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Transmittal Notice 2022-02

Date: 6/29/2022

Distribution: MnDOT Bridge Office Web Site

Issued by: MnDOT Bridge Office

Subject: MnDOT LRFD Bridge Design Manual Update

The MnDOT Bridge Office LRFD Bridge Design Manual (BDM) is available for download in Adobe PDF at:
http://www.dot.state.mn.us/bridge/
Check this Web site regularly for updates or to sign up for email update alerts.

Instructions (for two-sided printing):

1. Remove from the BDM:
    - Title Page
    - Section 2: Pages 2-7 & 2-8, 2-13 through 2-16, 2-59 through 2-62, 2-81 & 2-82, 2-105 & 2-106
    - Section 5: Pages 5-1 through 5-4, 5-31 & 5-32, 5-73 through 5-76, 5-93 through 5-126
    - Section 9: Pages 9-1 through 9-10
2. Print and insert in the BDM:
    - Title Page
    - Section 2: Pages 2-7a through 2-8b, 2-13 through 2-16, 2-59 through 2-62, 2-81 & 2-82, 2-105 & 2-106
    - Section 5: Pages 5-1 through 5-4, 5-31 & 5-32, 5-73 through 5-76, 5-93 through 5-126b
    - Section 9: Pages 9-1 through 9-10

BDM Update Summary

Revisions in the "JUNE 2022" update consist of the following changes:

Section 2 (LOOK FOR **3** VERTICAL BARS IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 2.1.2: Under "**Sidepaths (Shared-Use Paths) and Sidewalks on Bridges"**, lowered design speed at which barrier separation is required between roadway and sidepath/sidewalk from 50 mph to 45 mph. Also lowered design speed at which barrier separation needs consideration from 45 mph to 40 mph and revised bullets describing the context considerations.
- Article 2.1.3: Revised Table 2.1.3.1 to clarify minimum vertical clearances for pedestrian bridges and sign bridges. Also revised guidance for portal clearances for clarity and railroad requirements.
- Article 2.4.2.2: Under **Pay Quantities**, added sentence to clarify that computation of information quantities applies to both new bridge projects and existing bridge repair projects.

- Article 2.5.1.3: Revised Figure 2.5.1.3.1 to include a saw cut and seal in the new wearing course at the back of the abutment end block.
- Appendix 2-C: Under "**B. CONSTRUCTION NOTES**", deleted note that tells contractor pile loads were computed using LRFD methodology and directs them to special provisions. The special provision language is now in the MnDOT construction spec and use of LRFD methodology for piling has been normal practice for many years, so the note is no longer needed.

Section 5 (LOOK FOR **2** VERTICAL BARS IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 5.1.1: In Table 5.1.1, changed concrete mix designation for moment slabs from 3S52 to 3B52. Also revised design compressive strength limits for pretensioned superstructures to match values given in BDM Articles 5.4.3 and 5.4.6.
- Article 5.4.3: In second bullet prior to Figure 5.4.3.2, added sentence clarifying that not all bottom rows must be entirely filled with strands for an efficient design. Also, in first bullet prior to Figure 5.4.3.2, clarified instructions to place first straight strand in column immediately outside of the stirrup.
- Article 5.7.1: In ***L. Design Negative Moment Reinforcement***, corrected error in **Fatigue** regarding incorrect sign convention for fatigue check of top longitudinal bars.
- Article 5.7.2: The entire prestressed beam design example has been updated. The updated example includes changes to cross-section dimensions, span length, beam type, barrier type, beam initial & final concrete strength, prestress strand strength, and changes as needed to bring the design into conformance with the 2020 AASHTO Bridge Design Specifications.

Section 9 (LOOK FOR **3** VERTICAL BARS IN THE RIGHT MARGIN TO LOCATE THE CHANGES NOTED BELOW.)

- Article 9.1.1: In **Superstructure Drains**, revised guidance on deck drain outlets, extension below superstructure, and splash blocks.
- Article 9.2.1: In **Design**, revised guidance for crack control check of top transverse bars to assume 0.5 inches of wear for calculation of overall deck thickness, h.

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

Edward A. Lutgen

Digitally signed by Edward 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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JULY 2003 OCTOBER 2003 JANUARY 2004 APRIL 2004 OCTOBER 2004 DECEMBER 2004
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JUNE 2020 JANUARY 2021 APRIL 2021 OCTOBER 2021 FEBRUARY 2022 JUNE 2022

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.

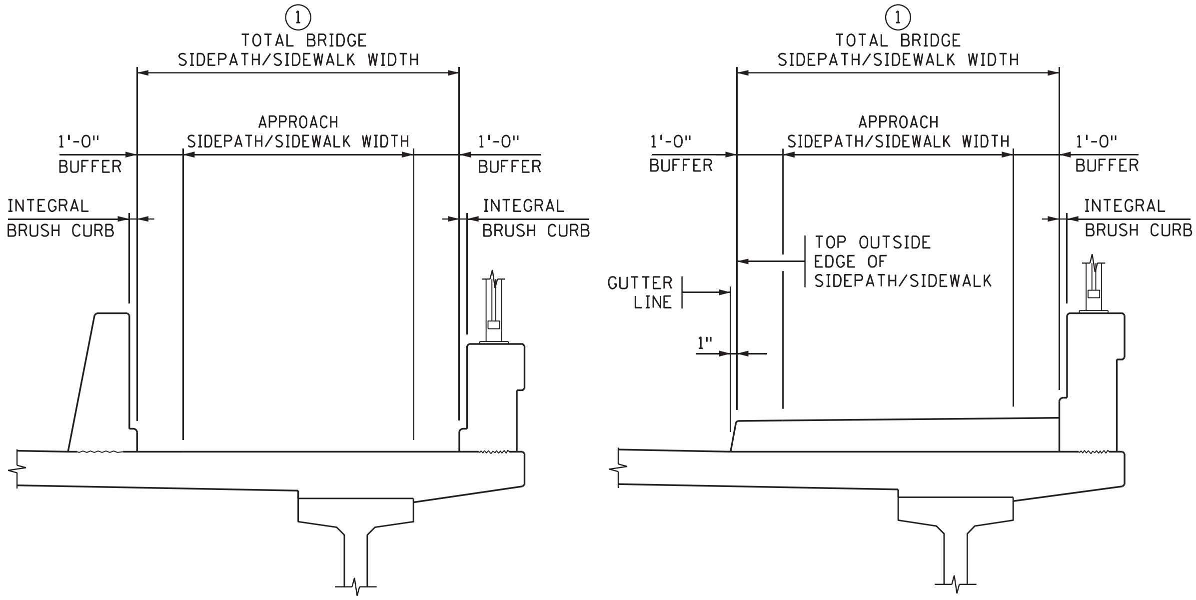
AASHTO defines high speed as design speeds 50 mph and greater, and low speed as design speeds 45 mph and less. Safety for both vehicles and pedestrians/bicycles must be considered when determining separation requirements on a bridge. A separation railing between the roadway and sidepath/sidewalk can 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.

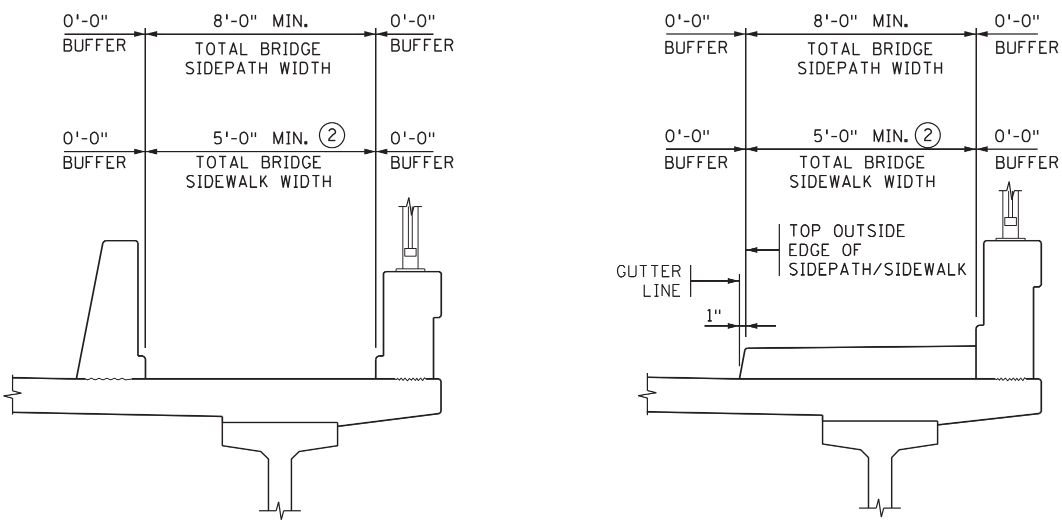


SIDEPATH/SIDEWALK WITH TRAFFIC BARRIER

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



SIDEPATH/SIDEWALK WITH TRAFFIC BARRIER

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

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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.

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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.

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.

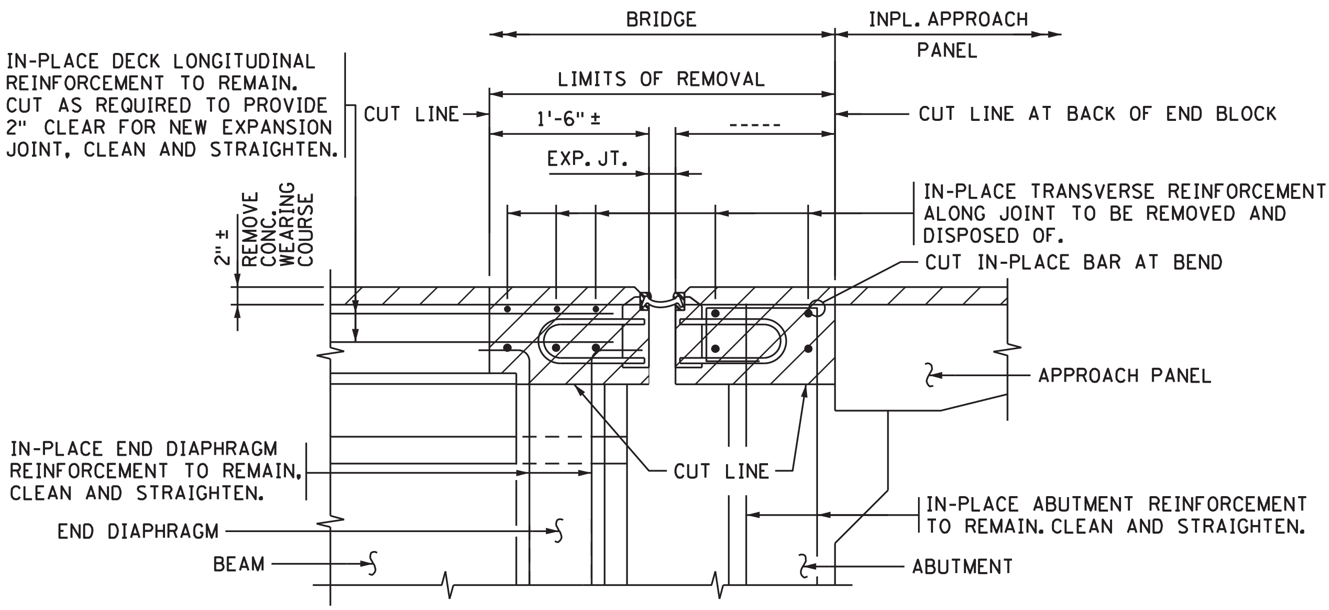
For complex projects, consider creating a separate plan sheet for pay items and notes for clarity.

Utilities

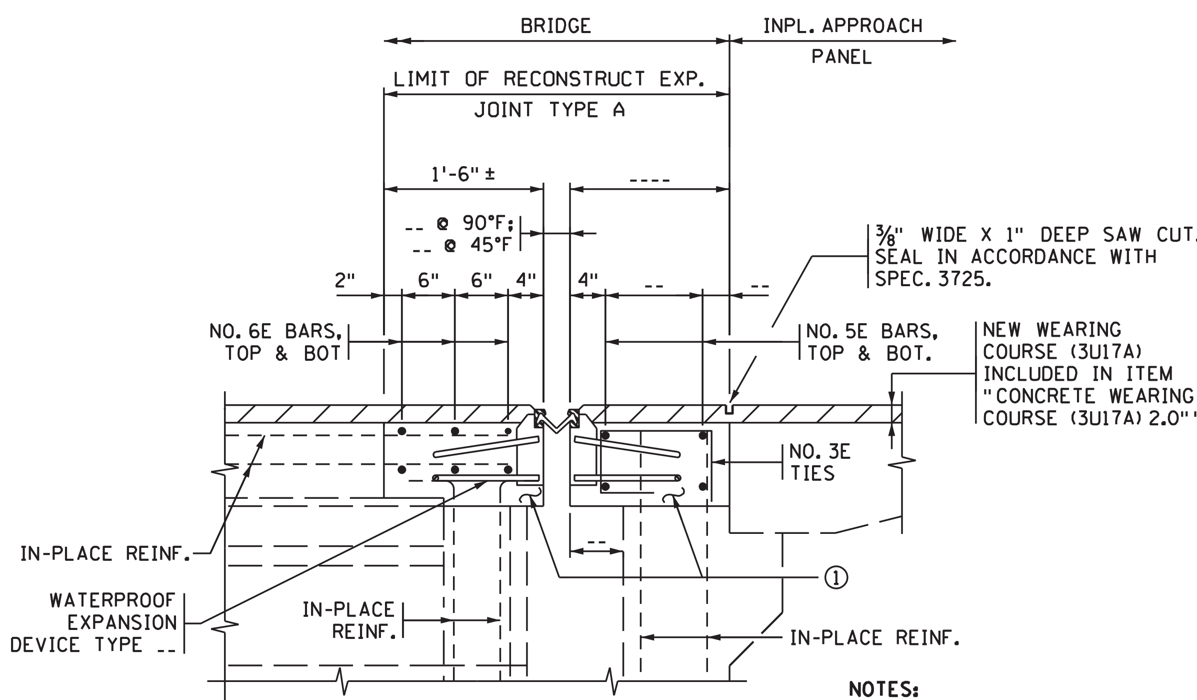
Show all utilities that may affect bridge construction. Note what is to be done with them (will they be moved, will they no longer be used or do they need to be protected during construction).

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.



IN-PLACE SECTION



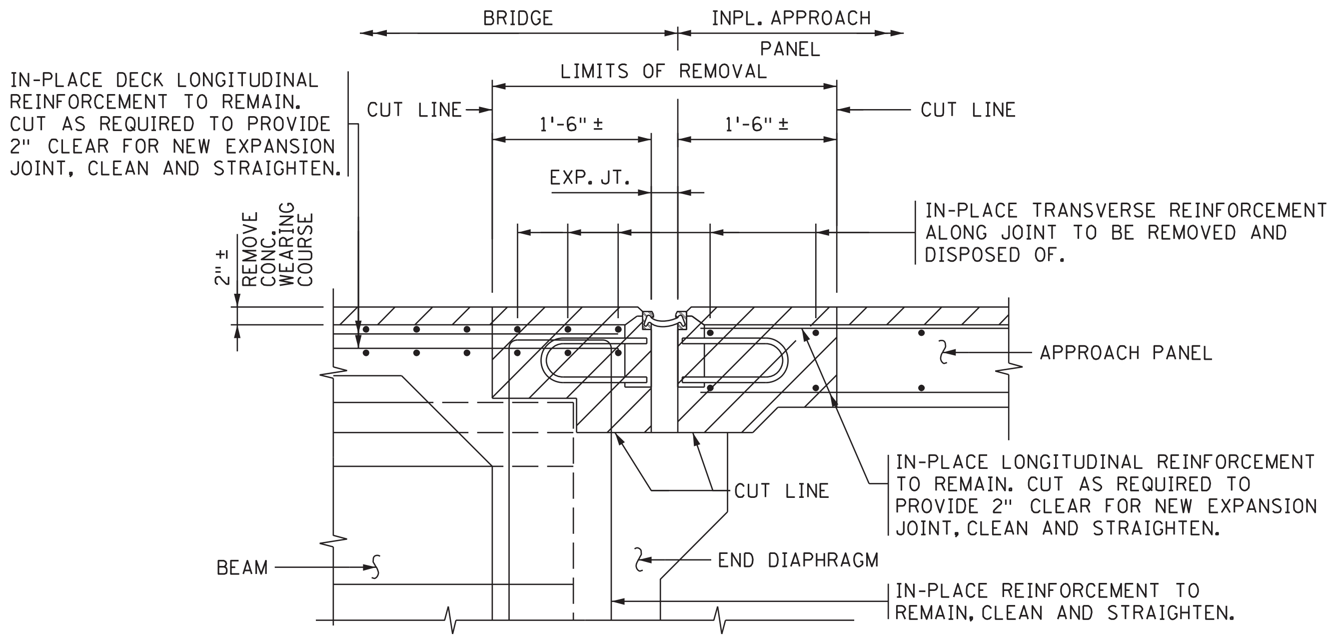
RECONSTRUCTED SECTION

NOTES:

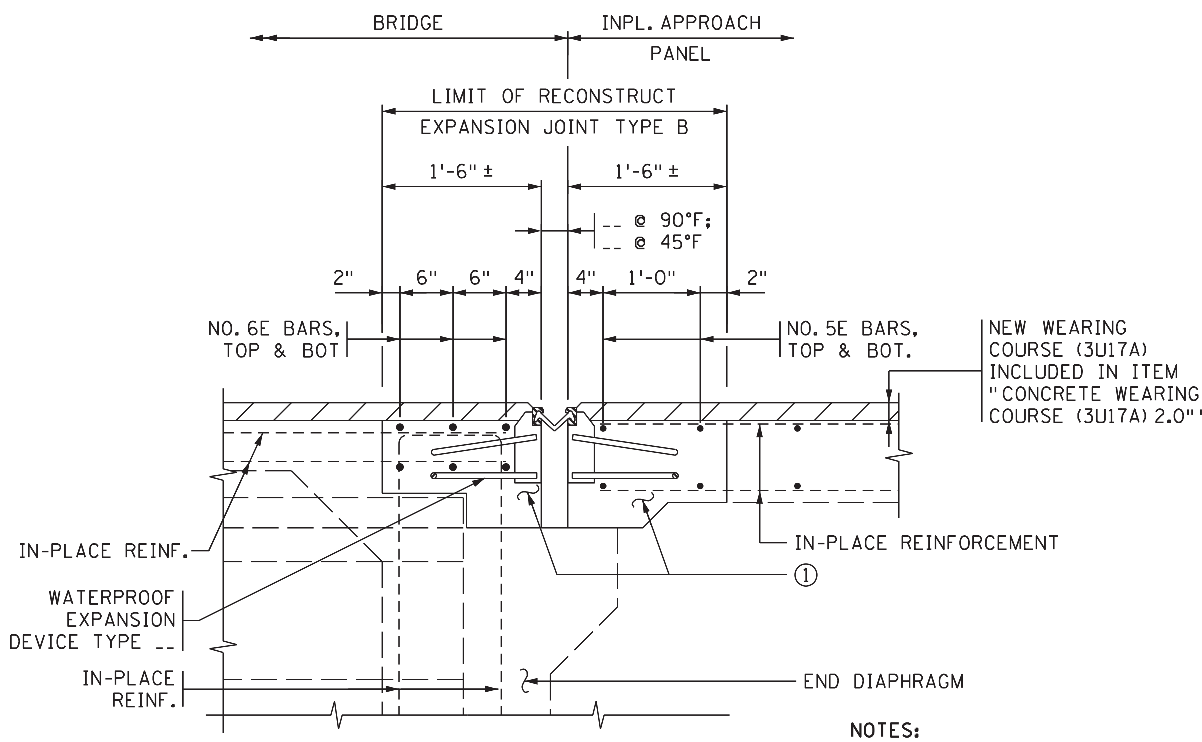
① USE HPC MIX IF ALREADY IN THE REPAIR PROJECT. OTHERWISE, USE 3Y47 CONCRETE MIX.

RECONSTRUCT EXPANSION JOINT TYPE A

Figure 2.5.1.3.1
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 B

Figure 2.5.1.3.2
Expansion Joints

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.]

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

[Use on all projects.]

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

[Use as required.]

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. THE CONTRACTOR'S ENGINEER SHALL DESIGN, AND THE CONTRACTOR SHALL CONSTRUCT A TEMPORARY BRACING SYSTEM AND/OR A DECK FALSEWORK/FORMWORK SYSTEM. THE SYSTEM SHALL PROVIDE 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.]

CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED 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. RESTORATION COSTS SHALL BE INCLUDED IN PRICE BID FOR STRUCTURE EXCAVATION.

[Use this note on railroad underpasses.]

________________ PIPE TO BE PLACED 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.]

5. CONCRETE STRUCTURES

Reinforced and prestressed concrete are used extensively in bridge projects. In addition to general design guidance and information on detailing practices, this section contains three design examples: a three-span reinforced concrete slab superstructure, a 63 inch pretensioned I-beam, and a three-span post-tensioned concrete slab superstructure.

5.1 Materials

For most projects, conventional materials should be specified. Standard materials are described in two locations: *MnDOT Standard Specifications for Construction* (MnDOT Spec.) and *Bridge Special Provisions*.

If multiple types of concrete or reinforcement are to be used in a project, it is the designer's responsibility to clearly show on the plans the amount of each material to be provided and where it is to be placed.

5.1.1 Concrete

MnDOT Specs. 2461 (cast-in-place) and 2462 (precast) identify and describe concrete mix types used by MnDOT. Based on their properties, different mixes are used for the various structural concrete components. Table 5.1.1.1 identifies the standard MnDOT concrete mix types to be used for different bridge components.

Four characters are typically used to identify a concrete mix. The first character designates the type of concrete (based on air entrainment requirements). The second character identifies the grade of concrete based on multiple characteristics (intended use, max. w/c ratio, max. cementitious content, max. supplementary cementitious materials, min. concrete strength, etc). The third character is the upper limit for the slump in inches. The fourth character identifies the coarse aggregate gradation. There are some exceptions to the above: MnDOT designed mixes (such as 3U17A low slump concrete), job mixes (JM) for box girders, and high performance concrete (HPC) mixes for bridge decks and slabs.

For HPC mixes, the first and second characters follow the description above. For monolithically poured decks, these are followed by either "HPC-M" or "LCHPC-M" (where the LC designates low cement). For decks that will receive a separate wearing course, these are followed by either "HPC-S" or "LCHPC-S" (where the LC designates low cement). For job mixes, the first character designates the type of concrete as above, but is followed by "JM" for mixes that will be determined by the Contractor.

In general, the standard concrete design strength is 4 ksi, and air entrained concretes are to be used for components located above footings and pile caps to enhance durability.

Table 5.1.1.1 Design Concrete Mix Summary

Location/Element	MnDOT Concrete Mix Designation	Design Compressive Strength (ksi)	Maximum Aggregate Size (in)
Cofferdam seals	1X62	5.0	1
Cast-in-place concrete piles and spread footing leveling pads	1P62	3.0	2
Drilled shafts and rock sockets	1X62 3X62	5.0 5.0	1 1
Footings and pile caps, except not for partially exposed abutment footings behind MSE walls.	1G52	4.0	1 ½ *
Abutment stems, wingwalls, pier columns, pier struts, pier caps, and moment slabs. Also includes partially exposed abutment footings behind MSE walls.	3B52	4.0	1 ½ *
Integral abutment diaphragms and pier continuity diaphragms	Same mix as used in deck	4.0	1
Pretensioned superstructures	1W82 or 3W82	5.0 – 9.5 at final 4.5 – 8.0 at initial	1
Cast-in-place and precast box girders	3JM	6.0 or higher	1
Monolithic decks and slabs	3YHPC-M, 3YLCHPC-M or 3Y47-M	4.0	1
Decks and slabs that will receive a 2 inch concrete wearing course	3YHPC-S, 3YLCHPC-S or 3Y47-S	4.0	1
Barriers, parapets, medians, sidewalks, and approach panels	3S52	4.0	1
Concrete wearing course	3U17A	4.0	$^5/_8$
MSE wall panels, PMBW blocks, and noise wall panels	3Y82	4.0	1
Cast-in-place wall stems	3G52	4.5	1 ½ *
Precast box culverts, arches, and 3-sided structures	3W82	5.0 or higher	1*

* For determination of s_{xe} per LRFD 5.7.3.4.2, use max aggregate size a_g = ¾”

Reinforced Concrete Sections

Base concrete modulus of elasticity computations on a unit weight of 0.145 kcf. Use a unit weight of 0.150 kcf for dead load calculations.

For structural modeling (determining design forces and deflections), use gross section properties or effective section properties. For redundant structures with redundant and nonprismatic members, model with nonprismatic elements.

[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.

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:

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

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

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 5.1.1.2 summarizes concrete properties for analysis and design:

Table 5.1.1.2
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 $f'_c \leq 6$ 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.

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 preferred for economic and safety reasons (i.e. – a design with more total strands consisting of many straight and few draped strands is more economical than a design with less total strands that has fewer straight and a higher number of draped strands).
- 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.

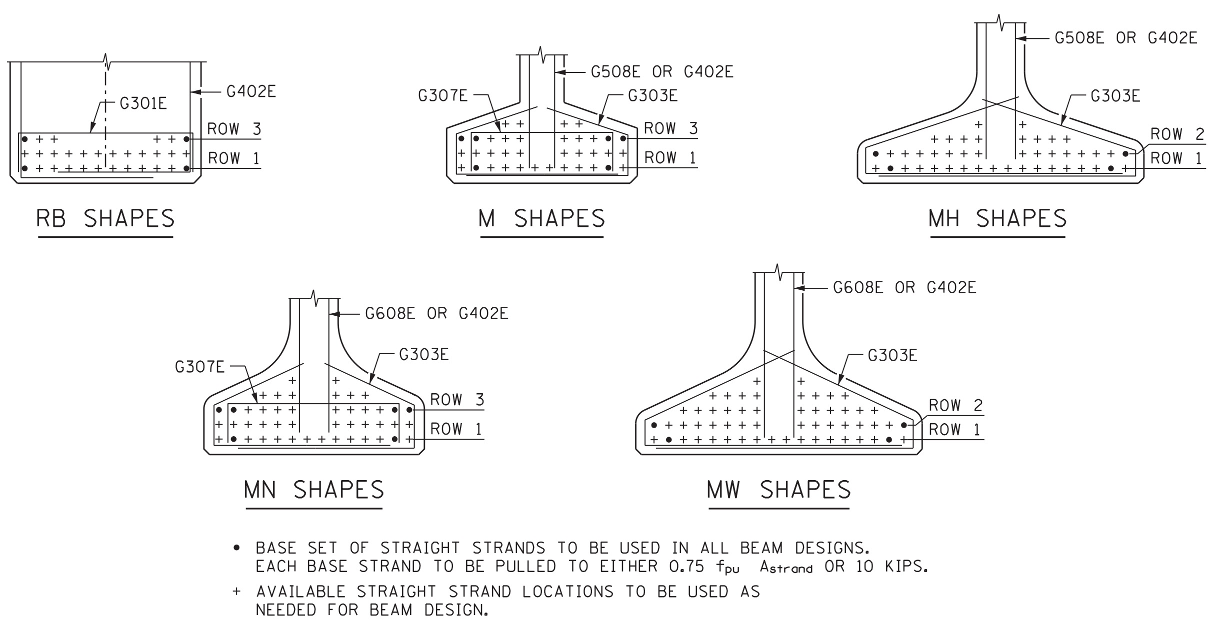


Figure 5.4.3.1

- 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.

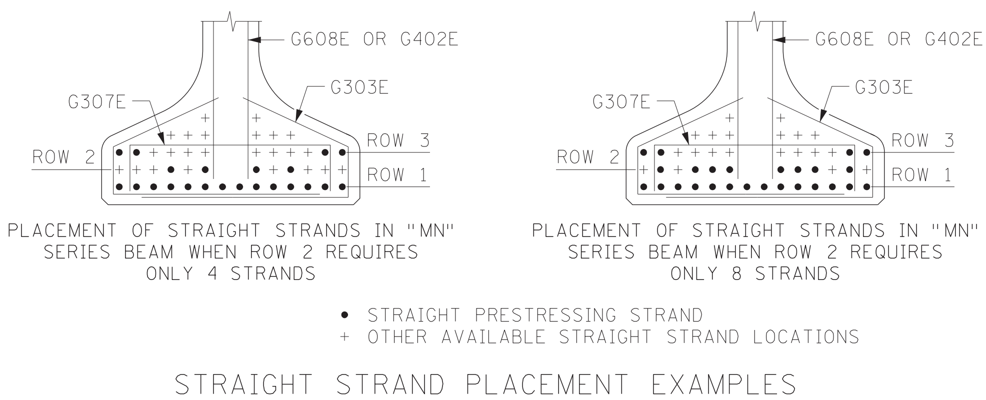


Figure 5.4.3.2

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.

Compute the actual stress in the reinforcement.

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d_s} = \frac{97 \cdot 12}{1.90 \cdot (16.67)} = 36.8 \text{ ksi} > 36.0 \text{ ksi} \qquad \text{NO GOOD}$$

Revise Interior Strip Top Longitudinal Bars

At this point, we can increase the bar size, change the bar spacing, or both. In order to keep the spacings consistent, try:

#9 bars @ 5" spacing, $A_s = 2.40 \text{ in}^2/\text{ft}$

Recalculate flexural resistance:

$d_s = 18.44$ in

$a = 3.53$ in

$M_r = 180.1 \text{ k-ft} > 144 \text{ k-ft}$ OK

$c = 4.15$ in

$\varepsilon_t = 0.0103 > 0.005$

Recalculate crack control checks:

$x = 6.01$ in

$j \cdot d_s = 16.44$ in

$f_{ss} = 29.5 \text{ ksi} < 36.0 \text{ ksi}$ OK

Actual cover $d_{covtop} = 3.0$ in

[BDM 5.3.2] For the calculation of d_c, use a maximum clear cover $d_{covtopcc}$ equal to 2.0 inches.

Then $d_c = d_{covtopcc} + 0.5 \cdot d_b = 2.0 + 0.5 \cdot 1.128 = 2.56$ in

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - h_{wear} - d_c)} = 1 + \frac{2.56}{0.7 \cdot (22 - 0.5 - 2.56)} = 1.19$$

Use $\gamma_e = 0.75$:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.19 \cdot 29.5} - 2 \cdot 2.56 = 9.8 \text{ in} > 5 \text{ in} \qquad \text{OK}$$

Similarly, for the exterior strip:

$x = 6.60$ in

$j \cdot d_s = 16.17$ in

$f_{ss} = 29.7 \text{ ksi} < 36.0 \text{ ksi}$ OK

$d_c = 2.64$ in

$\beta_s = 1.19$

$s_{max} = 9.6 \text{ in} > 5 \text{ in}$ OK

[5.5.3] **Fatigue**

The stress range in the reinforcement is computed and compared against code limits to ensure adequate fatigue resistance is provided.

[Table 3.4.1-1] Fatigue I: $U = 1.0 \cdot 1.75\,(LL + IM)$

[3.6.2.1] The dynamic load allowance for fatigue, IM = 15%

At Span Point 2.0 the one lane fatigue moments are:

Maximum positive moment = 33 kip-ft

Maximum negative moment = -252 kip-ft

Multiply the one lane moments by the dynamic load allowance, and the fatigue live load distribution factor to get the fatigue moments for a 1 foot wide strip. Note that negative moment causes tension in the top bars, but the sign convention for fatigue checks is + for tension and – for compression, so switch the signs on the moments to do this calculation:

Fatigue LL $M_{fatmin} = -(33) \cdot 1.15 \cdot 0.058 = -2.2$ kip-ft

Fatigue LL $M_{fatmax} = -(-252) \cdot 1.15 \cdot 0.058 = 16.8$ kip-ft

[5.5.3.1] AASHTO allows the use of uncracked section properties where the sum of the unfactored dead load stress and Fatigue I stress is less than $0.095\sqrt{f'_c}$ in tension. Conservatively assume the section is cracked.

Then using the equation previously used to check crack control stresses, determine the fatigue live load stress range Δf:

$$\Delta f = \frac{M_{fatmax} - M_{fatmin}}{A_s \cdot j \cdot d_s} = \frac{[16.8 - (-2.2)] \cdot 12}{2.40 \cdot 16.44} = 5.8 \text{ ksi}$$

For the Fatigue I load combination, the factored live load stress is:

$\gamma \cdot \Delta f = 1.75 \cdot 5.8 = 10.2$ ksi

[5.5.3.2] Now calculate the constant-amplitude fatigue threshold $(\Delta F)_{TH}$. The unfactored dead load moment at Span Point 2.0 is –52.4 kip-ft. Again, we will switch the sign on M_{DC} to match the sign convention for fatigue checks (+ for tension, - for compression):

$M_{DC} = -(-52.4) = 52.4$ kip-ft

$$f_{min} = \frac{\gamma \cdot M_{fatmin} + M_{DC}}{A_s \cdot j \cdot d_s} = \frac{[1.75 \cdot (-2.2) + 52.4] \cdot 12}{2.40 \cdot 16.44} = 14.8 \text{ ksi}$$

$$(\Delta F)_{TH} = 26 - \frac{22 \cdot f_{min}}{f_y} = 26 - \frac{22 \cdot (14.8)}{60} = 20.6 \text{ ksi} > 10.2 \text{ ksi} \quad \text{OK}$$

[5.6.3.3]

Check Minimum Reinforcement

To prevent a brittle failure, adequate flexural reinforcement needs to be placed in the cross section. For this check, zero wear is conservatively assumed.

Check that the reinforcement can carry the smaller of:

- Cracking moment, M_{cr}
- $1.33 \cdot M_u$

At Span Point 2.0:

$1.33 \cdot M_u = 1.33 \cdot 144 = 191.5$ kip-ft

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{4.0} = 0.48 \text{ ksi}$$

$$y_t = \frac{h}{2} = \frac{22.0}{2} = 11.0 \text{ in}$$

$$I_g = \frac{1}{12} \cdot b \cdot h^3 = \frac{1}{12} \cdot 12 \cdot (22)^3 = 10{,}648 \text{ in}^4$$

$$S_c = \frac{I_g}{y_t} = \frac{10648}{11.0} = 968 \text{ in}^3$$

For non-precast segmental structures, $\gamma_1 = 1.6$

MnDOT uses AASHTO M31 (ASTM A615) Grade 60 reinforcement in concrete bridge structures, so $\gamma_3 = 0.67$

$$M_{cr} = \gamma_3 \cdot (\gamma_1 \cdot f_r \cdot S_c) = \frac{0.67 \cdot (1.6 \cdot 0.48 \cdot 968)}{12} = 41.5 \text{ kip-ft} \quad \text{GOVERNS}$$

$$M_r = \phi \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

$$M_r = 0.9 \cdot (2.40) \cdot (60) \cdot \left(18.44 - \frac{3.53}{2}\right) \cdot \frac{1}{12}$$

$$= 180.1 \text{ kip-ft} > M_{cr} = 41.5 \text{ kip-ft} \qquad \text{OK}$$

Use #9 bars at 5 inches at Span Point 2.0

Similarly for the exterior strip:

At Span Point 2.0, $1.33 \cdot M_u = 240.7$ kip-ft

$M_{cr} = 41.5$ kip-ft

$M_r = 221.3$ kip-ft > 41.5 kip-ft

Use #10 bars at 5 inches at Span Point 2.0

[5.10.8.1.2a]
[5.10.8.1.2c]

Bar Cutoff Location

Determine the location where the 5 inch spacing can be increased to 10 inches. The moment capacity of #9 bars at 10 inches ($A_s = 1.20\ in^2$) for negative flexure is:

$$M_r = \phi \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

$$= 0.9 \cdot (1.20) \cdot (60) \cdot \left[18.44 - \frac{1.20 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)}\right] \cdot \frac{1}{12}$$

$$= 94.8 \text{ kip-ft}$$

For the interior strip, the negative bending moments are:

Span Point	$M_{Strength\ I}$ (kip-ft)/ft	$M_{Service\ I}$ (kip-ft)/ft
1.8	-65	-40
1.9	-98	-64
2.0	-144	-97
2.1	-79	-52
2.2	-41	-24

Knowing that span points are 3.6 feet apart in Span 1 and 4.5 feet apart in Span 2, the drop point locations which meet the Strength I negative bending moment of 94.8 kip-ft can be found.

For Span 1, interpolate between Span Points 1.8 and 1.9:

$$1.8 + \left(\frac{94.8 - 65}{98 - 65}\right) \cdot 0.1 = 1.89 \text{ or } 3.96 \text{ ft from Pier 1 centerline.}$$

For Span 2, interpolate between Span Points 2.0 and 2.1:

$$2.0 + \left(\frac{144 - 94.8}{144 - 79}\right) \cdot 0.1 = 2.08 \text{ or } 3.60 \text{ ft from Pier 1 centerline.}$$

The reinforcement must also meet the serviceability requirements at the theoretical drop point. Determine the drop point location based on the crack control requirements and compare with the drop points based on strength to see which ones govern.

[5.6.7]

For #9 bars @ 10", ($A_s = 1.20\ in^2$), $d_c = 2.56$ in:

5.7.2 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.

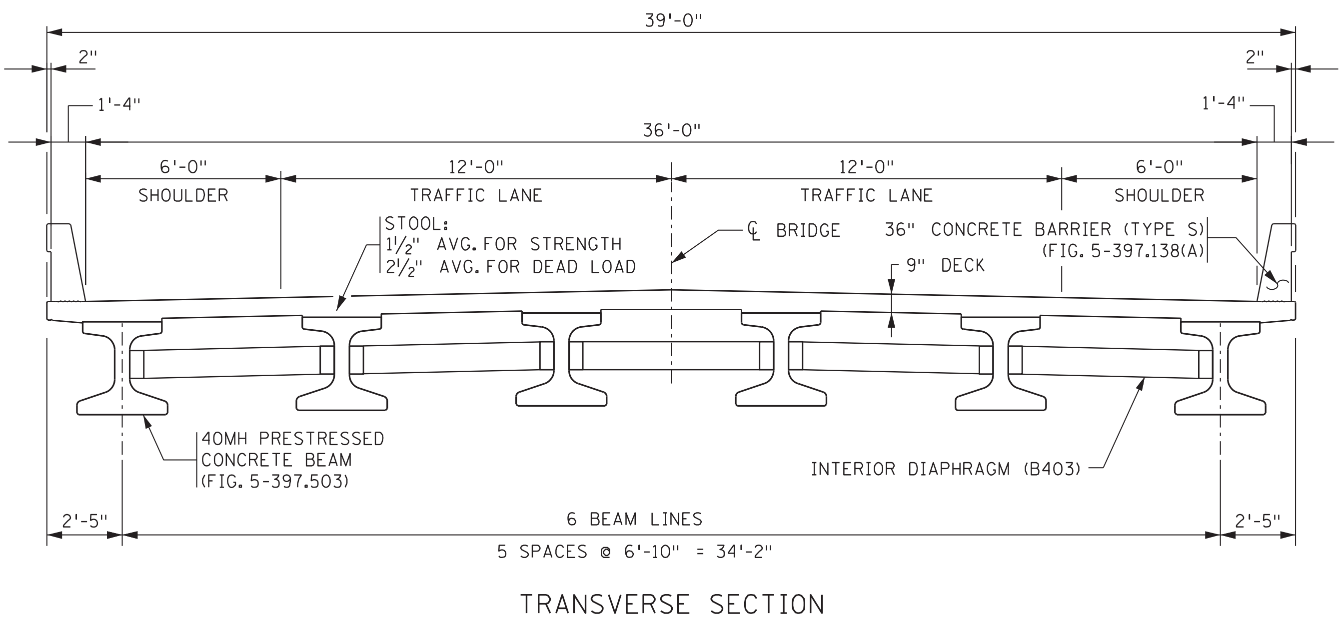
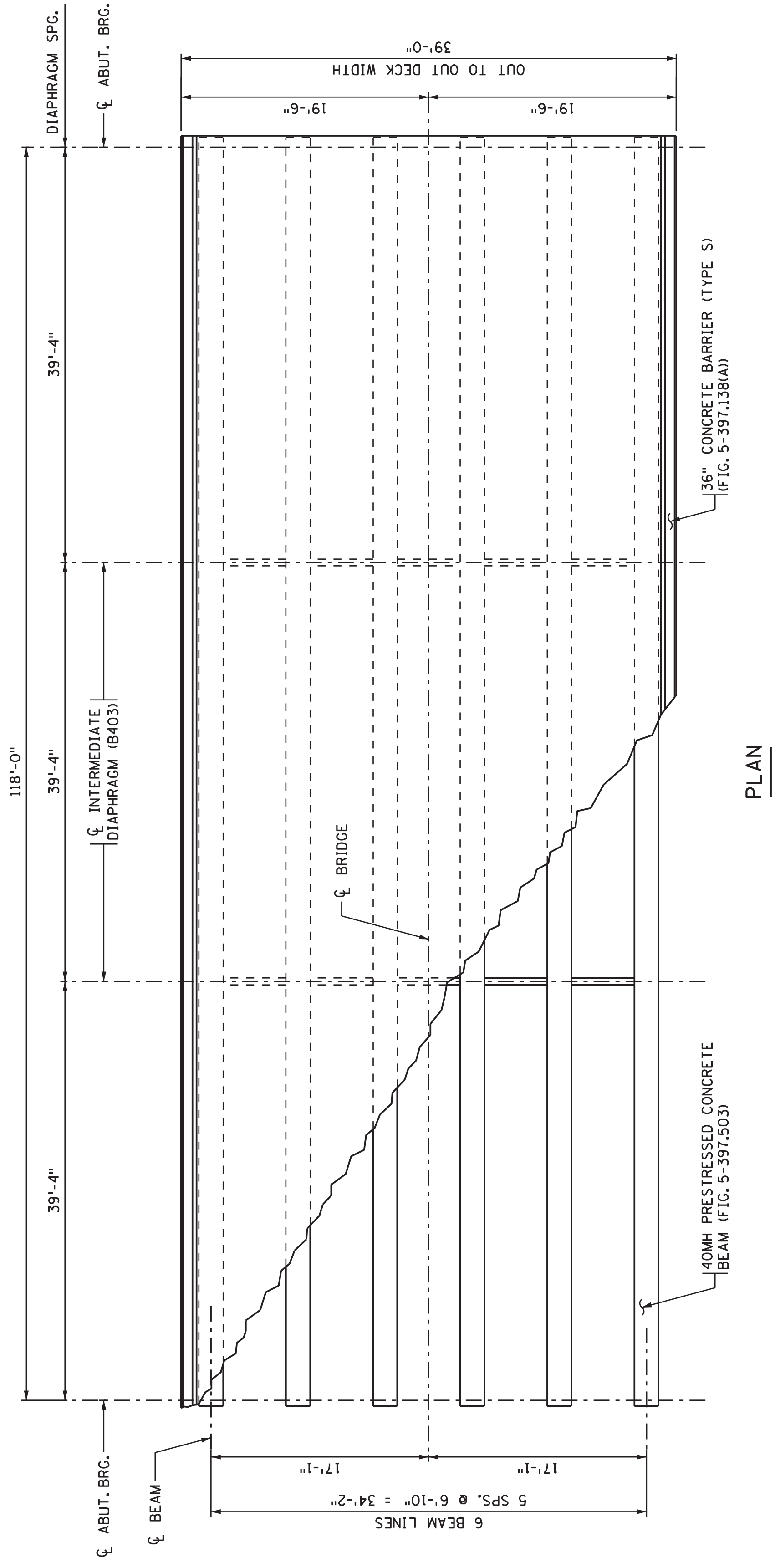


Figure 5.7.2.1



PLAN
39'-0"
OUT TO OUT DECK WIDTH
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)
℄ ABUT. BRG.
℄ BEAM
17'-1"
17'-1"
5 SPS. @ 6'-10" = 34'-2"
6 BEAM LINES

Figure 5.7.2.2

A. Materials

The modulus of elasticity E_c 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 5.7.2.1:

Table 5.7.2.1
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 5.7.2.2.

Table 5.7.2.2
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 = (deck centroid) – (beam centroid) = 45.75 – 18.07 = 27.68 in

$$K_g = n_{bm\_d} \cdot \left[I + A \cdot (e_g)^2\right] = 1.23 \cdot \left[149{,}002 + 704 \cdot (27.68)^2\right]$$

$$= 8.47 \times 10^5 \text{ in}^4$$

Check K_g limits: $1 \times 10^4 \le K_g = 8.47 \times 10^5 \le 7 \times 10^6$ OK

For one design lane loaded, the live load distribution factor for moment, gM_{int_1lane}, is:

$$gM_{int\_1lane} = 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12 \cdot L \cdot t_s^3}\right)^{0.1}$$

$$gM_{int\_1lane} = 0.06 + \left(\frac{6.83}{14}\right)^{0.4} \cdot \left(\frac{6.83}{118}\right)^{0.3} \cdot \left(\frac{8.47 \times 10^5}{12 \cdot 118 \cdot 8.5^3}\right)^{0.1}$$

gM_{int_1lane} = 0.378 lanes/beam

Two or more design lanes loaded:

$$gM_{int\_mlane} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12 \cdot L \cdot t_s^3}\right)^{0.1}$$

$$gM_{int\_mlane} = 0.075 + \left(\frac{6.83}{9.5}\right)^{0.6} \cdot \left(\frac{6.83}{118}\right)^{0.2} \cdot \left(\frac{8.47 \times 10^5}{12 \cdot 118 \cdot 8.5^3}\right)^{0.1}$$

gM_{int_mlane} = 0.538 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 = 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:

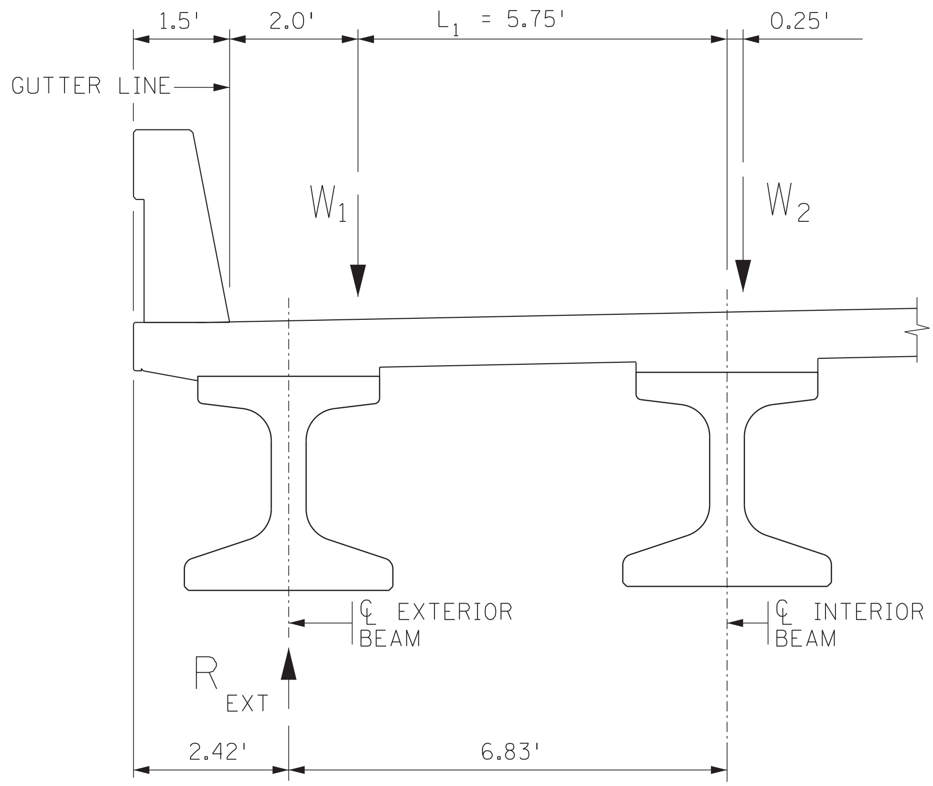


Figure 5.7.2.3

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:

[Table 3.6.1.1.2-1]

$W_1 = W_2 = 0.5$ 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]
[4.6.2.2.3a]

Live Load Distribution Factor for Shear – Interior Beams

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, which is 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]
[Table 3.6.1.1.2-1]

Live Load Distribution Factor for Deflection

The live load distribution factor for checking live load deflections assumes that the entire cross section participates in resisting the live load. The live load distribution factor for deflection 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 5.7.2.3 contains a summary of the live load distribution factors and Table 5.7.2.4 contains a summary of the load modifiers for this example.

Table 5.7.2.3
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 5.7.2.4 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 point, 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)

$$= X_{brgface} = \frac{L_{soleplate}}{2} = 7.5\ \text{in} = 0.63\ \text{ft}$$

Transfer Point

$$= X_{transfer} = 60 \cdot d_b - \frac{L_{soleplate}}{2} = (60 \cdot 0.6) - \frac{15}{2} = 28.5\ \text{in} = 2.38\ \text{ft}$$

Critical Shear Point (located at d_v from the inside face of bearing, calculations are shown in “F. Design Reinforcement for Shear”)

$$= X_{vcrit} = 4.07\ \text{ft}$$

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 5.7.2.4 and 5.7.2.5.

Table 5.7.2.4
Shear Force Summary (kips/beam)

Load Type/Combination		Brg CL (0.0')	Brg Face (0.63')	Trans Point (2.38')	Critical Shear Point (4.1')	0.1 Span Point (11.8')	0.2 Span Point (23.6')	0.3 Span Point (35.4')	0.4 Span Point② (47.2')	0.5 Span Point (59.0')
Dead Loads	Selfweight	45	44	43	42	36	27	18	9	0
	Stool	5	5	5	5	4	3	2	1	0
	Deck	45	45	44	42	36	27	18	9	0
	FWS	7	7	7	7	6	4	3	1	0
	Barrier	10	10	9	9	8	6	4	2	0
	Diaphragms	0	0	0	0	0	0	0	0	0
	Total	112	111	108	105	90	67	45	22	0
Live Loads ①	Uniform Lane	28	27	27	26	22	18	14	10	7
	Tandem + IM	48	48	47	46	43	38	33	28	23
	Truck + IM	64	64	63	62	57	50	43	36	29
	Governing LL Total (Truck + IM) + Lane	92	91	90	88	79	68	57	46	36
Strength I Load Comb ($1.25 \cdot DL + 1.75 \cdot LL$)		301	298	293	285	251	203	156	108	63
Service I Load Comb ($1.00 \cdot DL + 1.00 \cdot LL$)		204	202	198	193	169	135	102	68	36
Service III Load Comb ($1.00 \cdot DL + 0.80 \cdot LL$)		186	184	180	175	153	121	91	59	29

① All live loads include the interior beam live load distribution factor of 0.731 and IM of 0.33.
② Hold down point for draped strands.

Table 5.7.2.5
Bending Moment Summary (kip-ft/beam)

Load Type/Combination			Brg CL (0.0')	Brg Face (0.63')	Trans Point (2.38')	Critical Shear Point (4.1')	0.1 Span Point (11.8')	0.2 Span Point (23.6')	0.3 Span Point (35.4')	0.4 Span Point (47.2') ②	0.5 Span Point (59.0')
Dead Loads	DC1	Selfweight	0	28	104③	177	475	844	1108	1267④	1319
		Stool	0	3	12	21	55	98	129	147	153
		Deck	0	28	106	180	482	857	1124	1285	1338
		Diaphragms	0	0	0	1	2	4	6	7	7
		Total DC1	0	59	222	379	1014	1803	2367	2706	2817
	DC2	Barrier	0	6	23	39	103	184	241	276	287
		FWS	0	4	16	28	75	134	175	201	209
		Total DC2	0	10	39	67	178	318	416	477	496
	Total (DC1+DC2)		0	69	261	446	1192	2121	2783	3183	3313
Live Loads ①	Uniform Lane		0	13	47	80	216	384	503	575	599
	Tandem + IM		0	22	82	139	373	661	865	985	1020
	Truck + IM		0	29	110	187	499	877	1132	1283	1319
	Governing LL Total (Truck + IM) + Lane		0	42	157	267	715	1261	1635	1858	1918⑤
Strength I - Load Comb (1.25 · DL + 1.75 · LL)			0	160	601	1025	2741	4858	6340	7230	7498
Service I - Load Comb (1.00 · DL + 1.00 · LL)			0	111	418	713	1907	3382	4418	5041	5231
Service III – Load Comb (1.00 · DL + 0.80 · LL)			0	103	387	660	1764	3130	4091	4669	4847
Fatigue I – Load Comb (1.75 · LL)			-	-	-	-	-	-	-	-	1008

① All live loads include the interior beam live load distribution factor of 0.538 and IM of 0.33.

② Hold down point for draped strands.

③ Beam selfweight at strand release = 132 k-ft (beam in casting bed with span length equal to overall beam length of 119.25 ft).

Beam selfweight at erection on bearings = 104 k-ft (beam span length equal to design span of 118.0 ft).

④ Beam selfweight at strand release = 1295 k-ft (beam in casting bed with span length equal to overall beam length of 119.25 ft).

Beam selfweight at erection on bearings = 1267 k-ft (beam span length equal to design span of 118.0 ft).

⑤ Fatigue live load = 576 k-ft (includes interior beam live load distribution factor of 0.315 and IM of 0.15 applied to fatigue truck only).

E. Design Beam Pretensioning With Draped 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:

$$f_{serv3bot} = \left(\frac{M_{DC1}}{S_{gb}}\right) + \left(\frac{M_{DC2}}{S_{cb}}\right) + \left(\frac{M_{LL} \cdot 0.8}{S_{cb}}\right)$$

$$= \left(\frac{2817 \cdot 12}{8246}\right) + \left(\frac{496 \cdot 12}{12{,}917}\right) + \left(\frac{1918 \cdot 12 \cdot 0.8}{12{,}917}\right) = 5.99 \text{ ksi}$$

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 25%. This results in an effective prestress of

$$f_{pe} = 0.72 \cdot f_{pu} \cdot (1 - 0.25) = 0.72 \cdot 300 \cdot 0.75 = 162.0 \text{ ksi}$$

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 52 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 and include 8 draped strands. The centroid of this 52 strand pattern is:

$$y_{str} = \left[\frac{\sum \text{ (\# of strands)} \cdot \text{(y of strands)}}{\text{(total \# of strands)}}\right]$$

$$= \left[\frac{18 \cdot (2 + 4) + (10 \cdot 6) + (4 \cdot 8) + 2 \cdot (10)}{52}\right] = 4.23 \text{ in}$$

Using the centroid of this group as an estimate of the strand pattern eccentricity results in

$$e_{52} = y_g - y_{str} = 18.07 - 4.23 = 13.84$$

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

$$M_{p/s} = A_s \cdot f_{pe} \cdot e_{52} = n_{strands} \cdot A_{strand} \cdot f_{pe} \cdot e_{52}$$

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:

$$n_{strands} = \frac{f_{pscomp}}{A_{strand} \cdot f_{pe} \cdot \left(\frac{1}{A} + \frac{e_{52}}{S_b}\right)} = \frac{5.40}{0.217 \cdot 162.0 \cdot \left(\frac{1}{704} + \frac{13.84}{8246}\right)}$$

$$= 49.6 \text{ strands}$$

Try a strand pattern with 50 strands.

After reviewing Bridge Details Part II Figure 5-397.503, a trial strand pattern with maximum prestress eccentricity and 50 strands was selected, but calculations showed it did not meet the Service I compression stress limit at the beam ends immediately after strand release. A second trial strand pattern (see Figure 5.7.2.4) was then selected. The drape points were chosen to be at 0.40L = 47.2 ft from the centerline of bearing locations.

The properties of this strand pattern at midspan are:

$$y_{strand} = \left[\frac{(18 \cdot 2) + (16 \cdot 4) + (10 \cdot 6) + (4 \cdot 8) + (2 \cdot 10)}{50}\right] = 4.24 \text{ in}$$

$$e_{strand} = y_b - y_{strand} = 18.07 - 4.24 = 13.83 \text{ in}$$

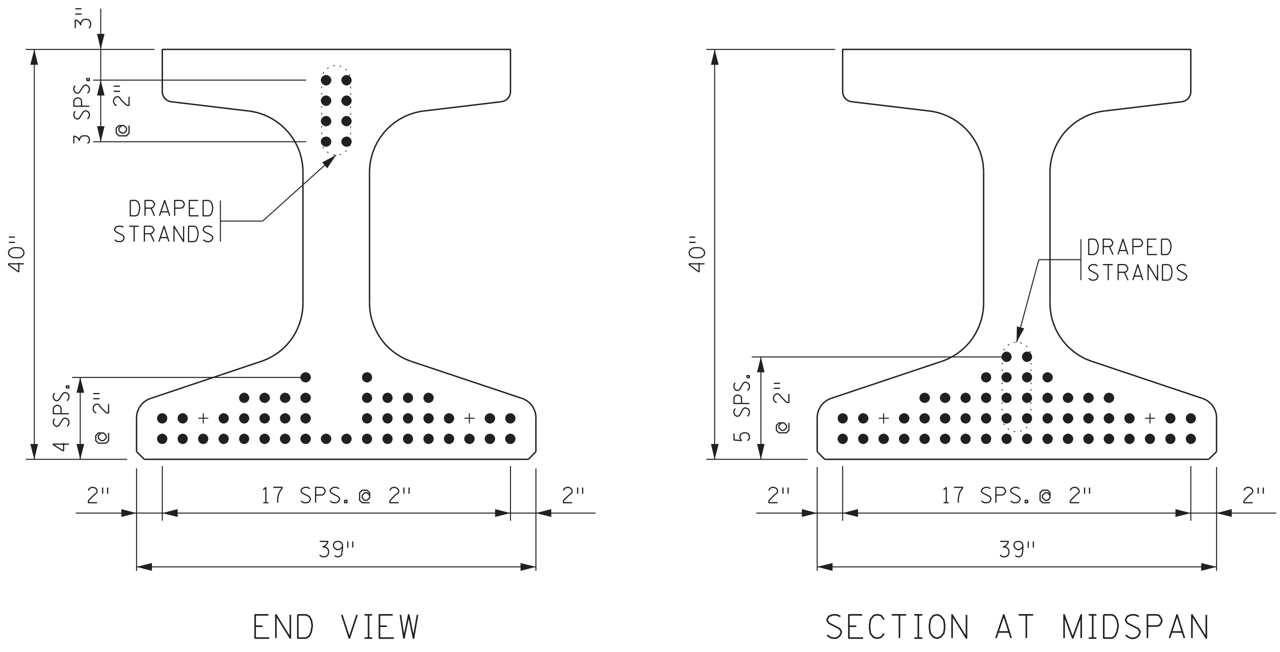


Figure 5.7.2.4

[5.9.3] **2. 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:

$$\Delta f_{pES} = \frac{A_{ps} \cdot f_{pbt} \cdot \left(I_g + e_m{}^2 \cdot A_g\right) - e_m \cdot M_g \cdot A_g}{A_{ps} \cdot \left(I_g + e_m{}^2 \cdot A_g\right) + \dfrac{A_g \cdot I_g \cdot E_{ci}}{E_p}}$$

$$A_{ps} = (\#\text{ of strands}) \cdot (\text{strand area}) = 50 \cdot 0.217 = 10.85\ \text{in}^2$$

$$f_{pbt} = f_{pj} = 216.0\ \text{ksi}$$

$$e_m = e_{strand} = 13.83\ \text{in}$$

$$\frac{A_g \cdot I_g \cdot E_{ci}}{E_p} = \frac{704 \cdot 149{,}002 \cdot 4578}{28{,}500} = 16{,}849{,}836\ \text{in}^6$$

$$A_{ps} \cdot \left(I_g + e_m{}^2 \cdot A_g\right) = 10.85\left[149{,}002 + (13.83)^2 \cdot (704)\right] = 3{,}077{,}660\ \text{in}^6$$

$$\Delta f_{pES} = \frac{216.0 \cdot (3{,}077{,}660) - 13.83 \cdot (1319) \cdot (12) \cdot (704)}{3{,}077{,}660 + 16{,}849{,}836} = 25.6\ \text{ksi}$$

[5.9.3.3] **Long Term Losses**

Use the approximate equation in the LRFD 5.9.3.3

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

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, $\Delta f_{pR} = 2.4$

$$\Delta f_{pLT} = 10.0 \cdot \frac{216.0 \cdot (10.85)}{704} \cdot 0.97(0.56) + 12.0(0.97)(0.56) + 2.4$$

$$= 27.0 \text{ ksi}$$

[5.9.3.1]

Total Losses

$$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pLT} = 25.6 + 27.0 = 52.6 \text{ ksi}$$

$$f_{pe} = f_{pj} - \Delta f_{pt} = 216.0 - 52.6 = 163.4 \text{ ksi}$$

$$\text{prestress loss percentage} = \frac{\Delta f_{pt}}{f_{pj}} \cdot 100 = \frac{52.6}{216.0} \cdot 100 = 24.4\ \%$$

Jacking force:

$$P_{jack} = A_{ps} \cdot \left(f_{pj}\right) = 10.85 \cdot (216.0) = 2344 \text{ kips}$$

Initial prestress force after transfer:

$$P_i = A_{ps} \cdot \left(f_{pj} - \Delta f_{pES}\right) = 10.85 \cdot (216.0 - 25.6) = 2066 \text{ kips}$$

Prestress force after all losses:

$$P_e = A_{ps} \cdot f_{pe} = 10.85 \cdot 163.4 = 1773 \text{ kips}$$

[5.9.2.3.1]

3. Stresses at Transfer (compression +, tension -)
Stress Limits for P/S Concrete at Release

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 other than the precompressed tensile zone without bonded reinforcement" for beams designed with draped strands. The limit is the smaller tension stress of:

$$f_{tlimrel1} = -\ 0.0948 \cdot \sqrt{f'_{ci}} = -\ 0.0948 \cdot \sqrt{8.0} = -\ 0.268 \text{ ksi}$$

or

$$f_{tlimrel2} = -\ 0.200 \text{ ksi}$$

Then $f_{tlimrel} = -0.200$ ksi

Check Release Stresses at Drape Point (0.40 Point of Span)

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.

$$P_i \cdot e_{strand} = 2066 \cdot 13.83 = 28{,}573 \text{ kip-in}$$

$$\text{Top stress due to P/S} = \left(\frac{P_i}{A_g}\right) - \left(\frac{P_i \cdot e_{strand}}{S_{gt}}\right) = \left(\frac{2066}{704}\right) - \left(\frac{28{,}573}{6794}\right)$$

$$= -1.27 \text{ ksi}$$

$$\text{Bottom stress due to P/S} = \left(\frac{P_i}{A_g}\right) + \left(\frac{P_i \cdot e_{strand}}{S_{gb}}\right) = \left(\frac{2066}{704}\right) + \left(\frac{28{,}573}{8246}\right)$$

$$= 6.40 \text{ ksi}$$

Selfweight moment at drape point = $M_{sw0.40}$ = 1295 kip-ft

$$\text{Top stress due to selfweight} = \left(\frac{M_{sw0.40}}{S_{gt}}\right) = \left(\frac{1295 \cdot 12}{6794}\right) = 2.29 \text{ ksi}$$

$$\text{Bottom stress due to selfweight} = \left(\frac{M_{sw0.40}}{S_{gb}}\right) = \left(\frac{1295 \cdot 12}{8246}\right) = -1.88 \text{ ksi}$$

Top stress at drape point = -1.27 + 2.29 = 1.02 ksi

$f_{tlimrel}$ = -0.200 ksi OK

Bottom stress at drape point = 6.40 - 1.88 = 4.52 ksi < 5.20 ksi OK

Check Release Stresses at End of Beam

The strands need to be draped to raise the eccentricity of the prestress force and limit the potential for cracking the top of the beams. Stresses are checked at the transfer point (60 bar diameters from the end of the beam), using the total length of the beam for selfweight moment calculations.

Centroid of strand pattern at the end of the beams:

$$y_{strand} = \left[\frac{(18 \cdot 2) + (14 \cdot 4) + (8 \cdot 6) + 2 \cdot (37+35+33+31+8)}{50}\right]$$

$$= 8.56 \text{ in}$$

Centroid of strand at the transfer point:

$$y_{strand} = 8.56 - \frac{60 \cdot 0.6}{118 \cdot 0.4 \cdot 12 + 7.5} \cdot (8.56 - 4.24) = 8.29 \text{ in}$$

The eccentricity of the strand pattern at the transfer point is:

$$e_{strand} = y_b - y_{strand} = 18.07 - 8.29 = 9.78 \text{ in}$$

The internal prestress moment is:

$$P_i \cdot e_{strand} = 2066 \cdot 9.78 = 20{,}205 \text{ kip-in}$$

$$\text{Top stress due to P/S} = \left(\frac{P_i}{A_g}\right) - \left(\frac{P_i \cdot e_{strand}}{S_{gt}}\right) = \left(\frac{2066}{704}\right) - \left(\frac{20{,}205}{6794}\right)$$
$$= -0.039 \text{ ksi}$$

$$\text{Bottom stress due to P/S} = \left(\frac{P_i}{A_g}\right) + \left(\frac{P_i \cdot e_{strand}}{S_{gb}}\right) = \left(\frac{2066}{704}\right) + \left(\frac{20{,}205}{8246}\right)$$
$$= 5.38 \text{ ksi}$$

$$\text{Top stress due to selfweight} = \left(\frac{M_{swtr}}{S_{gt}}\right) = \left(\frac{132 \cdot 12}{6794}\right) = 0.233 \text{ ksi}$$

$$\text{Bottom stress due to selfweight} = -\left(\frac{M_{swtr}}{S_{gb}}\right) = -\left(\frac{132 \cdot 12}{8246}\right) = -0.192 \text{ ksi}$$

Top stress at transfer point = -0.039 + 0.233 = 0.194 ksi

$f_{tlimrel} = -0.200$ ksi OK

Bottom stress at transfer point = 5.38 – 0.192 = 5.19 ksi < 5.20 ksi OK

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 is determined by substituting the calculated maximum compression stress for $f_{climrel}$ in the compression limit equation and solving for f'_{ci}.

Lowest required $f'_{cireq} = 5.19 / 0.65 = 7.98$ ksi

The bottom stress at release for this particular beam is essentially at the limit so the initial concrete strength cannot be reduced. If f'_{ci} could have been reduced, reanalysis of the beam would be necessary to ensure that stresses were still below the limits.

Proceed to the service and fatigue stress checks after final losses.

[5.9.2.3.2]

4. 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 end of beam 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

$$= -\left(\frac{M_{DC1}}{S_{gb}}\right) - \left(\frac{M_{DC2}}{S_{cb}}\right) - \left(\frac{M_{LL} \cdot 0.8}{S_{cb}}\right) + \left(\frac{P_e}{A_g}\right) + \left(\frac{P_e \cdot e_{strand}}{S_{gb}}\right)$$

$$= -\left(\frac{2817 \cdot 12}{8246}\right) - \left(\frac{496 \cdot 12}{12{,}917}\right) - \left(\frac{1918 \cdot 12 \cdot 0.8}{12{,}917}\right) + \left(\frac{1773}{704}\right) + \left(\frac{1773 \cdot 13.83}{8246}\right)$$

$= -0.494$ ksi

$f_{flimf} = -0.586$ ksi OK

Top stress due to all loads

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

$$= \left(\frac{1773}{704}\right) - \left(\frac{1773 \cdot 13.83}{6794}\right) + \left(\frac{2817 \cdot 12}{6794}\right) + \left[\frac{(496 + 1918) \cdot 12}{42{,}761}\right]$$

$= 4.56 \text{ ksi} < 5.70$ ksi OK

Top stress due to permanent loads

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

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

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

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

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

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

$$= 2.51 \text{ ksi} < 3.80 \text{ ksi} \quad \text{OK}$$

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 $0.9 \cdot f_{pu}$.

$$f_{py} = 0.9 \cdot f_{pu} = 0.9 \cdot 300 = 270 \text{ ksi}$$

[5.6.3.1.1]

$$k = 2 \cdot \left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 2 \cdot \left(1.04 - \frac{270}{300}\right) = 0.280$$

$$d_p = (\text{beam height}) + \text{stool} + \text{deck} - y_{strand}$$

$$= 40 + 1.5 + 8.5 - 4.24 = 45.76 \text{ in}$$

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

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_e + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

$$= \frac{10.85 \cdot 300}{0.85 \cdot 4.0 \cdot 0.85 \cdot 82.00 + 0.28 \cdot 10.85 \cdot \left(\dfrac{300}{45.76}\right)} = 12.67 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 12.67 = 10.77 \text{ in}$$

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

$$c = \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_c \cdot (b - b_w) \cdot h_f}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

$$= \frac{10.85 \cdot 300 - 0.85 \cdot 4.0 \cdot (82 - 34) \cdot 8.5}{0.85 \cdot 4.0 \cdot 0.85 \cdot 34 + 0.28 \cdot 10.85 \cdot \left(\dfrac{300}{45.76}\right)} = 15.81 \text{ in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 15.81 = 13.44 \text{ in}$$

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.

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 300 \cdot \left(1 - 0.28 \cdot \frac{15.81}{45.76}\right) = 271.0 \text{ ksi}$$

The internal lever arm between compression and tension flexural force components is:

$$d_p - \frac{a}{2} = 45.76 - \frac{13.44}{2} = 39.04 \text{ in}$$

Then:

$$M_n = A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_c \cdot (b - b_w) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

$$= 10.85 \cdot 271.0 \cdot 39.04 + 0.85 \cdot 4.0 \cdot (82 - 34) \cdot 8.5 \cdot \left(\frac{13.44}{2} - \frac{8.5}{2}\right)$$

$$= 118{,}218 \text{ kip-in} = 9852 \text{ kip-ft}$$

$M_r = \phi M_n = 1.0 \cdot 9852 = 9852$ kip-ft > $M_u = 7498$ kip-ft 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 $\varepsilon_{tl} = 0.005$

Referring to LRFD Figure C5.6.2.1-1 and using similar triangles in the prestressing strand, ε_t:

$$\varepsilon_t = (d_t - c) \cdot \left(\frac{\varepsilon_c}{c}\right) = (45.76 - 15.81) \cdot \left(\frac{0.003}{15.81}\right) = 0.0056 > 0.005$$

Therefore $\phi = 1.0$, which matches the assumption

[5.6.3.3]

6. Minimum Reinforcement

To prevent brittle failure, an adequate amount of reinforcement is required. Check that the section can carry the smaller of:

1) $1.33M_u$
2) 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 $\lambda = 1.0$.

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{9.5} = 0.74 \text{ ksi}$$

$$f_{cpe} = f_{peb} = \frac{P_e}{A_g} + \frac{P_e \cdot e_{strand}}{S_{gb}}$$

$$= \frac{1773}{704} + \frac{1773 \cdot 13.83}{8246} = 5.49 \text{ ksi}$$

$$M_{cr} = \gamma_3 \cdot \left[(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) \cdot S_{gbc} - M_{dnc} \cdot \left(\frac{S_{gbc}}{S_{gb}} - 1\right)\right]$$

$$= 1.0 \cdot \left[(1.6 \cdot 0.74 + 1.1 \cdot 5.49) \cdot 12{,}917 - 2817 \cdot 12 \cdot \left(\frac{12{,}917}{8246} - 1 \right) \right]$$

$$= 74{,}151 \text{ kip-in} = 6179 \text{ kip-ft} < 9972 \text{ kip-ft} \qquad M_{cr} \text{ GOVERNS}$$

$$M_r = \phi M_n = 9852 \text{ kip-ft} > 6179 \text{ kip-ft} \qquad \text{OK}$$

F. Design Reinforcement for Shear
[5.7]
[5.7.3.2]

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]

For beams with draped strands, 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 straight prestressing strands are considered.

The flexural tension side of the member is defined as:

$$\frac{h_{comp}}{2} = \frac{50}{2} = 25 \text{ in}$$

The centroid of the straight prestressing strands is at:

$$y_{sstr} = \left[\frac{(18 \cdot 2) + (14 \cdot 4) + (8 \cdot 6) + (2 \cdot 8)}{42} \right]$$

$$= 3.71 \text{ in}$$

With this strand centroid, d_p can be computed for the composite section:

$$d_p = h_{comp} - y_{sstr} = 50 - 3.71 = 46.29 \text{ in}$$

Recalculate the value of the compression block depth "a" considering only the straight prestressing strands:

$$A_{ps} = A_{sps} = (\#\text{ of straight strands}) \cdot (\text{strand area}) = 42 \cdot 0.217 = 9.11\ \text{in}^2$$

Begin by assuming the neutral axis lies in the deck.

[5.6.3.1.1]

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_e + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

$$= \frac{9.11 \cdot 300}{0.85 \cdot 4.0 \cdot 0.85 \cdot 82.00 + 0.28 \cdot 9.11 \cdot \left(\dfrac{300}{46.29}\right)} = 10.78\ \text{in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 10.78 = 9.16\ \text{in}$$

Compression block depth is greater than 8.5", the thickness of the slab, so T-section behavior must be considered.

$$c = \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_c \cdot (b - b_w) \cdot h_f}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_w + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

$$= \frac{9.11 \cdot 300 - 0.85 \cdot 4.0 \cdot (82 - 34) \cdot 8.5}{0.85 \cdot 4.0 \cdot 0.85 \cdot 34 + 0.28 \cdot 9.11 \cdot \left(\dfrac{300}{46.29}\right)} = 11.72\ \text{in}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 11.72 = 9.96\ \text{in}$$

$$d_v = d_p - \frac{a}{2} = 46.29 - \frac{9.96}{2} = 41.31\ \text{in}$$

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

$$d_v \geq 0.9\, d_e = 0.9\, d_p = 0.9\,(46.29) = 41.7\ \text{in}$$

Take d_v = 41.7 inches

Then the critical section for shear x_{vcrit} is:

x_{vcrit} = (0.5 · sole plate length) + d_v

= (0.5 · 15.0) + 41.7

= 49.2 in = 4.1 ft from centerline of bearing

Check Maximum Factored Shear Limit

From Table 5.7.2.4 the Strength I design shear at 4.1 ft is

$V_u = 285$ kips

The amount of force carried by the draped strands at their effective prestress level is:

$$P_{8d} = 8 \cdot 0.217 \cdot 163.4 = 283.7 \text{ kips}$$

The inclination of the draped strands is:

$$\varphi = \arctan\left[\frac{(34 - 7) / 12}{47.83}\right] = 2.69 \text{ degrees}$$

The vertical prestress component is:

$$V_p = P_{8d} \cdot \sin(\varphi) = 283.7 \cdot \sin(2.69) = 13.3 \text{ kips}$$

The superstructure is supported by an integral type abutment. Therefore, the nominal shear capacity of the section is limited to:

[5.7.3.3]

$$V_n = 0.25 \cdot f'_c \cdot d_v \cdot b_v + V_p = 0.25 \cdot 9.5 \cdot 41.7 \cdot 6.5 + 13.3 = 657 \text{ kips}$$

The maximum design shear the section can have is:

$$\phi_v \cdot V_n = 0.90 \cdot 657 = 591 \text{ kips} > 285 \text{ kips} \quad \text{OK}$$

Note that if the superstructure was supported by a parapet type abutment, which is 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 or semi-integral abutment 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 Figure 5.7.3.4.2-1. At x_{vcrit}, A_{ps} consists of only the straight strands.

Near the end of the beam, A_{ps} must also be reduced for development, so f_{ps} must be calculated again for the end section following the process shown previously:

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 300 \cdot \left(1 - 0.28 \cdot \frac{11.72}{46.29}\right)$$

$$= 278.7 \text{ ksi}$$

[5.9.4.3] Development length ℓ_d is:

$$\ell_d = \kappa \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b$$

$$= 1.6\left[278.7 - \frac{2}{3}(163.4)\right](0.6) = 163.0 \text{ in}$$

Transfer length ℓ_{tr} is:

$$\ell_{tr} = 60 \cdot d_b = 60\,(0.6) = 36.0 \text{ in}$$

At the critical section x_{vcrite} = (49.2 + 7.5) = 56.7 in from the beam end, the strand development fraction is:

$$F_{dev} = \frac{f_{pe}}{f_{ps}} + \frac{x_{vcrite} - \ell_{tr}}{\ell_d - \ell_{tr}}\left(1 - \frac{f_{pe}}{f_{ps}}\right)$$

$$= \frac{163.4}{278.7} + \frac{56.7 - 36.0}{163.0 - 36.0}\left(1 - \frac{163.4}{278.7}\right) = 0.65$$

Therefore, $A_{ps} = A_{sps} \cdot F_{dev}$

$$= 9.11 \cdot 0.65 = 5.92 \text{ in}^2$$

[5.7.3.4.2] Use LRFD equation 5.7.3.4.2-4 to compute the strain:

$$\varepsilon_s = \frac{\left(\left|\frac{M_u}{d_v}\right| + 0.5N_u + |V_u - V_p| - A_{ps} \cdot f_{po}\right)}{E_s \cdot A_s + E_p \cdot A_{ps}}$$

$$= \frac{\left[\left|\frac{1025 \cdot 12}{41.7}\right| + |285 - 13.3| - (5.92 \cdot 0.70 \cdot 300)\right]}{28{,}500 \cdot 5.92} = -0.00401$$

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

$$\varepsilon_s = \frac{\left(\left|\frac{M_u}{d_v}\right| + 0.5N_u + |V_u - V_p| - A_{ps} \cdot f_{po}\right)}{E_c \cdot A_{ct} + E_s \cdot A_s + E_p \cdot A_{ps}}$$

$$= \frac{\left[\left|\frac{1025 \cdot 12}{41.7}\right| + |285 - 13.3| - (5.92 \cdot 0.70 \cdot 300)\right]}{4899 \cdot 435 + 28{,}500 \cdot 5.92} = -0.00029$$

Computed strain limits:

$$-0.0004 < -0.00029 < 0.006 \quad \text{OK}$$

Compute the tensile stress factor β using equation 5.7.3.4.2-1

$$\beta = \frac{4.8}{1 + 750 \cdot \varepsilon_s} = \frac{4.8}{1 + 750 \cdot (-0.00029)} = 6.13$$

Compute the angle θ using equation 5.7.3.4.2-3

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500 \cdot (-0.00029) = 28.0 \text{ degrees}$$

Compute the concrete contribution:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 6.13 \cdot \sqrt{9.5} \cdot 6.5 \cdot 41.7 = 161.8 \text{ kips}$$

The required steel contribution is

$$V_s = V_n - V_c - V_p = \frac{V_u}{\phi_v} - V_c - V_p = \frac{285}{0.90} - 161.8 - 13.3 = 141.6 \text{ kips}$$

Find the required spacing of double leg #4 stirrups:

$$s = \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{V_s} = \frac{2 \cdot 0.20 \cdot 60 \cdot 41.7 \cdot \cot(28.0)}{141.6} = 13.3 \text{ in}$$

Try double leg stirrups at a 12 inch spacing near the end of the beam.

$$A_v = \frac{0.4 \cdot 12}{12} = 0.40 \text{ in}^2 / \text{ft} \qquad V_s = 156.9 \text{ kips}$$

[5.7.2.5] Check that the minimum transverse reinforcement requirement is satisfied:

$$\frac{A_{vmin}}{s} = 0.0316 \cdot \lambda \cdot \sqrt{f'_c} \cdot \frac{b_v}{f_y}$$

$$= 0.0316 \cdot 1.0 \cdot \sqrt{9.5} \cdot \frac{6.5}{60} \cdot 12 = 0.13 \frac{\text{in}^2}{\text{ft}} < 0.40 \frac{\text{in}^2}{\text{ft}} \quad \text{OK}$$

[5.7.2.6] Check maximum permitted stirrup spacing at x_{vcrite}:

$$v_u = \frac{V_u - \phi \cdot V_p}{\phi \cdot b_v \cdot d_v} = \frac{285 - 0.90 \cdot 13.3}{0.90 \cdot 6.5 \cdot 41.7} = 1.12 \text{ ksi}$$

$$v_{ulimit} = 0.125 \cdot f'_c = 0.125 \cdot 9.5 = 1.19 \text{ ksi} > 1.12 \text{ ksi}$$

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:

$A_{cv} = 22 \cdot 12 = 264.0 \text{ in}^2/\text{ft}$

[5.7.4.4]

The standard beam details require the top flanges of the beam to be roughened. Then:

$c = 0.28$ ksi $\mu = 1.0$ $K_1 = 0.3$ $K_2 = 1.8$ 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

$$T_{r2} = A_{ps} \cdot f_{ps} \cdot F_{dev} = 9.11 \cdot 278.7 \cdot 0.65 = 1650 \text{ kips}$$

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:

$$T_{u2} = \frac{M_u}{\varphi_f d_v} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta)$$

$$= \frac{1025 \cdot 12}{1.0 \cdot 41.7} + \left(\frac{285}{0.9} - 0.5 \cdot 156.9 - 13.3\right) \cdot \cot(28.0)$$

$$T_{u2} = 718.0 \text{ kips} < 1650 \text{ kips} \quad \text{OK}$$

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 jacking force to determine the required amount of steel.

The factored design splitting force is:

$$P_{split} = 1.0 \cdot 0.04 \cdot P_{jack} = 1.0 \cdot 0.04 \cdot 2344 = 93.8 \text{ kips}$$

The amount of resisting reinforcement is determined using a steel stress f_s of 20 ksi:

$$A_s = \frac{P_{split}}{f_s} = \frac{93.8}{20} = 4.69 \text{ in}^2$$

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:

$$\frac{A_s}{2 \cdot A_b} = \frac{4.69}{2 \cdot 0.31} = 7.6$$

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 seven sets of #5 stirrups (G508E) spaced at 2 1/2 inch centers.

$x_{splitting} = 2 + 7 \cdot 2.5 = 19.5 \text{ in} > 10 \text{ in}$

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.

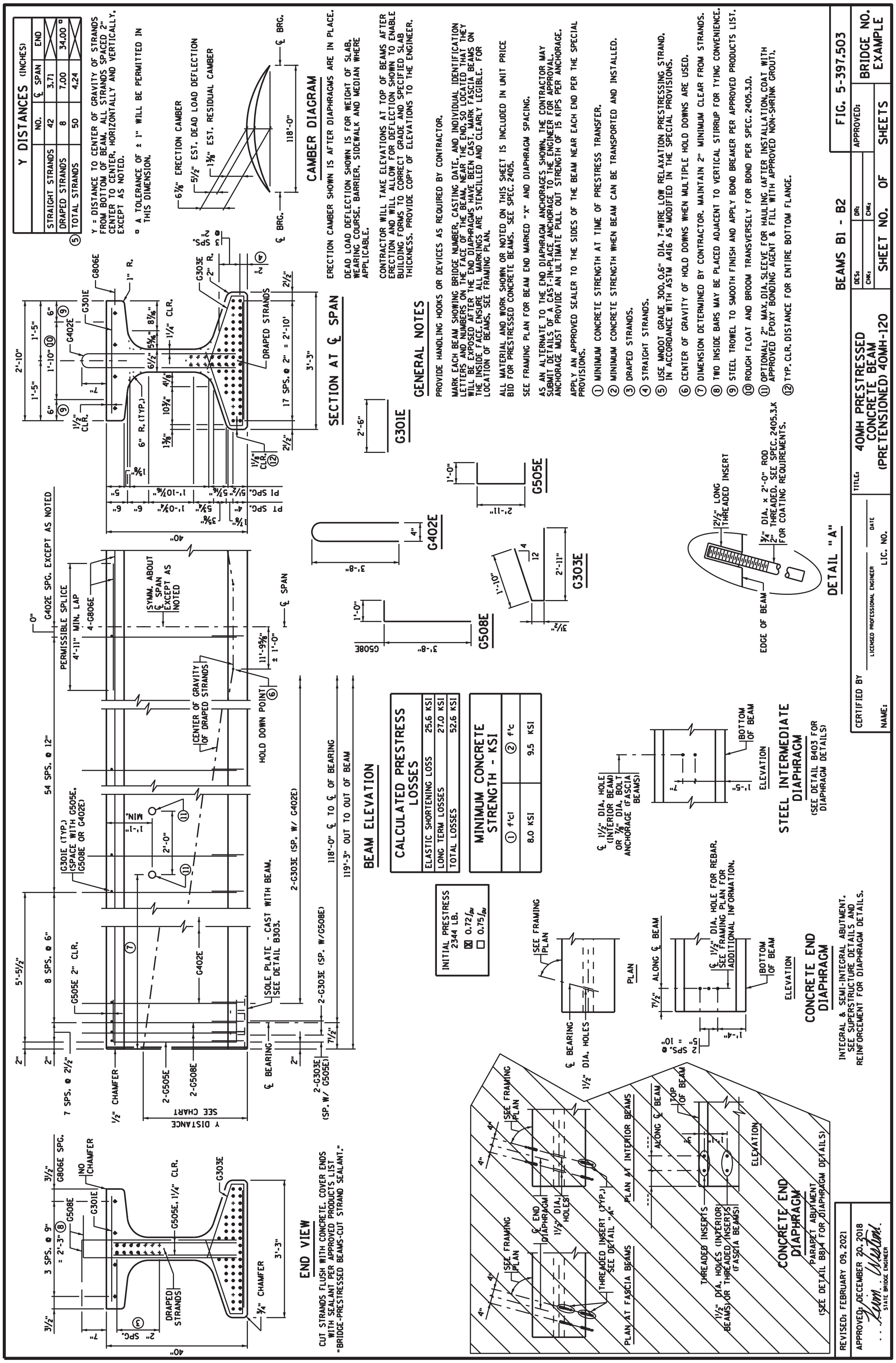

Y DISTANCES (INCHES)
STRAIGHT STRANDS 42 3.71
DRAPED STRANDS 8 7.00 34.00
TOTAL STRANDS 50 4.24
Y = DISTANCE TO CENTER OF GRAVITY OF STRANDS FROM BOTTOM OF BEAM. ALL STRANDS SPACED 2" CENTER TO CENTER, HORIZONTALLY AND VERTICALLY, EXCEPT AS NOTED.
A TOLERANCE OF ± 1" WILL BE PERMITTED IN THIS DIMENSION.
CAMBER DIAGRAM
ERECTION CAMBER SHOWN IS AFTER DIAPHRAGMS ARE IN PLACE.
SECTION AT ℄ SPAN
GENERAL NOTES
PROVIDE HANDLING HOOKS OR DEVICES AS REQUIRED BY CONTRACTOR.
SEE FRAMING PLAN FOR BEAM END MARKED "X" AND DIAPHRAGM SPACING.
APPLY AN APPROVED SEALER TO THE SIDES OF THE BEAM NEAR EACH END PER THE SPECIAL PROVISIONS.
BEAM ELEVATION
CALCULATED PRESTRESS LOSSES
ELASTIC SHORTENING LOSS 25.6 KSI
LONG TERM LOSSES 27.0 KSI
TOTAL LOSSES 52.6 KSI
MINIMUM CONCRETE STRENGTH - KSI
8.0 KSI 9.5 KSI
INITIAL PRESTRESS 2344 LB.
END VIEW
STEEL INTERMEDIATE DIAPHRAGM
CONCRETE END DIAPHRAGM
DETAIL "A"
40MH PRESTRESSED CONCRETE BEAM (PRETENSIONED) 40MH-120
BEAMS B1 - B2
FIG. 5-397.503
BRIDGE NO. EXAMPLE
REVISED: FEBRUARY 09, 2021
APPROVED: DECEMBER 20, 2018
STATE BRIDGE ENGINEER

Figure 5.7.2.5

5.7.3 Three-Span Haunched Post-Tensioned Concrete Slab Design Example

[Future manual content]

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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.

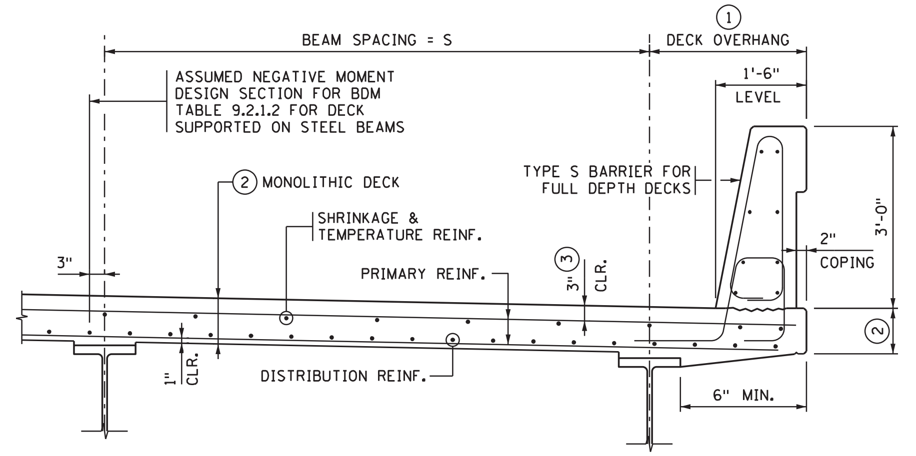
Avoid drainage details that include flat elements (grades less than 5%). Pipes and drainage elements with flat profiles tend to collect debris and plug.

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.

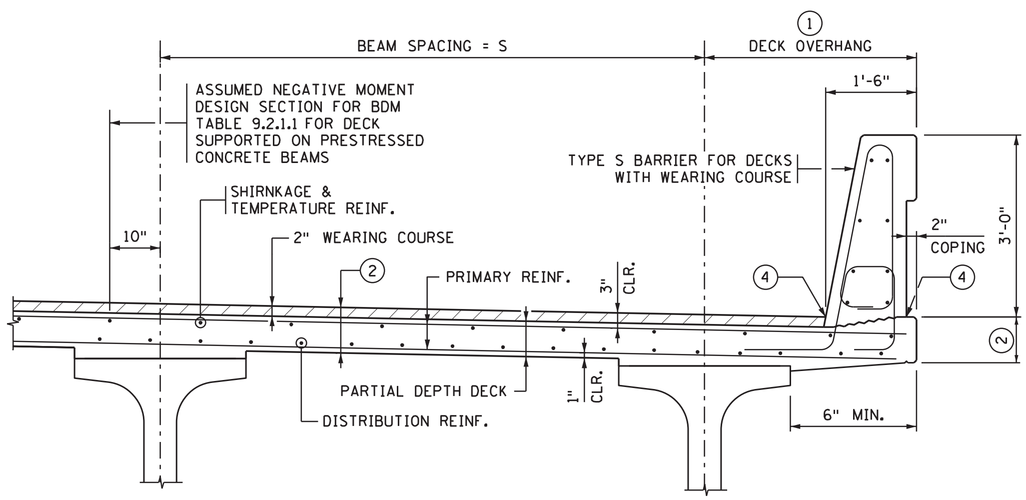
Provide the materials and gages for corrugated metal (C.M.) drains and semi-circle deck drains, such as those used on railroad bridges, in the plan details.

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 ($\gamma_e = 0.75$).

- 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.

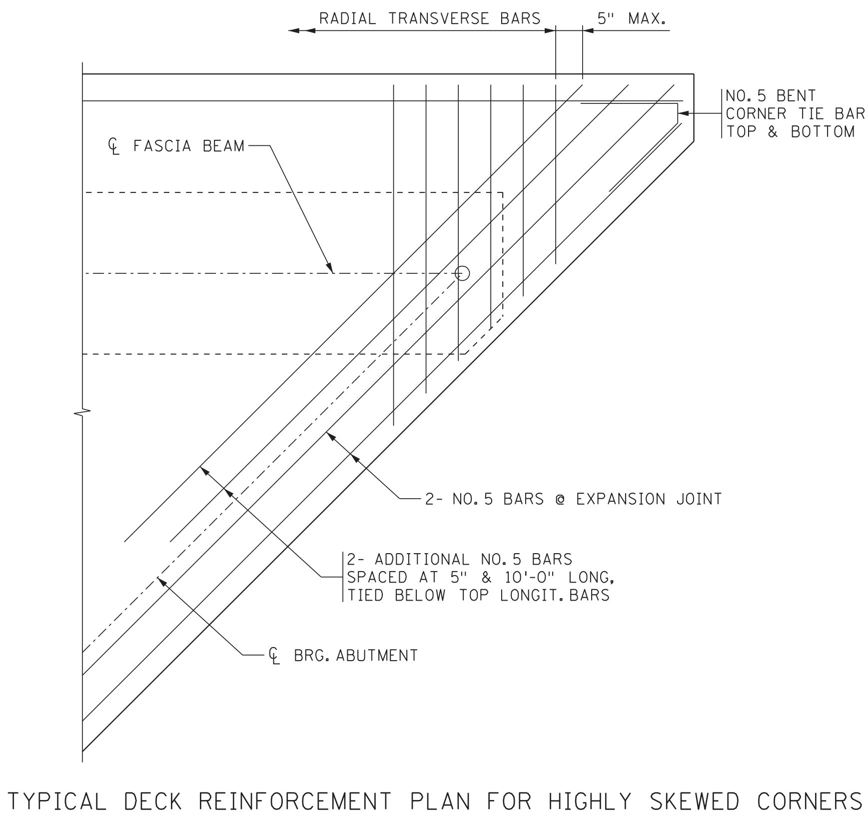


Figure 9.2.1.1

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.
- Provide additional longitudinal steel to minimize transverse deck cracking. See Figures 9.2.1.8 and 9.2.1.9.

- For decks supported on non-continuous prestressed beams, detail a partial depth sawcut in the deck over the pier backfilled with a sealant. See Figure 9.2.1.10.
- Place polystyrene on the corners of prestressed concrete beam bridges with skews greater than 20° to reduce wandering of the transverse deck crack at the centerline of pier. See Figure 9.2.1.10.

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.

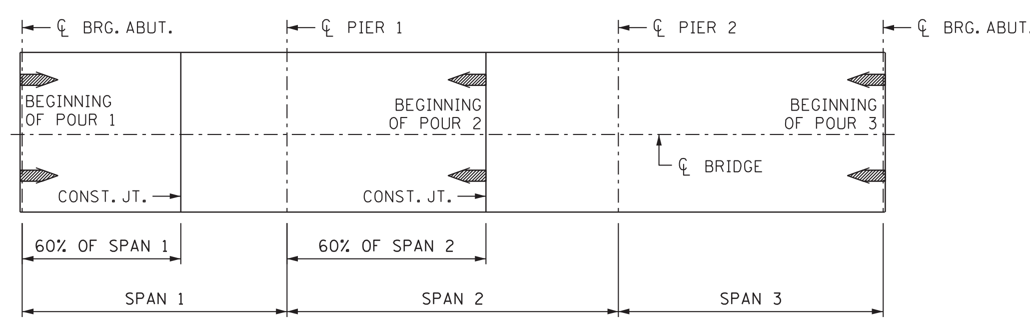


Figure 9.2.1.2

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.

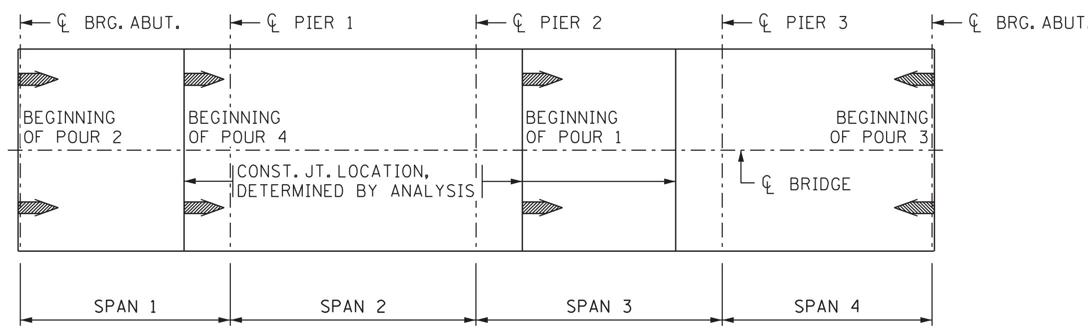


Figure 9.2.1.3

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.