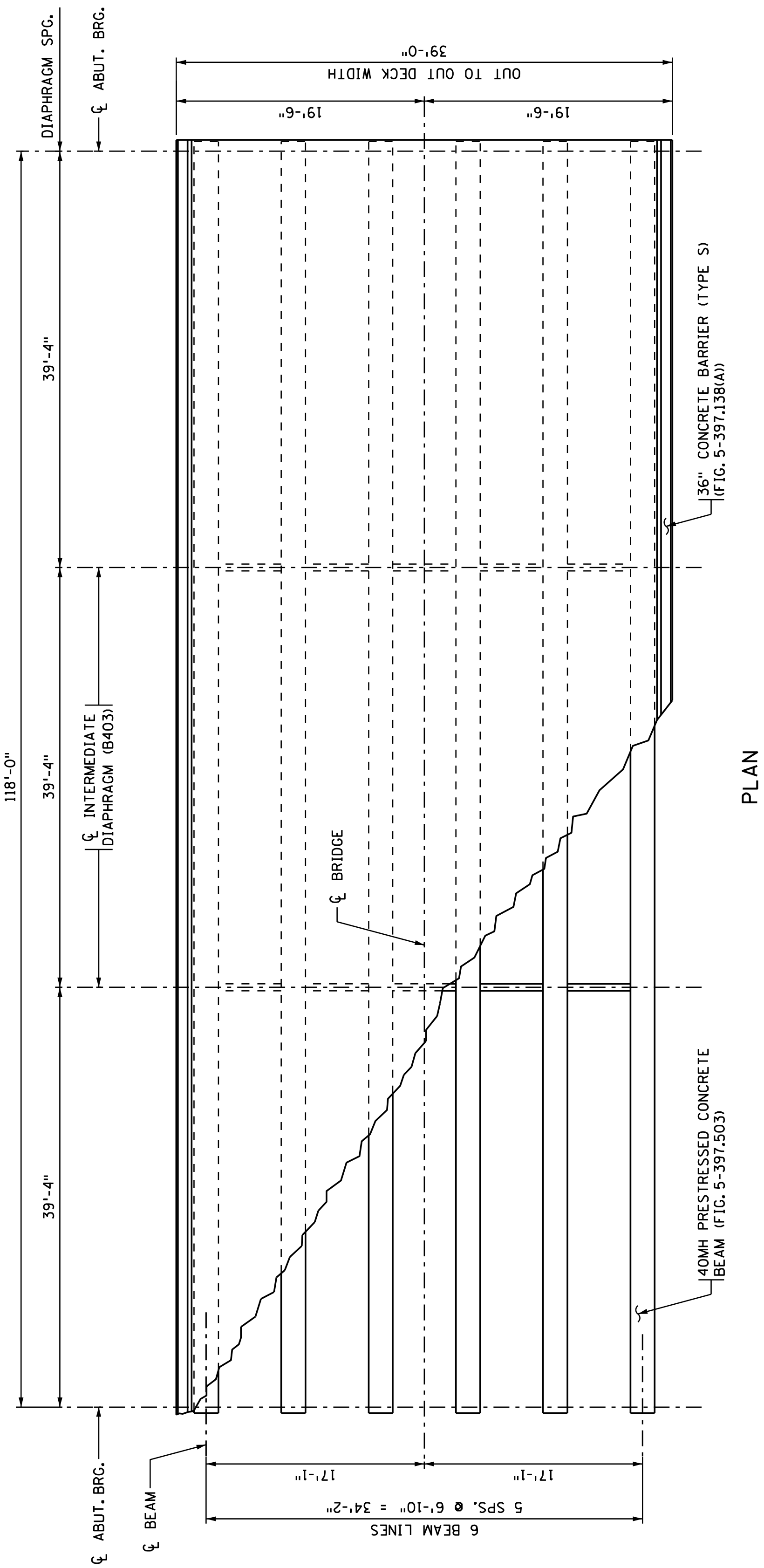


Figure 5.7.3.1



OUT TO OUT DECK WIDTH
39'-0"
19'-6"
19'-6"
DIAPHRAGM SPG.
℄ ABUT. BRG.
118'-0"
39'-4"
39'-4"
39'-4"
℄ INTERMEDIATE DIAPHRAGM (B403)
℄ BRIDGE
36" CONCRETE BARRIER (TYPE S) (FIG. 5-397.138(A))
40MH PRESTRESSED CONCRETE BEAM (FIG. 5-397.503)
PLAN
℄ ABUT. BRG.
℄ BEAM
17'-1"
17'-1"
5 SPS. @ 6'-10" = 34'-2"
6 BEAM LINES

Figure 5.7.3.2

A. Materials

The modulus of elasticity for high strength concrete suggested by ACI Committee 363 is used for the beam concrete. AASHTO Article 5.4.2.4 is used to calculate E_c for the deck and assumes a K_1 equal to 1.0. The composite deck is assumed to have a unit weight of 0.150 kcf for dead load computations and 0.145 kcf for E_c computations. The beam concrete is assumed to have a unit weight of 0.155 kcf for dead load computations.

The material and geometric parameters used in the example are shown in Table 5.7.3.1:

Table 5.7.3.1
Material Properties

Material Parameter		Prestressed Beam	Deck
Concrete	f'_{ci} at transfer	8.0 ksi *	---
	f'_c at 28 days	9.5 ksi *	4 ksi
	E_{ci} at transfer	$(1265 \cdot \sqrt{f'_{ci}}) + 1000$ = 4578 ksi	---
	E_c at 28 days	$(1265 \cdot \sqrt{f'_c}) + 1000$ = 4899 ksi	$120{,}000 \cdot K_1 \cdot (w_c)^2 \cdot (f'_c)^{0.33}$ = 3987 ksi
Steel	f_y for rebar	60 ksi	60 ksi
	f_{pu} for strand	300 ksi	---
	E_s for rebar	29,000 ksi	29,000 ksi
	E_p for strand	28,500 ksi	---
	Strand type	0.6 inch diameter 300 ksi, low relaxation	---

*These concrete compressive strength values are initial assumed values. Final values may differ based on adjustments for the actual and initial final service stresses.

B. Determine Cross-Section Properties for a Typical Interior Beam

The beams are designed to act compositely with the deck on simple spans. The deck consists of a 9 inch thick concrete slab. A $^1/_2$ inch of wear is assumed. A thickness of $8^1/_2$ inches is used for composite section properties. The stool height is assumed to be an average of $2^1/_2$ inches for dead load computations and $1^1/_2$ inches for section property computations.

[4.6.2.6.1]

The effective flange width, b_e, is equal to the average beam spacing:

$b_e = 82.00$ in

To transform the deck and stool concrete to beam concrete, use a modular ratio n_{d_bm} based on E_{cdeck} to E_{cbeam}:

$$n_{d\_bm} = \frac{E_{cdeck}}{E_{cbeam}} = \frac{3987}{4899} = 0.81$$

This results in a transformed effective flange width of:

$$b_{etrans} = n_{d\_bm} \cdot b_e = 0.81 \cdot 82 = 66.42 \text{ in}$$

Properties for an interior beam are given in Table 5.7.3.2.

Table 5.7.3.2
Cross-Section Properties

Parameter	Non-composite Section	Composite Section
Height of section, h	40.00 in	50.00 in
Deck thickness	---	8.50 in
Average stool thickness	---	1.50 in (section properties) 2.50 in (dead load)
Effective flange width, b_e	---	82.00 in (deck concrete) 66.42 in (beam concrete)
Area, A	704 in^2	1310 in^2
Moment of inertia, I	149,002 in^4	396,823 in^4
Centroidal axis height, y	18.07 in	30.72 in
Bottom section modulus, S_b	8246 in^3	12,917 in^3
Top section modulus, S_t	6794 in^3	25,410 in^3
Top of prestressed beam, S_{tbm}	6794 in^3	42,761 in^3

C. Live Load Distribution Factors and Load Modifiers

Assume that traffic can be positioned anywhere between the barriers.

$$\text{Number of design lanes} = \frac{\text{distance between barriers}}{\text{design lane width}} = \frac{36}{12} = 3$$

[4.6.2.2]

1. Determine Live Load Distribution Factors

Designers should note that the approximate live load distribution factor equations include the multiple presence factors.

[4.6.2.2.2]

Live Load Distribution Factor for Moment – Interior Beams

LRFD Table 4.6.2.2.1-1 lists the common deck superstructure types for which approximate live load distribution equations have been assembled. The cross section for this design example is Type (k). To ensure that the approximate distribution equations can be used, several parameters need to be checked.

1) 3.5 ft ≤ beam spacing = 6.83 ft ≤ 16.0 ft OK
2) 4.5 in ≤ slab thickness = 8.5 in ≤ 12.0 in OK
3) 20 ft ≤ span length = 118 ft ≤ 240 ft OK
4) 4 ≤ number of beams = 6 OK

The live load distribution factor equations use a K_g factor that is defined in LRFD Article 4.6.2.2.1. For determination of K_g, the beam concrete is transformed to deck concrete, so the modular ratio n_{bm_d} differs from n_{d_bm} calculated earlier.

$$n_{bm\_d} = \frac{E_{cbeam}}{E_{cdeck}} = \frac{4899}{3987} = 1.23$$

$$e_g = \text{(deck centroid)} - \text{(beam centroid)} = 45.75 - 18.07 = 27.68 \text{ in}$$

$$K_g = n_{bm\_d} \cdot \left[I + A \cdot (e_g)^2\right] = 1.23 \cdot \left[149{,}002 + 704 \cdot (27.68)^2\right]$$

$$= 8.47 \text{ x } 10^5 \text{ in}^4$$

Check K_g limits: $1 \text{ x } 10^4 \le K_g = 8.47 \text{ x } 10^5 \le 7 \text{ x } 10^6$ OK

For one design lane loaded, the live load distribution factor for moment, gM_{int_1lane}, is:

$$gM_{int\_1lane} = 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K}{12 \cdot L \cdot t_s^3}\right)^{0.1}$$

$$gM_{int\_1lane} = 0.06 + \left(\frac{6.83}{14}\right)^{0.4} \cdot \left(\frac{6.83}{118}\right)^{0.3} \cdot \left(\frac{8.47 \text{ x } 10^5}{12 \cdot 118 \cdot 8.5^3}\right)^{0.1}$$

$$gM_{int\_1lane} = 0.378 \text{ lanes/beam}$$

Two or more design lanes loaded:

$$gM_{int\_mlane} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K}{12 \cdot L \cdot t_s^3}\right)^{0.1}$$

$$gM_{int\_mlane} = 0.075 + \left(\frac{6.83}{9.5}\right)^{0.6} \cdot \left(\frac{6.83}{118}\right)^{0.2} \cdot \left(\frac{8.47 \text{ x } 10^5}{12 \cdot 118 \cdot 8.5^3}\right)^{0.1}$$

$$gM_{int\_mlane} = 0.538 \text{ lanes/beam}$$

[4.6.2.2.2d]

Live Load Distribution Factor for Moment - Exterior Beams

LRFD Table 4.6.2.2.2d-1 contains the approximate live load distribution factor equations for exterior beams. Type (k) cross-sections have a deck dimension check to ensure that the approximate equations are valid. The distance from the inside face of barrier to the centerline of the fascia beam is defined as d_e. For the example this distance is:

$$d_e = \text{deck overhang - deck coping - barrier width}$$

$$= \frac{(29 - 2 - 16)}{12} = 0.92 \text{ ft}$$

Check whether approximate equations can be used:

$-1.0 \text{ ft} \le d_e = 0.92 \text{ ft} \le 5.5 \text{ ft}$ OK

One design lane loaded:

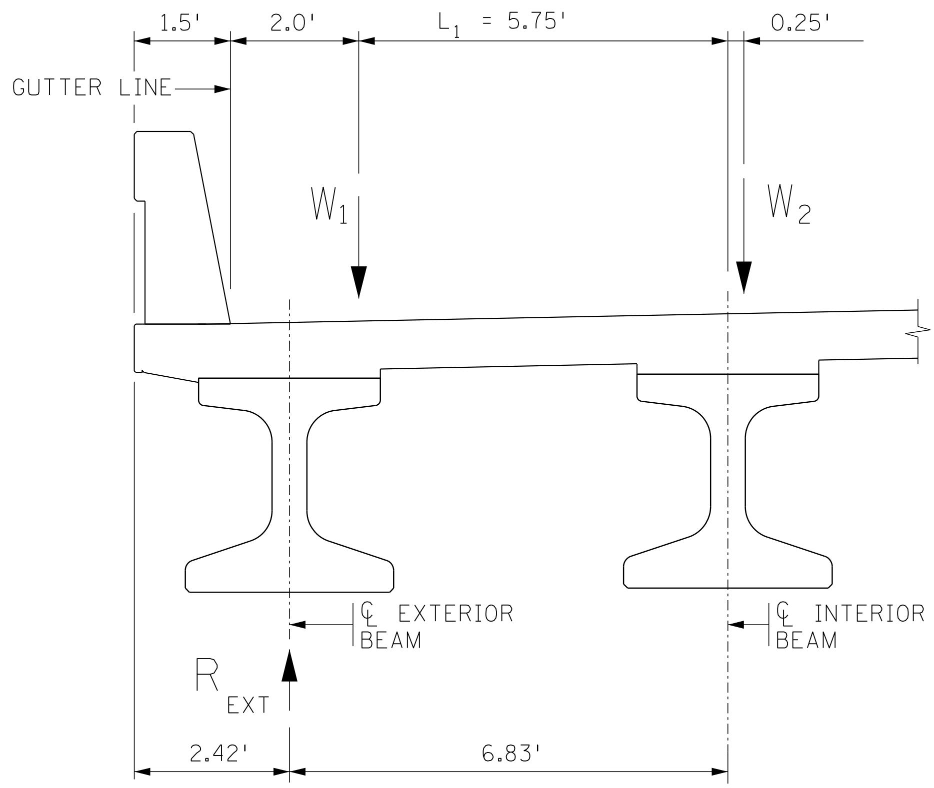


Figure 5.7.3.3

[Table 3.6.1.1.2-1]

Use the lever rule to determine the live load distribution factor for one lane. The exterior beam live load distribution factor is found by determining the exterior beam reaction and applying the multiple presence factor, m, for one lane:

$W_1 = W_2 = 0.5$ lanes

$$gM_{ext\_1lane} = R_{ext} \cdot m = \left(\frac{W_1 \cdot L_1}{S}\right) \cdot m = \left(\frac{0.5 \cdot 5.75}{6.83}\right) \cdot 1.20$$

$gM_{ext\_1lane} = 0.505$ lanes/beam

Two or more design lanes loaded:
The live load distribution factor is equal to the factor "e" multiplied by the interior girder live load distribution factor for two or more lanes.

$$e = 0.77 + \left(\frac{d_e}{9.1}\right) = 0.77 + \left(\frac{0.92}{9.1}\right) = 0.871$$

$$gM_{ext\_mlane} = e \cdot gM_{int\_mlane} = 0.871 \cdot 0.538 = 0.469 \text{ lanes/beam}$$

[4.6.2.2.2e] **Skew Factor**

No correction is necessary for a skew angle of zero.

[4.6.2.2.3] **Live Load Distribution Factor for Shear – Interior Beams**

[4.6.2.2.3a] LRFD Table 4.6.2.2.3a-1 can be used.

One design lane loaded:

$$gV_{int\_1lane} = 0.36 + \left(\frac{S}{25.0}\right) = 0.36 + \left(\frac{6.83}{25}\right) = 0.633 \text{ lanes/beam}$$

Two or more design lanes loaded:

$$gV_{int\_mlane} = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2 = 0.2 + \left(\frac{6.83}{12}\right) - \left(\frac{6.83}{35}\right)^2$$

$$= 0.731 \text{ lanes/beam}$$

[4.6.2.2.3b] **Live Load Distribution Factor for Shear – Exterior Beams**

One Design Lane Loaded:

Use the lever rule, which results in the same factor that was computed for flexure and is equal to 0.505 lanes/beam

Two or more design lanes loaded:

$$e = 0.6 + \left(\frac{d_e}{10}\right) = 0.6 + \left(\frac{0.92}{10}\right) = 0.692$$

The exterior beam shear live load distribution factor for two or more design lanes is determined by modifying the interior distribution factor:

$$gV_{ext\_mlane} = e \cdot gV_{int\_mlane} = 0.692 \cdot 0.731 = 0.506 \text{ lanes/beam}$$

[4.6.2.2.3c] **Skew Factor**

No correction is necessary for a skew angle of zero.

[2.5.2.6.2] **Live Load Distribution Factor for Deflection**

[Table 3.6.1.1.2-1] The live load distribution factor for checking live load deflections assumes that the entire cross section participates in resisting the live load. The deflection live load distribution factor is:

$$gD = \frac{(\#\text{ of lanes}) \cdot m}{(\#\text{ of beam lines})} = \frac{3 \cdot 0.85}{6} = 0.425 \text{ lanes/beam}$$

Live load Distribution Factor for Fatigue – Interior and Exterior Beams

[3.6.1.1.2] The fatigue limit state is to be analyzed for one traffic lane, but the multi-presence factor does not apply. The live load distribution factor for one lane is to be divided by 1.2 to account for this.

Interior Beam:

$$gF_{int\_1lane} = \frac{gM_{int\_1lane}}{1.2} = \frac{0.378}{1.2} = 0.315 \text{ lanes/beam}$$

Exterior Beam:

$$gF_{ext\_1lane} = \frac{gM_{ext\_1lane}}{1.2} = \frac{0.505}{1.2} = 0.421 \text{ lanes/beam}$$

Table 5.7.3.3 contains a summary of the live load distribution factors and Table 5.7.3.4 contains a summary of the load modifiers for this example.

Table 5.7.3.3
Live Load Distribution Factor Summary (lanes per beam)

Loading		Flexure	Shear	Deflection	Fatigue
Interior Beam	One Design Lane	0.378	0.633	-	0.315
	Two or More Design Lanes	0.538	0.731	0.425	-
Exterior Beam	One Design Lane	0.505	0.505	-	0.421
	Two or More Design Lanes	0.469	0.506	0.425	-

[1.3.3 – 1.3.5] **Table 5.7.3.4 Load Modifiers**

Modifier	Strength	Service	Fatigue
Ductility η_D	1.0	1.0	1.0
Redundancy η_R	1.0	1.0	1.0
Importance η_I	1.0	n/a	n/a
$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0	1.0

D. Shear Forces and Bending Moments

Four load combinations will be considered: Strength I, Service I, Service III, and Fatigue. As a result of the simple span configuration, only maximum γ_p values need to be considered.

Load effects related to settlement, thermal effects, water load, or stream pressure will not be considered.

[3.6.2]

Dynamic load allowance IM = 33%

$$\text{Beam Selfweight} = (704/144) \cdot (0.155\ \text{k/ft}^3) = 0.758\ \text{k/ft}$$

$$\text{Stool Weight} = (2.83\ \text{ft}) \cdot (0.208\ \text{ft}) \cdot (0.150\ \text{k/ft}^3) = 0.088\ \text{k/ft}$$

$$\text{Deck Weight} = (6.83\ \text{ft}) \cdot (0.75\ \text{ft}) \cdot (0.150\ \text{k/ft}^3) = 0.769\ \text{k/ft}$$

$$\text{Future Wearing Surface} = (0.020\ \text{k/ft}^2) \cdot (36\ \text{ft}) \cdot (1/6) = 0.120\ \text{k/ft}$$

$$\text{Barrier Weight} = 2 \cdot (0.496\ \text{k/ft}) \cdot (1/6) = 0.165\ \text{k/ft}$$

The load due to the intermediate diaphragms is calculated by referring to standard detail B403. For 40MH beams, the diaphragm consists of a steel C12 x 20.7 that is connected to the beams with 1.0′ x 1.0′ bent plates.

$$\text{Diaphragm Weight} \cong (6.83\ \text{ft}) \cdot (0.0207\ \text{k/ft}) + 2 \cdot (1.0\ \text{ft}) \cdot (1.0\ \text{ft}) \cdot \left(\frac{0.375\ \text{in}}{12\ \text{in/ft}}\right) \cdot \left(0.490\ \text{k/ft}^3\right) = 0.172\ \text{kips}$$

Critical locations along the beam need to be analyzed for moments and shear. These critical locations include: the inside face of bearing, prestress transfer points, critical shear point, and tenth points along the length of the beam. These locations, dimensioned from the beam centerline of bearing, are determined as follows:

Bearing Face (inside face of bearing is the point where a crack could start at the bottom of the beam, which is the inside edge of the sole plate)

$$= X_{brgface} = \frac{L_{soleplate}}{2} = 7.5\ \text{in} = 0.63\ \text{ft}$$

Initial Transfer Point

$$= X_{transfer1} = 60 \cdot d_b - \frac{L_{soleplate}}{2} = (60 \cdot 0.6) - \frac{15}{2} = 28.5\ \text{in} = 2.38\ \text{ft}$$

Bottom Flange Debonding Transfer Points (debonding limitations and the general guidance that helped establish these locations are discussed later in this example)

$$= X_{transfer2} = X_{transfer1} + 14 = 16.4\ \text{ft}$$

$$= X_{transfer3} = X_{transfer1} + 18 = 20.4\ \text{ft}$$

$$= X_{transfer4} = X_{transfer1} + 22 = 24.4\ \text{ft}$$

Critical Shear Point (located at d_v from the inside face of bearing, calculations are shown in "F. Design Reinforcement for Shear")

$= X_{vcrit} = 4.19$ ft

Tenth points are simply 0.1L, 0.2L, 0.3L, 0.4L, and 0.5L

The bending moments and shears for the dead and live loads were obtained with a line girder model of the bridge. They are summarized in Tables 5.7.3.5 and 5.7.3.6.

Table 5.7.3.5
Shear Force Summary (kips/beam)

Load Type/Combination		Brg CL (0.0')	Brg Face (0.63')	Trans Point #1 (2.38')	Critical Shear Point (4.2')	0.1 Span Point (11.8')	Trans Point #2 (16.4')	Trans Point #3 (20.4')	0.2 Span Point (23.6')	Trans Point #4 (24.4')	0.3 Span Point (35.4')	0.4 Span Point (47.2')	0.5 Span Point (59.0')
Dead Loads	Selfweight	45	44	43	42	36	32	29	27	26	18	9	0
	Stool	5	5	5	5	4	4	3	3	3	2	1	0
	Deck	45	45	44	42	36	33	30	27	27	18	9	0
	FWS	7	7	7	7	6	5	5	4	4	3	1	0
	Barrier	10	10	9	9	8	7	6	6	6	4	2	0
	Diaphragms	0	0	0	0	0	0	0	0	0	0	0	0
	Total	112	111	108	105	90	81	73	67	66	45	22	0
Live Loads①	Uniform Lane	28	27	27	26	22	20	19	18	17	14	10	7
	Tandem + IM	48	48	47	46	43	41	39	38	38	33	28	23
	Truck + IM	64	64	63	62	57	55	52	50	50	43	36	29
	Governing LL (Truck + IM) + Lane	92	91	90	88	79	75	71	68	67	57	46	36
Strength I Load Comb (1.25·DL+1.75·LL)		301	298	293	285	251	233	216	203	200	156	108	63
Service I Load Comb (1.00·DL+1.00·LL)		204	202	198	193	169	156	144	135	133	102	68	36
Service III Load Comb (1.00·DL+0.80·LL)		186	184	180	175	153	141	130	121	120	91	59	29

①All live loads include the interior beam live load distribution factors of 0.731 and IM of 0.33.

Table 5.7.3.6
Bending Moment Summary (kip-ft/beam)

Load Type/Combination			Brg CL (0.0')	Brg Face (0.63')	Trans Point #1 (2.38')	Critical Shear Point (4.2')	0.1 Span Point (11.8')	Trans Point #2 (16.4')	Trans Point #3 (20.4')	0.2 Span Point (23.6')	Trans Point #4 (24.4')	0.3 Span Point (35.4')	0.4 Span Point (47.2')	0.5 Span Point (59.0')
Dead Loads	DC1	Selfweight	0	28	104②	181	475	632	755	844	865③	1108	1267	1319
		Stool	0	3	12	21	55	73	88	98	100	129	147	153
		Deck	0	28	106	184	482	641	766	857	878	1124	1285	1338
		Diaph.	0	0	0	1	2	3	4	4	4	6	7	7
		Total DC1	0	59	222	387	1014	1349	1613	1803	1847	2367	2706	2817
	DC2	Barrier	0	6	23	39	103	137	164	184	188	241	276	287
		FWS	0	4	16	29	75	100	119	134	137	175	201	209
		Total DC2	0	10	39	68	178	237	283	318	325	416	477	496
	Total (DC1+DC2)		0	69	261	455	1192	1586	1896	2121	2172	2783	3183	3313
Live Loads①	Uniform Lane		0	13	47	82	216	287	343	384	393	503	575	599
	Tandem + IM		0	22	82	142	373	495	591	661	677	865	985	1020
	Truck + IM		0	29	110	192	499	661	786	877	897	1132	1283	1319
	Governing LL (Truck+IM) +Lane		0	42	157	274	715	948	1129	1261	1290	1635	1858	1918④
Strength I Load Comb (1.25·DL+1.75·LL)			0	160	601	1048	2741	3642	4346	4858	4973	6340	7230	7498
Service I Load Comb (1.00·DL+1.00·LL)			0	111	418	729	1907	2534	3025	3382	3462	4418	5041	5231
Service III Load Comb (1.00·DL+0.80·LL)			0	103	387	674	1764	2344	2799	3130	3204	4091	4669	4847
Fatigue I Load Comb (1.75·LL)			-	-	-	-	-	-	-	-	-	-	-	1008

①All live loads include the interior beam live load distribution factor of 0.538 and IM of 0.33.

②Beam selfweight at strand release = 132 k-ft (beam in casting bed with span length equal to overall beam length of 119.25)

Beam selfweight at erection on bearings = 104 k-ft (beam span length equal to design span of 118.0 ft)

③Beam selfweight at strand release = 893 k-ft (beam in casting bed with span length equal to overall beam length of 119.25 ft)

Beam selfweight at erection on bearings = 865 k-ft (beam span length equal to design span of 118.0 ft)

④Fatigue live load = 576 k-ft (includes interior beam live load distribution factor of 0.315 ad IM of 0.15 applied to fatigue truck only)

E. Design Beam Pretensioning With Debonded Strands for Control of End Stresses

Typically, the tension at the bottom of the beam at midspan in its final configuration after all losses have occurred dictates the required level of prestressing.

1. Estimate Required Prestress

Use the Service III load combination

Bottom of beam stress:

$$f_{serv3bot} = \left(\frac{M_{DC1}}{S_{gb}}\right) + \left(\frac{M_{DC2}}{S_{cb}}\right) + \left(\frac{M_{LL} \cdot 0.8}{S_{cb}}\right)$$

$$= \left(\frac{2817 \cdot 12}{8{,}246}\right) + \left(\frac{496 \cdot 12}{12{,}917}\right) + \left(\frac{1918 \cdot 12 \cdot 0.8}{12{,}917}\right) = 5.99 \text{ ksi}$$

For 300 ksi strands, MnDOT practice is to jack to an initial prestress force of $0.72f_{pu}$. As a starting point, the total prestress losses will be assumed to be 24%. This results in an effective prestress of

$$f_{pe} = 0.72 \cdot f_{pu} \cdot (1 - 0.24) = 0.72 \cdot 300 \cdot 0.76 = 164.2 \text{ ksi}$$

Strands are typically placed on a 2" grid. Referring to BDM Figures 5.4.6.2 to determine a starting point for the number of strands, assume 50 strands and choose a pattern that provides the greatest eccentricity for the prestressing force. This pattern will fill all the straight strand locations in the bottom flange. The centroid of this 50 strand pattern is:

$$y_{str} = \left[\frac{\sum \text{ (\# of strands)} \cdot \text{(y of strands)}}{\text{(total \# of strands)}}\right]$$

$$= \left[\frac{18 \cdot (2 + 4) + (10 \cdot 6) + (4 \cdot 8)}{50}\right] = 4.00 \text{ in}$$

Using the centroid of this group as an estimate of the strand pattern eccentricity results in

$$e_{50} = y_g - y_{str} = 18.07 - 4.00 = 14.07$$

The area, A_{strand}, of a 0.6" diameter 7-wire strand is 0.217 in^2

The axial compression produced by the prestressing strands is

$$P = A_s \cdot f_{pe} = n_{strands} \cdot A_{strand} \cdot f_{pe}$$

The internal moment produced by the prestressing strands is

$$M_{p/s} = A_s \cdot f_{pe} \cdot e_{50} = n_{strands} \cdot A_{strand} \cdot f_{pe} \cdot e_{50}$$

The allowable tension after losses = $0.19 \cdot \sqrt{f'_c} = 0.19 \cdot \sqrt{9.5} = 0.59$ ksi

This moment and axial compression from the prestress, f_{pscomp}, must reduce the bottom flange tension from 5.99 ksi tension to the allowable tension of 0.59 ksi.

$$f_{pscomp} = 5.99 - 0.59 = 5.40 \text{ ksi}$$

Knowing that

$$f_{pscomp} = \frac{P}{A} + \frac{M_{p/s}}{S_b}$$

and substituting and solving for $n_{strands}$, we get an estimate for the required number of strands:

$$n_{strands} = \frac{f_{pscomp}}{A_{strand} \cdot f_{pe} \cdot \left(\frac{1}{A} + \frac{e_{50}}{S_b}\right)} = \frac{5.40}{0.217 \cdot 164.2 \cdot \left(\frac{1}{704} + \frac{14.07}{8246}\right)}$$

$$= 48.5 \text{ strands}$$

Try a strand pattern with 48 strands to begin.

After reviewing Bridge Details Part II Figure 5-397.503, the trial strand pattern shown in Figure 5.7.3.4 was selected.

The properties of this strand pattern at midspan are:

$$y_{strand} = \left[\frac{18 \cdot (2 + 4) + (10 \cdot 6) + (2 \cdot 8)}{48}\right] = 3.83 \text{ in}$$

$$e_{strand} = y_b - y_{strand} = 18.07 - 3.83 = 14.24 \text{ in}$$

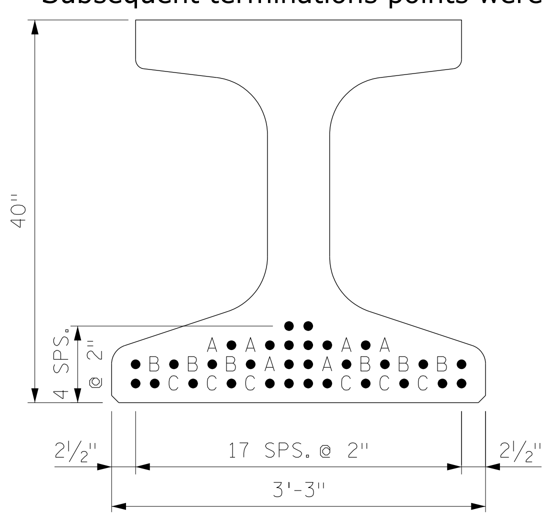
[5.9.4.3.3]

2. Debonding

Typically, tensile stress on the top flange of the beam near its ends dictate the amount of debonding. This example uses the maximum amount of debonding given the selected 48 strands. Debonding limitations are as follows:

- Maximum number of strands that terminate debonding at any given location:
    - 4 – When ten or fewer total strands are debonded.
    - 6 – When eleven or more total strands are debonded.
- Maximum of 45% of strands debonded in each row.
- Fully bonded strand location requirements:
    - MnDOT shape base strands (refer to Figure 5.4.3.1).
    - Bottom flange strands within the horizontal limits of web.
    - Outermost strands within the full-width section of the bottom flange.
- Alternate bonded and debonded strand locations both horizontally and vertically.
- Longitudinal spacing of debonding termination points shall be at least 60 d_b (3 ft).
- Debond strands symmetrically about beam centerline.
- For simple span precast, pretensioned girders, limit debonding lengths from the end of the beam to 20% of the total span length (23.6 feet).

After analyzing the section, the following layout which maximizes debonding, 18 debonded strands, was obtained. The initial termination point of 14 feet was chosen arbitrarily and will be confirmed later. Subsequent terminations points were chosen at a spacing of 4 ft.



END VIEW

END DEBONDING		
SYMBOL	NO.	LENGTH
A	6	14'-0"
B	6	18'-0"
C	6	22'-0"
TOTAL	18	

DEBONDING LENGTHS FOR A, B, AND C ARE MEASURED FROM EACH END OF BEAM.

Figure 5.7.3.4

Certain critical sections need to be tested. Depending on the stress checks at transfer, consider altering the number of debonded strands, debonding termination points, or add strands in the top flange. Before testing these critical locations, the losses due to prestressing need to be calculated.

[5.9.3]

3. Prestress Losses

Prestress losses are computed using the approximate method.

[5.9.3.2.3]

Elastic Shortening Loss

Use the alternative equation presented in the LRFD C5.9.3.2.3a.

$$\Delta f_{pES} = \frac{A_{ps} \cdot f_{pbt} \cdot \left(I_g + e_m{}^2 \cdot A_g\right) - e_m \cdot M_g \cdot A_g}{A_{ps} \cdot \left(I_g + e_m{}^2 \cdot A_g\right) + \dfrac{A_g \cdot I_g \cdot E_{ci}}{E_p}}$$

$$A_{ps} = (\#\text{ of strands}) \cdot (\text{strand area}) = 48 \cdot 0.217 = 10.42\ \text{in}^2 \quad (\text{midspan})$$

$$f_{pbt} = f_{pj} = 216.0\ \text{ksi}$$

$$e_m = e_{strandmid} = 14.24\ \text{in}$$

$$\frac{A_g \cdot I_g \cdot E_{ci}}{E_p} = \frac{704 \cdot (149{,}002) \cdot (4578)}{28{,}500} = 16{,}849{,}836\ \text{in}^6$$

$$A_{ps} \cdot \left(I_g + e_m{}^2 \cdot A_g\right) = 10.42\left[149{,}002 + (14.24)^2 \cdot (704)\right] = 3{,}040{,}112\ \text{in}^6$$

$$\Delta f_{pES} = \frac{216.0 \cdot (3{,}040{,}112) - 14.24 \cdot (1319) \cdot (12) \cdot (704)}{3{,}040{,}112 + 16{,}849{,}836} = 25.0\ \text{ksi}$$

[5.9.3.3]

Long Term Losses

Use the approximate equation in the LRFD 5.9.3.3

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pj} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

For an average humidity in Minnesota of 73%

$$\gamma_h = 1.7 - 0.01 \cdot H = 1.7 - 0.01 \cdot 73 = 0.97$$

$$\gamma_{st} = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 8.0} = 0.56$$

For low relaxation strand, $\Delta f_{pR} = 2.4$

$$\Delta f_{pLT} = 10.0 \cdot \frac{216.0 \cdot (10.42)}{704} \cdot 0.97(0.56) + 12.0(0.97)(0.56) + 2.4$$

$$= 26.3\ \text{ksi}$$

[5.9.3.1] **Total Losses**

$$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pLT} = 25.0 + 26.3 = 51.3 \text{ ksi}$$

$$f_{pe} = f_{pj} - \Delta f_{pt} = 216.0 - 51.3 = 164.7 \text{ ksi}$$

$$\text{prestress loss percentage} = \frac{\Delta f_{pt}}{f_{pj}} \cdot 100 = \frac{51.3}{216.0} \cdot 100 = 23.8\ \%$$

Jacking force:

$$P_{jack} = A_{ps} \cdot \left(f_{pj}\right) = 10.42 \cdot (216.0) = 2251 \text{ kips}$$

Initial prestress force after transfer through midspan:

$$P_i = A_{ps} \cdot \left(f_{pj} - \Delta f_{pES}\right) = 10.42 \cdot (216.0 - 25.0) = 1990 \text{ kips}$$

Prestress force after all losses through midspan:

$$P_e = A_{ps} \cdot f_{pe} = 10.42 \cdot 164.7 = 1716 \text{ kips}$$

Prestress Forces for Transfer Point #1

$$A_{ps} = (\#\text{ of strands}) \cdot (\text{strand area}) = 30 \cdot 0.217 = 6.51 \text{ in}^2$$

Initial prestress force after transfer at Transfer Point #1

$$P_i = A_{ps} \cdot \left(f_{pj} - \Delta f_{pES}\right) = 6.51 \cdot (216.0 - 25.0) = 1243 \text{ kips}$$

Prestress force after all losses at Transfer Point #1

$$P_e = A_{ps} \cdot f_{pe} = 6.51 \cdot 164.7 = 1072 \text{ kips}$$

[5.9.2.3.1] **4. Stresses at Transfer (compression +, tension -)**
Stress Limits for P/S Concrete at Release

This example checks the top stress at Transfer Point #1 for fully bonded strands (x=2.38’) and the bottom compression stress at Transfer Point #4 for strands that terminate at 22 feet (x=24.4′). These checks will help to determine if the amount of debonding is sufficient. Consider reducing the amount or length of debonding if compression stresses at transfer points are significantly passing. Consider increasing the amount of debonding (within allowable limits) or adding top flange strands if tension stresses at transfer points are failing. Only one tension and compression check at release are shown for brevity. As with all limit states checked in this example, additional locations are often required based on strand and debonding layout.

Compression in the concrete is limited to:

$$f_{climrel} = 0.65 \cdot f'_{ci} = 0.65 \cdot 8.0 = 5.20 \text{ ksi}$$

For tension, MnDOT uses the AASHTO Table 5.9.2.3.1b-1 stress limits for "areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force" for beam design. The tension in the concrete is limited to:

$$f_{tlimrel} = -\,0.24 \cdot \sqrt{f'_{ci}} = -\,0.24 \cdot \sqrt{8.0} = -\,0.68 \text{ ksi}$$

Confirm Bonded Reinforcement Tension Limit
Using the method presented in the LRFD C5.9.2.31b, confirm the required amount of bonded reinforcement is present in the top flange to resist the calculated tensile force. The total tensile force is determined by integrating the stress over the entire tensile zone. This can be estimated by summing the tensile force from discrete subzones using simplified beam geometry.

Centroid of strand pattern at Transfer Point #1:

$$y_{strand} = \left[\frac{(12 \cdot 2) + (10 \cdot 4) + (6 \cdot 6) + (2 \cdot 8)}{30}\right] = 3.87 \text{ in}$$

The eccentricity of the strand pattern at transfer point #1 is:

$$e_{strand} = y_b - y_{strand} = 18.07 - 3.87 = 14.20 \text{ in}$$

At this point, the beam is sitting in the casting bed. The beam will camber upward when the strands are released, so the span length used to determine the selfweight moment is the end-to-end beam length of 119.25 feet.

The internal prestress moment at Transfer Point #1 is:

$$P_i \cdot e_{strand} = 1243 \cdot 14.20 = 17{,}651 \text{ kip-in}$$

$$\text{Top stress due to P/S} = \left(\frac{P_i}{A_g}\right) - \left(\frac{P_i \cdot e_{strand}}{S_{gt}}\right) = \left(\frac{1243}{704}\right) - \left(\frac{17{,}651}{6794}\right)$$

$$= -0.83 \text{ ksi}$$

$$\text{Bottom stress due to P/S} = \left(\frac{P_i}{A_g}\right) + \left(\frac{P_i \cdot e_{strand}}{S_{gb}}\right) = \left(\frac{1243}{704}\right) + \left(\frac{17{,}651}{8246}\right)$$

$$= 3.91 \text{ ksi}$$

$$\text{Top stress due to selfweight} = \left(\frac{M_{swtr}}{S_{gt}}\right) = \left(\frac{132 \cdot 12}{6794}\right) = 0.23 \text{ ksi}$$

Bottom stress due to selfweight $= -\left(\frac{M_{swtr}}{S_{gb}}\right) = -\left(\frac{132 \cdot 12}{8246}\right) = -0.19$ ksi

Top stress = -0.83 + 0.23 = -0.60 ksi

Bottom stress = 3.91 – 0.19 = 3.72 ksi

$$\text{Depth of Neutral Axis} = \frac{0.60}{\left(\frac{0.60 + 3.72}{40}\right)} = 5.56 \text{ in}$$

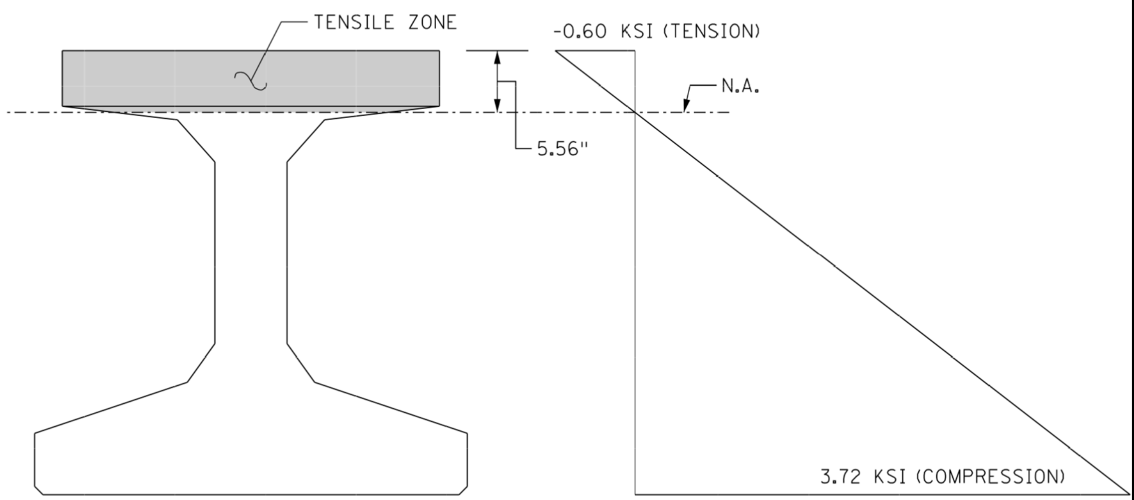


Figure 5.7.3.5

Using the stresses distribution and simplified beam geometry shown in Figure 5.7.3.5, the total tensile force at Transfer Point #1 is calculated in Table 5.7.3.7 and subsequent equations.

Table 5.7.3.7
Summation of Tension Force

Slice	y_{top} (in)	y_{bot} (in)	y_{cg} (in)	A_i (in^2)	f_i (ksi)	T_i (kips)
1	0.00	5.00	2.50	170.0	-0.33	56.1
2	5.00	5.56	5.26	16.3	-0.03	0.5

$$T = \sum f_i \cdot A_i = \sum T_i = 56.1+0.5 = 56.6 \text{ kips}$$

$$f_s = 0.5 \cdot f_y = 0.5 \cdot 60 = 30.0 \text{ ksi} \le 30.0 \text{ ksi}$$

$$A_{s\_req} = \frac{T}{f_s} = \frac{56.6}{30.0} = 1.89 \text{ in}^2$$

The above calculation shows 1.89 in^2 of developed reinforcement is required in the top flange to accommodate the release tension stress at Transfer Point #1. The standard amount of top flange reinforcement for a 40MH beam per Bridge Details Part II Figure 5-397.503 is four #8 bars (G806E). The development of the G806E bars can be calculated as follows:

[5.10.8.2.1]

$$l_{db} = 2.4 \cdot d_b \cdot \frac{f_y}{\sqrt{f'_c}} = 2.4 \cdot 1.0 \cdot \frac{60}{\sqrt{8}} = 50.9 \text{ in}$$

The following modification factors are assumed:

$\lambda = 1.0$

$\lambda_{rl} = 1.0$

$\lambda_{cf} = 1.5$

$\lambda_{er} = 1.0$

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} = \frac{1.0}{2.38+0} = \qquad \text{(conservatively assumes } k_{tr} = 0\text{)}$$

$$= 0.42 \qquad (0.4 \le \lambda_{rc} \le 1.0)$$

$$l_d = l_{db} \cdot \left(\frac{\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er}}{\lambda}\right) = 50.9 \cdot \left(\frac{1.0 \cdot 1.5 \cdot 0.42 \cdot 1.0}{1.0}\right) = 32.1 \text{ in}$$

Assuming a 1½ inch clear cover to the end of beam, this calculation shows the G806E bars are fully developed at Transfer Point #1. Although not quantified in this example, the top leg of two G505E bars also contribute to the bonded reinforcement area as the G806E bar develops at the end of beam.

$A_{s\_prov} = 4 \cdot 0.79 = 3.16 \text{ in}^2 \ge A_{s\_req} = 1.89 \text{ in}^2$ OK

Because the provided area of developed reinforcement (3.16 in^2) is larger than the required area (1.89 in^2), the previously calculated full tension stress limit ($f_{tlimrel}$ = -0.68 ksi) can be used. Alternatively, BDM Table 5.4.2.1 can be used when applicable, in lieu of the process shown in LRFD C5.9.2.31b, to confirm the required amount of developed reinforcement is present in the top flange for standard MnDOT beam shapes.

Check Release Stresses at Transfer Points #1 and #4

Top stress at Transfer Point #1 was calculated in the previous section as follows:

Top stress at Transfer Point #1 = -0.60 ksi

$f_{tlimrel}$ = -0.68 ksi OK

Bottom stress at Transfer Point #4 can be calculated as follows:

$P_i \cdot e_{strand} = 1990 \cdot 14.24 = 28{,}338$ kip-in

$$\text{Bottom stress due to P/S} = \left(\frac{P_i}{A_g}\right) + \left(\frac{P_i \cdot e_{strand}}{S_{gb}}\right) = \left(\frac{1990}{704}\right) + \left(\frac{28{,}338}{8246}\right)$$

$$= 6.26 \text{ ksi}$$

$$\text{Bottom stress due to selfweight} = -\left(\frac{M_{swtr}}{S_{gb}}\right) = -\left(\frac{893 \cdot 12}{8246}\right) = -1.30 \text{ ksi}$$

Bottom stress at Transfer Point #4 = 6.26 – 1.30 = 4.96 ksi < 5.20 ksi OK

If the tension stress at Transfer Point #1 or compressive stress at Transfer Point #4 fail, consider the following options to rectify the issue:

- Increase initial concrete compressive strength, f'_{ci} (up to 8 ksi)
- Increase the amount or length of debonding used
- Raise the center of gravity of prestressing strands
- Add permanent or temporary top flange strands.

The initial concrete strength, f'_{ci}, was assumed to be 8.0 ksi. For the most economical beam, the designer should choose the lowest required f'_{ci} for the beam. This can be determined by substituting the calculated maximum compression and tension in the stress limit equations and solving for f'_{ci}.

$$\text{Lowest required } f'_{cireqcomp} = \frac{f_{compTP\#4}}{0.65} = \left(\frac{4.96}{0.65}\right) = 7.63 \text{ ksi}$$

$$\text{Lowest required } f'_{cireqten} = \left(\frac{f_{tenTP\#1}}{0.24}\right)^2 = \left(\frac{-0.60}{0.24}\right)^2 = 6.25 \text{ ksi}$$

Controlling initial concrete strength $f'_{ci} = 7.7$ ksi

When updating the initial concrete strength, prestress losses must be recalculated. The reduced initial concrete strength of 7.7 ksi increases the prestress losses in the final condition. This loss of prestress force necessitates a final concrete strength, f'_c, of greater than 9.5 ksi, the maximum allowable. Due to this, the initial concrete strength will remain at 8.0 ksi despite release stress checks passing at a lower concrete strength.

Proceed to the service and fatigue stress checks after final losses.

[5.9.2.3.2]

5. Stresses at Service Loads (compression +, tension -)

Stress Limits for P/S Concrete after All Losses

Compression in the concrete is limited to (Service I Load Combination):

$f_{climf1} = 0.45 \cdot f'_c = 0.45 \cdot 9.5 = 4.28$ ksi

(for prestress and permanent loads)

Check the bottom stress at transfer points and the top stress at midspan against this limit.

$f_{climf2} = 0.60 \cdot \phi_w \cdot f'_c = 0.60 \cdot 1.0 \cdot 9.5 = 5.70$ ksi

(for live load, prestress, permanent loads, and transient loads)

Check the top stress at midspan against this limit.

[5.5.3.1]

Compression in concrete is limited to (Fatigue I Load Combination):

$f_{climfat} = 0.40 \cdot f'_c = 0.40 \cdot 9.5 = 3.80$ ksi

(for live load and $^1/_2$ of prestress and permanent loads)

Check the top stress at midspan against this limit.

Tension in the concrete is limited to (Service III Load Combination):

$f_{flimf} = -\,0.19 \cdot \sqrt{f'_c} = -\,0.19 \cdot \sqrt{9.5} = -\,0.586$ ksi

Check the bottom stress at midspan against this limit.

Check Stresses at Midspan After Losses:

Bottom stress

$$= -\left(\frac{M_{DC1}}{S_{gb}}\right) - \left(\frac{M_{DC2}}{S_{cb}}\right) - \left(\frac{M_{LL} \cdot 0.8}{S_{cb}}\right) + \left(\frac{P_e}{A_g}\right) + \left(\frac{P_e \cdot e_{strand}}{S_{gb}}\right)$$

$$= -\left(\frac{2817 \cdot 12}{8246}\right) - \left(\frac{496 \cdot 12}{12{,}917}\right) - \left(\frac{1918 \cdot 12 \cdot 0.8}{12{,}917}\right) + \left(\frac{1716}{704}\right) + \left(\frac{1716 \cdot 14.24}{8246}\right)$$

$= -0.585$ ksi < -0.586 ksi OK

Top stress due to all loads

$$= \left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2} + M_{LL}}{S_{gtc}}\right)$$

$$= \left(\frac{1716}{704}\right) - \left(\frac{1716 \cdot 14.24}{6794}\right) + \left(\frac{2817 \cdot 12}{6794}\right) + \left[\frac{(496 + 1918) \cdot 12}{42{,}761}\right]$$

$= 4.49$ ksi < 5.70 ksi OK

Top stress due to permanent loads

$$= \left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2}}{S_{gtc}}\right)$$

$$= \left(\frac{1716}{704}\right) - \left(\frac{1716 \cdot 14.24}{6794}\right) + \left(\frac{2817 \cdot 12}{6794}\right) + \left(\frac{496 \cdot 12}{42{,}761}\right)$$

$$= 3.96 \text{ ksi} < 4.28 \text{ ksi} \quad \text{OK}$$

Top stress due to fatigue live load plus ½ the sum of prestress and permanent loads

$$= \frac{1}{2}\left(\left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2}}{S_{gtc}}\right)\right) + \left(\frac{M_{LL}}{S_{gtc}}\right)$$

$$\frac{1}{2}\left(\left(\frac{1716}{704}\right) - \left(\frac{1716 \cdot 14.24}{6794}\right) + \left(\frac{2817 \cdot 12}{6794}\right) + \left(\frac{496 \cdot 12}{42{,}761}\right)\right) + \left(\frac{1008 \cdot 12}{42{,}761}\right)$$

$$= 2.26 \text{ ksi} < 3.80 \text{ ksi} \quad \text{OK}$$

Check the Compression Stresses at Transfer Points After Losses

Bottom flange stress at Transfer Point # 1 due to prestress and permanent loads

$$= \frac{P_e}{A_g} + \frac{P_e \cdot e_{strand}}{S_{gb}} - \left(\frac{M_{DC1}}{S_{gb}}\right) - \left(\frac{M_{DC2}}{S_{gbc}}\right)$$

$$= \frac{1072}{704} + \frac{1072 \cdot 14.20}{8246} - \left(\frac{222 \cdot 12}{8246}\right) - \left(\frac{39 \cdot 12}{12{,}917}\right)$$

$$= 3.01 \text{ ksi} < 4.28 \text{ ksi} \quad \text{OK}$$

Bottom flange stress at Transfer Point # 4 due to prestress and permanent loads

$$= \frac{P_e}{A_g} + \frac{P_e \cdot e_{strand}}{S_{gb}} - \left(\frac{M_{DC1}}{S_{gb}}\right) - \left(\frac{M_{DC2}}{S_{gbc}}\right)$$

$$= \frac{1716}{704} + \frac{1716 \cdot 14.24}{8246} - \left(\frac{1847 \cdot 12}{8246}\right) - \left(\frac{325 \cdot 12}{12{,}917}\right)$$

$$= 2.41 \text{ ksi} < 4.28 \text{ ksi} \quad \text{OK}$$

As discussed in the release stress calculations, the highest tensile stress at midpoint after losses is the most critical location (0.585 ksi < 0.586 ksi). This calculation confirms reducing initial concrete strength, thus increasing prestress losses, is not possible in the final condition. This closes the loop as to why the initial concrete strength will remain at 8.0 ksi.

The final concrete strength, f'_c, was assumed to be 9.5 ksi. For the most economical beam, the designer should choose the lowest required f'_c for the beam. This is determined by substituting the calculated maximum tensile stress for f_{flimf} in the tension limit equation and solving for f'_c.

$$\text{Lowest required } f'_c = \left(\frac{f_{flimf}}{0.19}\right)^2 = \left(\frac{-0.585}{0.19}\right)^2 = 9.48 \text{ ksi}$$

The assumed concrete strength cannot be reduced.

Keep f'_c = 9.5 ksi

[5.5.4]

6. Flexure – Strength Limit State

Resistance factors at the strength limit state are:

ϕ = 1.00 for flexure and tension (assumed)

ϕ = 0.90 for shear and torsion

ϕ = 1.00 for tension in steel in anchorage zones

Strength I design moment, M_u, is 7498 kip-ft at midspan.

From previous calculations, distance to strand centroid from bottom of the beam at midspan is:

y_{strand} = 3.83 in

Similar to Grade 270 strands, the yield strength, f_{py} is taken as $0.9 \cdot f_{pu}$.

$$f_{py} = 0.9 \cdot f_{pu} = 0.9 \cdot 300 = 270 \text{ ksi}$$

[5.6.3.1.1]

$$k = 2 \cdot \left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 2 \cdot \left(1.04 - \frac{270}{300}\right) = 0.28$$

d_p = (beam height) + stool + deck - y_{strand}

= 40 + 1.5 + 8.5 – 3.83 = 46.17 in

Begin by assuming the neutral axis lies in the deck.

For f'_c = 4.0 ksi, β_1 = 0.85 and α_1 = 0.85.

Then

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_e + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

$$= \frac{10.42 \cdot 300}{0.85 \cdot 4.0 \cdot 0.85 \cdot 82 + 0.28 \cdot 10.42 \cdot \left(\dfrac{300}{46.17}\right)} = 12.21 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 12.21 = 10.38 \text{ in}$$

Compression block depth is greater than the thickness of the slab (8.5 in), so T-section behavior must be considered. The "web width", b_w, of the T-section is the beam flange width, which is 34 in.

Then

$$c = \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_c \cdot (b - b_w) \cdot h_f}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

$$= \frac{10.42 \cdot 300 - 0.85 \cdot 4.0 \cdot (82 - 34) \cdot 8.5}{0.85 \cdot 4.0 \cdot 0.85 \cdot 34 + 0.28 \cdot 10.42 \cdot \frac{300}{46.17}} = 14.83 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 14.83 = 12.61 \text{ in}$$

The revised compression block depth is less than the thickness of the slab plus the flange thickness (15 in), so T-section behavior is confirmed. If the revised compression block depth had been greater than 15 inches, the section would be acting as a stepped T-section and a strain compatibility approach would have been necessary.

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 300 \cdot \left(1 - 0.28 \cdot \frac{14.83}{46.17}\right) = 273.0 \text{ ksi}$$

The internal lever arm between compression and tension flexural force components is:

$$d_p - \frac{a}{2} = 46.17 - \frac{12.61}{2} = 39.87 \text{ in}$$

Then:

$$M_n = A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_c \cdot (b - b_w) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

$$= 10.42 \cdot 273.0 \cdot 39.87 + 0.85 \cdot 4.0 \cdot (82 - 34) \cdot 8.5 \cdot \left(\frac{12.61}{2} - \frac{8.5}{2}\right)$$

$$= 116{,}267 \text{ kip-in} = 9689 \text{ kip-ft}$$

$$M_r = \phi M_n = 1.0 \cdot 9689 = 9689 \text{ kip-ft} > M_u = 7498 \text{ kip-ft} \quad \text{OK}$$

[5.5.4.2]

Validate the assumption of 1.0 for the resistance factor:

Concrete compression strain limit $\varepsilon_c = 0.003$

Reinforcement tension-controlled strain limit $\varepsilon_{tl} = 0.005$

Distance to extreme tension strand $d_t = 40 + 1.5 + 8.5 - 2 = 48$ in

Referring to LRFD Figure C5.6.2.1-1 and using similar triangles in the prestressing strand, ε_t:

$$\varepsilon_t = (d_t - c) \cdot \left(\frac{\varepsilon_c}{c}\right) = (48 - 14.83) \cdot \left(\frac{0.003}{14.83}\right) = 0.0067 > 0.005$$

Therefore $\phi = 1.0$, which matches the assumption

[5.6.3.3]

7. Minimum Reinforcement

To prevent brittle failure, an adequate amount of reinforcement is required. Check that the section can carry the smaller of:

3) $1.33M_u$
4) Cracking Moment, M_{cr}

At midspan, $1.33M_u = 1.33 \cdot 7498 = 9972$ kip-ft

Lightweight concrete is not being used, so concrete density factor $\lambda = 1.0$.

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{9.5} = 0.74 \text{ ksi}$$

$$f_{cpe} = f_{peb} = \frac{P_e}{A_g} + \frac{P_e \cdot e_{strand}}{S_{gb}}$$

$$= \frac{1716}{704} + \frac{1716 \cdot 14.24}{8246} = 5.40 \text{ ksi}$$

$$M_{cr} = \gamma_3 \cdot \left[(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) \cdot S_{gbc} - M_{dnc} \cdot \left(\frac{S_{gbc}}{S_{gb}} - 1\right)\right]$$

$$= 1.0 \cdot \left[(1.6 \cdot 0.74 + 1.1 \cdot 5.40) \cdot 12{,}917 - 2817 \cdot 12 \cdot \left(\frac{12{,}917}{8246} - 1\right)\right]$$

$= 72{,}872$ kip-in $= 6073$ kip-ft < 9972 kip-ft $\qquad M_{cr}$ GOVERNS

$M_r = \phi M_n = 9689$ kip-ft > 6073 kip-ft OK

F. Design Reinforcement for Shear
[5.7]

1. Vertical Shear Design

Determine d_v and Critical Section for Shear

Begin by determining the effective shear depth d_v at the critical section for shear. The critical location for shear x_{vcrit} is defined as the distance d_v from the internal face of support. The internal face is assumed to be at the inside edge of the 15 inch long sole plate.

The effective shear depth is taken as the greatest of:

[5.7.2.8]

$$d_v = d_p - \frac{a}{2} \qquad \text{or} \qquad 0.72h_{comp} \qquad \text{or} \qquad 0.9d_e$$

[5.7.3.4.2]

AASHTO is unclear on which strands to consider when determining d_p for calculating d_v. Considering LRFD Figure C5.7.2.8-1, it appears that d_v is based on calculating d_p and d_e for the strands found on the flexural tension side of the neutral axis. But for shear calculations, LRFD Article 5.7.3.4.2 and Figure 5.7.3.4.2-1 define A_{ps} as the strands found on the flexural tension side of ½ the height of the composite section. To keep computations simple, yet reasonably accurate, follow the LRFD Article 5.7.3.4.2 definition and consider only the prestressing strands found below ½ the height of the composite section when calculating d_p. Therefore, only bonded prestressing strands in the bottom flange are considered for debonded beams.

The flexural tension side of the member is defined as:

$$\frac{h_{comp}}{2} = \frac{50}{2} = 25 \text{ in}$$

The centroid of bonded bottom flange prestressing strands is at:

$$y_{sstr} = \left[\frac{(12 \cdot 2) + (10 \cdot 4) + (6 \cdot 6) + (2 \cdot 8)}{30}\right]$$

$$= 3.87 \text{ in}$$

With this strand centroid, d_p can be computed for the composite section:

$$d_p = h_{comp} - y_{sstr} = 50 - 3.87 = 46.13 \text{ in}$$

Recalculate the value of the compression block depth "a" considering only the bonded prestressing strands at the critical shear section:

$$A_{ps} = A_{sps} = (\#\text{ of bonded strands}) \cdot (\text{strand area}) = 30 \cdot 0.217 = 6.51 \text{ in}^2$$

Begin by assuming the neutral axis lies in the deck.

[5.6.3.1.1]

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_e + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

$$= \frac{6.51 \cdot 300}{0.85 \cdot 4.0 \cdot 0.85 \cdot 82 + 0.28 \cdot 6.51 \cdot \dfrac{300}{46.13}} = 7.85 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 7.85 = 6.67 \text{ in}$$

Compression block depth is less than 8.5", the thickness of the slab, so T-section behavior is not considered.

$$d_v = d_p - \frac{a}{2} = 46.13 - \frac{6.67}{2} = 42.80 \text{ in}$$

But the effective shear depth d_v need not be less than

$$d_v \geq 0.72 \cdot h_{comp} = 0.72 \cdot 50 = 36.0 \text{ in}$$

or

$$d_v \geq 0.9\, d_e = 0.9\, d_p = 0.9\,(46.13) = 41.5 \text{ in}$$

Take d_v = 42.8 inches

Then the critical section for shear x_{vcrit} is:

x_{vcrit} = (0.5 · sole plate length) + d_v

= (0.5 · 15.0) + 42.8

= 50.3 in = 4.2 ft from centerline of bearing

Check Maximum Factored Shear Limit

From Table 5.7.3.5 the Strength I design shear at 4.2 ft is

$V_u = 285$ kips

The girder is supported by an integral type abutment at both ends. Therefore, the nominal shear capacity of the section is limited to:

[5.7.3.3]

$$V_n = 0.25 \cdot f'_c \cdot d_v \cdot b_v + V_p = 0.25 \cdot 9.5 \cdot 42.8 \cdot 6.5 + 0 = 661 \text{ kips}$$

The vertical prestress component, V_p, is set to zero in the above equation due to all straight strands.

The maximum design shear the section can have is:

$$\phi_v \cdot V_n = 0.90 \cdot 661 = 595 \text{ kips} > 285 \text{ kips} \quad \text{OK}$$

Note that if the girder was supported by a parapet type abutment or pier without a continuity diaphragm, which are not built integrally with its support, the shear stress would have been limited to $0.18f'_c$ per AASHTO Article 5.7.3.2. Using an integral abutment, semi-integral abutment, or pier with a continuity diaphragm allows us to use the higher value from AASHTO Article 5.7.3.3.

Determine Longitudinal Strain ε_s

Assume that minimum transverse reinforcement will be provided in the cross section. As previously noted, A_{ps} includes only the area of prestressing steel found on the flexural tension side of the member, as defined in LRFD Figure 5.7.3.4.2-1. At x_{vcrit}, A_{ps} consists of only the bonded bottom flange strands.

Near the end of the beam and debonding, A_{ps} must also be reduced for development, so f_{ps} must be calculated again for the end section following the process shown previously:

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 300 \cdot \left(1 - 0.28 \cdot \frac{7.85}{46.13}\right)$$

$$= 285.7 \text{ ksi}$$

[5.9.4.3]

Strand development length ℓ_d is:

$$\ell_d = \kappa \cdot \left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b$$

$$= 1.6 \cdot \left[285.7 - \frac{2}{3}(164.7)\right] \cdot 0.6 = 168.9 \text{ in}$$

Note that a κ value of 1.6 is used at the x_{vcrit} location because all acting strands at this location have no debonded length. When calculating development where portions of the strand are debonded, a κ value of 2.0 is required per LRFD Article 5.9.4.3.3.

Transfer length ℓ_{tr} is:

$$\ell_{tr} = 60 \cdot d_b = 60\left(0.6\right) = 36.0 \text{ in}$$

At the critical section x_{vcrite} = (50.3 + 7.5) = 57.8 inches, which alters the previous critical section measurement (x_{vcrit}) from centerline of bearing to end of beam, the strand development fraction is:

$$F_{dev} = \frac{f_{pe}}{f_{ps}} + \frac{x_{vcrite} - \ell_{tr}}{\ell_d - \ell_{tr}}\left(1 - \frac{f_{pe}}{f_{ps}}\right)$$

$$= \frac{164.7}{285.7} + \frac{57.8 - 36.0}{168.9 - 36.0}\left(1 - \frac{164.7}{285.7}\right) = 0.65$$

Therefore, $A_{ps} = A_{sps} \cdot F_{dev}$

$$= 6.51 \cdot 0.65 = 4.23 \text{ in}^2$$

[5.7.3.4.2]

Use LRFD equation 5.7.3.4.2-4 to compute the strain:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5 \cdot N_u + |V_u - V_p| - A_{ps} \cdot f_{po}\right)}{E_s \cdot A_s + E_p \cdot A_{ps}}$$

$$= \frac{\left[\frac{|1048 \cdot 12|}{42.8} + |285 - 0| - (4.23 \cdot 0.70 \cdot 300)\right]}{28{,}500 \cdot 4.23} = -0.00257$$

Because the value is negative, the strain will be recalculated using an additional concrete term:

From Figure 5.4.6.1 of this manual, A_{ct} = 435 in^2

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5 \cdot N_u + |V_u - V_p| - A_{ps} \cdot f_{po}\right)}{E_c \cdot A_{ct} + E_s \cdot A_s + E_p \cdot A_{ps}}$$

$$= \frac{\left[\frac{|1048 \cdot 12|}{42.8} + |285 - 0| - (4.23 \cdot 0.70 \cdot 300)\right]}{4899 \cdot 435 + 28{,}500 \cdot 4.23} = -0.00014$$

Computed strain limits:

$$-0.0004 < -0.00014 < 0.006 \quad \text{OK}$$

Compute the tensile stress factor β using LRFD equation 5.7.3.4.2-1

$$\beta = \frac{4.8}{1 + 750 \cdot \varepsilon_s} = \frac{4.8}{1 + 750 \cdot (-0.00014)} = 5.36$$

Compute the angle θ using equation 5.7.3.4.2-3

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500 \cdot (-0.00014) = 28.51 \text{ degrees}$$

Compute the concrete contribution:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 5.36 \cdot \sqrt{9.5} \cdot 6.5 \cdot 42.8 = 145.2 \text{ kips}$$

The required steel contribution is

$$V_s = V_n - V_c - V_p = \frac{V_u}{\phi_v} - V_c - V_p = \frac{285}{0.90} - 145.2 = 171.5 \text{ kips}$$

Find the required spacing of double leg #4 stirrups:

$$s = \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{V_s} = \frac{2 \cdot 0.20 \cdot 60 \cdot 42.8 \cdot \cot(28.51)}{171.5} = 11.0 \text{ in}$$

Try double leg stirrups at a 11 inch spacing at the end of the beam.

$$A_v = \frac{0.4 \cdot 12}{11} = 0.44 \text{ in}^2 / \text{ft} \qquad V_s = 171.9 \text{ kips}$$

[5.7.2.5] Check that the minimum transverse reinforcement requirement is satisfied:

$$\frac{A_{vmin}}{s} = 0.0316 \cdot \lambda \cdot \sqrt{f'_c} \cdot \frac{b_v}{f_y}$$

$$= 0.0316 \cdot 1.0 \cdot \sqrt{9.5} \cdot \frac{6.5}{60} \cdot 12 = 0.13 \frac{\text{in}^2}{\text{ft}} < 0.44 \frac{\text{in}^2}{\text{ft}} \quad \text{OK}$$

[5.7.2.6] Check maximum permitted stirrup spacing at x_{vcrite}:

$$V_u = \frac{V_u - \phi \cdot V_p}{\phi \cdot b_v \cdot d_v} = \frac{285}{0.90 \cdot 6.5 \cdot 42.8} = 1.14 \text{ ksi}$$

$$V_{ulimit} = 0.125 \cdot f'_c = 0.125 \cdot 9.5 = 1.19 \text{ ksi} > 1.14 \text{ ksi}$$

Then the maximum spacing is the smaller of:

$s_{max} = 0.8 \cdot d_v = 0.8 \cdot 42.8 = 34.2$ in

or $s_{max} = 24$ in GOVERNS

$s_{max} = 24$ in > 11 in OK

Therefore, use double leg #4 stirrups at 11 inch spacing. Other sections are investigated similarly.

[5.7.4]

2. Interface Shear Transfer

The standard beam details require that the outer 6 inches on each edge of the top flange will be smooth with a bond breaker applied, which leaves 22 inches of the top flange to be roughened for engagement of shear transfer.

Then $b_{vi} = 22$ in

Calculate d_{vi} as the distance between the centroid of the tension steel at the critical shear section to the mid-thickness of the slab.

$$d_{vi} = h_{comp} - y_{sstr} - \frac{t_s}{2} = 50 - 3.87 - \frac{8.5}{2} = 41.9 \text{ in}$$

The Strength I vertical shear at the critical shear section due to all loads is:

$V_u = 285$ kip

$$v_{ui} = \frac{V_u}{b_{vi} \cdot d_{vi}} = \frac{285}{22 \cdot 41.9} = 0.31 \text{ ksi}$$

Interface shear force is:

$$V_{ui} = v_{ui} \cdot \frac{12 \text{ in}}{\text{ft}} \cdot b_{vi} = 0.31 \cdot 12 \cdot 22 = 81.8 \ \frac{\text{kip}}{\text{ft}}$$

Required nominal interface design shear is:

$$V_{nireq} = \frac{V_{ui}}{\varphi_v} = \frac{81.8}{0.90} = 90.9 \text{ kip/ft}$$

The interface area per 1 foot length of beam is:

$$A_{cv} = 22 \cdot 12 = 264.0 \text{ in}^2/\text{ft}$$

[5.7.4.4]

The standard beam details require the top flanges of the beam to be roughened. Then:

c= 0.28 ksi μ= 1.0 K_1= 0.3 K_2= 1.8 ksi

The upper limits on nominal interface shear are:

$$K_1 \cdot f'_c \cdot A_{cv} = 0.3 \cdot 4 \cdot 264.0 = 316.8 \text{ kip/ft} > 96.2 \text{ kip/ft} \quad \text{OK}$$

and

$$K_2 \cdot A_{cv} = 1.8 \cdot 264.0 = 475.2 \text{ kip/ft} > 96.2 \text{ kip/ft} \quad \text{OK}$$

The nominal interface shear resistance is:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} \cdot f_y + P_c\right)$$

$$P_c = 0.0 \text{ kip}$$

Substitute and solve for required interface shear steel:

$$A_{vfreq} = \frac{V_{nireq} - c \cdot A_{cv}}{\mu \cdot f_y} = \frac{90.9 - 0.28 \cdot 264.0}{1.0 \cdot 60} = 0.28 \text{ in}^2/\text{ ft}$$

[5.7.4.2]

Check minimum interface shear requirements:

The minimum requirement may be waived for girder-slab interfaces with the surface roughened to an amplitude of 0.25 in if the factored interface shear stress is less than 0.210 ksi.

$$v_{ui} = 0.31 \text{ ksi} > 0.210 \text{ ksi}$$

Then the minimum requirement cannot be waived.

The minimum required interface shear reinforcement is the lesser of:

$$A_{vfmin1} = \frac{0.05 \cdot b_v}{f_y} = \frac{0.05 \cdot 22}{60} = 0.018 \text{ in}^2/\text{ in} = 0.22 \text{ in}^2/\text{ ft}$$

or

$$A_{vfmin2} = \frac{1.33 \cdot V_{nireq} - c \cdot A_{cv}}{\mu \cdot f_y} = \frac{1.33 \cdot 90.9 - 0.28 \cdot 264}{1.0 \cdot 60}$$

$$= 0.78 \text{ in}^2/\text{ ft}$$

Then $A_{vfmin} = 0.22 \text{ in}^2/\text{ ft}$

The double leg #4 stirrup at 11″ spacing (A_v=0.44 in^2/ft) chosen earlier for vertical shear also meet the requirements for interface shear. Therefore, no additional reinforcement is required for interface shear.

Other sections are investigated similarly.

[5.7.3.5]

3. Minimum Longitudinal Reinforcement Requirement

The longitudinal reinforcement must be checked to ensure it is adequate to carry the tension caused by shear. The amount of strand development must be considered near the end of the beam. There are 2 cases to be checked:

Case 1: From the inside edge of bearing at the end supports out to the critical section for shear, the following must be satisfied, with $A_{ps} \cdot f_{ps}$ modified for development:

$$A_{ps} \cdot f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot\theta$$

A crack starting at the inside edge of the bearing sole plate will cross the center of gravity of the strands at:

$$x_{crack} = L_{soleplate} + y_{sstr} \cdot \cot(\theta) = 15 + 3.87 \cdot \cot(28.51) = 22.1 \text{ in}$$

The transfer length for 0.6" strands is: $\ell_{tr} = 36.0$ in

From the end of the beam to full transfer length, the strand stress increases linearly from zero to f_{pe}. Interpolate to find the tensile capacity of the bonded strands at the intersection with the assumed crack:

$$T_{r1} = A_{ps} \cdot f_{pe} \cdot \frac{x_{crack}}{\ell_{tr}} = 30 \cdot 0.217 \cdot 164.7 \cdot \frac{22.1}{36} = 658 \text{ kips}$$

The tension force to carry is:

$$T_{u1} = \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

$$= \left(\frac{285}{0.90} - 0.5 \cdot 171.9 - 0\right) \cdot \cot(28.51)$$

$$= 424.8 \text{ kips} < 658 \text{ kips} \quad \text{OK}$$

Case 2: At the critical section for shear, the following must be satisfied, with $A_{ps} \cdot f_{ps}$ modified for development:

$$A_{ps} \cdot f_{ps} \geq \frac{M_u}{\phi_f d_v} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

Use values calculated earlier to determine the tensile capacity at the critical section for shear:

$f_{ps}=285.7$ ksi

$A_{ps}= 6.51\ in^2$

Development fraction, $F_{dev}= 0.65$

$T_{r2} = A_{ps} \cdot f_{ps} \cdot F_{dev}= 6.51 \cdot 285.7 \cdot 0.65 = 1209$ kips

The factored moment M_u should be the moment concurrent with the factored shear V_u at x_{vcrit}. For simplicity, the maximum M_u at x_{vcrit} is used below.

Then the tension force to carry is:

$$T_{u2} = \frac{M_u}{\phi_f d_v} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

$$= \frac{1048 \cdot 12}{1.0 \cdot 42.8} + \left(\frac{285}{0.90} - 0.5 \cdot 171.9 - 0\right) \cdot \cot(28.51)$$

$T_{u2} = 718.6$ kips < 1209 kips OK

G. Design Pretensioned Anchorage Zone Reinforcement [5.9.4.4.1]

Splitting Reinforcement

To prevent cracking in the beam end due to the transfer of the prestressing force from the strands to the concrete, splitting reinforcement needs to be provided in the anchorage zone.

Use a load factor of 1.0 and lateral force component of 4% of the fully bonded strands to determine the required amount of steel.

The total prestressing force at transfer of bonded strands:

$$P_i = A_{ps} \cdot (f_{pj} - \Delta f_{pES}) = 6.51 \cdot (216.0 - 25.0) = 1243 \text{ kips}$$

The factored design splitting force is:

$$P_{split} = 1.0 \cdot 0.04 \cdot P_i = 1.0 \cdot 0.04 \cdot 1243 = 49.7 \text{ kips}$$

The amount of resisting reinforcement is determined using a steel stress f_s of 20 ksi:

$$A_s = \frac{P_b}{f_s} = \frac{49.7}{20} = 2.49\ in^2$$

This steel should be located at the end of the beam within a distance of:

$$\frac{h}{4} = \frac{40}{4} = 10 \text{ in}$$

The number of #5 double legged stirrups necessary to provide this area is:

$$\frac{A_s}{2 \cdot A_b} = \frac{2.49}{2 \cdot 0.31} = 4.02$$

The first set of stirrups (G505E) is located 2 inches from the end of the beam. See Figure 5.7.3.5.

Provide an additional four sets of #5 stirrups (G508E) spaced at 2 1/2 inch centers.

$x_{splitting} = 2 + 4 \cdot 2.5 = 12 \text{ in} > 10 \text{ in}$

Although the splitting reinforcement does not fit within h/4, #5 bars are the largest allowed and 2.5 inches is the tightest spacing allowed. This is OK per MnDOT practice.

[5.9.4.4.2]

Confinement Reinforcement

Reinforcement is required at the ends of the beam to confine the prestressing steel in the bottom flange. G303E bars (see Figure 5.7.2.5) will be placed at a maximum spacing of 6 inches out to 1.5d from the ends of the beam. For simplicity in detailing and ease of tying the reinforcement, space the vertical shear reinforcement with the confinement reinforcement in this area.

$$1.5\,d = 1.5 \cdot 40 = 60.0 \text{ in}$$

H. Determine Camber and Deflection
[2.5.2.6.2]
[3.6.1.3.2]
[5.6.3.5.2]

Camber Due to Prestressing and Dead Load Deflection

Using the memo to designers #2023-01, the camber due to prestress can be found. First, the strands must be separated into groups based on their debonded length. Then the eccentricity and prestress force just after transfer can be used to determine each group's contribution to release camber. The span length at release is the end-to-end length of the 119.25 feet since the beam is in the casting bed. Using the following equations, calculate the upward deflection values due to prestressing strand in Table 5.7.3.8.

Force in the strand:

$$P_t = A_{ps} \cdot \left(f_{pj} - \Delta f_{pES}\right)$$

Camber due to prestressing strands:

$$\Delta_{ps_{total}} = \sum_{i=1}^{n} \Delta_{ps_i}$$

$$\Delta_{ps_i} = \frac{P_{t_i} \cdot e_{s_i} \cdot L^2}{8 \cdot E_{ci} \cdot I} \qquad \text{(Straight Bonded Strands)}$$

$$\Delta_{ps_i} = \frac{P_{t_i} \cdot e_{s_i} \cdot \left[L^2 - \left(L_t + 2 \cdot L_{x_i}\right)^2\right]}{8 \cdot E_{ci} \cdot I} \qquad \text{(Debonded Strand)}$$

Where:

$\Delta_{ps_{total}}$ = upward camber of beam immediately after release, due to prestress alone (in)

Δ_{ps_i} = upward camber contribution immediately after release, due to individual strand group (in)

P_{t_i} = prestress force immediately after release of individual strand group (kips)

e_{s_i} = eccentricity of prestress force with respect to the beam centroid at midspan of individual strand group (in)

L = end to end length of beam (in)

L_t = transfer length of strand (in)

L_{x_i} = length of debonding from end of beam of individual strand group (in)

E_{ci} = modulus of elasticity of concrete at prestress transfer (ksi)

I = beam moment of inertia (in^4)

Table 5.7.3.8
Camber at Release Due to Prestressing Strands

Group	L_x (in)	e_s (in)	A_{ps} (in^2)	P_t (k)	Δ_{ps} (in)
1	0	14.20	6.51	1243	6.62 ①
2	168	12.74	1.30	248	1.11 ②
3	216	14.07	1.30	248	1.17 ②
4	264	16.07	1.30	248	1.26 ②
				Sum	10.16

① Calculated using "Straight Bonded Strands" equation.

② Calculated using "Debonded Strands" equation.

Downward deflection due to selfweight

$$\Delta_{sw} = \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I} = \frac{5 \cdot \frac{0.758}{12}(119.25 \cdot 12)^4}{384 \cdot 4578 \cdot 149{,}002} = 5.06 \text{ in}$$

Camber at release $\Delta_{rel} = \Delta_{ps} + \Delta_{sw} = 10.16 - 5.06 = 5.10$ in

To estimate camber at the time of erection the deflection components are multiplied by standard MnDOT multipliers. They are:

Release to Erection Multipliers:

Prestress = 1.4

Selfweight = 1.4

Camber and selfweight deflection values at erection are:

Prestress:	$1.4 \cdot 10.16 =$	14.22 in
Selfweight:	$1.4 \cdot (-5.06) =$	-7.08 in
Diaphragm DL:		-0.02 in
Deck and stool DL:		-5.12 in
Barrier:		-0.37 in

Note that the deflection values for diaphragms, deck, stool, and barrier are based on a span length of 118.0 feet.

The values to be placed in the camber diagram on the beam plan sheet are arrived at by combining the values above.

"Erection Camber" = 14.22 – 7.08 – 0.02 = 7.12 in say 7 1/8 in

"Est. Dead Load Deflection" = 5.12 + 0.37 = 5.49 in say 5 1/2 in

"Est. Residual Camber" = 7 1/8 – 5 1/2 = 1 5/8 in

Live Load Deflection

The deflection of the bridge is checked when subjected to live load and compared against the limiting values of $^{L}/_{800}$ for vehicle only bridges and $^{L}/_{1000}$ for bridges with bicycle or pedestrian traffic.

Deflection due to lane load is:

$$\Delta_{lane} = \left(\frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I}\right) = \left[\frac{5 \cdot \frac{0.64}{12} \cdot (118 \cdot 12)^4}{384 \cdot 4899 \cdot 396{,}823}\right] = 1.44 \text{ in}$$

Deflection due to a truck with dynamic load allowance is found using hand computations or computer tools to be:

$$\Delta_{truck} = 2.81 \text{ in}$$

Two deflections are computed and compared to the limiting values, that of the truck alone and that of the lane load plus 25% of the truck. Both deflections need to be adjusted with the live load distribution factor for deflection.

$$\Delta_1 = DF_\Delta \cdot \Delta_{truck} = 0.425 \cdot 2.81 = 1.19 \text{ in}$$

$$\Delta_2 = DF_\Delta \cdot (\Delta_{lane} + 0.25 \cdot \Delta_{truck}) = 0.425 \cdot (1.44 + 0.25 \cdot 2.81) = 0.91 \text{ in}$$

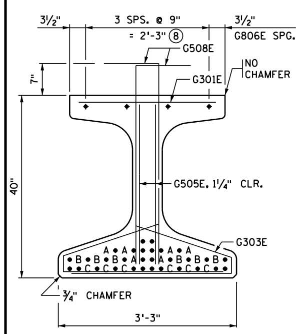
There is no bicycle or pedestrian traffic on the bridge, so the deflection limit is:

$$\frac{L}{800} = \frac{118 \cdot 12}{800} = 1.77 \text{ in} > \text{than } \Delta_1 \text{ or } \Delta_2 \qquad \text{OK}$$

I. Beam Sheet for Bridge Plan

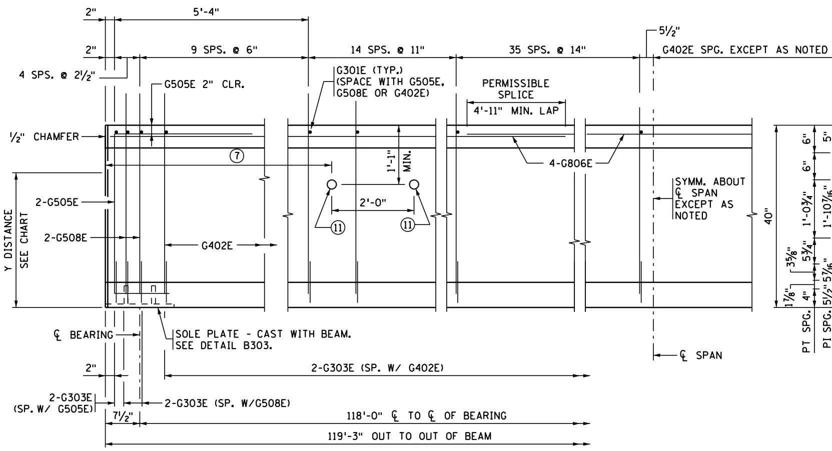
Figure 5.7.3.6 shows the detailed beam sheet for the debonded strand configuration that will be included in the bridge plan.

Figure 5.7.3.6

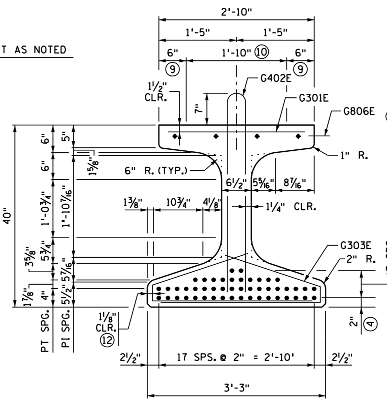


END VIEW

CUT STRANDS FLUSH WITH CONCRETE. COVER ENDS WITH SEALANT PER APPROVED PRODUCTS LIST "BRIDGE-PRESTRESSED BEAMS-CUT STRAND SEALANT."



BEAM ELEVATION



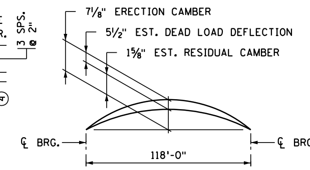
SECTION AT ℄ SPAN

Y DISTANCES

	NO.	Y (INCHES)
ROW 1 (BOTTOM)	18	2
ROW 2	18	4
ROW 3	10	6
ROW 4	2	8
⑤ TOTAL STRANDS	48	

Y = DISTANCE TO CENTER OF GRAVITY OF STRANDS FROM BOTTOM OF BEAM. ALL STRANDS SPACED 2" CENTER TO CENTER, HORIZONTALLY AND VERTICALLY, EXCEPT AS NOTED.

2 OPTIONAL ½" MIN. DIA. STRAIGHT STRANDS ARE NOT INCLUDED IN THIS TABLE.



CAMBER DIAGRAM

ERECTION CAMBER SHOWN IS AFTER DIAPHRAGMS ARE IN PLACE.

DEAD LOAD DEFLECTION SHOWN IS FOR WEIGHT OF SLAB, WEARING COURSE, BARRIER, SIDEWALK AND MEDIAN WHERE APPLICABLE.

CONTRACTOR WILL TAKE ELEVATIONS AT TOP OF BEAMS AFTER ERECTION AND WILL ALLOW FOR DEFLECTION SHOWN TO ENABLE BUILDING FORMS TO CORRECT GRADE AND SPECIFIED SLAB THICKNESS. PROVIDE COPY OF ELEVATIONS TO THE ENGINEER.

END DEBONDING

SYMBOL	NO.	LENGTH
A	6	14'-0"
B	6	18'-0"
C	6	22'-0"
TOTAL	18	

DEBONDING LENGTHS FOR A, B, AND C ARE MEASURED FROM EACH END OF BEAM.

INITIAL PRESTRESS
2,250,720 LB.
2 OPTIONAL ½" MIN. DIA. STRAIGHT TOP STRANDS ARE NOT INCLUDED IN THIS FORCE.
☒ $0.72 f_{pu}$
☐ $0.75 f_{pu}$

CALCULATED PRESTRESS LOSSES

ELASTIC SHORTENING LOSS	25.0 KSI
LONG TERM LOSSES	26.3 KSI
TOTAL LOSSES	51.3 KSI

MINIMUM CONCRETE STRENGTH - KSI

① f'ci	② f'c
8.0 KSI	9.5 KSI

GENERAL NOTES

PROVIDE HANDLING HOOKS OR DEVICES AS REQUIRED BY CONTRACTOR.

AT THE CONTRACTORS OPTION, 2 SYMMETRIC ½" MIN. DIA. STRANDS PULLED UP TO 5 KIPS EACH MAY BE ADDED TO THE TOP FLANGE. MAINTAIN THE VERTICAL PROFILE OF THESE STRANDS WITH A TOLERANCE OF ± 1" ALONG THE LENGTH OF THE BEAM AND MAINTAIN A MIN. OF 1¼" CLEAR COVER.

MARK EACH BEAM SHOWING BRIDGE NUMBER, CASTING DATE, AND INDIVIDUAL IDENTIFICATION LETTERS AND NUMBERS ON THE FACE OF THE BEAM, NEAR THE END, SO LOCATED THAT THEY WILL BE EXPOSED AFTER THE END DIAPHRAGMS HAVE BEEN CAST. MARK FASCIA BEAMS ON THE INSIDE FACE. ENSURE ALL MARKINGS ARE STENCILLED AND CLEARLY LEGIBLE. FOR LOCATION OF BEAMS, SEE FRAMING PLAN.

ALL MATERIAL AND WORK SHOWN OR NOTED ON THIS SHEET IS INCLUDED IN UNIT PRICE BID FOR PRESTRESSED CONCRETE BEAMS. SEE SPEC. 2405.

SEE FRAMING PLAN FOR BEAM END MARKED "X" AND DIAPHRAGM SPACING.

REFER TO "PRESTRESSED CONCRETE BEAM DIAPHRAGM CONNECTION DETAILS" SHEET INCLUDED IN THIS PLAN SET FOR CONCRETE END DIAPHRAGM AND STEEL INTERMEDIATE DIAPHRAGM DETAILS.

APPLY AN APPROVED SEALER TO THE SIDES OF THE BEAM NEAR EACH END PER THE SPECIAL PROVISIONS.

① MINIMUM CONCRETE STRENGTH AT TIME OF PRESTRESS TRANSFER.

② MINIMUM CONCRETE STRENGTH WHEN BEAM CAN BE TRANSPORTED AND INSTALLED.

~~③ DRAPED STRANDS.~~

④ STRAIGHT STRANDS.

⑤ USE MNDOT GRADE 300, 0.6" DIA. 7-WIRE LOW RELAXATION PRESTRESSING STRAND, IN ACCORDANCE WITH ASTM A416 AS MODIFIED IN THE SPECIAL PROVISIONS.

~~⑥ CENTER OF GRAVITY OF HOLD DOWNS WHEN MULTIPLE HOLD DOWNS ARE USED.~~

⑦ DIMENSION DETERMINED BY CONTRACTOR. MAINTAIN 2" MINIMUM CLEAR FROM STRANDS.

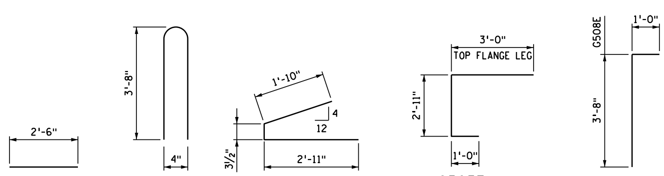
⑧ TWO INSIDE BARS MAY BE PLACED ADJACENT TO VERTICAL STIRRUP FOR TYING CONVENIENCE.

⑨ STEEL TROWEL TO SMOOTH FINISH AND APPLY BOND BREAKER PER APPROVED PRODUCTS LIST.

⑩ ROUGH FLOAT AND BROOM TRANSVERSELY FOR BOND PER SPEC. 2405.3D.

⑪ OPTIONAL: 2" MAX. DIA. SLEEVE FOR HAULING (AFTER INSTALLATION, COAT WITH APPROVED EPOXY BONDING AGENT & FILL WITH APPROVED NON-SHRINK GROUT).

⑫ TYP. CLR. DISTANCE FOR ENTIRE BOTTOM FLANGE.



G301E G402E G303E G505E G508E

MODIFICATIONS: REMOVED ALL REFERENCES TO DRAPED STRANDS AND INCORPORATED DEBONDED STRANDS.

REVISED: DECEMBER 21, 2022

APPROVED: DECEMBER 20, 2018
Kevin Western
STATE BRIDGE ENGINEER

REV. NO.	DATE	REVISION DESCRIPTION	BY

I HEREBY CERTIFY THAT THIS PLAN WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA

CERTIFIED BY ____ LICENSED PROFESSIONAL ENGINEER ____ DATE
NAME: ____ LIC. NO. ____

TITLE: 40MH PRESTRESSED CONCRETE BEAM (PRETENSIONED) 40MH-120

BEAMS B1 - B2

DES: DR: CHK: CHK:

SHEET NO. OF SHEETS

FIG. 5-397.503 MOD.

APPROVED:

BRIDGE NO. EXAMPLE

5.7.4 Three-Span Haunched Post-Tensioned Concrete Slab Design Example

[Future manual content]

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APPENDIX 5-A
MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

DECKS:

Top Transverse Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 for bar size and spacing. A Class A splice is provided where all top transverse bar splices occur between beams, with 50% of the bars spliced at a given location. A Class B splice is provided where 100% of the bars are spliced at a given location between beams or where 50% of the bars are spliced at a given location over beams. Avoid splicing 100% of bars over beams.

Top Transverse Deck Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	All Splices are Between Beams and 50% are at Same Location *(preferred)* (Class A)	100% of Splices at Same Location Between Beams or 50% of Splices at Same Location Over Beams (Class B)
3"	> 5"	#4	1'-6"	1'-11"
		#5	1'-10"	2'-5"
		#6	2'-2"	2'-10"
	5"	#4	1'-6"	1'-11"
		#5	1'-10"	2'-5"
		#6	2'-9"	3'-7"

Top Longitudinal Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 & Figure 9.2.1.8 or Table 9.2.1.2 & Figure 9.2.1.9 for bar size and spacing. Detail reinforcement such that no more than 50% of top longitudinal bars are spliced at any cross-section through the deck (Class A splice).

Top Longitudinal Deck Bar Lap Splice Lengths		
Concrete Cover to Bar Being Considered	Bar Size	Lap Splice Length (Class A)
≥ 3 1/2"	#4	1'-6"
	#5	1'-10"
	#6	2'-2"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

DECKS: (cont'd)

Bottom Transverse Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 for bar size and spacing. A Class A splice is provided where all bottom transverse bars are spliced over beams, with 50% of the bars spliced at a given location. A Class B splice is provided where 100% of the bars are spliced at a given location over beams or where 50% of the bars are spliced at a given location between beams. Avoid splicing 100% of bars between beams.

Bottom Transverse Deck Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	All Splices are Over Beams and 50% are at Same Location *(preferred)* (Class A)	100% of Splices at Same Location Over Beams or 50% of Splices at Same Location Between Beams (Class B)
1"	≥ 4"	#4	1'-10"	2'-5"
		#5	2'-9"	3'-6"
		#6	3'-9"	4'-10"

Bottom Longitudinal Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 & Figure 9.2.1.9 for bar size and spacing. A Class B splice is provided. Where possible, detail such that no more than 50% of the bottom longitudinal deck bars are spliced at a given cross-section through the deck.

Bottom Longitudinal Deck Bar Lap Splice Lengths			
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	50% of Splices at Same Location *(Preferred)* or 100% of Splices at Same Location (Class B)
≥ 1 1/2"	≥ 4"	#4	1'-11"
		#5	3'-0"
		#6	3'-7"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

ABUTMENTS:

Abutment and Wingwall Vertical Bars

Back face vertical bars are all spliced at the same location, so a Class B splice is used. See LRFD 5.10.8.4.3a. Front face bars are conservatively assumed to act as tension reinforcement, so compressive development lengths are not used in splice length computations. Although all front face bars are spliced at the same location, excess reinforcement is provided, so a Class A splice is used.

Abutment and Wingwall Vertical Bar Lap Splice Lengths					
Concrete Cover to Bar Being Considered	Bar Size	Back Face Bar Spacing (Class B)			Front Face Bar Spacing (Class A)
		4"	5"	≥6"	≥6"
≥ 2"	#4	--	--	--	1'-6"
	#5	3'-0"	2'-5"	2'-5"	1'-10"
	#6	3'-7"	3'-7"	3'-7"	2'-9"
	#7	4'-6"	4'-2"	4'-2"	3'-2"
	#8	5'-11"	4'-9"	4'-9"	3'-8"
	#9	7'-6"	6'-0"	5'-10"	--
	#10	9'-6"	7'-7"	7'-2"	--
	#11	11'-8"	9'-4"	8'-8"	--
	#14	--	13'-5"	11'-10"	--

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

ABUTMENTS: (cont'd)

Abutment and Wingwall Horizontal Bars

All horizontal bars are assumed to have more than 12" of concrete cast below. For integral abutment stem and diaphragm and for semi-integral abutment diaphragm, horizontal bars resist passive pressure loads, so a Class B splice is used. For parapet abutment stem and backwall and semi-integral abutment stem, horizontal bars are assumed to provide excess reinforcement, so a Class A splice is used. For long wingwalls on separate footings, horizontal bars become primary reinforcement, so a Class B splice is used.

Abutment and Wingwall Horizontal Bar Lap Splice Lengths					
Concrete Cover to Bar Being Considered	Bar Size	Integral Abutment Stem & Diaphragm and Semi-Integral Abutment Diaphragm Horizontal Bar Spacing (Class B)	Parapet Abutment Stem & Backwall and Semi-Integral Abutment Stem Horizontal Bar Spacing (Class A)	Wingwall Horizontal Bar Spacing (Class B)	
		≥ 6	≥ 6"	4"	≥ 5"
≥ 2"	#4	2'-6"	1'-11"	2'-6"	2'-6"
	#5	3'-1"	2'-5"	3'-4"	3'-1"
	#6	4'-0"	3'-1"	4'-0"	4'-0"
	#7	4'-8"	3'-7"	5'-1"	4'-8"
	#8	5'-4"	4'-1"	6'-8"	5'-4"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

PIERS:

Pier Cap Top Longitudinal Bars

All horizontal bars are assumed to have more than 12" of concrete cast below. For splices between columns (or between piles for a pile bent pier) where no more than 50% of the bars are spliced at the same location, a Class A splice is used. For all other cases, use a Class B splice.

Pier Cap Top Longitudinal Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	All Splices Located Between Columns and ≤ 50% of Bars Are Spliced at Same Location (Class A)				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
≥ 2 3/8"	#5	2'-7"	2'-5"	2'-5"	2'-5"	2'-5"
	#6	3'-1"	3'-1"	2'-10"	2'-10"	2'-10"
	#7	3'-11"	3'-7"	3'-7"	3'-7"	3'-7"
	#8	5'-2"	4'-1"	4'-1"	4'-1"	4'-1"
	#9	6'-6"	5'-3"	4'-9"	4'-8"	4'-8"
	#10	8'-3"	6'-7"	6'-0"	5'-6"	5'-6"
	#11	10'-2"	8'-2"	7'-5"	6'-10"	6'-8"
	#14	--	11'-9"	10'-8"	9'-9"	9'-1"
	Bar Size	All Splices Located Between Columns and > 50% of Bars Are Spliced at Same Location (Class B)				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
	#5	3'-4"	3'-1"	3'-1"	3'-1"	3'-1"
	#6	4'-0"	4'-0"	3'-8"	3'-8"	3'-8"
	#7	5'-1"	4'-8"	4'-8"	4'-8"	4'-8"
	#8	6'-8"	5'-4"	5'-4"	5'-4"	5'-4"
	#9	8'-6"	6'-9"	6'-2"	6'-0"	6'-0"
	#10	10'-9"	8'-7"	7'-10"	7'-2"	7'-2"
	#11	13'-3"	10'-7"	9'-8"	8'-10"	8'-7"
	#14	--	15'-3"	13'-10"	12'-9"	11'-10"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

PIERS: (cont'd)

Pier Cap Bottom Longitudinal Bars

For splices over columns (or over piles for a pile bent pier) where no more than 50% of the bars are spliced at the same location, a Class A splice is used. For all other cases, use a Class B splice.

Pier Cap Bottom Longitudinal Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	All Splices Located Over Columns and ≤ 50% of Bars Are Spliced at Same Location (Class A)				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
≥ 2 3/8"	#5	2'-3"	1'-10"	1'-10"	1'-10"	1'-10"
	#6	2'-9"	2'-9"	2'-2"	2'-2"	2'-2"
	#7	3'-6"	3'-2"	3'-2"	3'-2"	3'-2"
	#8	4'-6"	3'-8"	3'-8"	3'-8"	3'-8"
	#9	5'-9"	4'-7"	4'-2"	4'-1"	4'-1"
	#10	7'-4"	5'-10"	5'-4"	4'-11"	4'-10"
	#11	9'-0"	7'-2"	6'-7"	6'-0"	5'-10"
	#14	--	10'-4"	9'-5"	8'-8"	8'-1"
	Bar Size	All Splices Located Over Columns and > 50% of Bars Are Spliced at Same Location (Class B)				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
	#5	3'-0"	2'-5"	2'-5"	2'-5"	2'-5"
	#6	3'-7"	3'-7"	2'-10"	2'-10"	2'-10"
	#7	4'-6"	4'-2"	4'-2"	4'-2"	4'-2"
	#8	5'-11"	4'-9"	4'-9"	4'-9"	4'-9"
	#9	7'-6"	6'-0"	5'-5"	5'-4"	5'-4"
	#10	9'-6"	7'-7"	6'-11"	6'-4"	6'-4"
	#11	11'-8"	9'-4"	8'-6"	7'-10"	7'-7"
	#14	--	13'-5"	12'-3"	11'-3"	10'-5"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

PIERS: (cont'd)

Other Pier Cap Longitudinal Bars Located on Side Faces of Pier Cap

Longitudinal bars located on the side faces of pier caps (typically skin or shrinkage and temperature reinforcement) are assumed to have more than 12" of concrete cast below. For these bars, a Class B splice is used.

Lap Splice Lengths for Longitudinal Bars Located on Side Faces of Pier Cap		
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing ≥ 4" (Class B)
≥ 2 3/8"	#4	2'-6"
	#5	3'-4"
	#6	4'-0"
	#7	5'-1"

Pier Column Vertical Bars

For pier columns, all splices occur at the same location, so a Class B splice is used.

Pier Column Vertical Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing (Class B)				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
≥ 2 3/8"	#6	3'-7"	3'-7"	2'-10"	2'-10"	2'-10"
	#7	4'-6"	4'-2"	4'-2"	4'-2"	4'-2"
	#8	5'-11"	4'-9"	4'-9"	4'-9"	4'-9"
	#9	7'-6"	6'-0"	5'-5"	5'-4"	5'-4"
	#10	9'-6"	7'-7"	6'-11"	6'-4"	6'-4"
	#11	11'-8"	9'-4"	8'-6"	7'-10"	7'-7"
	#14	--	13'-5"	12'-3"	11'-3"	10'-5"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

SLAB SPANS:

Top Bars

This table applies to both top longitudinal and transverse bars. All bars are assumed to have more than 12" of concrete cast below. A Class B splice is used.

Top Longitudinal and Transverse Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing (Class B)				
		4"	5"	6"	7"	≥ 8"
≥ 3"	#4	2'-6"	2'-6"	2'-6"	2'-6"	2'-6"
	#5	3'-4"	3'-1"	3'-1"	3'-1"	3'-1"
	#6	4'-0"	4'-0"	3'-8"	3'-8"	3'-8"
	#7	5'-1"	4'-8"	4'-8"	4'-4"	4'-4"
	#8	6'-8"	5'-4"	5'-4"	4'-11"	4'-11"
	#9	8'-6"	6'-9"	6'-0"	6'-0"	6'-0"
	#10	10'-9"	8'-7"	7'-2"	6'-9"	6'-9"
	#11	13'-3"	10'-7"	8'-10"	7'-7"	7'-6"
	#14	--	15'-3"	12'-9"	10'-11"	9'-11"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

Notes: > Splice lengths are based on BDM Tables 5.2.2.1 and 5.2.2.2.
> Assumes use of epoxy coated bars. Excess reinforcement factor λ_{er} = 1.0.
> Class A splice length is equivalent to the rebar development length, ℓ_d.

SLAB SPANS: (cont'd)

Bottom Bars

The table applies to both bottom longitudinal and transverse bars. A Class B splice is used.

Bottom Longitudinal and Transverse Bar Lap Splice Lengths			
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing (Class B)	
		4"	≥ 5"
≥ 1 1/2"	#4	1'-11"	1'-11"
	#5	3'-0"	3'-0"
	#6	3'-7"	3'-7"
	#7	4'-8"	4'-8"
	#8	5'-11"	5'-11"
	#9	7'-6"	7'-3"
	#10	9'-6"	8'-11"
	#11	11'-8"	10'-7"
	#14	--	14'-4"

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9. DECKS AND DECK SYSTEMS

Reinforced concrete decks on girders are the predominant type of deck used on highway bridges in Minnesota. The deck is the structural element that transfers vehicle and pedestrian loads to the girders. It is analyzed as a continuous beam with the girders acting as supports. The top and bottom primary moment resisting reinforcement runs transversely in the deck. The concrete stool between the girder top flange and the deck bottom varies to allow placement of the deck to the proper elevation.

Only reinforced concrete decks supported on girders are covered in this section. Practices for slab type bridges, where the superstructure does not contain girder supports, are located in Article 5.3 of this manual.

Timber decks may be used on secondary roads and temporary bridges as part of the superstructure. Guidance for the design of timber decks is provided in Section 8.

Specialized deck systems are used for railroad bridges. A common design is a thru-girder system with floor beams supporting a bent plate. This channel shaped bent plate holds the ballast on which the rails are supported. These specialized deck systems are not currently covered in this manual.

9.1 General

Bridge Deck Protection Policy

Refer to BDM Article 2.4.1.1.2 for the bridge deck protection policy.

9.1.1 Deck Drainage

Deck Drainage Considerations

The design of a deck requires:

- Removing water from the driving surface using a crown cross-section to protect against potential hydroplaning.
- Channeling drainage water away from the bridge and features below the bridge using road grades and end slopes respectively.

Deck drains and drainage systems on bridges are strongly discouraged due to their high maintenance requirements. Debris tends to build up in the drains, causing plugging of the system. Drainage systems are also prone to leakage, which is especially a problem for box type structures where the system runs inside the box. Bridges with lengths less than 500 feet that are located over lakes or streams can usually be designed such that deck drains are not necessary. Bridges that are longer than 500 feet may have problems with deck flooding in severe rainstorms, and may require deck drains. The Bridge Waterways Unit will work with the Bridge Preliminary

Plans Unit to investigate the need for deck drains and include the requirements, if any, in the Preliminary Bridge Plan.

Superstructure Drains
When drainage systems are required on bridges, avoid direct runoff into "waters of the state", as defined in *Mn. Statute 115.01, Subdivision 22*.

Extend drains 6 inches below the superstructure to prevent corrosion and deterioration of the superstructure from wind-blown water on bridges where deicing chemicals are applied. Drains need only be extended 1 inch below the bottom of the superstructure where deicing chemicals are not used (typically only on local road system). See Standard Bridge Details B701, B702, B703, B705, and B706.

Avoid drain outlets over roadways, shoulders, sidewalks/trails, streams, railroad tracks, and end slopes. If drains are required, ensure that proper clearance is maintained for drains placed over roadways, including consideration of future roadway expansion beneath a bridge. If drain outlets cannot be avoided over waterways, consult with Bridge Hydraulics and District Bridge Maintenance to determine if potential impact/damming from flowing debris or ice can occur at the site, and revise drain extension length noted above as needed.

Drains placed over riprap require the area to be grouted, or a grouted flume section provided. At down spouts or deck drains, provide splash blocks, including locations where water drains onto concrete slope paving.

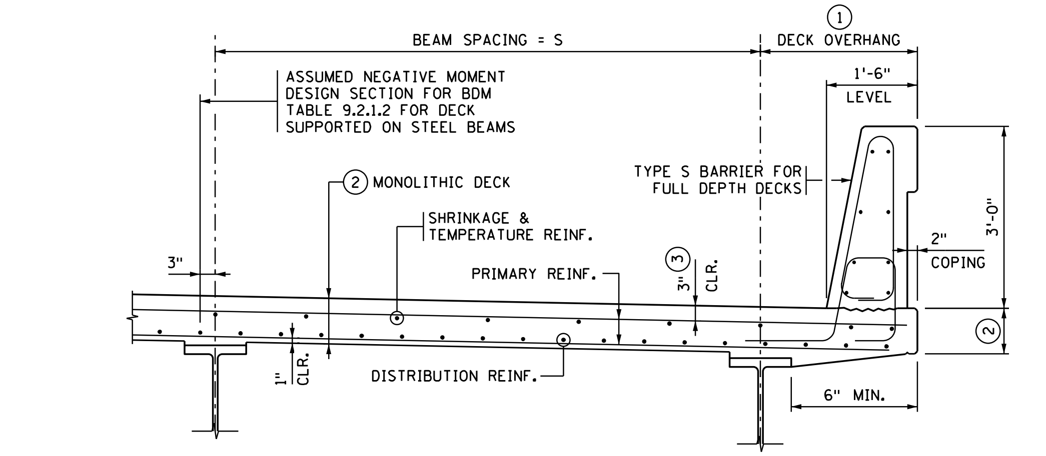
Avoid drainage details that include flat elements (grades less than 5%). Pipes and drainage elements with flat profiles tend to collect debris and plug.

Note that special drainage requirements are necessary for bridges where a Corps of Engineers "404 permit" is required. The Bridge Waterways Unit may also require the addition of containment and treatment features to the project for bridges located in or near scenic waterways or near public water supply sources.

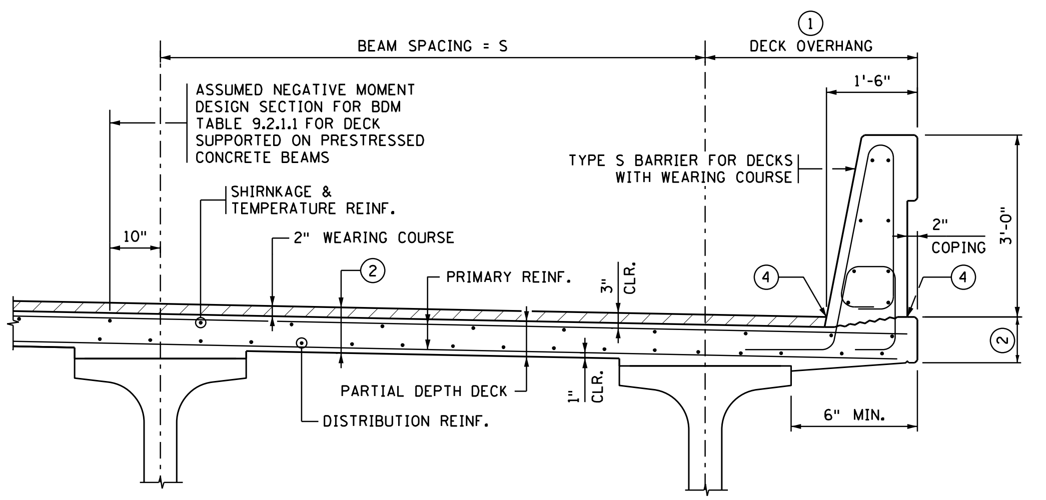
Provide the materials and gages for corrugated metal (C.M.) drains and semi-circle deck drains, such as those used on railroad bridges, in the plan details.

9.2 Concrete Deck on Beams

Figure 9.2.1 illustrates the two most common concrete deck systems used. See the deck protection policy in BDM Article 2.4.1.1.2 for determination of which deck system to use for a given bridge project. The upper portion of the figure shows a monolithic deck constructed with a single concrete pour. The lower portion illustrates a deck with a wearing course.



FULL DEPTH DECK SYSTEM



PARTIAL DEPTH DECK + 2" WEARING COURSE

GEOMETRY AND DESIGN ASSUMPTIONS FOR MOST COMMON CONCRETE DECK SYSTEMS

NOTES:

EITHER DECK SYSTEM CAN BE USED WITH EITHER BEAM TYPE.

REINFORCEMENT CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER MAY DIFFER FOR STAINLESS OR GFRP BARS.

1. PER BDM ARTICLE 2.4.1.1.1 GENERALLY LIMIT THE OVERHANG TO THE SMALLEST OF:
   - BEAM DEPTH
   - 40% OF BEAM SPACING
   - DECK COPING WIDTH + BARRIER WIDTH + 1'-0" + ½ FLANGE WIDTH
2. FOR VEHICULAR BRIDGES, REQ'D. DECK THICKNESS AND EDGE OF DECK THICKNESS IS DEPENDENT ON BEAM SPACING (SEE TABLES 9.2.1.1 & 9.2.1.2). FOR PEDESTRIAN BRIDGES, USE 7" DECK THICKNESS WITH NO WEARING COURSE.
3. FOR PEDESTRIAN BRIDGES, PROVIDE 2" CLEAR COVER.
4. GUTTER LINE AND COPING ELEVATION TO MATCH.

Figure 9.2.1

9.2.1 Deck Design and Detailing

Design

The default reinforcement bar type used in concrete bridge decks is an epoxy coated bar with a yield strength, F_y, equal to 60 ksi which meets the material requirements of ASTM A615.

In special cases, as outlined in Technical Memorandum No. 17-02-B-01, use stainless steel reinforcement and design accordingly. Note that stainless steel bars have a higher yield strength and different clear cover requirements.

Use of glass fiber reinforced polymer (GFRP) reinforcement bars has been limited to a small number of specific projects and is not to be specified unless approved by the State Bridge Design Engineer.

For design of conventionally reinforced concrete decks, the following requirements apply:

- Use the traditional approximate method of analysis for design of the top and bottom transverse reinforcement. Do not use the empirical deck design method in LRFD Article 9.7.2.
- For analysis, assume the deck is a continuous transverse strip with the beams below as supports.
- For skews less than or equal to 20°, detail deck transverse bars parallel to the skew. For design of the transverse bars, use the beam spacing measured along the skew for the deck span length.
- For skews greater than 20°, detail deck transverse bars at right angles to the centerline of roadway. For design of the transverse bars, use the beam spacing measured normal to the roadway centerline for the deck span length.
- For dead load, include deck self-weight plus a future wearing course of 20 psf. Apply a load factor of 1.25.
- If LRFD Appendix A4 assumptions and limitations are met, use the live load moments provided in LRFD Table A4-1 for design. Apply negative moment live load at the design section specified in LRFD Article 4.6.2.1.6.
- For decks without a wearing course, assume ½ inch of wear when determining structural depth, d, for the bottom transverse reinforcement. For decks with a wearing course, do not include the wearing course (sacrificial) when determining structural depth, d, for the bottom transverse reinforcement.
- Check crack control per LRFD Article 5.6.7 using the Class 2 exposure condition (γ_e = 0.75).

- For LRFD Article 5.6.7 crack control check of top transverse bars:
  - Although actual top concrete clear cover may exceed 2 inches, calculate center of bar cover, d_c, for top transverse bars using a maximum clear concrete cover equal to 2 inches.
  - For determination of strain ratio, β_s, assume 0.5 inches of wear for calculation of overall deck thickness, h.
- For bottom longitudinal reinforcement, provide distribution reinforcement per LRFD Article 9.7.3.2. For bridges with varying beam spacing, base the distribution reinforcement for each unit (where a unit is defined as the number of spans between expansion joints) on the widest beam spacing found within the unit.
- For the deck region in non-pier areas, provide top longitudinal reinforcement that meets the requirements for shrinkage and temperature reinforcement in LRFD Article 5.10.6.
- For the deck region over/near a pier, provide top longitudinal reinforcement consistent with the superstructure modeling assumptions:
  - Where deck is continuous, but beams are not continuous, provide reinforcement per Figure 9.2.1.8.
  - Where deck and prestressed beams are continuous, design reinforcement for factored negative moment.
  - Where deck and steel beams are continuous, design reinforcement for factored negative moment and meet requirements of LRFD Article 6.10.1.7. See Figure 9.2.1.9 for additional information.
- See Memo to Designers #2020-01 for guidance on deck overhang design.

Tables 9.2.1.1 and 9.2.1.2 provide minimum reinforcement requirements based on the traditional deck design method for decks supported on precast pretensioned concrete beams and steel beams, respectively. The tables may be used for all LRFD deck designs that fit the assumptions, as well as for decks of bridges originally designed by the AASHTO Standard Specifications Load Factor method (bridge widenings).

See Memo to Designers #2020-01 for discussion on requirements for deck overhang reinforcement when Tables 9.2.1.1 and 9.2.1.2 are used for the deck design.

Decks with geometry or loads that fall outside the Table 9.2.1.1 and 9.2.1.2 assumptions require a special design.

Geometry

Figures 9.2.1.4 through 9.2.1.7 show standard practice deck details. Typical deck reinforcement layouts at deck edges and medians are illustrated in the figures.

Use a uniform deck thickness for all spans based on the minimum thickness required for the widest beam spacing. For new bridges, use a 9 inch minimum deck thickness on all vehicular structures and a 7 inch minimum deck thickness on pedestrian bridges. For bridge repair projects on vehicular bridges, a lesser deck thickness (8 inch minimum) may be used when approved by the Regional Bridge Construction Engineer to achieve an acceptable load rating.

For the edge-of-deck thickness, use a uniform thickness in all spans. Use an edge-of-deck thickness that is equal to the deck thickness specified in BDM Tables 9.2.1.1 and 9.2.1.2 (typically 9", except for wide beam spacings). In the special case where a deck thickness less than 9" is specified (e.g., redecking of a bridge with existing deck thickness equal to 8½"), provide an edge-of-deck thickness equal to 9".

The standard height for bridge sidewalks at the gutter line is 6 inches above the top of roadway. For bridge medians, match approach roadway median shape and height as shown in the preliminary bridge plan.

Dimension the bottom of deck on the outside of the fascia beam at 1 inch below the top of the beam for prestressed concrete beams. For steel beams, detail the bottom of deck on the outside of the fascia beam to meet the bottom of the top flange. See Figures 9.2.1.4 through 9.2.1.7.

Check the slope of the bottom of the deck on overhangs. Confirm that the bottom edge of the deck is higher than the location next to the beam top flange.

Detailing

For main transverse deck reinforcement, provide straight bars located in both the top and the bottom reinforcing mats. Refer to Memo to Designers #2020-01 for some exceptions to this in the deck overhang where hooked bars are required.

The main transverse reinforcement will vary with the beam spacing. For skewed bridges where the beam spacing changes from one span to another, continue the reinforcement for the wider beam spacing until the reinforcement is completely outside of the span with the wider beam spacing.

For the acute corners of highly skewed bridges, detail the deck reinforcement as follows: In addition to the 2-#5 bars that run parallel to the expansion joint at the end of the deck, place 2 top mat #5 bars that are 10 feet long and run parallel to the joint with a spacing of 5 inches. Also, run a series of radial transverse bars that shorten as they progress into the corner. Finally, place a bent bar in the corner that ties to the outside deck longitudinal bar and the end bar running parallel to the joint. See Figure 9.2.1.1.

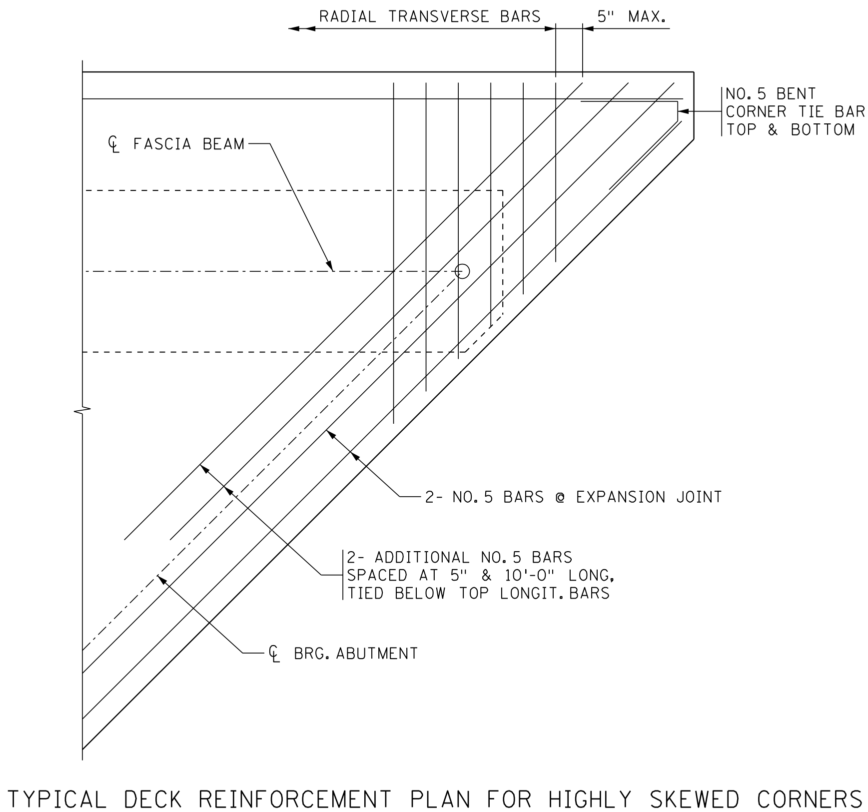


Figure 9.2.1.1

Add a longitudinal tie at the end of the deck if the deck projects past the end of the diaphragm more than 1 foot.

For bridges with transverse deck reinforcement parallel to the skew, dimension transverse bar spacing along edge of deck.

Several detailing practices are to be used near piers:

- Detail longitudinal steel (temperature and distribution) as continuous over piers, excluding raised sidewalks as stated below.
- Provide additional longitudinal steel to minimize transverse deck cracking. See Figures 9.2.1.8 and 9.2.1.9.

- For decks supported on prestressed concrete beams without a continuity diaphragm:
  - Detail a control joint consisting of a partial depth sawcut in the deck at the centerline of all piers and fill with a sealant to produce a controlled crack. See Figure 9.2.1.10.
  - For bridges with skewed piers, place polystyrene on the corners of the beams to reduce wandering of the controlled transverse deck crack at the centerline of pier. See Figure 9.2.1.10.
  - For decks with raised sidewalks or medians that cross the pier, provide a 1 inch deep by ⅜ inch wide V shaped control joint formed using a hand trowel in sidewalks or medians at the centerline of all piers. Do not include control joints at other locations along raised sidewalks or medians. Detail the longitudinal reinforcement in raised sidewalks as discontinuous through the control joint over the pier. Seal raised sidewalk control joints in accordance with Spec. 3722.

Deck Placement Sequence

One contributor to through-deck transverse cracking is inadequate sequencing of deck pours. Provide a deck placement sequence for the following types of bridges:

- Bridges with decks wider than 90 feet.
- Continuous bridges with spans exceeding 150 feet.
- Bridges where the concrete placement rate is lower than 60% of the span length per hour. (Note that a single pump truck can be assumed to maintain a pour rate of 70 cubic yards per hour.)

The overall goal of the deck placement sequence is have minimal tension in the deck due to self-weight when the entire deck is complete.

Generally, for continuous superstructures containing span lengths between 150 and 200 feet, locate the transverse construction joint for the first pour at the 0.6 point of the first span. Start the following pour at the 0.6 point of the adjacent span and proceed toward and terminate at the end of the previous pour. Continue this pattern for all interior spans. The last placement will extend from the end of the bridge to the previous placement. A typical deck placement sequence for a 3 span bridge fitting the above criteria is shown in Figure 9.2.1.2.

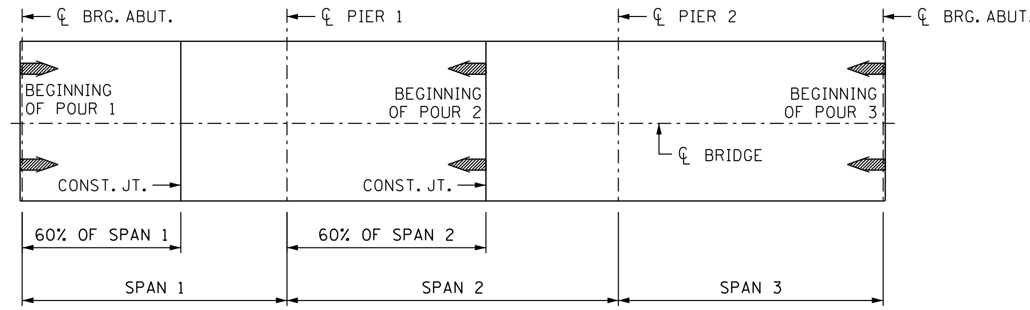


Figure 9.2.1.2

For continuous superstructures containing span lengths greater than 200 feet, conduct an analysis to determine construction joint locations for the deck pour sequence. For the initial trial, set the joint locations at points of dead load contraflexure. Choose a pour sequence that minimizes upward deflections in previously placed spans (i.e. longer pour sections should be placed before shorter adjacent sections). Place positive moment sections prior to negative moment sections. Next, analyze the deck for the initial trial pour sequence to determine the cumulative stresses in the deck. Then begin adjusting construction joint locations and reanalyzing until the pour sequence with the lowest tension stresses in the deck is reached. An acceptable pour sequence for a multi-span bridge fitting the above criteria is shown in Figure 9.2.1.3. Since adjacent spans may not be poured within 72 hours of each other, the second pour is permitted to be the next most flexible section after the first pour. Note that the third and fourth pours require placement of both positive and negative moment sections. If the contractor will be unable to complete the placement of the entire section in one pour, the positive moment area is to be placed first followed by the negative sections.

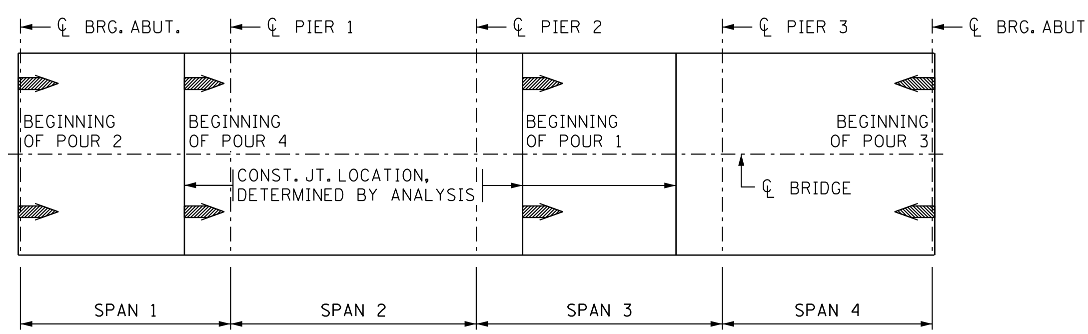


Figure 9.2.1.3

For superstructures which consist of a series of simply supported spans that require a deck placement sequence, locate transverse construction joints at the end of a span.

Where possible, consider orienting the direction of the concrete pours in the uphill direction, allowing gravity to reduce possible tension in the deck.

On bridges with strip seal expansion joints where a deck pour begins at the abutment, investigate the effects of beam end rotation. Too much rotation can negatively affect the joint size or cause deck and end block formwork to fail if not appropriately handled during concrete placement. If this is a problem, consider revising the pour sequence or placing the abutment end block after the deck is complete.

In all cases, a minimum of 72 hours is required between adjacent deck pours.

For unusual span length configurations, discuss the deck placement sequence with the Regional Bridge Construction Engineer.

Design Assumptions for Deck Reinforcement Tables 9.2.1.1 and 9.2.1.2:

- Transverse reinforcement is based on the traditional deck design method.
- Concrete strength, f'_c = 4 ksi.
- Epoxy coated steel rebar strength, f_y = 60 ksi.
- Dead load includes deck self-weight plus a future wearing course of 0.020 ksf. A load factor of 1.25 was applied to the future wearing course. Dead load bending moment is based on equation $M_{DC} = w_{DC} \cdot L^2/10$.
- Positive live load moments were taken from LRFD Table A4-1.
- Negative live load moments were interpolated from values in LRFD Table A4-1.
- Concrete clear cover for top transverse reinforcement is 3”.
- Concrete clear cover for bottom transverse reinforcement is 1”.
- For decks without a wearing course, ½” wear was assumed in determining the structural depth, d_s, for the bottom transverse reinforcement.
- For decks with a wearing course, the 2" wearing course was not used (sacrificial) in determining structural depth, d_s, for the bottom transverse reinforcement.
- Crack control used a Class 2 exposure condition (γ_e = 0.75).
- For crack control check, although the actual concrete clear cover to top transverse bars exceeds 2 inches, d_c calculation used a maximum clear concrete cover equal to 2 inches.
- LRFD Art. 9.7.2.4 (under empirical design) requires that the ratio of the effective beam spacing to slab thickness be less than 18. The slab thicknesses given in the tables fit these requirements and are similar to what MnDOT has used successfully in the past.
- Bottom longitudinal reinforcement is distribution reinforcement per LRFD Article 9.7.3.2.

EPOXY COATED STEEL REINFORCEMENT FOR DECKS SUPPORTED ON PRECAST PRETENSIONED CONCRETE BEAMS

Negative moment design section is assumed at 10 inches from centerline for I-beams, based on $^{1}/_{3}$ of the M-series beam top flange per LRFD Article 4.6.2.1.6 (conservative for MN-series and MW-series beams). Similarly, negative moment design section is assumed at 8.7 inches from centerline for rectangular beams, which is based on 1/3 of the rectangular beam width.

Maximum Beam Spacing ①	Transverse Reinforcement Size and Spacing				Deck Thickness T ②	Longitudinal Reinforcement Size and Spacing, Bottom Mat ③	Longitudinal Reinforcement Size and Spacing, Top Mat ③
	Bottom Mat		Top Mat				
	With Wearing Course	Without Wearing Course	Deck on I-Beam	Deck on Rectangular Beam			
5'-0"	4 @ 6.5"	4 @ 9"	4 @ 10"	4 @ 9.5"	9''	4 @ 9"	4 @ 1'-6"
5'-6"	4 @ 6.5"	4 @ 8.5"	4 @ 9"	4 @ 8.5"	9''	4 @ 9"	4 @ 1'-6"
6'-0"	4 @ 6.5"	4 @ 8"	4 @ 8.5"	4 @ 8"	9''	4 @ 9"	4 @ 1'-6"
6'-6"	4 @ 6"	4 @ 8"	4 @ 8"	4 @ 7.5"	9''	4 @ 9"	4 @ 1'-6"
7'-0"	4 @ 5.5"	4 @ 7.5"	4 @ 7.5"	4 @ 7"	9''	4 @ 8"	4 @ 1'-6"
7'-6"	4 @ 5.5"	4 @ 7"	4 @ 7"	4 @ 6.5"	9''	4 @ 8"	4 @ 1'-6"
8'-0"	4 @ 5"	4 @ 6.5"	4 @ 6.5"	4 @ 6.5"	9''	4 @ 7"	4 @ 1'-6"
8'-6"	5 @ 7.5"	4 @ 6"	4 @ 6.5"	4 @ 6"	9''	4 @ 7"	4 @ 1'-6"
9'-0"	5 @ 7"	4 @ 6"	4 @ 6"	4 @ 6"	9''	5 @ 10"	4 @ 1'-6"
9'-6"	5 @ 6.5"	4 @ 5.5"	4 @ 6"	4 @ 5.5"	9''	5 @ 9"	4 @ 1'-6"
10'-0"	5 @ 6"	4 @ 5.5"	4 @ 5.5"	4 @ 5"	9''	5 @ 8"	4 @ 1'-6"
10'-6"	5 @ 6"	4 @ 5"	4 @ 5"	5 @ 6.5"	9''	5 @ 8"	4 @ 1'-6"
11'-0"	5 @ 5.5"	5 @ 7.5"	5 @ 6"	5 @ 6"	9''	5 @ 8"	4 @ 1'-6"
11'-6"	5 @ 5.5"	5 @ 7"	5 @ 5.5"	5 @ 5.5"	9''	5 @ 8"	4 @ 1'-6"
12'-0"	5 @ 5"	5 @ 6.5"	5 @ 5.5"	5 @ 5.5"	9''	5 @ 7"	4 @ 1'-6"
12'-6"	6 @ 7"	5 @ 6.5"	5 @ 5"	5 @ 5"	9''	5 @ 7"	4 @ 1'-6"
13'-0"	6 @ 7"	5 @ 6.5"	5 @ 5"	5 @ 5"	9.5''	5 @ 7"	4 @ 1'-6"
13'-6"	6 @ 7.5"	5 @ 6.5"	5 @ 5"	5 @ 5"	9.75''	5 @ 8"	4 @ 1'-6"
14'-0"	6 @ 7"	5 @ 6.5"	5 @ 5"	6 @ 6"	10''	5 @ 8"	4 @ 1'-6"
14'-6"	6 @ 7.5"	5 @ 6.5"	5 @ 5"	6 @ 6"	10.25''	5 @ 8"	4 @ 1'-6"
15'-0"	6 @ 7.5"	5 @ 6.5"	5 @ 5"	6 @ 6"	10.5''	5 @ 8"	4 @ 1'-6"

① For skews ≤ 20°, beam spacing is measured along the skew.
For skews > 20°, beam spacing is measured normal to roadway centerline.

② Deck thickness includes wearing course.

③ Reinforcement shown is for deck regions in non-pier areas only and is based on LRFD 5.10.6. Note that additional reinforcement is required for deck regions over/near piers. See Figure 9.2.1.8 for additional top longitudinal reinforcement required in deck regions over/near piers when only deck is continuous. For beams made continuous, design longitudinal reinforcement in deck regions over/near piers for factored negative moment.

Table 9.2.1.1

EPOXY COATED STEEL REINFORCEMENT FOR DECKS SUPPORTED ON STEEL BEAMS

Negative moment design section is assumed at 3 inches from centerline of beam, based on 1/4 of a 12 inch top flange per LRFD Article 4.6.2.1.6.

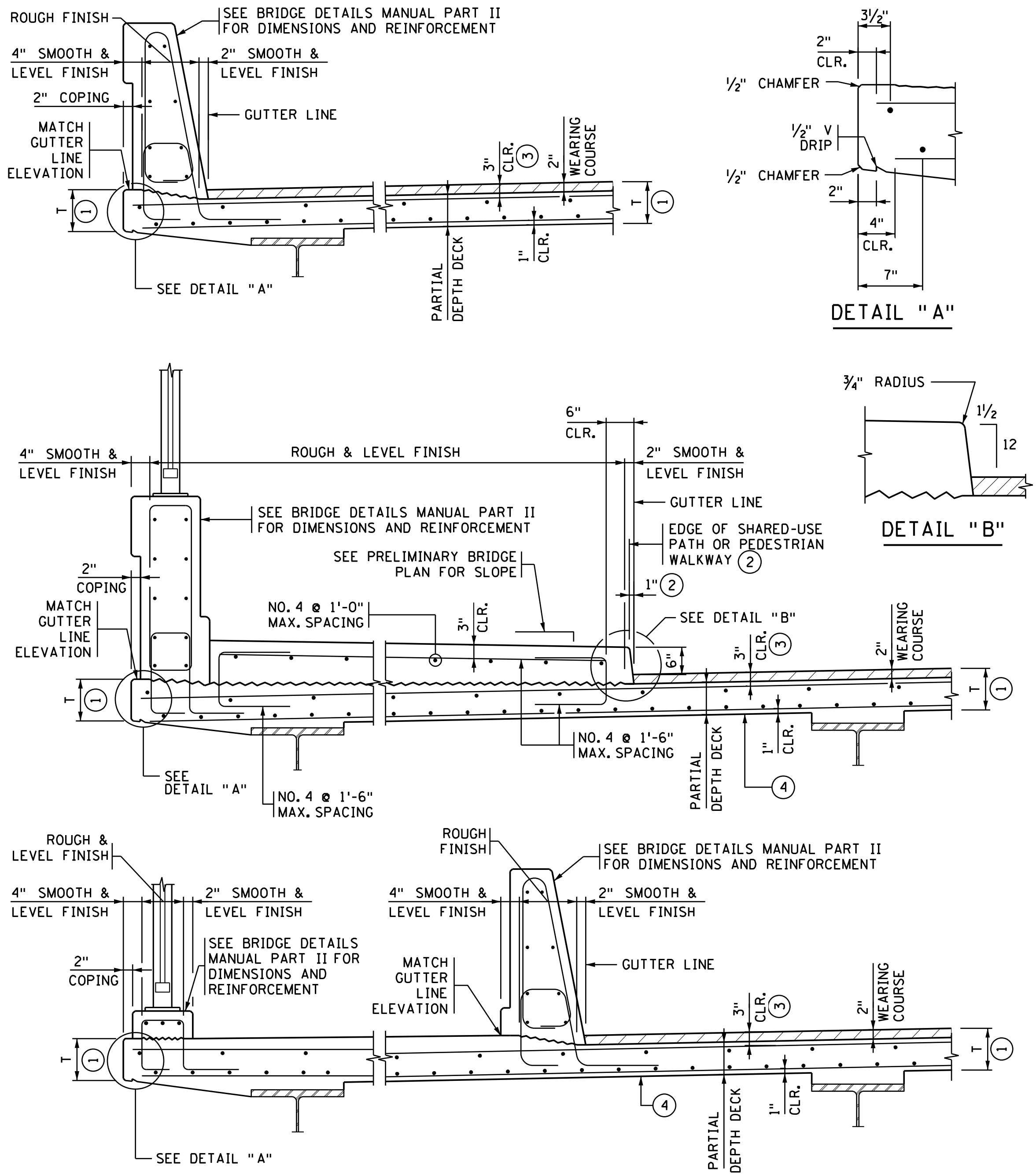
Maximum Beam Spacing ①	Transverse Reinforcement Size and Spacing			Deck Thickness T ②	Longitudinal Reinforcement Size and Spacing, Bottom Mat ③	Longitudinal Reinforcement Size and Spacing, Top Mat ③
	Bottom Mat		Top Mat			
	With Wearing Course	Without Wearing Course				
5'-0"	4 @ 6.5"	4 @ 9"	4 @ 8"	9''	4 @ 9"	4 @ 1'-6"
5'-6"	4 @ 6.5"	4 @ 8.5"	4 @ 7"	9''	4 @ 9"	4 @ 1'-6"
6'-0"	4 @ 6.5"	4 @ 8"	4 @ 6.5"	9''	4 @ 9"	4 @ 1'-6"
6'-6"	4 @ 6"	4 @ 8"	4 @ 6"	9''	4 @ 9"	4 @ 1'-6"
7'-0"	4 @ 5.5"	4 @ 7.5"	4 @ 5.5"	9''	4 @ 8"	4 @ 1'-6"
7'-6"	4 @ 5.5"	4 @ 7"	4 @ 5.5"	9''	4 @ 8"	4 @ 1'-6"
8'-0"	4 @ 5"	4 @ 6.5"	4 @ 5"	9''	4 @ 7"	4 @ 1'-6"
8'-6"	5 @ 7.5"	4 @ 6"	5 @ 6.5"	9''	4 @ 7"	4 @ 1'-6"
9'-0"	5 @ 7"	4 @ 6"	5 @ 6.5"	9''	4 @ 6"	4 @ 1'-6"
9'-6"	5 @ 6.5"	4 @ 5.5"	5 @ 6"	9''	4 @ 6"	4 @ 1'-6"
10'-0"	5 @ 6"	4 @ 5.5"	5 @ 6"	9''	4 @ 5"	4 @ 1'-6"
10'-6"	5 @ 6"	4 @ 5"	5 @ 5.5"	9''	4 @ 5"	4 @ 1'-6"
11'-0"	5 @ 6"	4 @ 5"	5 @ 5.5"	9.25''	4 @ 5"	4 @ 1'-6"
11'-6"	5 @ 6"	4 @ 5"	5 @ 5"	9.5''	4 @ 5"	4 @ 1'-6"
12'-0"	5 @ 6"	5 @ 7.5"	5 @ 5"	9.75''	4 @ 6"	4 @ 1'-6"
12'-6"	5 @ 6"	5 @ 7.5"	5 @ 5"	10''	4 @ 6"	4 @ 1'-6"
13'-0"	5 @ 6"	5 @ 7.5"	6 @ 6.5"	10.25''	4 @ 6"	4 @ 1'-6"
13'-6"	5 @ 6"	5 @ 7"	6 @ 6.5"	10.5''	4 @ 6"	4 @ 1'-6"
14'-0"	5 @ 6"	5 @ 7"	6 @ 6.5"	10.75''	4 @ 6"	4 @ 1'-6"
14'-6"	5 @ 5.5"	5 @ 7"	6 @ 6.5"	11''	4 @ 6"	4 @ 1'-6"
15'-0"	5 @ 5.5"	5 @ 7"	6 @ 6.5"	11.25''	4 @ 6"	4 @ 1'-6"

① For skews ≤ 20°, beam spacing is measured along the skew.
For skews > 20°, beam spacing is measured normal to roadway centerline.

② Deck thickness includes wearing course.

③ Reinforcement shown is for positive moment region only and is based on LRFD 5.10.6. Where deck and steel beams are continuous, design longitudinal reinforcement in negative moment regions for the factored negative moment and meet requirements of LRFD Article 6.10.1.7. See Figure 9.2.1.9 for longitudinal reinforcing requirements in negative moment regions.

Table 9.2.1.2

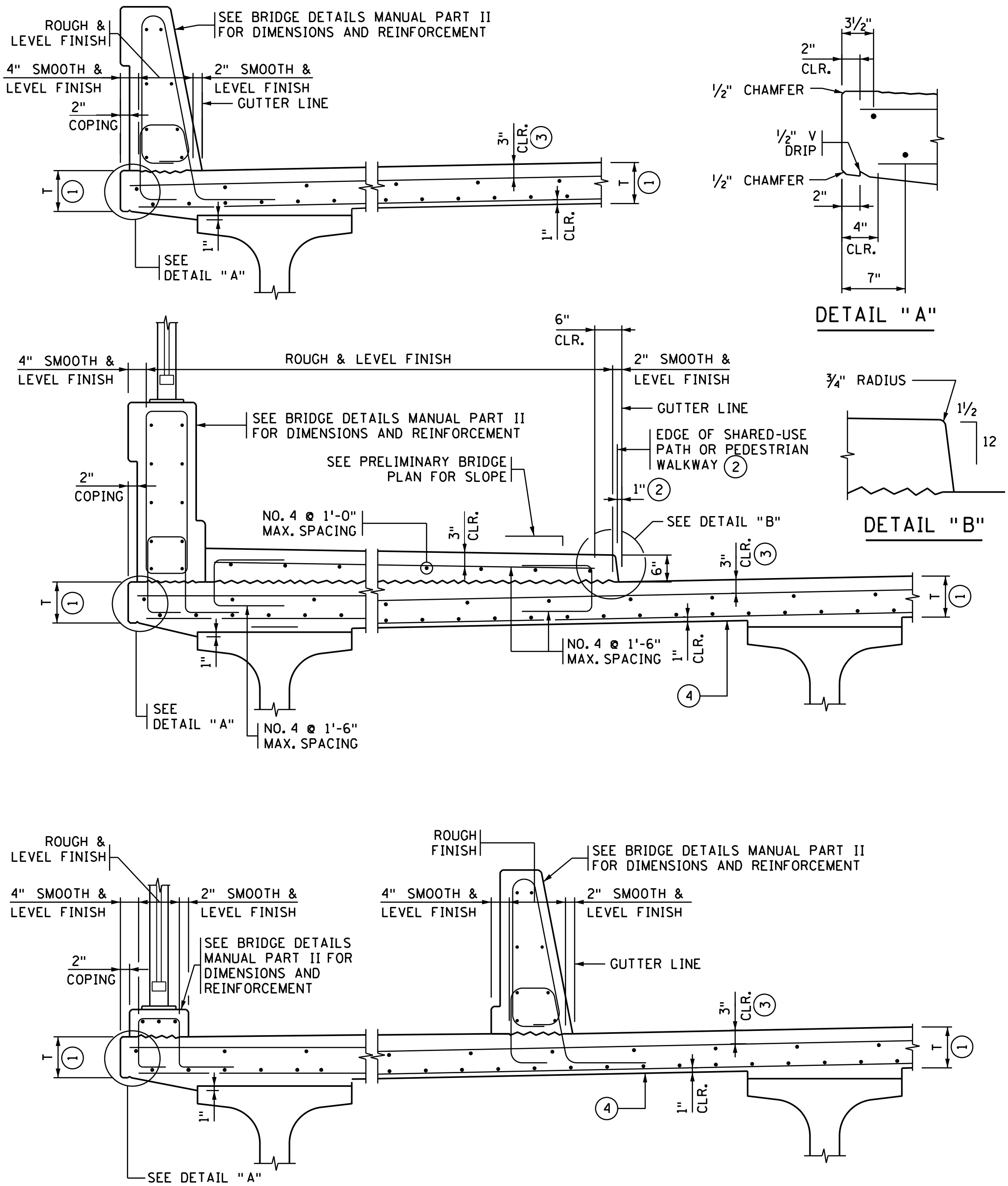


1. T EQUALS DECK THICKNESS GIVEN IN TABLE 9.2.1.1 OR 9.2.1.2 OF THIS MANUAL.
2. LOCATION OF TOP EDGE OF SHARED-USE PATH OR PEDESTRIAN WALKWAY IS DEFINED AS 1" OUTSIDE OF GUTTER LINE.
3. CONCRETE CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER MAY DIFFER FOR OTHER BAR TYPES.
4. FOR CROWN SECTION SLOPE BOTTOM OF DECK TO MATCH TOP OF ROADWAY SLOPE. CONTINUE SLOPE UNDER SIDEWALK TO FASCIA BEAM. FOR SUPERELEVATED SECTION WITH SIDEWALK ON HIGH SIDE OF DECK, ADJUST FASCIA BEAM VERTICAL LOCATION AS NEEDED TO PROVIDE A MINIMUM PARTIAL DEPTH DECK THICKNESS EQUAL TO T-2.

CONCRETE DECK REINFORCEMENT SECTIONS

(WITH CONCRETE WEARING COURSE)

Figure 9.2.1.4



(1) T EQUALS DECK THICKNESS GIVEN IN TABLE 9.2.1.1 OR 9.2.1.2 OF THIS MANUAL.

(2) LOCATION OF TOP EDGE OF SHARED-USE PATH OR PEDESTRIAN WALKWAY IS DEFINED AS 1" OUTSIDE OF GUTTER LINE.

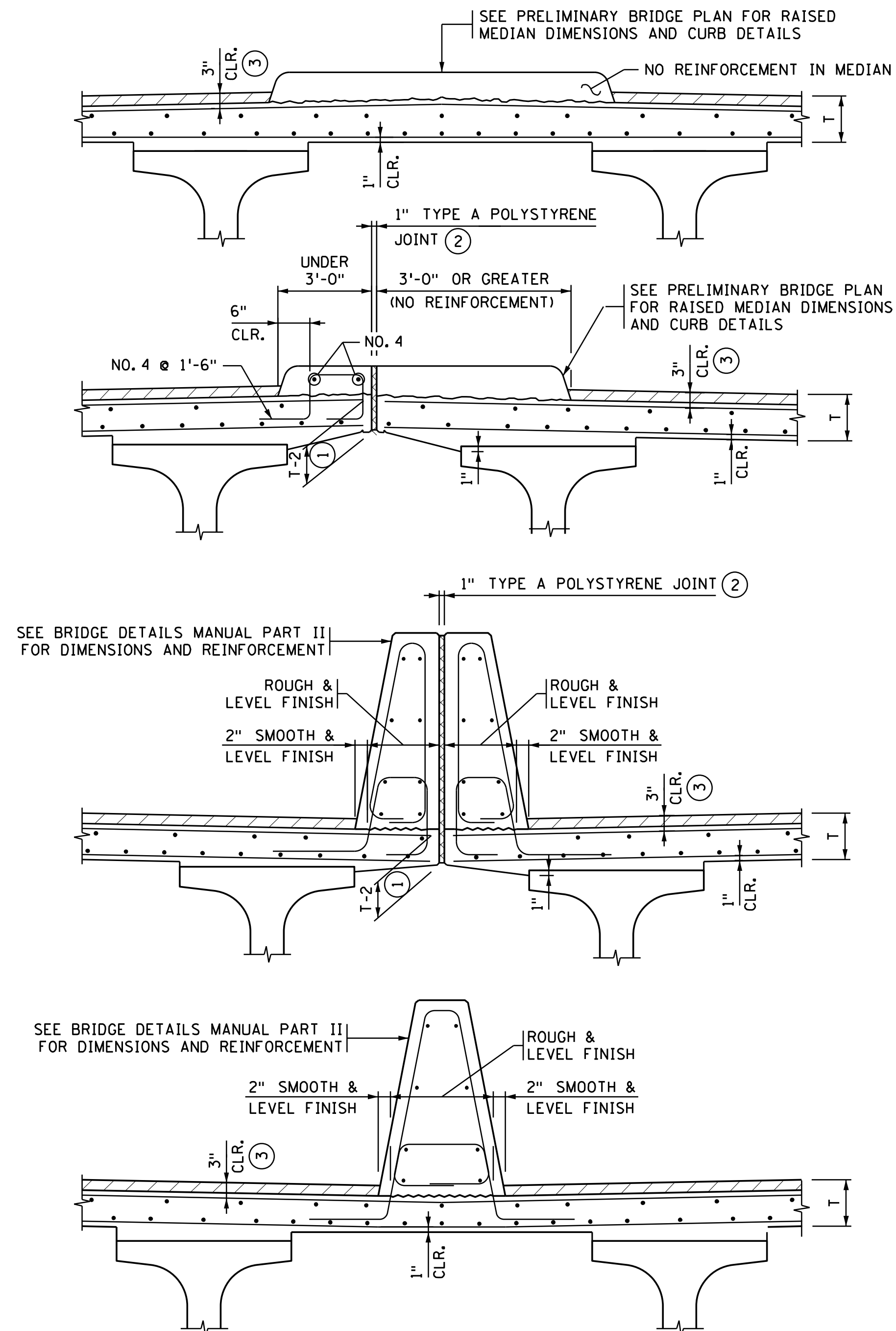
(3) CONCRETE CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER MAY DIFFER FOR OTHER BAR TYPES.

(4) FOR CROWN SECTION SLOPE BOTTOM OF DECK TO MATCH TOP OF ROADWAY SLOPE. CONTINUE SLOPE UNDER SIDEWALK TO FASCIA BEAM. FOR SUPERELEVATED SECTION WITH SIDEWALK ON HIGH SIDE OF DECK, ADJUST FASCIA BEAM VERTICAL LOCATION AS NEEDED TO PROVIDE A MINIMUM PARTIAL DEPTH DECK THICKNESS EQUAL TO T.

CONCRETE DECK REINFORCEMENT SECTIONS

(WITHOUT CONCRETE WEARING COURSE)

Figure 9.2.1.5



NOTES:

IF BOTH SHOULDER AREAS SLOPE INTO BARRIER BOTH ADJACENT BEAMS MAY HAVE TO DROP TO PREVENT NEGATIVE STOOLS.

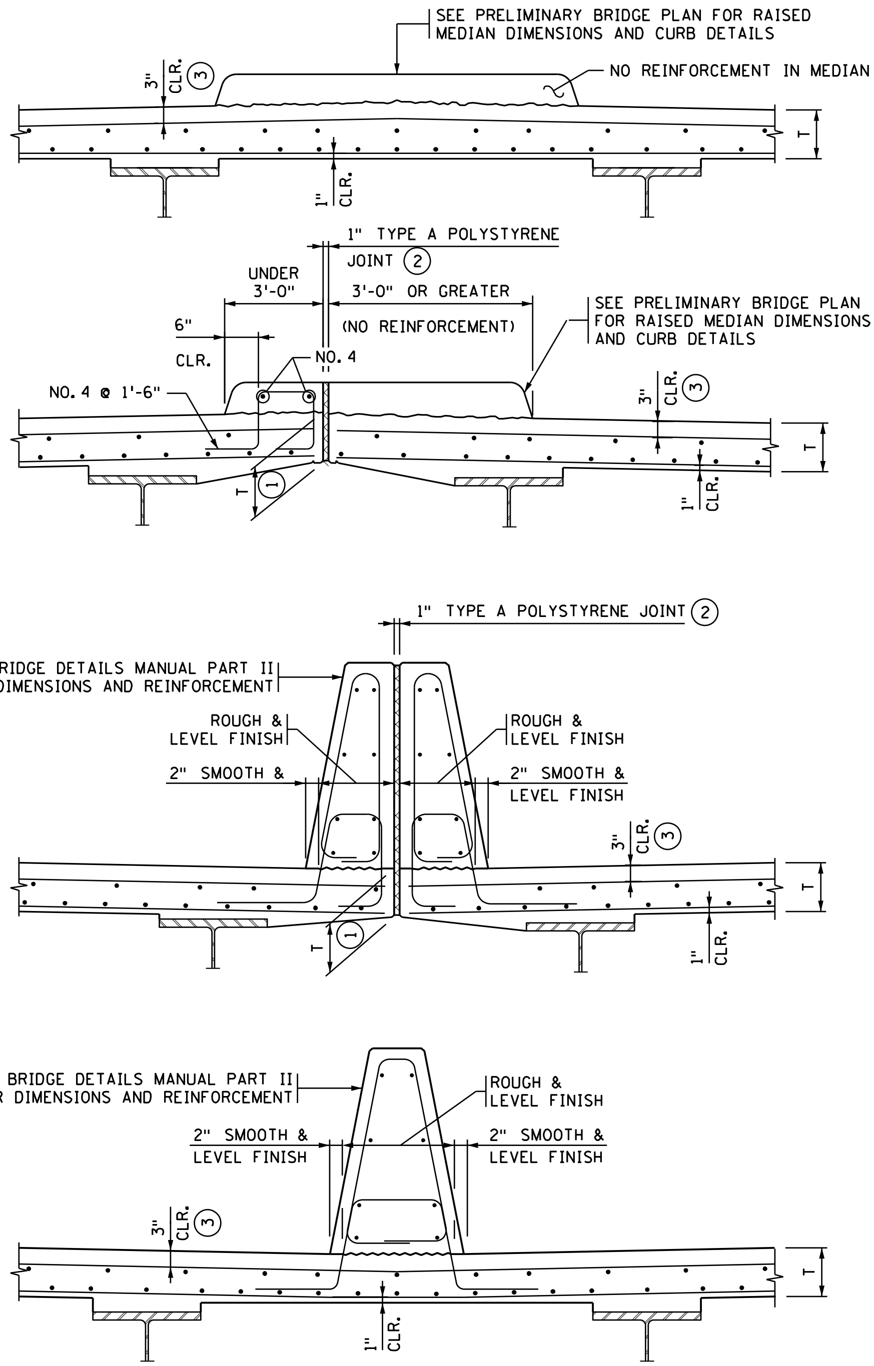
(1) T EQUALS DECK THICKNESS GIVEN IN TABLE 9.2.1.1. OR 9.2.1.2 OF THIS MANUAL.

(2) PROVIDE SPLIT MEDIAN CAP WHEN REQUIRED BY REGIONAL BRIDGE CONSTRUCTION ENGINEER.

(3) CONCRETE CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER FOR OTHER BARS MAY DIFFER.

CONCRETE DECK REINFORCEMENT SECTIONS
(WITH CONCRETE WEARING COURSE)

Figure 9.2.1.6



NOTES:

IF BOTH SHOULDER AREAS SLOPE INTO BARRIER BOTH ADJACENT BEAMS MAY HAVE TO DROP TO PREVENT NEGATIVE STOOLS.

1. T EQUALS DECK THICKNESS GIVEN IN TABLE 9.2.1.1. OR 9.2.1.2 OF THIS MANUAL.
2. PROVIDE SPLIT MEDIAN CAP WHEN REQUIRED BY REGIONAL BRIDGE CONSTRUCTION ENGINEER.
3. CONCRETE CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER FOR OTHER BARS MAY DIFFER.

CONCRETE DECK REINFORCEMENT SECTIONS
(WITHOUT CONCRETE WEARING COURSE)

Figure 9.2.1.7

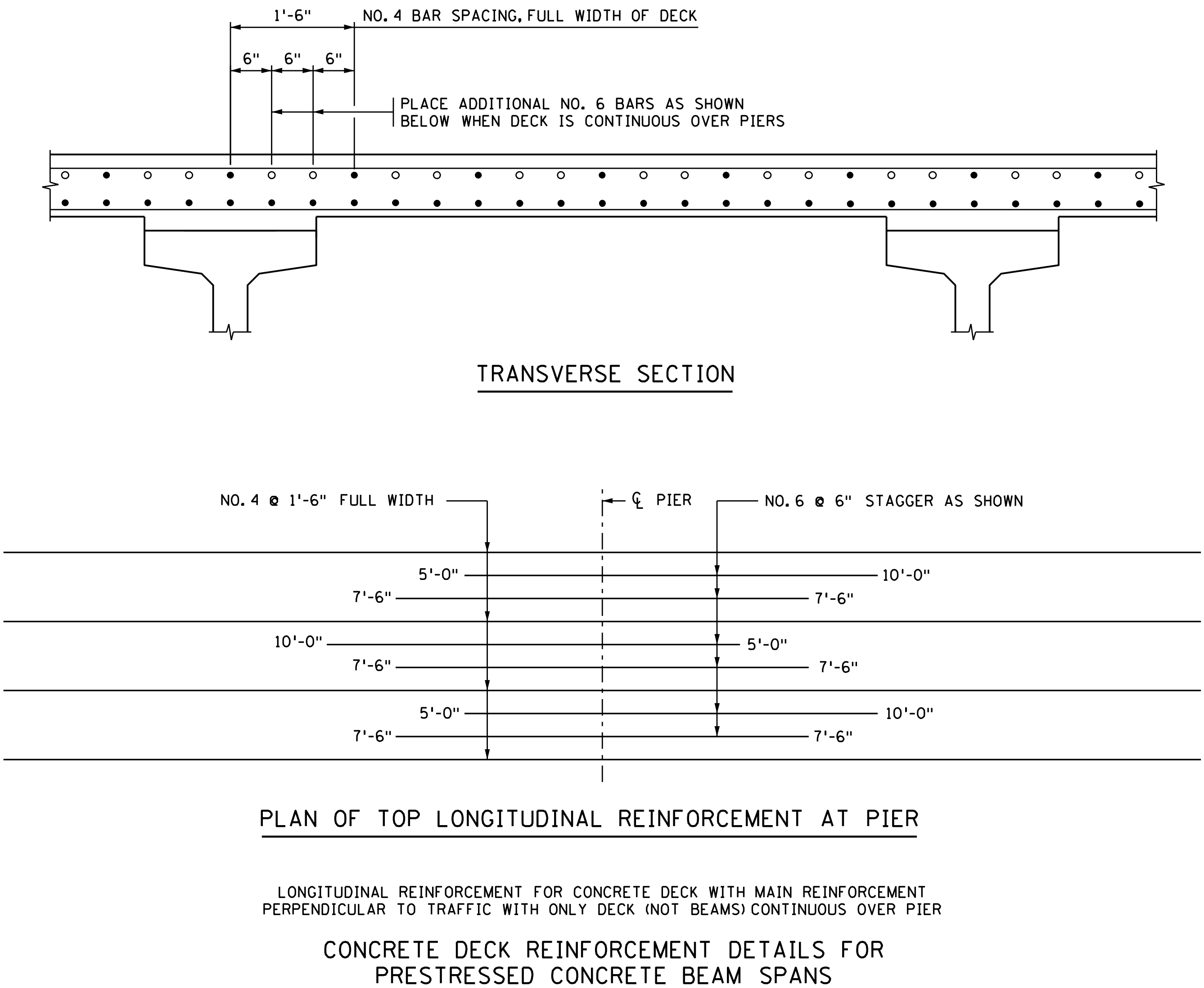
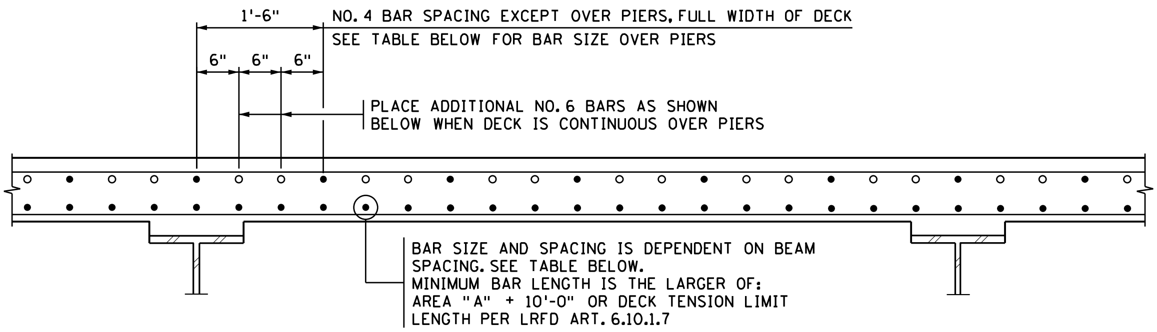
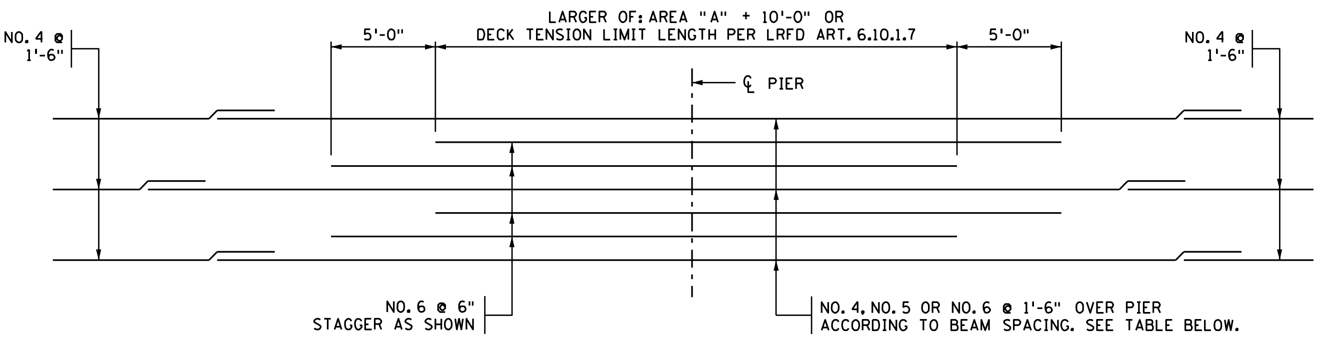

1'-6"
NO. 4 BAR SPACING, FULL WIDTH OF DECK
6"
6"
6"
PLACE ADDITIONAL NO. 6 BARS AS SHOWN
BELOW WHEN DECK IS CONTINUOUS OVER PIERS
TRANSVERSE SECTION
NO. 4 @ 1'-6" FULL WIDTH
℄ PIER
NO. 6 @ 6" STAGGER AS SHOWN
5'-0"
10'-0"
7'-6"
7'-6"
10'-0"
5'-0"
7'-6"
7'-6"
5'-0"
10'-0"
7'-6"
7'-6"
PLAN OF TOP LONGITUDINAL REINFORCEMENT AT PIER
LONGITUDINAL REINFORCEMENT FOR CONCRETE DECK WITH MAIN REINFORCEMENT
PERPENDICULAR TO TRAFFIC WITH ONLY DECK (NOT BEAMS) CONTINUOUS OVER PIER
CONCRETE DECK REINFORCEMENT DETAILS FOR
PRESTRESSED CONCRETE BEAM SPANS

Figure 9.2.1.8



TRANSVERSE SECTION



PLAN OF TOP LONGITUDINAL REINFORCEMENT AT PIER

THROUGH REINFORCEMENT OVER PIERS		
BEAM SPACING	BOTTOM LONGITUDINAL	TOP LONGITUDINAL
UP TO 9'-6"	NO. 4 @ 6"	NO. 4 @ 1'-6"
OVER 9'-6" TO 10'-6"	NO. 4 @ 5"	NO. 4 @ 1'-6"
OVER 10'-6" TO 11'-6"	NO. 4 @ 5"	NO. 5 @ 1'-6"
OVER 11'-6" TO 12'-6"	NO. 4 @ 6"	NO. 5 @ 1'-6"
OVER 12'-6" TO 14'-6"	NO. 5 @ 6"	NO. 6 @ 1'-6"
OVER 14'-6"	SPECIAL DESIGN	

PERMISSIBLE SPLICES IN REINFORCEMENT BARS TO BE LOCATED AWAY FROM ℄ OF PIER AND ALTERNATED ON EACH SIDE OF PIER

LONGITUDINAL REINFORCEMENT FOR CONCRETE DECK WITH MAIN REINFORCEMENT PERPENDICULAR TO TRAFFIC AND CONTINUOUS OVER 3 OR MORE BEAMS

CONCRETE DECK REINFORCEMENT DETAILS FOR CONTINUOUS STEEL BEAM SPANS (AASHTO LRFD ART. 6.10.1.7)

Figure 9.2.1.9

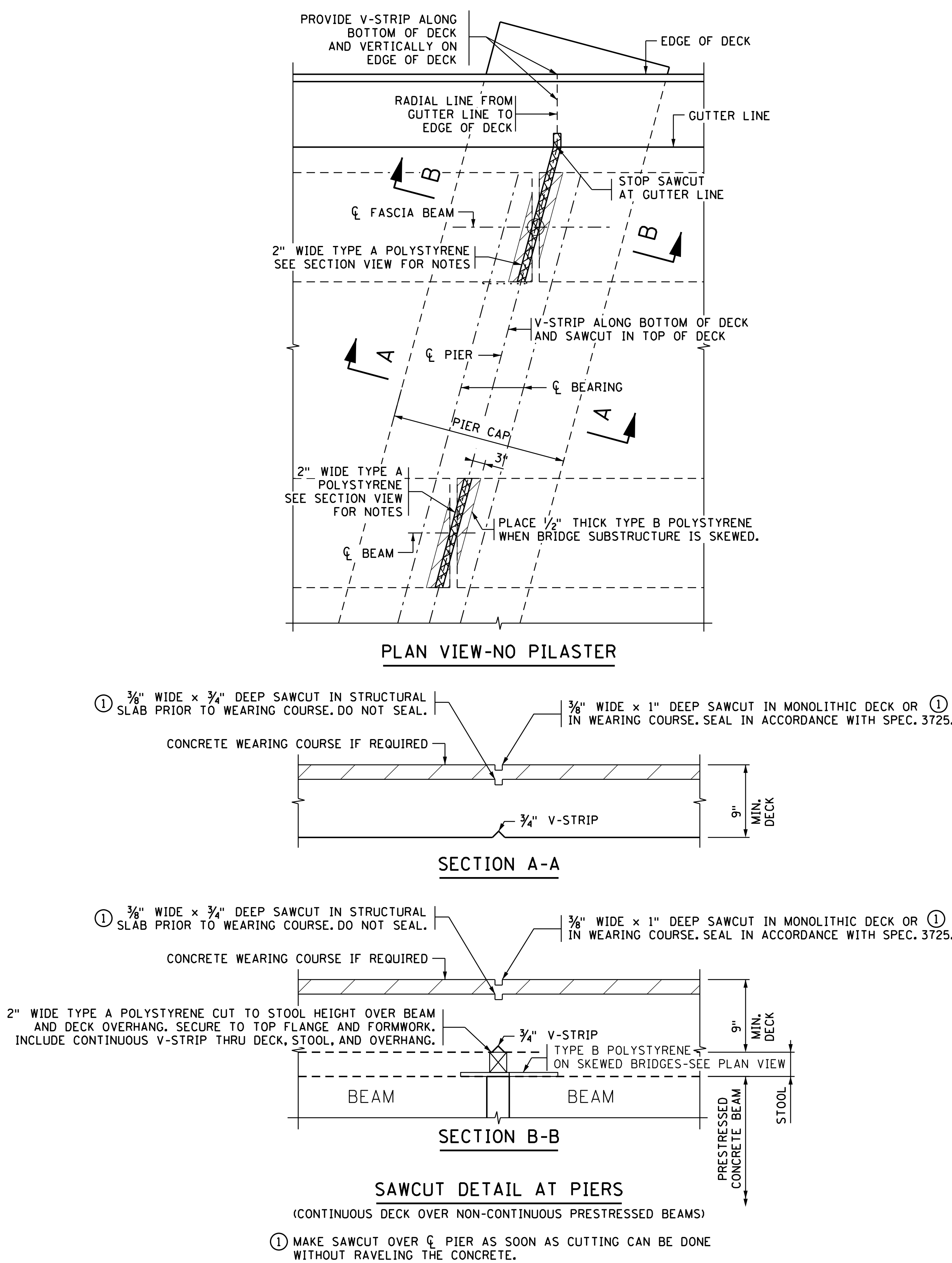


Figure 9.2.1.10

9.3 Reinforced Concrete Deck Design Example

This example demonstrates the design of a reinforced concrete deck supported on MN63 pretensioned concrete I-beams. The first part describes the design of the deck interior region (between the fascia beams) and the second part provides design procedures for the deck overhang region.

[4.6.2.1]

The deck is designed using the traditional approximate analysis method. The deck is assumed to carry traffic loads to the beam supports via one-way slab or beam action. The beams are parallel to the direction of traffic and the substructures are not skewed, so the primary reinforcement for the deck is placed perpendicular to the beams. Distribution steel is placed parallel to the beams.

[9.7.3.2]

The reinforced concrete deck section with wearing course is illustrated in Figure 9.3.1.

A. Material and Design Parameters

[9.7.1.1]
[9.7.1.3]

Deck

Unit weight of deck and wearing course (for loads), w_c = 0.150 kcf
Unit weight of deck and wearing course (for E_c), w_{cE} = 0.145 kcf
Skew angle of bridge, θ = 0 degrees
Out-to-out bridge deck transverse width, b_{deck} = 52.00 ft = 624 in
Weight of future wearing course, w_{fws} = 0.020 kcf
Yield strength of reinforcing bars, f_y = 60 ksi
Reinforcing bar modulus of elasticity, E_s = 29,000 ksi
28 day concrete strength, f'_c = 4 ksi
Center-to-center beam spacing, L_s = 9.00 ft
Railing weight, $w_{barrier}$ = 0.513 klf (see Std. Figure 5-397.139(B))
Beam flange width, b_f = 34 in (MN63 Prestressed I-Beam)
Deck overhang = 3.50 ft

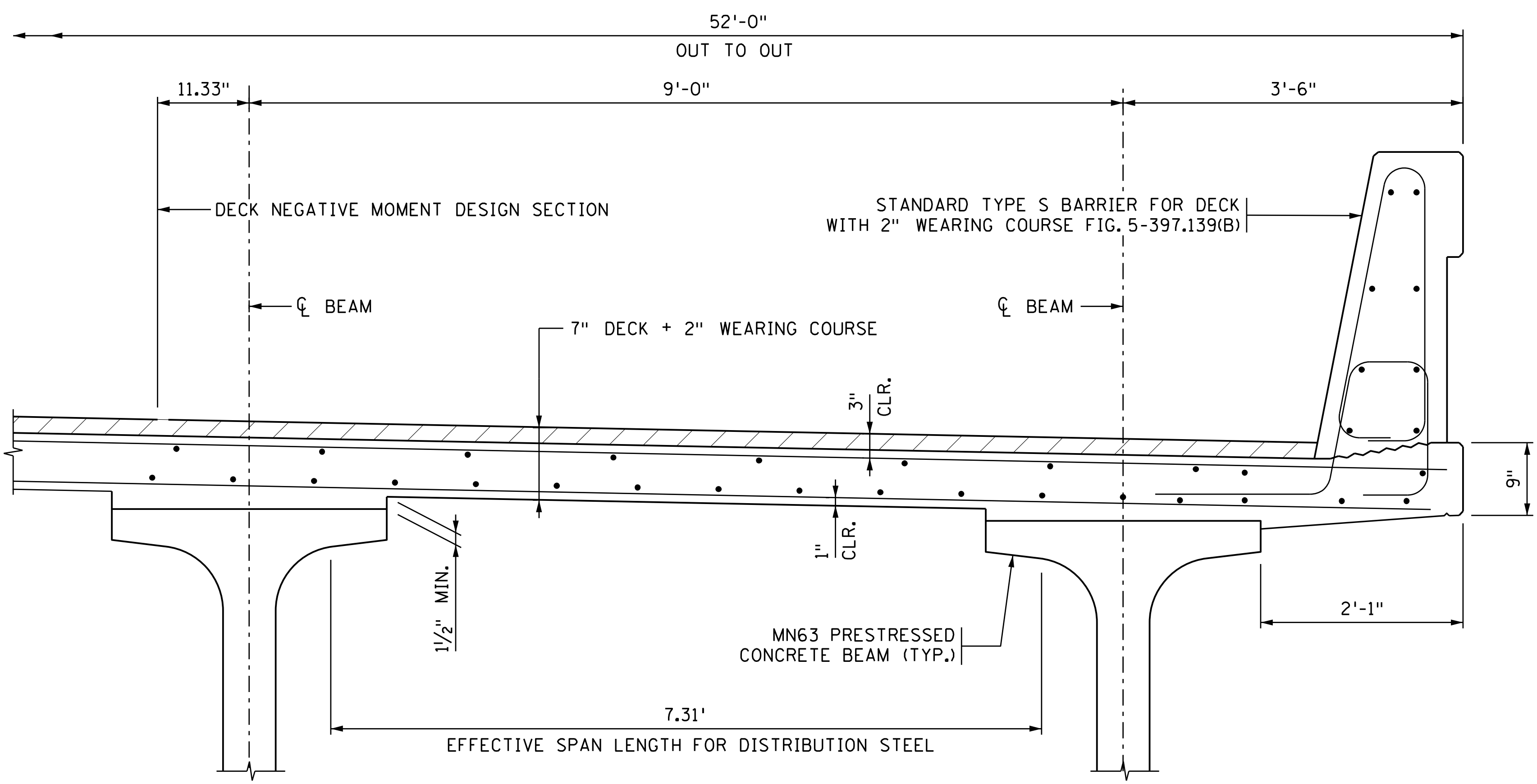
B. Structural Analysis of Interior Region

[9.6.1]

The deck is modeled as a continuous beam on pinned supports provided at the centerline of the supporting beams. The beams are assumed to be rigid, not permitting vertical movement. Recognizing that beams have top flanges that provide support for the deck over a finite dimension, the specifications permit designing negative moment reinforcement for locations that are offset from the centerline of the beam.

[4.6.2.1.6]

For prestressed beams, negative moments are checked at the design section located $^1/_3$ of the flange width away from the beam centerline, with maximum offset of 15 inches. For the top flange width of 34 inches, check negative moments at a location 11.33 inches away from beam centerline. (Note that this differs from the design section chosen for the deck reinforcement tables, which are conservatively based on M-series beams with 30 inch flanges.) The design is based on a unit strip one foot wide.



PARTIAL DECK SECTION

Figure 9.3.1

C. Live Loads [Appendix A4]

The AASHTO LRFD Specifications contain tables listing the design live load moments (positive and negative) for decks supported on different beam spacings. The tabularized moments are for a one foot wide strip.

The limitations for use of the tables include a check on the overhang dimension. A minimum of 1.75 feet from the centerline of the fascia beam is permitted. The maximum overhang permitted, L_{ohmax}, is the lesser of:

$L_{ohmax} = 6.00$ ft

or

$L_{ohmax} = 0.625 \cdot L_s = 0.625 \cdot 9.00 = 5.63$ ft GOVERNS

For this example the overhang check falls within the limits:

1.75 ft < 3.50 ft < 5.63 ft OK

The overhang dimension checks are satisfied, as are all other parameters specified for use of the design live load moment tables.

Interpolate Design Live Load Moments

LRFD Table A4-1 lists the following design live load moments for a beam spacing of 9.0 ft:

Positive moment	= 6.29 kip-ft
Negative moment (9 in)	= 4.28 kip-ft
Negative moment (12 in)	= 3.71 kip-ft

Interpolate to obtain the negative moment at the design section (11.33 inches away from the center of the supporting beam):

$$M_{LL(neg)} = 4.28 - \left(\frac{11.33 - 9}{12 - 9}\right) \cdot (4.28 - 3.71) = 3.84 \text{ kip-ft}$$

The values in LRFD Table A4 -1 include the multiple presence and dynamic load allowance factors.

D. Dead Loads

The dead load moments are based on the self-weight of the 7 inch partial depth deck, the 2 inch wearing course, and a 0.020 ksf future wearing surface.

Depth of concrete deck, $d_{deck} = 7 + 2 = 9$ in

Dead loads will be computed for a strip of deck 1 foot wide. MnDOT practice is to simplify the dead load bending moment calculations by computing both the positive and negative dead load bending moments using:

$$M_{DC} = \frac{W_{DC} \cdot L_s^2}{10}$$

Deck and Wearing Course Load:

$$W_{deck} = w_c \cdot d_{deck} = (0.150) \cdot 9 \cdot \frac{1}{12} = 0.11 \text{ klf}$$

Future Wearing Surface Load:

W_{fws} = 0.02 klf

Combined Dead Load:

$W_{DC} = W_{deck} + W_{fws} = 0.11 + 0.02 = 0.13$ klf

Dead Load Bending Moment:

$$M_{DC} = \frac{0.13 \cdot 9^2}{10} = 1.05 \text{ kip-ft}$$

E. Flexural Design Moments [1.3.3 – 1.3.5]

The load modifiers for the deck design are:

$\eta_D = 1.00$
$\eta_R = 1.00$
$\eta_I = 1.00$

Then $\eta_{cum} = \eta_D \cdot \eta_R \cdot \eta_I = 1.00$

[Table 3.4.1-1]

Use the load factors provided in LRFD Article 3.4.1 to generate the Strength I and Service I design moments.

Strength I Limit State Loads

$U_1 = \eta_{cum} \cdot (1.25 \cdot DC + 1.75 \cdot LL)$

Negative Design Moment:
$M_{u(neg)} = 1.00 \cdot [1.25 \cdot (1.05) + 1.75 \cdot (3.84)] = 8.03$ kip-ft

Positive Design Moment:
$M_{u(pos)} = 1.00 \cdot [1.25 \cdot (1.05) + 1.75 \cdot (6.29)] = 12.32$ kip-ft

Service I Limit State Loads

$S_1 = \eta_{cum} \cdot (1.0 \cdot DC + 1.0 \cdot LL)$

Negative Design Moment:
$M_{s(neg)} = 1.00 \cdot [1.0 \cdot (1.05) + 1.0 \cdot (3.84)] = 4.89$ kip-ft

Positive Design Moment:

$$M_{s(pos)} = 1.00 \cdot [1.0 \cdot (1.05) + 1.0 \cdot (6.29)] = 7.34 \text{ kip-ft}$$

F. Top Steel (Negative Moment)

[5.6.3]

Flexure Strength Check

The top reinforcement has a clear cover of 3 inches (which includes the 2 inch wearing course). Design the negative moment reinforcement assuming a singly reinforced cross section.

[5.5.4.2]

Assume the section is tension-controlled and the flexural resistance factor, $\phi = 0.90$.

Based on BDM Table 9.2.1.1, try #4 bars with a 6 inch center-to-center spacing.

Determine depth, d_s, from extreme compression fiber to tension reinforcement.

$$d_s = d_{deck} - \text{cover} - \frac{1}{2} \cdot d_b = 9 - 3 - \frac{1}{2} \cdot 0.5 = 5.75 \text{ in}$$

Width of compression face of member, b = 12 in

Area of top steel provided is:

$$A_{s(top)} = A_b \cdot \left(\frac{12}{\text{bar spacing}}\right) = 0.20 \cdot \left(\frac{12}{6}\right) = 0.40 \ \frac{\text{in}^2}{\text{ft}}$$

Then:

$$a = c \cdot \beta_1 = \frac{A_{s(top)} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.40 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.59 \text{ in}$$

$$\phi \cdot M_n = \phi \cdot A_{s(top)} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 0.9 \cdot 0.40 \cdot 60 \cdot \left(5.75 - \frac{0.59}{2}\right) \cdot \frac{1}{12}$$

$$= 9.82 \text{ kip-ft} > 8.03 \text{ kip-ft} \qquad \text{OK}$$

[5.5.4.2]

Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the section in compression:

$$c = \frac{a}{\beta_1} = \frac{0.59}{0.85} = 0.69 \text{ in}$$

[5.6.2.1]

Concrete compression strain limit $\varepsilon_c = 0.003$

Reinforcement tension-controlled strain limit ε_{tl} = 0.005

$$\varepsilon_t = (d_s - c) \cdot \left(\frac{\varepsilon_c}{c}\right) = (5.75 - 0.69) \cdot \left(\frac{0.003}{0.69}\right) = 0.0220 > \varepsilon_{tl} = 0.005$$

Therefore, $\phi = 0.9$

[5.6.7]

Crack Control Check

The LRFD crack control check places a limit on the spacing of reinforcement to prevent severe and excessive flexural cracking. This is accomplished by limiting the spacing of reinforcing bars as follows:

$$s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Also, the stress in the reinforcement, f_{ss}, is limited to:

$f_{ss} \leq 0.6 \cdot f_y = 0.6 \cdot 60 = 36.0$ ksi

Per Article 5.3.2 of this manual, use a maximum clear cover of 2.0 inches to compute d_c. Assuming #4 bars are used:

$d_c = 2.0 + 0.5 \cdot d_b = 2.0 + 0.5 \cdot 0.50 = 2.25$ in

The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. To simplify the calculation, the section is assumed to be singly reinforced.

[5.4.2.4 & 5.6.1]

Referring to Figure 9.3.2, determine the distance, x, from the bottom of the deck to the neutral axis:

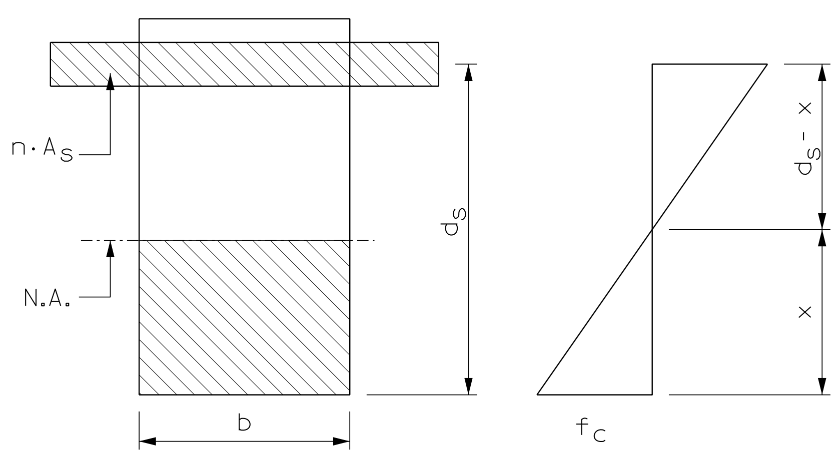


Figure 9.3.2

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{120{,}000 \cdot K_1 \cdot w_{cE}^2 \cdot f_c'^{\,0.33}} = \frac{29{,}000}{120{,}000 \cdot 1.0 \cdot 0.145^2 \cdot 4^{0.33}} = 7.27$$

$$n \cdot A_s = 7.27 \cdot 0.40 = 2.908$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d_s - x)$$

$$\frac{12 \cdot x^2}{2} = 2.908 \cdot (5.75 - x)$$

solving, $x = 1.44$ in

Determine the lever arm between service load flexural force components:

$$j \cdot d_s = d_s - \frac{x}{3} = 5.75 - \frac{1.44}{3} = 5.27 \text{ in}$$

The stress in the reinforcement when subjected to the Service I moment is:

$$f_{ss} = \frac{M_{s(neg)}}{A_s \cdot j \cdot d_s} = \frac{4.89 \cdot 12}{0.40 \cdot 5.27} = 27.8 \text{ ksi} < 36.0 \text{ ksi} \qquad \text{OK}$$

Find β_s. For determination of deck depth, h, conservatively assume 0.5 inches of wear:

$$h = d_{deck} - 0.5 = 9 - 0.5 = 8.5 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.25}{0.7 \cdot (8.5 - 2.25)} = 1.51$$

For severe exposure, use γ_e=0.75. Then, maximum bar spacing, s_{max}, is:

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.51 \cdot 27.8} - 2 \cdot 2.25 = 8.01 \text{ in} > 6 \text{ in} \qquad \text{OK}$$

[5.6.3.3]

Minimum Reinforcement

Check that reinforcement can carry the smaller of:

- Cracking moment, M_{cr}
- $1.33 \cdot M_{u(neg)}$

Conservatively assume the full 9 inch deep section for the minimum reinforcement check:

$$S_{deck} = \frac{b \cdot (d_{deck})^2}{6} = \frac{12 \cdot 9^2}{6} = 162 \text{ in}^3$$

[5.4.2.6]

Determine the modulus of rupture, f_r:

λ = 1.0 for normal weight concrete

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'c} = 0.24 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

Take γ_1 = 1.60 and γ_3 = 0.67 for ASTM Grade 60 reinforcement.

Combining these parameters results in a cracking moment, M_{cr}, of:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_{deck} = 0.67 \cdot 1.60 \cdot 0.48 \cdot 162 \cdot \frac{1}{12} = 6.95 \text{ kip-ft}$$

Compare this to:

$$1.33 \cdot M_{u(neg)} = 1.33 \cdot 8.03 = 10.68 \text{ kip-ft} > 6.95 \text{ kip-ft}$$

Use the M_{cr} value to check minimum reinforcement.

$\phi \cdot M_n$ = 9.82 kip-ft > 6.95 kip-ft OK

G. Bottom Steel (Positive Moment) [5.6.3]

Flexure Strength Check

The bottom reinforcement has a clear cover of one inch. Because the wearing course may be removed in future milling operations, do not include it in structural capacity computations. Design the positive moment reinforcement assuming a singly reinforced cross section.

[5.5.4.2]

Assume the section is controlled in tension and the flexural resistance factor, ϕ = 0.90.

Based on BDM Table 9.2.1.1, try #5 bars with a 7 inch center-to-center spacing.

Determine the depth, d_s, from extreme compression fiber to tension reinforcement.

$$d_s = d_{deck} - \text{cover} - \text{wear course} - \frac{1}{2} \cdot d_b = 9 - 1 - 2 - \frac{1}{2} \cdot 0.63 = 5.69 \text{ in}$$

Width of compression face of member, b = 12 in

Area of top steel provided is:

$$A_{s(bot)} = A_b \cdot \left(\frac{12}{\text{bar spacing}}\right) = 0.31 \cdot \left(\frac{12}{7}\right) = 0.53 \ \frac{in^2}{ft}$$

Then:

$$a = c \cdot \beta_1 = \frac{A_{s(bot)} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.53 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.78 \text{ in}$$

$$\phi \cdot M_n = \phi \cdot A_{s(bot)} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 0.9 \cdot 0.53 \cdot 60 \cdot \left(5.69 - \frac{0.78}{2}\right) \cdot \frac{1}{12}$$

$$= 12.64 \text{ kip-ft} > 12.32 \text{ kip-ft} \qquad \text{OK}$$

[5.5.4.2] Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the section in compression:

$$c = \frac{a}{\beta_1} = \frac{0.78}{0.85} = 0.92 \text{ in}$$

Concrete compression strain limit $\varepsilon_c = 0.003$

Reinforcement tension-controlled strain limit $\varepsilon_{tl} = 0.005$

$$\varepsilon_t = (d_s - c) \cdot \left(\frac{\varepsilon_c}{c}\right) = (5.69 - 0.92) \cdot \left(\frac{0.003}{0.92}\right) = 0.0156 > \varepsilon_{tl} = 0.005$$

Therefore, $\phi = 0.9$

[5.6.7] **Crack Control Check**

As noted previously, the limit on spacing of reinforcement for crack control is:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Also, the stress in the reinforcement, f_{ss}, is limited to:

$f_{ss} \le 0.6 \cdot f_y = 0.6 \cdot 60 = 36.0$ ksi

For #5 bars with 1 inch of cover, d_c is:

$d_c = \text{cover} + 0.5 \cdot d_b = 1.00 + 0.5 \cdot 0.625 = 1.31$ in

Compute the stress in the reinforcement using a cracked section analysis of a singly reinforced section. Begin by locating the neutral axis.

As calculated previously, $n = 7.27$

$n \cdot A_s = 7.27 \cdot 0.53 = 3.853$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d_s - x)$$

$$\frac{12 \cdot x^2}{2} = 3.853 \cdot (5.69 - x)$$

solving, x = 1.62 in

Determine the lever arm between service load flexural force components.

$$j \cdot d_s = d_s - \frac{x}{3} = 5.69 - \frac{1.62}{3} = 5.15 \text{ in}$$

The stress in the reinforcement when subjected to the Service I design moment is:

$$f_{ss} = \frac{M_{s(pos)}}{A_s \cdot j \cdot d_s} = \frac{7.34 \cdot 12}{0.53 \cdot 5.15} = 32.3 \text{ ksi} < 36.0 \text{ ksi} \qquad \text{OK}$$

Find β_s. For determination of deck depth, h, conservatively ignore the 2 inch wearing course:

$$h = d_{deck} - 2.0 = 9 - 2 = 7.0 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{1.31}{0.7 \cdot (7.0 - 1.31)} = 1.33$$

For severe exposure, use $\gamma_e = 0.75$. Then, maximum bar spacing, s_{max}, is:

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.33 \cdot 32.3} - 2 \cdot 1.31 = 9.60 \text{ in} > 7 \text{ in} \qquad \text{OK}$$

[5.6.3.3]

Minimum Reinforcement Check

Check that reinforcement can carry the smaller of:

- Cracking moment, M_{cr}
- $1.33 \cdot M_{u(neg)}$

Conservatively assuming the full 9 inch deep section for the minimum reinforcement check, S = 162 in^3 (previously calculated).

[5.4.2.6]

Also, the modulus of rupture, f_r = 0.48 ksi (previously calculated)

Taking γ_1 = 1.60 and γ_3 = 0.67 for ASTM Grade 60 reinforcement, the cracking moment, M_{cr}, is:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_{deck} = 0.67 \cdot 1.60 \cdot 0.48 \cdot 162 \cdot \frac{1}{12} = 6.95 \text{ kip-ft}$$

Compare this to:

$$1.33 \cdot M_{u(pos)} = 1.33 \cdot 12.32 = 16.39 \text{ kip-ft} > 6.95 \text{ kip-ft}$$

Use the M_{cr} value to check minimum reinforcement.

$\phi \cdot M_n$ = 12.65 kip-ft > 6.95 kip-ft OK

H. Bottom Longitudinal Reinforcement

[9.7.3.2]

As part of the Traditional Design Method an "equivalent width method" for reinforced bridge deck designs is utilized. To ensure proper load distribution, reinforcement placed perpendicular to the primary reinforcement must be provided in the bottom mat. This reinforcement is a fraction of the primary steel required for positive moment. For decks where the primary reinforcement is placed perpendicular to traffic, the longitudinal reinforcement requirement in the bottom mat is:

$$PCT = \left(\frac{220}{\sqrt{S_e}}\right) \leq 67\%$$

where S_e is the effective span length in feet

[9.7.2.3]

The effective span length is a function of the beam spacing and type of beam. For prestressed concrete I-beam sections, the effective span length, S_e, is:

S_e = beam spacing – top flange width + one flange overhang

$$= 9.00 - \frac{34}{12} + \frac{13.75}{12} = 7.31 \text{ ft}$$

$$PCT = \left(\frac{220}{\sqrt{S_e}}\right) = \left(\frac{220}{\sqrt{7.31}}\right) = 82.6\% \geq 67\%$$

Use 67% of the primary steel in the bottom mat.

The required area of steel is:

$$A_{s(req)} = 0.67 \cdot A_{s(bot)} = 0.67 \cdot 0.53 = 0.36 \text{ in}^2/\text{ft}$$

Try #5 bars on 10 inch centers. Area of steel provided equals:

$$A_{s(prov)} = A_b \cdot \left(\frac{12}{\text{spacing}}\right) = 0.31 \cdot \left(\frac{12}{10}\right) = 0.37 \frac{\text{in}^2}{\text{ft}} > 0.36 \frac{\text{in}^2}{\text{ft}} \qquad \text{OK}$$

I. Top Longitudinal Reinforcement [5.10.6]

The top longitudinal bars must meet the shrinkage and temperature reinforcement requirements.

The least width $b = b_{deck} = 624$ in
Take the least depth, h, as equal to the full deck thickness (conservative), which is 9 inches.

Then:

$$A_{stemp} \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} = \frac{1.30 \cdot 624 \cdot 9}{2 \cdot (624 + 9) \cdot 60} = 0.096 \ \frac{in^2}{ft}$$

In addition:

$0.11 \ in^2/ft \leq A_{stemp} \leq 0.60 \ in^2/ft$
and
maximum bar spacing is 18 inches

Therefore, use #4 bars spaced at 18 inches ($A_s = 0.13 \ in^2/ft$) for the top longitudinal reinforcement.

MnDOT includes additional reinforcement over the piers when the deck is continuous, but the beams are not continuous. The additional reinforcing consists of two #6 bars placed on 6 inch centers between the top mat #4 bars. Refer to Figure 9.2.1.8 for typical reinforcement detailing.

Figure 9.3.3 illustrates the final reinforcement layout for the interior region of the deck.

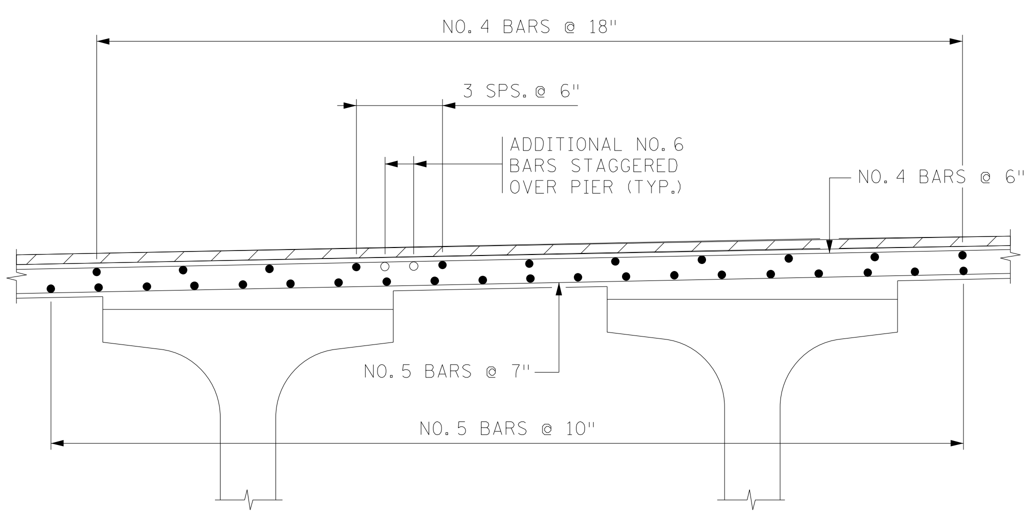


Figure 9.3.3

J. Structural Analysis of Deck Overhang Region [A13.4.1]

Figure 9.3.4 illustrates the deck overhang region. Four cases must be considered for the deck overhang design:

Case 1: Extreme Event II evaluated at the toe of the barrier for the dead load plus horizontal collision force.
Case 2: Extreme Event II evaluated at the edge of the beam flange for the dead load plus horizontal collision force plus live load.
Case 3: Strength I evaluated at the edge of the beam flange for the dead load plus live load.
Case 4: Extreme Event II evaluated at the edge of the beam flange for the dead load plus vertical collision force plus live load.

For this example, the distance from the edge of flange to the gutter line is less than 1 foot, so a live load wheel load is not considered. Also, the dead load moment is a small fraction of the moment due to the collision load, so the higher Strength I load factor for dead load does not have an appreciable effect on the Strength I load combination when comparing it to Extreme Event II. Therefore, by inspection, Case 3 will not govern over Cases 1 and 2, so Case 3 calculations are not included in this example.

K. Overhang Region Analysis, Case 1

Geometry and Loads

Case 1 is Extreme Event II checked at the toe of the barrier for dead load and the horizontal collision force. Referring to Figure 9.3.4, determine the center of gravity location for the barrier by considering the area of a rectangular block that encompasses the entire barrier cross-section and subtracting components ①, ②, and ③. Results are shown in Table 9.3.1:

Table 9.3.1 Determination of Barrier Center of Gravity Location

Component Description	Width (in)	Height (in)	Area (in^2)	Moment Arm From Barrier Toe (in)	Area · Moment Arm (in^3)
Block encompassing barrier	18.38	38.00	698.44	9.19	6418.66
① (triangle)	7.38	38.00	-140.22	2.46	-344.94
② (rectangle)	2.00	25.00	-50.00	17.38	-869.00
③ (triangle)	16.38	2.00	-16.38	10.92	-178.87

Total = 491.84 in^2 Total = 5025.85 in^3

Then C.G. location, x_{cg}, from barrier toe is: $$x_{cg} = \frac{5025.85}{491.84} = 10.22 \text{ in}$$

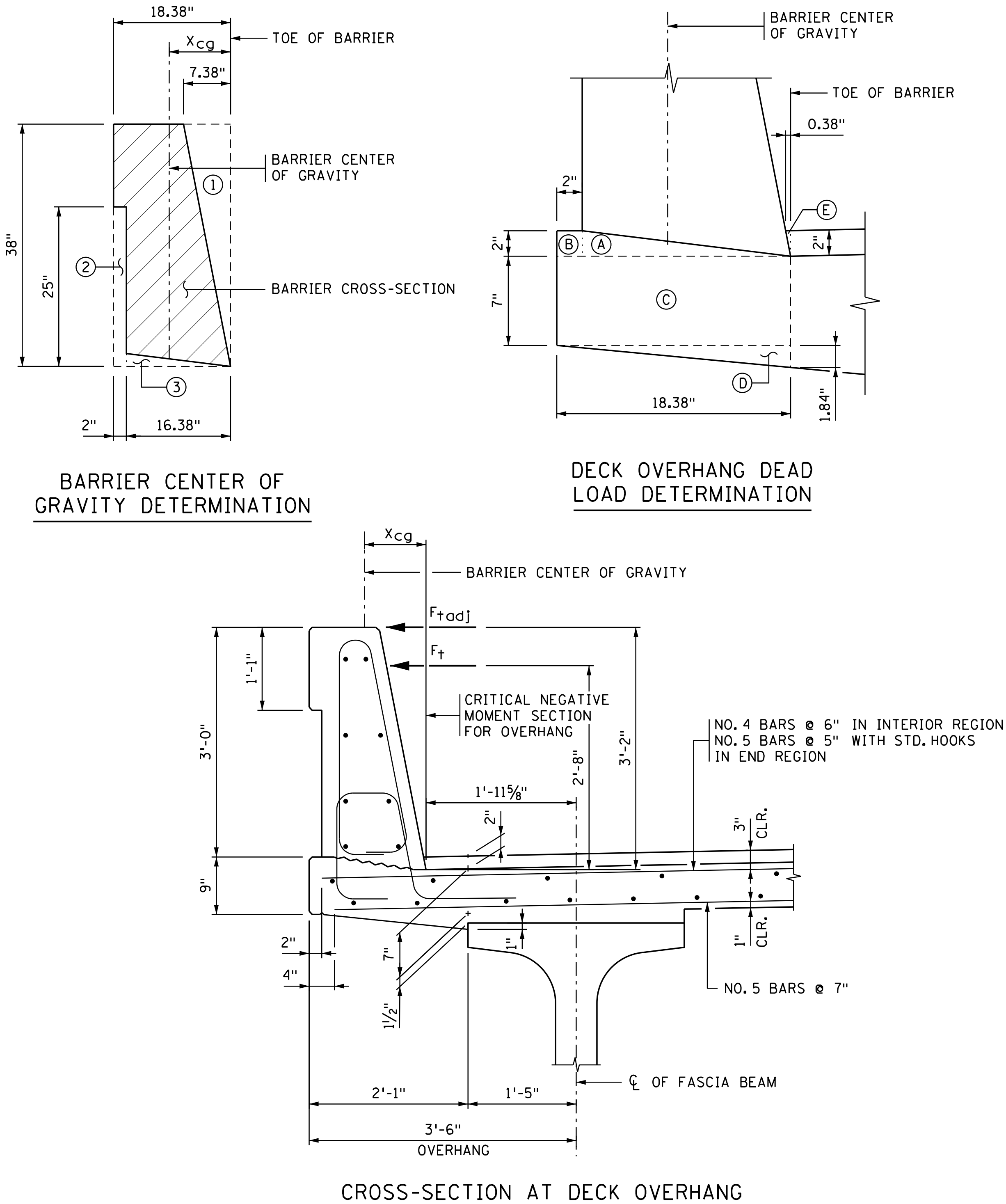

18.38"
Xcg
TOE OF BARRIER
7.38"
BARRIER CENTER OF GRAVITY
1
2
BARRIER CROSS-SECTION
38"
25"
3
2"
16.38"
BARRIER CENTER OF GRAVITY DETERMINATION
BARRIER CENTER OF GRAVITY
TOE OF BARRIER
0.38"
2"
E
B
A
2"
2"
7"
C
D
18.38"
1.84"
DECK OVERHANG DEAD LOAD DETERMINATION
Xcg
BARRIER CENTER OF GRAVITY
Ftadj
Ft
1'-1"
CRITICAL NEGATIVE MOMENT SECTION FOR OVERHANG
NO. 4 BARS @ 6" IN INTERIOR REGION
NO. 5 BARS @ 5" WITH STD. HOOKS IN END REGION
3'-0"
2'-8"
3'-2"
1'-11⅝"
2"
3" CLR.
9"
1"
1" CLR.
2"
4"
7"
1½"
NO. 5 BARS @ 7"
℄ OF FASCIA BEAM
2'-1"
1'-5"
3'-6"
OVERHANG
CROSS-SECTION AT DECK OVERHANG

Figure 9.3.4

Overhang = 3.50 ft = 42 in
Distance from centerline of beam to edge of top flange = 17 in
Edge of deck to critical negative moment section at barrier toe,
L_{crit} = 18.38 in
Deck thickness, h_{deck_toe}, at barrier toe (ignoring wearing course):

$$h_{deck\_toe} = (9 - 2) + \left(\frac{18.38}{42 - 17}\right) \cdot [(2 + 7 + 1.5 + 1) - 9] = 8.84 \text{ in}$$

Referring again to Figure 9.3.4, determine dead load moments acting on the deck at the toe of barrier. Results are shown in Table 9.3.2:

Table 9.3.2 Determination of Dead Load Moments at Barrier Toe for a 1 ft. Deck Strip Width

Component Description	Width (in)	Height (in)	w_{DC} (kips)	Moment Arm From Barrier Toe (in)	Unfactored Moment M_{DC} (kip-ft)
Barrier			0.513	10.22	0.437
A (triangle)	16.38	2.00	0.017	10.92	0.015
B (rectangle)	2.00	2.00	0.004	17.38	0.006
C (rectangle)	18.38	7.00	0.134	9.19	0.103
D (triangle)	18.38	1.84	0.018	6.13	0.009
E (triangle)	0.38	2.00	0.000	0.13	0.000

Total w_{DC} = 0.686 kips for 1 ft strip width

Total M_{DC} = 0.570 kip-ft for a 1 ft strip width

[A13.2]

Collision Force Tension and Bending Moment

(Note: The remainder of this design example is correct only for deck overhang design using the outdated NCHRP Report 350 loads. Use the methodology shown in this example along with the loads determined using Memo to Designers #2020-01 for a deck overhang design that will meet the current MASH loading requirements.)

Using the yield line analysis method of LRFD Appendix A13, values for the nominal resistance, R_w, flexural resistance about the horizontal axis, M_c, and critical wall length, L_c, were calculated (not included here) for a 36 inch Type S barrier mounted on a deck that includes a wearing course (Std. Figure 5-397.139(B)):

Barrier int.: R_{w_int} = 117.4 kips M_{c_int} = 17.1 kip-ft/ft L_{c_int} = 10.9 ft
Barrier end: R_{w_end} = 71.8 kips M_{c_end} = 22.8 kip-ft/ft L_{c_end} = 5.0 ft

For a barrier meeting NCHRP Report 350 Test Level 4:

Transverse collision load F_t = 54 kips

Height of load application H_e = 32 in (distance above top of wearing course)

Because the yield line equations in LRFD assume the collision load is applied at the top of the barrier, adjust F_t for the difference between the barrier height and height of application. Refer to Figure 9.3.4. Note that the barrier sits on the partial depth deck with its toe 2 inches below the top of wearing course:

$H_{barrier}$ = 38 in

$$F_{tadj} = F_t \cdot \left(\frac{H_e + 2}{H_{barrier}}\right) = 54 \cdot \left(\frac{32 + 2}{38}\right) = 48.3 \text{ kips}$$

Because the barrier capacity can be excessively large compared to the collision load, MnDOT requires that the deck overhang be designed to resist a transverse collision force equal to the lesser of the barrier capacity R_w or $^4/_3 \cdot F_{tadj}$:

$F_{coll\_int} = R_{w\_int} = 117.4$ kips

$F_{coll\_end} = R_{w\_end} = 71.8$ kips

or

$$= \frac{4}{3} \cdot F_{tadj} = \frac{4}{3} \cdot 48.3 = 64.4 \text{ kips} \quad \text{GOVERNS IN BOTH REGIONS}$$

Since R_{w_int} and R_{w_end} do not govern, the M_c values must also be adjusted to correspond with the collision load:

$$M_{cadj\_int} = \frac{F_{coll\_int}}{R_{w\_int}} \cdot M_{c\_int} = \frac{64.4}{117.4} \cdot 17.1 = 9.4 \ \frac{\text{kip-ft}}{\text{ft}}$$

$$M_{cadj\_end} = \frac{F_{coll\_end}}{R_{w\_end}} \cdot M_{c\_end} = \frac{64.4}{71.8} \cdot 22.8 = 20.5 \ \frac{\text{kip-ft}}{\text{ft}}$$

For deck overhang design, assume that the collision load is distributed over a length of $L_{c\_int} + 2 \cdot H_{barrier}$ for the interior overhang region and a length of $L_{c\_end} + H_{barrier}$ for the end overhang region. Then:

$$F_{cadj\_int} = \frac{F_{coll\_int}}{L_{c\_int} + 2 \cdot H_{barrier}} = \frac{64.4}{10.9 + 2 \cdot \frac{38}{12}} = 3.7 \text{ kips/ft}$$

$$F_{cadj\_end} = \frac{F_{coll\_end}}{L_{c\_end} + H_{barrier}} = \frac{64.4}{5.0 + \frac{38}{12}} = 7.9 \text{ kips/ft}$$

The resulting load M_{cadj} is located at the top of the partial depth deck at the toe of the barrier. Translate this load to the center of the partial depth deck for design of the deck overhang. Referring to Figure 9.3.5, first find the eccentricity:

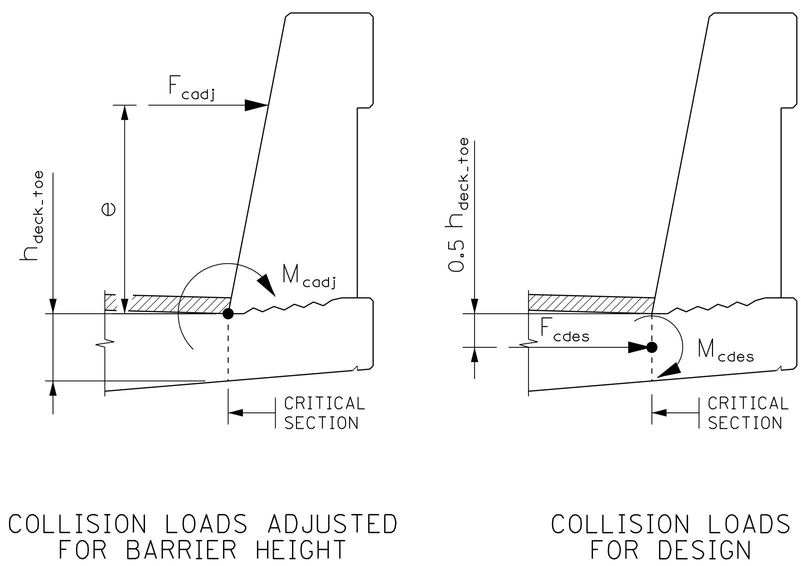


Figure 9.3.5

$$e_{int} = \frac{M_{cadj\_int}}{F_{cadj\_int}} = \frac{9.4}{3.7} = 2.54 \text{ ft}$$

$$e_{end} = \frac{M_{cadj\_end}}{F_{cadj\_end}} = \frac{20.5}{7.9} = 2.59 \text{ ft}$$

Then:

$F_{cdes\_int} = F_{cadj\_int} = 3.7$ kips/ft

$M_{cdes\_int} = F_{cdes\_int} \cdot (e_{int} + 0.5 \cdot h_{deck\_toe})$

$$= 3.7 \cdot \left(2.54 + 0.5 \cdot \frac{8.84}{12}\right) = 10.8 \frac{\text{kip-ft}}{\text{ft}}$$

$F_{cdes\_end} = F_{cadj\_end} = 7.9$ kips/ft

$$M_{cdes\_end} = F_{cdes\_end} \cdot (e_{end} + 0.5 \cdot h_{deck\_toe})$$
$$= 7.9 \cdot \left(2.59 + 0.5 \cdot \frac{8.84}{12}\right) = 23.4\ \frac{\text{kip-ft}}{\text{ft}}$$

Extreme Event II Limit State Bending Moment

Total factored loads are:

[A13.4.1]

$$M_{u\_int} = 1.00 \cdot M_{DC} + 1.00 \cdot M_{cdes\_int}$$
$$= 1.00 \cdot 0.57 + 1.00 \cdot 10.8 = 11.4 \text{ kip-ft/ft}$$

$$P_{u\_int} = 1.00 \cdot F_{DC} + 1.00 \cdot F_{cdes\_int}$$
$$= 1.00 \cdot 0.0 + 1.00 \cdot 3.7 = 3.7 \text{ kips/ft}$$

$$M_{u\_end} = 1.00 \cdot M_{DC} + 1.00 \cdot M_{cdes\_end}$$
$$= 1.00 \cdot 0.57 + 1.00 \cdot 23.4 = 24.0 \text{ kip-ft/ft}$$

$$P_{u\_end} = 1.00 \cdot F_{DC} + 1.00 \cdot F_{cdes\_end}$$
$$= 1.00 \cdot 0.0 + 1.00 \cdot 7.9 = 7.9 \text{ kips/ft}$$

The eccentricity of P_u is:

$$e_{u\_int} = \frac{M_{u\_int}}{P_{u\_int}} = \frac{11.4}{3.7} = 3.08 \text{ ft} = 36.96 \text{ in}$$

$$e_{u\_end} = \frac{M_{u\_end}}{P_{u\_end}} = \frac{24.0}{7.9} = 3.04 \text{ ft} = 36.48 \text{ in}$$

Resistance of Deck Interior Overhang Region

The overhang must resist both axial tension and bending moment. The capacity of the overhang will be determined by considering the tension side of the structural interaction diagram for a one foot wide portion of the overhang. See Figure 9.3.6.

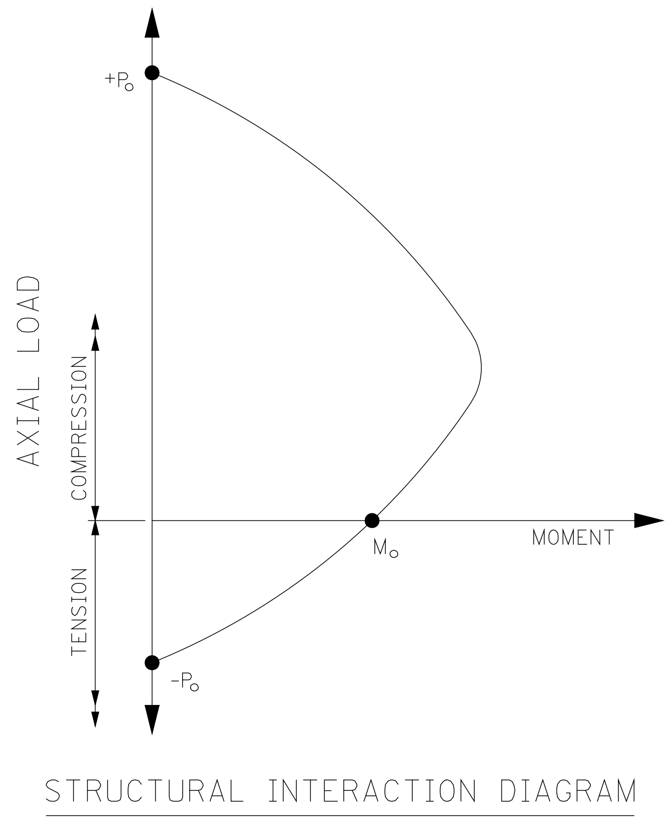


Figure 9.3.6

Check if the reinforcement chosen for the deck interior region (between the fascia beams) will be adequate for the overhang. The deck interior region reinforcement is:

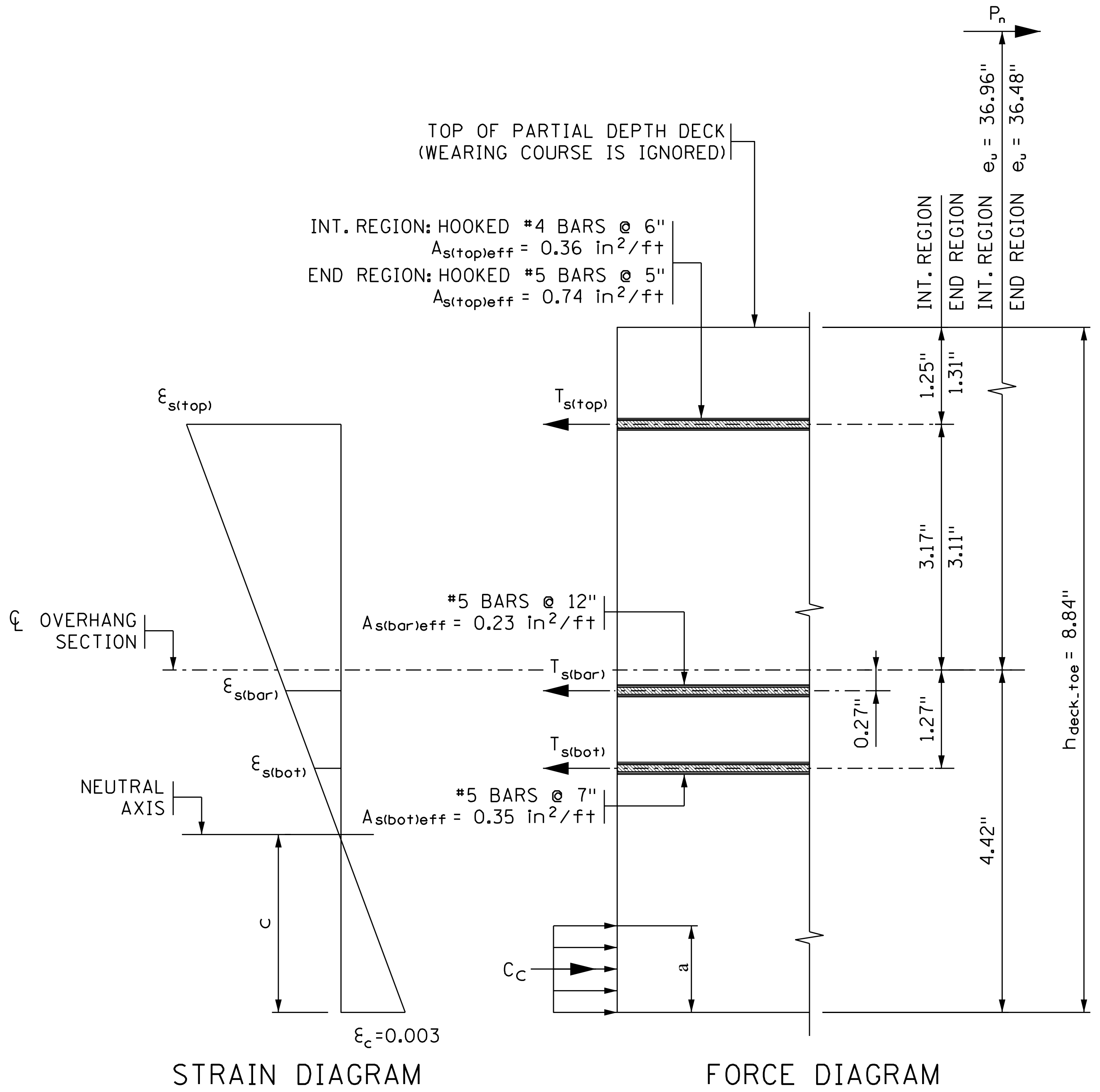
Top reinforcement – #4 bars @ 6" ($A_{s(top)}$ = 0.40 in^2/ft)

Bottom reinforcement – #5 bars @ 7" ($A_{s(bot)}$ = 0.53 in^2/ft)

Note that the front leg of the barrier bar also contributes to the strength of the overhang. The barrier reinforcement is:

Barrier front leg - #5 bars @ 12" ($A_{s(bar)}$ = 0.31 in^2/ft)

Referring to Figure 9.3.7, determine the capacity of the overhang section for the eccentricity e_{u_int} equal to 36.96 inches.



Reinforced Concrete Section at Toe of Barrier
Figure 9.3.7

First, determine the effective area of reinforcement for the deck bars located at the toe of the barrier. From Figure 5.2.2.2 of this manual, the development length, ℓ_d, for the deck bars are as follows:

For #4 top bars @ 6", cover is > 3" outside barrier toe, which results in an ℓ_d = 18".

For #5 bottom bars @ 7", cover varies with 1" minimum.

Using the 1" minimum cover results in an ℓ_d = 33".

For #5 barrier bars @ 12", minimum cover is 2.38" to the top of deck at the inside of the barrier toe (ignoring wearing course), which results in an ℓ_d = 22".

Then:

For #4 top bars, $\ell_{davailable} = L_{crit}$ – (top bar end cover)
= 18.38 – 2.0 = 16.38 in

$$A_{s(top)eff} = A_{s(top)} \cdot \frac{\ell_{davailable}}{\ell_d} = 0.40 \cdot \frac{16.38}{18} = 0.36 \frac{in^2}{ft}$$

For #5 bottom bars, $\ell_{davailable} = L_{crit}$ – (bottom bar end cover)
= 18.38 – 4.0 = 14.38 in

$$A_{s(bot)eff} = A_{s(bot)} \cdot \frac{\ell_{davailable}}{\ell_d} = 0.53 \cdot \frac{14.38}{33} = 0.23 \frac{in^2}{ft}$$

For #5 barrier bars (R501E), the bar is considered fully developed on the outside of the barrier toe due to the bend. On the traffic side of the barrier toe, refer to Figure 9.3.8 to determine how much of the 18" bar leg extends beyond the toe.

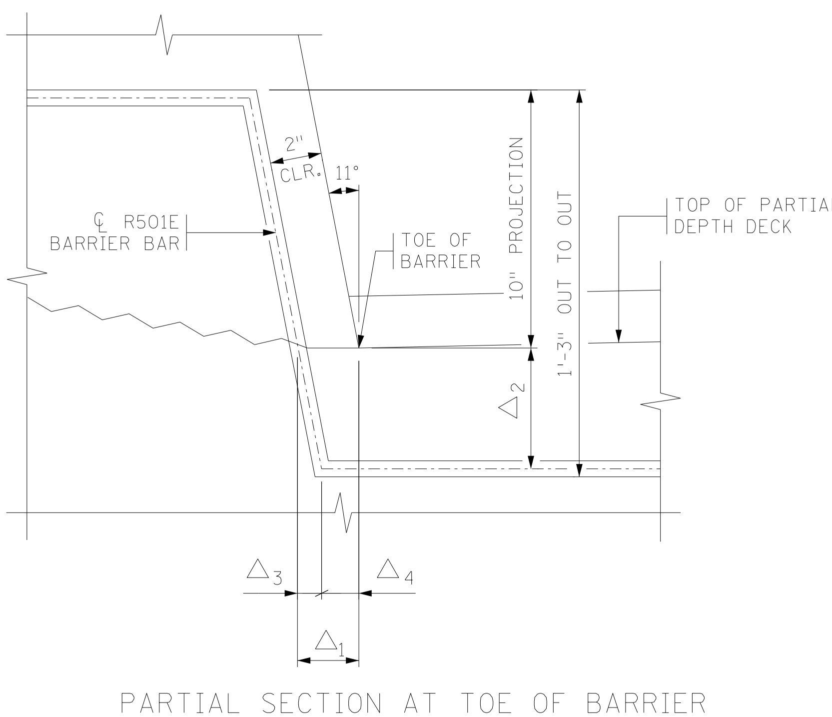


Figure 9.3.8

$$\Delta_1 = \frac{2 + 0.5 \cdot (0.625)}{\cos 11°} = 2.36 \text{ in}$$

$\Delta_2 = 15 - 10 - 0.5 \cdot (0.625) = 4.69$ in

$\Delta_3 = 4.69 \cdot (\tan 11°) = 0.91$ in

$\Delta_4 = 2.36 - 0.91 = 1.45$ in

Then:

$\ell_{davailable} = 18 - 1.45 = 16.55$ in

$$A_{s(bar)eff} = A_{s(bar)} \cdot \frac{\ell_{davailable}}{\ell_d} = 0.31 \cdot \frac{16.55}{22} = 0.23 \ \frac{in^2}{ft}$$

Now determine the distance from the bottom of the section to the neutral axis, c. Start by assuming that for all reinforcement, $\varepsilon_s > \varepsilon_y$.

Then:

$f_s = E_s \cdot \varepsilon_y = f_y$

$T_{s(top)} = A_{s(top)eff} \cdot f_y = 0.36 \cdot 60 = 21.60$ kips/ft

$T_{s(bot)} = A_{s(bot)eff} \cdot f_y = 0.23 \cdot 60 = 13.80$ kips/ft

$T_{s(bar)} = A_{s(bar)eff} \cdot f_y = 0.23 \cdot 60 = 13.80$ kips/ft

$T_{s(tot)} = 21.60 + 21.00 + 13.80 = 49.20$ kips/ft

The total compression force C_c is:

$C_c = 0.85 \cdot f'_c \cdot b \cdot a = 0.85 \cdot 4.0 \cdot 12.0 \cdot 0.85 \cdot c = 34.68 \cdot c$

Referring to Figure 9.3.7, find c by taking moments about P_n:

$21.60 \cdot (36.96 - 3.17)$
$+ 13.80 \cdot (36.96 + 1.27)$
$+ 13.80 \cdot (36.96 + 0.27)$
$- \quad 34.68 \cdot c \cdot (36.96 + 4.42 - 0.5 \cdot 0.85 \cdot c) = 0$

Solving, we get c = 1.25 in

Check if original assumption was correct, that $\varepsilon_s > \varepsilon_y$:

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

$$\varepsilon_{s(top)} = (8.84 - 1.25 - 1.25) \cdot \left(\frac{0.003}{1.25}\right) = 0.01522 > 0.00207$$

$$\varepsilon_{s(bot)} = (4.42 - 1.27 - 1.25) \cdot \left(\frac{0.003}{1.25}\right) = 0.00456 > 0.00207$$

$$\varepsilon_{s(bar)} = (4.42 - 0.27 - 1.25) \cdot \left(\frac{0.003}{1.25}\right) = 0.00696 > 0.00207$$

Therefore the assumption was correct.

Then,

$C_c = 34.68 \cdot c = 34.68 \cdot 1.25 = 43.35$ kips/ft

And,

$P_n = T_{s(top)} + T_{s(bot)} + T_{s(bar)} - C_c$

$= 21.60 + 13.80 + 13.80 - 43.35 = 5.85$ kips/ft

[1.3.2.1] The resistance factor ϕ for Extreme Event II limit state is 1.0. Therefore,

$\phi \cdot P_n = 1.0 \cdot 5.85 = 5.85$ kips/ft > 3.7 kips/ft OK

$\phi \cdot M_n = \phi \cdot P_n \cdot e_u$

$$= 1.0 \cdot 5.85 \cdot 36.96 \cdot \frac{1}{12} = 18.02 \ \frac{\text{kip-ft}}{\text{ft}} > 11.4 \ \frac{\text{kip-ft}}{\text{ft}} \quad \text{OK}$$

Therefore, the deck interior overhang region reinforcement is adequate.

Resistance of Deck End Overhang Region

The process for checking the end overhang region is the same as for the interior region. Use Table 9.2.1.1 for the bottom reinforcement and modify the top reinforcement in regions near an expansion joint. For a 36 inch Type S barrier supported by prestressed beams, try the following reinforcement:

Top reinforcement – Hooked #5 bars @ 5" ($A_{s(top)} = 0.74$ in^2/ft) over a distance of 8 feet from the joint.

Bottom reinforcement – #5 bars @ 7" ($A_{s(bot)} = 0.53$ in^2/ft)

In the end region, the front leg of the barrier bar also contributes to the strength of the overhang. There is some variation in bar spacing in the end region, but we will conservatively use:

Barrier front leg - #5 bars @ 12" ($A_{s(bar)} = 0.31$ in^2/ft)

First, determine the effective area of reinforcement for the deck bars located at the toe of the barrier. From Figure 5.2.2.6 of this manual, the development length, ℓ_d, for the hooked top bars is:

For #5 top bars @ 5" with side cover > 2.5" and 2" end cover, $\ell_d = 12''$

$\ell_{davailable} = 16.38'' > 12''$, so top bars are fully developed.

$A_{s(top)eff} = 0.74$ in^2/ft

The other bars are unchanged, so:

$A_{s(bot)eff} = 0.23\ in^2/ft$

$A_{s(bar)eff} = 0.23\ in^2/ft$

Now determine the distance from the bottom of the section to the neutral axis, c. Start by assuming that for all reinforcement, $\varepsilon_s > \varepsilon_y$. Then:

$f_s = E_s \cdot \varepsilon_y = f_y$

$T_{s(top)} = A_{s(top)eff} \cdot f_y = 0.74 \cdot 60 = 44.40$ kips/ft

$T_{s(bot)} = A_{s(bot)eff} \cdot f_y = 0.23 \cdot 60 = 13.80$ kips/ft

$T_{s(bar)} = A_{s(bar)eff} \cdot f_y = 0.23 \cdot 60 = 13.80$ kips/ft

$T_{s(tot)} = 44.40 + 13.80 + 13.80 = 72.00$ kips/ft

The total compression force C_c is:

$C_c = 0.85 \cdot f'_c \cdot b \cdot a = 0.85 \cdot 4.0 \cdot 12.0 \cdot 0.85 \cdot c = 34.68 \cdot c$

Referring to Figure 9.3.7, find c by taking moments about P_n:

$44.40 \cdot (36.48 - 3.11)$

$+ 13.80 \cdot (36.48 + 1.27)$

$+ 13.80 \cdot (36.48 + 0.27)$

$- \quad 34.68 \cdot c \cdot (36.48 + 4.42 - 0.5 \cdot 0.85 \cdot c) = 0$

Solving, we get c = 1.80 in

Check if original assumption was correct, that $\varepsilon_s > \varepsilon_y$:

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

$$\varepsilon_{s(top)} = (8.84 - 1.31 - 1.80) \cdot \left(\frac{0.003}{1.80}\right) = 0.00955 > 0.00207$$

$$\varepsilon_{s(bot)} = (4.42 - 1.27 - 1.80) \cdot \left(\frac{0.003}{1.80}\right) = 0.00225 > 0.00207$$

$$\varepsilon_{s(bar)} = (4.42 - 0.27 - 1.80) \cdot \left(\frac{0.003}{1.80}\right) = 0.00392 > 0.00207$$

Therefore the assumption was correct.

Then the compression force C_c is:

$C_c = 34.68 \cdot c = 34.68 \cdot 1.80 = 62.42$ kips/ft

And,

$P_n = T_{s(top)} + T_{s(bot)} + T_{s(bar)} - C_c$

$= 44.40 + 13.80 + 13.80 - 62.42 = 9.58$ kips/ft

[1.3.2.1] The resistance factor ϕ for Extreme Event II limit state is 1.0. Therefore,

$\phi \cdot P_n = 1.0 \cdot 9.58 = 9.58$ kips/ft > 7.9 kips/ft OK

$\phi \cdot M_n = \phi \cdot P_n \cdot e_u$

$$= 1.0 \cdot 9.58 \cdot 36.48 \cdot \frac{1}{12} = 29.12\ \frac{\text{kip-ft}}{\text{ft}} > 24.0\ \frac{\text{kip-ft}}{\text{ft}} \qquad \text{OK}$$

Therefore, the end region deck reinforcement is adequate for the end overhang region under Case 1.

L. Overhang Region Analysis, Case 2

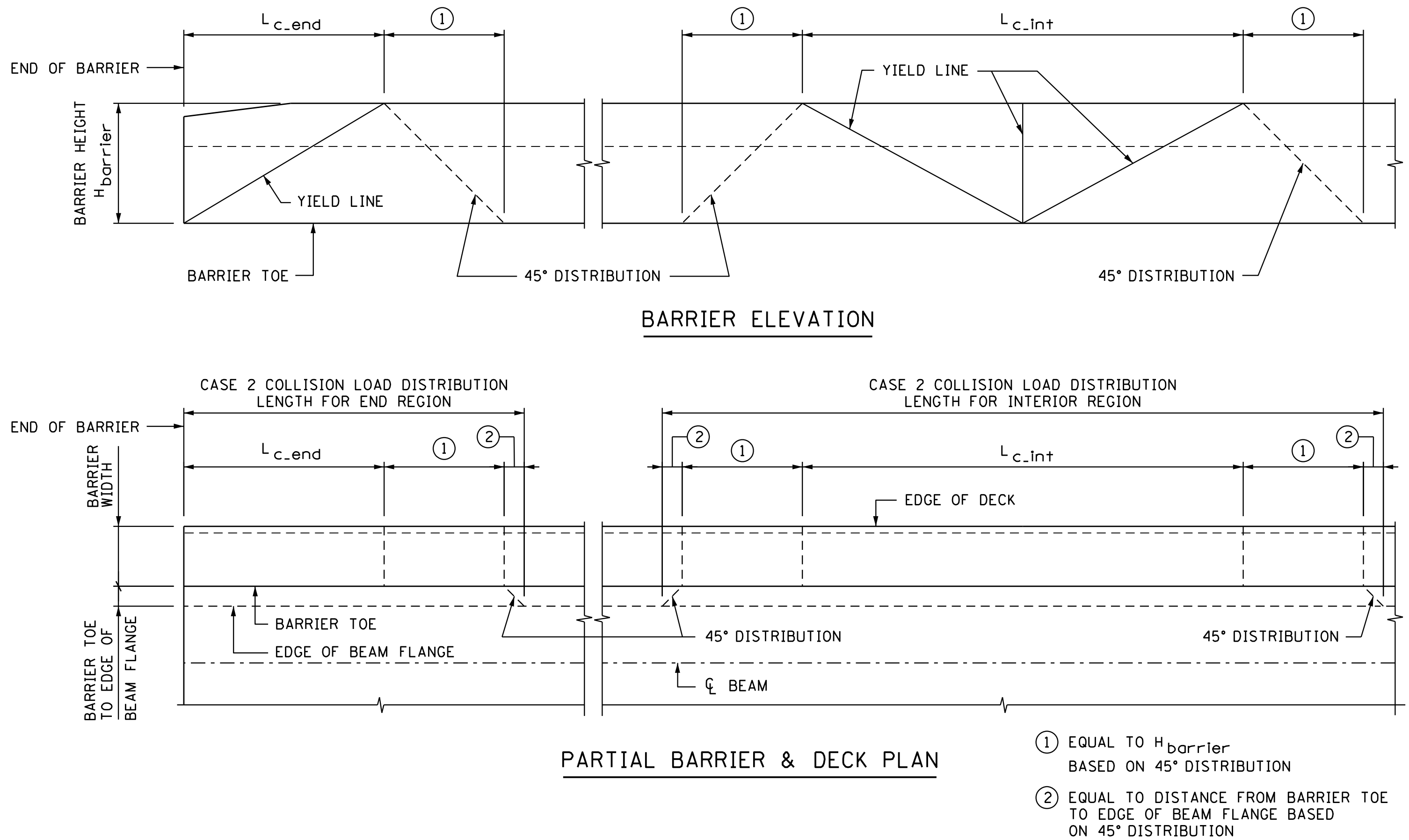
Case 2 is Extreme Event II checked at the edge of the beam flange for the dead load plus horizontal collision force plus live load. As noted earlier, live load is not considered due to the overhang geometry. At the edge of the beam flange, the analysis is very similar to that done for Case 1, except the dead load moment will be greater, the cross-section will be deeper, and the $A_{s(bar)eff}$ will be substantially less. Because the process is similar, the calculations are not shown and only the important values are included below to confirm the overhang is adequate.

Table 9.3.3 shows the loads for Case 2 and Table 9.3.4 shows results for the overhang resistance to the loads.

Table 9.3.3 – Determination of Loads for Case 2 Applied to a 1 ft Wide Strip

	Total w_{DC} (kips)	Total M_{DC} (kip-ft)	M_{cadj} (kip-ft)	F_{cadj} (kips) ①	e (ft)	F_{cdes} (kips)	M_{cdes} (kip-ft)	M_u (kip-ft)	P_u (kips)	e_u (ft)
Interior region	0.763	0.970	9.4	3.5	2.7	3.5	10.8	11.8	3.5	3.37
End region	0.763	0.970	20.5	7.4	2.8	7.4	23.6	24.6	7.4	3.32

① Because section for analysis is at the beam flange, the collision load is distributed over:
$L_c + 2 \cdot H_{barrier} + 2 \cdot$ (distance from edge of beam flange to barrier toe) for interior region.
$L_c + 1 \cdot H_{barrier} + 1 \cdot$ (distance from edge of beam flange to barrier toe) for end region.
See Figure 9.3.9.



Lc_end
①
①
Lc_int
①
END OF BARRIER
YIELD LINE
BARRIER HEIGHT Hbarrier
YIELD LINE
BARRIER TOE
45° DISTRIBUTION
45° DISTRIBUTION
BARRIER ELEVATION
CASE 2 COLLISION LOAD DISTRIBUTION LENGTH FOR END REGION
CASE 2 COLLISION LOAD DISTRIBUTION LENGTH FOR INTERIOR REGION
END OF BARRIER
Lc_end
①
②
②
①
Lc_int
①
②
BARRIER WIDTH
EDGE OF DECK
BARRIER TOE
EDGE OF BEAM FLANGE
45° DISTRIBUTION
45° DISTRIBUTION
℄ BEAM
BARRIER TOE TO EDGE OF BEAM FLANGE
PARTIAL BARRIER & DECK PLAN
① EQUAL TO Hbarrier BASED ON 45° DISTRIBUTION
② EQUAL TO DISTANCE FROM BARRIER TOE TO EDGE OF BEAM FLANGE BASED ON 45° DISTRIBUTION

Figure 9.3.9

Table 9.3.4 – Determination of Deck Overhang Resistance for a 1 ft Wide Strip

	Overhang Reinforcement	Effective A_s (in^2)	Assumed T_s (kips) ①	Arm to ϕP_n (in)	c (in)	Calculated $\varepsilon_s > \varepsilon_y$?	C_{conc} (kips)	ϕP_n (kips)	ϕM_n (kip-ft)
Interior region	Top bars: #4 @ 6″	0.40	24.00	36.94	1.34	0.01547 > ε_y OK	46.47	6.3	21.33
	Bottom bars: #5 @ 7″	0.34	20.40	41.38		0.00553 > ε_y OK			
	Barrier bars: #5 @ 12″	0.14	8.40	40.38		0.00777 > ε_y OK			
End region	Top bars: hooked #5 @ 5″	0.74	44.40	36.40	1.83	0.01043> ε_y OK	63.46	9.7	32.34
	Bottom bars: #5 @ 7″	0.34	20.40	40.78		0.00325> ε_y OK			
	Barrier bars: #5 @ 12″	0.14	8.40	39.78		0.00489> ε_y OK			

① Assumed T_s = (Effective A_s) · f_y

Based on the table values, the overhang deck reinforcement is adequate for Case 2.

M. Overhang Region Analysis, Case 4

The vertical collision load, F_v, for NCHRP Report 350 Test Level 4 represents the weight of a vehicle lying on top of the barrier and is applied over a length, L_v:

F_v = 18 kips

L_v = 18 feet.

The moment arm, L_{arm}, is:

L_{arm} = (overhang) − (coping) − (barrier width) − 0.5 · b_f + x_{cg}

= 42 − 2 − 16.38 − 0.5 · 34 + 10.22

= 16.84 in = 1.40 ft

Then, conservatively using a distribution length equal to L_v:

$$M_{vcollision} = \frac{F_v}{L_v} \cdot L_{arm} = \frac{18}{18} \cdot 1.40 = 1.40 \ \frac{\text{kip-ft}}{\text{ft}}$$

Dead load is the same as calculated previously for Case 2:

M_{DC} = 0.970 kip-ft/ft

Then M_u for Case 4 is:

Case 4 $M_u = M_{vcollision} + M_{DC}$

$= 1.40 + 0.970$

$= 2.37$ kip-ft/ft << Case 2 $M_u = 11.8$ kip-ft/ft

Therefore, by inspection, Case 4 is satisfied.

The deck overhang reinforcement is adequate for all cases.

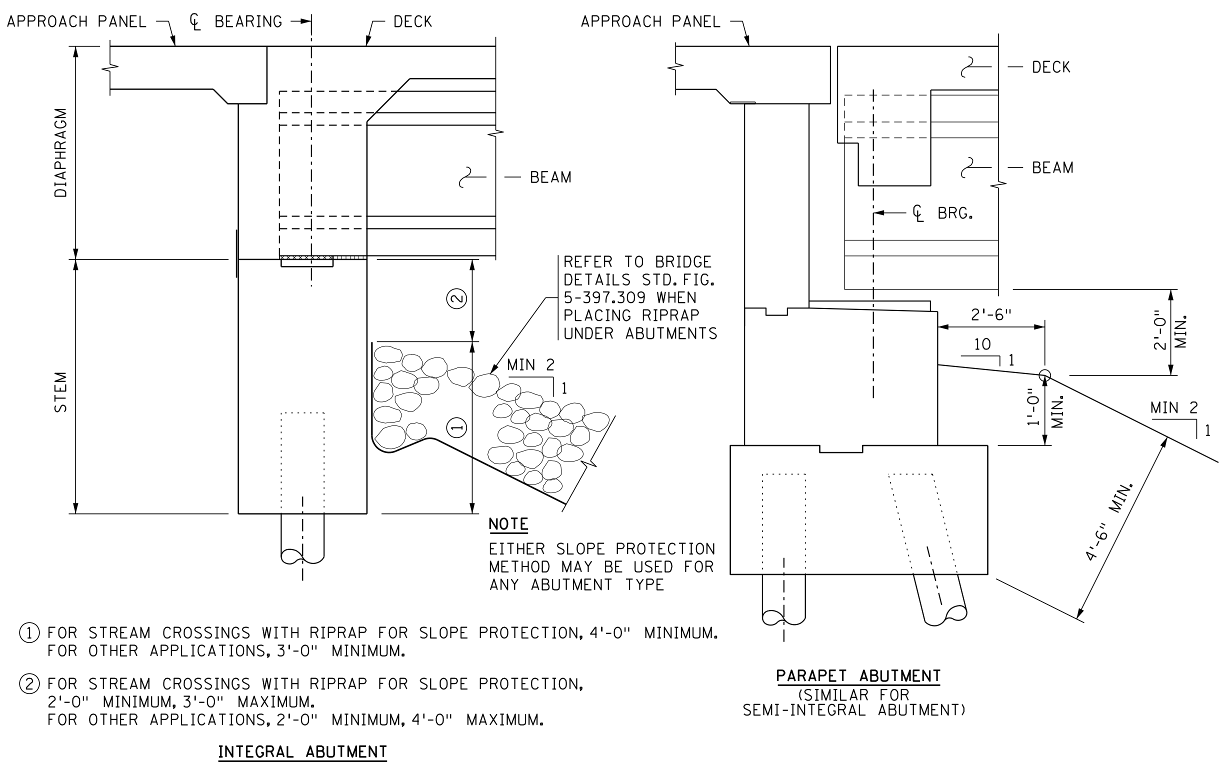


Figure 11.1.2
Cover and Clearance Requirements

For skewed abutments, acute angles are not allowed at corners where wingwalls intersect with the abutment stem. Instead, provide a 6 inch minimum chamfer or "square up" the corner to the wingwall at all acute angle corners.

Provide shrinkage and temperature reinforcement per BDM Article 5.2.6.

Detail sidewalk paving brackets with the same width and elevation as the roadway paving bracket. Sidewalks are to be supported on abutment diaphragm or abutment backwalls and detailed to "float" along adjacent wingwalls.

For semi-integral and parapet abutments, avoid projections on the back of abutments that are less than 4'-6" below grade. If shallower projections are necessary, slope the bottom to minimize frost heave effects.

For additional guidance on reinforcement detailing, see the web published document, *Suggested Reinforcement Detailing Practices,* which can be found at http://www.dot.state.mn.us/bridge/standards.html.

11.1.1 Integral Abutments

An integral abutment consists of an abutment stem supported by a single line of piles. The superstructure girders or slab bear on the stem. An abutment diaphragm is poured with the deck and encases the ends of the girders. The diaphragm is connected to the stem, making the superstructure integral with the abutment. Figure 11.1.1.2 shows typical integral abutment cross-section details and reinforcement. Figure 11.1.1.3 shows typical partial elevation details and reinforcement. Figure 11.1.1.4 shows Section A-A through the partial elevation. The reinforcement in these figures is typical for an integral abutment design based on the **Integral Abutment Reinforcement Design Guide** found in this section. For abutments that do not meet the design guide criteria, these figures may not accurately reflect the final abutment design.

Geometry

Use a minimum thickness of 3 feet for the abutment stem. For skewed bridges, increase the abutment thickness to maintain a minimum of 5 inches between the beam end and the approach slab seat (See Figure 11.1.1.2). Set the abutment stem height to be as short as practical while meeting the embedment and exposure limits shown in Figure 11.1.2. The requirements for stem height and embedment below grade are dependent on the site:

- For stream crossings where riprap is used for slope protection, set the abutment stem height on the low side of the bridge to 6 feet, with 4 feet below grade and 2 feet exposure. This is to ensure that the soil below the riprap covers the piles so they do not become exposed and susceptible to corrosion over the life of the bridge.

- For other applications, set the abutment stem height on the low side of the bridge to 5 feet, with 3 feet below grade and 2 feet exposure.

Note that the 4'-6" minimum depth below grade requirement for abutment footings does not apply to integral abutment stems.

Orient H-piling such that weak axis bending occurs under longitudinal bridge movements. Minimum pile penetration into abutment stem is 2’-6”. Avoid using 16” CIP and HP 14 piles or larger because of limited flexibility.

When the angle between the back face of wingwall and back face of abutment is less than 135 degrees, provide a 2’-0” x 2’-0” corner fillet on the back face of the wingwall/abutment connection. Include the fillet along the height of the abutment stem only, stopping it at the top of the stem.

NOTES:

1. CONSTRUCTION JOINT AT TOP OF ABUTMENT STEM WITH KEYWAYS BETWEEN BEAMS.
2. PERMISSIBLE CONSTRUCTION JOINT WITH KEYWAY, IF UPPER PORTION OF WINGWALL IS PLACED WITH DIAPHRAGM AND DECK.
3. PERMISSIBLE CONSTRUCTION JOINT WITH KEYWAY (ABOVE ABUTMENT STEM), IF UPPER PORTION OF WINGWALL IS PLACED WITH ABUTMENT.
4. MEMBRANE WATERPROOFING SYSTEM IF CONSTRUCTION JOINT IS USED.

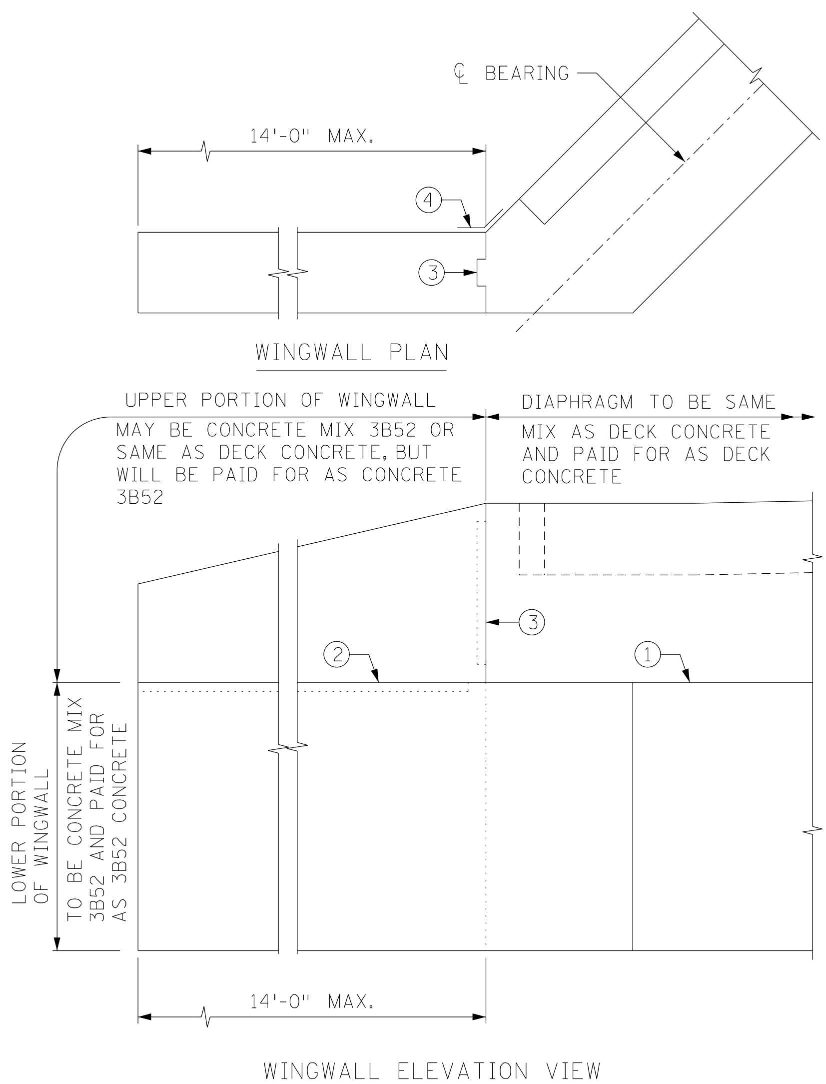


Figure 11.1.1.1b
Permissible Construction Joints For Integral Abutments With Flared Wingwalls

For new bridges, tie the approach panel to the bridge with stainless steel dowel bars that extend at a 45 degree angle out of the diaphragm through the paving bracket seat and forms a 180 degree hook with 4½ inches of clear cover to the top of the approach panel. (See bar S605S, Figure 11.1.1.2.) For repair projects, provide an epoxy coated dowel rather than stainless steel due to the shorter remaining life of the bridge. Include a ½ x 7 inch bituminous felt strip on the bottom of the paving bracket to allow rotation of the approach panel.

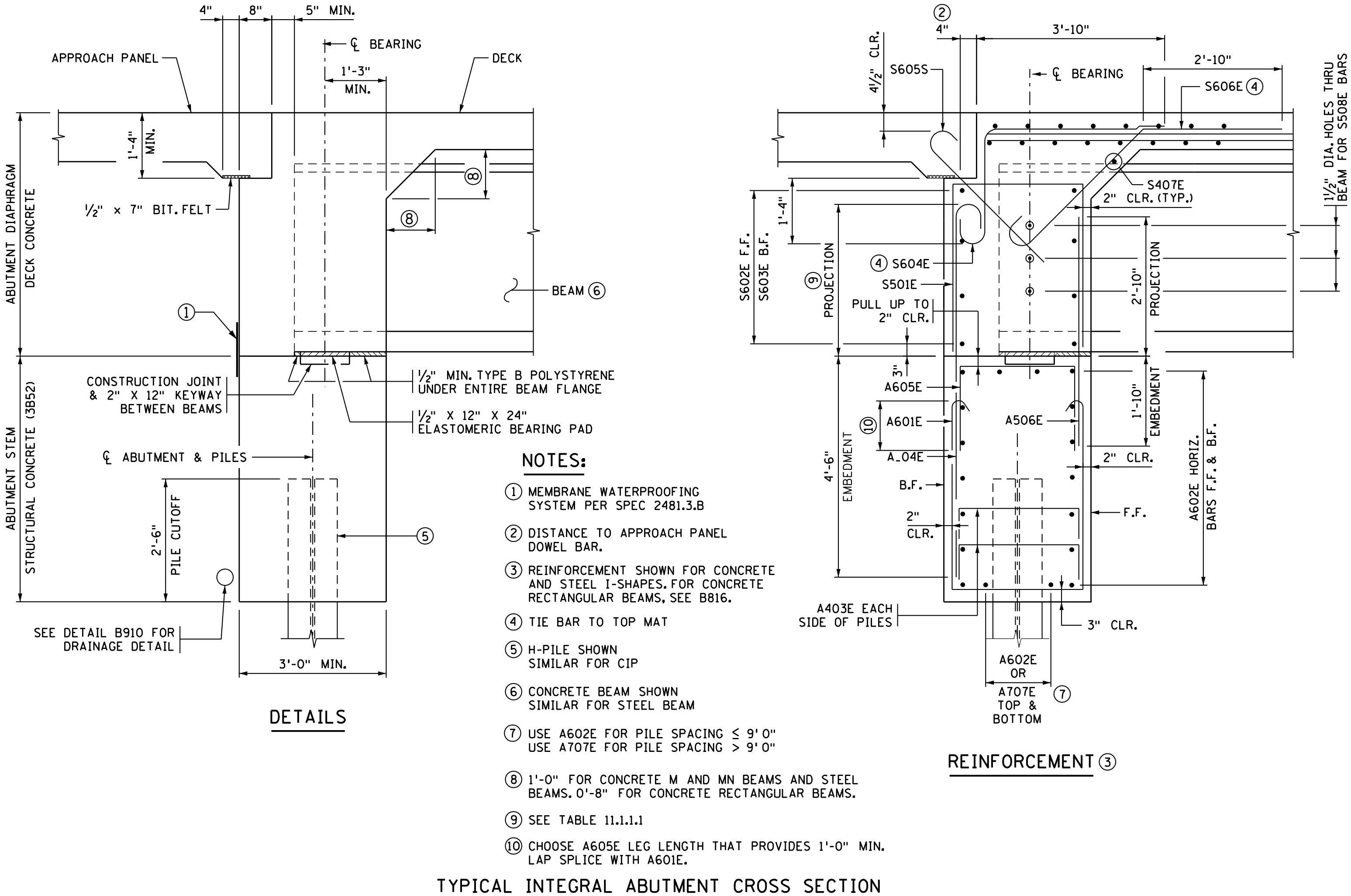
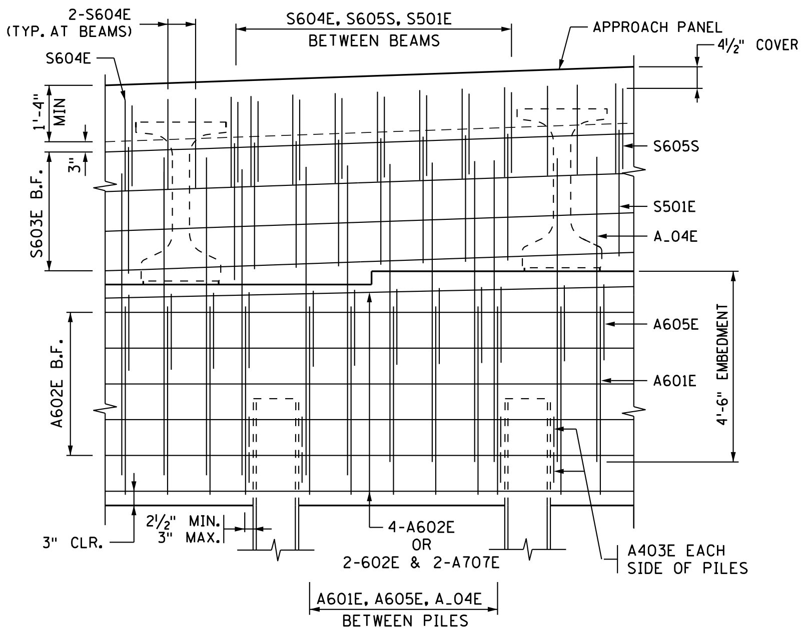
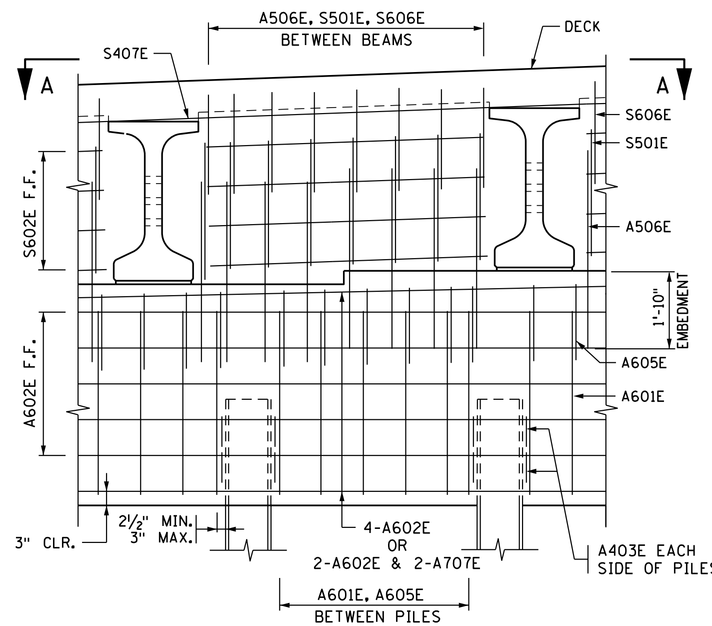


Figure 11.1.1.2

Figure 11.1.1.3



BACK FACE REINFORCEMENT ELEVATION



FRONT FACE REINFORCEMENT ELEVATION

INTEGRAL ABUTMENT REINFORCEMENT

BAR	SHAPE	LOCATION
A601E		STEM STIRRUP
A602E		STEM HORIZONTAL
A403E		PILE TIE
A_04E		STEM B.F. VERTICAL
A605E		STEM SEAT TIE
A506E		STEM F.F. VERTICAL
A707E		STEM HORIZONTAL
S501E		DIAPHRAGM TRANSVERSE
S602E		DIAPHRAGM F.F. HORIZ.
S603E		DIAPHRAGM B.F. HORIZ.
S604E		DIAPH./DECK TIE
S605S		DIAPH./APPROACH TIE
S606E		DIAPH./FILLET TIE
S407E		DIAPHRAGM F.F. HORIZ.
S508E		DIAPHRAGM THRU. BEAMS

NOTES:

ELEVATION DOES NOT SHOW HORIZONTAL BARS THROUGH BEAMS

ELEVATION DOES NOT SHOW DECK AND APPROACH PANEL REINFORCEMENT

TYPICAL INTEGRAL ABUTMENT PARTIAL ELEVATION

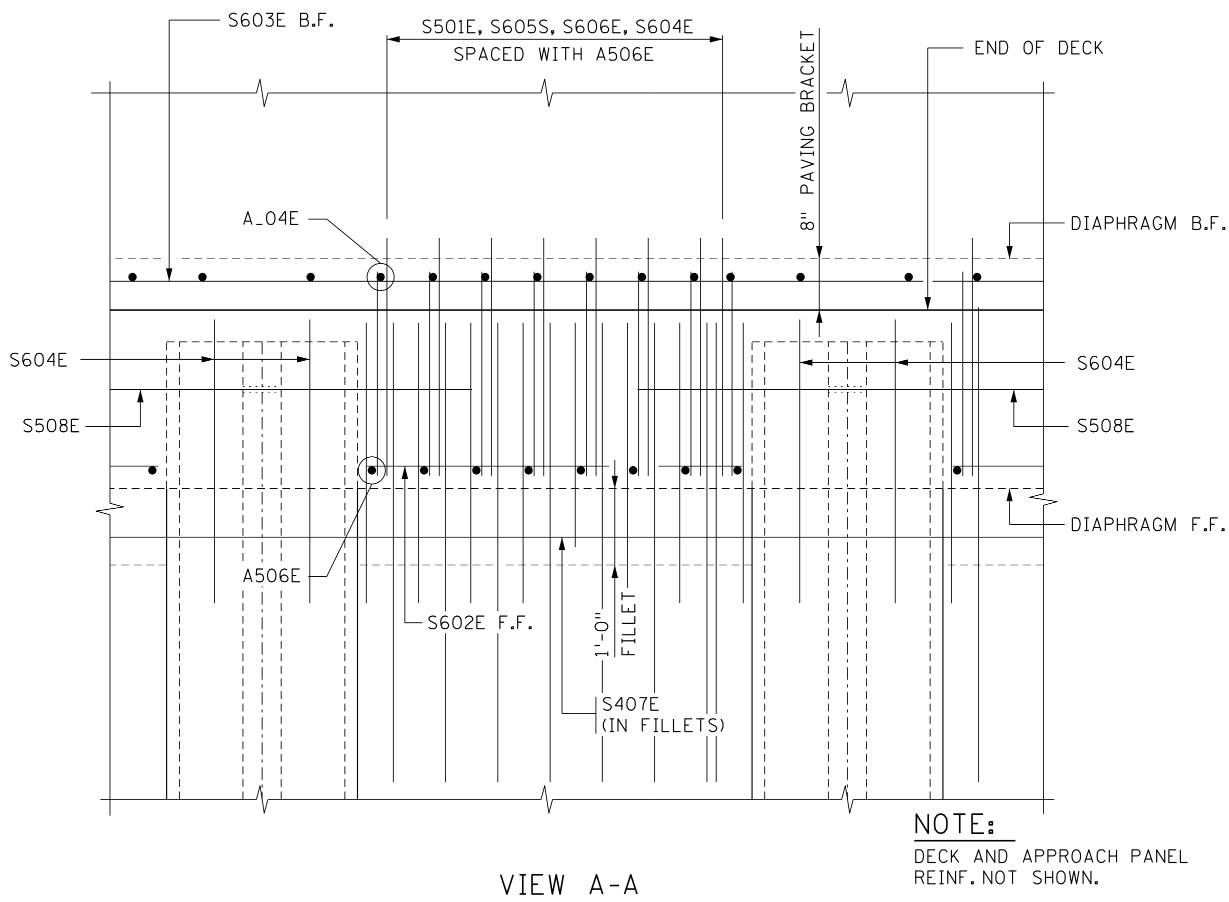


Figure 11.1.1.4

Integral Abutment Reinforcement Design Guide

Integral abutment reinforcement may be designed using the following guidance on beam and slab span bridges where all of the following criteria are met:

- All requirements of Articles 11.1 and 11.1.1 of this manual are met
- Beam height ≤ 72”
- Beam spacing ≤ 13’-0”
- Pile spacing ≤ 11’-0”
- Factored pile bearing resistance ϕR_n ≤ 165 tons
- Maximum abutment stem height ≤ 7’-0”
- Deck thickness plus stool height ≤ 15.5”

For beam heights that fall in between current MnDOT prestressed beam sizes (i.e. steel beams), use the values corresponding to the next largest beam height in the tables. Detail reinforcement using Figures 11.1.1.2 through 11.1.1.4.

For abutment stem shear reinforcement, use #6 bars spaced at a maximum of 12 inches between piles along the length of the abutment. These bars are designated A601E and A605E in Figures 11.1.1.2 and 11.1.1.3.

For abutment stem back face vertical dowels, select bar size, spacing and length from Table 11.1.1.1. Embed dowels 4'-6" into the stem. These bars are designated A_04E in Figures 11.1.1.2 and 11.1.1.3. Where table shows a maximum spacing of 12", space A_04E dowels with the abutment stem shear reinforcement (A601E) between piles. Where table shows a maximum spacing of 6", space every other A_04E dowel with the abutment stem shear reinforcement (A601E) between piles.

Table 11.1.1.1 Abutment Stem Back Face Vertical Dowels (A_04E) Minimum Required Bar Size and Length

Beam Size (in)	Bar Size & Max Spacing	Bar Projection into Abutment Diaphragm
14	#5 @ 12"	8"
18	#6 @ 12"	1'-0"
22	#6 @ 12"	1'-4"
27	#6 @ 12"	1'-6"
36	#7 @ 12"	2'-3"
45	#7 @ 12"	3'-0"
54	#6 @ 6"	3'-9"
63	#6 @ 6"	4'-6"
72	#6 @ 6"	5'-3"

For abutment stem front face vertical dowels, use #5 bars spaced at a maximum of 12 inches between beams. These bars are designated A506E in Figures 11.1.1.2 through 11.1.1.4. Do not space with the other abutment stem reinforcement, but instead space with the abutment diaphragm transverse bars (S501E).

For abutment stem front and back face horizontal reinforcement, use #6 bars spaced at a maximum of 9 inches. These bars are designated A602E in Figures 11.1.1.2 and 11.1.1.3. Account for changes in abutment seat height by varying bar spacing or the number of bars.

For the abutment stem top and bottom longitudinal bars, use 4-#6 bars on the top and bottom faces of the stem for piles spaced at 9 feet or less. These bars are designated A602E in Figures 11.1.1.2 and 11.1.1.3. When pile spacing

w_p = passive pressure calculated at the elevation of the bottom of abutment diaphragm and applied as a uniform pressure on the abutment stem

= $p_p \cdot h_{stem}$

L = pile spacing

Design abutment stem top and bottom horizontal bars for vertical loads due to girder reactions, including dynamic load allowance of 33%. Consider the stem to be a continuous beam with piles as supports. Also, check that the front and back face horizontal bars meet the longitudinal skin reinforcement provisions of LRFD Article 5.6.7.

Similar to abutment stem, design abutment diaphragm horizontal bars for the passive soil pressure which results when the bridge expands. For this case, consider the diaphragm to be a continuous beam with the superstructure girders as supports.

For crack control checks, assume a Class 1 exposure condition (γ_e=1.00).

For size and spacing of all other abutment diaphragm bars, refer to the **Integral Abutment Reinforcement Design Guide.**

11.1.2 Semi-Integral Abutments

Semi-integral abutments are similar to integral abutments in that the superstructure and approach panel are connected and move together. Unlike integral abutments, the superstructure is supported on bearings that allow movement independent from the abutment stem. The abutment stem is stationary and is supported by a spread footing or a pile cap on multiple rows of piles. Figure 11.1.2.1 illustrates typical semi-integral abutment cross-section details and reinforcement.

Geometry

Skews on semi-integral abutments are limited to 30 degrees when wingwalls are parallel to the roadway in order to prevent binding of the approach panel/wingwall interface during thermal movement. For other wingwall configurations, bridge length and skew limits are the same as those for integral abutments. Whenever the skew is greater than 30 degrees, provide a concrete guide lug to limit unwanted lateral movement.

Refer to Figure 11.1.2 for minimum cover and clearance requirements.

Provide a minimum abutment stem thickness of 4′-0″. If abutment aesthetics are required, include the rustication depth within the 4′-0″

minimum thickness (i.e., 3′-10″ minimum structural concrete and 2″ rustication).

Provide pedestals under the bearings and slope the bridge seat between pedestals to provide drainage toward the abutment front face. A standard seat slope provides one inch of fall from the back of the seat to the front of the seat. In no case should the slope be less than 2 percent. Set pedestals back 2 inches from front face of abutment. Minimum pedestal height is to be 3 inches at front of pedestal. Preferred maximum pedestal height is 9 inches. Provide #5 reinforcing tie bars at 6 inch to 8 inch centers in both directions under each bearing. For bearing pedestals over 9 inches tall, provide column ties in addition to other reinforcement. Provide 2 inches of clear cover for horizontal pedestal bars in the bridge seat. Provide a minimum of 2 inches of clear distance between anchor rods and reinforcing tie bars.

Provide a 3 inch minimum horizontal gap between the abutment diaphragm lug and abutment stem.

When the angle between the back face of wingwall and back face of abutment is less than 135 degrees, provide a 2′-0″ x 2′-0″ corner fillet on the back face of the wingwall/abutment connection. Extend the fillet from the top of footing to the top of abutment stem on the back face.

Provide a vertical construction joint at the abutment to wingwall connection. Detail the joint location with the goal of making it inconspicuous by considering the wingwall layout, abutment skew angle, fascia beam offset distance from the abutment edge, and aesthetic treatment. For wingwall layout parallel to the roadway, the preferred construction joint location is through the thickness of the abutment in a plane coincident with the back face of the wingwall. For abutments with geometry or aesthetic features that preclude this, another location such as at a vertical rustication line in the abutment or wingwall front face is appropriate. When aesthetic features govern the joint location, the Preliminary Bridge Plans Engineer will provide acceptable construction joint locations in the preliminary plan based on guidance from the Bridge Architectural Specialist. Avoid horizontal construction joints in the wingwall unless absolutely needed. If horizontal joints are needed, locate the joints at a rustication line.

Provide 1 inch of Type B (low density) polystyrene in the vertical gap between the end diaphragm and back face of wingwall. Also, provide 1 inch of Type A (high density) polystyrene in the horizontal gap between the end diaphragm lug and abutment stem. Additionally, provide a membrane waterproofing system with a 1 inch backer rod to allow movement to occur without tearing the waterproofing. Note that the membrane waterproofing and backer rod are included in item “Structural Concrete (___)” and the geotextile filter is included in item “Bridge Slab Concrete (___)”. See Figures 11.1.2.1 and 11.1.2.2 for details.

Place 1½ inches of Type B (low density) polystyrene between the edge of the approach panel and the back face of the wingwall to minimize binding of the approach panel on the wingwall interface during thermal movement. See approach panel standard plan sheets 5-297.225 and .229 for more details.

Detail semi-integral abutments with a drainage system behind the wall (Detail B910). Outlet the 4 inch drains through the wingwalls and backslopes.

For new bridges, tie the approach panel to the bridge with stainless steel dowel bars that extend at a 45 degree angle out of the diaphragm through the paving bracket seat and forms a 180 degree hook with 4½ inches of clear cover to the top of the approach panel. (See bar #6S, Figure 11.1.2.1.) For repair projects, provide an epoxy coated dowel rather than stainless steel due to the shorter remaining life of the bridge. Include a ½ inch x 7 inch bituminous felt strip on the bottom of the paving bracket to allow rotation of the approach panel.

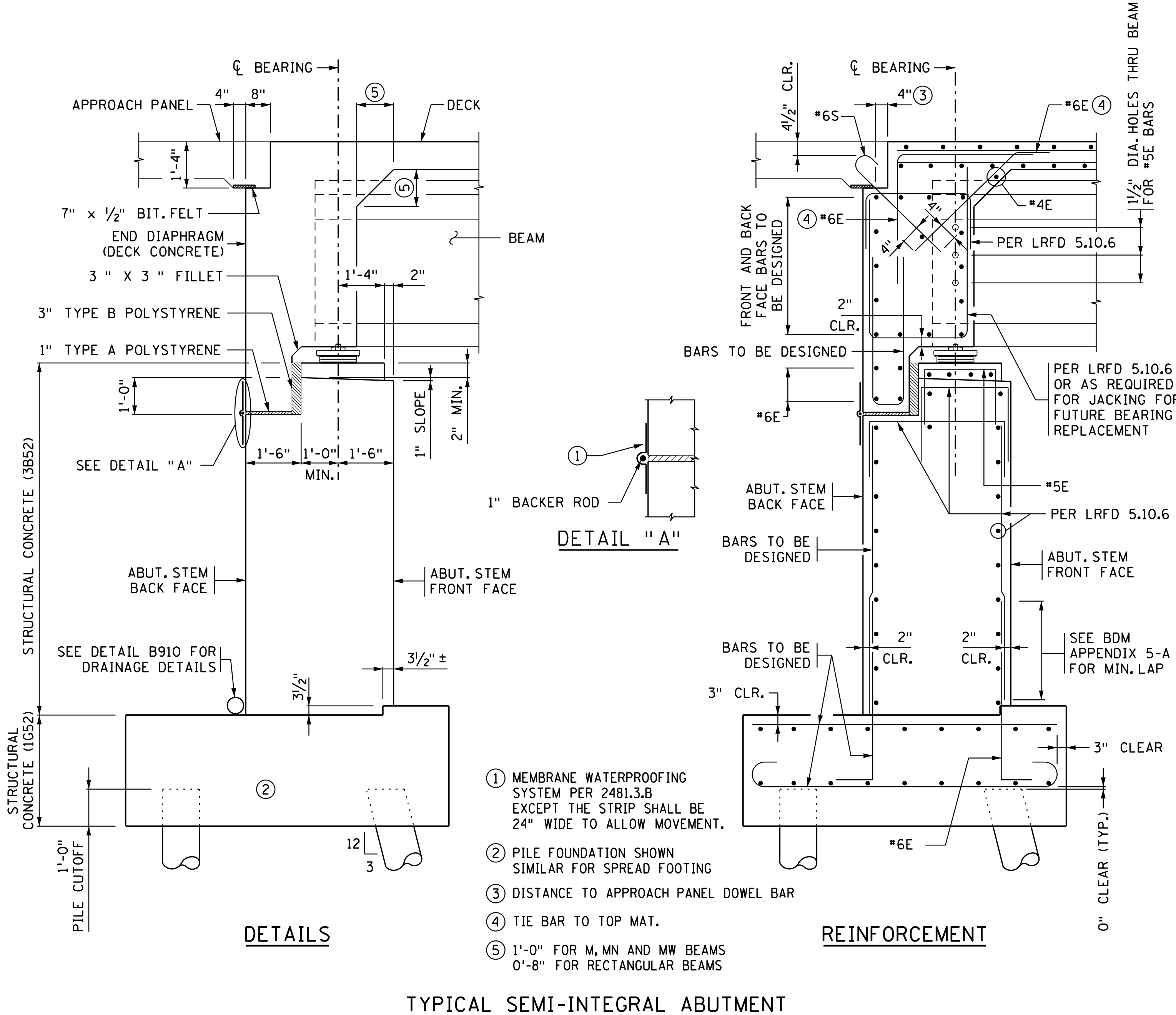


Figure 11.1.2.1

Distribute superstructure loads (dead load and live load) over the entire length of abutment. For live load, apply the number of lanes that will fit on the superstructure adjusted by the multiple presence factor.

For resistance to lateral loads, see Article 10.2 of this manual to determine pile resistance in addition to load resisted by battering.

Design footing thickness such that no shear reinforcement is required. Performance of the Service I crack control check per LRFD 5.6.7 is not required for abutment footings.

Design abutment stem and backwall for horizontal earth pressure and live load surcharge loads.

For stem and backwall crack control check, assume a Class 1 exposure condition (γ_e = 1.00).

For skewed bridges, place end diaphragm transverse bars parallel to the beams.

11.1.3.1 Low Abutments

Low abutments shall have vertical contraction joints at about a 32 foot spacing. (See Detail B801.)

Detail low abutments with a drainage system (Detail B910). Outlet the 4 inch drains through the wingwalls and backslopes.

Figure 11.1.3.1.1 contains typical dimensions and reinforcing for low parapet abutments.

11.1.3.2 High Abutments

High abutments shall have vertical construction joints (with keyways) at about a 32 foot spacing.

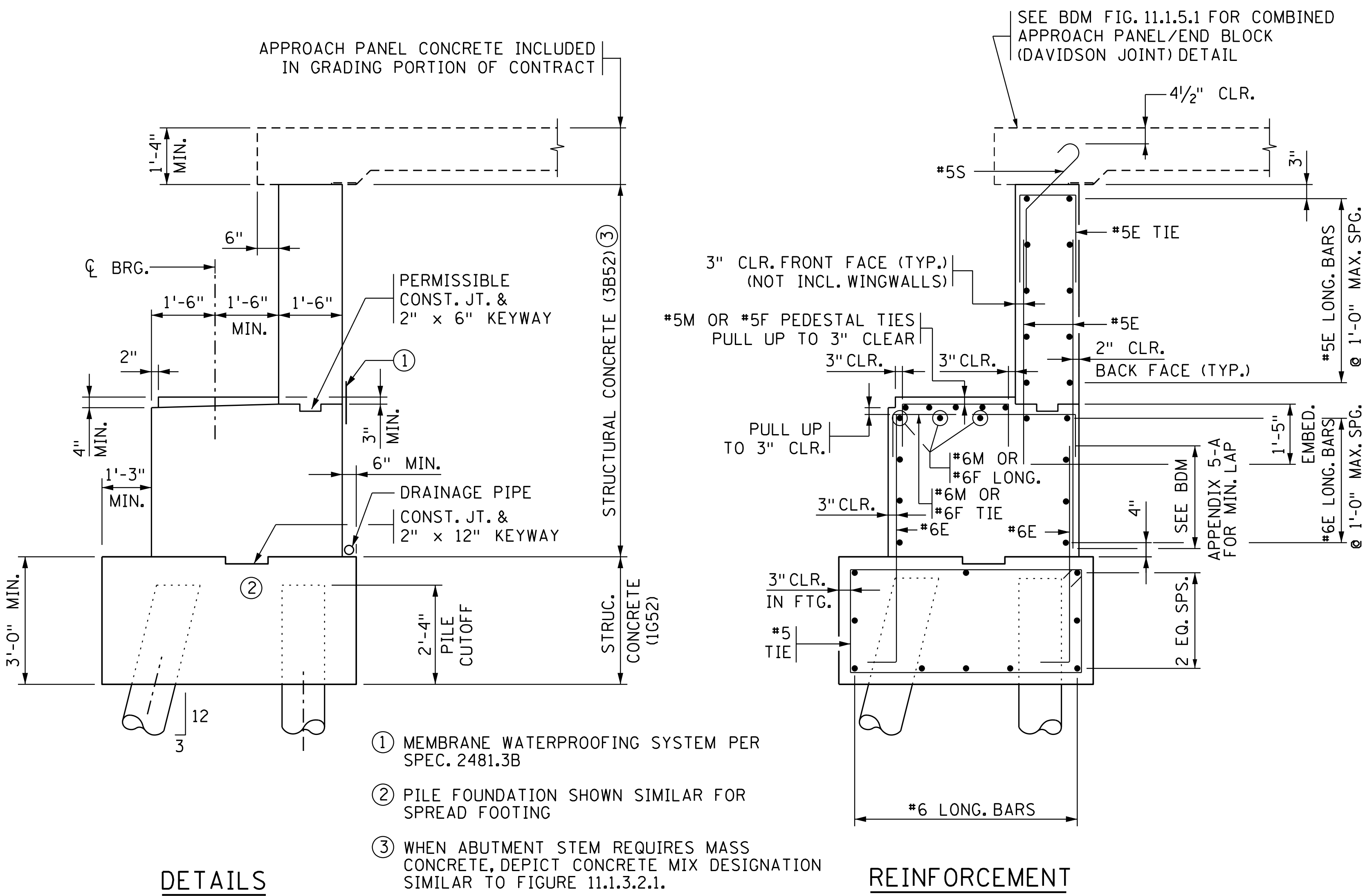
Detail high abutments with a drainage system (Detail B910). Outlet the 4 inch drains through the wingwalls and backslopes. Granular backfills at railroad bridge abutments typically includes perforated pipe drains.

Figure 11.1.3.2.1 illustrates typical high abutment dimensions and reinforcing.

11.1.3.3 Abutments Behind MSE Walls

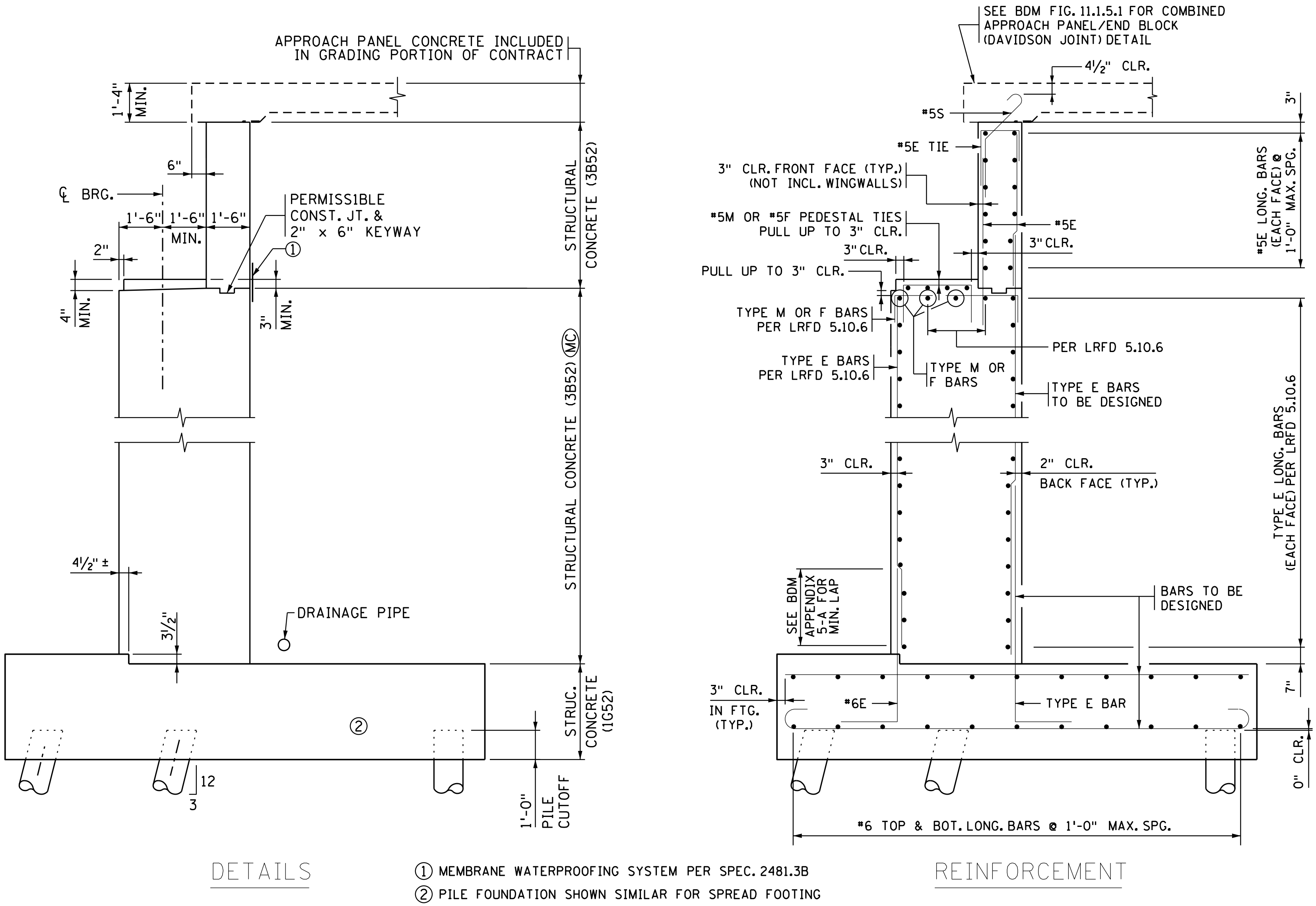
[Future manual content]

Figure 11.1.3.1.1



TYPICAL LOW PARAPET ABUTMENT

Figure 11.1.3.2.1



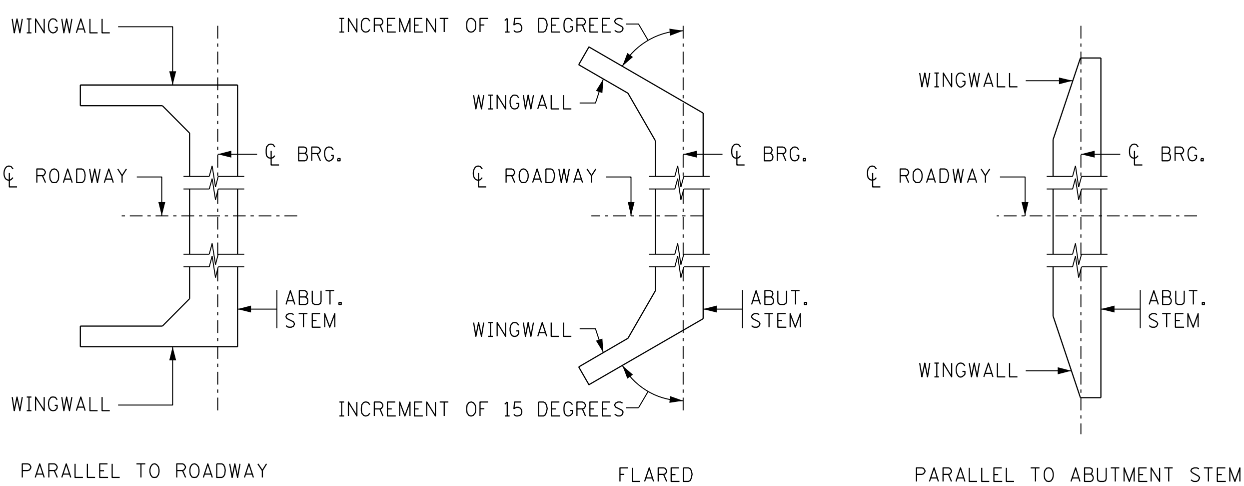
TYPICAL HIGH PARAPET ABUTMENT

11.1.4 Wingwalls

Wingwalls are the retaining portion of the abutment structure that are located outside the abutment stem.

11.1.4.1 Wingwall Geometry

Wingwalls can be oriented parallel to the roadway, flared, or parallel to the abutment stem. See Figure 11.1.4.1.1.



WINGWALL ORIENTATION

Figure 11.1.4.1.1

Factors to consider when choosing wingwall orientation include:

- Flared wingwalls
  This configuration is generally the most economical because it provides the shortest length of wall. This makes flared wingwalls the preferred wingwall when there are no aesthetic or lateral constraints.

  For river/stream crossings, a flared wingwall configuration generally provides the best hydraulic flow coefficient, thereby reducing scour implications.

  A flared wingwall configuration can provide the lowest visual impact, which makes it an appropriate choice when aesthetics do not require high visibility features.

  Set the flare angle between the wingwall and centerline of bearing in increments of 15 degrees. When bridge skew results in an optimum flare angle between wingwall and roadway alignment of less than 25 degrees, consider changing to a parallel to roadway length, pier fixity, and wingwall flare angle, loading from passive soil pressure should be considered.

- For wingwalls oriented parallel to the abutment stem, design for passive soil pressure loading.

For semi-integral and parapet abutment wingwalls, use the following guidance (see Figure 11.1.4.2.1):

- Design the vertical back face wingwall dowels to resist the entire moment caused by the horizontal loads.
- Assume active soil pressure using an equivalent fluid weight of 0.033 kcf.
- Design wingwall horizontal back face reinforcement at end of footing to resist loads applied to horizontal cantilever region.
- Depending upon the wingwall height tied to the abutment stem and the length of wingwall supported by the footing, consider analyzing wingwall as a plate fixed on 2 edges to:
  - determine the stem-to-wingwall horizontal reinforcement.
  - determine the front face reinforcement in wingwall center region.

  For all wingwalls with a height greater than 20 feet, a plate analysis is required.
- Provide reinforcement through the construction joint at the intersection of the wing and abutment wall to transfer wingwall loads to the abutment, if applicable.
- Where abutment wall and wingwalls are supported by a common pile supported footing:
  - Design the piles considering that the wingwalls, abutment wall, and common footing act as a system. Based on the wingwall length and footing geometry, use engineering judgement to determine which piles can share the loads applied to abutment wall and wingwall. For example, consider that the piles under the abutment wall will share in resisting the wingwall lateral loads, provided that the wingwall footing shear resistance is adequate to transfer the lateral loads. Conversely, the wingwall piles will share in resisting the abutment wall lateral loads.
  - Provide one set of pile tables in the bridge plan for each abutment based on the pile with the maximum computed load. Tables are shown in BDM Appendix 2-C under G. **FOUNDATIONS**. Although computed loads for abutment wall piles and wingwall piles will likely differ, all of the piles in the abutment will be driven to the same criteria.

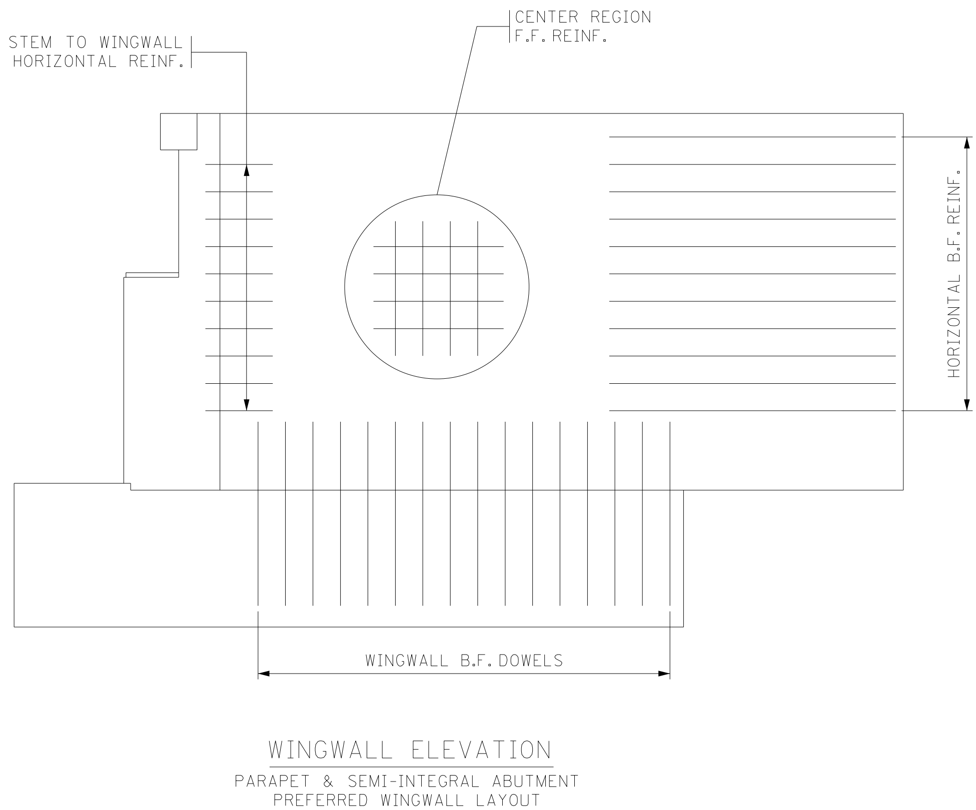


Figure 11.1.4.2.1

When checking crack control for wingwalls, use the Class 1 exposure condition (γ_e = 1.00).

11.1.5 Bridge Approach Panels

Details for bridge approach panels for concrete and bituminous roadways are typically included in the roadway plans and are provided on roadway Standard Plans 5-297.223 through 5-297.228.

Approach panels are a roadway pay item. The preliminary bridge plan provides information to the roadway designer regarding the appropriate expansion joint type, sill detail, and abutment type. Coordinate approach panel curb and median transitions with roadway designers.

The bridge designer is responsible for the design of barriers or parapets on the approach panel. This includes the reinforcement and any cover plates needed (refer to bridge Standard Figure 5-397.635).

For approach panels with a skew exceeding 15 degrees, the bridge designer is also responsible for completing the bill of reinforcement for the approach panel (refer to roadway Standard Plan 5-297.224). An approach panel guidance document along with other design and construction resources are available on the MnDOT Design Standards Unit website.

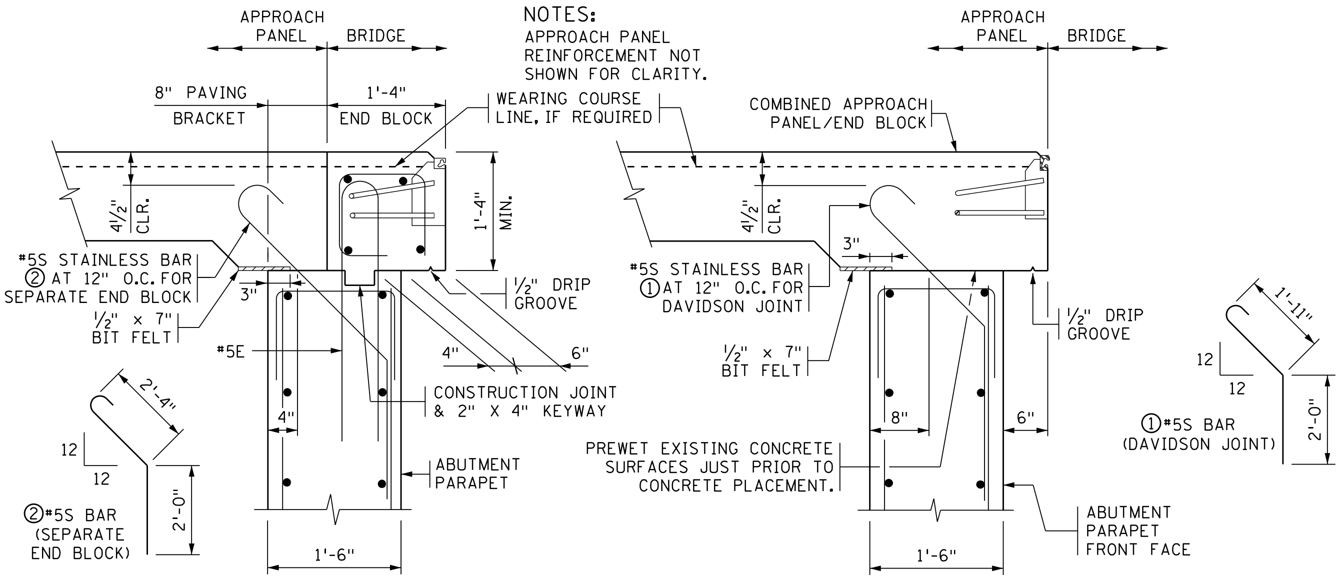
Use a concrete wearing course on approach panels when the bridge deck has a concrete wearing course. The wearing course will be placed on the bridge superstructure and the approach panels at the same time. Therefore, include the wearing course quantity for both the approach panels and the superstructure when computing the wearing course pay item quantity for the bridge plan.

Past practice (before 2020) for parapet abutments was to cast an end block on top of the parapet and include an 8 inch paving bracket to support the approach panel. See Figure 11.1.5.1a. The approach panel was cast separately, leaving a cold joint between the approach panel and the end block. When a separate end block was used, the "BEGIN BRIDGE" and "END BRIDGE" locations were at the cold joint location on the approach panel side of the end block.

Current practice for most parapet abutments is to cast the approach panel and end block monolithically without a cold joint as shown in Figure 11.1.5.1b. This was named the "Davidson joint," after Dave Davidson from District 1 who suggested the change. Note that for this case, the "BEGIN BRIDGE" and "END BRIDGE" locations are at the approach panel side of the expansion joint. Use of a separately cast end block (Figure 11.1.5.1a) still applies to parapet abutments for the following cases:

1) Long-term approach settlement greater than ½" is expected.
2) Modular joints are used at the abutment.
3) At the discretion of the Regional Bridge Construction Engineer.

For the Davidson joint, set the top of parapet 1′-4" minimum below the top of roadway surface. Include the #5S stainless steel bars, which tie the approach panel to the bridge, in the abutment bar list for both the Davidson joint and separate end block detail. Also, contact the road designer to coordinate the length and limits of the approach panel.



(a) Appr. Panel w/ Separate End Block ***(b) Davidson Joint - Combined Appr. Panel/End Block***

Parapet Abutment End Block Reinforcement

Figure 11.1.5.1

Cantilevered Approach Panels

It is preferable to locate bridge rail sections that extend beyond the bridge and connect to guardrail on the approach panel. Typically, this is done by locating the abutment wingwalls outside the bridge rail. However, historic details and geometric constraints occasionally require the bridge rail to be located above the wingwall. In these situations, consider using a bridge rail on cantilevered approach panel over the wingwall or retaining wall (not including MSE walls). Refer to Figure 11.1.5.2. Use the following guidance for selecting potential situations for use of a cantilevered approach panel:

- For existing bridges, consider a cantilevered approach panel where the existing rail is mounted on the wingwall for parapet, integral, or fixed semi-integral abutment type bridges with:
    - A planned redeck with approach panel replacement.
    - A planned mill and overlay with end post or approach panel replacement.
- For new bridges, consider a cantilevered approach panel for parapet abutment bridges where the wingwalls are prohibited from being placed outside the approach panel due to project constraints. Ensure the wingwall extends the full length of the approach panel.

If the approach panel is cantilevered over the wingwall (regardless of abutment type), consider the potential effects of settlement if the wingwall does not extend to the end of the approach panel. It may be prudent to consider shortening the length of the approach panel. Cantilevered approach panels should not be used where post-construction embankment settlements are predicted to exceed 1 inch.

Cantilevered approach panels should not be used on contraction type abutments. Coordinate with the retaining wall designers if there are retaining wall tie-ins.

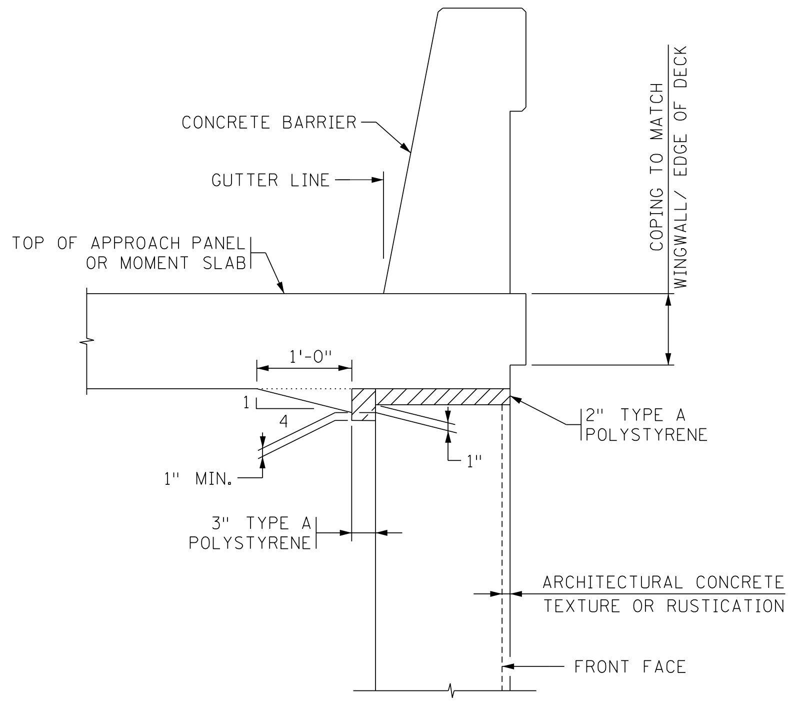


Figure 11.1.5.2

11.1.6 Bridge Approach Treatment

For typical new bridge projects, the preliminary bridge plan provides information to the roadway designer regarding the appropriate bridge approach treatment detail to include in the roadway plans (for a bridge with integral abutments or a bridge with abutments on a footing). For repair projects and other projects where no separate grading plans are prepared, make sure that bridge approach treatments are consistent with the appropriate roadway Standard Plan 5-297.233 or 5-297.234.

11.2 Piers

A wide variety of pier types are used in bridge construction. The simplest may be pile bent piers where a reinforced concrete cap is placed on a single line of piling. A more typical pier type is a cap and column pier, where columns supported on individual footings support a common cap. The spacing of columns depends on the superstructure type, the superstructure beam spacing, the column size, and the aesthetic requirements. A typical cap and column pier for a roadway may have from three to five columns. At times wall piers may be used to support superstructures. Where extremely tall piers are required, hollow piers may be considered. Specialty bridges such as segmental concrete bridges may use double-legged piers to reduce load effects during segmental construction.

11.2.1 Geometrics

When laying out piers, consider the economy to be gained from reusing forms (both standard and non-standard) on different piers constructed as part of a single contract.

Dimension piles, footing dimensions, and center of columns to working points.

For pier caps (with cantilevers) supported on multiple columns, space the columns to balance the dead load moments in the cap.

Provide a vertical open joint in pier caps that have a total length exceeding 100 feet. The design may dictate that additional pier cap joints are necessary to relieve internal forces.

Label the ends of piers (South End, North End, etc.).

Concrete Pier Columns
The minimum column diameter or side of rectangular column is 2′-6″.

To facilitate the use of standard forms, detail round and rectangular pier columns and pier caps with outside dimensions that are multiples of 2 inches. As a guide, consider using 2′-6″ columns for beams 3′-0″ or less

$$V_{u,RowI} = (\text{Pile Fraction Outside Critical Section}) \left(\frac{\text{Pile Reaction}}{\text{Pile Spacing}}\right)$$

$$V_{u,RowI} = (4.08/12) \cdot (199.7/8.00) = 8.5 \text{ kips/ft width}$$

The shear due to the Row III piles governs.

[5.7.3.3] There is no shear reinforcement, so the nominal shear capacity of the footing is:

$$V_n = V_c$$

An upper limit is placed on the maximum nominal shear capacity a section can carry. The maximum design shear for the footing heel is:

$$V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_{v,heel} = 0.25 \cdot 4.0 \cdot 12.0 \cdot 28.92 = 347.0 \text{ kips}$$

The concrete shear capacity of a section is:

$$V_C = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

[5.7.3.4.1] The distance from the point of zero shear to the backface of the abutment x_{vo} is:

$$X_{vo} = 54.0 + 6.0 = 60.0 \text{ in}$$

$$3 \cdot d_v = 3 \cdot 28.92 = 86.8 \text{ in} > 60.0 \text{ in}$$

Therefore, $\beta = 2.0$

For a 1 ft. wide section, substituting values into V_c equation produces:

$$V_C = 0.0316 \cdot 2.0 \cdot \sqrt{4} \cdot 12 \cdot 28.92 = 43.9 \text{ kips}$$

This results in:

$$V_n = V_c = 43.9 \text{ kips} < 347.0 \text{ kips} \qquad \text{(OK)}$$

Including the shear resistance factor, the shear capacity is found to be:

$$V_r = \phi V_n = 0.90 \cdot 43.9 = 39.5 \text{ kips} > 22.3 \text{ kips} \qquad \text{(OK)}$$

[5.12.8.6.1]

Check Two-Way Shear in Footing

Punching of an individual pile through the abutment footing is checked next. The critical section for two-way shear is located at $0.5 d_v$ from the perimeter of the pile. The Row I pile at the corner is the governing case because it has the largest load with the shortest length of critical section. See Figure 11.4.1.5.

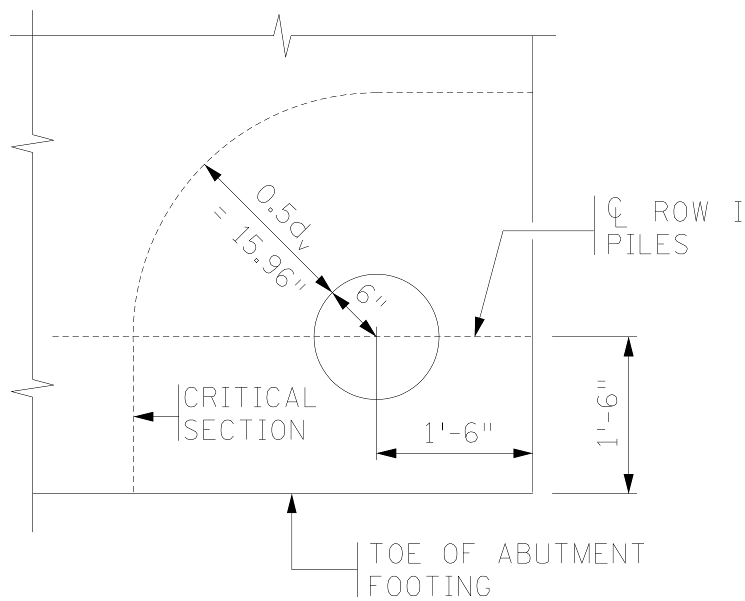


Figure 11.4.1.5
Partial Footing Plan

The length of the critical section is:

$$b_o = 18 + 0.5 \cdot \pi \cdot (15.96 + 6) + 18 = 70.5 \text{ in}$$

[5.12.8.6.3]

$$\phi V_n = \phi \left(0.126 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_o \cdot d_{v,toe}\right)$$

$$= 0.90\ (0.126 \cdot 1 \cdot \sqrt{4} \cdot 70.5 \cdot 31.92)$$

$$= 510.4 \text{ kips}$$

V_u = Row I Factored Pile Load = 199.7 kips < 510.4 kips (OK)

14. JOINTS AND BEARINGS

Expansion joints and bearings provide mechanisms to accommodate movements of bridges without generating excessive internal forces. This section provides guidance on joint and bearing selection and the movement and loads that must be used in their designs.

14.1 Bridge Movements and Fixity

To determine movements for bearings and joints, the point of fixity must be established for the bridge or bridge segment. The point of fixity is the neutral point on the bridge that does not move horizontally as the bridge experiences temperature changes. Use the following guidance concerning bridge fixity:

1) For single span structures, fix the bearings at the low end of the bridge.
2) For typical two-span structures, fix the bearings at the pier. For bridges with tall or flexible piers that are located on slopes, the superstructure may tend to move toward the downhill end, causing maintenance problems at the uphill end due to a wider joint than anticipated. For these bridges, consider providing fixed bearings at the downhill abutment.
3) For structures with three or more spans, investigate the longitudinal stiffness of the bridge. The longitudinal stiffness is a function of the interaction between pier stiffnesses, bearing types and joint locations. Consider the following:
    a) The number and location of expansion joints is determined based on a maximum joint opening of 4 inches at the ends of the bridge. When joint openings exceed 4 inches, two options are available:
        i) The preferred option is to provide additional joints at the piers to split the superstructure into segments.
        ii) Provide modular expansion joints at bridge ends only.
    b) For each bridge or bridge segment, provide fixed bearings at a minimum of two piers to provide increased resistance to longitudinal movements.
    c) Provide fixed bearings at all tall pier locations. Tall or flexible piers deflect prior to mobilizing the translational capacity of the bearing.
    d) Bridges with tall or flexible piers that are located on slopes may tend to move toward the downhill end, causing maintenance problems at the uphill end due to a wider joint than anticipated. For these bridges, consider providing fixed bearings at the downhill abutment.

e) A combination of fixed, expansion, guided, and limited expansion bearings can be provided at the piers to accommodate the movements for the bridge or bridge segments.

f) Based on the point of fixity of each segment, the maximum movements can be determined for the design of joints and bearings.

14.2 Expansion Joints [14.5.3.2]

Minnesota bridges with parapet type abutments typically have strip seal expansion joints at the abutments to isolate superstructure movements from the abutments. When the maximum joint openings at the abutments exceed 4 inches additional joints are needed at piers or modular joints are required at the abutments.

Do not use elastomeric compression seal expansion joints.

14.2.1 Thermal Movements [Table 3.4.1-1]

Design joint openings for movements associated with a temperature range of 150°F (-30°F to 120°F). For strip seal expansion joints on typical bridges, use a load factor for movement of 1.0. (Note that this value differs from the LRFD Specification based on past performance of joints in Minnesota.) For strip seal expansion joints on non-typical bridges and for all modular expansion joints, use a load factor for movement of 1.2 per LRFD Article 3.4.1. See BDM Article 3.10.1 for the definition of typical and non-typical bridges.

[5.4.2.2] [6.4.1]

The coefficients of thermal expansion are:

- Concrete: 6.0×10^{-6} per °F
- Steel: 6.5×10^{-6} per °F

14.2.2 Strip Seal Expansion Joints

For movements of up to 4 inches, use strip seal expansion devices. Design joints to have a minimum opening of $^1/_2$ inch between the steel elements (extrusions) of the joint.

[14.5.3.2]

To provide a reasonably smooth roadway surface, the maximum width of expansion openings is limited to 4 inches (measured perpendicular to joint) on roadway bridges. The maximum width for pedestrian bridges is 5 inches. Detail cover plates on sidewalks, medians, and pedestrian bridges to cover the opening.

The standard strip seal device is a Type 4.0, which has a movement capacity of 4 inches. Bridges on a horizontal curve or with a skew over 30° must accommodate "racking" or transverse movements as well. For these situations use a Type 5.0 strip seal (5 inch capacity). Type 5.0 strip seals can also be used on pedestrian bridges.

For skews less than 30°:

- For expansion distance less than 150'-0", dimension opening at 2 inches at all temperatures.
- For expansion distance greater than or equal to 150'-0", dimension opening at $1^1/_2$ inches at 90°F. Also determine and show dimension at 45°F, checking that the opening at -30°F does not exceed 4 inches. If so, reduce accordingly at 45°F and 90°F.

For skews greater than or equal to 30°:

- Dimension opening at $1^1/_2$ inches at 90°F. Also determine and show dimension at 45°F, checking that the opening at -30°F does not exceed 4 inches. If so, reduce accordingly at 45°F and 90°F.

14.2.3 Modular Expansion Joints

When dividing a bridge into segments will not reduce the joint movement to less than 4 inches, use modular expansion joints. Provide a joint setting schedule with modular joints that lists the opening the joint should have at different construction temperatures. Show joint openings for a temperature range from 45°F to 90°F in 15°F increments.

Note that conventional modular joints are one-directional units. Bridges with skews or horizontal curvature may require the use of "swivel" modular joints. These accommodate lateral movement as well as longitudinal movements.

All modular joints will be built as a single unit to a length up to 70 feet without field splice.

14.2.4 Expansion Joint Detailing

When laying out the expansion joint device, kinked bends are not allowed in the strip seal gland in plan view due to constructability issues, however, are allowed in the joint opening as discussed below. Referring to Figure 14.2.4.1, use the following guidance for varying levels of expansion joint skews:

- For skews up to and including 20°, detail the expansion joint opening as straight from edge of deck to edge of deck for barriers and parapets.
- For skews greater than 20° and up to and including 50°:
    - For barriers, detail the expansion joint opening as straight between the top inside edge of the sloped face. Kink the joint opening at top inside edge of the sloped face so it is normal with outside edge of deck.

  - For parapets (with or without a brush curb) and curbs under a metal railing/fence, detail the expansion joint opening as straight to a line offset 9 inches from the front face of parapet/curb (or brush curb). Kink the joint opening at the 9 inch offset point so it is normal to the outside edge of deck.
  - For parapets (with or without a brush curb) adjacent to a raised sidewalk, detail the expansion joint opening as straight between the inside front face of parapet (or brush curb). Kink the joint opening at front face so it is normal to the outside edge of deck.
- For skews greater than 50°, curve the expansion joint ends for barriers and parapets. Use a 2'-0" radius for new bridges. A minimum radius of 1'-6" is allowed on bridge repair projects. Terminate the curved section 6 inches from gutter line.

Use bend-up details for all bridges with curbs, barriers, or parapets. For bridges with skewed joints, verify that the bend-up details do not project out of the front face of concrete.

Use snowplow protection for expansion joint devices (Bridge Details Part II Fig. 5-397.628) when joints are skewed greater than 15° and less than 50°.

On the appropriate expansion joint standard sheet, show the elevation at the top of the extrusion at crown break points and gutter lines. For bridges that require it, dimension the lengths for straight and curved portions of the expansion joint.

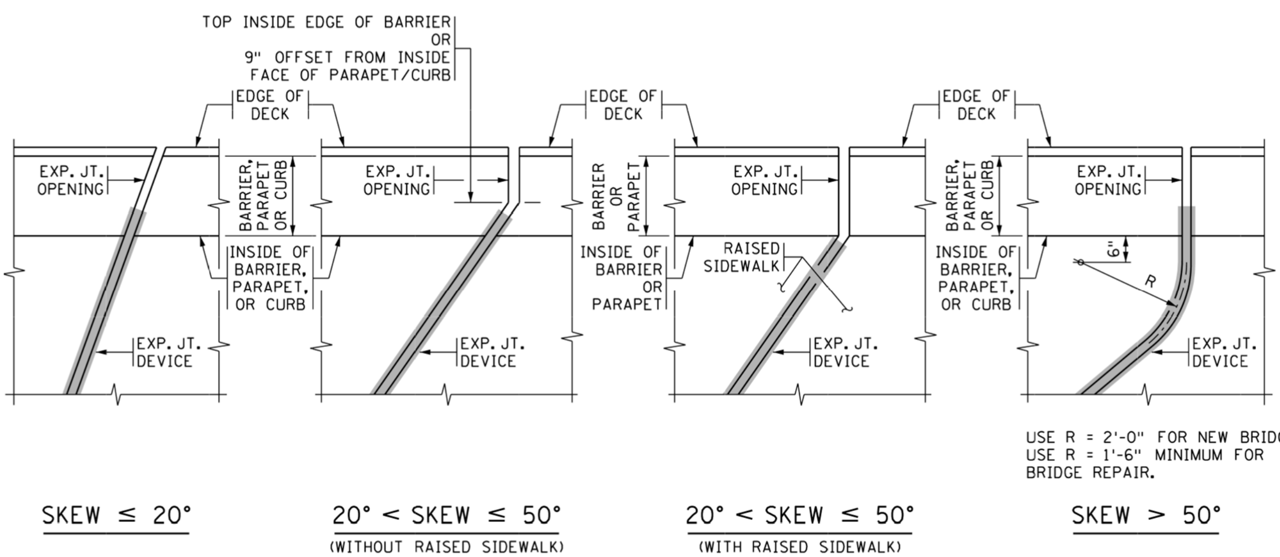


Figure 14.2.4.1
Expansion Joint Details

14.3 Bearings

The purpose of a bridge bearing is to transmit loads from the superstructure to the substructure while facilitating translation and rotation. Four types of bearings are typically used:

1) Expansion Bearing:
   - Transfers vertical load
   - Allows lateral movement in two directions
   - Allows longitudinal rotation
2) Guided Expansion Bearing:
   - Transfers vertical load and lateral load in one direction
   - Allows lateral movement in one direction
   - Allows longitudinal rotation
3) Limited Expansion Bearing:
   - Transfers vertical load and lateral load
   - Allows limited lateral movement in one direction
   - Allows longitudinal rotation
4) Fixed Bearing:
   - Transfers vertical load and lateral load
   - Resists lateral movement
   - Allows longitudinal rotation

All of Minnesota is located in Seismic Zone 1. See Article 3.7 of this manual for Seismic Zone 1 requirements for fixed and expansion bearings.

14.3.1 Loads and Movements

Design bearings for movements associated with a temperature range of 150°F (-30°F to 120°F) and a base construction temperature of 45°F.

Design elastomeric bearings for service loads and without Dynamic Load Allowance (IM).

[14.6.1]

Uplift at bearings is not permitted. Check bearings for uplift using the Strength I load combination with the minimum load factor for dead load.

For post-tensioned concrete bridges, consider the movement due to concrete creep for design of the bearings and expansions joints.

14.3.2 Bearing Details

Identify the type of bearing used at each support location on the superstructure framing plan.

For bearing components, the length is measured parallel to the centerline of the beam and the width is measured perpendicular to the centerline of the beam.

Check the dimensions of the bearing. Check that bearings have adequate clearance to other bearings (pier locations), are consistent with the beam end details (pier and abutment locations), and have adequate clearance to vertical faces of supporting elements. For fixed bearings, provide a minimum of 1 inch clear from the face of the bearing seat to the bearing pad or masonry plate. For expansion bearings, increase this minimum dimension to 3 inches.

Locate bearing anchor rods to permit field drilling of holes and provide 2 inch minimum clearance to reinforcement in bridge seat.

Bearings typically provide a modest amount of lateral restraint. However, designers must consider whether or not additional restraint needs to be provided. Typically, this additional restraint is provided by reinforced concrete guide lugs in the substructure or slotted hole fixed bearing assemblies adjacent to the center beam at expansion piers and abutments for bridges on large skews or curves. A 1 inch clear dimension must be provided between elements for either of these restraint methods. Provide additional restraint for pedestrian bridges.

The service life of bearings is less than the anticipated service life of a bridge. To simplify future maintenance operations and potential replacement, provide adequate clearance for the installation of jacks (at least 6 inches) and also provide a jacking load path. The load path may involve properly designed and detailed diaphragms or a suitable superstructure element.

When the slope of steel beam or plate girder superstructures exceeds 3%, incorporate tapered sole plates into the bearings. When the slope of prestressed beam superstructures exceeds 3%, incorporate tapered bearing plates into the bearings, using standard detail B309 as a guide.

For bridge bearings that have a masonry plate (e.g. - new disc bearings or existing lubricated bronze plate bearings being replaced in kind), include a 1/8 inch 60 durometer plain elastomeric pad between the bridge seat and new masonry plate to provide proper load distribution.

14.3.3 Elastomeric Bearings

Use of elastomeric bearings is preferred over other types of bearings. Fixed and expansion elastomeric bearing types are used most frequently.

MnDOT’s standard elastomeric bearings include:

- Detail B304: Fixed bearing used on bridge repair projects for replacement of existing bearings where there is insufficient height available for a curved plate bearing assembly, consisting of a steel reinforced elastomeric pad with bearing plate and anchor rods, sized to match the height of the existing bearing.

- Detail B305: Fixed bearing used at integral abutments or at piers with continuity diaphragms consisting of a plain elastomeric pad.
- Detail B309: Fixed bearing used at integral abutments or at piers with continuity diaphragms when grades exceed 3%, consisting of a plain elastomeric pad with a tapered bearing plate.
- Detail B310: Fixed bearing for prestressed concrete beams consisting of a plain elastomeric pad with a curved plate to allow rotation, and anchor rods for fixity.
- Detail B311: Expansion bearing for prestressed concrete beams consisting of a steel reinforced elastomeric pad with a curved plate to allow rotation.
- Detail B354: Fixed bearing for steel beams consisting of a plain elastomeric pad with a curved plate to allow rotation, and anchor rods for fixity.
- Detail B355: Expansion bearing for steel beams consisting of a steel reinforced elastomeric pad with a curved plate to allow rotation.

Note that the use of plain elastomeric pads is currently limited per Memo to Designers (2012-01) due to issues of excessive pad deformation. For all fixed curved plate bearing assemblies (Details B310 and B354), plain elastomeric bearing pads are replaced with cotton-duck bearing pads of the same size as required for a plain pad. However, the guidance regarding plain elastomeric pads within this article has been retained until a final policy decision is made regarding their use.

14.3.3.1 Design

Use the tables found in Article 14.7 of this manual whenever possible for consistency and economy among bearing designs.

Design elastomeric bearings using Method A of the AASHTO LRFD Specifications.

[Table 14.7.6.2-1] Design using an elastomer hardness of 60 durometers. The minimum shear modulus (G) for this material is 130 psi. The maximum shear modulus is 200 psi.

Except for special designs, use steel with a yield strength F_y equal to 36 ksi for all bearing assembly plates.

For MnDOT bridges with curved plate bearings, rotations need not be considered in the design.

For maximum compressive stress checks, use the minimum shear modulus value.

Holes are not permitted in elastomeric bearings.

14.3.3.1.1 Size and Stability

Although not an AASHTO requirement, MnDOT has historically used the following limits for the shape factor, S, for plain pads and internal laminates of steel reinforced pads with good success:

$$5.0 \le S \le 10.0$$

For fixed bearings, use $^1/_2$ inch or $^3/_4$ inch thickness plain pads. For expansion bearings, use $^3/_8$ inch, $^1/_2$ inch, or $^3/_4$ inch thickness internal laminates with $^1/_8$ inch thick steel reinforcing plates and $^1/_4$ thick cover layers.

Round dimensions for elastomeric bearings to the nearest 2 inch increment. For "RB", "M", and "MN" series prestressed beams, the minimum length (A) is 12 inches and the minimum width (B) is 24 inches. For "MW" series prestressed beams, the minimum length (A) is 16 inches and the minimum width (B) is 36 inches. For steel beams, the minimum length (A) is 8 inches. The width (B) shall not be less than the bottom flange width and not more than 2 inches greater than the bottom flange width for steel beams.

Based on the past performance of elastomeric bearings, MnDOT places a limit on the plan aspect ratio of a bearing. The length (A) is limited by the following equation:

$$B \le 2.5 \cdot A$$

[14.7.6.3.6]

Additionally, the total elastomer thickness for the bearing (h_{rt}) must be no more than $^1/_3$ of the bearing pad length and width:

$$h_{rt} \le \frac{A}{3} \text{ and } \frac{B}{3}$$

14.3.3.2 Fixed Bearings
[14.7.6.3.2]

Design fixed elastomeric bearings for a maximum compressive stress of 0.880 ksi. This includes a 10% increase for fixity.

Provide transverse fixity for $^2/_3$ of beams at fixed piers or fixed abutments for widths along skew greater than 70'-0".

14.3.3.3 Expansion Bearings
[14.7.6.3.2]

Design expansion elastomeric bearings to be steel-reinforced, with a maximum compressive stress equal to the lesser of 1.25GS or 1.25 ksi.

[14.7.6.3.4]
[Table 3.4.1-1]

In order to accommodate shear deformation in the pad due to thermal movement, the total height or thickness of elastomer (h_{rt}) must be greater than twice the maximum design movement. The LRFD Specifications list a load factor of 1.2 to be used for thermal movement calculations. However, based on past performance of bearings, use a load factor of 1.3 with half the design temperature range (75°F) when computing movement Δ_s for the shear deformation check.

Timely delivery of elastomeric bearings has been an issue in the past. In an effort to improve availability and encourage fabricators/contractors to stockpile pads, the number of laminates has been standardized into two groupings for "RB", "M", and "MN" series prestressed beam expansion elastomeric bearings. Where possible:

- For design movements $\Delta_s \leq 1.00''$, use a 12" x 24" pad with 3 – ½" thick laminates
- For pads where $1.00'' <$ design movement $\Delta_s \leq 1.75''$, use a 12" x 24" pad with 6 – ½" laminates

14.3.3.3.1 Minimum Compressive Load
[C14.8.3.1]

LRFD Article C14.8.3.1 states that bearings should be anchored securely to the support to prevent their moving out of place. It further states that elastomeric bearings may be left without anchorage provided adequate friction is available and that a design coefficient of friction equal to 0.20 may be assumed between elastomer and concrete or steel. The minimum horizontal resistance to slippage of the bearing is:

$$H_{bres} = 0.20 \cdot P_{min}$$

The factored horizontal shear force H_{bu} generated in the bearing due to temperature movement is:

[14.6.3.1]

$$H_{bu} = G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

Equating H_{bres} and H_{bu} and solving for the minimum compressive load, P_{min} results in:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

For the minimum compressive load check, use the maximum shear modulus value (0.200 ksi). For calculation of Δ_u, LRFD Article 3.4.1 specifies a load factor of 1.2. However, based on past performance of bearings, use a load factor of 1.0 with half the design temperature range (75°F) to calculate Δ_u.

If the check is not satisfied, revise the number and/or thickness of the laminates as needed. If the requirement still cannot be met, the standard curved plate expansion bearing assemblies (B311 and B355) contain a 3/8" x 3/8" bar welded to the bearing plate. This can be considered as a mechanism that secures the pads.

14.3.4 Disc Bearings

Use disc bearings where the loads are too high or the movements and rotations are too large to be readily accommodated with elastomeric bearings.

To reduce the possibility of generating large lateral forces in wide bridges supported on disc bearings, do not use guided or fixed bearings for beam lines outside of the center 45 feet of the bridge (distance measured along the substructure).

Due to a variety of preferences among disc bearing fabricators, explicit bearing details are not provided in the plans. Instead, provide a schematic of the bearings and all applicable design loads and movements in the plans. Using the provided data, the fabricator will determine the size of all of the bearing components, from the masonry plate to the sole plate.

Contact bearing fabricators to determine estimated bearing assembly heights for inclusion in the bridge plan. Also provide the appropriate standard plan note (see Appendix 2-C.J) in the bridge plan regarding the estimated bearing assembly heights and adjustments to seat elevations.

Fixed disc bearings allow for limited rotation, but no movement.

Guided expansion disc bearings allow for free movement in one direction and provide rotational capacity. However, movement perpendicular to the free movement direction is restrained. For curved bridges, assume the free movement direction to be along a chord connecting the ends of the beam. Guide bars must resist a minimum of 15% of the vertical service limit state load applied to the bearing.

Expansion disc bearings provide for rotation and unguided movement in all horizontal directions.

For computation of movement for design of disc bearings, use a load factor of 1.2.

For disc bearings where blockouts are incorporated into the plans, calculate X and Y coordinates for individual anchor rods and center of anchor rod groups in advance of letting and include them with the final calculations.

14.3.5 Other Types of Bearings

Pot Bearings
Do not use pot bearings on roadway bridges.

Steel Bearings
This type of bearing does not contain elastomeric components to accommodate horizontal movement. Rather, horizontal movement takes place at the interface of a machined masonry plate and a lubricated bronze plate. Bridge Details Part I B351, B352, and B353 detail fixed, expansion, and guided expansion steel bearings respectively. They have all been archived, but can be retrieved if necessary for a repair plan. Note that these bearings are used for bridge repair projects only and are not for new construction.

Modify the standard bearings as necessary to accommodate unusually wide flanges or to provide movement capacities greater than those permitted with the standard details.

Check the clearances on the guide bars for curved bridges.

To reduce the possibility of generating large lateral forces in wide bridges supported on steel bearings, do not use guided or fixed bearings for beam lines outside of the center 45'-0" of the bridge (distance measured along the substructure).

Bearings for Railroad Bridges
Due to the extremely large loads associated with railroad bridges, spherical bearings, rocker bearings, disc bearings, or pot bearings are normally required. Rocker bearings may be considered for other applications where there is a combination of large load and large movement.

14.4 Curved Plate Design

Width
For prestressed concrete beams, set the width (H) equal to the bearing pad width (B) plus 2 inches. The width may change slightly (2 inches to 4 inches) for special designs. For steel beams, set the width equal to the bearing pad width (B).

Thickness

Use the LRFD design method for determining the curved plate thickness. Although AASHTO LRFD allows the nominal flexural resistance of a section to be taken as the plastic moment of a section, MnDOT limits the nominal flexural resistance to the yield moment. For steel elements in flexure use

[6.5.4.2] a resistance factor, ϕ_r equal to 1.0.

The all-around weld, together with the friction between plates, causes the curved plate and bearing plate to act compositely. Therefore, the thickness for design can be considered to include the curved plate thickness plus the bearing plate thickness. The minimum thickness for curved plates is $1^1/_4$ inches. When greater thickness is required, increase plate thickness in $^1/_4$ inch increments.

Length

The minimum length (G) for the curved plate is $4^1/_2$ inches. The next permitted length is 6 inches, after which the length may be increased by increments of 2 inches up to a maximum of 12 inches. If, when designing the bearing plate, the required bearing plate thickness exceeds 2 inches, increase the length of the curved plate to reduce the length of the cantilever for the bearing plate design. Increase the curved plate length until the required bearing plate thickness alone and the required plate thickness for the curved plate based on composite design are approximately equal.

Radius

The radius of curved plates is to be no less than 16 inches. Check contact stresses to make sure that an adequate radius is provided. Based on past satisfactory performance of curved plate bearing assemblies, use LRFD

[14.7.1.4] Equations C14.7.1.4-1 and C14.7.1.4-2 for determination of curved plate radius. If the resulting radius exceeds 24 inches, a special design must be completed using LRFD Equation 14.7.1.4-1 and steel with a yield strength F_y equal to 50 ksi.

14.5 Bearing Plate Design

Width

For prestressed concrete beams, set the width (E) equal to the curved plate width (H) plus 1 inch for expansion bearings. For fixed bearings, set the width (E) equal to the beam bottom flange width plus 8 inches. For steel beams, set the width (E) equal to the curved plate width (B) plus 2 inches for expansion bearings and plus 10 inches for fixed bearings.

Length

Set the length of the bearing plate (C) 2 inches larger than the bearing pad length (A).

Thickness

Use the LRFD design method for determining the bearing plate thickness. Although AASHTO LRFD allows the nominal flexural resistance of a section to be taken as the plastic moment of a section, MnDOT limits the nominal flexural resistance to the yield moment. For steel elements in flexure use

[6.5.4.2] a resistance factor, ϕ_r, equal to 1.0.

The minimum thickness for bearing plates is $1^1/_2$ inches. When greater thickness is required, increase plate thickness in $^1/_4$ inch increments.

14.6 Sole Plate Design (Steel Beams)

Width

Set the width of the sole plate 2 inches larger than the curved plate width (B). The width cannot be equal to the beam flange width because of the fillet weld used to attach the sole plate to the flange. Increase the sole plate width by 1 inch if this occurs.

Length

The minimum length is 6 inches. Also, the length shall not be less than the curved plate length (G).

Thickness

The minimum sole plate thickness is $1^1/_4$ inches. When greater thickness is required, increase plate thickness in $^1/_8$ inch increments.

When the bearing pad width exceeds the bottom flange width, the sole plate must be designed as a cantilever to resist the load from the pad that extends outside the flange. Use the LRFD design method. Although AASHTO LRFD allows the nominal flexural resistance of a section to be taken as the plastic moment of a section, MnDOT limits the nominal flexural resistance to the yield moment. For steel elements in flexure use a

[6.5.4.2] resistance factor, ϕ_r, equal to 1.0.

14.7 Tables

The following tables contain standard curved plate bearing designs for prestressed concrete and steel beam superstructures based on the guidance given in this manual.

Table 14.7.1 Fixed Curved Plate Bearing Assembly for Prestressed Concrete Beams (B310)

Table 14.7.2	Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311)
Table 14.7.3	Elastomeric Bearing Pad Thickness for Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311)
Table 14.7.4	Fixed Curved Plate Bearing Assembly for Steel Beams (B354)
Table 14.7.5	Expansion Curved Plate Bearing Assembly for Steel Beams (B355)
Table 14.7.6	Elastomeric Bearing Pad Thickness for Expansion Curved Plate Bearing Assembly for Steel Beams (B355)

The curved plate thicknesses, the bearing plate thicknesses, and the steel beam sole plate thicknesses given in the tables were designed for the LRFD Strength I limit state by applying the live load factor of 1.75 to the maximum service load determined for the elastomeric pad design. This ensures a conservative design for the plates without knowing the exact mix of dead load and live load.

Use the tables whenever possible to increase consistency and economy among bearing designs. When calculated loads and/or movements fall outside the limits given in the table, two options are available to designers:

1) Complete a special elastomeric bearing design. For this case, use LRFD Equation 14.7.1.4-1 for determination of curved plate radius. Also, use steel with a yield strength equal to 50 ksi for the curved plate. Modify the B-Detail by specifying that the curved plate shall comply with MnDOT Spec. 3310. ***[14.7.1.4]***
2) Use a disc bearing.

Table 14.7.1
Fixed Curved Plate Bearing Assembly for Prestressed Concrete Beams (B310)

Beam Series	Max Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thickness (in)	Shape Factor	Bearing Plate Size (in) ②			Curved Plate Size (in) ②			Min Radius (in)	Anchor Rod Factored Shear Resistance (kips) ③
		A	B			C	E	F	G	H	J		
RB, M, and MN	253	12	24	$\frac{1}{2}$	8.0	14	①	$1\frac{1}{2}$	$4\frac{1}{2}$	26	$1\frac{1}{4}$	16	14.4
	295	14	↓	↓	8.8	16	↓	↓	6	↓	↓	↓	↓
	337	16	↓	↓	9.6	18	↓	2	↓	↓	↓	↓	12.3
	380	18	↓	$\frac{3}{4}$	6.9	20	↓	↓	8	↓	↓	↓	10.7
	422	20	↓	↓	7.3	22	↓	$2\frac{1}{4}$	↓	↓	↓	20	10.0
MH	316	12	30	$\frac{1}{2}$	8.6	14	47	$1\frac{1}{2}$	$4\frac{1}{2}$	32	$1\frac{1}{4}$	16	14.4
	369	14	↓	↓	9.6	16	↓	↓	6	↓	↓	↓	↓
MW	270	16	36	$\frac{1}{2}$	11.1	18	47	$1\frac{1}{2}$	$4\frac{1}{2}$	38	$1\frac{1}{4}$	16	14.4
	350	↓	↓	↓	↓	↓	↓	↓	6	↓	↓	↓	↓
	506	↓	↓	↓	↓	↓	↓	2	↓	↓	↓	↓	12.3
	570	18	↓	↓	12.0	20	↓	↓	8	↓	↓	↓	↓

Single 1½ inch diameter pintle has a factored shear resistance of 50.3 kips. Two included in typical B310 bearing assembly.

① 34" for all "RB" and "M" series beams.
38" for all "MN" series beams.

② Plates are conservatively designed for $1.75 \cdot (\text{Max Service DL+LL})$.

③ Based on single 1½ inch diameter anchor rod. Two included in typical B310 bearing assembly.

Table 14.7.2
Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311)

Beam Series	Max Service DL+LL (Kips)	Bearing Pad Size (in)		Laminate Thickness (in)	Max Number of Laminates ①	Shape Factor	Bearing Plate Size (in) ②			Curved Plate Size (in) ②			Min Radius (in)
		A	B				C	E	F	G	H	J	
RB, M, and MN	300	12	24	$\frac{1}{2}$	7	8.0	14	27	$1\frac{1}{2}$	$4\frac{1}{2}$	26	$1\frac{1}{4}$	16
	360	↓	↓	↓	↓	↓	14	↓	$1\frac{3}{4}$	↓	↓	↓	↓
	420	14	↓	↓	8	8.8	16	↓	↓	6	↓	↓	19
MH	395	12	30	$\frac{1}{2}$	7	8.6	14	33	$1\frac{1}{2}$	$4\frac{1}{2}$	32	$1\frac{1}{4}$	16
MW	270	16	36	$\frac{3}{4}$	6	7.4	18	39	$1\frac{1}{2}$	$4\frac{1}{2}$	38	$1\frac{1}{4}$	16
	350	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
	480	↓	↓	↓	↓	↓	↓	↓	$1\frac{3}{4}$	6	↓	↓	↓
	630	↓	↓	↓	↓	↓	↓	↓	2	↓	↓	↓	↓

Single 1½ inch diameter pintle has a factored shear resistance of 50.3 kips. Two included in typical B311 bearing assembly.

① See Table 14.7.3 for determination of required number of laminates.

② Plates are conservatively designed for $1.75 \cdot (\text{Max Service DL+LL})$.

Table 14.7.3
Elastomeric Bearing Pad Thickness for Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311) ①②

Interior Laminate Thickness (in)	D (in) ③	Number of Laminates	Total Elastomer Thickness, h_{rt} (in) ③	Maximum Movement Δ_s (in) ④
1/2"	1 1/4	1	1	1/2
	1 7/8	2	1 1/2	3/4
	2 1/2	3 ⑤	2	1
	3 1/8	4	2 1/2	1 1/4
	3 3/4	5	3	1 1/2
	4 3/8	6 ⑤	3 1/2	1 3/4
	5	7	4	2
	5 5/8	8	4 1/2	2 1/4
3/4"	1 1/2	1	1 1/4	7/8
	2 3/8	2	2	1
	3 1/4	3	2 3/4	1 3/8
	4 1/8	4	3 1/2	1 3/4
	5	5	4 1/4	2 1/8
	5 7/8	6	5	2 1/2

① Table is based on requirements of *AASHTO LRFD Bridge Design Specs*. Art. 14.7.6.3.4:

$$h_{rt} \geq 2\Delta_s$$

Engineer must also check that the minimum compressive load requirement (discussed in Article 14.3.3.3.1) is satisfied. Specifically:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

where P_{min} is the minimum factored load ($0.9 \cdot DC + 1.75 \cdot LL_{min}$), G is equal to the maximum shear modulus value (0.200 ksi), A_{pad} is the plan area of the bearing pad, and Δ_u is the movement of the bearing pad from the undeformed state using a 75°F temperature.

② Engineer must also check the elastomeric bearing pad for compression deflection based on the requirements from *AASHTO LRFD Bridge Design Specifications* Articles 14.7.6.3.3 and 14.7.5.3.6.

③ Pad thickness D includes h_{rt} and 1/8" steel reinforcement plates. Total elastomer thickness h_{rt} includes interior laminates plus 1/4" cover layers.

④ Maximum movement Δ_s is the movement of the bearing pad from the undeformed state to the point of maximum deformation. Use a 75°F temperature change with a 1.3 load factor for calculation of maximum movement.

⑤ For "RB", "M", and "MN" series prestressed beam expansion elastomeric bearings, the number of laminates has been standardized for the movements that are most often encountered.

- If $\Delta_s \leq 1.00''$, use 3 – ½" laminates.
- If $1.00'' < \Delta_s \leq 1.75''$, use 6 – ½" laminates.

Table 14.7.4 – Fixed Curved Plate Bearing Assembly for Steel Beams (B354)

Beam Flange Min. Width (in)	Beam Flange Max. Width (in)	Max. Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thick. (in)	Shape Factor	Bearing Plate Size (in) ①			Curved Plate Size (in) ①			Min. Radius (in)	Sole Plate Size (in)			Anchor Rod Factored Shear Resistance (kips) ②
			A	B			C	E	F	G	B	H		Length	Width	Thick.	
12	14	81	8	14	$\frac{1}{2}$	5.1	10	24	$1\frac{1}{2}$	$4\frac{1}{2}$	14	$1\frac{1}{4}$	16	6	16	$1\frac{1}{4}$	14.4
↓	↓	116	10	↓	↓	5.8	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	147	12	↓	↓	6.5	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	172	14	↓	↓	7.0	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
14	16	97	8	16	$\frac{1}{2}$	5.3	10	26	$1\frac{1}{2}$	$4\frac{1}{2}$	16	$1\frac{1}{4}$	16	6	18	$1\frac{1}{4}$	14.4
↓	↓	140	10	↓	↓	6.2	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	168	12	↓	↓	6.9	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	197	14	↓	↓	7.5	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
↓	↓	225	16	↓	↓	8.0	18	↓	↓	6	↓	↓	17	↓	↓	↓	↓
16	18	113	8	18	$\frac{1}{2}$	5.5	10	28	$1\frac{1}{2}$	$4\frac{1}{2}$	18	$1\frac{1}{4}$	16	6	20	$1\frac{1}{4}$	14.4
↓	↓	158	10	↓	↓	6.4	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	190	12	↓	↓	7.2	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	221	14	↓	↓	7.9	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
↓	↓	253	16	↓	↓	8.5	18	↓	2	6	↓	↓	↓	↓	↓	↓	12.3
↓	↓	285	18	↓	↓	9.0	20	↓	↓	8	↓	↓	19	8	↓	↓	↓
18	20	130	8	20	$\frac{1}{2}$	5.7	10	30	$1\frac{1}{2}$	$4\frac{1}{2}$	20	$1\frac{1}{4}$	16	6	22	$1\frac{1}{4}$	14.4
↓	↓	176	10	↓	↓	6.7	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	211	12	↓	↓	7.5	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	246	14	↓	↓	8.2	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
↓	↓	281	16	↓	↓	8.9	18	↓	2	6	↓	↓	↓	↓	↓	↓	12.3
↓	↓	316	18	↓	↓	9.5	20	↓	↓	8	↓	↓	18	8	↓	↓	↓
↓	↓	352	20	↓	↓	10.0	22	↓	$2\frac{1}{4}$	↓	↓	↓	22	↓	↓	↓	11.5

Table 14.7.4 *(Cont.)* **– Fixed Curved Plate Bearing Assembly for Steel Beams (B354)**

Beam Flange Min. Width (in)	Beam Flange Max. Width (in)	Max. Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thick. (in)	Shape Factor	Bearing Plate Size (in) ①			Curved Plate Size (in) ①			Min. Radius (in)	Sole Plate Size (in)			Anchor Rod Factored Shear Resistance (kips) ②
			A	B			C	E	F	G	B	H		Length	Width	Thick.	
20	22	193	10	22	$\frac{1}{2}$	6.9	12	32	$1\frac{1}{2}$	$4\frac{1}{2}$	22	$1\frac{1}{4}$	16	6	24	$1\frac{1}{4}$	14.4
↓	↓	232	12	↓	↓	7.8	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	271	14	↓	↓	8.6	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
↓	↓	309	16	↓	↓	9.3	18	↓	2	6	↓	↓	↓	↓	↓	↓	12.3
↓	↓	348	18	↓	↓	9.9	20	↓	↓	8	↓	↓	17	8	↓	↓	↓
↓	↓	387	20	↓	$\frac{3}{4}$	7.0	22	↓	$2\frac{1}{4}$	↓	↓	↓	21	↓	↓	↓	10.0
↓	↓	426	22	↓	↓	7.3	24	↓	$2\frac{1}{2}$	↓	↓	↓	25	↓	↓	↓	9.4
22	24	211	10	24	$\frac{1}{2}$	7.1	12	34	$1\frac{1}{2}$	$4\frac{1}{2}$	24	$1\frac{1}{4}$	16	6	26	$1\frac{1}{4}$	14.4
↓	↓	253	12	↓	↓	8.0	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	295	14	↓	↓	8.8	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
↓	↓	337	16	↓	↓	9.6	18	↓	2	6	↓	↓	↓	↓	↓	↓	12.3
↓	↓	380	18	↓	$\frac{3}{4}$	6.9	20	↓	↓	8	↓	↓	↓	8	↓	↓	10.7
↓	↓	422	20	↓	↓	7.3	22	↓	$2\frac{1}{4}$	↓	↓	↓	20	↓	↓	↓	10.0
↓	↓	464	22	↓	↓	7.7	24	↓	$2\frac{1}{2}$	↓	↓	↓	24	↓	↓	↓	9.4
24	26	274	12	26	$\frac{1}{2}$	8.2	14	36	$1\frac{1}{2}$	$4\frac{1}{2}$	26	$1\frac{1}{4}$	16	6	28	$1\frac{1}{4}$	14.4
↓	↓	320	14	↓	↓	9.1	16	↓	$1\frac{3}{4}$	↓	↓	↓	↓	↓	↓	↓	13.3
↓	↓	366	16	↓	↓	9.9	18	↓	2	6	↓	↓	↓	↓	↓	↓	12.3
↓	↓	411	18	↓	$\frac{3}{4}$	7.1	20	↓	↓	8	↓	↓	↓	8	↓	↓	10.7
↓	↓	457	20	↓	↓	7.5	22	↓	$2\frac{1}{4}$	↓	↓	↓	19	↓	↓	↓	10.0
↓	↓	503	22	↓	↓	7.9	24	↓	$2\frac{1}{2}$	↓	↓	↓	23	↓	↓	↓	9.4

Single 1½ inch diameter pintle has a factored shear resistance of 50.3 kips. Two included in typical B354 bearing assembly.
① Plates are conservatively designed for 1.75 · Max Service DL+ LL
② Based on single 1½ inch diameter anchor rod. Two included in typical B354 bearing assembly.

Table 14.7.5 – Expansion Curved Plate Bearing Assembly for Steel Beams (B355)

Beam Flange Min. Width (in)	Beam Flange Max. Width (in)	Max. Service DL+LL (kips)	Bearing Pad Size (in)		Laminate Thick. (in)	Max. Number of Lamin-ates ①	Shape Factor	Bearing Plate Size (in) ②			Curved Plate Size (in) ②			Min. Radius (in)	Sole Plate Size (in)		
			A	B				C	E	F	G	B	H		Length	Width	Thick.
12	14	123	8	14	$^3/_8$	5	6.8	10	16	$1^1/_2$	$4^1/_2$	14	$1^1/_4$	16	6	16	$1^1/_4$
↓	↓	175	10	↓	↓	7	7.8	12	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	210	12	↓	↓	9	8.6	14	↓	$1^3/_4$	↓	↓	↓	21	↓	↓	↓
14	16	147	8	16	$^3/_8$	5	7.1	10	18	$1^1/_2$	$4^1/_2$	16	$1^1/_4$	16	6	18	$1^1/_4$
↓	↓	200	10	↓	↓	7	8.2	12	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	240	12	↓	↓	9	9.1	14	↓	$1^3/_4$	↓	↓	↓	19	↓	↓	↓
16	18	172	8	18	$^3/_8$	5	7.4	10	20	$1^1/_2$	$4^1/_2$	18	$1^1/_4$	16	6	20	$1^1/_4$
↓	↓	225	10	↓	↓	7	8.6	12	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	270	12	↓	↓	9	9.6	14	↓	$1^3/_4$	↓	↓	↓	17	↓	↓	↓
↓	↓	315	14	↓	$^1/_2$	8	7.9	16	↓	↓	6	↓	↓	23	↓	↓	↓
18	20	198	8	20	$^3/_8$	5	7.6	10	22	$1^1/_2$	$4^1/_2$	20	$1^1/_4$	16	6	22	$1^1/_4$
↓	↓	250	10	↓	↓	7	8.9	12	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	300	12	↓	↓	9	10.0	14	↓	$1^3/_4$	↓	↓	↓	↓	↓	↓	↓
↓	↓	350	14	↓	$^1/_2$	8	8.2	16	↓	↓	6	↓	↓	22	↓	↓	↓
20	22	275	10	22	$^3/_8$	7	9.2	12	24	$1^1/_2$	$4^1/_2$	22	$1^1/_4$	16	6	24	$1^1/_4$
↓	↓	330	12	↓	$^1/_2$	↓	7.8	14	↓	$1^3/_4$	↓	↓	↓	↓	↓	↓	↓
↓	↓	385	14	↓	↓	8	8.6	16	↓	↓	6	↓	↓	21	↓	↓	↓
22	24	300	10	24	$^3/_8$	7	9.4	12	26	$1^1/_2$	$4^1/_2$	24	$1^1/_4$	16	6	26	$1^1/_4$
↓	↓	360	12	↓	$^1/_2$	↓	8.0	14	↓	$1^3/_4$	↓	↓	↓	↓	↓	↓	↓
↓	↓	420	14	↓	↓	8	8.8	16	↓	↓	6	↓	↓	20	↓	↓	↓
24	26	390	12	26	$^1/_2$	7	8.2	14	28	$1^3/_4$	$4^1/_2$	26	$1^1/_4$	16	6	28	$1^1/_4$
↓	↓	455	14	↓	↓	8	9.1	16	↓	↓	6	↓	↓	19	↓	↓	↓

Single 1½ inch diameter pintle has a factored shear resistance of 50.3 kips. Two included in typical B355 bearing assembly.

① See Table 14.7.6 for determination of required number of laminates.

② Plates are conservatively designed for 1.75 · Max Service DL+ LL

Table 14.7.6
Elastomeric Bearing Pad Thickness for Expansion Curved Plate Bearing Assembly for Steel Beams (B355)①②

Interior Laminate Thickness (in)	D (in) ③	Number of Laminates ④	Total Elastomer Thickness, h_{rt} (in) ③	Maximum Movement (in) ⑤
	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{16}$
	$1\frac{5}{8}$	2	$1\frac{1}{4}$	$\frac{5}{8}$
	$2\frac{1}{8}$	3	$1\frac{5}{8}$	$\frac{13}{16}$
	$2\frac{5}{8}$	4	2	1
	$3\frac{1}{8}$	5	$2\frac{3}{8}$	$1\frac{3}{16}$
$\frac{3}{8}$"	$3\frac{5}{8}$	6	$2\frac{3}{4}$	$1\frac{3}{8}$
	$4\frac{1}{8}$	7	$3\frac{1}{8}$	$1\frac{9}{16}$
	$4\frac{5}{8}$	8	$3\frac{1}{2}$	$1\frac{3}{4}$
	$5\frac{1}{8}$	9	$3\frac{7}{8}$	$1\frac{15}{16}$
	$5\frac{5}{8}$	10	$4\frac{1}{4}$	$2\frac{1}{8}$
	$6\frac{1}{8}$	11	$4\frac{5}{8}$	$2\frac{5}{16}$
	$1\frac{1}{4}$	1	1	$\frac{1}{2}$
	$1\frac{7}{8}$	2	$1\frac{1}{2}$	$\frac{3}{4}$
	$2\frac{1}{2}$	3	2	1
	$3\frac{1}{8}$	4	$2\frac{1}{2}$	$1\frac{1}{4}$
	$3\frac{3}{4}$	5	3	$1\frac{1}{2}$
$\frac{1}{2}$"	$4\frac{3}{8}$	6	$3\frac{1}{2}$	$1\frac{3}{4}$
	5	7	4	2
	$5\frac{5}{8}$	8	$4\frac{1}{2}$	$2\frac{1}{4}$
	$6\frac{1}{4}$	9	5	$2\frac{1}{2}$
	$6\frac{7}{8}$	10	$5\frac{1}{2}$	$2\frac{3}{4}$
	$7\frac{1}{2}$	11	6	3

① Table is based on requirements of AASHTO LRFD Article 14.7.6.3.4: $h_{rt} \geq 2\Delta_3$. Engineer must also check that the minimum compressive load requirement (discussed in Article 14.3.3.3.1 of this manual) is satisfied:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

where P_{min} is the minimum factored load ($0.9 \cdot DC + 1.75 \cdot LL_{min}$), G is equal to the maximum shear modulus value (0.200 ksi), A_{pad} is the plan area of the bearing pad, an Δ_u is the movement of the bearing pad from the undeformed state using a 75°F temperature change with a 1.0 load factor.

② Engineer must also check the elastomeric bearing pad for compression deflection based on the requirements from AASHTO LRFD Bridge Design Specifications Articles 14.7.6.3.3 and 14.7.5.3.6.

③ Pad thickness D includes h_{rt} and $\frac{1}{8}$" steel reinforcement plates. Total elastomer thickness h_{rt} Includes interior laminates plus $\frac{1}{4}$" cover layers.

④ Engineer must also check that $S_i^2/n < 22$ to meet requirements of AASHTO LRFD Article 14.7.6.3.4.

⑤ Maximum movement Δ_s is the movement of the bearing pad from the undeformed state to the point of maximum deformation. Use a 75°F temperature change with a 1.3 load factor for calculation of maximum movement.

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14.8 Design Examples

Two design examples follow. The first is a fixed elastomeric bearing. The second is an expansion elastomeric bearing.

14.8.1 Fixed Elastomeric Bearing Design Example [14.7.6]

Note that the use of plain elastomeric pads is currently limited per Memo to Designers (2012-01) due to issues of excessive pad deformation. For all fixed curved plate bearing assemblies (Details B310 and B354), plain elastomeric bearing pads are replaced with cotton-duck bearing pads of the same size as required for a plain pad. However, the following design example has been retained until a final policy decision is made regarding their use.

This example illustrates the design of a fixed curved plate elastomeric bearing for a prestressed concrete beam bridge. The bearing is based on Bridge Details Part I B310. The elastomeric bearing pad is designed using Method A (LRFD Article 14.7.6). Figure 14.8.1.1 shows the bearing components.

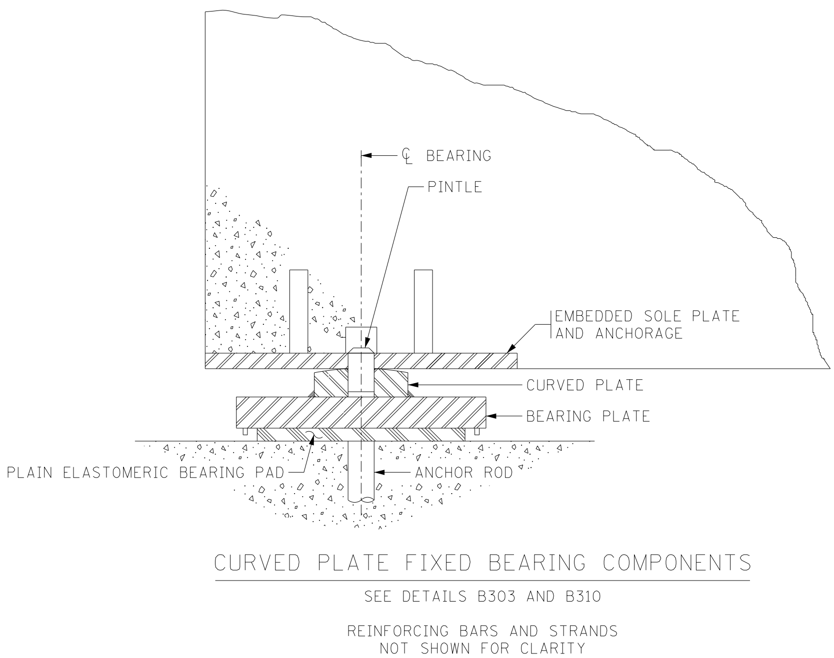


Figure 14.8.1.1

After the maximum reaction is calculated, the bearing design should be selected from the standard tables found in BDM Article 14.7. If a standard design will not work due to unusual loads or geometric constraints, a custom design will be required.

This example will outline the procedure to custom design a fixed elastomeric bearing. First, design the elastomeric pad. Next determine the steel plate requirements for the rest of the bearing assembly.

A. Design Elastomeric Bearing Pad [14.7.6]

The prestressed beam for this example is an MN63, which has a bottom flange width equal to 30 inches. The design loads are given as follows:

Dead Load = P_{dl} = 156 kips

Maximum Live Load = 97.6 kips (Does not include IM)

Minimum Live Load = P_{llmin} = 0 kips

Combining the loads results in the following:

Maximum service limit state load $P_s = P_{dl} + P_{llmax}$
$= 156 + 97.6 = 253.6$ kips

Minimum strength limit state load $P_{umin} = 0.9 \cdot P_{dl} + 1.75 \cdot P_{llmin}$
$= 0.9 \cdot 156 + 1.75 \cdot 0$
$= 140.4$ kips
Therefore, there is no uplift.

[Table 14.7.6.2-1]

MnDOT Spec. 3741 specifies an elastomeric pad with a hardness of 60 durometers. Per LRFD Table 14.7.6.2-1, the shear modulus G for design ranges from 0.130 to 0.200 ksi.

The minimum bearing pad dimensions for a prestressed beam are:
Length A = 12 in
Width B = 24 in

Assuming a plain pad thickness D = 0.50 in,

[14.7.5.1]

$$\text{Shape factor } S = \frac{A \cdot B}{2 \cdot D \cdot (A + B)} = \frac{12 \cdot 24}{2 \cdot 0.5 \cdot (12 + 24)} = 8.0$$

[14.7.6.3.2]

The allowable compressive stress σ_{sall} for plain pads is the smaller of:

$\sigma_{sall} = 1.00 \cdot G \cdot S$
$= 1.00 \cdot 0.130 \cdot 8.0$
$= 1.04$ ksi

or $\sigma_{sall} = 0.80$ ksi. < <u>GOVERNS</u>

The allowable is increased by 10% for a fixed bearing because shear deformation is prevented.

$\sigma_{sallfixed} = 1.10 \cdot 0.80 = 0.88$ ksi

Then the maximum service limit state stress is:

$$\text{Actual}\ \sigma_s = \frac{P_s}{A \cdot B} = \frac{253.6}{12 \cdot 24} = 0.88\ \text{ksi}$$

There are two geometric checks on the bearing pad to ensure that it has good proportions. First, in plan, the length of the long side can be no more the 2.5 times the length of the short side. Second, the height of the elastomeric portion can be no more than $^1/_3$ the length of the short side of the pad.

$$2.5 \cdot A = 2.5 \cdot 12 = 30\ \text{in} \geq 24\ \text{in} \qquad \text{OK}$$

[14.7.6.3.6]

$$\frac{A}{3} = \frac{12}{3} = 4\ \text{in} > 0.50\ \text{in} = h_{rt}$$

Therefore, use a 12″ x 24″ x ½″ plain pad.

B. Curved Plate Design

Set the curved plate width 2 inches wider than the bearing pad.

$$H = B + 2 = 24 + 2 = 26\ \text{in}$$

The all-around weld, together with the friction between plates, causes the curved plate and bearing plate to act compositely. Therefore, the thickness for design can be considered to include the curved plate thickness plus the bearing plate thickness.

Begin by checking the thickness for a curved composite plate with a length of 4.5 inches. If, when designing the bearing plate, the required bearing plate thickness exceeds 2 inches, increase the length of the curved plate to reduce the length of the cantilever for the bearing plate design. Increase the curved plate length until the required bearing plate thickness alone and the required plate thickness for the curved plate based on composite design are approximately equal.

Curved Plate Length = G = 4.5 in.

The radius of the contact surface is the first parameter to determine for the curved plate. The radius of the curved plate is a function of the yield strength of the steel and the load intensity.

The sole plate width minus the chamfers at each side is greater than the length of the curved plate. Then the contact length of the sole plate with the curved plate is equal to the length of the curved plate minus the pintles and the associated bevels around each of the pintles. See Figure 14.8.1.2.

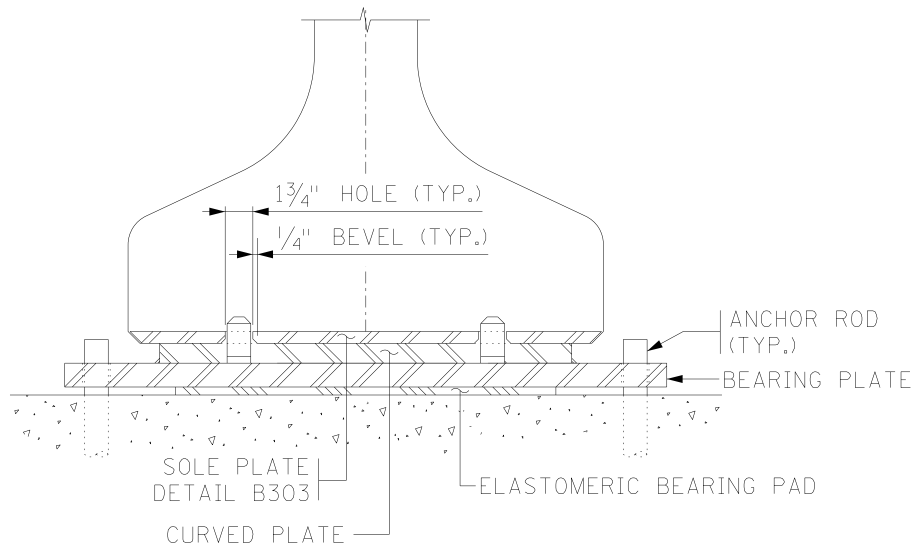


Figure 14.8.1.2

Contact length L_{sp} is equal to

$$L_{sp} = 26 - 2 \cdot (2.25) = 21.50 \text{ in}$$

[14.7.1.4] Based on past satisfactory performance of curved plate bearing assemblies, the minimum radius permitted is determined with LRFD Equation C14.7.1.4-1 and C14.7.1.4-2. Start by assuming the diameter d is 25 inches or less, so use the first equation. Rearranging the equation to solve for diameter results in the following:

$$d_{min} = \frac{20 \cdot p}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left(\frac{P_s}{L_{sp}}\right)}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left(\frac{253.6}{21.50}\right)}{0.6 \cdot (36 - 13)}$$

$$= 17.1 \text{ in} < 25.0 \text{ in}$$

The assumption was correct. Then the radius R_{min} = 8.55 inches.

The radius of curved plates is to be no less than 16 inches. Therefore, specify the minimum radius for the curved plate to be 16 inches.

The required thickness of the curved composite plate is based on a simple model in which a uniform pressure is applied to the bottom of the plate and the reaction is a line load. See Figure 14.8.1.3. Use strength limit state loads for flexural design of the steel plates.

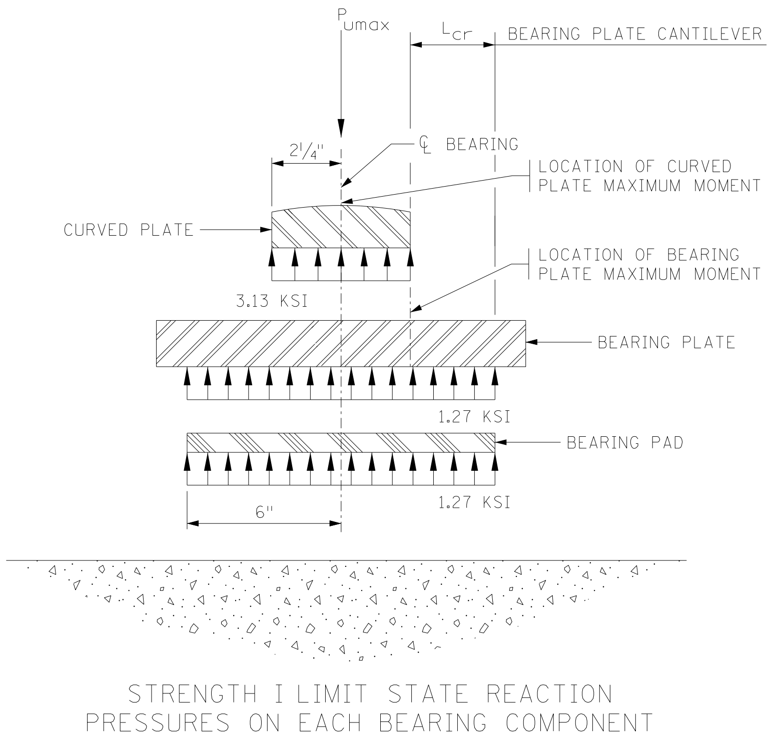


Figure 14.8.1.3

Maximum strength limit state load $P_{umax} = 1.25 \cdot P_{dl} + 1.75 \cdot P_{llmax}$
$= 1.25 \cdot 156 + 1.75 \cdot 97.6$
$= 365.8$ kips

Pressure on the composite plate is:

$$\sigma_{cp} = \frac{P_{umax}}{G \cdot H} = \frac{365.8}{4.5 \cdot 26} = 3.13 \text{ ksi}$$

Maximum moment on the composite plate is:

$$M_{ucp} = \sigma_{cp} \cdot \frac{G}{2} \cdot \frac{G}{4} \cdot H = 3.13 \cdot \frac{4.5}{2} \cdot \frac{4.5}{4} \cdot 26 = 206.0 \text{ kip-in}$$

[6.12.2.2.7]

Consider the plate to be fully laterally supported. The AASHTO LRFD specifications allow the nominal flexural resistance of a rectangular section to be taken as the plastic moment. However, MnDOT limits the nominal resistance to the yield moment. Find the required composite plate thickness such that the yield moment, M_y, of the section will have adequate capacity to resist the design moment, M_{ucp}.

[6.5.4.2]

For steel elements in flexure, ϕ_f=1.0.

The flexural resistance, M_r, of the composite plate section is:

$$M_r = \phi_f M_{ncp} = \phi_f M_y = \phi_f S_{cp} F_y$$

The section modulus of the composite plate is:

$$S_{cp} = \frac{H \cdot J^2}{6}$$

where J = thickness of composite plate

Then by substitution:

$$M_r = \phi_f \cdot \left(\frac{H \cdot J^2}{6}\right) \cdot F_y$$

Set the flexural resistance of the composite plate section equal to the design moment:

$$M_{ucp} = \phi_f \cdot \left(\frac{H \cdot J^2}{6}\right) \cdot F_y$$

Solve for composite plate thickness:

$$J \geq \sqrt{\frac{6 \cdot M_{ucp}}{\phi_f \cdot F_y \cdot H}} = \sqrt{\frac{6 \cdot 206.0}{1.0 \cdot 36 \cdot 26}} = 1.15 \text{ in}$$

The standard curved plate thickness is 1¼ inches, so composite action does not need to be considered. Use a 1¼ thick curved plate.

C. Bearing Plate Design

Per Detail B310, the length (C) is set at 2 inches longer than the pad length. This provides room for the keeper bar to be attached to the bottom of the bearing plate. The width (E) is set 8 inches greater than the beam bottom flange width. This provides room on each side for the anchor rods.

$$E = b_f + 8 = 30 + 8 = 38 \text{ in}$$

$$C = A + 2 = 12 + 2 = 14 \text{ in}$$

The bearing plate is assumed to act as a cantilever (See Figure 14.8.1.3) that carries the maximum strength limit state load to the curved plate. The cantilever length is half the difference in length between the bearing pad and the curved plate.

$$\sigma_{bp} = \frac{P_{umax}}{A \cdot B} = \frac{365.8}{12 \cdot 24} = 1.27 \text{ ksi}$$

$$L_{cr} = \frac{A}{2} - \frac{G}{2} = \frac{12}{2} - \frac{4.5}{2} = 3.75 \text{ in.}$$

$$M_{ubp} = \sigma_{bp} \cdot \frac{L_{cr^2}}{2} \cdot E = 1.27 \cdot \frac{3.75^2}{2} \cdot 38 = 339.3 \text{ kip-in}$$

[6.12.2.2.7]

Again, the AASHTO LRFD Specifications allow the nominal flexural capacity to be set equal to the plastic moment of the plate. However, MnDOT limits the nominal capacity of the plate to the yield moment. Find the required bearing plate thickness such that the yield moment, M_y, of the section will have adequate capacity to resist the design moment, M_{ubp}.

The flexural resistance, M_r, of the bearing plate section is:

$$M_r = \phi_f M_{nbp} = \phi_f M_y = \phi_f S_{bp} F_y$$

The section modulus of the bearing plate is:

$$S_{bp} = \frac{E \cdot F^2}{6}$$

Where F = thickness of bearing plate

Then by substitution:

$$M_r = \phi_f \cdot \left(\frac{E \cdot F^2}{6} \right) \cdot F_y$$

Set the flexural resistance of the bearing plate section equal to the design moment:

$$M_{ubp} = \phi_f \cdot \left(\frac{E \cdot F^2}{6} \right) \cdot F_y$$

Solve for bearing plate thickness:

$$\text{Min.} \quad F = \sqrt{\frac{6 \cdot M_{ubp}}{\phi_f \cdot F_y \cdot E}} = \sqrt{\frac{6 \cdot 339.3}{1.0 \cdot 36 \cdot 38}} = 1.22 \text{ in}$$

The standard bearing plate thickness is 1½ inches, so use a 1½ thick bearing plate.

D. Anchor Rods/Pintles

The Detail B310 standard set of two $1^1/_2$ inch anchor rods is designed for a combination of moment and shear. The standard set of two $1^1/_2$ inch pintles is designed for shear only.

[6.7.6.2.1]

The combined moment and shear of a single anchor rod can be calculated using LRFD Equation 6.7.6.2.1-1, replacing M_u with V_u multiplied by the bearing pad thickness plus half the bearing plate thickness.

$$\frac{6.0 \cdot M_u}{\phi_f \cdot D^3 \cdot F_y} + \left(\frac{2.2 \cdot V_u}{\phi_v \cdot D^2 \cdot F_y}\right)^3 = \frac{6.0 \cdot V_u \cdot \left(0.5 + \frac{1.5}{2}\right)}{1.0 \cdot 1.5^3 \cdot 36} + \left(\frac{2.2 \cdot V_u}{1.0 \cdot 1.5^2 \cdot 36}\right)^3 \leq 0.95$$

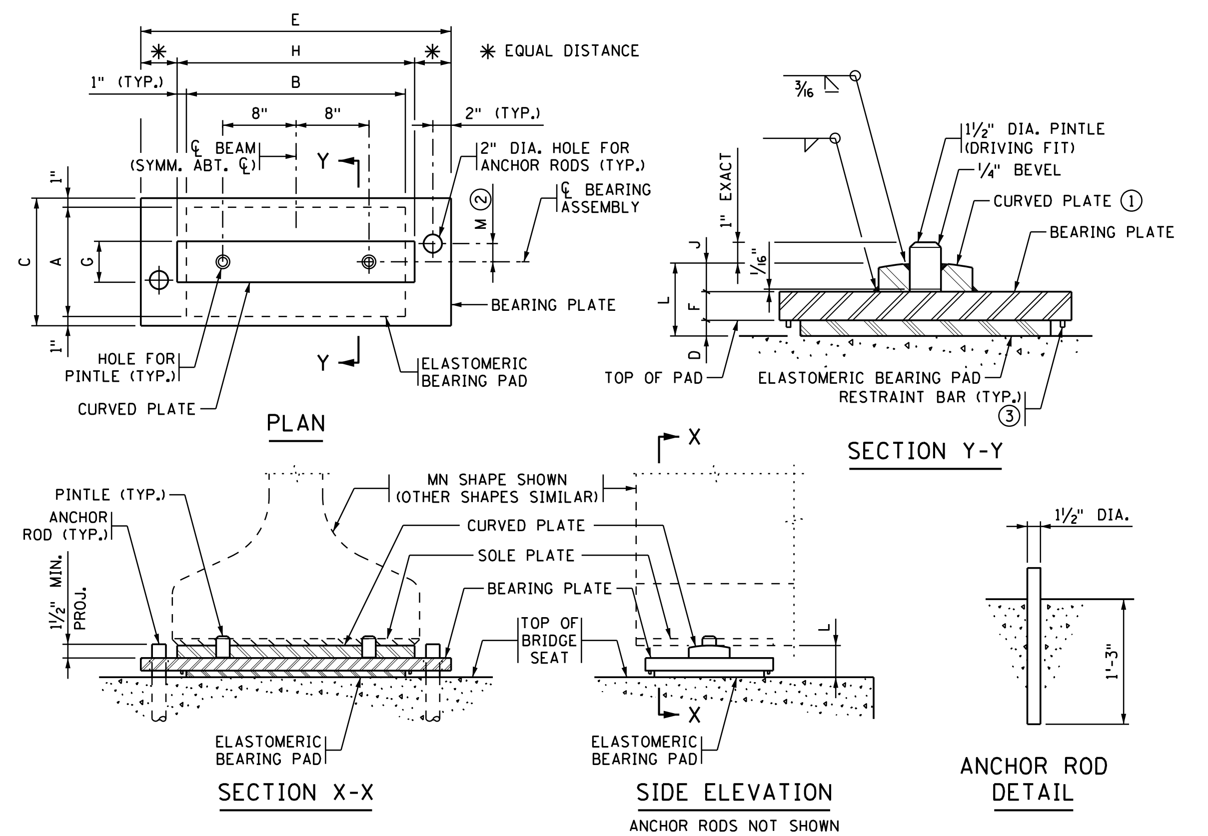
Solving for the allowable shear produces a factored load capacity of 14.4 kips per anchor rod (28.8 kips for two).

Designing the pintles for shear produces a factored load capacity of 50.3 kips per pintle (100.6 kips for two).

For many projects, such as the superstructure assumed for this design example, the capacity of the anchor rods and pintles will be adequate by inspection. For projects where two or more piers are fixed or where significant longitudinal forces are anticipated, evaluate the capacity of the anchor rods and pintles.

The anchor rod offset dimension (M) is to be calculated such that the anchor rods are located along the beam centerline of bearing. In this case, the skew is zero, so M=0 inches.

The bearing design is summarized in Figures 14.8.1.4 and 14.8.1.5.



TABLE

ASSEMBLY TYPE	LOCATION	BEAM SIZE	BEARING PAD SIZE			SHAPE FACTOR	BEARING PLATE SIZE			CURVED PLATE SIZE				ANCHOR ROD OFFSET		ASSY. HEIGHT	RESTRAINT PATTERN ③
			A	B	D		C	E	F	G	H	J	R ①	+/- ②	M	L	
F1	PIER	MN63	12"	24"	½"	8.0	14"	38"	1½"	4½"	26"	1¼"	16"		0"	3¼"	A-1

NOTES:

PROVIDE ELASTOMERIC MATERIALS AND PAD CONSTRUCTION IN ACCORDANCE WITH SPEC. 3741.

PROVIDE STEEL PLATES IN ACCORDANCE WITH SPEC. 3306.

PROVIDE ROUND BARS IN ACCORDANCE WITH SPEC. 3306 FOR USE AS ANCHOR RODS. GALVANIZE IN ACCORDANCE WITH SPEC. 3394.

PROVIDE PINTLES IN ACCORDANCE WITH SPEC. 3309.

GALVANIZE STRUCTURAL STEEL BEARING ASSEMBLY AFTER FABRICATION IN ACCORDANCE WITH SPEC. 3394, EXCEPT AS NOTED.

PAYMENT FOR BEARING ASSEMBLY INCLUDES ALL MATERIAL ON THIS DETAIL.

① THE MIN. RADIUS IS 16" UNLESS OTHERWISE SPECIFIED IN THE TABLE. THE MAX. RADIUS IS 24". FINISH TO 250 MICRO. THE FINISHED THICKNESS OF THE PLATE MAY BE 1/16" LESS THAN SHOWN.

② "+" DENOTES OFFSET AS SHOWN.
"-" DENOTES OFFSET OPPOSITE OF SHOWN.

③ REFER TO BEARING PAD RESTRAINT B-DETAIL FOR ADDITIONAL INFORMATION AND DETAILS.

APPROVED: DECEMBER 20, 2018 Kevin Western STATE BRIDGE ENGINEER	STATE OF MINNESOTA DEPARTMENT OF TRANSPORTATION CURVED PLATE BEARING ASSEMBLY (PRESTRESSED CONCRETE BEAMS) (FIXED)	REVISION 10-22-2019 12-21-2022	DETAIL NO. B310

Figure 14.8.1.4

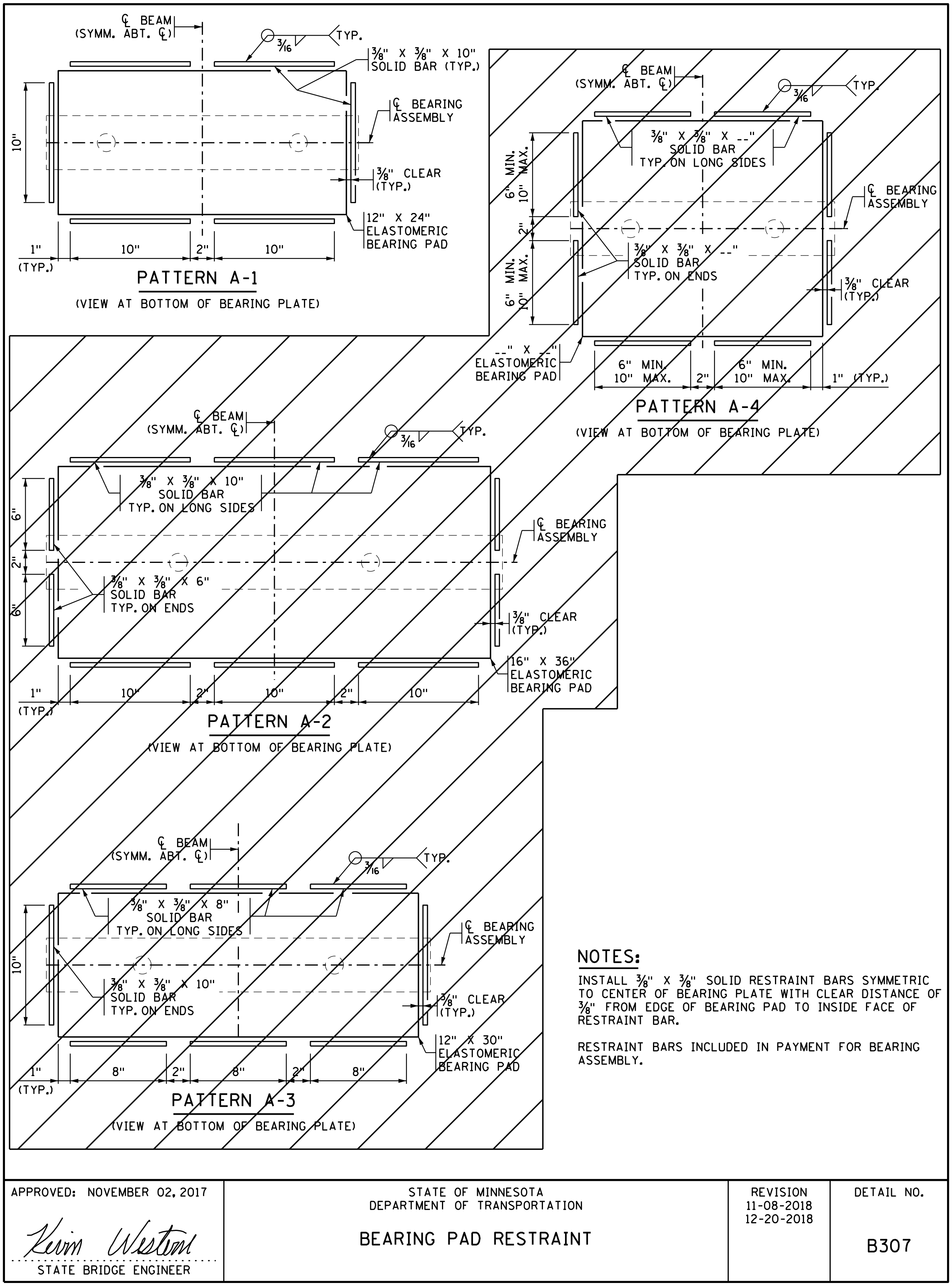
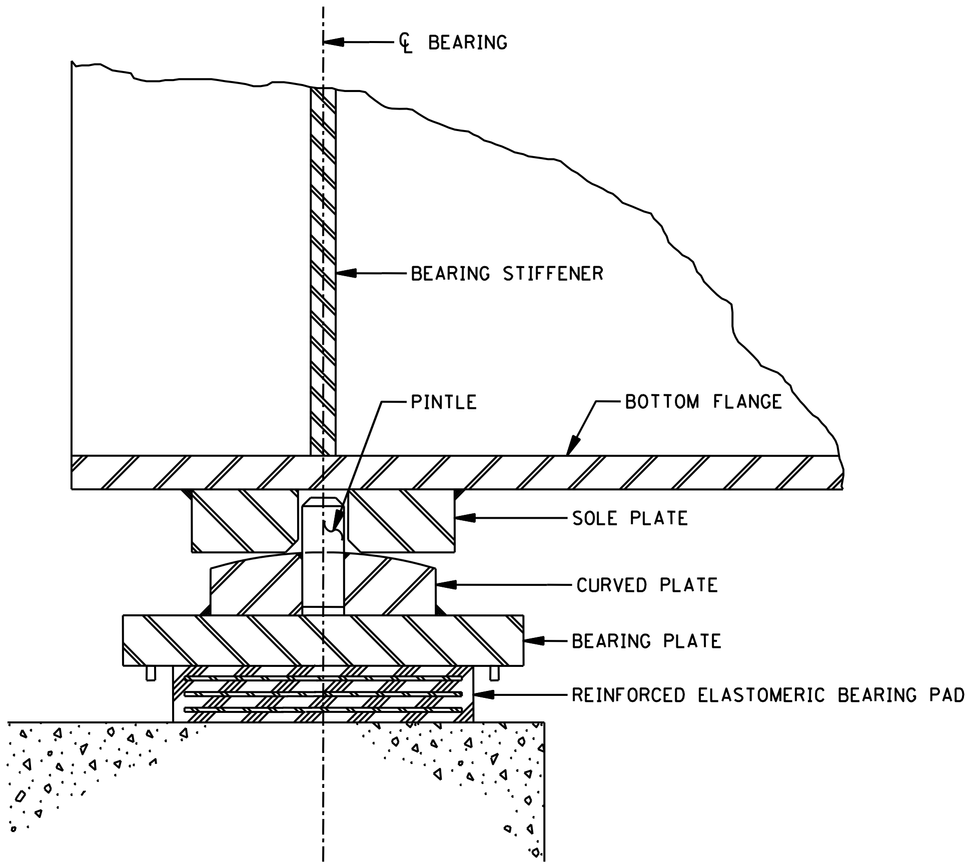


Figure 14.8.1.5

14.8.2 Expansion Elastomeric Bearing Design Example [14.7.6]

This example illustrates the design of an expansion curved plate elastomeric bearing for a steel plate girder bridge. The bearing is based on Bridge Details Part I, B355. The elastomeric bearing pad is designed using Method A (LRFD Article 14.7.6). Figure 14.8.2.1 labels the primary components for this type of bearing.



CURVED PLATE EXPANSION BEARING COMPONENTS

SEE DETAIL B355

Figure 14.8.2.1

After the maximum reaction is calculated, the bearing design should be selected from standard bearing tables in Article 14.7 of this manual. If a standard design will not work due to unusual loads or geometric constraints, a custom design will be required.

This example will outline the procedure to custom design an expansion elastomeric bearing. First determine the size of the pad required. Next determine the steel plate requirements for the rest of the assembly.

Two movements are accommodated with this type of bearing, rotation and horizontal translation. The rotation takes place at the interface between the sole plate and the curved plate. The horizontal translation takes place in the reinforced elastomeric bearing pad.

A. Design Reinforced Elastomeric Bearing Pad

The bearing pad needs sufficient plan area to ensure that compression stresses are below the limit. It also needs sufficient thickness to accommodate the horizontal translation. Begin by determining the design movements and loads for the bearing.

Design Movements

The bearing is located at the abutment of a two-span steel plate girder bridge with equal spans of 152'-0". Fixity is assumed at the pier.

Expansion length = L_{exp} = 152 ft.

[6.4.1] Coefficient of thermal expansion for steel = $\alpha_{steel} = 0.0000065$

Design temperatures:

Base Construction Temperature: $T_{constr} = 45\,°F$
Low Temperature: $T_{low} = -30\,°F$
High Temperature: $T_{high} = 120\,°F$
Temperature Fall: $T_{fall} = T_{constr} - T_{low} = 75°F$
Temperature Rise: $T_{rise} = T_{high} - T_{constr} = 75\,°F$

Movement for minimum compressive stress check (Load Factor = 1.0)

$$\Delta_u = 1.0 \cdot T_{fall} \cdot \alpha_{steel} \cdot L_{exp} = 1.0 \cdot 75 \cdot 0.0000065 \cdot 152 \cdot 12 = 0.89 \text{ in}$$

Movement for shear deformation check (Load Factor = 1.3)

$$\Delta_s = 1.3 \cdot T_{fall} \cdot \alpha_{steel} \cdot L_{exp} = 1.3 \cdot \Delta_u = 1.3 \cdot 0.89 = 1.16 \text{ in}$$

Design Loads

The design loads for the bearing are given as follows:

Dead load $= P_{dl} = 117$ kips

Maximum live load (without IM) = $P_{ll_{max}} = 108$ kips

Minimum live load (without IM) = $P_{ll_{min}} = -15$ kips

The bearing is sized using the maximum service limit state load:

$$P_{smax} = P_{dl} + P_{llmax} = 117 + 108 = 225 \text{ kips}$$

The minimum compressive load check is made with Strength I limit state load:

$$P_{umin} = 0.9 \cdot P_{dl} + 1.75 \cdot P_{llmin} = 0.9 \cdot 117 + 1.75 \cdot (-15) = 79.1 \text{ kips}$$

Size Elastomeric Bearing Pad

[Table 14.7.6.2-1]

MnDOT Spec. 3741 specifies an elastomeric pad with a hardness of 60 durometers. Per LRFD Table 14.7.6.2-1, the shear modulus G for design ranges from 0.130 to 0.200 ksi.

In order to accommodate shear deformation in the pad due to thermal movement, the total thickness of elastomer must be at least twice the design movement. The movement Δ_s with the 1.3 multiplier is used for this check.

[Eq. 14.7.6.3.4-1]

$$\text{Minimum } h_{rt} = 2 \cdot \Delta_s = 2 \cdot 1.16 = 2.32 \text{ in}$$

Thickness of cover elastomer laminate, $h_{cover} = 0.25$

Try an internal elastomer laminate thickness, $h_{ri} = 0.375$ in

Thickness of steel plates, $h_s = 0.125$ in

Determine the number of internal laminates, n, required:

$$n = \frac{\text{Min } h_{rt} - 2 \cdot h_{cover}}{h_{ri}} = \frac{2.32 - 2 \cdot 0.25}{0.375} = 4.85$$

Use 5 internal laminates.

Number of steel plates, $n_s = n + 1 = 6$

Total elastomer thickness:

$$h_{rt} = 2 \cdot (h_{cover}) + n \cdot (h_{ri}) = 2 \cdot (0.25) + 5 \cdot (0.375) = 2.375 \text{ in}$$

Height of reinforced elastomeric pad, $D = h_{rt} + n_s \cdot h_s$

$$= 2.375 + 6 \cdot 0.125 = 3.125 \text{ in}$$

For preliminary pad sizing, assume the pad allowable compression is 1.25 ksi. Round the pad width and length dimensions to even inch dimensions. For steel beams, the width (B) must be at least the bottom flange width and not more than 2 inches greater than the bottom flange width. In this case, the bottom flange width is 20 inches.

Try a pad width, B = 20 in

Solve for the minimum pad length (A):

$$A_{min} = \frac{P_{smax}}{1.25 \cdot B} = \frac{225}{1.25 \cdot 20} = 9.00 \text{ in}$$

Try a pad length, A = 10 in

Shape Factor Check

Check the shape factor of the internal laminate:

$$S_i = \frac{A \cdot B}{2 \cdot (A + B) \cdot h_{ri}} = \frac{10 \cdot 20}{2 \cdot (10 + 20) \cdot 0.375} = 8.89$$

$5.0 \le S_i = 8.89 \le 10.0$ OK

[14.7.6.1]

Check that shape factor requirements of AASHTO LRFD Art. 14.7.6.1 are met:

For this check, if cover layer thickness is greater than or equal to one-half the internal laminate thickness, n may be increased 0.5 for each cover layer.

Then

$$\frac{S_i^2}{n} = \frac{8.89^2}{5 + 0.5 + 0.5} = 13.17 < 22 \qquad \text{OK}$$

Compute the shape for the cover layers for later use in the deflection computations.

[14.7.5.1]

$$S_c = \frac{A \cdot B}{2 \cdot (A + B) \cdot h_{ri}} = \frac{10 \cdot 20}{2 \cdot (10 + 20) \cdot 0.25} = 13.33$$

Pad Dimensional Checks

Check that the bearing satisfies aspect ratio checks. The total elastomeric thickness, h_{rt}, must be less than $^1/_3$ the length of the pad's shortest side.

[14.7.6.3.6]

$$\frac{A}{3} = \frac{10}{3} = 3.33 \text{ in} > 2.375 \text{ in} \quad \text{OK}$$

Also check that maximum pad dimension (B) is no greater than 2.5 times the smallest pad dimension (A):

$2.5 \cdot A = 2.5 \cdot 10 = 25 \text{ in} > 20 \text{ in}$ OK

[14.7.6.3.2]

Maximum Compressive Stress Check

Now check the maximum compressive stress in the pad. Use the minimum shear modulus for this computation (G_{min} = 0.130 ksi).

The allowable compressive stress σ_{sall} is the smaller of:

$$\sigma_{sall} = 1.25 \cdot G_{min} \cdot S_i$$
$$= 1.25 \cdot 0.130 \cdot 8.89$$
$$= 1.44 \text{ ksi}$$

or σ_{sall} = 1.25 ksi. GOVERNS

Then the maximum service limit state stress is:

$$\text{Actual } \sigma_s = \frac{P_s}{A \cdot B} = \frac{225}{10 \cdot 20} = 1.13 \text{ ksi} < 1.25 \text{ ksi} \quad \text{OK}$$

[14.7.6.3.3]
[14.7.5.3.6]

Compressive Deflection

To ensure that joints and appurtenances perform properly, the vertical deflection in elastomeric bearings is checked. Due to the nonlinear behavior of the elastomer, the movement associated with live load is computed by subtracting the dead load deflection from the total load deflection.

Begin by determining the average vertical compressive stress in the bearings under dead load alone and under total load.

$$\sigma_{dl} = \frac{P_{dl}}{A \cdot B} = \frac{117}{10 \cdot 20} = 0.585 \text{ ksi}$$

$$\sigma_{tl} = \frac{P_{tl}}{A \cdot B} = \frac{225}{10 \cdot 20} = 1.125 \text{ ksi}$$

Using the stress strain figure for 60 durometer reinforced bearings shown in AASHTO LRFD Figure C14.7.6.3.3-1, the strain was estimated in the interior laminates and the cover layers and summarized in Table 14.8.2.1.

Table 14.8.2.1 Estimated Strains

Laminate	Load	S	Stress (ksi)	Estimated Compressive Strain ε (%)
Interior	Dead Load	8.89	0.585	2.6%
	Total Load	8.89	1.125	4.2%
Cover	Dead Load	13.33	0.585	2.2%
	Total Load	13.33	1.125	3.7%

The initial compressive deflection of a single interior laminate under total load is:

[14.7.6.3.3]

$$\Delta_{tlhri} = \varepsilon \cdot h_{ri} = 0.042 \cdot h_{ri} < 0.090 \cdot h_{ri} \quad \text{OK}$$

With five interior laminates and two cover layers the deflection under total load is:

$$\Delta_{tl} = 5 \cdot \varepsilon_{ri} \cdot h_{ri} + 2 \cdot \varepsilon_{cover} \cdot h_{rcover}$$

$$= 5 \cdot 0.042 \cdot 0.375 + 2 \cdot 0.037 \cdot 0.25 = 0.097 \text{ in}$$

The deflection under dead load is:

$$\Delta_{dl} = 5 \cdot \varepsilon_{ri} \cdot h_{ri} + 2 \cdot \varepsilon_{cover} \cdot h_{rcover}$$

$$= 5 \cdot 0.026 \cdot 0.375 + 2 \cdot 0.022 \cdot 0.25 = 0.060 \text{ in}$$

[Table 14.7.6.2-1] The deflection due to creep is:

$$\Delta_{cr} = 0.35 \cdot \Delta_{dl} = 0.35 \cdot 0.060 = 0.021 \text{ in}$$

[C14.7.5.3.6]

The difference between the two deflections is the estimated live load deflection. The total deflection due to live load plus creep should be no greater than $^{1}/_{8}$ inch.

$$\Delta_{ll} = \Delta_{tl} - \Delta_{dl} = 0.097 - 0.060 = 0.037 \text{ in}$$

$$\Delta_{ll} + \Delta_{cr} = 0.037 + 0.021 = 0.058 \text{ in} < 0.125 \text{ in} \quad \text{OK}$$

Minimum Compressive Load Check

Using the equation derived in BDM Article 14.3.3.3.1:

$$\text{Req'd. } P_{umin} \geq 5 \cdot G_{max} \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

$$= 5 \cdot 0.200 \cdot 10 \cdot 20 \cdot \frac{0.89}{2.375} = 74.9 \text{ kips}$$

Actual P_{umin} = 79.1 kips > 74.9 kips OK

[14.7.5.3.5] **Check Service and Fatigue of Steel Reinforcement Plates**

Check the service and fatigue limit states for the steel plates. At the service limit state the following equation must be satisfied:

$$h_s \geq \frac{3 \cdot h_{max} \cdot \sigma_s}{F_y}$$

The yield strength of the steel plates (F_y) is 36 ksi.

$$h_{max} = h_{ri} = 0.375 \text{ in}$$

σ_s = 1.13 ksi

$$\text{Min. } h_s = \frac{3 \cdot h_{max} \cdot \sigma_s}{F_y} = \frac{3 \cdot 0.375 \cdot 1.13}{36} = 0.035 \text{ in} < 0.125 \text{ in} \quad \text{OK}$$

At the fatigue limit state, the following equation must be satisfied:

$$h_s \geq \frac{2 \cdot h_{max} \cdot \sigma_L}{\Delta_{FTH}}$$

[Table 6.6.1.2.5-3] where, Δ_{FTH} = 24 ksi (Category A steel detail).

Note that the live load used for this check is not based on reactions from the fatigue truck and is not factored according to the fatigue limit state. Rather, it is the maximum live load for the service limit state with a load factor equal to 1.0.

$$\sigma_L = \frac{P_{llmax}}{A \cdot B} = \frac{108}{10 \cdot 20} = 0.540 \text{ ksi}$$

Minimum steel plate thickness for this check is

$$\text{Min. } h_s = \frac{2 \cdot h_{max} \cdot \sigma_L}{\Delta_{FTH}} = \frac{2 \cdot 0.375 \cdot 0.540}{24} = 0.017 < 0.125 \text{ in} \quad \text{OK}$$

Use a 10" x 20" x $3^1/_8$" bearing pad, composed of two $^1/_4$ inch cover laminates, five $^3/_8$ inch interior laminates, and six $^1/_8$ inch steel plates.

B. Curved Plate Design

The thickness of the plate is H. The curved plate has a width (B), which is equal to the width of the bearing pad. The length (G) is determined in an iterative process with the thickness. Begin by checking the thickness for a curved composite plate with a length of 4.5 inches. If, when designing the bearing plate, the required bearing plate thickness exceeds 2 inches, increase the length of the curved plate to reduce the length of the cantilever for the bearing plate design. Increase the curved plate length until the required bearing plate thickness alone and the required composite plate thickness for the curved plate design become approximately equal.

Try a 20" x 4.5" curved plate (B = 20) in, G = 4.5 in).

First, determine the radius of the contact surface. The radius of the curved plate is a function of the yield strength of the steel and the load intensity.

The contact length of the sole plate with the curved plate is equal to the curved plate width minus the pintles and bevels. Refer to Figure 14.8.2.2.

Contact length L_{sp} is equal to

$$L_{sp} = 20 - 2 \cdot (1.75) - 2 \cdot (0.25) = 15.50 \text{ in}$$

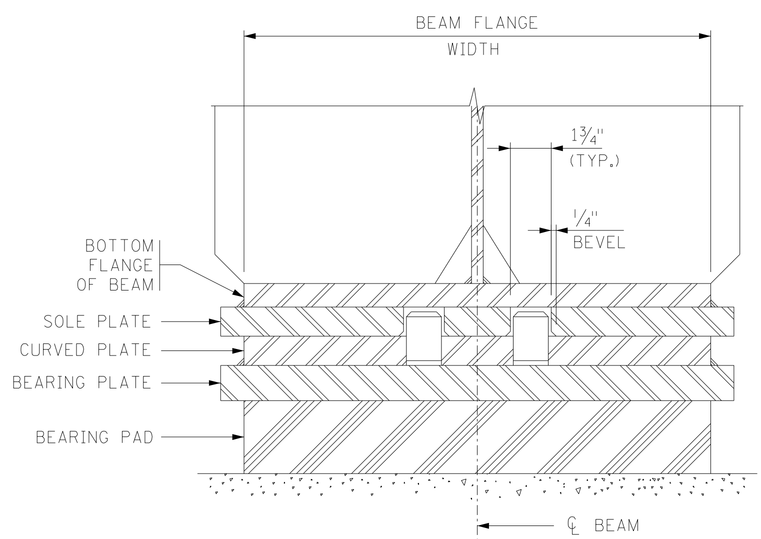


Figure 14.8.2.2

[14.7.1.4]

Based on past satisfactory performance of curved plate bearing assemblies, the minimum radius permitted is determined with LRFD Equation C14.7.1.4-1 and C14.7.1.4-2. Start by assuming the diameter d is 25 inches or less, so use the first equation. Rearranging the equation to solve for diameter results in the following:

$$d_{min} = \frac{20 \cdot p}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left(\frac{P_s}{L_{sp}}\right)}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left(\frac{225}{15.50}\right)}{0.6 \cdot (36 - 13)} = 21.0 \text{ in} < 25.0 \text{ in}$$

The assumption was correct. Then the radius R_{min} = 10.5 in

The radius of curved plates is to be no less than 16 inches. Therefore, specify the minimum radius for the curved plate to be 16 inches.

Use strength limit state loads for flexural design of the curved plate.

The maximum Strength I limit state load, P_{umax}, is:

$$P_{umax} = 1.25 \cdot P_{dl} + 1.75 \cdot P_{llmax}$$

$$= 1.25 \cdot 117 + 1.75 \cdot (108) = 335.3 \text{ kips}$$

Pressure on the composite plate is:

$$\sigma_{cp} = \frac{P_{umax}}{G \cdot B} = \frac{335.3}{4.5 \cdot 20} = 3.73 \text{ ksi}$$

Maximum moment on the composite plate is:

$$M_{ucp} = \sigma_{cp} \cdot \frac{G}{2} \cdot \frac{G}{4} \cdot B = 3.73 \cdot \frac{4.5}{2} \cdot \frac{4.5}{4} \cdot 20 = 188.8 \text{ kip-in}$$

[6.12.2.2.7]

Consider the plate to be fully laterally supported. The AASHTO LRFD Specifications allow the nominal flexural resistance of a rectangular section to be taken as the plastic moment. However, MnDOT limits the nominal resistance to the yield moment. Find the required composite plate thickness such that the yield moment, M_y, of the section will have adequate capacity to resist the design moment, M_{ucp}.

[6.5.4.2]

For steel elements in flexure, ϕ_f=1.0.

The flexural resistance, M_r , of the composite plate section is:

$$M_r = \phi_f \cdot M_{ncp} = \phi_f \cdot M_y = \phi_f \cdot S_{cp} \cdot F_y$$

The section modulus of the composite plate is:

$$S_{cp} = \frac{B \cdot H^2}{6}$$

where H = thickness of composite plate

Then by substitution:

$$M_r = \phi_f \cdot \left(\frac{B \cdot H^2}{6}\right) \cdot F_y$$

Set the flexural resistance of the composite plate section equal to the design moment:

$$M_{ucp} = \phi_f \cdot \left(\frac{B \cdot H^2}{6}\right) \cdot F_y$$

Solve for composite plate thickness:

$$H \geq \sqrt{\frac{6 \cdot M_{ucp}}{\phi_f \, F_y \cdot B}} = \sqrt{\frac{6 \cdot 188 \cdot 8}{1.0 \cdot 36 \cdot 20}} = 1.25 \text{ in}$$

The standard curved plate thickness is 1¼ inches, so composite action does not need to be considered. Use a 1¼ thick curved plate.

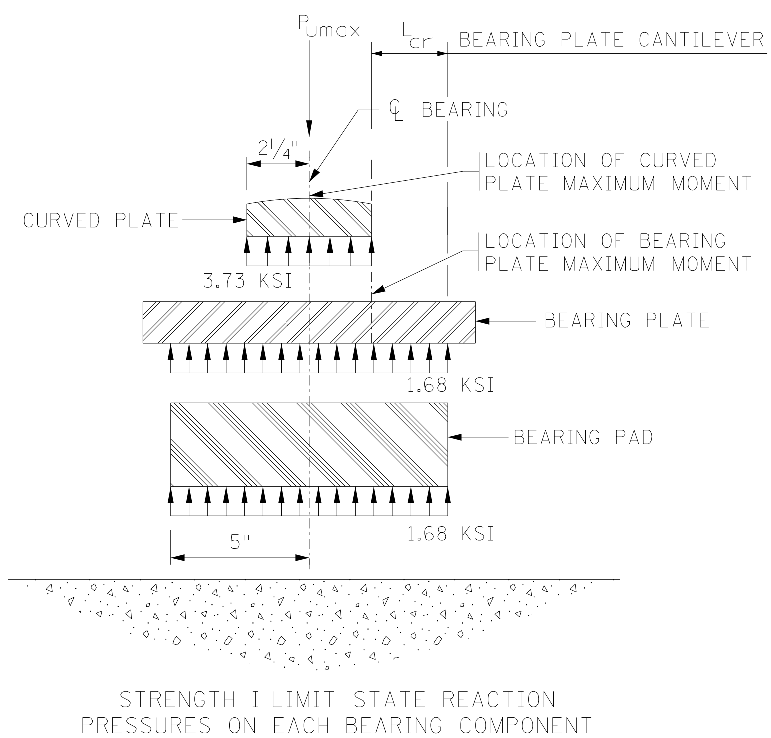


Figure 14.8.2.3

C. Bearing Plate Design

Now determine the thickness of the bearing plate. The bearing plate has plan dimensions that are slightly larger than the bearing pad to provide adequate space for the attachment of the keeper bar. One inch is provided on all sides for this purpose.

Bearing Plate width, E = 22 in

Bearing Plate length, C = 12 in

The bearing plate is assumed to act as a cantilever that carries the bearing pad pressure back to the curved plate. See Figure 14.8.2.3.

The cantilever length is half the difference in length between the bearing pad and the curved plate.

$$\sigma_{bp} = \frac{P_{umax}}{A \cdot B} = \frac{335.3}{10 \cdot 20} = 1.68 \text{ ksi}$$

$$L_{cr} = \frac{A}{2} - \frac{G}{2} = \frac{10}{2} - \frac{4.5}{2} = 2.75 \text{ in}$$

$$M_{ubp} = \sigma_{bp} \cdot \frac{L_{cr}^2}{2} \cdot E = 1.68 \cdot \frac{2.75^2}{2} \cdot 22 = 139.8 \text{ kip-in}$$

Again, the AASHTO LRFD specifications allow the nominal flexural capacity to be set equal to the plastic moment of the plate. However, MnDOT limits the nominal capacity of the plate to the yield moment. Find the required bearing plate thickness such that the yield moment, M_y, of the section will have adequate capacity to resist the design moment, M_{ubp}.

The flexural resistance, M_r, of the bearing plate section is:

$$M_r = \phi_f \cdot M_{nbp} = \phi_f \cdot M_y = \phi_f \cdot S_{bp} \cdot F_y$$

The section modulus of the bearing plate is:

$$S_{bp} - \frac{E \cdot F^2}{6}$$

where F = thickness of bearing plate

Then by substitution:

$$M_r = \phi_f \cdot \left(\frac{E \cdot F^2}{6}\right) \cdot F_y$$

Set the flexural resistance of the bearing plate section equal to the design moment:

$$M_{ubp} = \phi_f \cdot \left(\frac{E \cdot F^3}{6}\right) \cdot F_y$$

Solve for bearing plate thickness:

$$F \geq \sqrt{\frac{6 \cdot M_{ubp}}{\phi_f \, F_y \cdot E}} = \sqrt{\frac{6 \cdot 139.8}{1.0 \cdot 36 \cdot 22}} = 1.03 \text{ in}$$

The standard bearing plate thickness is 1½ inches, so use a 1½ thick bearing plate.

D. Sole Plate Constraints

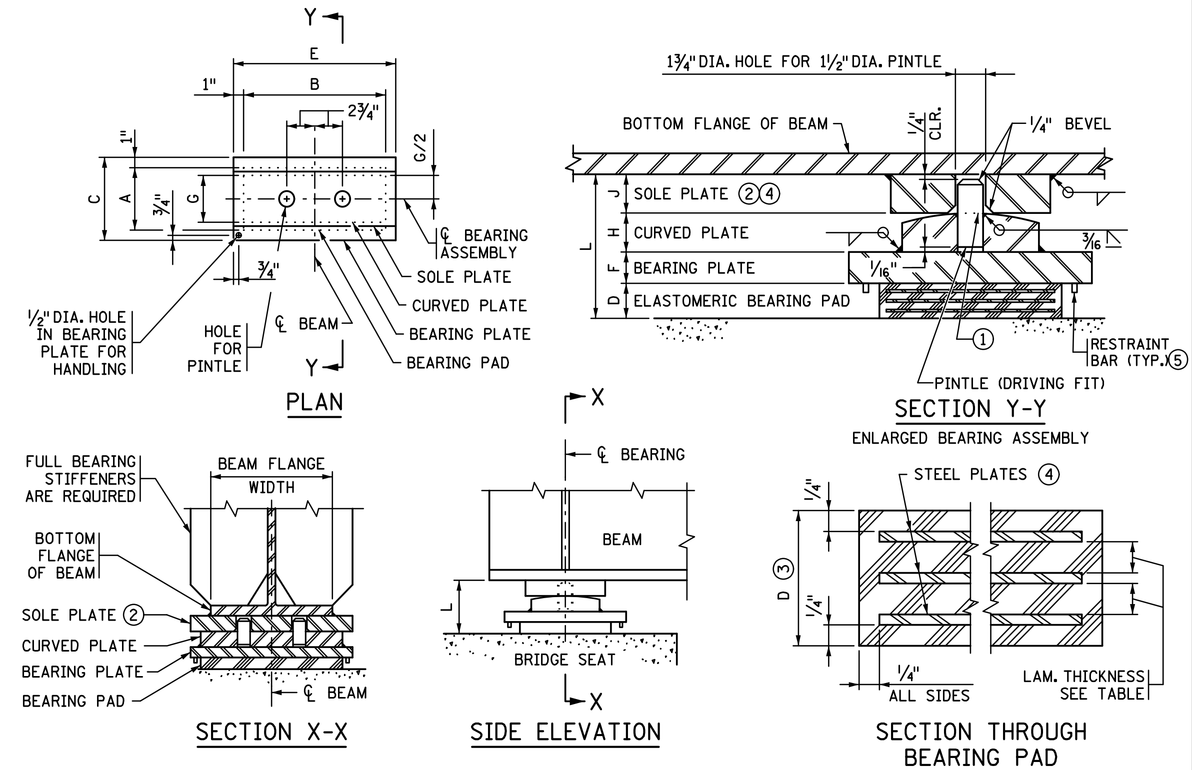
Set the sole plate width 2 inches greater than the curved plate width and check that it is sufficiently wider than the beam bottom flange to allow welding.

Sole plate width = 20 + 2 = 22 in > 20 in flange OK

The sole plate length must be 6 inches minimum, but not less than the curved plate length. Therefore, set sole plate length equal to 6 inches.

The minimum sole plate thickness is $1^1/_4$ inches. When the bearing pad width exceeds the bottom flange width, the sole plate must be designed as a cantilever to resist the load from the pad that extends outside the flange. For our case, the bottom flange width equals the pad width, so set sole plate thickness equal to $1^1/_4$ inches.

The bearing design is summarized in Figures 14.8.2.4 and 14.8.2.5.



TABLE

ASSEMBLY TYPE	LOCATION	BEAM FLANGE WIDTH	BEARING PAD SIZE A	BEARING PAD SIZE B	BEARING PAD SIZE D	STEEL PLATES NO.	STEEL PLATES THICK.	LAMINATES NO.	LAMINATES THICK.	SHAPE FACTOR	BEARING PLATE SIZE C	BEARING PLATE SIZE E	BEARING PLATE SIZE F	CURVED PLATE SIZE G	CURVED PLATE SIZE B	CURVED PLATE SIZE H	CURVED PLATE SIZE R ①	SOLE PLATE SIZE WID.	SOLE PLATE SIZE LEN.	SOLE PLATE SIZE J ②	PINTLE DIA.	ASSY. HEIGHT L	RESTRAINT PATTERN ⑤
E	ABUT.	20"	10"	20"	3 1/8"	6	1/8"	5	3/8"	8.89	12"	22"	1 1/2"	4 1/2"	20"	1 1/4"	16"	22"	6"	1 1/4"	1 1/2"	7 7/8"	A-3

NOTES:

PROVIDE ELASTOMERIC MATERIALS AND PAD CONSTRUCTION PER SPEC. 3741.

PROVIDE STEEL PLATES PER SPEC. 3306 EXCEPT THE SOLE PLATE. PROVIDE SOLE PLATE WITH THE SAME MATERIAL SPECIFICATION AS THE STEEL BEAMS.

PROVIDE PINTLES PER SPEC. 3309.

GALVANIZE STRUCTURAL STEEL BEARING ASSEMBLY AFTER FABRICATION PER SPEC. 3394, EXCEPT AS NOTED.

PAYMENT FOR BEARING ASSEMBLY INCLUDES ALL MATERIAL ON THIS DETAIL EXCEPT THE SOLE PLATE. THE SOLE PLATE IS INCLUDED IN THE WEIGHT OF STRUCTURAL STEEL.

① THE MIN. RADIUS IS 16" UNLESS OTHERWISE SPECIFIED IN THE TABLE. THE MAX. RADIUS IS 24". FINISH TO 250 MICRO. THE FINISHED THICKNESS OF THE PLATE MAY BE 1/16" LESS THAN SHOWN.

② WHEN THE SOLE PLATE IS TAPERED, DIMENSIONS "J" AND "L" ARE THICKNESS OF SOLE PLATE AND BEARING ASSEMBLY AT CENTERLINE OF BEARING.

③ THE TOTAL THICKNESS SHOWN INCLUDES THE STEEL PLATES.

④ DO NOT GALVANIZE THIS PLATE.

⑤ REFER TO BEARING PAD RESTRAINT SHEET FOR ADDITIONAL INFORMATION AND DETAILS.

DESIGN DATA:

MAX. FACTORED SHEAR RESISTANCE:
- 50.3 KIPS PER 1 1/2" DIA. PINTLE

APPROVED: NOVEMBER 22, 2002	STATE OF MINNESOTA DEPARTMENT OF TRANSPORTATION	REVISED	DETAIL NO.
Daniel L Dorgan STATE BRIDGE ENGINEER	CURVED PLATE BEARING ASSEMBLY (STEEL BEAMS) (EXPANSION)	08-10-2006 12-17-2008 11-03-2015 11-02-2017	B355

Figure 14.8.2.4

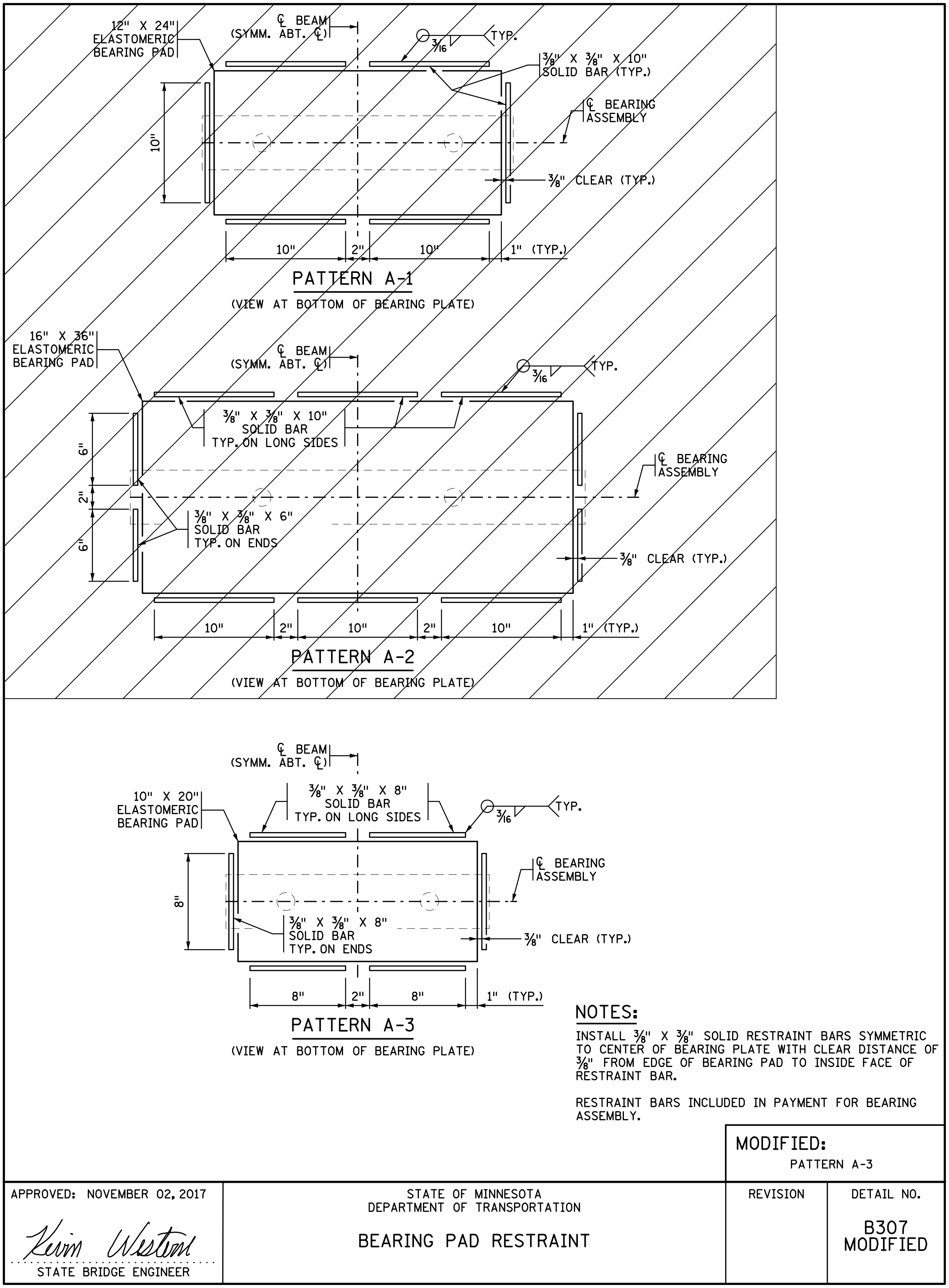


Figure 14.8.2.5

DEPARTMENT OF TRANSPORTATION
Mail Stop 610
3485 Hadley Avenue North
Oakdale, MN 55128

Office Tel: 651-366-4506
Office Fax: 651-366-4497

Memo

Date: 11/3/2023

To: Bridge Design Engineers

From: Arielle Ehrlich 2023.11.03 08:11:11 -05'00'
State Bridge Design Engineer

RE: Memo to Designers #2023-01: Debonded Strands in Prestressed Concrete Beams

Introduction

This memo provides guidance on the use of debonded strands in prestressed concrete beams.

Debonded strands are used to reduce release stresses at the beam ends while maintaining flexural capacity at the midspan. This is accomplished by intentionally debonding the strands at the beam ends by mechanical means so that concrete does not bond to the strand, typically through the use of a sheathing applied to the debonded region. Debonding is only used on straight strands and is an alternative to the draped strands historically used by MnDOT.

MnDOT has historically required the use of fully bonded straight or draped strands only, not allowing debonded strands due to concerns with potential water and chloride intrusion at the beam ends. However, satisfactory in-service performance for beams with debonded strands located in states with similar climates has led to decreased concerns about corrosion related to debonded strands. Additionally, straight strands pose less of a safety hazard to fabricators than draped strands, and the use of debonded strands allows for safer fabrication without sacrificing efficiency.

Effective immediately, use debonded straight strands in prestressed concrete beams provided that both beam ends are encased in concrete to mitigate concerns regarding water intrusion. This typically limits the use of debonded strands to single span structures with integral or semi-integral abutments.

Design Requirements

General

Except as identified in this memo, design prestressed beams with debonded strands using the requirements for prestressed beams specified in the MnDOT *LRFD Bridge Design Manual* (BDM) and the current edition of the AASHTO *LRFD Bridge Design Specifications* (AASHTO). All AASHTO references will be for the 9th Edition, 2020, as it is the current edition as of publication of this memo.

AASHTO mentions both debonded and unbonded strands throughout Section 5. It is important to note that debonded and unbonded strands are different. AASHTO Article 5.2 defines a debonded strand as a “pretensioned prestressing strand that is bonded for a portion of its length and intentionally debonded elsewhere through the use of mechanical or chemical means.” Unbonded strands are defined as strands “that are effectively bonded at only their anchorages and intermediate bonded sections.” This memo addresses debonded strands only.

Do not combine debonded strands and draped strands within the same beam.

Resistance Factors

For beams that include debonded strands, use resistance factors for prestressed concrete sections per AASHTO Article 5.5.4.2 (i.e. $\phi = 1.00$ for tension-controlled flexure and $\phi = 0.90$ for shear and torsion). Note that the sixth bullet in AASHTO Article 5.5.4.2 specifies a lower resistance factor for shear and torsion for sections with debonded strands, but it applies only to post-tensioned segmental construction.

Shear

When determining the effective shear depth, d_v, neglect all debonded strands over their debonded length. As a result, the effective shear depth will vary over the length of the beam and must be accounted for appropriately in calculations.

It should also be noted that the vertical portion of prestressing force, V_p, will be zero since there are no draped strands.

Strand Placement

AASHTO Article 5.9.4.3.3 provides several restrictions for debonded strands at the ends of beams, including limits on the number of debonded strands per row, distribution of debonded strands, and locations where debonded strands cannot be placed. Follow the guidance of AASHTO Article 5.9.4.3.3 except as defined below.

For all I-beams with debonded strands, use fully bonded strands for the outermost strands in the bottom flange and for all strands located within the projection of the web width. For all rectangular beams with debonded strands, uniformly distribute the debonded strands across the width of the section and provide fully bonded strands for the outermost strands. Additionally, bond all base strands for I-beams and rectangular beams, as shown on the standard beam sheets and BDM Figure 5.4.3.1, for their entire length.

Top Flange Strands

Ideally, use of debonding at the beam ends and accounting for bonded mild reinforcement will keep stresses near the beam ends below the AASHTO Article 5.9.2.3.1b tension limit at release:

$$0.24\lambda\sqrt{f'_{ci}}$$

However, in some cases where the amount of debonded strands has reached the AASHTO limit, additional strands may be required in the top flange to satisfy the tension stress limit at release. Fully bonded straight strands located in the top flange raise the center of gravity of the pretensioning force and therefore reduce the effectiveness of the prestressing at midspan. Debonding these top flange strands over the middle portion of the span may be necessary to keep beam stresses at midspan below the AASHTO Article 5.9.2.3.2b tension limit after losses:

$$0.19\lambda\sqrt{f'_{ci}} \leq 0.6 \text{ ksi}$$

In addition to controlling release stresses, top flange strands may also be necessary for handling and transportation stability concerns in long span beams per AASHTO Article 5.5.4.3. Strands located in the top flange that require debonding over the middle portion of the span shall be treated as temporary strands per AASHTO Article 5.9.4.5. They will require access pockets in the beam for cutting of the strands before deck placement. The designer must investigate if permanent diaphragms (when applicable) can safely accommodate any sweep or vertical deflection created during the temporary strand cutting process, or if temporary bracing is required until after all temporary strands are cut.

Place all temporary strands in one row, 3 inches from the top of beam. Limit the number of temporary strands to four or as directed by MnDOT fabricators. Horizontally locate temporary strands symmetrically about the beam centerline as shown in Figure 1. Permanent top flange strands (not debonded) may be placed in the two web strand columns at 2 inch spacing increments starting 3 inches from the top of beam.

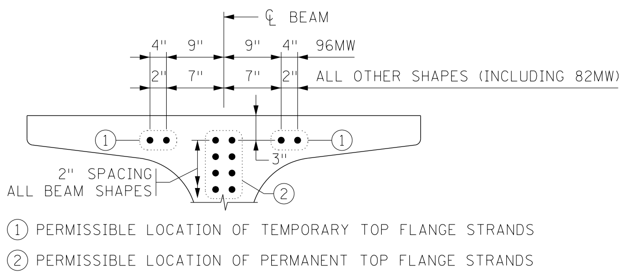


Figure 1

Prestressing Development

Per AASHTO Article 5.9.4.3.1, the stress in prestressing steel is assumed to increase linearly from zero to the effective stress after losses over the strand's transfer length. From the end of the transfer length to the end of development length the stress is assumed to increase linearly from the effective stress after losses to the stress at nominal resistance. This is the case for both bonded and debonded strands. As a result, the main difference between the design of a beam with debonded strands and a beam with draped strands is the amount of bookkeeping required.

With no debonding, all strands are fully transferred at the same point in the beam. This is also true for the development length. For beams with debonded strands, these points vary over the length of the beam. Due to the debonding, there will be several different locations at which various strands will become fully transferred or developed, increasing the locations needing to be checked. Note that AASHTO Article 5.9.4.3.3 requires a value of κ=2.0 when determining the development length of a debonded strand.

Camber

When determining estimated camber use the following equations from *Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures – Design Examples*, Publication No. FHWA-NHI-15-058, April 2007 Revised August 2015. The equations are found in Design Step 5.6.7:

$$\Delta_{ps_{total}} = \sum_{i=1}^{n} \Delta_{ps_i}$$

$$\Delta_{ps_i} = \frac{P_{t_i} \cdot e_{s_i} \cdot L^2}{8 \cdot E_{ci} \cdot I}$$ (straight bonded strands and temporary top strands)

$$\Delta_{ps_i} = \frac{P_{t_i} \cdot e_{s_i} \cdot \left[L^2 - \left(L_t + 2 \cdot L_{x_i}\right)^2\right]}{8 \cdot E_{ci} \cdot I}$$ (strands debonded at girder ends)

where:

$\Delta_{ps_{total}}$ = upward camber of beam immediately after release, due to prestress alone (in)

Δ_{ps_i} = upward camber contribution immediately after release, due to individual strand group (in)

P_{t_i} = prestress force immediately after release of individual strand group (kips)

e_{s_i} = eccentricity of prestress force with respect to the beam centroid at midspan of individual strand group (in)

L = end to end length of beam (in)

L_t = transfer length of strand (in)

L_{x_i} = length of debonding from end of beam of individual strand group (in)

E_{ci} = modulus of elasticity of concrete at prestress transfer (ksi)

I = beam moment of inertia (in^4)

Note that P_{t_i}, e_{s_i}, and L_{x_i} in the above equations are based only on the strands in question (i.e. only straight fully bonded strands, temporary top strands, or individual debonded strand groups). The total camber due to prestressing is then the summation of each individual camber from the above equations. For beams with debonded strands, follow current BDM guidance regarding use of camber multipliers and when a refined camber analysis is required.

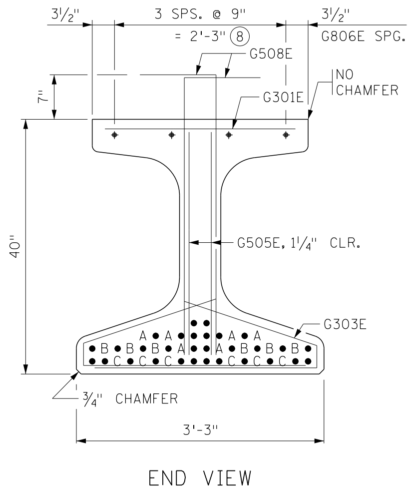
Include a construction stage in beam camber design for beams with temporary top flange strands debonded at midspan. For beams using camber multipliers, the added stage should remove the effect of temporary top flange strands after the camber multipliers are applied to all strands as described above. The effect of temporary top flange strands can be removed by using the camber equation for “temporary top strands” above and substituting updated values for time dependent variables P and E. This estimates the strand cutting process at a later date assuming the bonded region for top flange temporary strands is kept minimal. For beams that require a refined camber analysis, similarly add a construction stage for the removal of temporary strands debonded at midspan, incorporating time dependent properties of the concrete and prestressed losses.

Plan Development

Until there are standard beam sheets for debonding, modify the standard beam sheet for the correct size beam as follows:

- Remove all references to draped strands from the sheet.
- Modify the end view by labeling debonded strands with letters. The set of strands with the shortest debonded length should be labeled “A”. Continue through the alphabet as needed. See Figure 2 below for an example.

- Include an “End Debonding” table near the “End View”. The table should include the debonding symbol, number of strands, and length of debonding from end of beam. See Figure 2 for an example.



END DEBONDING		
SYMBOL	NO.	LENGTH
A	6	14'-0"
B	6	18'-0"
C	6	22'-0"
TOTAL	18	

DEBONDING LENGTHS FOR A, B, AND C ARE MEASURED FROM EACH END OF BEAM.

Figure 2

- Modify the “Y Distances” table to show the total number of strands per row and the row’s corresponding center of gravity. See Figure 3 for an example.

Y DISTANCES		
	NO.	Y (INCHES)
ROW 1 (BOTTOM)	18	2
ROW 2	18	4
ROW 3	10	6
ROW 4	2	8
5 TOTAL STRANDS	48	

Y = DISTANCE TO CENTER OF GRAVITY OF STRANDS FROM BOTTOM OF BEAM. ALL STRANDS SPACED 2" CENTER TO CENTER, HORIZONTALLY AND VERTICALLY, EXCEPT AS NOTED.

Figure 3

- For the special case where strands in the top flange require debonding over the middle portion of the span, include details for access pockets in the beams and notes/special provisions regarding cutting of the strands before deck placement. Space access pockets at 2 foot increments longitudinally, starting 2 feet from the bonded zone. Place access pockets symmetrically about the beam centerline at the low end of beam. See Figure 4 for an example.

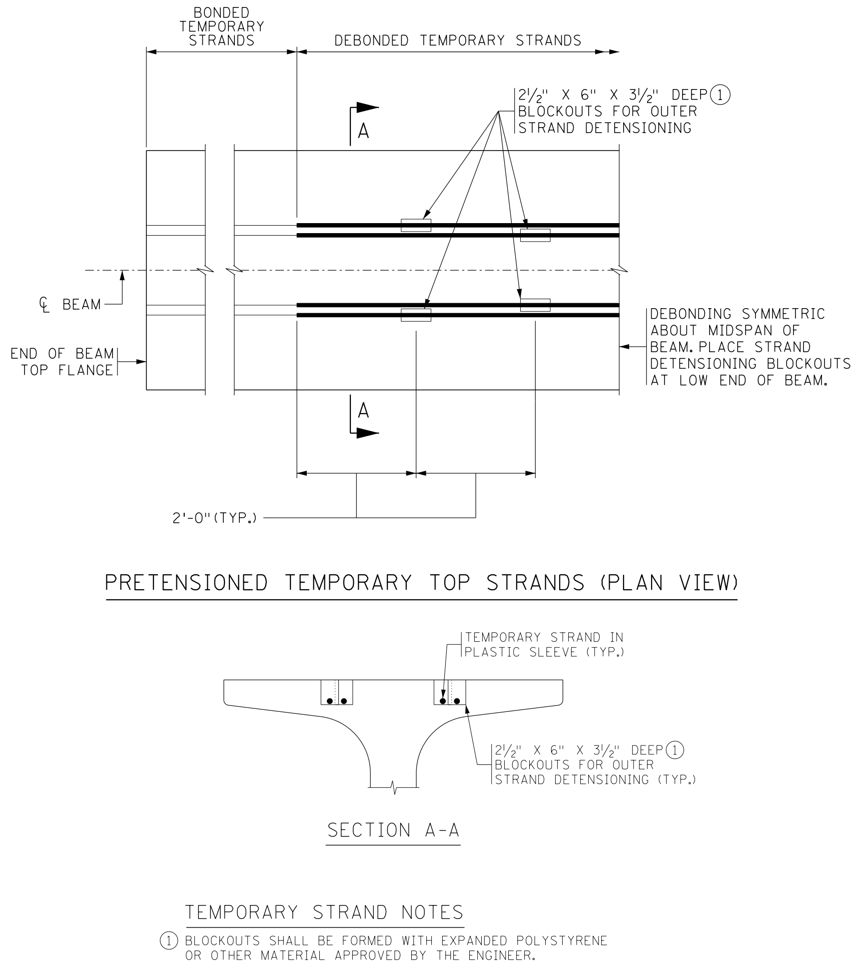


Figure 4

Bridge Special Provision Requirements

A standard special provision for beams that include debonded strands is available as part of the *2020 "SB" Bridge Special Provisions* which can be downloaded from the Bridge Office web site at:

http://www.dot.state.mn.us/bridge/construction.html

The special provision is denoted as SB2020-2405.7 and describes proper fabrication practices for prestressed girders containing debonded strands. Requirements for material properties are provided, as well as instructions for proper sealing of debonded regions to prevent moisture ingress. The sheathing must be sealed within the formwork prior to casting the girder concrete, as well as at the girder ends after the strands have been released. Oversized rigid sheathing is the required debonding material to ensure adequate space for strand dilation and prevent cement paste leakage into the sheathing. Double slit sheath debonding is only allowed in remedial cases to the allowable length listed in the special provisions. The strand release pattern used for girders containing debonded strands also differs from the typical strand release pattern specified for prestressed girders with draped strands. This alternative strand release pattern for girders containing debonded strands is described in SB2020-2405.4B.

For questions about this memo, please contact Karl Johnson (karl.johnson@state.mn.us or 651-366-4521) or Arielle Ehrlich (arielle.ehrlich@state.mn.us or 651-366-4506).