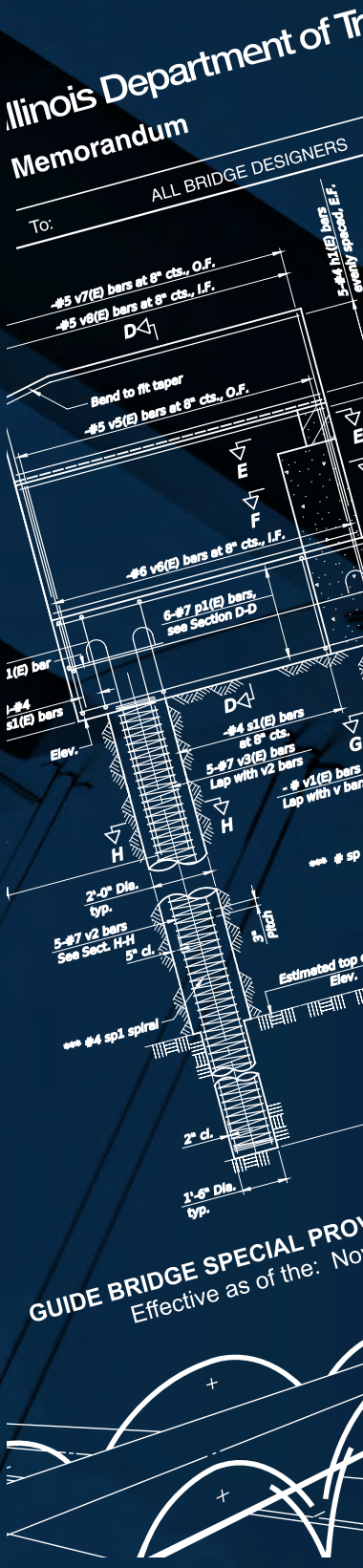


2025 BRIDGE MANUAL

Illinois Department of Transportation

STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION BUREAU OF BRIDGES AND STRUCTURES	**FILE CLASSIFICATION:** Bridge Manual
	BRIDGE MANUAL CHANGE LETTER NO. *25-01*
To: *All holders of the Bridge Manual*	**DATE ISSUED:** *January 31, 2025*

The 2025 Bridge Manual is available on the IDOT website. This manual shall replace the 2023 Bridge Manual in its entirety.

The primary revisions in the 2025 Bridge Manual are presented below.

Incorporated ABD Memos

Incorporated numerous ALL BRIDGE DESIGNER (ABD) memorandums with some minor revisions as observed through our vetting process. Because memorandums contain both planning and design policies, incorporations may occur over several Bridge Manual Sections. Refer to the change bars on the left side of the Bridge Manual pages and see the list below.

ABD #	ABD Title	Primary Locations of 2025 Incorporation
ABD 24.4	Updated Strand Patterns and Charts for IL-Beams	2.3.6.1.3B; 3.4
ABD 23.3	Full Lane Sealant Waterproofing System	3.2.13
ABD 23.2	Headed Reinforcement	Entire Manual
ABD 23.1	Nonredundant Steel Tension Members in Design	3.3.4.1; 3.3.4.2; 3.3.4.2.2; 3.1.13; 3.1.14.8(2k); 3.1.14.8(3l, 3m, 3n)
ABD 22.8	Modular Expansion Joint Details and Re-Certification	3.3.23; 3.6.2.3
ABD 22.7	Noise Abatement Walls	3.12
ABD 22.3	High Load Multi-Rotational Bearings (Revised)	3.7.5
ABD 15.5	Revised Load Rating Guidance for Bolted and Riveted Gusset Plates in Steel Truss Bridges	Structural Services Manual
ABD 06.1	Structural Concrete Repair	Structural Services Manual

General Updates

1. Revision of body text: Addition of change bars along left border to indicate
    a. additions / deletions / revisions.
    b. editorial reorganization and updates.
    c. corrected IDOT website links.
    d. update of words/terms for consistency with AASHTO LRFD Bridge Design Specifications and the IDOT SSRBC.
2. Revision of figures
    a. Update of CADD figures from v8i to CONNECT environment.
    b. See “Topic-Specific Updates” and “Section-Specific Updates” below for significant figure revisions.
3. Incorporation of AASHTO LRFD 10th Edition Specifications, specifically:
    a. Revised Specifications cross-references.
    b. Revised tension development lengths (see “Topic-Specific Updates” below).

Topic-Specific Updates

1. Included references to the IDOT Seismic Manual.
2. MSE-wall-type abutments: reorganization to present preferred standards.
    a. Section 3.8.9 (Vaulted Abutments) was moved to a new Section 3.8.14.
        i. Associated text references and cross-references re-numbered.
        ii. Associated figures moved & re-numbered.
    b. Section 3.11.1.4 (Retaining Walls > Mechanically Stabilized Earth (MSE) > Abutments) was moved to Section 3.8.9 with additional new policies and new figures.
3. Reinforcement Bar tension development and laps: updates for consistency with AASHTO 5.10.8.2.
    a. Expansion of Section 3.1.4.
    b. Revision and expansion of Appendix A3.4.
    c. Revision of applicable lap lengths in figures (manual-wide).
4. Update of figures with headed bars for proper payment (as “Bar terminators paid for separately”) and typical “_ - #X Bar terminators” note.
5. Deck Drainage strategies updated in the Planning and Design Sections
    a. Specified a revised hierarchy to select off-bridge drainage features, drainage scuppers, and floor drains.
    b. Specified a new Table 2.3.6.1.8-1 to be placed on TSLs to coordinate the Q (cfs) of off-bridge drainage and roadway plan needs with the District.
6. Removed references to Base Sheet SB-1 and promoted General Note 13 for beam depths > 30 in.

Section 1 Updates

1. Section 1.1: Included standard Document Control and Revision, Distribution, and Revision History.
2. Section 1.1.2: Clarified scope of Displacement-Based and Force-Based seismic design.
3. Section 1.4.1: Revised BBS 2750 form with revised sample figure.

Section 2 Updates

1. Section 2.2.3.3: Clarified reference of “Bridge Condition Report Procedures and Practices” for reuse of substructure elements.
2. Section 2.2.3.4.3: Clarified traffic items to be identified in the BCR.
3. Section 2.3.2:
    a. Replaced “Retained” height with “Exposed” height for cases that require submittal of TSL & BCR.
    b. Full update of cases when TSL submittal is not required.
4. Section 2.3.3: Updated clarification of submittals/timelines.
5. Figure 2.3.6.1.2-12: Corrected Figure title from 42x36 to 42x48.
6. Section 2.3.6.1.3 charts reorganized.
    a. Section 2.3.6.1.3A groups together the body text and planning charts for PPC I-Beams and Bulb-T Beams.
    b. Section 2.3.6.1.3B groups together the body text and planning charts for PPC IL-Beams.
7. Section 2.3.6.1.1:
    a. Reorganized the text of Coating Strategies 1-4 to match the order of Figure 2.3.6.1.1-1.
    b. Provided AGA websites for “Galvanized Steel” to assist with estimations of vat width/depths.
    c. Updated components to be included in minimum roadway clearance.
8. Section 2.3.6.1.6: Clarified use of preformed joint seals & neoprene joints.
9. Section 2.3.6.1.7:
    a. Included reference to upcoming “Local Only” Base sheets for lower test level railings.
    b. Clarified use of 44 in. Constant-Slope parapet for “when at least one of” listed scenarios occurs.
    c. Clarified requirement of when to specify Highway Standards TBT Type 6 (Highway Standard 631031) & TBT Type 5 (Highway Standard 631026).
10. Section 2.3.6.1.8:
    a. Full section update to preferred drainage strategies.
    b. Addition of new figures and planning tables.
    c. *Bridge Drainage Scuppers*; *Floor Drains; Off Bridge Drainage* Sections: Full update.
    d. *Drainage Scupper Location by Hydraulic Analysis* Section: updated rainfall intensity (I) to 7 in./hr.
11. Section 2.3.6.2:
    a. Section 2.3.6.2.1: Addition of MSE as an abutment type; added planning guidance and manual cross-references.
    b. *Integral Abutment, Semi-Integral Abutment, Closed Abutment,* and *Vaulted Abutment* sections: updated planning guidance and manual cross-references.
    c. Section 2.3.6.2.2:
        i. Updated planning considerations for the 3 pier types.
        ii. *Column Web-Wall Drilled Shaft Bent Pier* section: Revised usage of pier type without cofferdams .
    d. Figure 2.3.6.2.2-1: Revised limits of H for 3 column case & updated notes.
12. Section 2.3.6.3.1: Clarified metal shell piles as the preferred pile type when feasible.

13. Section 2.3.6.3.2:
    a. *Design Scour Table for Bridges* section: Revised Item #4 for method and timing of submitting BBS SCE form.
    b. *Item 113 Rating - Table A / B / C* body-text sections: Clarified maximum design scour depths caused by overtopping.
    c. *Item 113 Rating - Table D*: New body-text section and Table 2.3.6.3.2-D.
14. Section 2.3.6.3.3:
    a. Figure 2.3.6.3.3-1: Updated layout; updated handles of 10' riprap limits.
    b. Figure 2.3.6.3.3-3: Added additional column to dimension table.
    c. Figure 2.3.6.3.3-5: Revised end of cap heights to be consistent with Section 3.8; revised step heights of Sketch 4.
    d. Figure 2.3.6.3.3-6 & Figure 2.3.6.3.3-7: Updated figure titles.
15. Section 2.3.6.4.2: Clarified exceptions to requirement of Type 2 Cofferdam.
16. Section 2.3.7:
    a. Section 2.3.7.1.1: Clarified applicability of vehicle collision force.
    b. Section 2.3.7.1.2: Clarified vertical clearance.
    c. Figure 2.3.7.4.1-1: Updated crown labels for "Raised-Curb Medians".
    d. Section 2.3.7.6 & 2.3.7.7: Full revision.
    e. Section 2.3.7.8: ADT clarified as "current" when checking > 10,000.
17. Section 2.3.8: corrected differential dead load deflection limit to 2 inches.
18. Section 2.3.10:
    a. Clarified scope of Displacement-Based and Force-Based seismic design in all sub-sections.
    b. 2.3.10.3: Provided sample of how to fill out Seismic Data tables for TSL.
    c. Figure 2.3.10-1 through Fig 2.3.10-4: Updated "Code" to "Bridge Design Specifications" in notes.
19. Section 2.3.11:
    a. Replaced "cast-in-place" with CIP in appropriate locations.
    b. Section 2.3.11.1.2: Added item 6c for cases to avoid.
    c. Section 2.3.11.2.1: Clarified Item #4 for Design Fill.
    d. Figure 2.3.11.2.2-3: Added missing line for Circle Note (1).
    e. Section 2.3.11.2.3: Revised "abutment" scour to "local" scour.
    f. Section 2.3.11.4.1:
        i. Fifth paragraph to end of Section: Full update on quantities, special provision, and conversion from cubic yards to tons; removal of "*The Rockfill shall be capped…*" note.
        ii. Figure 2.3.11.4.1-1: Full figure update.
20. Section 2.3.13.1 through Section 2.3.13.3: Added additional items to all Checklists.
21. Section 2.3.15: Updated responsibilities, submittal timing, and submittal method for various subsections.
22. Section 2.4.2.3: Full update; clarified partial removal of the backwall as complete backwall removal can be unfeasible without damaging the existing abutment.

Section 3 Updates

1. Section 3.1:
    a. Section 3.1.3: Full General Note update.
    b. Section 3.1.6: Clarified “Total Bill of Material” directions for staged construction and dual structures.
    c. Section 3.1.12.1: Removed reference to base sheet SB-1 and promoted General Note 13.
    d. Figure 3.1.12.2: Provided sample deflection table.
    e. Figure 3.1.12.2-1 & Figure 3.1.12.3-1: Added note to clarify dashes.
    f. Section 3.1.13: Incorporated elements of ABD 23.1.
    g. Section 3.1.14:
        i. Added numbered headings for sub-sections.
        ii. 3.1.14.8, #2: Incorporated elements of ABD 23.1.
2. Section 3.2:
    a. Figure 3.2.2.1-1:
        i. Increased edge beam depth due to increased cover.
        ii. Added 5" min. dimension from beam to edge of edge beam.
        iii. Added * note to more dimensions on x(E) bar.
        iv. Added ** note to provide clear cover at flange for skewed structures.
    b. Figure 3.2.2.1-2:
        i. Increased 4" min. dimension to 5" min. from beam to edge of edge beam.
        ii. Added * note to more dimensions on x(E) bar.
        iii. Added ** note to provide clear cover at flange for skewed structures.
        iv. Removed threaded dowel rods, and replaced with headed a3(E) bars.
    c. Section 3.2.3: Clarified area of steel calculations required for different detailing orientations.
    d. Figure 3.2.4-1 & Figure 3.2.4-4:
        i. Deck thickness at overhang revised to 9½“.
        ii. Added Protective Coat Area.
        iii. Added reference to Table 3.2.9-1 for minimum overhang.
    e. Figure 3.2.4-2 & Figure 3.2.4-5:
        i. Changed clear cover in parapet from 2" min. to 2".
        ii. Labeled bottom of cork joint in parapet.
        iii. Corrected distance from bottom distribution reinforcement to steel beam to match 3.2.2.
    f. Figure 3.2.4-8 & Figure 3.2.4-9: Increased aluminum sheet thickness to 3/16" minimum.
    g. Figure 3.2.4-11: Clarified location of formed joint.
    h. Figure 3.2.4-13: Decreased chamfer at minimum fillet from ¾" to ½".
    i. Section 3.2.4.5: New section for aesthetic steel bridge railings.
    j. Section 3.2.5: Clarified type of curb shown in figures in this section.
    k. Figure 3.2.5-1:
        i. Increased aluminum sheet thickness to 3/16" minimum. Updated coating of aluminum sheet.
        ii. Added reference to Figure 3.2.7-1.

l. Figure 3.2.5-2: Updated callout for expansion anchor or ferrule loop. Updated * note to allow epoxy coating.
m. Figure 3.2.7-2:
    i. Added callout to slope bottom of deck and provide a ¾" drip notch.
    ii. Added * note giving guidance for 2" open joints.
    iii. Updated PJF callout.
    iv. Updated trapezoidal opening shape.
n. Table 3.2.9-1: New table for minimum fascia beam overhang.
o. Figure 3.2.9-2 & Figure 3.2.9-3: Added notes for changes when using 8" drains; added cross reference to Table 3.2.9-1.
p. Figure 3.2.10-1 through Figure 3.2.10-5: Updated all figures for 9”-wider light pole blister.
q. Figure 3.2.11-1: Increased bottom reinforcement clear cover to 1½”.
r. Figure 3.2.11-2 & Figure 3.2.11-3: Corrected AASHTO references, letters C and D.
s. Section 3.2.12: Updated precast approach slab nomenclature to partial depth precast approach slab.
t. Section 3.2.13: New section and figure for deck treatment guidance.

3. Section 3.3:
    a. Section 3.3.3: Added policy on eliminating uplift.
    b. Section 3.3.4.2: New section to incorporate elements of ABD 23.1.
    c. Section 3.3.5: Full update of second paragraph.
    d. Figure 3.3.9-1: Changed bottom clear cover of the rebar from 1½” minimum to 1½”.
    e. Figure 3.3.21-3: Updated equations in letter e.
    f. Section 3.3.22 & Section 3.3.23: Cleaned up weld detailing in figures.
    g. Figure 3.3.22-1: Added new view adjacent to abutment showing first interior cross-frame/diaphragm for steel beams and plate girders on integral and semi-integral abutments.
    h. Figure 3.3.22-3: Added plate girder web depth column. Combined details into one drawing.
    i. Figure 3.3.22-4:
        i. Changed name to Weld and Clip Limits and Details.
        ii. Added alternate web weld detail.
        iii. Added guidance for when to use each detail.
        iv. Combined figures from Bridge Manual 2023 Figure 3.3.22-4, Figure 3.3.23-4, and Figure 3.3.23-6.
    j. Figure 3.3.23-7:
        i. Moved Detail A and B from the following figure onto this one.
        ii. Updated weld detail to include guidance for when cross-frames are galvanized.
    k. Figure 3.3.23-8:
        i. Added Isometric view.
        ii. Moved Detail A and B to Figure 3.3.23-7.
    l. Figure 3.3.29.2-1: Clarified material of drip plate.
    m. Section 3.3.29.3: Clarified the content GBSP 83.
    n. Section 3.3.29.3 & 3.3.29.4: Added AASHTO LFRD reference for considering bolt relaxation.
4. Section 3.4:
    a. Consolidation of data between Table 3.4.3.2-1 and Figure 3.4.3.2-7.

   b. Section 3.4.3.3: Revised fillet threshold & updated Figure 3.4.3.3-1 accordingly.
   c. Section 3.4.4.2: moved paragraph previously after Table 3.4.4.2-1 to below criteria #9.
   d. Figure 3.4.6-1: Revised name; corrected value assignments of 55,400 & 41,400.
   e. Replaced Figure 3.4.6-3 with Table 3.4.6-3 & Table 3.4.6-4; consolidated Figure 3.4.6-3 into a revised/single Figure 3.4.6-2 (per ABD 24.4).
   f. Section 3.4.9:
      i. Figure 3.4.9-1 & Figure 3.4.9-3: Updated bond breaker notes.
      ii. Figure 3.4.9-4 & Figure 3.4.9-5: Replaced old Figure 3.4.9-4.
      iii. Figure 3.4.9-7 & Figure 3.4.9-8: Replaced old Figure 3.4.9-6 & Figure 3.4.9-7.
      iv. Updated all figure number references.
5. Section 3.5:
   a. Section 3.5.1: Imported correction factor paragraphs/tables from design guide.
   b. Figure 3.5.7.4-2 & Figure 3.5.7.4-3: Corrected label of C(E) bars to “Bar B10(E)” and increased leg length.
   c. Figure 3.5.9-2: Updated grade of steel plates to 50 ksi.
6. Section 3.6:
   a. Section 3.6.2.1: Updated variable definitions for “A”.
   b. Figure 3.6.2.1-1 through Figure 3.6.2.1-3: Added notes to indicate “After grinding (when specified)”.
   c. Figure 3.6.2.1-6 & Figure 3.6.2.2-10: 3/8” DIA for countersunk bolts now a “min.” diameter.
   d. Figure 3.6.2.2-4 & Figure 3.6.2.2-8: Updated weld callouts.
   e. Section 3.6.2.3: Updated variable definitions for T.
   f. Figure 3.6.2.3-1 through Figure 3.6.2.3-12, Figure 3.6.2.3-14 and15: Modified since ABD with updated figure titles.
   g. Figure 3.6.2.3-1 & Figure 3.6.2.3-7: Modified since ABD with:
      i. callout that modular joint shall be flush with final deck surfaces.
      ii. ** note for rehabilitation/repair projects.
   h. Figure 3.6.2.3-4: Modified since ABD to clarify x1(E) bar laps.
   i. Figure 3.6.2.3-15: Modified since ABD to clarified Auxiliary top flange equation and variables.
7. Section 3.7:
   a. Figure 3.7.1.2-1: Added elevation at pier view. Added pintle detail.
   b. Figure 3.7.1.2-4:
      i. Corrected Sole Weld Design Equation.
      ii. Removed “c=0.144 for f_y=36 ksi” note for industry trends.
      iii. Added subscripts to length and width of sole and masonry plates. Variables now match HLMR equations.
      iv. Changed sole weld design * note to ** note for clarity.
      v. Updated linework.
   c. Table 3.7.2-1: Changed “Length” to “Minimum Length” in column header.
   d. Table 3.7.3-1: Anchor bolt shear capacities increased. Corrected the equation for F.
   e. Figure 3.7.4-15, Figure 3.7.4-16, & Figure 3.7.4-18: Clarified H.S. bolts' galvanizing note.
   f. Figure 3.7.4-20: Clarified * note. Corrected weld detailing.

   g. Figure 3.7.4-21: Removed “c=0.144 for f_y=36 ksi” note for industry trends.
   h. Figure 3.7.4-24: Corrected figure title to Type 2 and 3 Bearings.
8. Section 3.8:
   a. Section 3.8.1: Clarified dynamic loading.
   b. Section 3.8.3: Added items 1-3 for utilization list.
   c. Figure 3.8.3-1: Added PJF.
   d. Figure 3.8.3-3: Deleted heads off of s10(E) bars.
   e. Figure 3.8.3-4: Updated “bearing pads” references.
   f. Figure 3.8.3-12: Updated caption for upper-left “END ELEVATION” to include up to IL90.
   g. Section 3.8.4: Introduced new Section 3.8.4.1 for rehabilitation and reconstruction scope of work.
   h. Figure 3.8.4-3: Clarified *** note for drop-down of diaphragm concrete to top of bottom flange.
   i. Figure 3.8.7-2 through Figure 3.8.7-4: Full revisions.
   j. Section 3.8.9 moved to Section 3.8.14:
      i. Added new figures and policies for MSE-wall-type abutments.
   k. Section 3.8.14: New section of existing policies moved from 3.8.9:
      i. Old Figure 3.8.9-1 now new Figure 3.8.14-1: Updated View A-A.
      ii. Old Figure 3.8.9-4 now new Figure 3.8.14-4: full revision.
      iii. Old Figure 3.8.9-5 through Figure 3.8.9-6 now new Figure 3.8.14-5 through Figure 3.8.14-6: full revision.
      iv. Old Figure 3.8.9-5 now new Figure 3.8.14-5: Updated Section A-A, notes, and plan view.
      v. Old Figure 3.8.9-6 now new Figure 3.8.14-6: Updated linework at span ends.
9. Section 3.9:
   a. Figure 3.9.1-1: Added new Pile Orientation Figure.
   b. Figure 3.9.5-2: Added new Alternate Solid Wall Figure.
   c. Old Figure 3.9.5-2 now moved to Figure 3.9.5-3: removed PJF.
10. Section 3.10: No significant revisions.
11. Section 3.11:
   a. Figure 3.11.1.1-1: Increased the thickness of the coping overhang from 4” to 5” minimum. Increased coping height from 1’-9” to 2’-2”. Added ¾” continuous drip groove. Added bond breaker between front of panel and coping overhang.
   b. Section 3.11.1.1: Added plan notes for fences mounted to a CIP coping.
   c. Figure 3.11.1.1-2 & Figure 3.11.1.1-3: Added new figures for chain link fence mounted on top of MSE wall CIP coping.
   d. Section 3.11.1.2: Renamed to ‘Vertical and Horizontal Obstructions’. Added new policy for geometry and impact loading requirements for flexible post and beam barriers driven into MSE wall.
   e. Figure 3.11.1.2-3: Added new figure for MSE wall horizontal obstructions.
   f. Section 3.11.1.3: Added new policy for impact loading requirements for MSE walls when supporting an anchorage slab.
   g. Figure 3.11.1.3-1 & Figure 3.11.1.3-2:
      i. Increased overhang height from 2" to 4" minimum.
      ii. Increased overhang width 4" to 5" minimum.
      iii. Increased PJF thickness from 1/2" to 1".
      iv. Added new table and updated * note for impact loading for soil reinforcement.

v. Added ** note to overhang height, showing limits of bond breaker.
vi. Revised the term “Moment Slab” to “Anchorage Slab” for consistency throughout this manual.

h. Figure 3.11.1.3-2: Increased max overhang height from 1'-0" to 1'-2" for easier constructability.
i. Figure 3.11.2-1 & Figure 3.11.3.2.1-5: Added 1'-0" dimension showing combined thickness of the wall and geocomposite wall drain.
j. Figure 3.11.3.3-1: Aligned the back of the concrete facing with the inside edge of the pile flange.
k. Figure 3.11.3.3-2:
    i. Increased overhang height from 2" to 4" minimum.
    ii. Increased overhang width from 4" to 5" minimum.
    iii. Increased PJF thickness from 1/2" to 1".
    iv. Added ** note to overhang height, showing limits of bond breaker.
    v. Revised the term “Moment Slab” to “Anchorage Slab” for consistency throughout this manual.
l. Figure 3.11.3.3-4: Clarified railing callout.
m. Figure 3.11.5.4-2: Labeled Bearing Plate.

12. Section 3.12:
    a. Section 3.12.1.2, Item 2 “Design Loading”: Modified since ABD with additional Design Loads notes for applicable NAW types.
    b. Section 3.12.1.2, Item 5 “General Plan and Elevation Sheets”:
        i. Category “Elevation Views”: Modified since ABD with additional plan note for ground mounted noise abatement walls (NAWs) where the plans show stair-stepped lines for the wall or ground line.
        ii. Category “Section Thru Wall”: Modified since ABD with additional items to be provided in the section view; a separate profile grade sketch shall also be provided.
    c. Figure 3.12-7: Updated vertical dimension handles of:
        i. 1’-4” anchor rods in parapet.
        ii. 6’-0” vertical parapet height.
    d. Figure 3.12-11: Corrected Section C-C arrows on Section B-B view.
13. Section 3.13:
    a. Figure 3.13.3-1: Updated * note.
14. Section 3.14:
    a. Revised the term “Slopewall” to “Slope Wall” for consistency with SSRBC.
    b. Section 3.14.1: Updated General Note.
    c. Figures 3.14.1-1 through 3: Added Construction Berm label
    d. Figure 3.14.4.1-3: Added approach slab; corrected Section B-B arrows; clarified bench geometry.
    e. Figure 3.14.5-1: Removed older flat bench geometry.
15. Section 3.15: Revised seismic terminologies for Bridge Operational Category (formerly Importance Category).

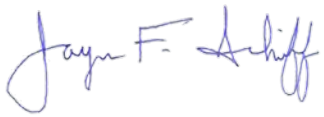
Appendix Updates

1. Figure A2.3: Revised description “SOLID PIERS” offset
2. Figure A2.5: Consolidated “FOR PARAPETS” figures; showed constant slope.
3. A3 Design Aids > Design Layout Examples – Top of Slab Elevations: Example Sheets moved to A.3.10.1 through A.3.10.5, and old border/fonts removed.

The Planning and Design Detail CADD Libraries, which reflect many of the figures in the Bridge Manual, will be issued soon to correspond to the 2025 Bridge Manual. Also, there are minor revisions necessary to the Guide Bridge Special Provisions (GBSPs) and the Design Guides which will follow soon. Questions regarding the 2025 Bridge Manual may be sent to DOT.BBS.COMSUGGEST@Illinois.gov .

Implementation

The 2025 Bridge Manual details and policies shall be effective for all projects with TSL's approved March 1, 2025, and beyond and may also be implemented for projects currently under development as determined by the District. The 2023 Bridge Manual will remain available on the IDOT website.



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PLP/BridgeManualChangeLetter25-01

Bridge Manual

Prepared by:

Bureau of Bridges & Structures
Office of Highways Project Implementation

Agency:

Illinois Department of Transportation

Place of Publication:

Springfield, Illinois

2025

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Section 1 Introduction

1.1 Bridge Policy

As directed by the Engineer of Bridges & Structures, it is the responsibility of the Engineer of Bridge Design to develop, maintain and administer the policies that govern the planning, design and preparation of plans and specifications for all structures under the jurisdiction of the Office of Highways Project Implementation and for structures under the jurisdiction of other agencies when Departmental approval of the project is required by State Statute.

Document Control and Revision

The Illinois Department of Transportation Bridge Manual is owned by the Office of Highways Project Implementation, Bureau of Bridges & Structures. The manual is reviewed during use for adequacy and updated by the Bureau of Bridges & Structures as necessary to reflect current policy. Interim policies that develop before the publication of the next Bridge Manual or supplement manuals are documented according to Section 1.1.3.

Distribution

This manual is intended to be used electronically as it includes hyperlinks within and resources external to the document. Portable Document Format (PDF) has been selected as the primary distribution format, and the official version of the manual is available on the Policy Center site.

The information contained in this manual is current as the date of issuance. Employees are responsible for ensuring use of the most current version of any document. All current policy documents are available on the Policy Center site.

Revision History

Refer to the “Bridge Manual Change Letter” at the front of this manual.

This Manual, supplemented by the Culvert Manual, Drainage Manual, Geotechnical Manual, Sign Structures Manual, Seismic Manual, and Structural Services Manual (all available at the primary Bridges & Structures Page), is the vehicle by which these policies are controlled. Presented herein is a compilation of design and plan presentation procedures, specification interpretations, standard practices, and details which constitute “policy”. While this manual attempts to unify and clarify bridge and structure

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policies performed by or for the IDOT Office of Highways Project Implementation, it does not preclude justifiable exceptions, subject to the approval of the Engineer of Bridges & Structures, provided these exceptions are based upon sound engineering principles. Good design practice will always require a combination of basic engineering principles, experience, and judgment in order to furnish the best possible structure, within reasonable economic limitations, to suit an individual site. The policies in this manual have been established primarily for application to typical highway structures using conventional construction methods. These polices are subject to re-examination and may not be applicable to long span, complex curved, or high clearance structures such as major river crossings or multi-level interchange complexes.

1.1.1 Applicable Bridge and Structure Design Specifications

The latest version of the AASHTO LRFD Bridge Design Specifications and the latest version of the AASHTO Guide Specifications for LRFD Seismic Bridge Design to the Design Specifications, combined with this manual and the aforementioned supplemental manuals shall be used to develop plans and specifications for new structures. See Section 2.1.2 for specification guidance on rehabilitation projects. If there is a discrepancy between the AASHTO LRFD Bridge Design Specifications and the Department policies, the Department policies shall govern.

1.1.2 Seismic Design of Bridges

To scope whether a bridge project qualifies for a force-based or displacement-based design approach, refer to the latest All Bridge Designer (ABD) Memoranda available at the primary Bridges & Structures Page. This manual contains Force-Based Seismic Design and Details; however, the Force-Based details are also suitable for Displacement-Based designs in SDC-A.

Displacement-Based Seismic Design

Refer to the IDOT Seismic Manual available at the primary Bridges & Structures Page.

Force-Based Seismic Design (previous policy when approved by Department)

The design of bridges to resist seismic loadings has become an increasing focus since the mid to late 1990's in Illinois. This has become more apparent with the adoption of the 1000 yr. design return period earthquake loading into the LRFD Code in 2008. The previous earthquake design loading was for a 500 yr. return period earthquake. The 500 yr. design return period earthquake as specified in the AASHTO Standard Specifications for Highway Bridges, Division I and IA (LFD), however, is still relevant primarily for bridges in Illinois which are undergoing seismic retrofit.

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The policies and details within this manual meet the minimum AASHTO requirements for Seismic Performance Zone 1 (LRFD) and Category A (LFD) with a low probability of being exceeded during the normal life expectancy for a bridge. Bridges and their components that are designed or retrofitted to resist Zone 1 or Category A forces and constructed in accordance with the design details contained in this manual should not experience total collapse, but may sustain repairable damage due to seismically induced ground shaking. Structures in Seismic Performance Zones 2, 3 & 4 (LRFD) or Categories B, C & D (LFD) will require additional analysis as per the appropriate AASHTO Specifications for Seismic Design. However, there should also be a low probability of collapse for structures in these Zones or Categories if the guidance given in this manual is followed.

1.1.3 Manual Updates, All Bridge Designer Memoranda (ABD), and Other Supplemental Electronic Documents

The Bureau of Bridges & Structures (BBS) Manuals are continually reviewed by the BBS Policies, Standards & Specifications Unit. Updates to the manuals are issued as frequently as needed. Interim updates are issued through memos posted on the internet. The most current manuals and information regarding IDOT bridge policy, documents and procedures are available at the primary Consultant Resources Page.

The following is an abbreviated list of Memoranda and Supplemental Electronic Documents available at the web page address given above:

1. All Bridge Designers Memoranda (ABD Memos)
2. All Geotechnical Manual User Memoranda (AGMU Memos)
3. Design Guides
4. Approved TSL Plans
5. CADD Cell Libraries, Base Sheets and Detailing Practices
6. Guide Bridge Special Provisions (GBSP)
7. Bridges & Structures Forms

ABD and AGMU Memos are directives from the Bureau of Bridges & Structures advising designers on policy changes. Typically, after a certain trial period, the policies implemented through these memos are eventually incorporated into the manuals.

Design Guides, Approved TSL plans, and CADD related drawings and documents are considered supplemental, but part of the Bridge Manual. These and other available documents are dated and revised as IDOT policy or the AASHTO LRFD Code changes.

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The IDOT BBS subscriptions service informs subscribers of changes and updates to information on the BBS web pages. Users of the BBS manuals should subscribe to this service which is available at the web page address given above.

Comments, suggestions, and questions from users of the BBS manuals should be sent to: dot.bbs.comsuggest@illinois.gov.

1.2 Organizations and Functions

The Bureau of Bridges & Structures is a part of the Office of Highways Project Implementation. The Engineer of Bridges & Structures, as head of the Bureau, is responsible for planning, developing and maintaining the State's bridge and structural engineering program, policies, specifications and standards which will facilitate the best utilization of resources for accomplishing the objectives of the Office of Highways Project Implementation. The Bureau of Bridges & Structures also provides the Project Implementation area of the Division with structural and geotechnical expertise during the construction phase of bridge projects.

To fulfill these responsibilities, the Bureau is organized as illustrated in the Organization Charts found on the Bureau's website at Contact Directory.

1.2.1 Services Development Section

This Section is responsible for managing the Bureau's operating budget, personnel and salary administration, business service activities, fiscal payment processing, typing services, file maintenance, and administrative staff support.

1.2.2 Bridge Planning Section

This Section is under the direct supervision of the Engineer of Bridge Planning. It is composed of three units that are responsible for project programming, and preliminary design of bridges and structures. These are the Bridge Planning Unit, Hydraulics Unit, and Foundations and Geotechnical Unit.

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1.2.2.1 Bridge Planning Unit

Under the supervision of the Unit Chief, each Planning Group is responsible for project programming and monitoring, conducting special engineering studies and reports, and the analysis and approval of man hour requirements for structural engineering/consulting engineering agreements. They review and approve Bridge Condition Reports (BCR), prepare Type, Size and Location (TSL) Plans, and review those prepared by consultants. The BCR details the scope of work for a bridge project and is utilized in the District Project Report to secure design approval. The TSL plan documents the fundamental features of the structure and is used to obtain final preliminary approval of the details for the basic project parameters and utility assessments required for the final design.

The Bridge Planning Unit also prepares detailed economic evaluations of alternate structure types and configurations, conducts structural analyses and aesthetic studies, and formulates the basic type of proposed structures utilizing current State and Federal design policies.

1.2.2.2 Hydraulics Unit

Under the supervision of the Unit Chief, the Hydraulics Unit is responsible for the review and approval of the Hydraulic Report (HR) for bridge projects and the Pump Station Hydraulic Report (PSHR) for storm water pumping stations. For bridge projects, the approval means the waterway opening properly addresses policy and practice controlled in the IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) and satisfies any external regulatory requirements. Recommendations generated from Hydraulic Unit approvals are provided to the Bridge Planning Unit and the Foundations and Geotechnical Unit in support of TSL plan development. The Hydraulics Unit assists both Units in further refinement of preliminary structure details relating to hydraulics, such as bridge alignment\skew, substructure placement, low beam elevation and scour countermeasures. Beyond this involvement in TSL plan development, the unit assists the Bridge Design Section and other Central Office Bureaus within the Plan, Special Provision & Estimates (PS&E) process. The Hydraulics Unit also assists the Bridge Investigations and Repair Plans Unit with review and preparation of bridge scour countermeasure plans.

The unit is also responsible for the development and implementation of all drainage policy, practice and technical hydrologic and hydraulic (H&H) procedures utilized by the Department. This policy is disseminated through the IDOT Drainage Manual. The Drainage Manual is the primary reference for H&H studies that include, in addition to structure Hydraulic Reports; Location Drainage Studies for roadway improvements, storm drain and detention analysis, and bridge scour evaluation\countermeasure design and drainage connection permits. The Hydraulics Unit provides technical support of these District

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efforts, and also disseminates new methodologies, FHWA activities, research products, and software investigations relating to hydraulics. Technical support is delivered via statewide meetings\informal training sessions such as the Annual Hydraulic Engineers Meeting (AHEM), which is organized by the Hydraulics Unit. The unit also delivers or coordinates formal H&H training for IDOT, Local Agency and consulting personnel through the Bureau of Employee Services, NHI and other agencies. The Hydraulics Unit Chief rates Statement of Experience and Financial Condition (SEFC) submittals from consulting firms seeking pre-qualification for IDOT hydraulic work and also is charged with national IDOT representation on several AASHTO, FHWA and NCHRP panels. Finally, the unit creates and leads the IDOT technical review panel for hydraulic research projects initiated by our primary research arm, the Illinois Center for Transportation (ICT).

1.2.2.3 Foundations and Geotechnical Unit

Under the supervision of the Unit Chief, the Foundations and Geotechnical Unit provides a wide spectrum of geotechnical services to the Central Bureaus and Districts within the Department, consultants or contractors working for the Department, other State and Local Agencies, academia, and other DOTs and national organizations outside the State.

The unit provides geotechnical services during all project phases of planning, design and construction. In the planning phase, the unit assesses proper subsurface exploration and geotechnical recommendations, which are contained in the Structure Geotechnical Report (SGR), appropriate for use in finalizing the TSL and developing the final design of bridges, retaining walls, and multiple box culverts. This is accomplished by either preparing or providing approvals of district's or consultant's SGR's during TSL reviews. In the design phase, the unit assists the design section with retaining wall, drilled shaft, repair plan, traffic signal, light tower, and sign structure foundation design. In the construction phase, the Foundations and Geotechnical Unit provides special provisions, assists in resolution of construction issues and review of construction submittals including temporary soil retention, tieback design, drilled shaft installation, and value engineering. The unit conducts forensic investigations, foundation underpinning/repairs, and subgrade or slope failure retrofit designs. The unit assists with maintenance and updating of the Department's IDOT Geotechnical Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical) as well as updating all geotechnical/foundation aspects of the Bridge Manual. The unit also serves as a conduit for technology transfer by providing various training classes to Districts and their consultants/contractors, facilitating and organizing seminars and conferences, and disseminating innovative research findings to all geotechnical personnel working for the Department (Districts and consultants). In addition to geotechnical research studies on structures, it serves on various State and national technical review panels/committees overseeing sponsored research projects.

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1.2.3 Bridge Design Section

This Section is under the direct supervision of the Engineer of Bridge Design. It is composed of four units. These are the In-House Bridge Design and Review Unit; Consultant Bridge Design and Construction Review Unit; the Policy, Standards, and Final Plan Control Unit; and the Sign Structures, Shop Drawings and Steel Fabrication Inspection Unit. The Bridge Design Section is responsible for the design and preparation of bridge and structure plans for the Department; and the initiation, development and dissemination of design policies, procedures and structural theories to be used in the selection, proportioning, and detailing of members and components employed in any bridge or structure type on Illinois' highway system. In addition, it is responsible for the development of specifications and special provisions for all materials and procedures as they relate to the use and application in bridges and structures; the preparation of bridge and structure estimates of cost and time; the evaluation and utilization of new structure types, products, techniques and materials; and the review and resolution of structural construction submittals. The Design Section is also responsible for the review and approval of shop plans as well as steel fabrication inspections.

1.2.3.1 Policy, Standards, and Final Plan Control Unit

Under the supervision of the Unit Chief, this unit develops and maintains all bridge and structure design policies including the Bridge Manual and Culvert Manual, Design Guides, and Base Sheets. In addition, the unit analyzes, reviews and develops standards for special structures, highway appurtenances, and specialized design and construction concepts developed by outside agencies. It designs new specialized structure components utilized by the Bridge Design and Review Units in structural plan preparation; seeks out, evaluates and develops policy and guidelines for implementation of new design and construction techniques, products and materials; monitors structure related Standard Specifications for revision of the Standard Specifications for Road and Bridge Construction; reviews all mis-fabricated and damaged precast prestressed beams; assists CBM with the continued development of the Manual for Fabrication of Precast Prestressed Concrete Products; reviews, approves and maintains the lists of prequalified structural systems (PQSS); and develops the Bureau's technical programs. Technical programs include bridge and structural engineering software, and computer systems. To help accomplish these tasks this unit and other units serve or assist on numerous technical review panels, AASHTO committees, ICT and other research projects. The Unit Chief represents the Bureau on the Department's Specification Committee.

The Policy, Standards, and Final Plan Control Unit also conducts final reviews which ensure compatibility of roadway and bridge plans; evaluates, develops and recommends approval of bridge special provisions; serves on the paint committee; manages the Department's paint contracts, and prepares cost and time

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estimates for new bridge designs, products or construction methods. In order to comply with Federal Regulations and to ensure that the Department obtains a proportional share of Federal funds, the unit also tracks and assembles bridge construction cost information for submittal to the Federal Highway Administration.

1.2.3.2 Bridge Design and Review Units

The Bridge Design and Review Units consist of the Consultant Bridge Design and Construction Review Unit and the In-House Bridge Design and Review Unit. Under the supervision of the Unit Chiefs, these units perform the analysis and evaluation of structural designs; develops and prepares bridge and structural plans for use on the State Highway System; and performs the analysis, evaluation and approval of final contract plans for bridges and structures prepared for the Department or Local Agencies by consultants. The Consultant Bridge Design and Construction Review Unit additionally provides guidance to Department personnel, consultants, and other agencies for the development of structure contract plans and specifications; reviews structural construction submittals, and reviews, evaluates and assists in the resolution of structure construction problems and corrective actions; develops policies and procedures for design and plan preparation by outside agencies or consultants for structures on the State Highway System; and evaluates consultant performance on Department managed structural projects.

The Bridge Design & Construction Review Unit Chief assists the Design and Review Units, Policy Standards and Specifications Unit, and the Shop Drawing & Steel Fabrication Inspection Unit on construction means, methods, procedures, and practices on matters related to structure construction, fabrication, specifications, and policy.

1.2.3.3 Sign Structures, Shop Drawings & Steel Fabrication Inspection Unit

Under the supervision of the Unit Chief, this unit is responsible for the review and approval of Shop Drawings covering steel, aluminum, and prestressed concrete elements for bridge and sign structures as well as structural evaluation of Shop Drawings for precast box culverts, three sided precast concrete structures and noise walls. The unit provides shop quality assurance services during the fabrication, painting and non-destructive testing of structural steel bridge components and aluminum sign structures. The unit also acts as adviser in matters associated with fabrication and non-destructive testing of steel and aluminum. In addition, it is responsible for the development and maintenance of the Sign Structures Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Fabrication) which covers design policy, plan standards, inspection of sign structures, and structural evaluation of light towers and other special traffic structures.

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1.2.4 Structural Services Section

This Section is under the direct supervision of the Engineer of Structural Services. It is composed of four units. These are the Bridge Investigations and Repair Plans Unit; the Local Bridge Unit; the Structural Ratings and Permits Unit; and the Bridge Management and Inspection Unit. These units are responsible for the structural investigations of existing bridges, the development or review of repair plans, determination of bridge load-carrying capacity, establishment of posted weight limits, evaluation of overweight permit vehicle movements, maintenance of bridge plan archives, review and approval of Local Agency bridge construction projects, and managing the inventory and inspection of the State's bridges.

1.2.4.1 Bridge Investigations and Repair Plans Unit

Under the supervision of the Unit Chief, this unit performs field investigations to identify the cause or extent of structural deficiencies, develops repair alternatives, and prepares plans to correct deficiencies related to the deterioration of structural members or accidental damage. Field investigations performed by the unit also include those to evaluate reoccurring deficiencies associated with standard structural detailing practices to identify the elements contributing to the deficiencies and to offer solutions. The unit provides guidance to Department personnel, consulting engineers and other agencies engaged in the development of repair or maintenance projects, and this guidance is provided on a project specific basis and through the maintenance and updating of information contained in the Repair Section of the Structural Services Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Repairs), for which the unit is responsible. Repair, maintenance, and minor bridge rehabilitation projects prepared by Department personnel or consulting engineers are reviewed by the unit for comment and approval prior to being accepted for advertisement as a contract for letting. The unit also provides assistance to Department implementation personnel as required to resolve construction issues during the implementation of the projects reviewed or prepared by the unit. An inventory of Contract plans and As-Built plans is actively maintained by the unit for all bridges directly under Department jurisdiction for reference during bridge maintenance, repair or rehabilitation projects. Additional information related to unit procedures may be found in the Repair Section of the Structural Services Manual.

1.2.4.2 Local Bridge Unit

Under the supervision of the Unit Chief, this unit provides administrative and technical expertise to Local Agencies concerning local bridge matters and assists the Bureau of Local Roads & Streets during the development of policies and procedures for Local Agency bridges. All Local Agency bridge rehabilitation and replacement projects utilizing Federal, State or motor fuel tax funds, and other local projects requiring Department approval by State Statutes, are reviewed by the unit during project development to the

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degree necessary to ensure structural adequacy and compliance with Department policies and procedures. As part of the project review, the unit provides coordination with the Department of Natural Resources, using information received from the Local Agency, to obtain approvals for local bridge projects to proceed to letting. The unit provides services to Counties, as required by State Statute, leading to the development of Contract plans for bridge construction.

Unit personnel conduct field inspections and perform analyses to determine the load-carrying capacity of existing bridges in response to Local Agency requests or to address changes in bridge conditions routinely reported by Local Agency inspection staff. The unit establishes weight limits to be enforced for Local Agency bridges to ensure highway safety and assists agencies in developing repairs to improve the condition of Local Agency bridges or to avoid the implementation of weight limits. Local Agencies coordinate with the unit to resolve construction issues and to evaluate permit requests for overweight vehicles. Additional information related to unit procedures may be found in the Bureau of Local Roads & Streets Manual available at the primary LPA Resources Page.

1.2.4.3 Structural Ratings and Permits Unit

Under the supervision of the Unit Chief, this unit performs analysis and evaluations to determine the load-carrying capacity of new and existing bridges under Department jurisdiction, as required by State Statute and Federal Regulations. When necessary, unit personnel perform field inspections of severely damaged or deteriorated bridges to obtain information regarding essential bridge elements for use in evaluating load-carrying capacity. When necessary to address existing structural conditions, the unit issues directives to place weight restrictions on existing bridges under State jurisdiction to ensure highway safety. The unit works cooperatively with the other units of the Structural Services Section for identifying repair alternatives to correct deficiencies that would otherwise require the implementation of a weight restriction. The Bureau of Operations routinely coordinates the review of overweight permit requests with the unit prior to authorizing the movement of overweight vehicles to ensure that highway infrastructure is not damaged. In order to ensure that bridge load-carrying capacities can be determined in an expeditious manner, the unit maintains databases of structural information for use during the evaluation of overweight permit vehicle movements or the effect of damage or deterioration on bridge load-carrying capacity. The movement of heavy construction equipment across existing bridges to facilitate construction projects is evaluated by this unit, as well as the feasibility of placing additional wearing surface on existing bridges located within the limits of roadway resurfacing projects. The unit reviews proposed legislative changes to the Illinois Vehicle Code to determine the effect of the changes on highway system bridges, and provides coordination with other Units and Bureaus for developing comments in regard to anticipated effects. Unit personnel assist the Office of Planning and Programming during the maintenance and updating of the Structure Information and Procedure Manual (at the primary Bridges & Structures Page

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> Specific Scope of Services **>** Bridge Management and Inspections) to ensure compliance with the bridge safety and inventory provisions of the National Bridge Inspection Standards, and represents the Department in matters pertaining to the maintenance, revision or updating of bridge rating specifications that may be proposed by the American Association of State Highway and Transportation Officials (AASHTO). Additional information related to unit procedures may be found in the Ratings and Permits Section of the Structural Services Manual.

1.2.4.4 Bridge Management and Inspection Unit

Under the supervision of the Unit Chief, this unit provides oversight of the bridge inspection procedures utilized by the Department and Local Agencies to ensure bridge safety as required by the National Bridge Inspection Standards provided in Title 23 of the Code of Federal Regulations, Part 650.301. Unit personnel assist the Office of Planning and Programming during the maintenance and updating of the Structure Information and Procedure Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Bridge Management and Inspections) to ensure compliance with the bridge safety and inventory provisions of the National Bridge Inspection Standards for Local Agency bridges. This unit also manages the Illinois bridge and tunnel inventory to ensure that needed repairs are accomplished in a timely manner to maximize the lifespan of all structures in accordance with Federal Regulations; works to ensure the Illinois Bridge Inspection Program is in compliance with the FHWA 23 Metrics; reviews Improvement Plans and Plans of Corrective Action written to meet compliance with FHWA; and performs quality assurance reviews of other agencies to ensure that local agency inspection programs meet the standards of the NBIS. These other agencies include the Illinois State Toll Highway Authority, county, municipal and other state government agencies. To accomplish these tasks the unit interacts routinely with FHWA, other Units and Bureaus, central headquarters offices, District bridge inspection staff and Local Agencies that have bridge inspection responsibilities.

The Unit Chief is the Statewide Program Manager for bridge inspections in Illinois and is responsible for setting bridge inspection policy to ensure bridges serving roadways in Illinois remain safe for all users of the highway system. The Statewide Program Manager approves inspection personnel to perform inspections or manage inspection programs in the State of Illinois. This unit ensures that all personnel involved in bridge inspections are appropriately trained; schedules training classes for state personnel and consultants performing Illinois bridge inspections; and is responsible for the development and maintenance of Section 3 of the Structural Services Manual which summarizes the official bridge inspection policies for the State of Illinois. The unit is also the Bureau representative for AASHTOWare Bridge Management (BrM) software. Additional information related to unit procedures may be found in the Inspection Section of the Structural Services Manual.

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1.3 Preparation of Bridge and Structure Plans

The preparation of State Bridge and Structure plans begin by the submittal of the Structure Report, Bridge Condition Report and where applicable, the Hydraulic Report. The information contained in these reports is the basis upon which the structure is hydraulically and geometrically proportioned. Sufficient data shall be furnished to fully delineate all field conditions. Particular care shall be taken to supply complete information on existing structures planned to be incorporated into the plans. The authority to proceed with subsurface investigations and preparation of the Structure Geotechnical Report (SGR) is given after the Bridge Condition Report is approved. The subsurface investigation results and the SGR shall then be submitted to the Bureau of Bridges & Structures (BBS) with the Type, Size & Location (TSL) plans.

The TSL is prepared from the data furnished in the Structure, Bridge Condition, Hydraulic and Structure Geotechnical Reports and illustrates the general plan and elevation of the structure, including descriptions and treatments of the general structure components. It is presented to the Regional Engineer and to other agencies for their concurrence in relation to overall features of the structure. In addition to the Regional and District Offices, approval from other agencies, when required, include the U.S. Department of Transportation, the Federal Highway Administration, the Illinois Department of Natural Resources, the U.S. Army Corp of Engineers, railroads, utilities, and the U.S. Coast Guard (when navigable streams are affected).

The Plan Development Outline (PDO), as detailed in Section 3.1.13, presents key design elements, criteria, deliverables and schedule information related to the specific structure plans, to help ensure the project structure Final Plan development stays on scope and schedule. The PDO shall be prepared and submitted after the TSL is approved and early in the plan development phase.

The Final Design plans constitute the single most important document necessary for the construction of structures. They provide dimensions, details, and material strengths for all structure components. The Final plans should agree with the approved TSL, SGR and PDO. If it is found necessary to deviate from the TSL plan during design, prior approval shall be obtained from the Engineer of Bridges & Structures before such change can be incorporated. Since any deviation could involve concurrence by other applicable agencies, sufficient time shall be allowed for processing.

After the Final plans are completed, they shall be submitted to the Engineer of Bridges & Structures for approval and signature to denote acceptance of the plans. The plans are then stored until placed on contract. During this time, if policy changes dictate, the plans may be updated when practical or necessary in accordance with current design policy and then will be transmitted to the Project Development and Implementation Section of the Bureau of Design & Environment for contract

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processing. The process for consultant prepared Final Plans is slightly different. In these cases, the Final Plans are held at the District until placed on contract letting, then updated if necessary and submitted to BBS for final signature and letting. The process for preparation of preliminary submittal and Final plans for Local Agency structures is similar, and submittals are processed through the Bureau of Local Roads & Streets to the Local Bridge Unit. See the Bureau of Local Roads & Streets Manual available at the primary LPA Resources Page for guidelines.

1.4 Consulting Engineers

Consulting engineers may be retained by the Districts for the design of bridges and other structures. The consultant shall be prepared to undertake all of the necessary tasks required for the production of the Final plans as per the standard of practice and in conformance with the policies and requirements of the Department. These tasks include: field site investigation, preparation of the Structure Report, preparation of the Bridge Condition Report, hydraulic survey and preparation of the Hydraulic Report, subsurface investigation and preparation of the Structure Geotechnical Report, economic studies, bridge and wall type selection studies, preparation of the TSL, PDO, Final Plans, PS&E and Shop Plan review. During the construction phase of the project, it shall be the consultant's responsibility to resolve all construction issues resulting from Design plan errors or inconsistencies.

Included in this manual are guidelines and requirements to assist with the development of the TSL, PDO and Final plans. Adherence to these guidelines will help to facilitate expeditious review and approval of plans and minimize last minute changes and delays. The guidelines are presented in the following manner:

1. Checklist for preparation of Type Size and Location (TSL) Plans (See Section 2.3)
2. Plan Development Outline (PDO) Guidelines (See Section 3.1)
3. Checklist for preparation of Final Plans (See Section 3.1)

Structure plans prepared by design consultants for the Department shall be approved by the Engineer of Bridges & Structures prior to letting for construction. To accomplish this, the Bureau of Bridges & Structures requires specific submittals for review and approval. These submittals include a Plan Development Outline, Final Structure Plans for structural review and approval, and the Final Plans and Specifications for letting.

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1.4.1 Quality Verification Statement

A Quality Verification Statement and requested documentation (Figure 1.4.1-1) shall be completed by the consultant and shall accompany all "Final Structure Plans" for State projects. "Final Structure Plans" are defined as the first submittal of the Structure Contract plans, including Special Provisions, to the Bureau of Bridges & Structures for review.

The Quality Verification Statement attests that the plans prepared by consultants are completed by the firm and checked prior to first submittal to the Bureau of Bridges & Structures. It is intended to emphasize that the responsibility for ensuring plan quality rests with the consulting firm, and to give the Bureau of Bridges & Structures a confidence level that the firm has completed all the necessary work for a structurally safe, cost efficient, and well-detailed structure conforming to Department requirements and policies.

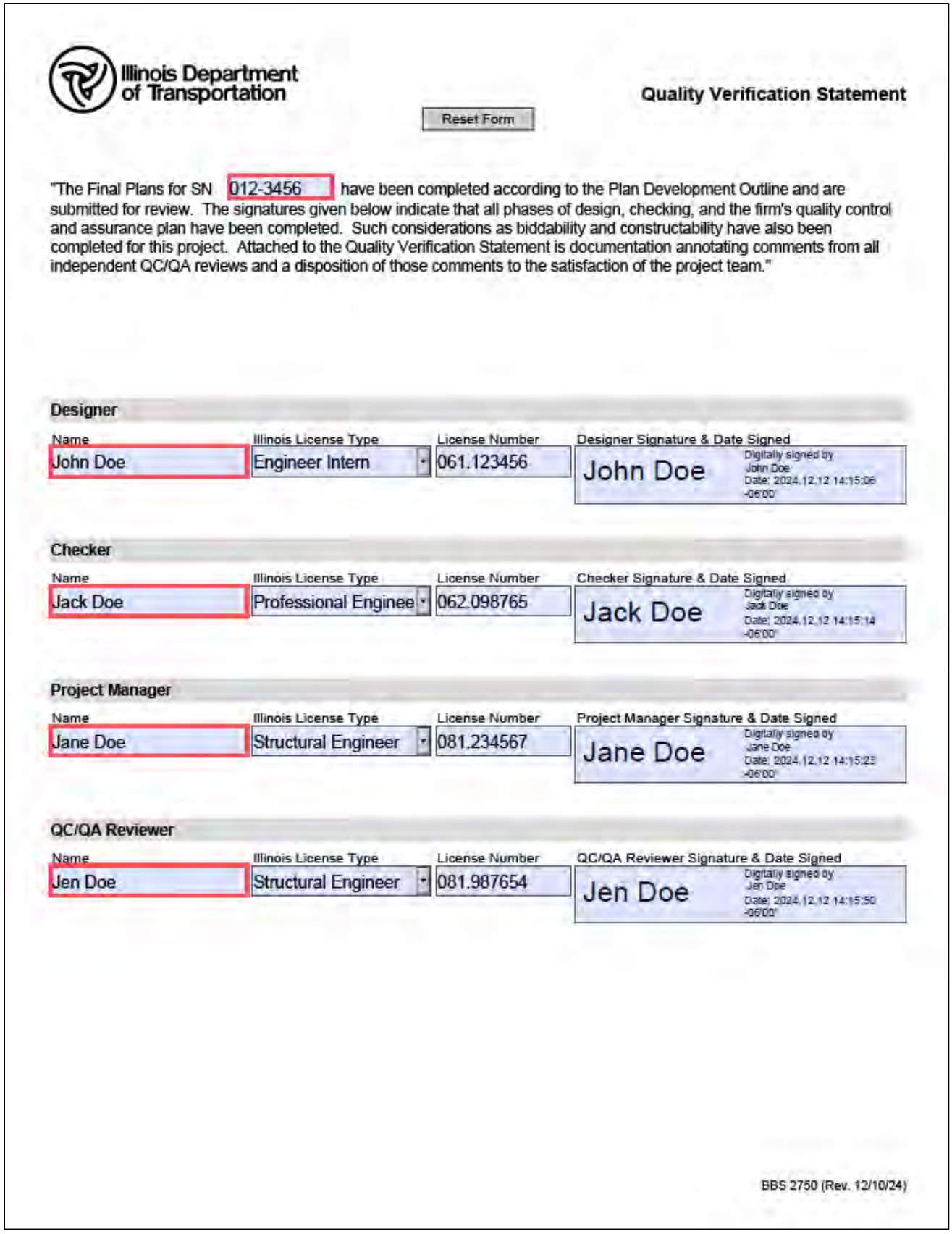
As a minimum, the Bureau of Bridges & Structures will review the Plan Development Outline, perform a rating analysis of the main load-carrying members for capacity verification purposes only, and review the pay items, notes, and Special Provisions for biddability. Some projects will be further reviewed for structural adequacy (splices, shear studs, bearings, substructure units, etc.) and adequate detailing at the discretion of the Bureau of Bridges & Structures.

Since the review and processing of Local Agency (LA) bridge projects varies from project to project, the Quality Verification Statement will not be required for most LA projects. The Quality Verification Statement will not be required for LA projects when the bridge plans are accepted by the Department based upon certification in accordance with the Bureau of Local Roads & Streets Manual (available at the primary LPA Resources Page) Section 11-7.03.1, or Section 23-7.02.1 for Federal Aid Projects. The Quality Verification Statement shall be submitted with bridge plans for LA projects that cannot be accepted based on certification, and are therefore scheduled to be reviewed by the Bureau of Bridges & Structures prior to acceptance by the Department.

The QC/QA review shall be completed by an Illinois Licensed Structural Engineer and shall not be the same engineer as the designer, checker, or sealer of the plans.

The quality verification statement form BBS 2750 is available for printing online at the primary Forms Page.

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Illinois Department of Transportation

Quality Verification Statement

Reset Form

"The Final Plans for SN 012-3456 have been completed according to the Plan Development Outline and are submitted for review. The signatures given below indicate that all phases of design, checking, and the firm's quality control and assurance plan have been completed. Such considerations as biddability and constructability have also been completed for this project. Attached to the Quality Verification Statement is documentation annotating comments from all independent QC/QA reviews and a disposition of those comments to the satisfaction of the project team."

Designer

Name	Illinois License Type	License Number	Designer Signature & Date Signed
John Doe	Engineer Intern	061.123456	John Doe Digitally signed by John Doe Date: 2024.12.12 14:15:06 -06'00'

Checker

Name	Illinois License Type	License Number	Checker Signature & Date Signed
Jack Doe	Professional Enginee	062.098765	Jack Doe Digitally signed by Jack Doe Date: 2024.12.12 14:15:14 -06'00'

Project Manager

Name	Illinois License Type	License Number	Project Manager Signature & Date Signed
Jane Doe	Structural Engineer	081.234567	Jane Doe Digitally signed by Jane Doe Date: 2024.12.12 14:15:23 -06'00'

QC/QA Reviewer

Name	Illinois License Type	License Number	QC/QA Reviewer Signature & Date Signed
Jen Doe	Structural Engineer	081.987654	Jen Doe Digitally signed by Jen Doe Date: 2024.12.12 14:15:50 -06'00'

BBS 2750 (Rev. 12/10/24)

Figure 1.4.1-1

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Section 2 Planning

2.1 General Planning Process

The planning processes in this manual are associated with bridges/structures and the bridge approach elements to establish a structure type, size, and location as the basis for final design plan preparation.

2.1.1 Goal and Tasks

The bridge and structure planning process encompasses the evaluation of site information, the application of established policies and practices, and the consideration of alternates and their respective economic evaluations for the purpose of establishing the most appropriate bridge or structure configuration. Specific factors such as cost, safety, function, hydraulics, geotechnical, geometric, structural and aesthetics are all considered in this process.

There are several primary tasks involved in the planning process of a bridge or structure. The responsibility for completing individual tasks falls to either an IDOT District Office, a consultant hired by the Department, or the Bureau of Bridges & Structures in various combinations. The Planning Section of the Bridge Manual is a guide for policy and the preparation of the primary tasks which include:

1) Bridge Condition Report (BCR)
2) Structure Geotechnical Report (SGR)
3) Hydraulic Report (HR)
4) Type, Size and Location Plan (TSL)
5) Utility attachments

Additional Manuals and guides may also be found in documents issued by other Bureaus as referenced.

The Planning Section also provides input to the Districts or the Bureau of Design & Environment (BDE) for:

1. Consultant Man Hour Evaluation
2. Consultant Prequalification
3. Project Programming

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2.1.2 Design Specifications

The latest version of the AASHTO LRFD Bridge Design Specifications and the latest version of the AASHTO Guide Specifications for LRFD Seismic Bridge Design to the Design Specifications are the primary design codes for new structures. The AASHTO LRFD Bridge Design Specifications and the FHWA Seismic Retrofitting Manual for Highway Structures (2006) are the primary design codes for all ,reconstructed, and rehabilitated structures such as deck replacement, structure widenings and extensions. For structure rehabilitation projects, if the LRFD Specifications cannot be satisfied by either the existing beams or by the existing substructure being re-used, the design specifications of the existing structure shall be utilized with approval by the BBS. The design specification for a particular project shall be clearly indicated on the TSL plan and other pertinent planning documents. Any deviations will require written approval from the Engineer of Bridges & Structures. See Section 1.1 for additional information. New construction, reconstruction or rehabilitation projects shall use the 1000 yr. seismic data as discussed in Section 2.3.

2.1.3 Context Sensitive Solutions

Context Sensitive Solutions (CSS) is an interdisciplinary approach that seeks effective multimodal transportation solutions by working with stakeholders to develop, build and maintain cost effective transportation facilities which fit into and reflect a project's surroundings – its "context". Chapter 19 of the Bureau of Design & Environment Manual contains guidelines and polices for implementation of CSS on highway and structure projects. The CSS process is administered by the District and may be found at the primary CSS Page.

2.1.4 Structure Types

There are several main types of structures which require the Bureau of Bridges & Structures' (BBS) involvement and approval. These are:

1. Bridges
2. Multi-Barrel Box Culverts
    a. Cast-In-Place (CIP)
    b. Precast
3. 3-Sided Precast Concrete Structures
4. Walls
    a. Retaining
    b. Noise
5. Pump Stations
6. Sign Structures
7. Corrugated Structural Plate Structures

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2.1.5 Planning Tasks

2.1.5.1 Bridge Condition Report (BCR)

Bridge Condition Reports are intended to provide a format for Districts to document a proposed scope-of-work for an existing structure to the Bureau of Bridges & Structures. A BCR may be completed by District Personnel or a consultant hired by the Department. The BCR provides clearly documented communication between the Bureau of Bridges & Structures and the Districts or its consultants.

The BCR documents a bridge or structure's current physical condition and functionality. It also addresses structural and safety deficiencies, and finally proposes a scope-of-work. All pertinent information which is required to support the proposed scope-of-work is contained in a BCR.

A complete guide to compiling a BCR can be found in the IDOT document "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning. Requirements for BCR's for Local Agency projects are similar, and follow the requirements of Sections 10-2.03(b) and 22-2.06(a) of the Bureau of Local Roads & Streets Manual available at the primary LPA Resources Page. See also Section 2.2 for further discussion of BCRs.

2.1.5.2 Type Size and Location (TSL) Plan

The Type Size and Location plan forms the basis for final plan preparation, which is used for construction of the structure. TSLs are also used to obtain an agreement between a District or its consultant and the BBS along with other applicable parties such as railroads, the Federal Highway Administration (FHWA), the US Coast Guard (USCG), the Illinois Department of Natural Resources (IDNR) and its Historic Preservation Division (formerly the Illinois Historic Preservation Agency), the Army Corp of Engineers, municipalities and private developers. Additional approval process information is given in Section 2.3.

Bridge or structure type, size and location are established under the principles of overall project economy and safety and are subject to the various site factors and conditions unique to a project. Consequently, detailed structure configurations are based upon comprehensive geometric, hydraulic, geotechnical, structural, aesthetic and economic analyses.

TSL Guidelines, policies, and samples are given in Section 2.3.

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Requirements for TSL submittals for Local Agency projects are similar to those for State projects, and follow the requirements of Section 10-2.03(c) of the Bureau of Local Roads & Streets Manual available at the primary LPA Resources Page.

2.1.5.3 Structure Geotechnical Report (SGR) and Hydraulic Report (HR)

The Structure Geotechnical Report provides the engineers responsible for development of the TSL and the design plans with the geotechnical information and recommendations needed to plan and design the foundations for a specific structure. See Section 2.3 and the IDOT Geotechnical Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical) for more information and guidance on developing and compiling SGRs.

The Hydraulic Report plays an important role in determining the scope of a project. Hydraulic issues, such as scour, estimated water surface elevations and the waterway information table are also addressed in the HR. See Sections 2.2 and 2.3 along with the IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) for more information and guidance on developing and compiling HRs.

2.1.5.4 Consultant Man Hour Evaluations

Consultant man hour estimates shall be submitted to the District. The BBS reviews structure related man hours as requested by the district; however, estimates for BCRs, retaining walls less than 7 feet of exposed height, and single cell box culverts less than 12 feet are reviewed by the Districts. The BBS will be available for assistance upon request.

There are two options for which the BBS can have involvement in consultant man hour estimates for TSL plans and Structure plans. These are:

Option 1: If a time constraint is given to have an agreement signed, a District can request the BBS to provide an independent estimate prior to the selection of a consultant by the Department through the Professional Transportation Bulletin (PTB) selection process. This option allows the Districts to negotiate directly with the consultant in a timely manner for a set of agreed upon man hours. The BBS may be contacted during the negotiation phase to help resolve any questions. The District is asked to send the original consultant estimate to the BBS prior to negotiation in order to keep BBS records current.

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Option 2: If a District sends a consultant estimate to the BBS for review and comment, the BBS will perform an independent estimate. Comparisons will be made with previous projects to ensure similar scopes-of-work were assumed. The BBS will then forward its recommended estimate to the District. The District negotiates and discusses any discrepancies with the consultant in order to arrive at an agreed upon set of man hours. The BBS may be contacted during this negotiation phase to help resolve any questions.

Additional notes that may apply to the two options above are:

1. Estimates for the Structure Geotechnical Report should be coordinated through the District Geotechnical Engineer. The BBS Foundations and Geotechnical Unit may be contacted for assistance.
2. QC/QA, and Management and Administration estimates are related to the entire project and therefore should be reviewed by the District.
3. Estimates should be separated into specific tasks. I.e., TSL plan, PDO, Structure plan, Shop Drawings, etc. Structure plan estimates should include an itemized breakdown.
4. It may be necessary to defer the Structure plan estimate until after the TSL plan is approved and the structure type (or scope-of-work) is more clearly defined. Structure plan estimates are then negotiated as a supplement.

2.1.5.5 Utility Attachments

Utility attachments to bridges and structures require approval from the Regional Engineer. Applications for a permit are then submitted to the Central Bureau of Operations for review of compliance with policy and method of attachment. Utility companies who wish to attach their facilities to traffic structures under the jurisdiction of the Department are subject to assessment charges. If the Central Bureau of Operations approves the method of attachment, the BBS Planning Section will conduct a structural feasibility analysis and compute the assessment charge. Guidelines and policies for utility attachment to structures are given in Section 2.5.

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2.1.6 Responsibilities

2.1.6.1 District

The District is responsible for providing the Bridge Condition Report (BCR), and the Structure Geotechnical Report (SGR) to the BBS for review and approval. The District reviews TSL plans for conformance within various Bureaus, Project Report agreements, Roadway plans, and other non-structural project requirements. In addition, the District is responsible for directing and supervising work and man hours performed by its consultants which include those that compile Structure Geotechnical and Hydraulic Reports (with guidance from the BBS as needed).

2.1.6.2 Consultant

Consultants may be hired by the Department to help the District compile Bridge Condition Reports, Hydraulic Reports and Structure Geotechnical Reports and/or to develop Type,Size and Location plans. In addition to specific guidance provided by the Districts, consultants are required to follow the policies and procedures of relevant IDOT documents as well as the appropriate AASHTO or AREMA Design Specifications. In particular, a consultant should rely on the guidelines and polices referenced in Section 2.1 and this manual.

2.1.6.3 Bureau of Bridges & Structures Planning Section

Overall, the Bureau of Bridges & Structures Planning Section responsibilities vary from production to oversight and guidance. The BBS Planning Section provides oversight and approval for all BCR's, SGR's and consultant developed TSL plans submitted by the Districts. Approvals or rejections are documented by memorandum with recommendations/revisions as required. Also, the BBS Planning Section can be called upon by the Districts to develop In-house TSL plans. See Section 2.3 for more details regarding the development and approval of the TSL plan process. The Foundations and Geotechnical Unit of the BBS reviews and approves all SGR's and compiles them for in-house projects. The BBS Planning Section is available upon request for guidance and interpretation of various design specifications and Departmental policies and procedures.

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2.1.7 Reference Manuals

In addition to this manual and the AASHTO "LRFD Bridge Design Specifications," familiarity with the following manuals and documents is necessary to properly develop a BCR, SGR, Hydraulic Report, and TSL plan for a structure over which the State has review authority:

- Bureau of Bridges & Structures (primary Bridges & Structures Page)
    - Bridge Condition Report Procedures and Practices
    - Seismic Manual
    - Geotechnical Manual
    - Drainage Manual
    - Culvert Manual
    - Structural Services Manual
    - Sign Structures Manual
    - Memoranda to All Bridge Designers (ABD)
    - Memoranda to All Geotechnical Manual Users (AGMU)
- Bureau of Design & Environment (primary Highways Page)
    - Standard Specifications for Road and Bridge Construction
    - Supplemental Specifications and Recurring Special Provisions
    - Bureau of Design & Environment Manual
    - Highway Standards Manual
    - Memoranda to All Regional Engineers
- Bureau of Local Roads & Streets (primary LPA Resources Page)
    - Bureau of Local Roads & Streets Manual
- Other Federal/National Documents
    - FWHA Retrofitting Manual for Highway Bridges
    - AREMA Manual for Railway Engineering
    - AASHTO Guide Specification for Bridge Railings
    - AASHTO Policy on Geometric Design of Highways and Streets
    - AASHTO Guide Specification for Fatigue Evaluation of Existing Steel Bridges
    - AASHTO Guide for the Development of Bicycle Facilities
    - NCHRP 341 Guidelines for the Use of Weathered Steel
    - AASHTO Guide Specifications for LRFD Seismic Bridge Design
    - AASHTO Roadside Design Guide

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These reference materials form the basic criteria which control all BCR's, SGR's, Hydraulic Reports and TSL plans prepared for the State. With the exception of the documents published by AASHTO, AREMA, NCHRP and FHWA, the references above can all be obtained from the issuing Bureau or from the IDOT at the primary Doing Business Page.

An "exception to policy" shall be secured from the appropriate Bureau before any design or detail outside the guides and controls of the referenced policy manuals may be utilized on any TSL plan.

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2.2 Bridge Condition Reports

2.2.1 Definition

The purpose of a BCR is to establish a scope-of-work with regard to the extent of repair, replacement (partial or total), widening or other improvements. The BCR allows the Bureau of Bridges & Structures to determine the most cost-effective method for addressing reported structural, geometric or hydraulic deficiencies with options to restore structural adequacy and functionally serviceability. The BCR contains a comprehensive recommendation for the proposed scope-of-work, along with supporting information which is submitted by the District to the Bureau of Bridges & Structures for review and concurrence. After concurrence is obtained, the approval memorandum issued by the BBS is submitted to the District and is incorporated into the District Project Report.

The complete guide to prepare a BCR can be found in the IDOT document "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning. Additional BCR requirements are found in Chapter 39 of the Bureau of Design & Environment Manual available at the primary Highways Page, and similar requirements for BCRs for Local Agency structures follow the requirements of Sections 10-2.03(b) and 22-2.06(a) of the Bureau of Local Roads & Streets Manual available at the primary LPA Resources Page.

2.2.2 BCR Types

There are several possible formats for a BCR. Each require varying degrees of detail for how information is reported as well as how much. See the flowcharts in Section 1.5 "Bridge Condition Report Procedures and Practices." The possible formats or types of BCR's are briefly described below.

2.2.2.1 Bridge Condition Report

The format of the Bridge Condition Report is an extensive and detailed description of a structure. It allows the Bureau of Bridges & Structures to make correct structural, economic and policy decisions for the cost-effective expenditure of bridge rehabilitation funds. Therefore, the BCR shall provide:

1. A description of the physical conditions and deficiencies that mandate repair or replacement.
2. A verification of the apparent soundness of any substructure elements recommended for reuse along with the economic advantage gained by their reuse.
3. A statement of any geometric or hydraulic improvement required.
4. A recommendation for the proposed scope-of-work.

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5. A statement regarding the maintenance of traffic during the rehabilitation.
6. A Proposed Structure Sketch. If the recommended scope-of-work is total replacement, it should address the approximate dimensions of a replacement structure, but not so precisely that configuration refinements resulting from subsequent hydraulic, soils, structural, or economic studies are restricted.

The Bureau of Design & Environment Manual and the Bureau of Local Roads & Streets Manual refer to this BCR type as the "in-depth" BCR to discern it from other shortened formats.

2.2.2.2 Abbreviated Bridge Condition Report

The Abbreviated Bridge Condition Report (ABCR) is a short format standard form and is applicable to structures where the proposed scope of work is apparent without an in-depth study. When the anticipated scope of work for a structure is easily determined due to horizontal or vertical realignment deficiencies, severe physical deterioration, structural insufficiency, or hydraulic inadequacy, the detailed collection and analysis of information required for a BCR is not warranted. An ABCR is also applicable when documentation is necessary to indicate the anticipated work is minor in nature and the structure is in good condition.

Under certain circumstances (for complete replacement projects with specific funding and sufficiency rating), a one-page-format BCR with supplemental attachments may suffice for a Local Agency structure only if previously discussed with the District and Local Bridge Unit. The Bureau of Design & Environment Manual & the Bureau of Local Roads & Streets Manual refer to this BCR type as the "abbreviated" (with lower-case "a") BCR to discern it from the "in-depth" BCR.

2.2.2.3 Bridge Gapping

For structures to be gapped (allowed to remain in place with no or minor work) within a 3R type highway project, the District shall submit a memorandum outlining the project intent along with attaching the Master Structure Report (S-107) and the most recent NBIS inspection report which will suffice as documentation.

2.2.2.4 Bridge Condition Report Not Required

SMART and 3P projects do not require submittal of a BCR. However, if a structure lies within the limits of these type projects, coordination shall be initiated with the Bureau of Bridges & Structures before determining resurfacing options across the bridge. Bridge Repair Projects financed by maintenance

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funds are not evaluated by the Bureau of Bridges & Structures unless specifically requested by the District.

2.2.3 BCR Content

2.2.3.1 Bridge Inspection and Documentation

Since the BCR selects the scope-of-work to be performed, it is imperative that the information presented be as thorough and detailed as possible. This allows for an accurate and in-depth evaluation of the scope-of-work recommendations. Of particular concern is the physical condition of all elements to be retained for reuse in a rehabilitation project. All potential problems such as scour, shifted or frozen bearings, out-of-plumb elements, substructure movements, deterioration, anticipated vertical or horizontal alignment changes, and structurally significant cracks should be reported and accompanied by explanatory sketches and photographs to aid the evaluation of the recommended scope-of-work. Photographs and properly scaled drawings are valuable tools which provide a permanent record of the conditions existing at the time of inspection and are of great use in evaluating the suitability of reusing specific structural elements. The photographs and sketches should be of sufficient number to cover all appropriate areas of the structure.

2.2.3.2 Delamination Surveys

Delamination surveys for bridge decks are usually conducted when it is unclear if the level of deterioration dictates deck repair or replacement. Decks which have a small area, are beyond repair by visual inspection, are functionally obsolete, or exhibit little or no deterioration generally do not warrant a delamination survey. Since some delamination surveys may interpret the debonding of wearing surfaces as delaminations, the surveys should be closely coordinated with both the top and bottom of deck inspections to aid in estimating areas of deck repair. There are several test methods and procedures to choose from when conducting a delamination survey.

2.2.3.3 Bridge Condition and Geometric Analysis

There are various geometries related to a bridge or structure which should be analyzed, evaluated and documented along with the structure itself. The roadway geometrics, for which a bridge is a small part, as well as the roadways passing under the structure should be evaluated for conformance with Departmental policies. Vertical and horizontal clearances underneath the superstructure, and clear width of the deck itself should also be reviewed for conformance with Departmental policies.

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Structural adequacy and condition of the deck, superstructure and substructure elements should also be analyzed, evaluated and documented. Reuse of bridge elements such as primary beams generally depends upon their rated load capacity. Generally, an inventory rating factor of 1.0 or greater and a structural condition evaluation of "6" or greater are required to "do nothing" or reuse. Reuse of other bridge elements such as bearings and joints typically depend upon their condition. For reuse of substructure elements, see Section 3.5.7 of the "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning.

The reuse of bridge components for which the original plans are not available is not recommended. Proposals of this nature will be considered only when the Bridge Survey provides complete information on the component's soundness, make-up and dimension, and the proposed loading conditions will remain essentially unchanged.

Note that economics plays a pivotal role in all recommendations made by the engineer during the BCR process.

Detailed guidelines and requirements, as noted previously, can be found in the "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning. The Bridge Planning Section is also available, upon request, to assist with the evaluation of problem structures or site locations as well as to clarify current Departmental policies.

2.2.3.4 BCR Final Recommendation

2.2.3.4.1 Scope-of-Work

Proposing or recommending a scope-of-work for a project, with a synthesis of critical information about the structure, is required. These critical pieces of information and factors include the structure's condition and load capacity, geometric and hydraulic acceptability, economic evaluation as well as what is termed "exterior constraints."

"Exterior constraints" is a term used to describe issues which bear directly on the feasibility of a project. These may include adverse effects on traffic control, unacceptable user delay, emergency need of repair, and availability of funding.

When exterior constraints influence the scope-of-work decision, they should be thoroughly analyzed and documented.

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2.2.3.4.2 Bridge Width

The proposed bridge width should be included in the proposed scope-of-work and detailed in the structure sketch. The required bridge width is a function of traffic, design speed, existing roadway features and the proposed roadway improvement. Urban bridge widths for rehabilitation/reconstruction projects generally match the roadway template. Detailed guidelines on required bridge widths can be found in Chapters 39, and 44 through 50 of the Bureau of Design & Environment Manual available at the primary Highways Page. Any exceptions to the bridge width policies require the District to submit proper justification and documentation for consideration by the Bureau of Bridges & Structures and the Bureau of Design & Environment.

As structures are an extension of the adjacent roadway, structures should, whenever possible, duplicate the accommodations made for on the roadway for bicyclist or other multi-modal accommodations. These projects should be coordinated with the District and the BDE Bicycle and Pedestrian Coordinator. Policies and procedures are given in Chapter 17 of the BDE Manual available at the primary Highways Page.

2.2.3.4.3 Maintenance of Traffic

Construction in stages, or detour of traffic during construction should be identified in the BCR. When traffic is recommended to be maintained for a project, i.e., construction is staged, it becomes an important aspect of the proposed scope-of-work. An evaluation of lane widths, existing superstructure condition, structural adequacy, soil retention, etc. during each stage of construction are required. In particular, ensure that the lane used for Stage I traffic is correct and its condition will last the duration of Stage I construction. If this is not feasible, posting the structure, providing a beam replacement contract prior to Stage I traffic, or redirecting the traffic via lane-closure/detour should be determined and discussed with the District. It will not be necessary to show a detailed staging sequence, however the feasibility of stage construction shall be properly documented in the BCR. For existing concrete slab bridges and concrete box culverts, with a condition rating of 4 of less, obtain concrete cores to determine stage construction feasibility. See Section 3.6.3 of the "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning.

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2.2.3.4.4 Proposed Structure Sketch

A “Proposed Structure Sketch” shall be included with the Bridge Condition Report. Details such as railing, superstructure and substructure types need not be shown. However, the approximate structure length, pier locations (when environmental or hydraulic concerns mandate a specific location or omission), the general structure configuration, i.e., open abutment, closed abutment or culvert, and recommended structure width should be indicated. All other details, unless required to secure approval, should generally be omitted to allow the designer the necessary freedom to select the most appropriate structure design. Figure 2.2.3.4.4-1 presents an example of a Proposed Structure Sketch to be included within the BCR.

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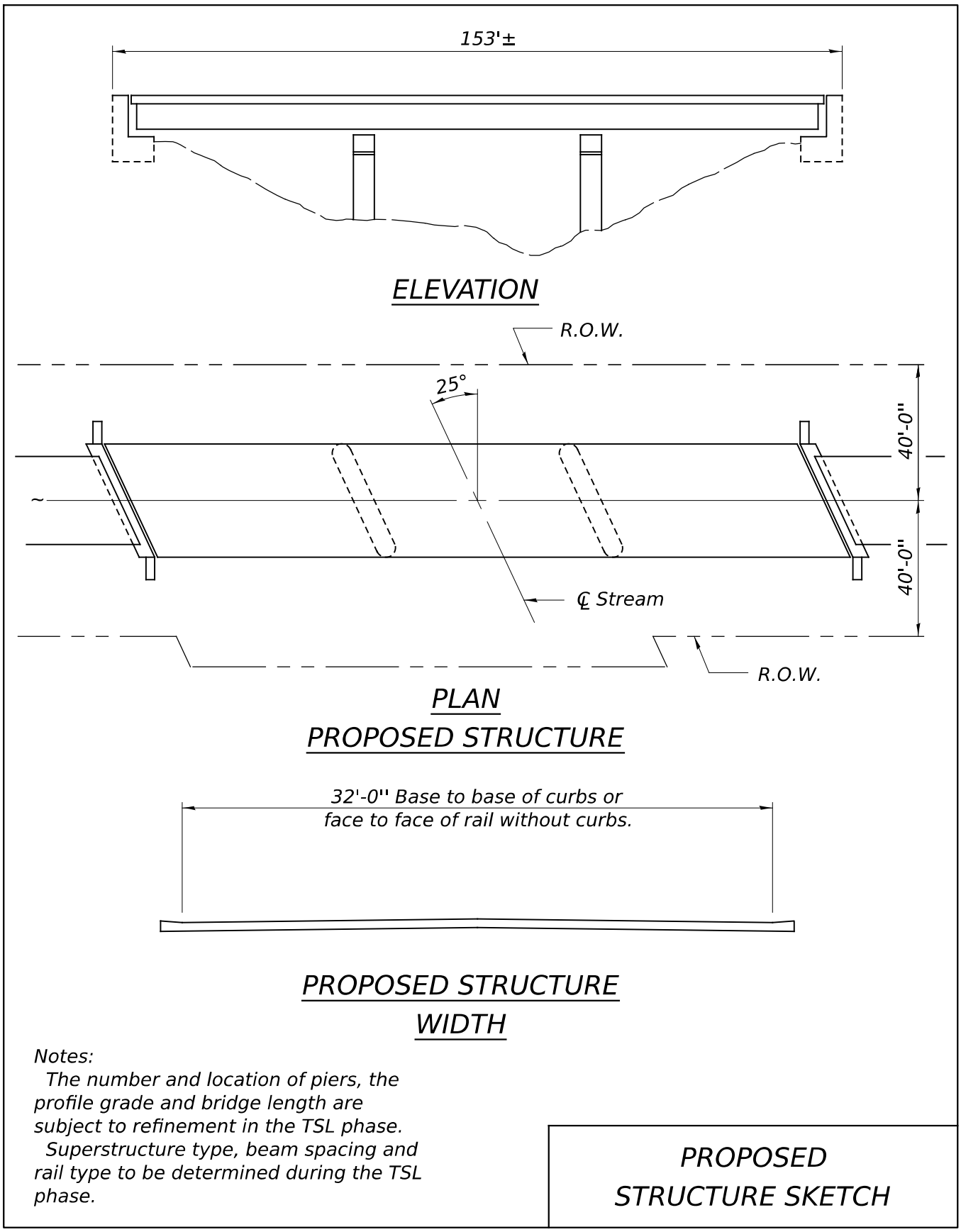


Figure 2.2.3.4.4-1

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2.2.4 Hydraulic Analysis Summary

For bridges or other structures at stream crossings, the hydraulic capacity should be reviewed when appropriate. A review of any existing hydraulic capacity analysis results and records of flooding should be made, if available. Changes since initial construction in the channel location or hydraulic opening through the structure should be noted.

Where the existing vertical alignment is to be maintained and there is no history of serious hydraulic deficiencies at the location, the existing bridge waterway opening may usually be retained.

For the following cases, development of a formal Hydraulic Report is required:

1. Bridge Replacement
2. Superstructure Replacement
3. Bridge Widening Requiring Additional Substructure
4. Reductions to the Hydraulic Opening Through the Structure

Detailed guidance on writing a Hydraulic Report is available in the "IDOT Drainage Manual" at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics.

2.2.5 BCR Submittals and Timelines

The BCR for a typical bridge or structure should be submitted 30 months before a project's date of letting. For complex bridges, the submittal time is increased to about 40 months. The engineer responsible for completing the BCR should take this into account when compiling a BCR. A copy of the BCR should be submitted to the BBS at DOT.BBS.Planning@illinois.gov for review.

2.2.6 Example Bridge Condition Report Format

Example formats for completing a BCR can be found in "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning.

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2.3 Type Size and Location Plans

2.3.1 Introduction

Since a TSL plan is generally utilized as the General Plan sheet for the final design plans, care should be exercised in its layout and presentation. Unnecessary details, out-of-proportion drawings, and non-standard lettering should be avoided. The plan and elevation views should be presented on the same sheet to provide a clear picture of the complete structure. In addition, a section through the superstructure, a section through a pier with an expansion joint, a pier sketch and the sequence of staging should be provided, as applicable. A TSL plan can require multiple sheets because of overall length and/or complexity of the structure. Detailed dimensioning outside of that necessary to establish geometric and structural controls is not desirable; however, the engineer should make the necessary calculations and scaled sketches to assure a well-proportioned and aesthetic structure.

See Section 2.1 for information on the purpose and need for a TSL plan.

2.3.2 Submittal Requirements

Generally, a TSL is required to be submitted for a project when at least some portion of the scope-of-work is structural. The cases are essentially the same as when a BCR is required. These are:

1. Bridge Replacement
2. Bridge Reconstruction
3. Bridge Rehabilitation with at Least Some Major Work
    a. Deck Replacement
    b. Superstructure/Substructure Widening
4. Permit Projects (See Section 2.6)
5. Retaining Walls with an Exposed Height of 7 ft. or Greater
6. Multiple Barrel Culverts (Cast-In-Place)
7. Multiple Barrel Culverts (Precast) on an Interstate System
8. Three Sided Structures
9. Pedestrian Bridges
10. Pedestrian Tunnels

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Cases where a TSL is typically not required to be submitted include when the scope-of-work is only:

1. Bridge Deck Repair, and deck overlays
2. Minor Bridge Repair, beam repair, and substructure repair
3. Bridge to Remain in Place
4. Retaining Walls with Less Than 7 ft. of Exposed Height
5. Multiple cell precast culverts not on the interstate system.
6. Joint Removal with Link Slab Repair detail.
7. Single Box Culverts (which are covered in the Culvert Manual)

Projects requiring TSL submittal generally require final structure plan submittal with final plan and PDO review. Projects not requiring a TSL generally do not require final structure plan submittal, as further described in Section 3.1.15.

2.3.3 Submittals and Timelines for TSL Plans

2.3.3.1 In-House TSL Plans

The TSL plan is initiated by the District with the submittal of the Structure Report (BBS Form 153) and associated attachments to BBS at DOT.BBS.Planning@illinois.gov. For typical structures, the Structure Report is submitted to the BBS 24 months prior to the project's date of letting and is increased to 27 months for complex structures.

2.3.3.2 Consultant TSL Plans

A copy of the TSL plan (in an approved digital format) along with the Structure Report (BBS Form 153), Scour Critical Evaluation Coding Report (with final TSL, if the structure crosses a waterway), Structure Geotechnical Report, detailed Plan and Profile sheet, and associated attachments are submitted by the District or by the consultant (with permission from the District) to the BBS at DOT.BBS.Planning@illinois.gov. Consultant TSLs shall be submitted to the BBS 15 months prior to the project's date of letting and is increased to 18 months for complex structures.

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2.3.4 Preliminary Guidelines, Investigations and Reports

2.3.4.1 Structure Report

2.3.4.1.1 General

A Structure Report (BBS Form 153, available at the primary Forms Page) shall accompany all TSL plan submittals for review and approval or in-house TSL development requests by the District. The Structure Report can be prepared by Department personnel, or by a consultant (when requested by the District). The Structure Report is the means of providing the Bridge Planning Unit of the BBS the necessary information and documentation to properly accommodate these requests. The District shall review and check Structure Reports developed by consultants for accuracy prior to the submittal of a TSL plan for review and approval.

2.3.4.1.2 Content

The Structure Report provides for the comprehensive reporting of data pertinent to a proposed bridge/culvert/retaining wall project. These data include:

1. Project identification, location, and programming
2. Highway, railroad and/or stream/river data
3. Structure Geotechnical Report and soil boring responsibility
4. Detailed Plan and Profile sheets.
5. Special requirements/recommendations for the composition of the structure
6. Utility accommodation, traffic handling, lighting needs and permit requirements
7. Attachments

Plans, drawings or photographs necessary to define special conditions or information, such as highway and railroad templates, plan and profile sheet(s), existing survey data, underpassing roadway and railroad profile grades and cross sections, and deck drainage calculations should be provided, if applicable, as attachments to the Structure Report. In addition, unless previously submitted, the Bridge Condition Report and Approved Waterway Information Table shall accompany the Structure Report, if applicable.

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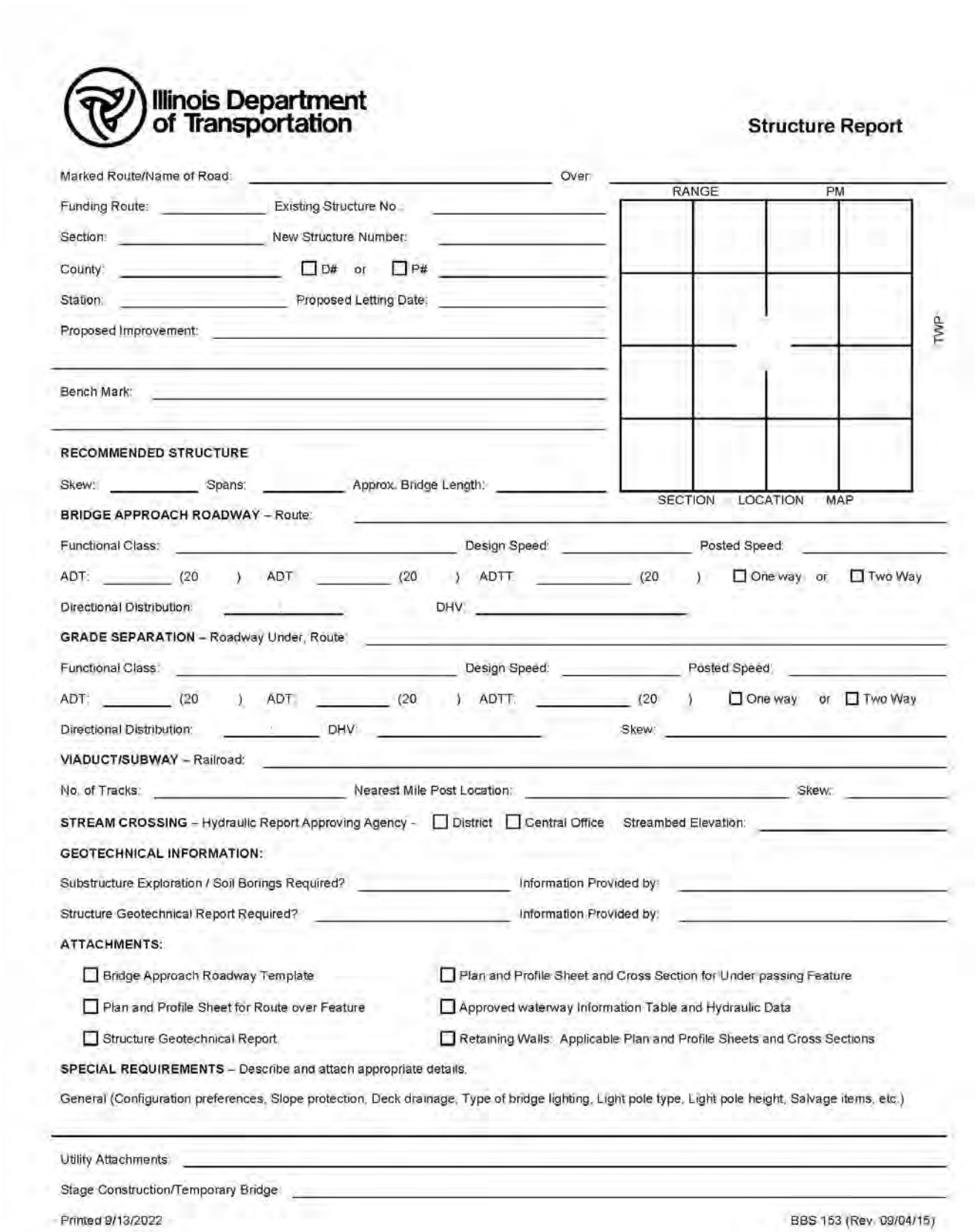
2.3.4.1.3 Preparation and Submittal

The efficiency and timeliness of the preparation and review of a TSL plan are highly contingent upon the completeness of information in the Structure Report. Therefore, all items in the report shall be appropriately addressed with non-applicable items so designated. All required attachments to the report shall be provided. When filling out the Structure Report, the consultant should contact the District regarding any questions they may have.

Upon receipt by the Bridge Planning Section, the Structure Report will be reviewed for completeness of information and, if found acceptable, used as the basis for the review of a consultant TSL plan, or the initiation and subsequent development of a TSL plan and Structure Geotechnical Report by the BBS.

A sample Structure Report form is shown in Figure 2.3.4.1.3-1 and can be found as form BBS 153 in the "Bridges and Structures" Section at the primary IDOT Forms page.

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Illinois Department of Transportation

Structure Report

Marked Route/Name of Road: _____ Over: _____

Funding Route: _____ Existing Structure No.: _____

Section: _____ New Structure Number: _____

County: _____ ☐ D# or ☐ P# _____

Station: _____ Proposed Letting Date: _____

Proposed Improvement: _____

Bench Mark: _____

RANGE PM TWP

SECTION LOCATION MAP

RECOMMENDED STRUCTURE

Skew: _____ Spans: _____ Approx. Bridge Length: _____

BRIDGE APPROACH ROADWAY – Route: _____

Functional Class: _____ Design Speed: _____ Posted Speed: _____

ADT: _____ (20) ADT: _____ (20) ADTT: _____ (20) ☐ One way or ☐ Two Way

Directional Distribution: _____ DHV: _____

GRADE SEPARATION – Roadway Under, Route: _____

Functional Class: _____ Design Speed: _____ Posted Speed: _____

ADT: _____ (20) ADT: _____ (20) ADTT: _____ (20) ☐ One way or ☐ Two Way

Directional Distribution: _____ DHV: _____ Skew: _____

VIADUCT/SUBWAY – Railroad: _____

No. of Tracks: _____ Nearest Mile Post Location: _____ Skew: _____

STREAM CROSSING – Hydraulic Report Approving Agency - ☐ District ☐ Central Office Streambed Elevation: _____

GEOTECHNICAL INFORMATION:

Substructure Exploration / Soil Borings Required? _____ Information Provided by: _____

Structure Geotechnical Report Required? _____ Information Provided by: _____

ATTACHMENTS:

☐ Bridge Approach Roadway Template

☐ Plan and Profile Sheet and Cross Section for Under passing Feature

☐ Plan and Profile Sheet for Route over Feature

☐ Approved waterway Information Table and Hydraulic Data

☐ Structure Geotechnical Report

☐ Retaining Walls: Applicable Plan and Profile Sheets and Cross Sections

SPECIAL REQUIREMENTS – Describe and attach appropriate details.

General (Configuration preferences, Slope protection, Deck drainage, Type of bridge lighting, Light pole type, Light pole height, Salvage items, etc.)

Utility Attachments: _____

Stage Construction/Temporary Bridge: _____

Printed 9/13/2022 BBS 153 (Rev. 09/04/15)

Figure 2.3.4.1.3-1

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2.3.4.2 Hydraulic Report Coordination

The engineer responsible for development of the TSL should obtain or check the status of the Hydraulic Report at the time when the TSL is in its initial stages of formulation if the structure involves a waterway crossing. Since the Hydraulic Report is often initiated in the BCR phase, it is important that as the TSL development progresses, any structure type or length changes that would affect the Waterway Information Table, scour calculations, or other recommendations be relayed to the Hydraulics Engineer for reevaluation and possible revision. There can be special concerns regarding scour calculations that were not foreseen during the BCR phase. The final number of piers, their location, skew, stem widths, footing widths and footing elevations will affect this calculation, and thus communication between the hydraulics engineer and the engineer responsible for TSL development plays a key role. In addition to the Waterway Information Table and scour calculations, the Hydraulic Report provides the information needed to calculate estimated water surface elevations (EWSE), which are used to evaluate the need for and required height of cofferdams, and/or the need for permanent casing of drilled shafts.

2.3.4.3 Structure Geotechnical Report and Subsurface Investigation

A significant amount of geotechnical information and recommendations are required to select the most appropriate structure type, size and location for a project. This information is provided via the Structure Geotechnical Report (SGR). The SGR ensures geotechnical responsibilities are properly assigned, documented, and approved in a consistent manner statewide. The SGR serves to verify all geotechnical issues affecting a structure have been identified and taken into account by the engineer responsible for TSL development. In some cases, a design phase geotechnical memo will be required to provide the design parameters and foundation treatments to be used by the structural design engineer during Final Contract plan development. Guidance and policies for preparation of SGR's are given in the Geotechnical Memorandums and Provisions (i.e. AGMUs at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) and in the "Foundation Component Selection" Section 2.3 herein.

For consultant projects, as soon as a project's BCR is approved, the District should determine which individuals or parties will be responsible for the subsurface investigation and the SGR. Both decisions impact the Professional Transportation Bulletin (PTB) scope, consultant selection, man hour negotiations as well as other issues, and thus should be completed at the earliest possible time. In general, the geotechnical responsibilities related to structures involve subsurface exploration or investigation, geotechnical analyses, and foundation design recommendations, all of which shall be contained in the final SGR or design phase geotechnical memorandum. Subsurface exploration shall be conducted by either the District Geotechnical Engineer or a geotechnical consultant. For consultant prepared TSL

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plans, the SGR shall be completed by the District Geotechnical Engineer, a geotechnical consultant, or the structural consultant. The BBS Foundations and Geotechnical Unit will complete the SGR for Structure plans prepared by the BBS. In all cases, the Foundations and Geotechnical Unit reviews and approves all SGR's during the BBS's process of review and approval of TSL plans.

The engineer responsible for TSL development should establish contact with the geotechnical engineer responsible for performing the subsurface investigation immediately after the District requests initiation of TSL plan preparation. Together, they should formulate an exploration and testing plan tailored to the needs of the anticipated geotechnical analyses and foundation design recommendations. A timeline which meets the expected SGR and TSL completion date should also be established. The engineer responsible for TSL development should conduct preliminary analyses to determine possible structure type(s). These analyses should consider existing foundations and conditions as well as anticipated foundation locations, elevations and loadings, the potential need for any new fills or cuts, and any other pertinent information. The engineer responsible for TSL development should provide all pertinent information to the geotechnical engineer to help ensure a proper and complete subsurface investigation. This information should take into account existing foundation elevations, loads and new fills/cuts. Continued coordination between the geotechnical engineer and the engineer responsible for TSL development is recommended up until actual mobilization of drilling operations to ensure the subsurface investigation is relevant and completed to meet the TSL development schedule.

2.3.4.4 TSL Guidelines

The following general guidelines for various aspects of the TSL plan can be used to establish the most cost-effective bridge type and size and to locate the substructure components appropriately.

2.3.4.4.1 Bridge Length

Bridge Length is determined by the location of the abutments. The location of the abutments is dependent on bridge opening requirements, and the method used to terminate the approach embankment and transition to the structure. Where the embankment is to be terminated by means of a stable end slope, an "open" abutment is located at or near the top of the end slope. End slopes shall be 2:1 or as otherwise established by the Structure Geotechnical Report stability analysis. Where the embankment is to be terminated at a vertical plane, a "closed" or earth retaining abutment is located at that plane. The use of an end slope to terminate the embankment results in a longer bridge than one using a closed abutment; however, overall bridge costs are generally lower with the open abutment design because of the high cost of closed abutments.

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Closed abutments are generally designed as a reinforced concrete retaining wall supported on a large spread footing, drilled shafts or a pile supported footing. Closed abutments are seldom economical and should not be used without a detailed cost investigation unless site conditions dictate its use.

Open abutments generally consist of a single or staggered rows of piles, drilled shafts, or a spread footing supporting a concrete cap block. Vaulted abutments are a combination of closed and open abutments used at grade separation or interchange locations.
For open stub abutments, two primary dimensions have been established to assist with bridge layout:

A_{bw} = Front face of backwall to CL Brg. (in.)
A_{ba} = Back of abutment to CL Brg. (in.)

These dimensions provide a consistent starting point during the TSL (Type, Size and Location) development stage and assist in establishing the bridge stationing. There are numerous factors that can affect the location of the centerline of bearing such as skew, bearing size, beam size, expansion joint size and type, clearances, profile grade, end diaphragms, and seismic seat lengths. Figure 2.3.4.4.1-1 through Figure 2.3.4.4.1-2 presents a summary of policy references for these many variables. High skew and high expansion scenarios may require further evaluation beyond these equations.

See the following definition sheets & Table 2.3.4.4.1-1 for A_{bw} Constraints & reference temperatures for expansion opening scenarios.

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Variable definitions for Figure 2.3.4.4.1-1 through Figure 2.3.4.4.1-2 and Table 2.3.4.4.1-1 are as follows:

Variables at Right Angles (inches):

R_1 = Minimum clear from front face of backwall to end of beam *,**
R_2 = Minimum clear from end of deck to end of beam
R_3 = Minimum clear from front face of backwall to top plate *
R_4 = Minimum clear from front face of backwall to bottom plate *
R_5 = CL bearing to front face of abutment
R_6 = Minimum clear from bottom plate to front face of abutment
R_7 = Minimum clear from end of beam to front face of abutment *,**
A_{bw} = Front face of backwall to CL bearing
A_{ba} = Back of abutment to CL bearing
d, b_{tf}, b_{bf} = Beam depth, top flange width, and bottom flange width (respectively)
L_b = Bottom bearing plate dimension perpendicular to beam web
W_b = Bottom bearing plate dimension parallel to beam web

** For Initial TSL Planning stage, calculated at 50°F*

*** For Final Design stage, refine at min/max temperature for controlling limit states*

Variables Along CL Beam:

E_1 = Distance from beam end to CL bearing (inches)
E_2 = Sloped concrete beam encroachment to backwall (inches) = $d \sin\left[\tan^{-1} G\right]$, where

G = Beam slope at CL bearing toward backwall
(Decimal, foot/foot; positive for sloping toward abutment backwall)

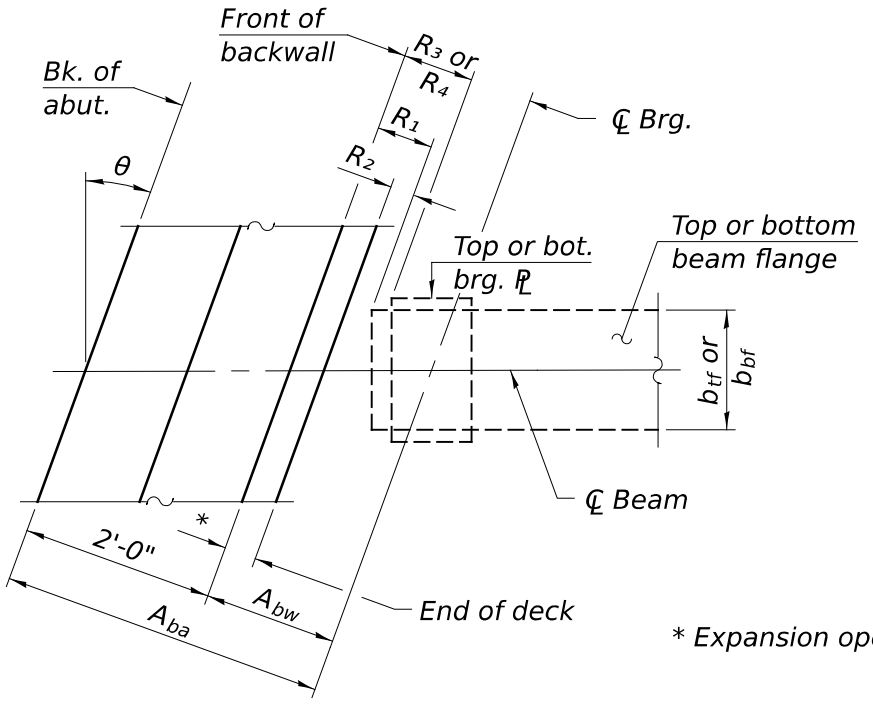
Stationing (along CL beam, no horizontal curves):

$STA_{℄\ Brg.} = STA_{Bk.\ Abut.} \pm L_{ba} = STA_{Bk.\ Abut.} \pm A_{ba} / \cos\theta$, where

$STA_{℄\ Brg.}$, $STA_{Bk.\ Abut.}$ = Typical Roadway Stationing (###+##.##, feet)
L_{ba}, $STA_{℄\ Brg.}$, $STA_{Bk.\ Abut.}$ all measured along CL beam
θ = bridge skew (degrees), and L_{ba} & A_{ba} converted to feet

See the “Substructure Component Selection” Section 2.3 herein for more complete guidance on substructure selection.

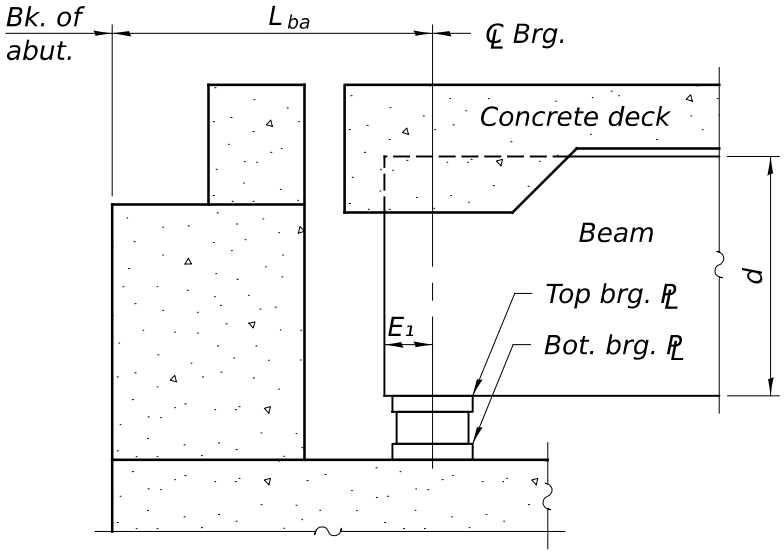
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PARTIAL PLAN

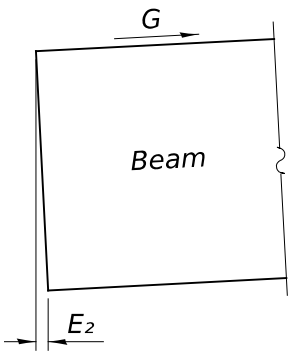
(Clearances behind bearing)

$$A_{bw}\ (in) = R_1 + \cos\left[\text{abs}\left\{\theta - \tan^{-1}\left(\frac{b_{tf}/2}{E_1 + E_2}\right)\right\}\right]\sqrt{(E_1 + E_2)^2 + (b_{tf}/2)^2}$$



SECTION THRU ABUT.

(Dimensions along ℄ beam. Standard version shown, corbel version similar.)



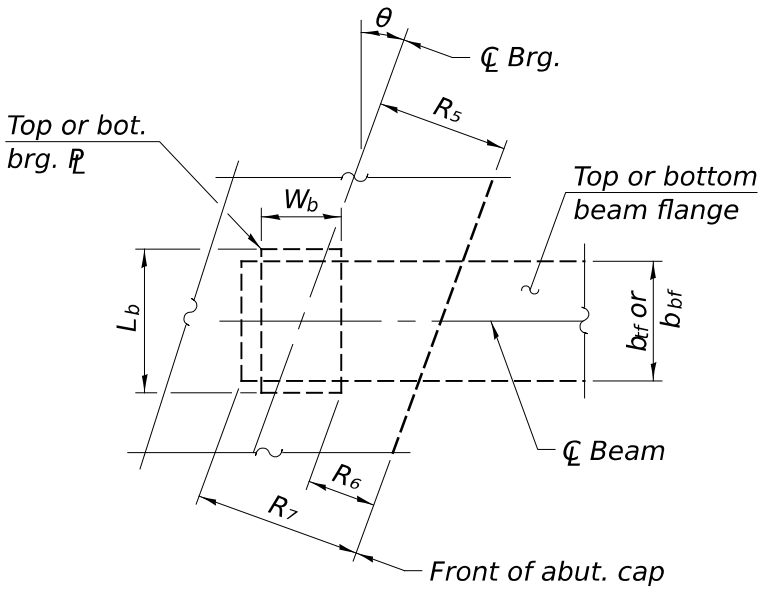
SLOPED BEAM

(Concrete beams shown, steel beam ends may be cut vertical.)

STUB ABUTMENT ℄ BEARING LAYOUT

Figure 2.3.4.4.1-1

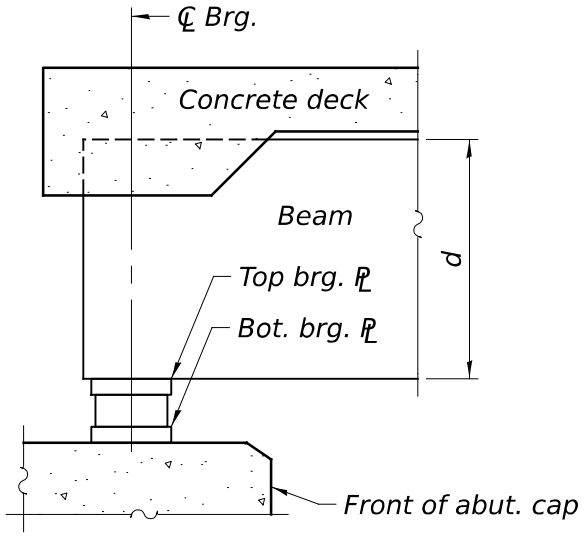
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PARTIAL PLAN

(Clearances in front of bearing)

$$R_5\,(in) = R_6 + \cos\left[\text{abs}\left\{\theta - \tan^{-1}\left(\frac{L_b\,/\,2}{W_b\,/\,2}\right)\right\}\right]\sqrt{(L_b\,/\,2)^2 + (W_b\,/\,2)^2}$$



SECTION THRU ABUT.

(Dimensions along ℄ beam)

STUB ABUTMENT
SEAT WIDTH LAYOUT

Figure 2.3.4.4.1-2

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Element	Variable or Constraint	Policy Source
Substructure	General/Initial Planning Dimensions	
	$2'\text{-}0''$ typ + A_{bw} + $R_5 \geq 4'\text{-}9''$	Figure 3.8.5-1 & Figure 3.8.5-2
	Seismic Seat Requirements	
	$R_7 / \cos\theta \geq N_{IDOT}$ $A_{ba} + R_5 - (2'\text{-}0''$ typ + $R_1) \geq N_{AASHTO}$	N_{IDOT}: Section 3.15.4.2 or Seismic Manual N_{AASHTO}: AASHTO LRFD 4.7.4.4
Superstructure	PPC-I, PPC-IL, Bulb T Beams	
	d, b_{tf}, b_{bf}	Table 3.4.3.2-7
	IL-27 Beam Shapes: E_1	Figure 3.4.7-1 & Figure 3.4.7-2
	IL-36 thru IL-90 Beam Shapes: E_1	Figure 3.4.7-1 & Figure 3.4.7-3
	Steel Beams	
	W_B, and L_B	Figure 3.7.1.2-4
	E_1, ($\theta \geq 20°$ with Pedestals for End Cross-Frames)	Section 3.3.23
Bearing	Expansion Bearings	
	$R_1 \geq 5''$	Figure 3.8.5-9
	$E_1 \geq 8''$	
	Elastomeric Bearings	
	Minimum W_B and L_B	Section 3.7.4: Figures 3.7.4-2 and beyond
	Fixed Bearings, PPC-I, PPC-IL, and Bulb T Beams	
	$E_1 = 8''$	Figure 3.7.1.2-3
	Minimum W_B and L_B	
	Fixed Bearings with Sealed Fixed Joints & $\theta \leq 20°$	
	$R_2 \geq 3''$	For $\theta > 20°$, clip Bulb-T top flanges similar to Figure 3.8.3-14 to meet R_2 constraint
	$R_7 \geq 2''$	
	Fixed Bearings with Anchor Bolts	
	$R_1 \geq 4''$	Clip bott. PL up to 3"x3" at R_6 || to θ to meet anchor bolt, constructability, and connection clearances
	$E_1 \geq 5½''$	
	$R_6 \geq 2''$	

Table 2.3.4.4.1-1

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2.3.4.4.2 Pier Location and Type

The number, type and location of the piers are determined in such a manner as to produce optimum bridge economy within the constraints of horizontal and vertical clearance requirements, stream flow requirements and aesthetics. Bridge piers are generally of two basic types; pile or drilled shaft bents, and piers with footings. Bent piers consist of a single row of piles or drilled shafts supporting a bearing cap. Where required for aesthetic or hydraulic purposes, the extension of the piles above the ground may be encased to produce a solid wall.

Footing supported piers are of many types. Footing may be “spread” (soil or rock supported) or may be supported by drilled shafts or piling. Pier shafts may be solid walls (or web walls), walls with cantilever extensions (hammerheads) or may consist of a multi-column frame mounted on a plinth or crash wall.

See the “Pier Type” Section 2.3 herein for more complete guidance on pier type selection.

2.3.4.4.3 Superstructure Types

Figure 39-3B in Chapter 39 of the Bureau of Design & Environment Manual (available at the primary Highways Page) provides a list of commonly employed superstructure types, the span lengths for which they are applicable, and the approximate construction depth (profile grade to low beam) required for their use. Superstructures may be of any of the types listed for the span length ranges indicated. The figure is a good source for a rough estimate prior to the initiation of the TSL plan development process and can be a helpful tool for the District when evaluating profile grades.

See the “Superstructure Component Selection” Section 2.3 herein for more complete guidance on this and other aspects of superstructure planning.

2.3.5 TSL Plan Types

This section presents a brief overview of specific types of information that shall be presented on a TSL plan for the following structure types: Highway Bridges, Railroad Bridges; Culverts and Three-Sided Structures; Pedestrian Bridges; and Retaining Walls. The TSL plan shall be an 11 in. by 17 in. drawing (or drawings) with the standard of preparation being the same as that required for Final Contract plans. The plan sheet shall be presented in a form that will allow for its eventual refinement as the "General Plan and Elevation" sheet or the cover sheet for Contract Bridge plans.

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See comprehensive "TSL checklist" in Section 2.3.13 for the development of TSL plans for bridges, culverts, and three sided structures, and Appendix A1.3 provides online links to Approved TSL plans for reference.

2.3.5.1 Highway Bridges

The following is a partial list of items which shall be shown on a TSL plan (as applicable): elevation and plan view, cross section through superstructure, outlines of existing structure, location sketch, waterway information, profile grade data, design specifications, roadway classification data, sketch of typical pier in elevation, stage construction order and limits, foundation type at each substructure, etc.

All aesthetic details for a structure shall be finalized during the TSL phase. Special aesthetic treatments and special bridge features should also be illustrated; however, data and dimensions subject to refinement in the detailed structural analysis should be omitted.

2.3.5.2 Railroad Bridges

The primary differences between highway and railway bridge TSLs are: the AREMA Railway Bridge Design Specifications govern the design, railroad approval of the TSL is required, and stage construction and traffic control differ.

2.3.5.3 Culverts and Three-Sided Structures

For culverts, a cross section through the barrel should be shown on the TSL. A longitudinal section which includes lane, shoulder, and median widths as well as roadway cross slopes shall also be given. Special considerations for a culvert TSL include indicating the type of wingwalls proposed. See the "Culvert and Three-Sided Precast Concrete Structure Selection Process" and "Type Size and Location Presentation" Section 2.3 herein.

Three sided structures are typically shown as culverts on a TSL plan.

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2.3.5.4 Pedestrian Bridges

Pedestrian bridges have a minimum vertical clearance which is greater than that required for highway bridges. See Chapter 39 of the Bureau of Design & Environment Manual available at the primary Highways Page. They shall also meet the Americans with Disabilities Act (ADA) requirements and the AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges. See GBSP 33 for other requirements.

2.3.5.5 Retaining Walls

Walls with an exposed height (defined as the difference in elevation between the finished grade behind the wall and the finished grade in front of the wall) of 7 ft. or greater require a TSL plan. In addition, walls below this height with unique retention conditions such as tiered walls, walls with large/steep back slopes, walls designed to retain slope failures, and walls retaining railroads or disturbance sensitive property may also require a TSL plan. The following is a partial list of items which shall be shown on a TSL plan (as applicable): elevation and plan view, typical wall section, location sketch, roadway profile grade data, design specifications, etc. Contact the BBS if the need for a TSL plan remains uncertain.

All aesthetic details for a retaining wall shall be finalized during the TSL phase. Special aesthetic treatments and special wall features should also be illustrated; however, data and dimensions subject to refinement in the detailed structural analysis should be omitted.

See the “Type Size and Location Presentation” Section 2.3 herein, which gives a comprehensive checklist for preparation of TSL plans for retaining walls. Appendix A1.3 provides online links to Approved TSL plans for reference.

2.3.6 Bridge Type Study

A Bridge Type Study is the process by which the most appropriate structure type for a given location is determined and is a synthesis of the necessary economic, aesthetic and site evaluations which lead to that selection. A well-conceived Bridge Type Study considers the structure types feasible for the site parameters or environmental commitments, provides the reasoning for eliminating or developing particular alternates as well as compiles cost estimates for all alternates considered and finally the rationale for the selection of the structure type chosen. Essentially, a Bridge Type Study is an important phase in the TSL plan preparation process.

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In each project, the Bridge Type Study is a part of the planning computations which justify the TSL plan and as such is not submitted for review. However, for major river crossings or when requested by the Bureau of Bridges & Structures, a Bridge Type Study becomes a formal report requiring the approval of the Engineer of Bridges & Structures before preparation of the TSL plan can commence. Such a report would provide additional written treatments concerning economic evaluations for the viable alternates, span length versus pier height studies for the approaches, pier type structural and aesthetic studies, main spans and the approaches structure type aesthetic studies, and architectural presentations of the alternate systems presented in the report. Since AASHTO Specifications do not specifically address some of the long span bridge types associated with major river crossings, the report should also document unusual design procedures contemplated, deviations from or variations of AASHTO Specifications to be used, special materials or details proposed or tests anticipated.

Economic Evaluation: It is the philosophy of the Bureau of Bridges & Structures that all structures are to be planned within the constraints of site requirements and policy such that the selected bridge configuration will result in the minimum structure cost. The minimum structure cost shall be established on the basis of initial structure cost with due consideration given to replacement and maintenance costs.

Increasing minimum costs are justifiable when it will result in either the least overall highway project cost, reduced annual maintenance costs or where other intangible benefits are derived. The use of cost premiums shall be supported by proper economic documentation.

The following features are examples of cost premiums:

1. Bridge length in excess of that required by clearance or waterway opening requirements.
2. Bridge widths in excess of that required by structure width policy.
3. Bridge superstructure depth greater than the most economical.
4. Bridge length in excess of that required to avoid conflict between new and old substructure units.
5. Accelerated Bridge Construction (ABC) to minimize disruption to traffic and local businesses.
6. Alternate coating/protection systems for beams and rebar to extend structure life. See Section 2.3.6.1.1.

Aesthetics: Each structure should be evaluated for aesthetics. It is seldom practical to provide cost premium aesthetic treatments without a specific demand, but careful attention to the details of the lines and forms used will generally result in a pleasing structure appearance.

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Some basic aesthetic guidelines are:

1. Avoid mixing structural support systems, i.e., trusses and beams, or column piers with solid piers.
2. Whenever possible, use one or no more than two beam depths in a structure length. Avoid sandwiching shallow spans between two deeper spans and utilizing very slender superstructures over massive piers.
3. Abrupt changes in beam depth should be avoided when possible. Whenever sudden changes in the depth of beams in adjacent spans are required, care should be taken in the development of details at pier locations. If thoughtfully considered, treatment of these depth transition piers can create an attractive and pleasing appearance which will complement the aesthetics of the overall project.
4. The lines should be simple and without excessive curves, insets, offsets and ornamentation.
5. All structures should blend with their environment.

One of the most significant design factors contributing to the aesthetic quality of a highway might variously be termed unity, consistency, coherence or continuity—that quality which makes it appear the whole has been consciously designed to present a “highway theme”. Highways are not, from an aesthetic design point, easily divisible, particularly the modern interstate or freeway with long sight distances. Therefore, every element in the highway complex should relate directly or indirectly to the others if the desired theme is to be realized.

Because of the typically great extent of modern multi-lane freeways, it is inappropriate to follow a single theme for the full extent of a highway. Changes in the character of the terrain and in the culture of the various areas traversed will facilitate the blending or graceful transitioning from one basic set of design concepts to another.

The thematic concept for highway design can normally be accomplished within the general guides of the standards developed by the Department for both structures and roadways requiring only minimal special designs and accomplished with minor project cost increases.

It is anticipated that special situations and projects will arise where new concepts and details will require development to fulfill the aesthetic needs of a given project. In particular, Context Sensitive Solutions (CSS) will be a requirement on highway and structure projects. Details and concepts as a result of CSS should be coordinated with the appropriate District and the Bureau of Bridges & Structures. See Section 2.1 for additional information on CSS.

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2.3.6.1 Superstructure Component Selection

Figure 39-3B in Chapter 39 of the Bureau of Design & Environment Manual (available at the primary Highways Page) provides a list of commonly employed superstructure types; the span ranges for which they are applicable; and the approximate construction depth (profile grade to low beam) required for their use. Superstructures may be of any of the types listed for the span length ranges indicated. Where two or more types are applicable to the span length and depth requirements of the site, the choice shall be made on the basis of comparative cost. The values provided are general guidelines for setting profile grades, sizing waterway openings and estimating cost for a proposed structure and should not be used for detailed TSL determination.

2.3.6.1.1 Structural Steel

All wide flange beams and plate girders shall be designed for composite action in both positive and negative moment regions; see Section 3.3 for more information. Planners and designers shall perform additional analysis for steel structures as follows:

- Bridge skews less than 10 degrees; skews exceeding 60 degrees (see Analysis Procedures in Section 3.1.12.1)
- Lateral stresses for bridge skews less than 45 degrees; skews exceeding 60 degrees (see prescriptive or policy measures in Section 3.3.5)
- Interior diaphragm and cross-frame orientation for bridge skews less than 10 degrees; skews exceeding 20 degrees (see Section 3.3.22)

Long span steel plate girders may be evaluated for geometric and cost advantages of utilizing HPS 70 ksi hybrid flanges at the piers or throughout the beam length, and haunched girders for bridges with vertical clearance issues. When practical, i.e., cost, constructability, vertical clearances, etc., web depths should meet AASHTO 2.5.2.6.2 and 2.5.2.6.3 live load truck criteria. The recommended minimum wide flange section shall be W27. Due to constructability and serviceability concerns, contact the BBS before using wide flange sections shallower than W27. See the “Maintenance of Traffic” Section 2.3 herein, “stage construction” limitations Section 3.3.25, and “minimum deck overhang” limitations Section 3.2.9.

Structural steel shall utilize the materials designated in Table 6.4.1-1 of the AASHTO LRFD Bridge Design Specifications.

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Coating Strategies for New Steel Structures

Bridge coating systems are primarily used to increase the service life of structural steel. The coating may also be viewed as an aesthetic treatment. To provide a bridge coating system that is functional, durable, and aesthetically pleasing, a bridge coating strategy must be determined. IDOT currently supports four bridge coating strategies:

- Hot-Dip Galvanized Steel
- Metallized Steel
- Weathering Steel
- Painted Steel

The weathering steel strategy requires painted or metallized beam ends. The painted steel strategy may utilize metallized beam ends. Galvanized and metallized steel may utilize a paint topcoat for aesthetics.

The coating strategy is typically chosen at the structure planning phase, and is indicated on the Type, Size and Location (TSL) as follows:

- If galvanized or metallized steel is chosen, this shall be stated on the TSL general plan and elevation sheet via a general note, e.g., “All structural steel shall be galvanized” or “All structural steel shall be metallized”.
- If weathering steel is chosen, this is reflected on the TSL via the grade of steel (Grade 50W is shown).
- If painted steel is chosen, this shall be stated on the TSL general plan and elevation sheet via a general note, e.g., “All structural steel shall be painted”.

The decision of a coating strategy should be coordinated with District Studies and Plans Engineers, District Bridge Maintenance Engineers and/or District Paint Technicians, and will be reviewed by the Bureau of Bridges & Structures Bridge Planning Unit at the TSL review phase for Department-owned structures. The coating strategy for Local Public Agency (LPA) structures shall be determined by the Owner, in consultation with the design engineer. The LPA may consult with the Bureau of Bridges & Structures.

A brief explanation of each of the four coating strategies is given below, followed by a summary of their relative costs, durability, and aesthetics.

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1) Galvanized Steel (Hot-Dipped)

Galvanized steel is a coating strategy wherein the structural steel is dipped into a galvanizing kettle of zinc, which allows the zinc to form a metallurgical bond with the steel. This strategy provides the highest level of cathodic protection and is more adherent than paint or metallizing. It is the most resistant to corrosion, with a design life expected to be equal to the design life of the bridge. For this reason, it is the most desirable option.

In general, galvanizing is a viable option for structures with individual span lengths less than or equal to 120 feet and very little curvature. These structures tend to have shallow and comparatively lightweight sections that will meet the requirements in Table 2.3.6.1-1. An in-depth list of preferred and maximum dimensions and segment weights is provided in Table 2.3.6.1-1.

There are multiple galvanizers available to hot-dip galvanize beams. Each of these galvanizers have kettles that vary in size. They also have different cranes with variable weight capacities available to lift the beam segments. In order to maximize the amount of available galvanizers, beam segments shall follow the geometric requirements in Table 2.3.6.1-1. This table uses assumptions for vat width/depth, girder curvature, and progressive dipping techniques presented on the AGA website, which may vary between galvanizing plants. A "beam segment" refers to a portion of a beam between a beam end and a splice, or the portion of the beam between two splices. The "preferred" column gives values which most hot-dip galvanizers can accommodate. The "maximum" column gives a value that should not be exceeded because it would be very difficult for the majority of hot-dip galvanizers to galvanize the beams. Note that the "width" must also include any curvature on curved beams.

	Preferred	Maximum
Length (ft)	50	75*
Width (ft)	4	9
Depth (ft)	6	12
Weight (kips)	10	20

Table 2.3.6.1-1: Beam Segment Requirements for Hot-Dip Galvanizing
*(Data as of 2024. *Contact AGA for the latest data in the region)*

To meet the requirements in the table above, segments may be required to be shortened to accommodate for curvature in highly curved structures. Shorter beam segments require additional splices to the structure to accommodate the galvanizing. This may not always be a viable option depending upon the structure configuration. For example, splices are discouraged over live railroad and in high moment areas of a beam. If splices in these locations are required, they will typically be assembled on the ground,

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and multiple segments will be lifted into place simultaneously, which may complicate erection. Bridge planners should look at the geometry of the structure and required beam segments when determining if galvanized steel is an option. Excessively heavy beams, high curvature, deep girders, or longer required segments will often preclude the use of galvanized steel. Minimum roadway vertical clearance, shall include the splice bolts, if located over roadway.

Because recoating is not required, galvanized steel is a preferred option where recoating is difficult, such as in high-traffic areas or bridges over railroads.

Cost: Galvanized steel upfront costs are generally 40% higher than weathering steel, and 25% higher than painted steel. The upfront costs are generally slightly higher than metallized steel. The life-cycle costs of galvanized steel are reduced by the fact that recoating is not required.

Durability: Galvanized steel is considered to be the most durable bridge coating strategy. It has a design life that is anticipated to be equal to the life of the structure. Recoating is not required.

Aesthetics: Galvanized steel is a dull gray color that may appear to be patchy, especially in the first few years of the structure's life. It may not be considered to be aesthetically pleasing. The color also may change over time as the bridge ages. Galvanized steel fascia beams may be painted to improve aesthetics at an additional cost. Painting requirements are found in the GBSP 83 (Hot Dip Galvanizing for Structural Steel) which references Section 506 of the Standard Specifications for Road and Bridge Construction regarding paint systems. The additional up-front cost of painting fascias of galvanized steel structures may be assumed to be 5% of the total steel cost. However, planners should note that aesthetic painting will likely require repainting in the future, thereby increasing the life-cycle cost. More information on the painting of galvanized steel is given in Section 3.3.29.

2) Metallized Steel

Metallized steel is a coating strategy wherein a metallic thermal spray is shop-applied to the structural steel. This spray is considered to have a similar adherence as paint, but is more resistant to thermal cycles than paint. Therefore, it is a more resistant bridge coating strategy than painted steel and weathering steel, but less resistant than galvanized steel.

Metallized steel does not have the design and detailing considerations required of galvanized steel. There are no maximum segment lengths, weights, or curvature requirements. It is available as an option for full bridge coating if galvanized steel is preferred but cannot be utilized due to geometric or weight constraints.

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Cost: Metallized steel upfront costs are generally similar to galvanized steel for full structure coatings. For metallizing small areas near deck joints on weathering steel or painted steel projects, the overall cost of adding metallizing to these projects is small.

Durability: Metallized steel is a durable bridge coating strategy. It has a design life that is anticipated to be equal to 1.5 to 2 times that of painted steel. Metallized steel should be anticipated to be re-metallized once over the course of its service life.

Aesthetics: Metallized steel is a dull, patchy, gray color that becomes increasingly darker as the structure ages, eventually becoming a charcoal gray. The color may not be consistent in tone across the structure. It may not be considered to be aesthetically pleasing. A clear coat may be added to seal the metallizing and limit further changes in color. Metallized steel fascias may be painted to improve aesthetics. These aesthetic options are more clearly explained in GBSP 82 (Metallizing of Structural Steel). Similar to painting galvanized fascias, the additional cost of painting metallized fascias may be assumed to be 5% of the total structural steel cost. Also, similar to galvanized steel, planners should note that any aesthetic painting will likely require repainting in the future, thereby increasing the life-cycle cost.

Refer to the flowchart in Figure 2.3.6.1.1-1 for bridge planners to aid in choosing the correct coating strategy. Note that each decision in the flowchart has a number attached to it. The number corresponds to Q&A Items 1-11 immediately following the flowchart.

3) Weathering Steel

Weathering steel is a coating strategy wherein the steel relies on a dense metal oxide patina to resist corrosion. This coating system is effective when the steel can remain relatively dry and free of chlorides, i.e., salt. Locations where weathering steel cannot remain dry and free of chlorides, such as areas near deck joints, are coated with either paint or metallizing.

When conditions are appropriate (see below), unpainted AASHTO M270 Grade 50W may be used for new and reconstructed bridges with consent from the District and when structurally necessary AASHTO M270 Grade HPS 70W may be used when authorized by the Bureau of Bridges & Structures. When unpainted weathering steel is not applicable, AASHTO M270 Grade 50 should be specified for primary members. For bridge widening projects, the section properties of the existing members should be matched.

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Unpainted AASHTO M270 Grade 50W (Weathering Steel) is permitted for bridges when criteria of the Federal Highway Administration Technical Advisory (T 5140.22) "Uncoated Weathering Steel in Structures" (1989)[1] are met.

The most common use of weathering steel is in stream crossing applications, where it is not generally seen by the motoring public. However, weathering steel should not be used in situations where there will be constant humidity due to the proximity of water. For stream crossings/crossings over running water, if the distance between the typical streamflow elevation and the bottom of the beams is less than 8 feet, weathering steel shall not be used. For crossings over lakes/stagnant bodies of water, if the distance between normal pool elevation and the bottom of the beams is less than 10 feet, weathering steel shall not be used. These 8 ft. and 10 ft. dimensions are taken from FHWA Technical Advisory 5140.22, which is available at the following location:

[1]http://www.fhwa.dot.gov/bridge/t514022.cfm

All surfaces are blast cleaned to remove mill scale and to promote a uniform weathering appearance. Also, protection measures for substructure concrete surfaces vulnerable to staining shall be as directed by the District. See Section 3.1.3 for applicable General Notes when weathering steel is specified.

For bridges over waterways with significant boat traffic, some districts have opted to use weathering steel despite the aesthetic concerns. Painting of all areas of weathering steel within 10 feet of substructure units has been performed to limit staining. Other details such as drip plates may also limit staining of substructure units. These are viable options and may be utilized at the discretion of the district.

Cost: Weathering steel is the least expensive coating strategy, with initial per-pound costs that are around 90% of painted steel and 70% of galvanized or metallized steel. Comparatively, there is very little up-front cost for using weathering steel and it does not require recoating. The additional cost of painting or metallizing beam ends is marginal when compared to the total cost of the structural steel.

Durability: Weathering steel is durable if it can remain dry and free of chlorides. Planners should note that this is not always possible, and that areas near deck joints and bridge fascias have been known to develop pack rust concerns. Due to these concerns, some districts are not in favor of utilizing weathering steel.

Aesthetics: Weathering steel drips rust when wet, which is not considered to be aesthetically pleasing. Planners should account for this when choosing a coating strategy.

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4) Painted Steel

Painted steel is a coating strategy wherein three coats of paint are applied to the steel. The first coat, or primer, consists of a zinc-rich paint that is intended to provide cathodic protection to the steel. The second coat, or intermediate coat, is intended to provide protection to the primer. The third coat, or topcoat, is an aesthetic treatment that also protects the intermediate coat from environmental elements and UV effects.

IDOT currently uses two paint systems for new steel. The systems are abbreviated by the material properties of their three coats:

- IZ/AC/AC- Inorganic Zinc-Rich Primer (IZ), Acrylic Intermediate Coat (AC), Acrylic Topcoat (AC)
- OZ/E/U- Organic Zinc-Rich Primer (OZ), Epoxy Intermediate Coat (E), Urethane Topcoat (U)

IZ/AC/AC is the preferred system for new painted structures. The Department has had over 30 years of experience with this primer. When compared to the OZ/E/U system, it provides better corrosion protection. The IZ primer is sensitive to imperfections in the steel and therefore the primer must be applied in the shop. It requires exposure to humidity to complete its curing, which requires the midcoat and topcoats to be applied in the field. AC coats also require a higher ambient temperature (55 degrees Fahrenheit) to cure and may not be applicable in cold-weather work. Note that the use of heaters to help cure paint systems is common. Therefore, the outside ambient temperature may be considerably less than 55 degrees and the paint system will still cure appropriately due to the heaters. However, the IZ/AC/AC system should not be used in extreme cold such as the middle of winter in northern Illinois, as the heaters may not be able to adequately warm the containment.

OZ/E/U utilizes a primer with a quicker recoat window and cures independently of the ambient humidity, which allows the entire system to be applied in the shop. This is advantageous when conditions require that steel be erected quickly, i.e., traffic control is a concern. OZ/E/U also has a lower curing temperature (35 degrees Fahrenheit) and may be utilized in colder conditions than the IZ/AC/AC system. The system is not preferred for new steel but may be chosen if traffic control or temperature requirements are a concern.

The Central Bureau of Materials maintains an approved list of paint products that are compliant with the above systems.

Recoating of painted steel, which is typically required every 25 years, can be very expensive, and negatively affects the life-cycle cost of the bridge. For this reason, painted steel also may be metallized

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at beam ends to increase the service life of the coating system at these locations. The additional cost of metallizing beam ends is marginal when compared to the total cost of the structural steel, and a topcoat may be provided over the metallizing for aesthetic purposes.

Painted steel is typically used in grade separation applications, where the motoring public is cognizant of the aesthetics of the structure.

Cost: Painted steel upfront costs are generally 10% higher than weathering steel, and 20% lower than galvanized or metallized steel. Life-cycle costs are substantially higher than any other system due to recoating requirements.

Durability: Painted steel requires recoating approximately every 25 years, sometimes sooner if located in poor environmental conditions. It should be anticipated that a painted steel bridge will be repainted twice over the course of its service life. It is seen as more durable than weathering steel but less durable than galvanized or metallized steel.

Aesthetics: Painted steel is considered to be aesthetically pleasing. IDOT currently allows four standard colors for painted steel. See Section 3.3.29 for design information.

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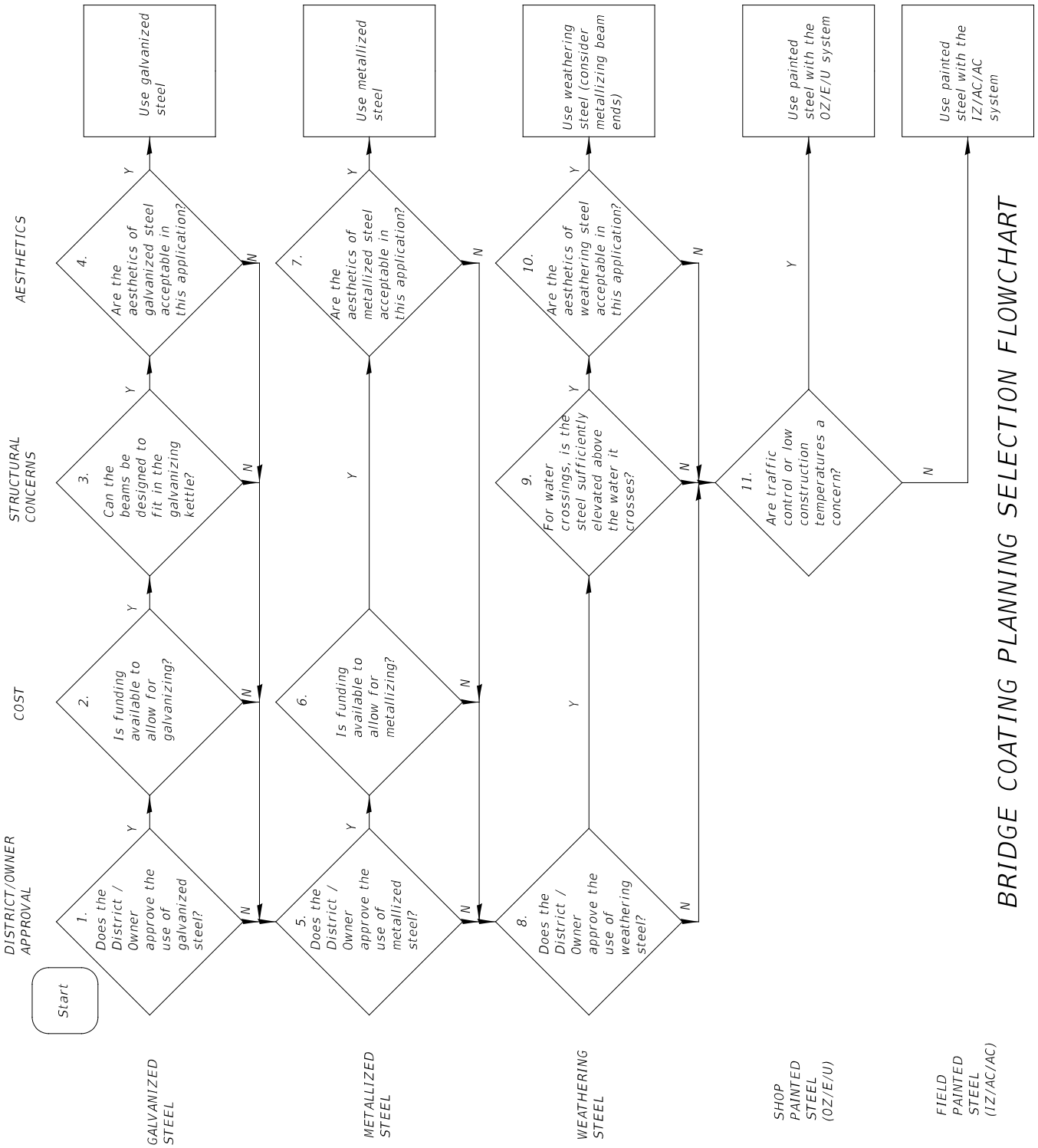


Figure 2.3.6.1.1-1

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Q&A Items for Bridge Coating Section Flowchart

1. *Does the District/Owner approve the use of galvanized steel?*
   Bridge planners should verify with District Studies and Plans Engineers and District Bridge Maintenance Engineers and/or District Paint Technicians, or Owner for LPA structures whether the district/owner has a preference for or against galvanized steel.

2. *Is funding available to allow for galvanizing?*
   Galvanized steel costs around 40% more than weathering steel and 25% more than painted steel. For bridges with large amounts of steel, this is a substantial additional cost. Bridge planners should evaluate whether this additional cost is acceptable to the district/owner.

3. *Can the beams be designed to fit in the galvanizing kettle?*
   In general, steel structures with individual span lengths less than or equal to 120 feet and very little curvature will meet the requirements of Table 2.3.6.1-1 which gives the beam segment requirements.

   Additional splices to be placed in high stress regions such as midspan or near piers and splices to be placed over live rail traffic are discouraged because they may complicate erection procedures.

4. *Are the aesthetics of galvanized steel acceptable in this situation?*
   Bridge planners should inquire if the dull gray and potentially patchy color is aesthetically acceptable in the location of the bridge. If not, bridge planners should inquire if the district/owner is amenable to fascia painting over the galvanizing, knowing it that will incur an additional cost of around 5% of the steel cost and the fascias will eventually need to be repainted.

5. *Does the District/Owner approve the use of metallized steel?*
   Bridge planners should verify with District Studies and Plans Engineers and District Bridge Maintenance Engineers and/or District Paint Technicians, or Owner for LPA structures, whether the district/Owner has a preference for or against metallized steel.

6. *Is funding available to allow for metallizing?*

   Metallized steel can be 10% to 40% more in cost. For bridges with large amounts of steel, this is a substantial additional cost. Bridge planners should evaluate whether this additional cost is acceptable to the district/owner.

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Q&A Items for Bridge Coating Section Flowchart (Continued)

7. *Are the aesthetics of metallized steel acceptable in this situation?*
   Bridge planners should inquire if the dull gray and potentially patchy color is aesthetically acceptable in the location of the bridge. If not, bridge planners should inquire if the district/owner is amenable to fascia painting over the metallizing, knowing that that will incur an additional cost of around 5% of the steel cost and the fascias will eventually need to be repainted.

8. *Does the District/Owner approve the use of weathering steel?*
   Bridge planners should verify with District Studies and Plans Engineers and District Bridge Maintenance Engineers and/or District Paint Technicians, or Owner for LPA structures, whether the district/owner has a preference for or against weathering steel.

9. *For water crossings, is the steel sufficiently elevated above the water it crosses?*
   Bridges with steel beams too close to constant bodies of water are not good candidates for weathering steel because the constant humidity will generate pack rust. Bridge planners should verify that the steel beams are at least 8 ft. above the average streamflow elevation for stream crossings/running water or 10 ft. above normal pool elevations of lakes/stagnant water they cross.

10. *Are the aesthetics of weathering steel acceptable in this situation?*
    Weathering steel bridges tend to cause rust stains on adjacent substructure units and roadways. Bridge planners should verify that this is not an aesthetic concern prior to choosing weathering steel as a paint strategy. In general, weathering steel should not be used on structures at grade crossings or bridges over boat traffic. In the past, structures with weathering steel over boat traffic have been painted at substructure units to reduce the amount of staining at these locations. This is a viable option and may be used at the discretion of the district/owner.

11. *Are traffic control or low construction temperatures a concern?*
    Bridge planners should choose field painting with the IZ/AC/AC system whenever feasible. Non-feasibility occurs when traffic control concerns limit the amount of time available to paint the bridge, or low erection temperatures (i.e. middle of winter in the northern portion of Illinois) will affect the curing of the paint system.

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2.3.6.1.2 Prestressed Concrete Deck Beams

Precast Prestressed Concrete Deck Beams are available in 6 different depths (11" thru 42") and can be an economical option for structures with spans ranging from 15 to 100 feet. Some advantages include relatively shallow overall superstructure depth, and reduced construction time. A reinforced, non-composite, 5 inch minimum Concrete Wearing Surface (CWS) shall be used on these types of structures for State routes. An initial 1 ¼ inch minimum Hot Mix Asphalt (HMA) wearing surface plus a waterproofing membrane system per Section 581 of the Standard Specifications may be used in lieu of the CWS on LPA projects. Approach slabs are required on State projects but are optional for Local projects. Base Sheets depicting the preferred application of deck beam superstructures are available at the IDOT CADD Bridge Cell Library.

PPC deck beams shall not be used on bridges with large vertical curves, superelevation, superelevation transitions, or with skew angles greater than 35 degrees. Also, changes in beam depths from span to span are not desirable. The standard details, base sheets and charts do not address these conditions. Exceptions may be allowed on a case by case basis when very unique circumstances dictate a need but are subject to approval by the Bureau of Bridges & Structures. Deck beam structures with all new substructure units and a total length equal to or less than 300 feet shall be fixed at all substructure units. An analysis for thermal forces is not required for structures within the 300 feet length limitation. Longer structures generally should incorporate an expansion joint; however, longer structures with all fixed supports may be permitted on a project-by-project basis in which case all thermal forces shall be accounted for in the design.

The selection charts illustrate the relationships between beam size and beam strand patterns, and utilize bar graphs to depict the maximum span length for each loading combination. The charts are configured such that the strand patterns are listed on the y-axis, the span lengths are listed on the x-axis. There is one chart for each beam size. For more detailed policies on the PPC deck beams see Section 3.5 and the "LRFD PPC Deck Beam Design" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design.

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The deck beam charts were developed using the loading cases and Design Criteria shown below:

Loading cases for 11 inch beams:

1. HMA wearing surface + Type CO-10 railing and curb.
2. Concrete wearing surface + future wearing surface + Constant slope parapet.

Loading cases for 17 inch thru 42 inch beams:

1. Bare deck beams + future wearing surface + Steel Railing.
2. HMA wearing surface + future wearing surface + Steel Railing.
3. Concrete wearing surface + future wearing surface + Steel Railing.
4. Concrete wearing surface + future wearing surface + Constant slope parapet.

Where:

Bare deck beams	=	No initial wearing surface
Concrete wearing surface	=	70 pounds per square foot
HMA wearing surface	=	40 pounds per square foot
Future wearing surface	=	50 pounds per square foot
Steel Railing	=	120 pounds per foot (includes Type SM, SMX and IL-OH railing)
Type CO-10 railing and curb	=	245 pounds per foot.
Constant slope parapet	=	570 pounds per foot (includes 39” and 44” parapet)

Additional Design Criteria:

1. LRFD 9th Edition.
2. ½ inch diameter low relaxation seven-wire strands, f_{pu} = 270,000 psi.
3. Concrete beam strengths f'_c of 6,000 psi with release strengths f'_{ci} of 5,000 psi.
4. HL-93 live load.
5. Live load distribution according to “(h)-connected only enough to prevent relative vertical displacement at the interface” of AASHTO Table 4.6.2.2.2b-1 (Contact BBS if additional information regarding this distribution is required)
6. Interior beam design.
7. 1 inch camber for dead load calculations.
8. Concrete wearing surface is considered non-composite.
9. Parapets, railings and curbs are distributed over 3 beams.

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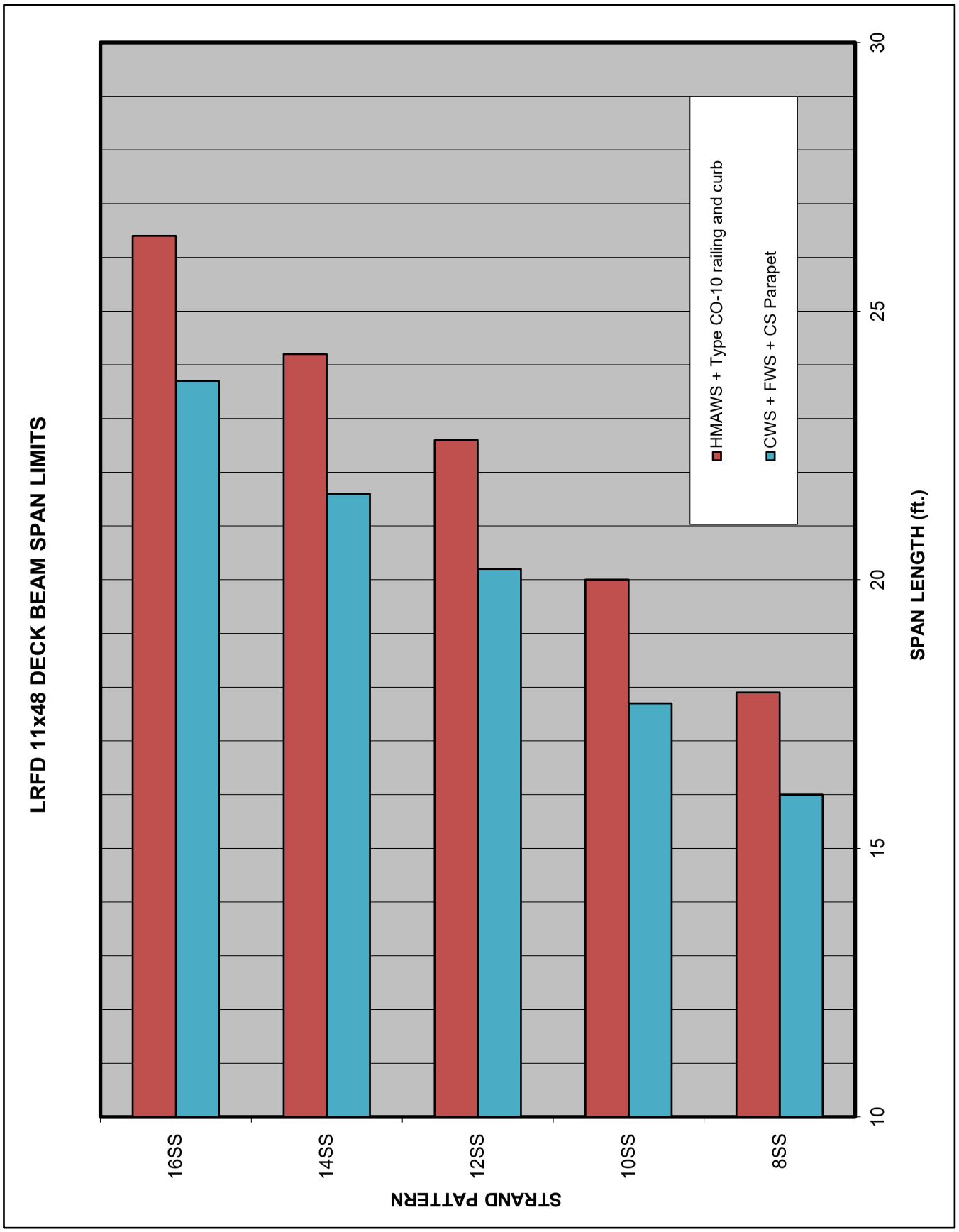


Figure 2.3.6.1.2-1

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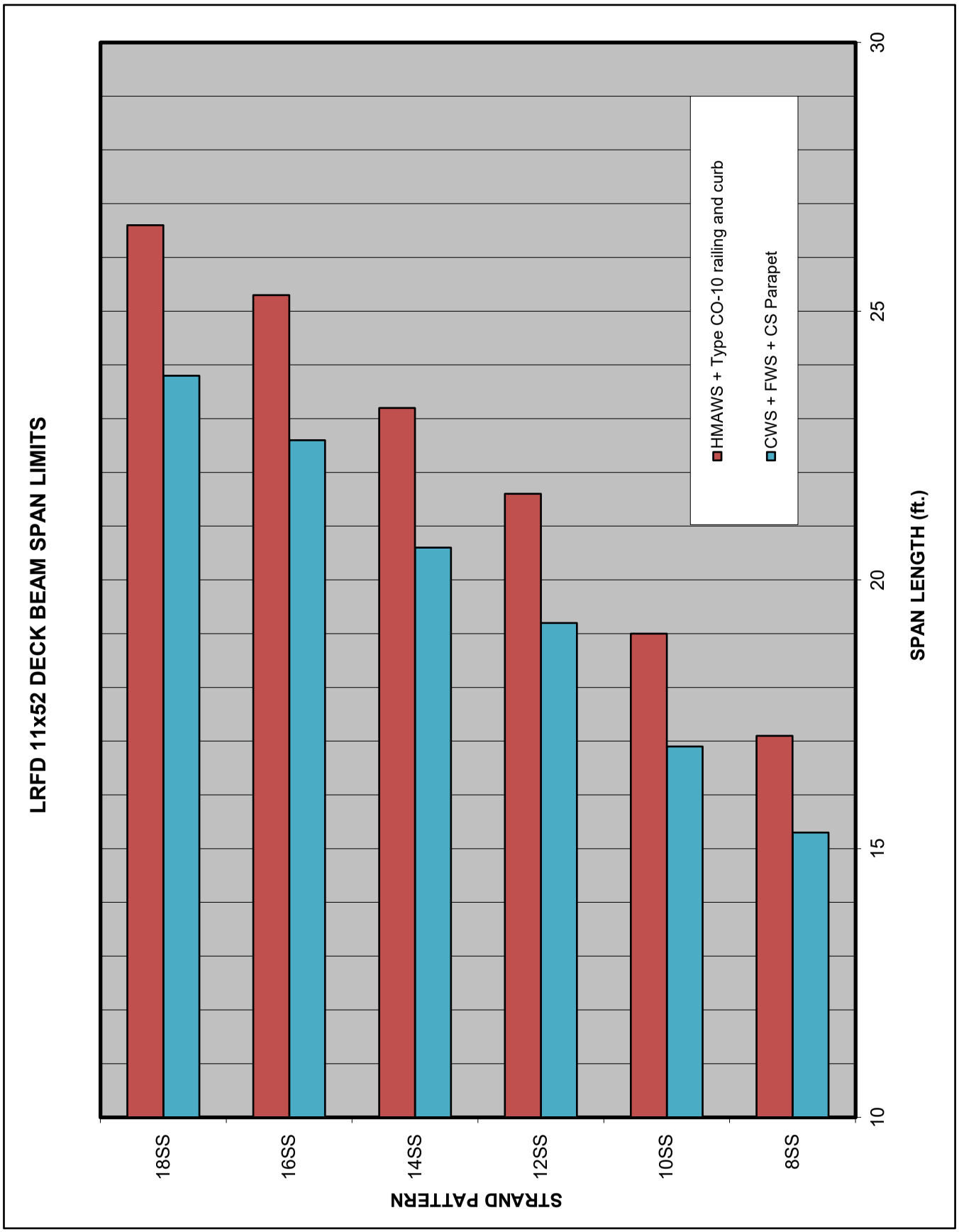


Figure 2.3.6.1.2-2

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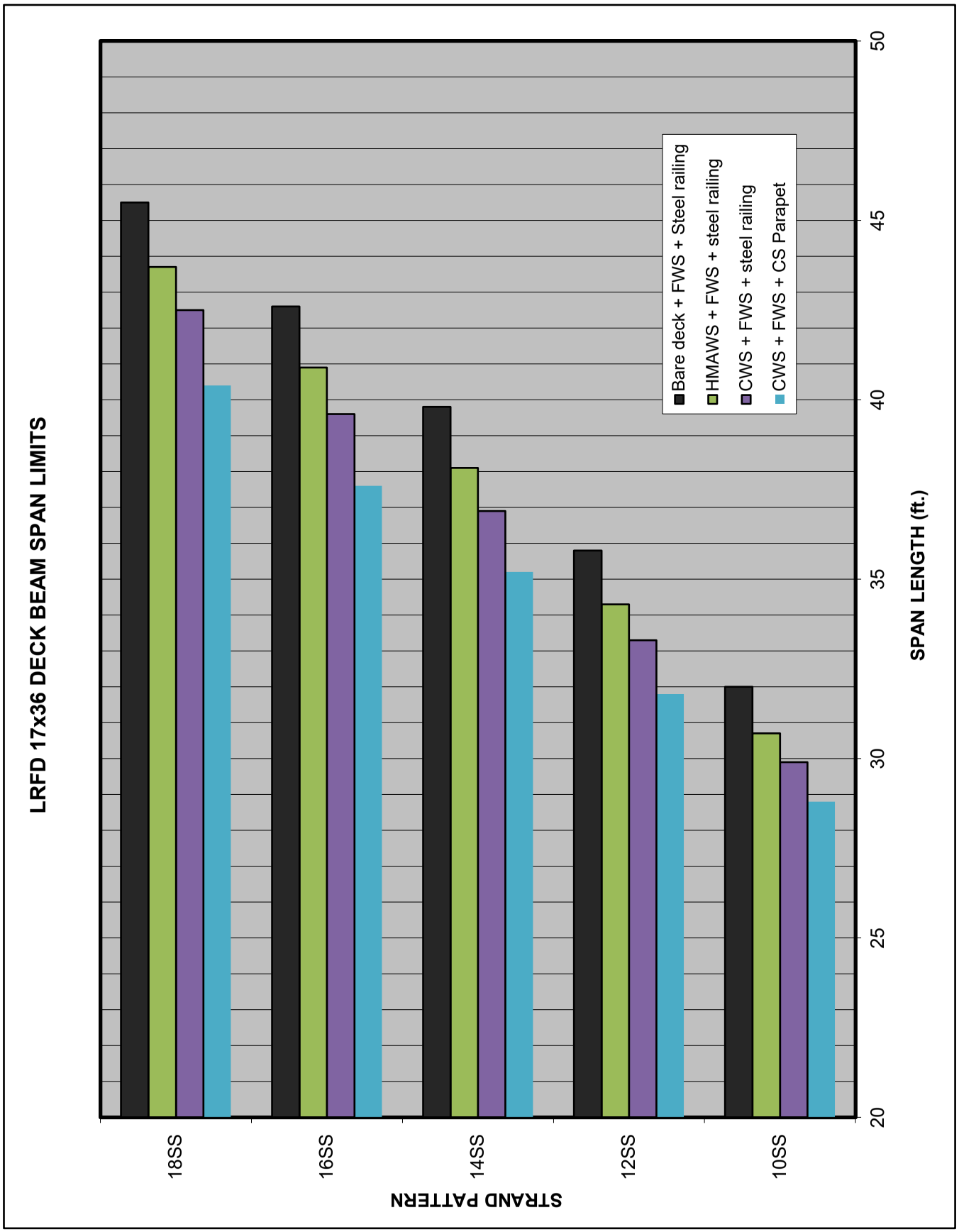


Figure 2.3.6.1.2-3

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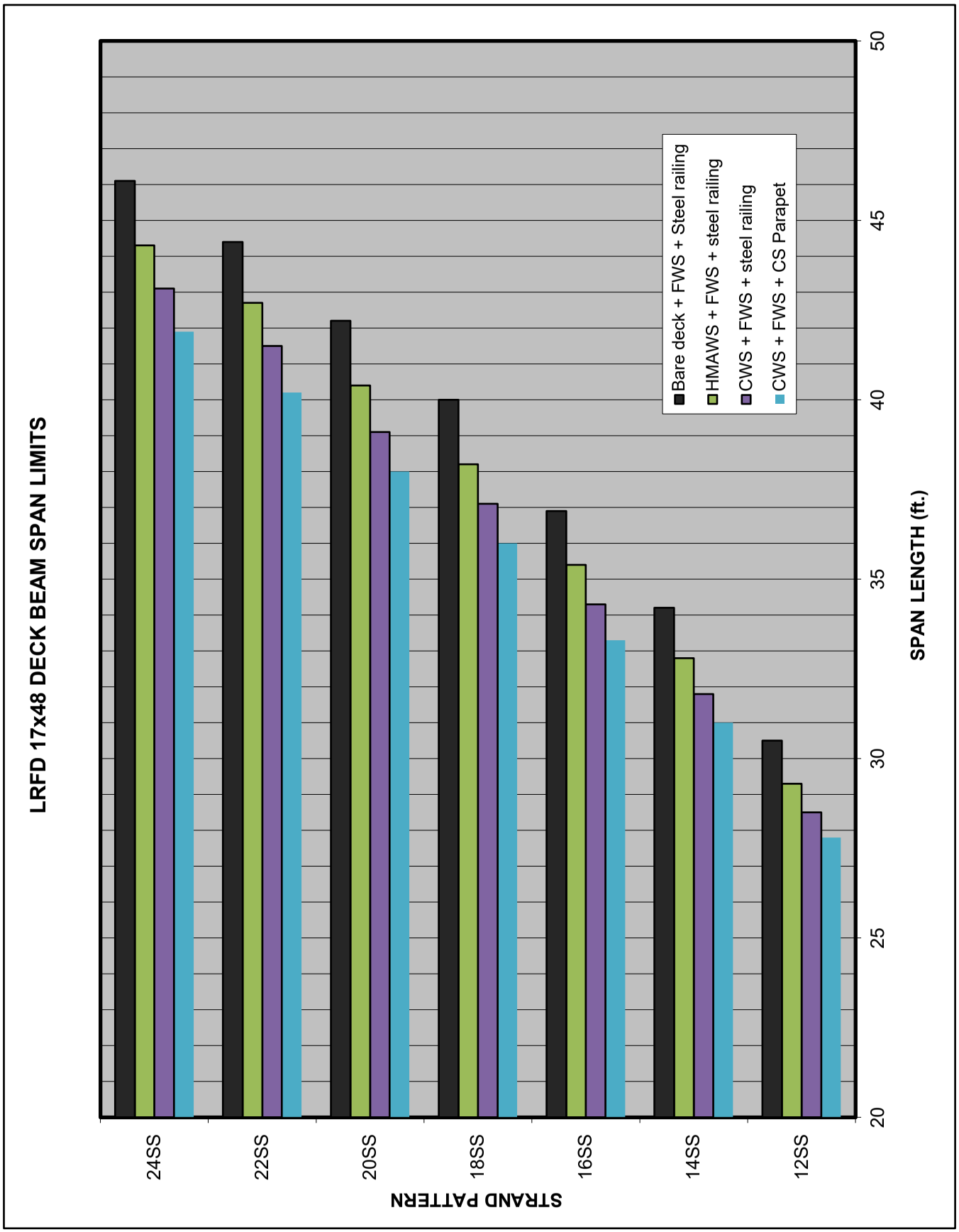


Figure 2.3.6.1.2-4

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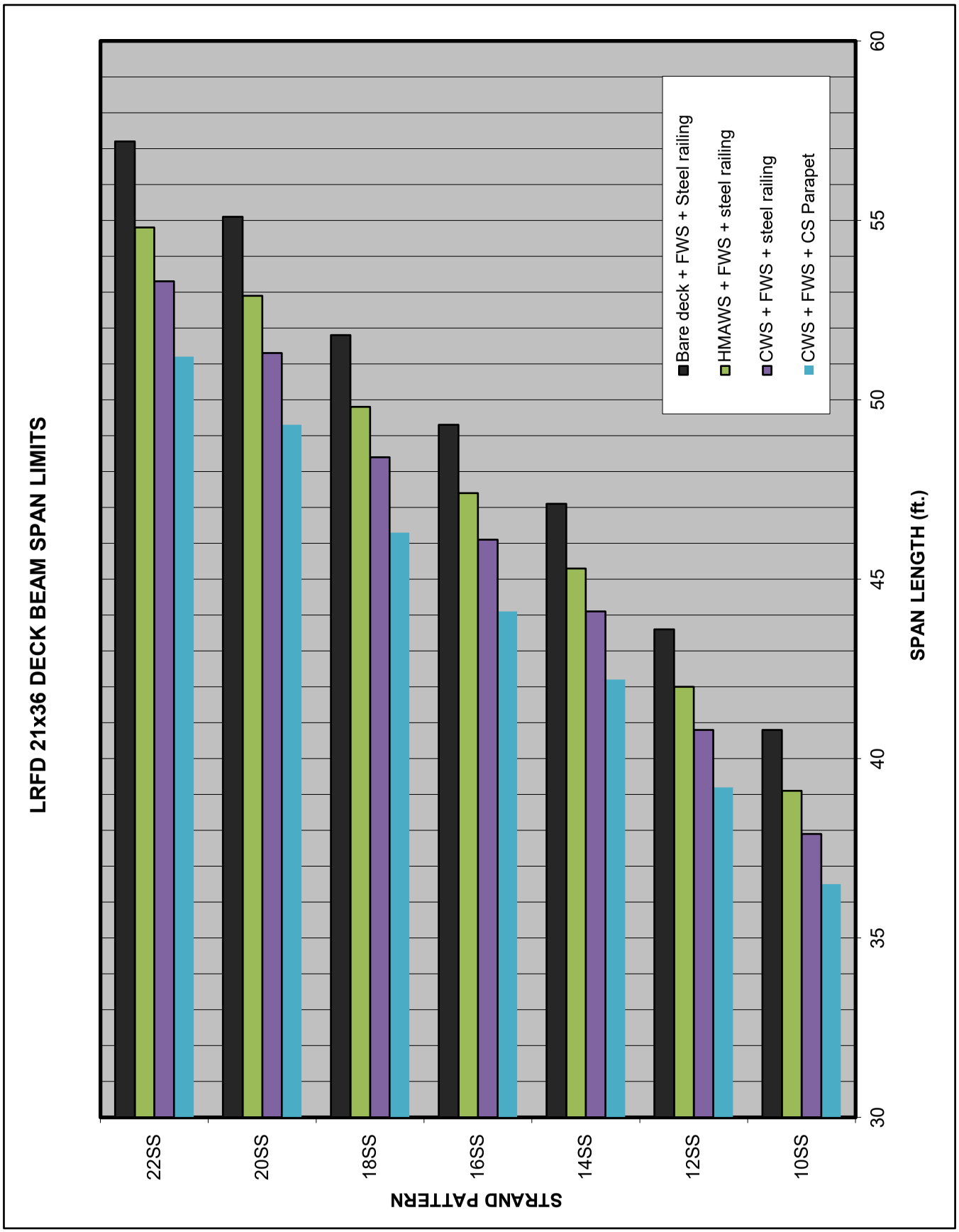


Figure 2.3.6.1.2-5

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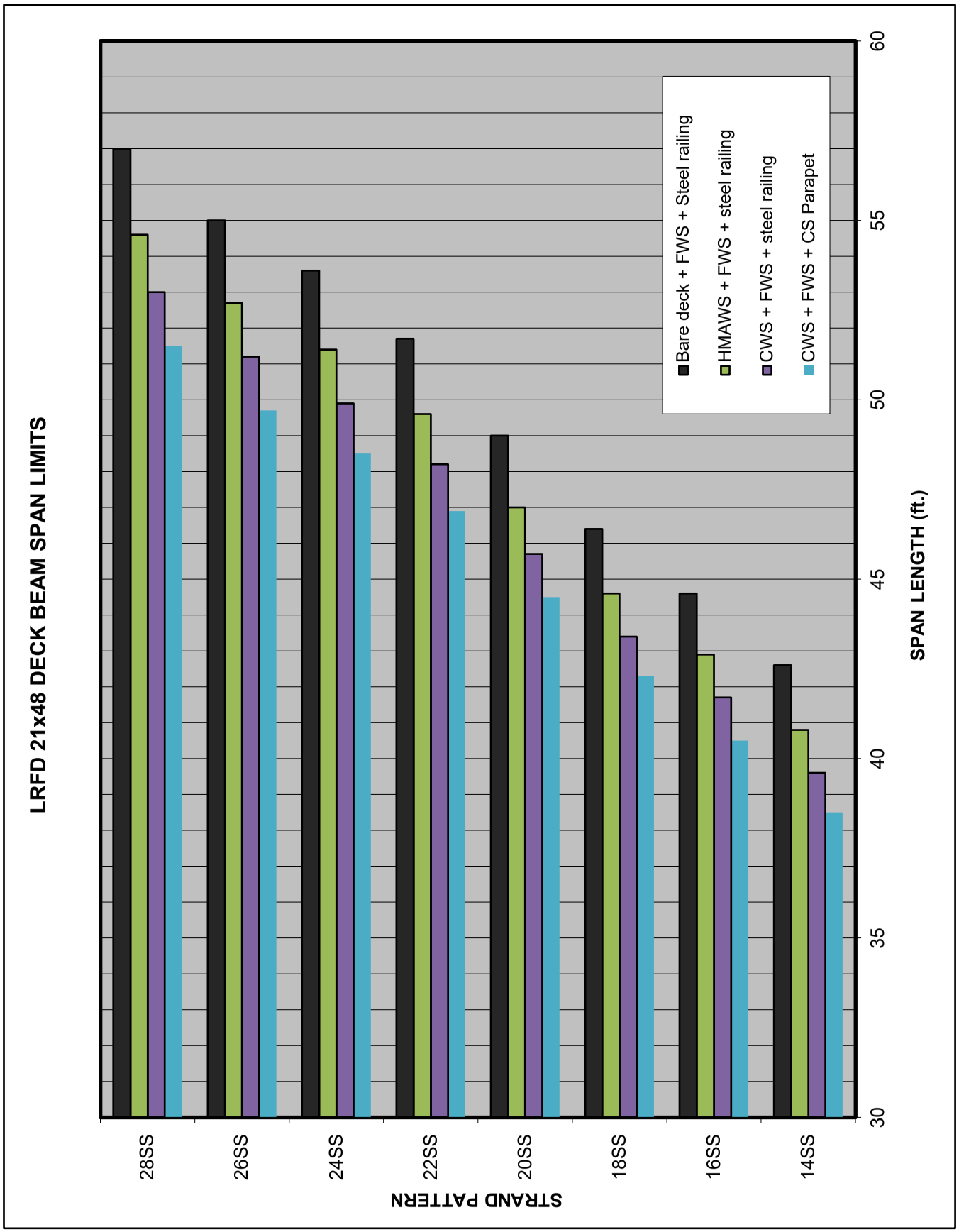


Figure 2.3.6.1.2-6

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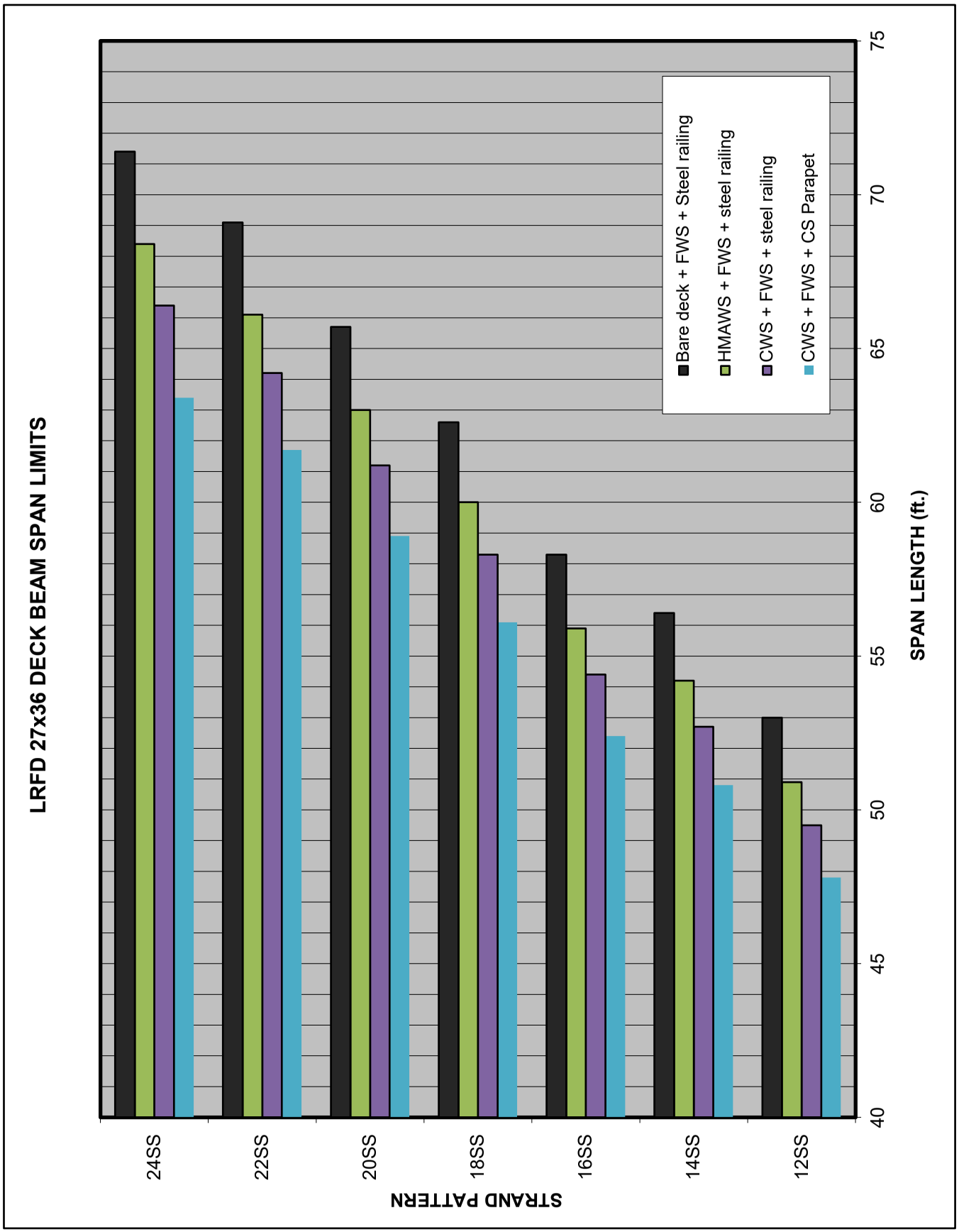


Figure 2.3.6.1.2-7

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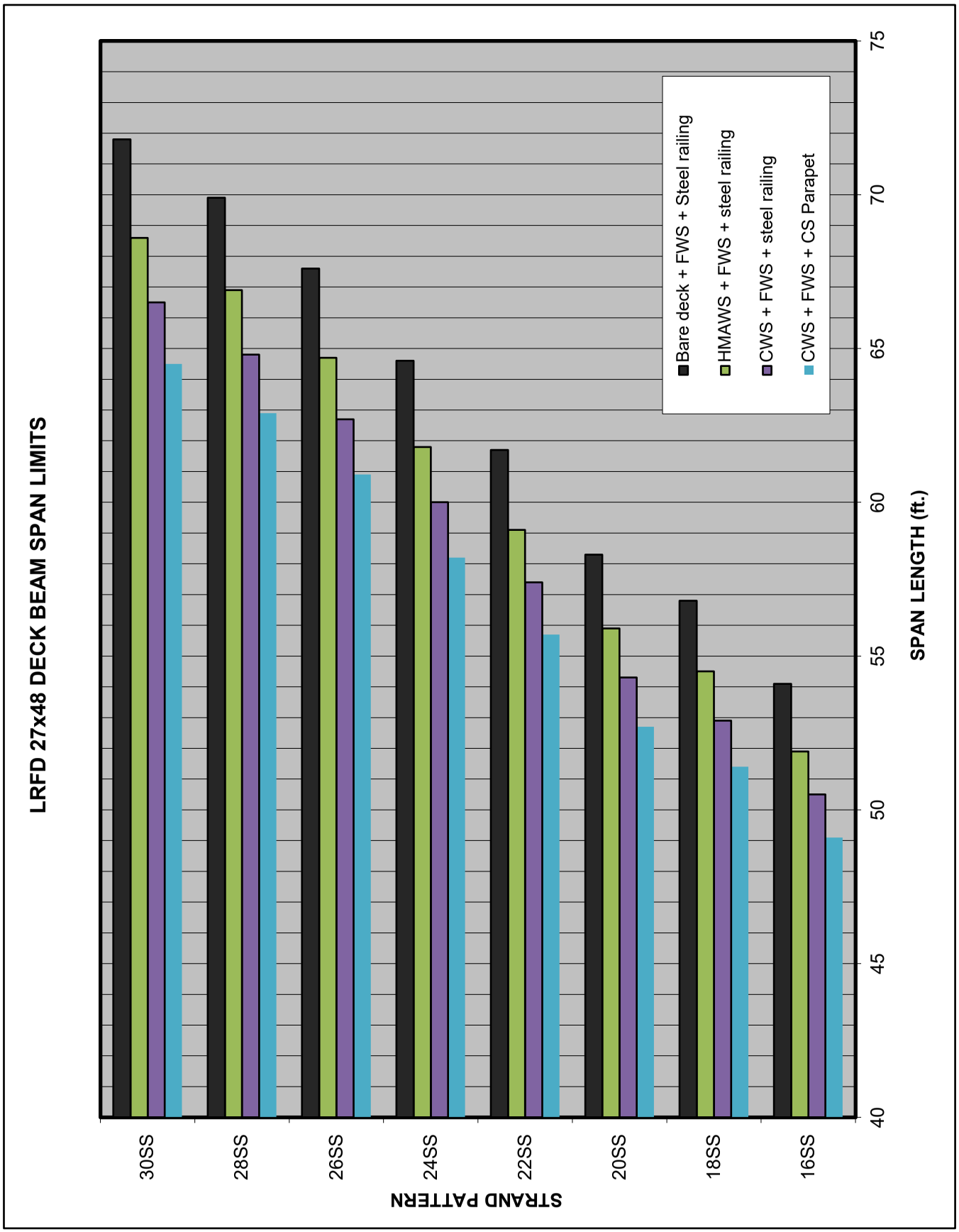


Figure 2.3.6.1.2-8

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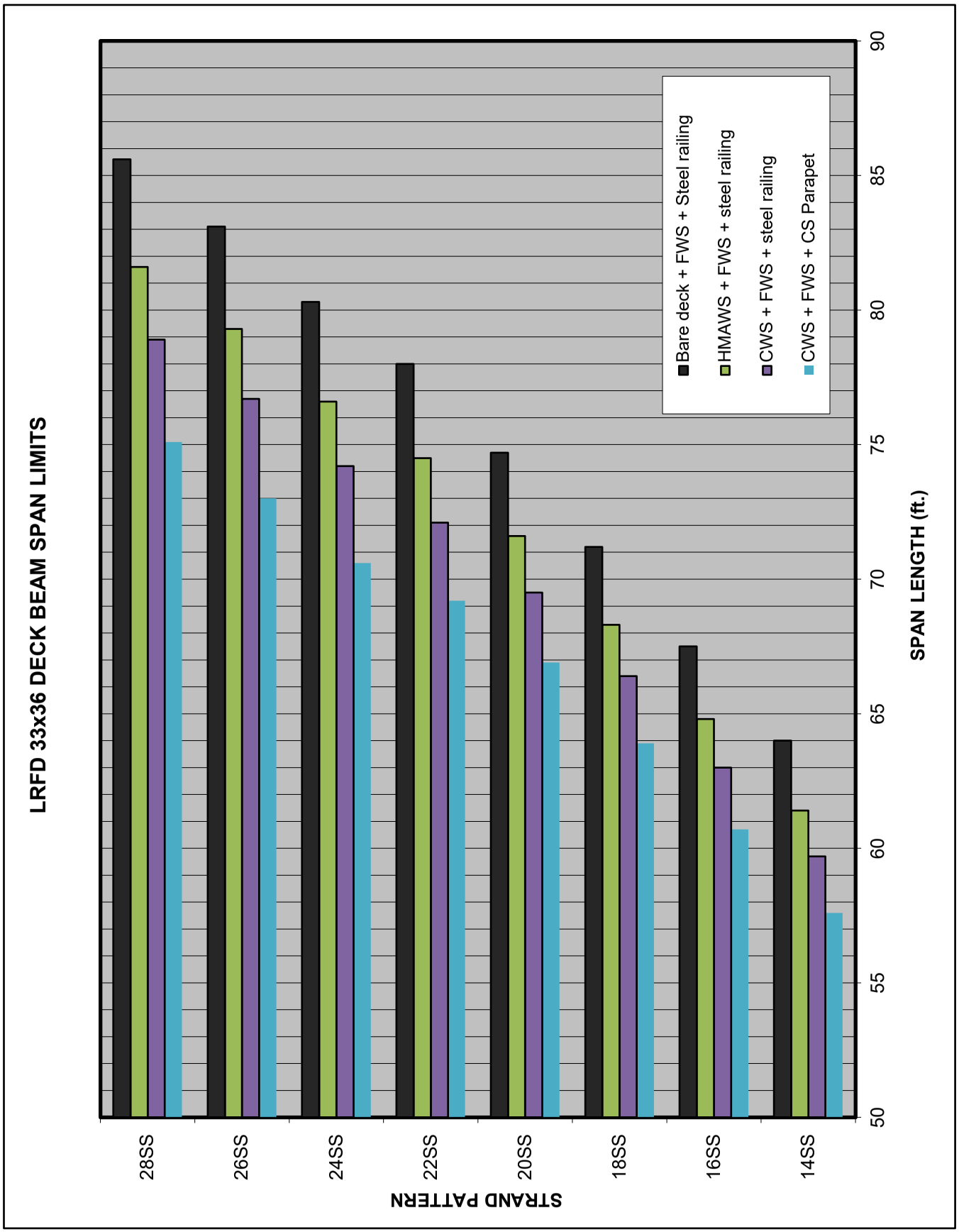


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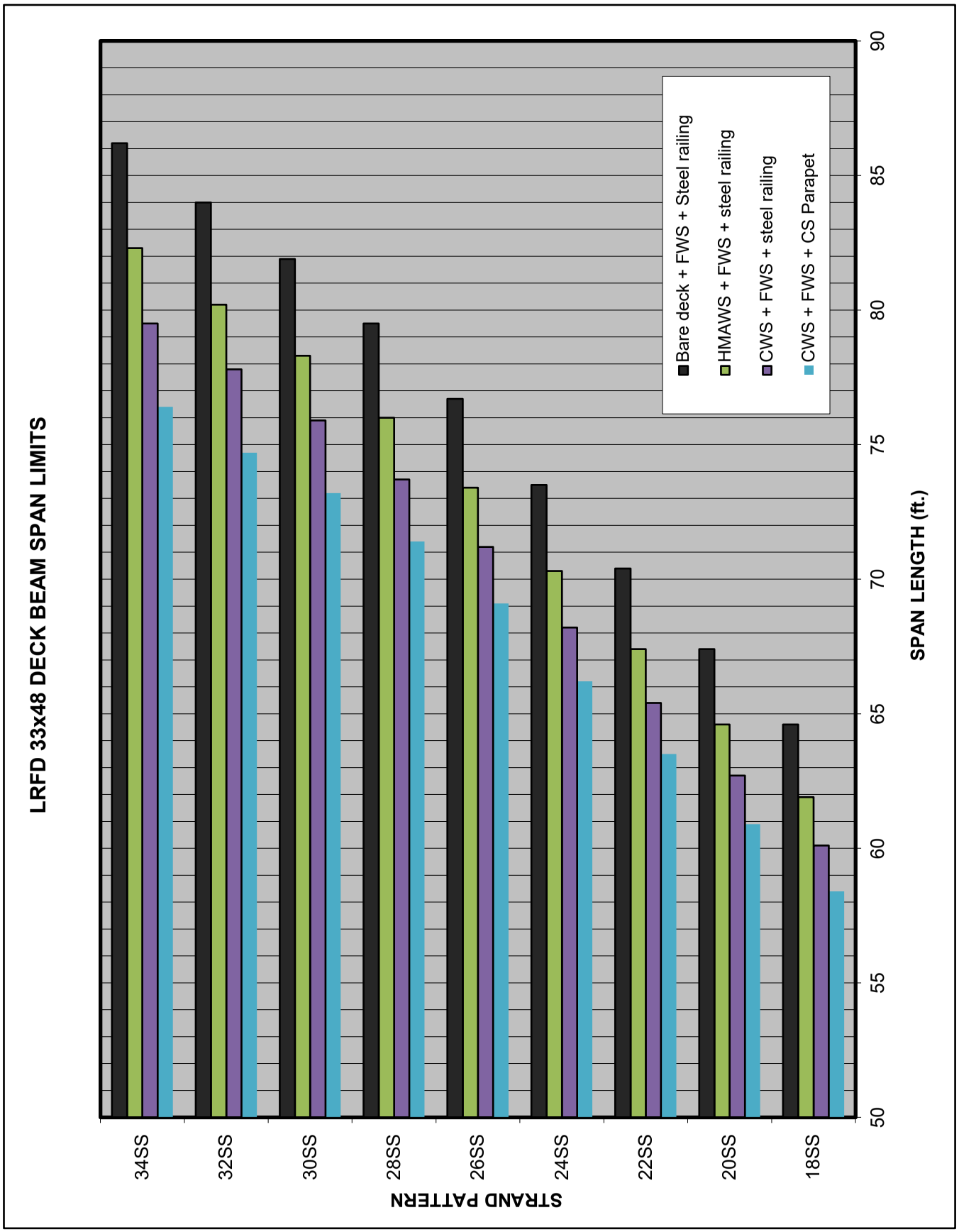


Figure 2.3.6.1.2-10

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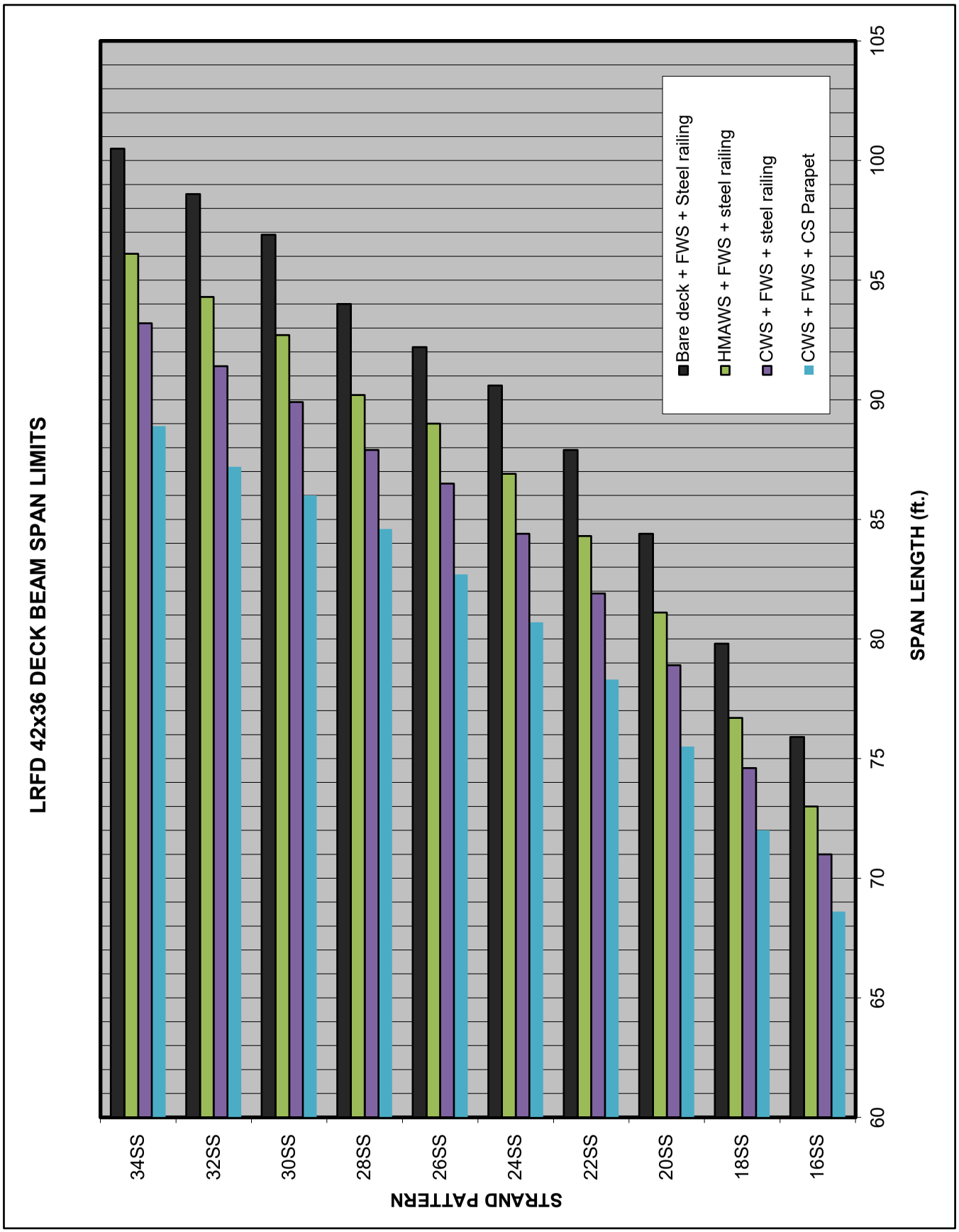


Figure 2.3.6.1.2-11

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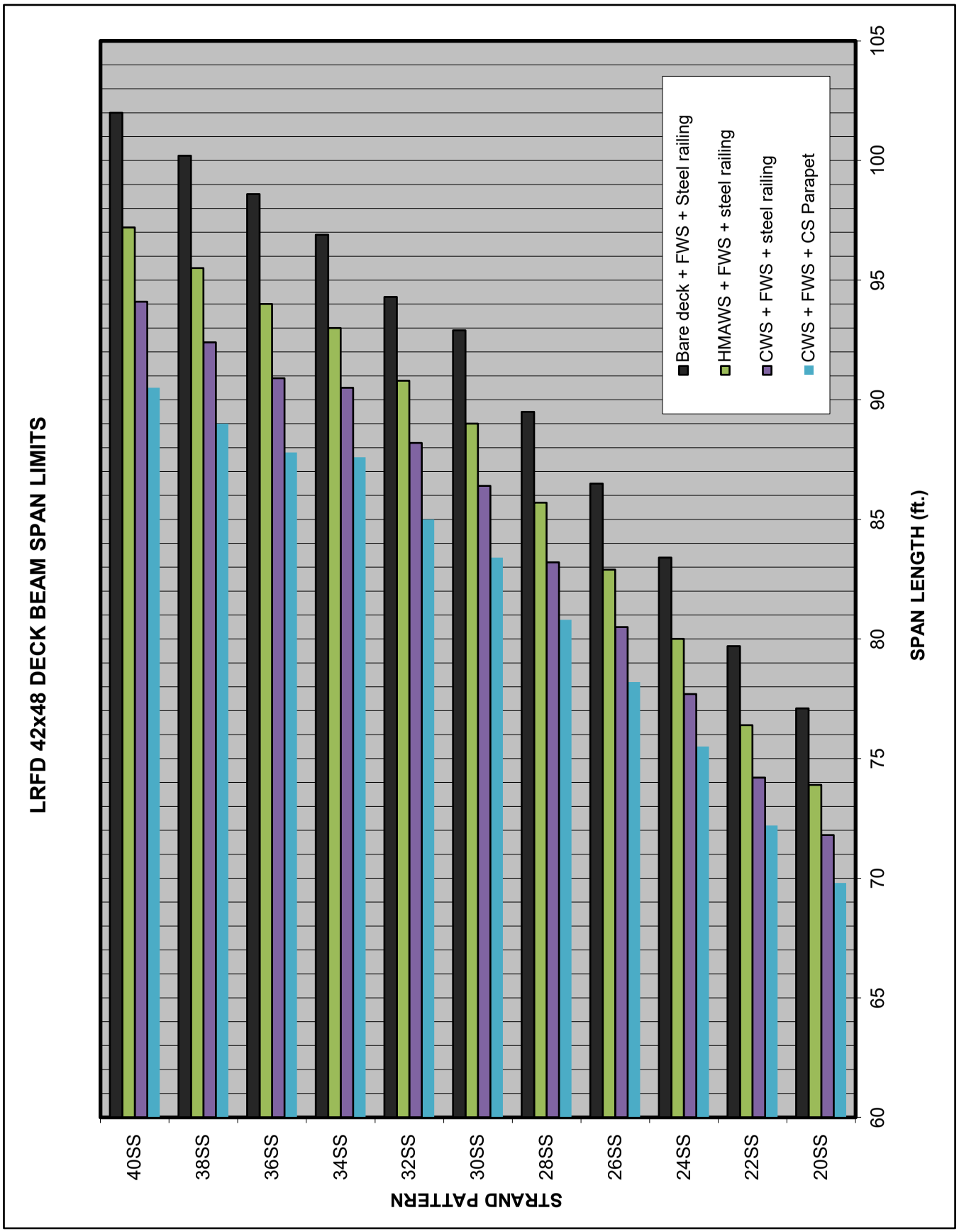


Figure 2.3.6.1.2-12

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2.3.6.1.3A Prestressed Concrete I-Beams and Bulb-T Beams

For PPC I-beams and bulb T-beams, selection charts have been developed to aid in determining beam size, beam spacing, and beam strand patterns for a given span length. These charts illustrate a bar graph depicting the span ranges for each strand pattern with beam spacings ranging from 4 ft. 6 in. to 9 ft. 0 in. The charts are configured such that the strand patterns are listed on the y-axis, the span lengths are listed on the x-axis and the beam spacings are listed on the bars of the bar graph. There are two charts for each beam, one for simple span designs and one for multi-span designs. The scales depicting the span length ranges for the x-axis were chosen for presentation purposes and therefore the absolute minimum span length for the strand pattern may not be defined on the chart. These limitations are available in Tables 3.4.3.1-1 through 3.4.3.1-25. When possible, the minimum span length limitations are depicted by darkening the bars for the lower boundary.

To use the chart, enter a span length starting from the bottom of the chart and go up until a strand pattern is intersected with a beam spacing equal to or greater than the desired beam spacing. For example, a 36 in. I-beam with a 57 ft. span and 6 ft. beam spacing would require strand pattern 18DS or 20DSH with 18DS being the most economical.

The charts for PPC I-beams (Figure 2.3.6.1.3-1 through Figure 2.3.6.1.3-8) and bulb T-beams (Figure 2.3.6.1.3-9 through Figure 2.3.6.1.3-12) were developed using the following criteria:

1. LRFD 3rd Edition with 2005 interims
2. ½ inch diameter low relaxation seven-wire strands, f_{pu} = 270,000 psi.
3. Concrete beam strengths f'_c of 6,000 psi and 7,000 psi with release strengths f'_{ci} of 5,000 psi and 6,000 psi respectively. Concrete deck strength of 4,000 psi.
4. HL-93 live loading using simplified distribution. See Section 3.1.12.
5. 8 inch deck thickness
6. 1 inch average fillet height for dead load only. Fillet not included in section properties.
7. 6 beam lines.
8. Standard F-shape concrete barrier weighing 450 pounds per linear foot.
9. 50 psf future wearing surface.
10. Multi-span charts are based on two equal spans.

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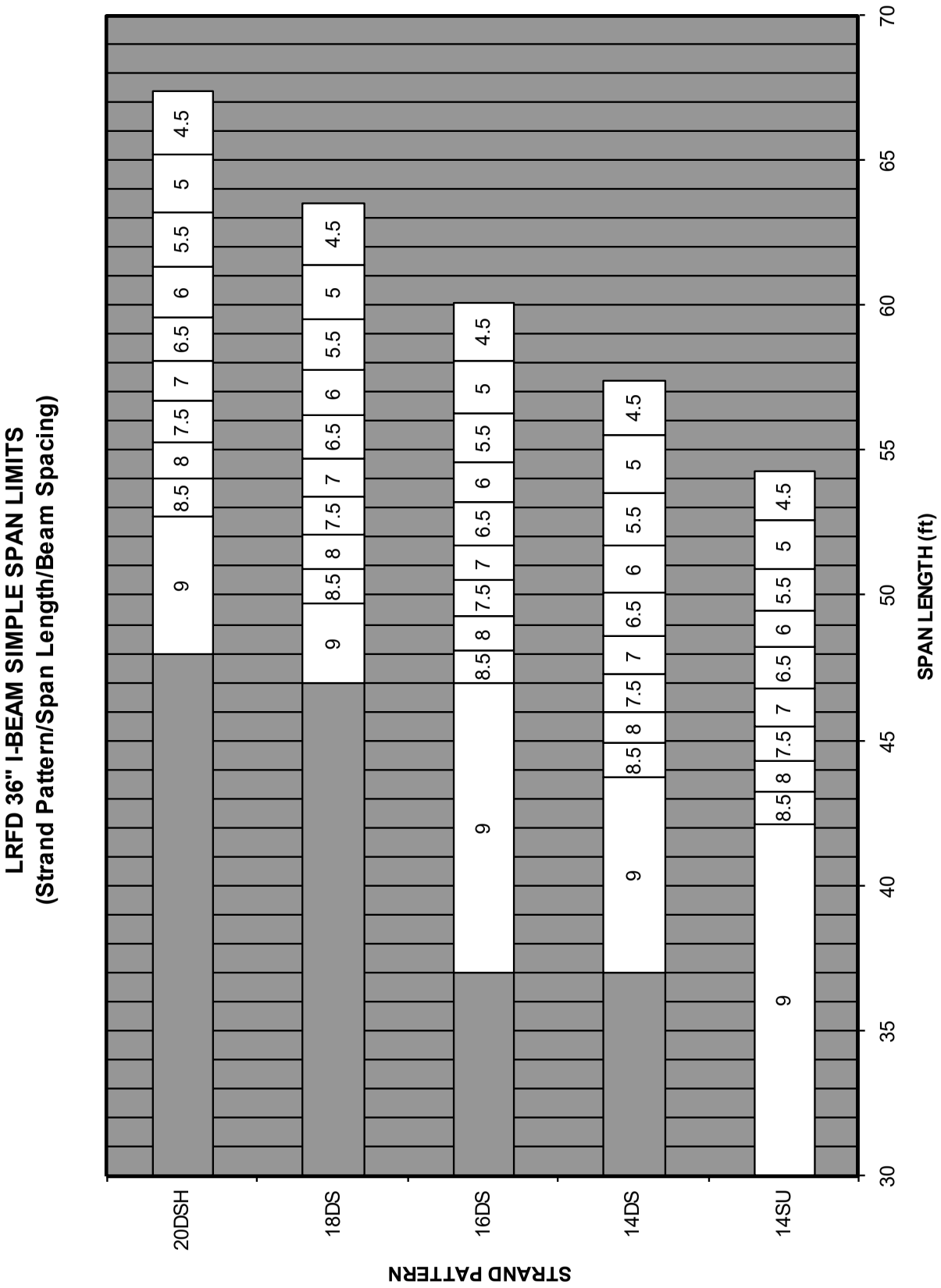


Figure 2.3.6.1.3-1

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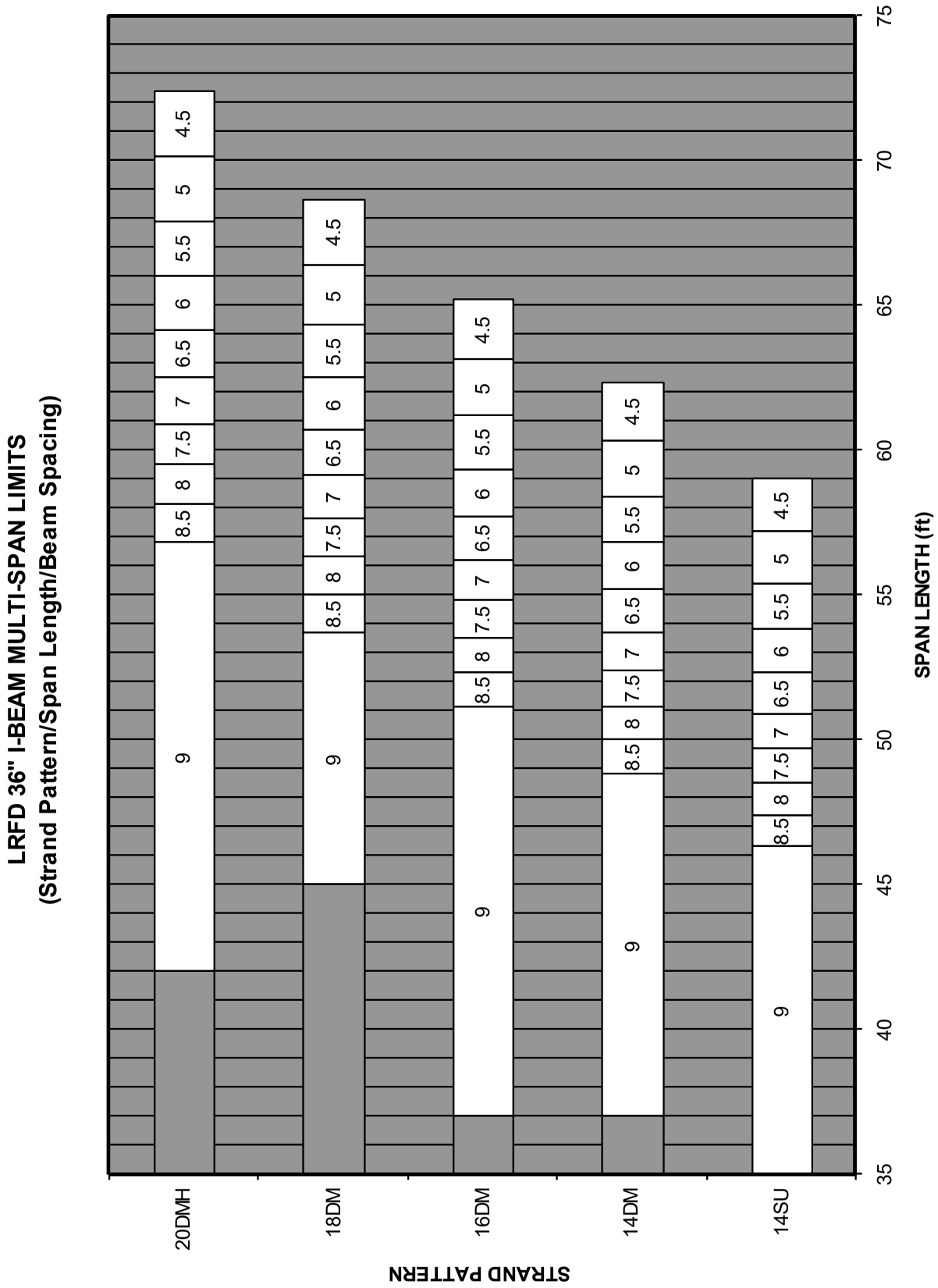


Figure 2.3.6.1.3-2

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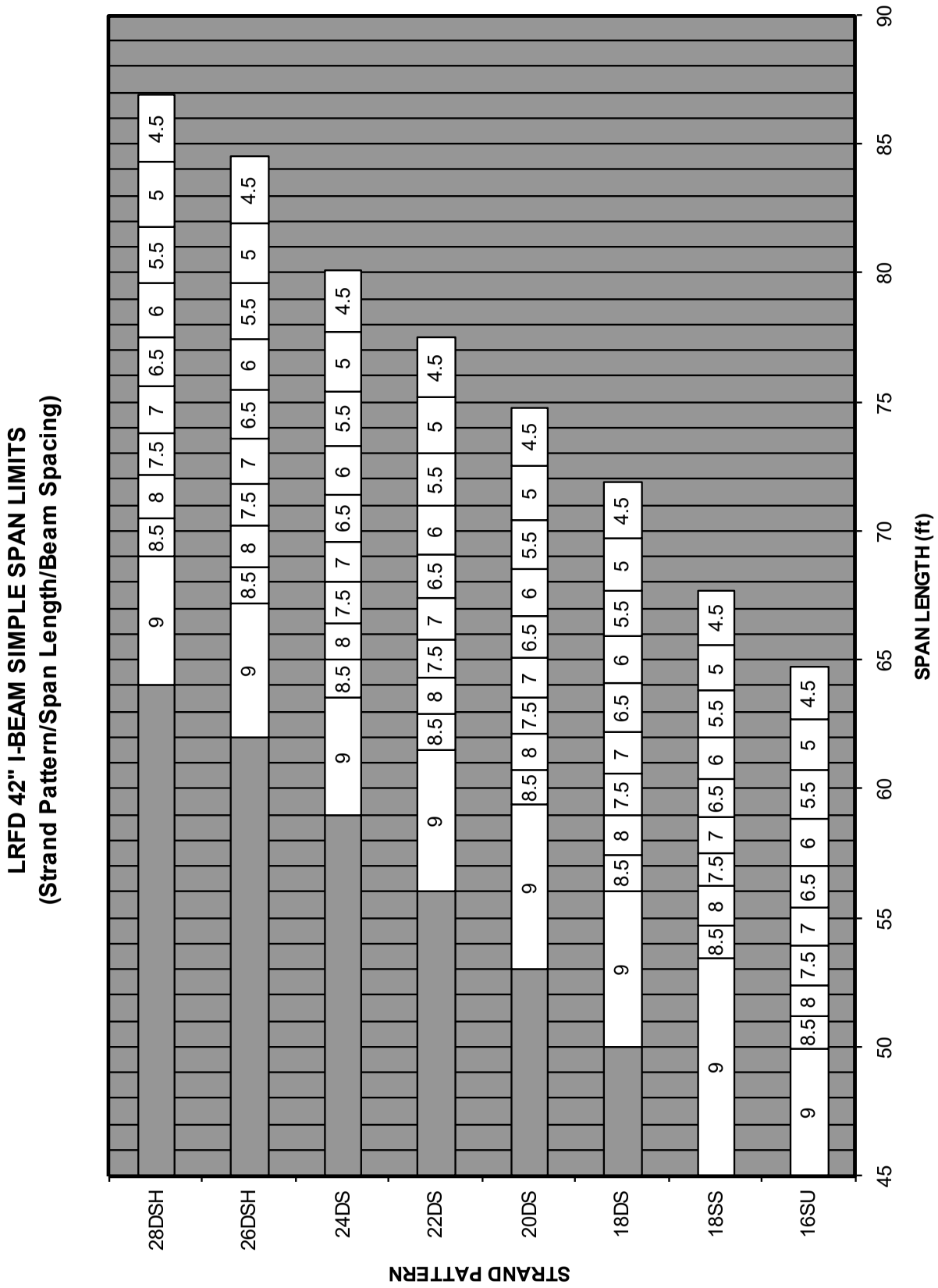
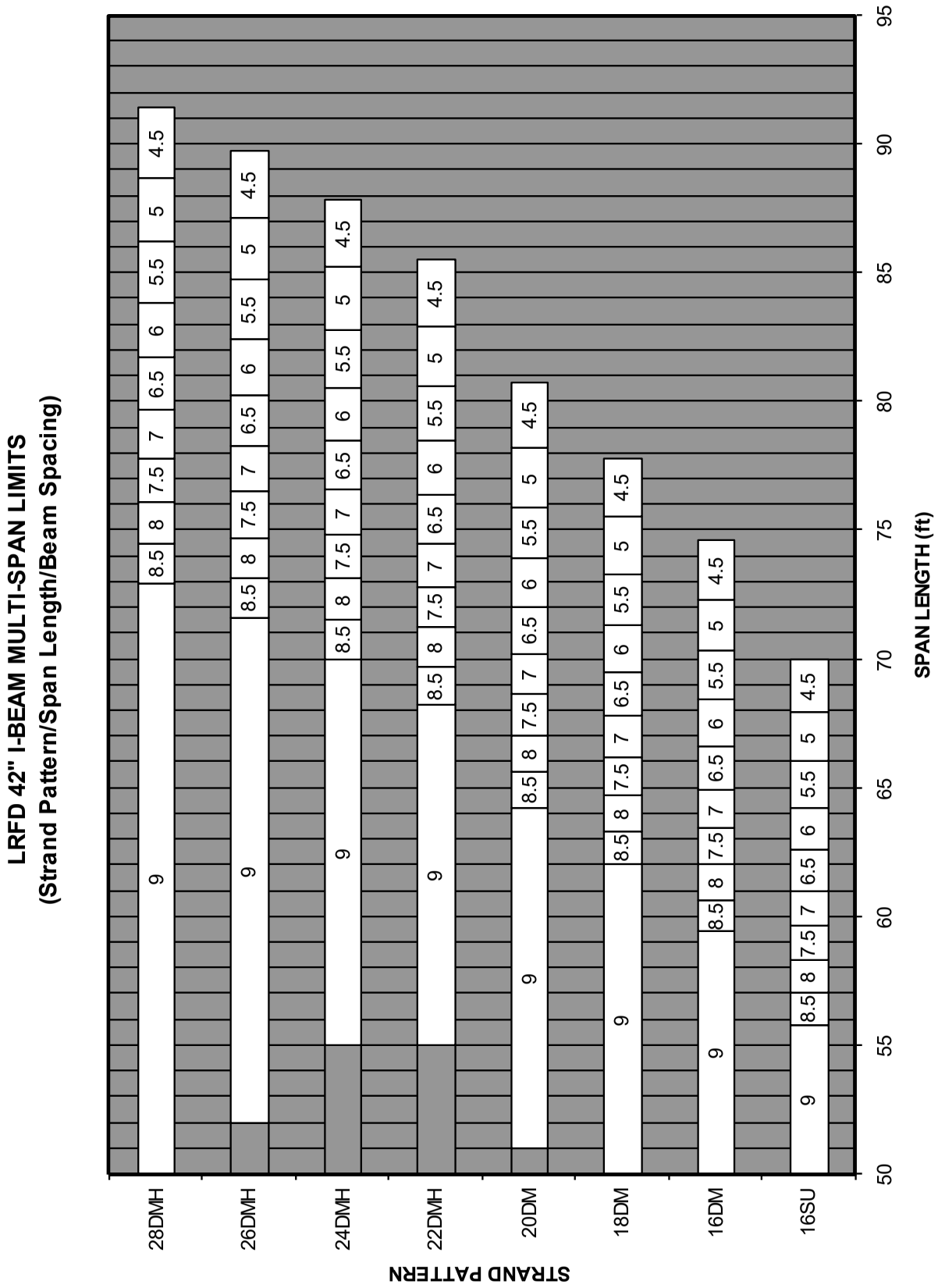


Figure 2.3.6.1.3-3

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Figure 2.3.6.1.3-4

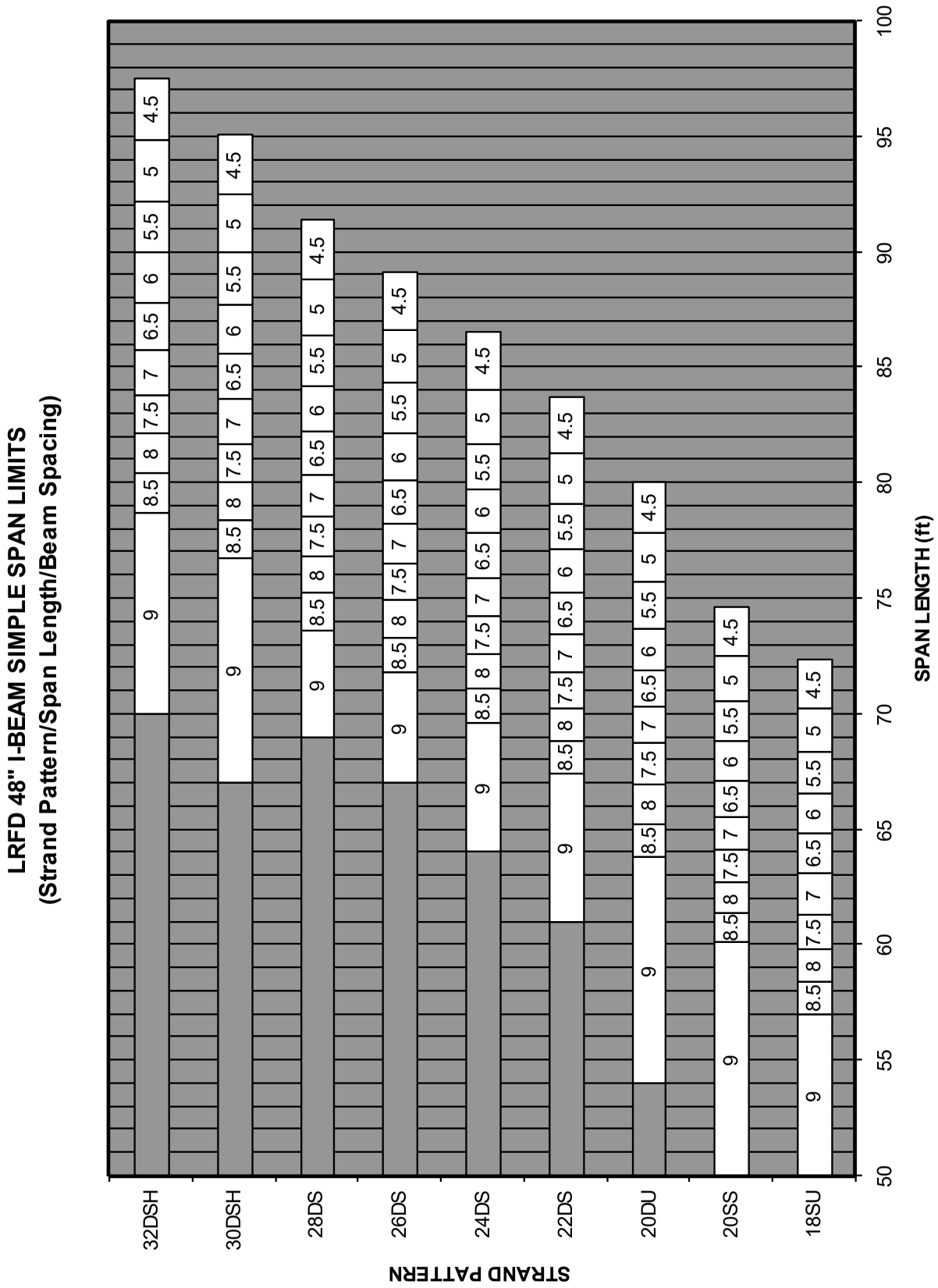


Figure 2.3.6.1.3-5

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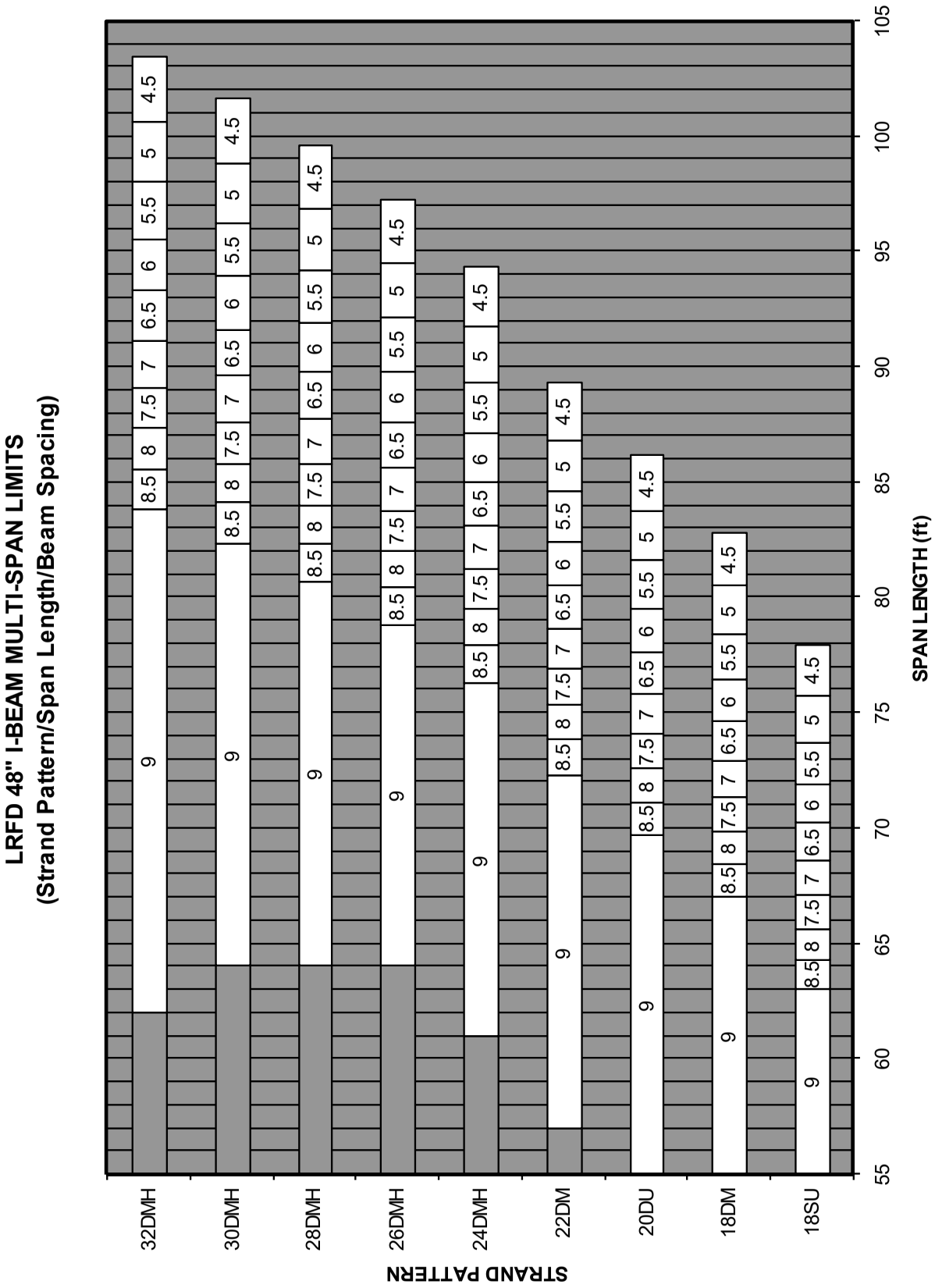


Figure 2.3.6.1.3-6

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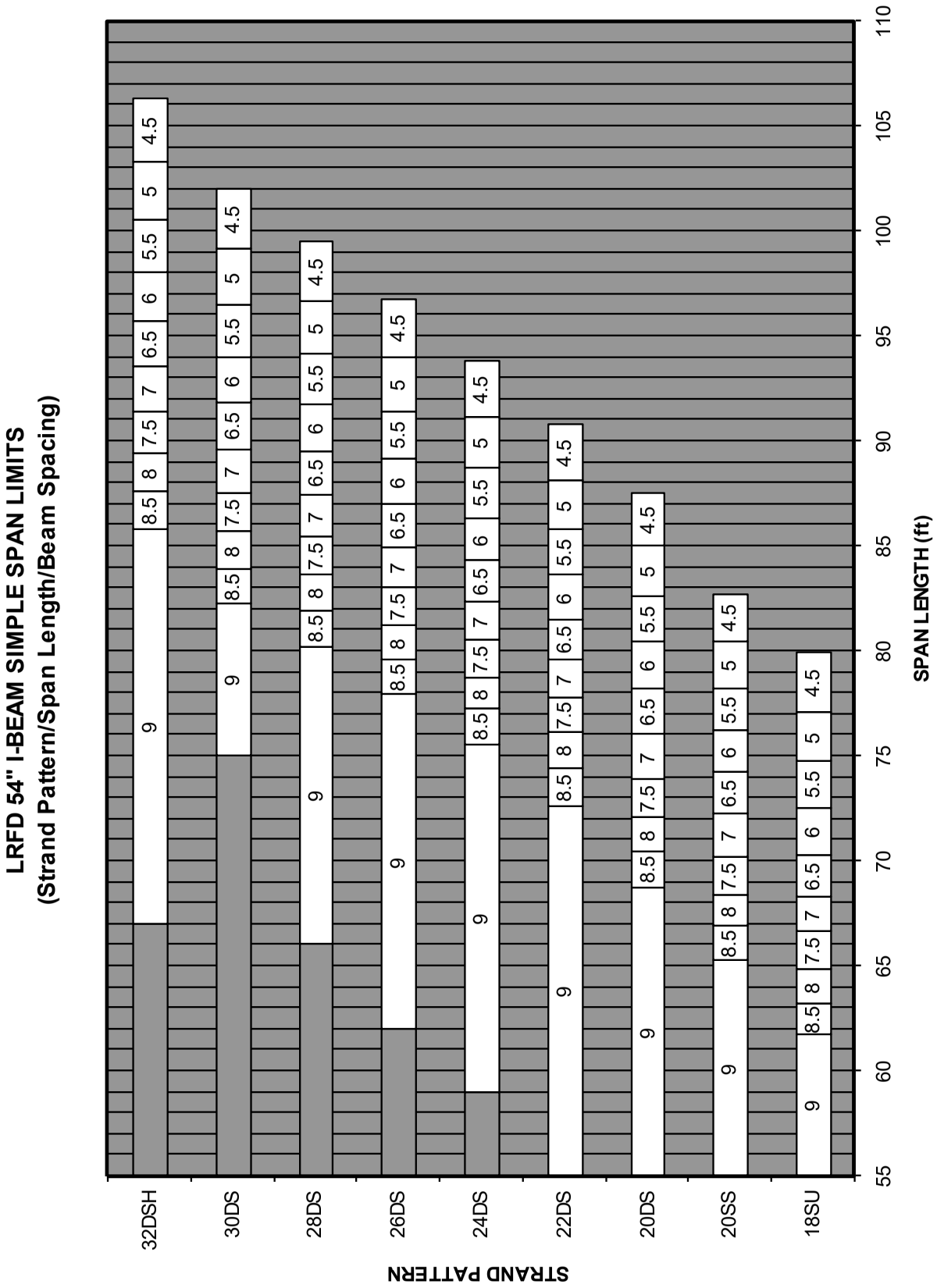


Figure 2.3.6.1.3-7

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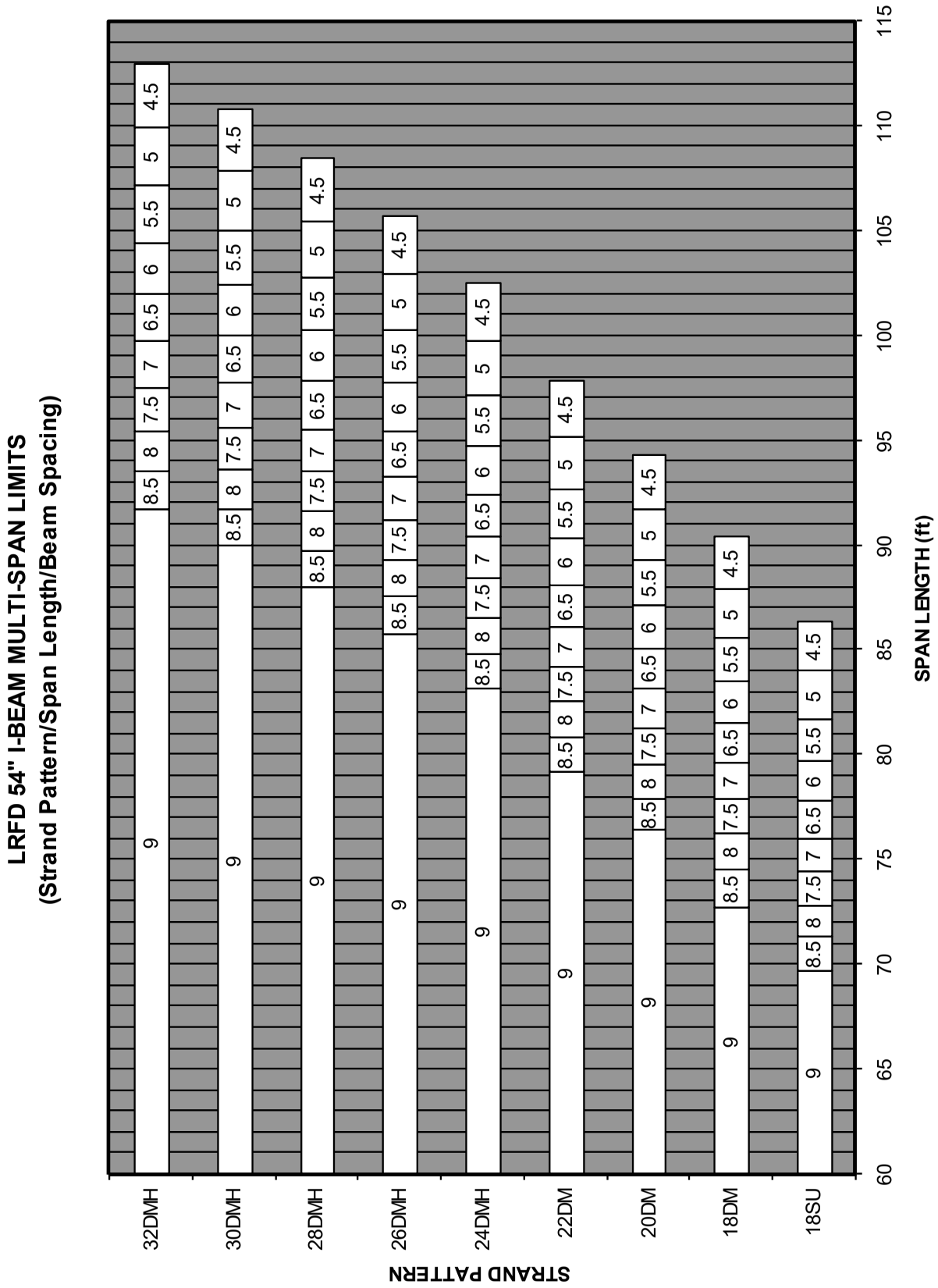


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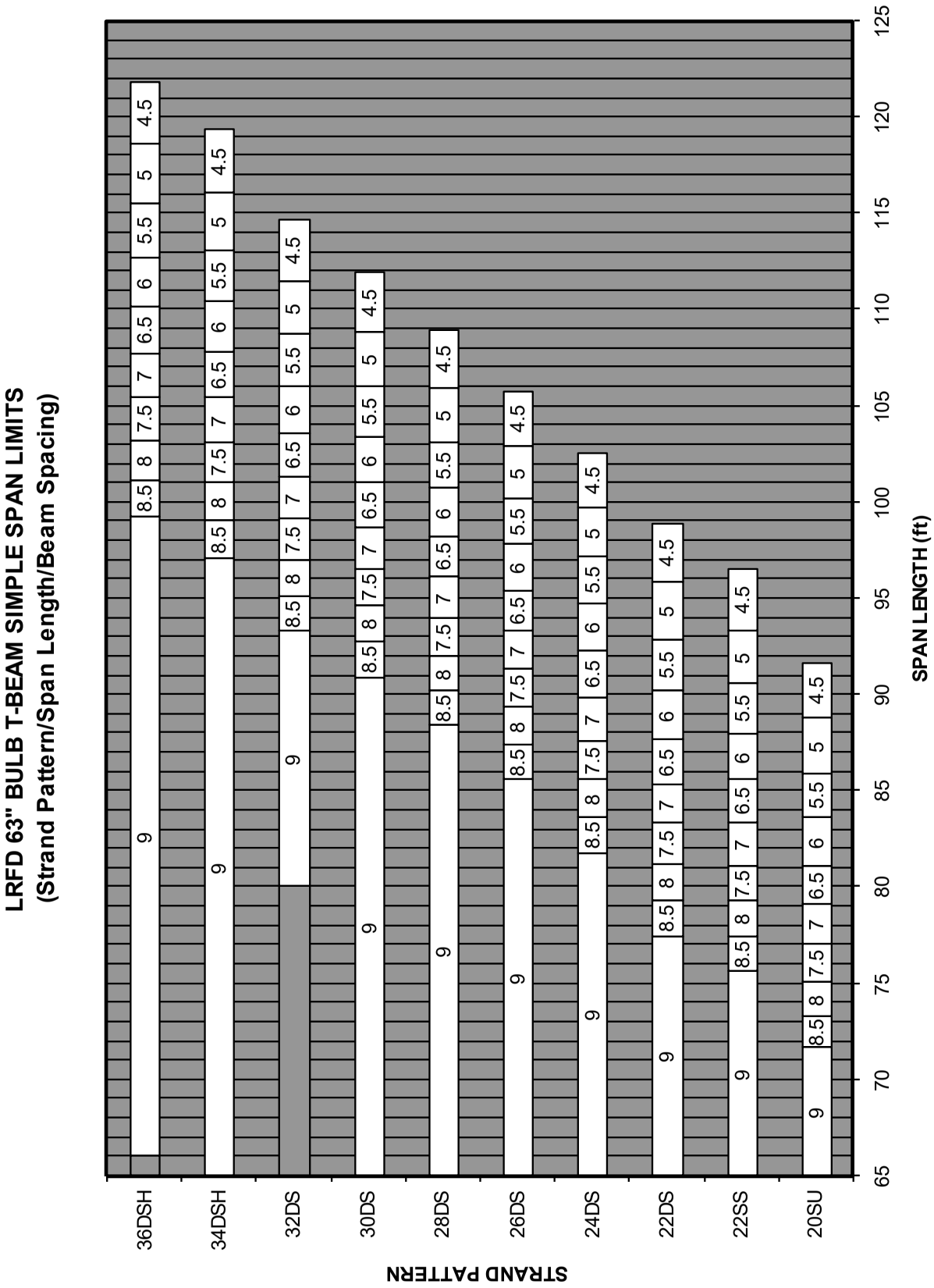


Figure 2.3.6.1.3-9

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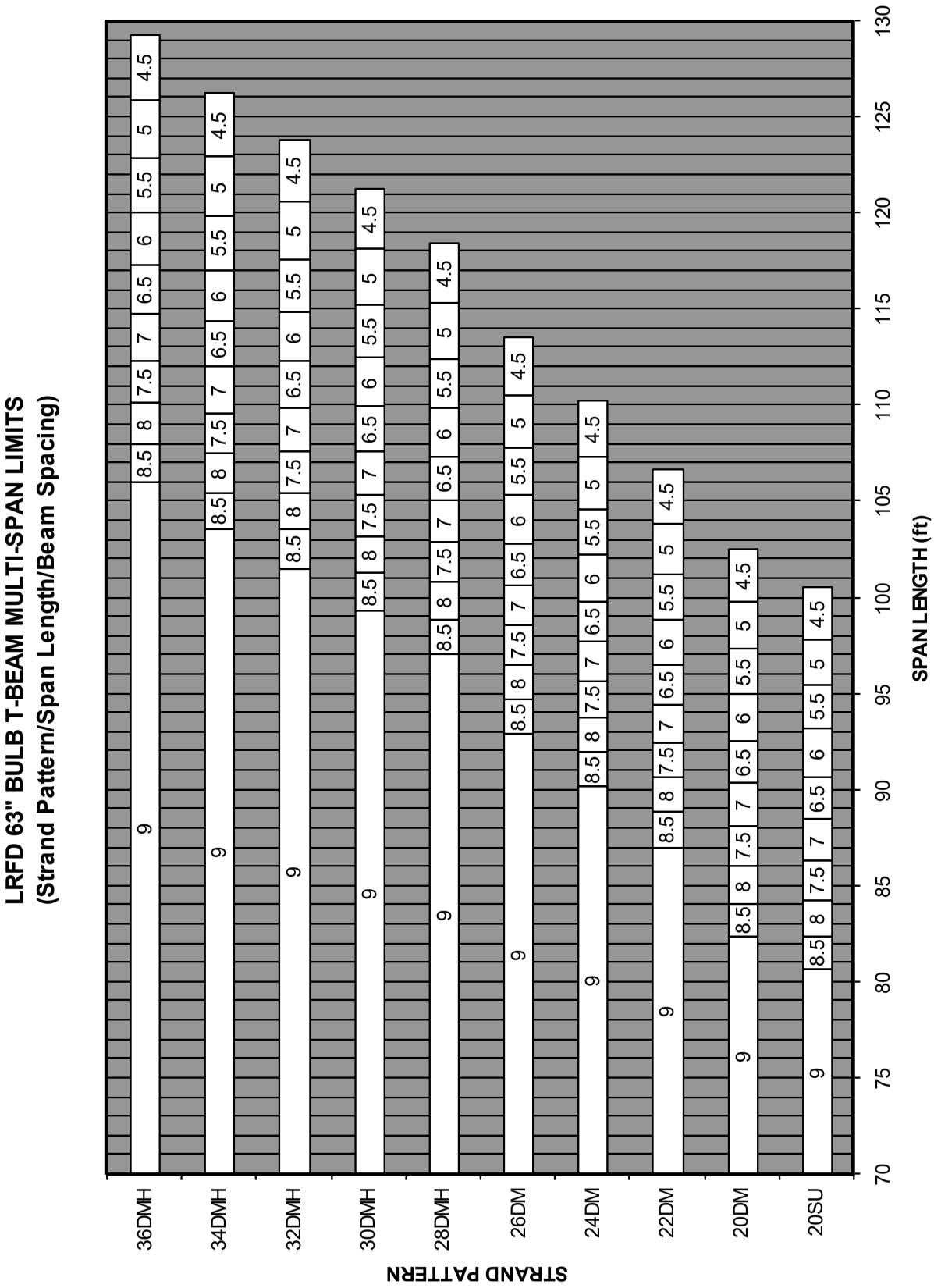


Figure 2.3.6.1.3-10

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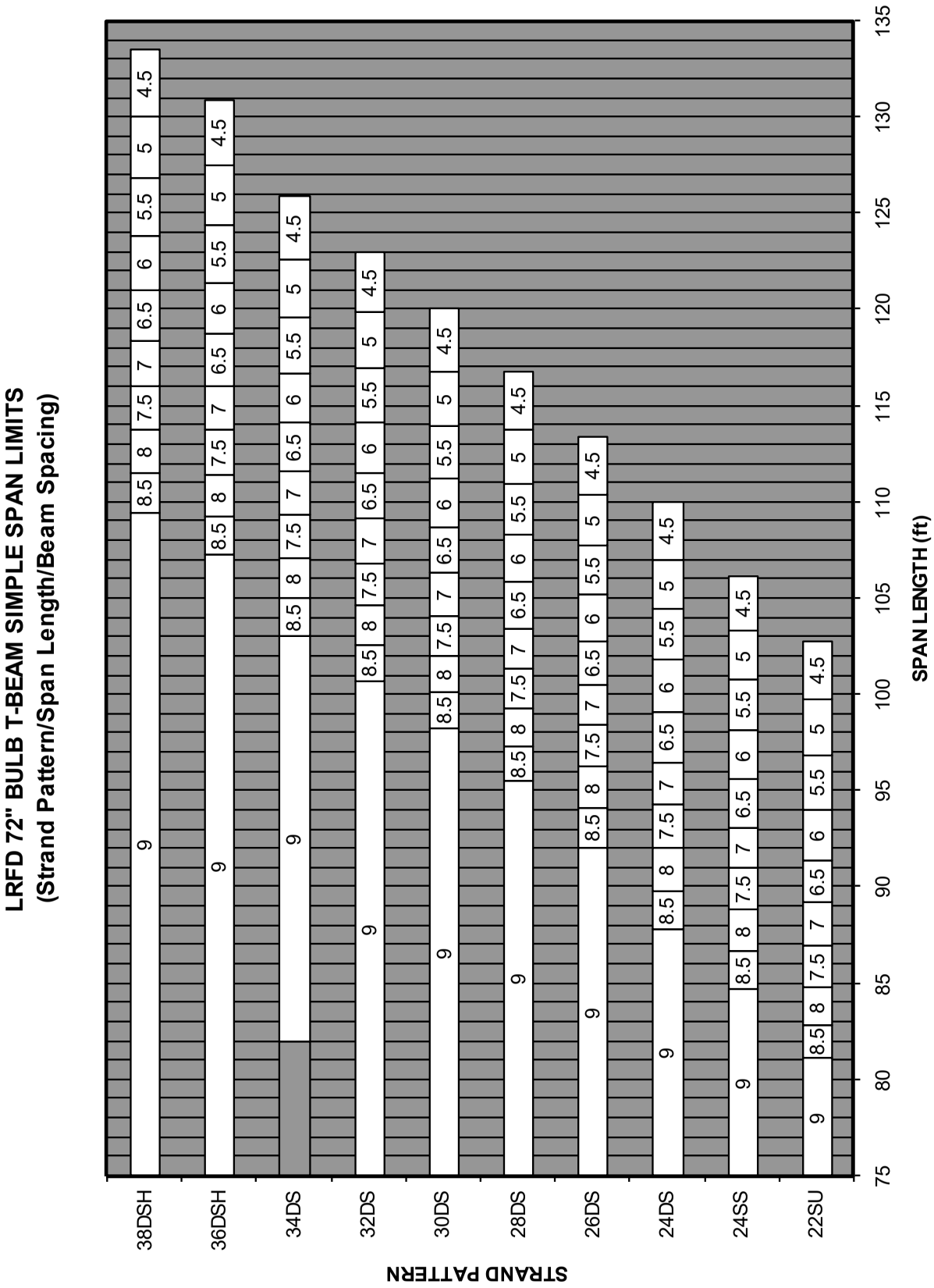


Figure 2.3.6.1.3-11

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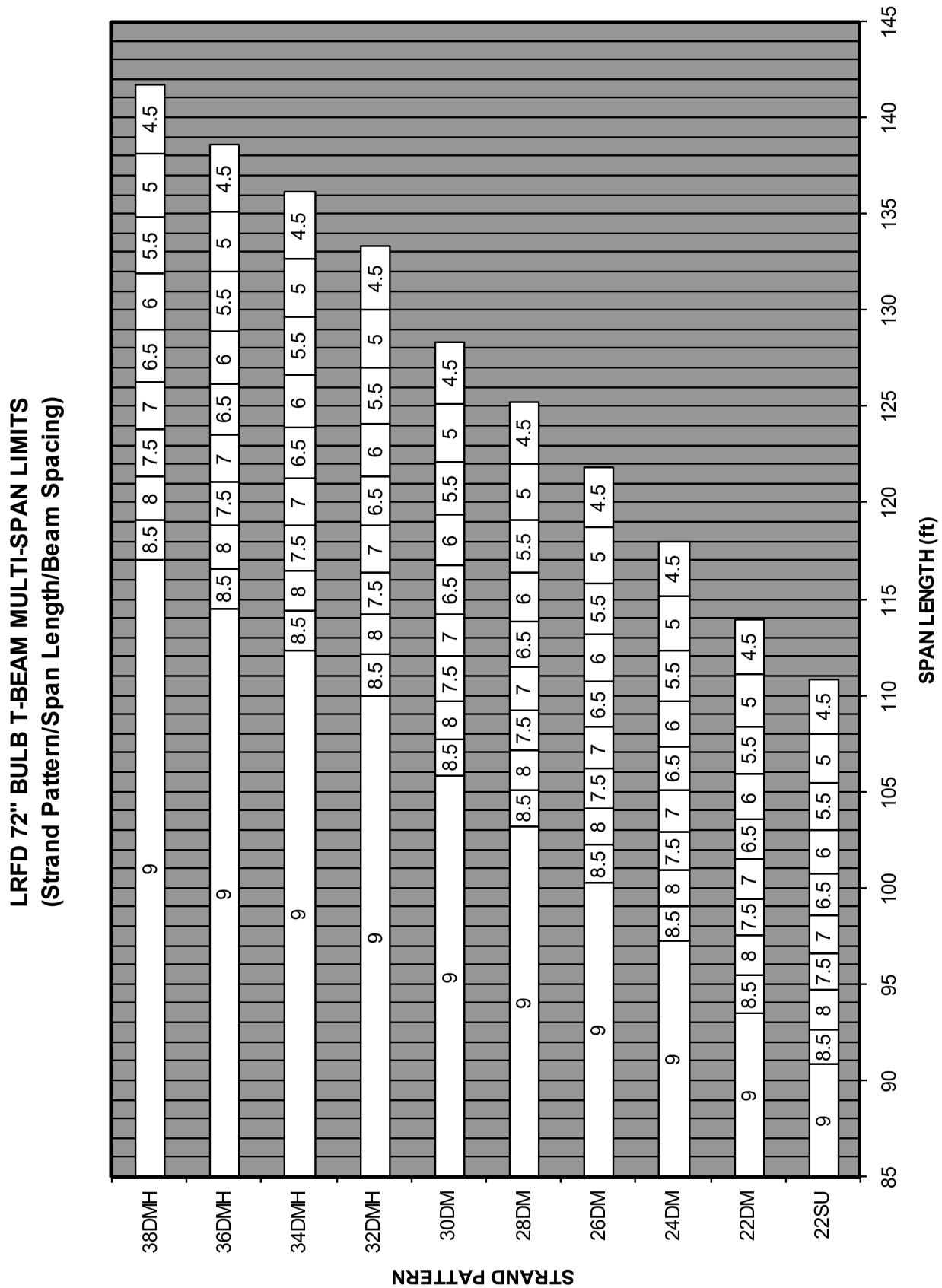


Figure 2.3.6.1.3-12

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2.3.6.1.3B Prestressed Concrete IL-Beams (PPC IL-Beams)

In 2015, the Department released new precast prestressed concrete (PPC) beam sections known as "IL-Beams" ranging in depth from 27 inches to 72 inches. These beam sections have successfully provided longer spans with shallower beam depths and reduced the number of beam lines by allowing wider beam spacings. In 2021, the Department elected to expand the standard suite of IL beams with 81 inch and 90 inch deep sections. These deeper beam sections (IL81-3838 and IL90-3838) provide additional superstructure options for bridge planners by utilizing the same IL-Beam concrete strength, design methodology, and beam shapes, with the addition of two more 9" incremental web increases. The exception is that the narrow 24-inch top flange option is not available on these deeper sections due to stability concerns.

Multi-span structures with spans near the maximum limits may experience deck reinforcement congestion in the negative moment region due to the required longitudinal deck reinforcement. In these rare instances, 80 ksi reinforcement may be used in the deck to reduce the congestion. Projects requiring 80 ksi reinforcement will require designers to calculate development lengths independent of those found in the Bridge Manual Appendix.

The integral abutment pile selection spreadsheet located on the web site has been updated to include these new beam sizes. Bridge planners considering an integral structure should continue to use this spreadsheet to evaluate pile options based on their proposed beam size, structure length, soil stiffness and structure skew. In some cases, in order to maintain a jointless structure, it may be necessary to improve the soil conditions by precoring holes for piles and backfilling with bentonite or utilizing a semi-integral abutment structure.

Figure 2.3.6.1.3-13 and Figure 2.3.6.1.3-14 are simple and multi-span planning selection charts showing a range of span lengths available based on beam size and beam spacing. The beam spacing ranges are indicated on the chart bars and range from 6 to 10 feet for these deeper IL-Beams. Beam spacing less than 6 feet is not included for the following reasons:

a. The IL-Beams are more cost effective when using larger beam spacings and fewer beam lines.
b. Tighter beam spacings are challenging to accommodate stage construction lines; do not provide as much space for negative moment reinforcement in the deck slab over piers which often requires grade 80 reinforcement; and they make beam inspection more difficult.
c. Standard overhangs are typically one-half the beam spacing and consequently tighter beam spacings make it very difficult to provide deck drainage.
d. The Department was unable to find any cases where the 5 feet and 5.5 feet beam spacings were utilized since the IL-beams were released.

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The multi-span charts are based on two equal spans. Additionally, the site location and delivery route shall be considered when specifying these beams because the fabricator shall be capable of satisfying the transportation requirements of Section 3.6.4 of the Manual for Fabrication of Precast Prestressed Concrete Products.

The maximum span lengths shown are typically controlled by the Service III tensile check in the bottom fiber of the beam at the location of maximum moment. However, maximum span lengths may also be controlled by either the maximum lift capacity of most fabricators equipment, which is 250,000 pounds, or negative camber considerations. The Department does not permit a negative camber resulting from dead load deflection. Camber generally increases with time; therefore, beams shipped and locked in place with a composite deck earlier than the assumed waiting period are more likely to have negative camber issues as well as larger fillets and dead load. These charts were developed using a minimum time between beam casting and deck placement of 45 days. The beam shipping policies may be found in the Manual for Fabrication of Precast Prestressed Concrete Products.

Figures 2.3.6.1.3-15 through 2.3.6.1.3-27 consist of preliminary simple span design charts for the IL-Beam sections. They indicate the maximum and minimum span for each strand pattern and beam spacing. These charts provide the designer with a starting point when selecting a strand pattern, but they are not to be used in lieu of computations for the final design. The simple span strand patterns should work for multi-span bridges. Slight design adjustments to address end stresses at interior supports, such as increasing the number of debonded strands or increasing the debonded lengths, may be necessary. Additional draping should be avoided but may also be utilized provided the total limit of 6 draped strands (for the IL27 through and IL72 beam sections) and 10 draped strands (for the IL81 and IL90 beam sections) is not exceeded.

These charts can be used to help choose an appropriate beam size for a given bridge. They also provide designers with a good starting point when selecting a strand pattern. They are not to be used in lieu of computations for the final design of a structure.

For Standard Strand Pattern tables and for descriptions on chart development, see Section 3.4.3.

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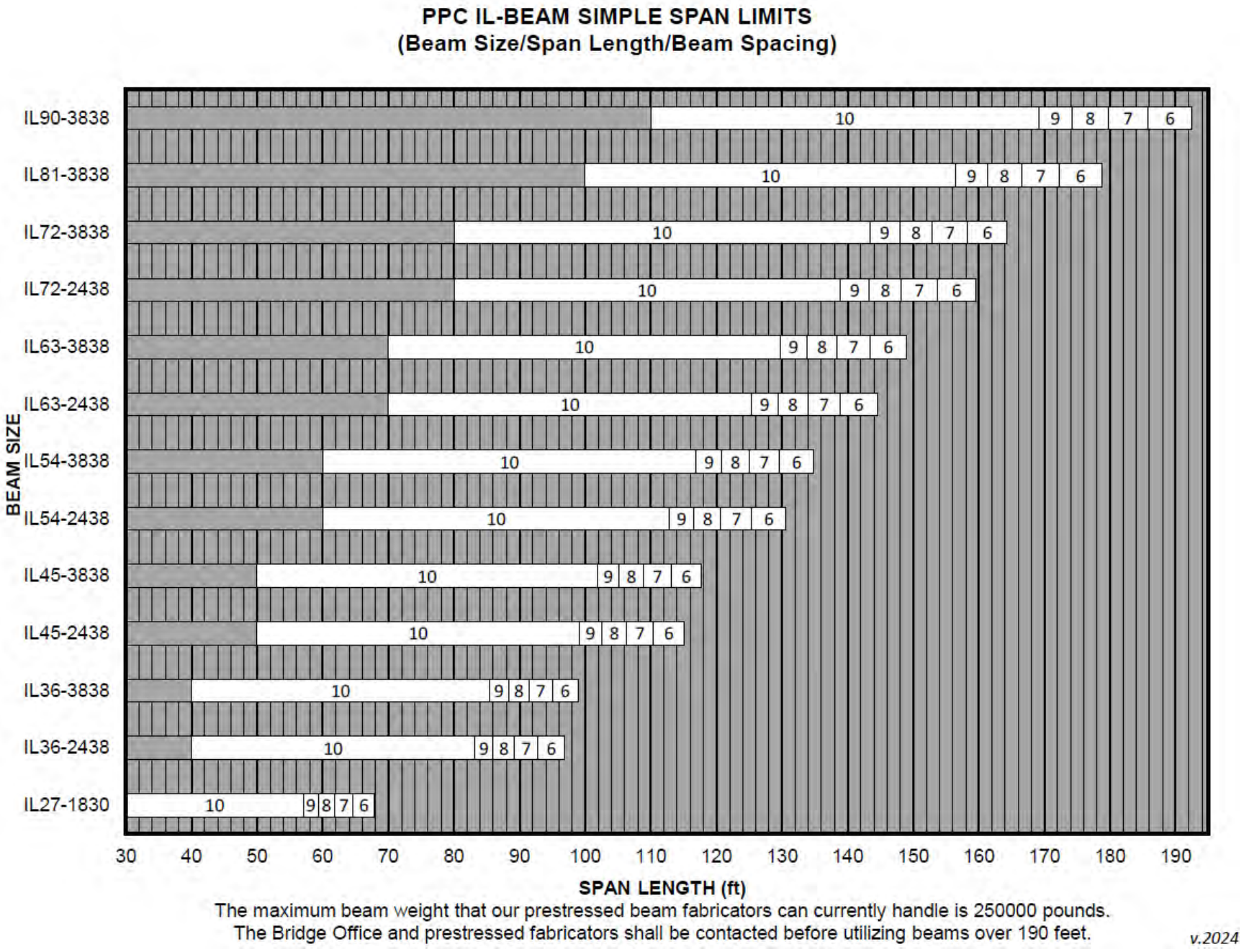


Figure 2.3.6.1.3-13

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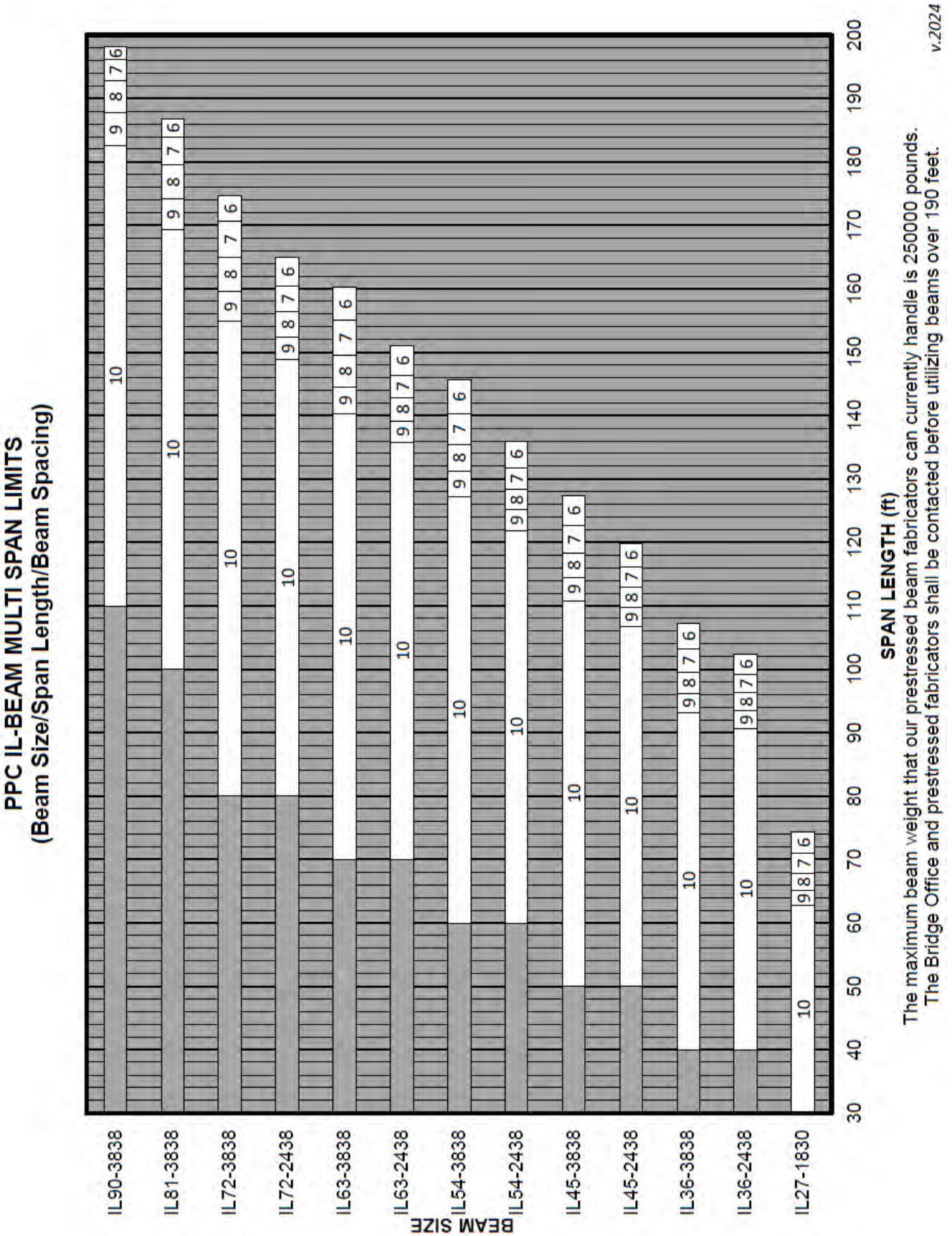


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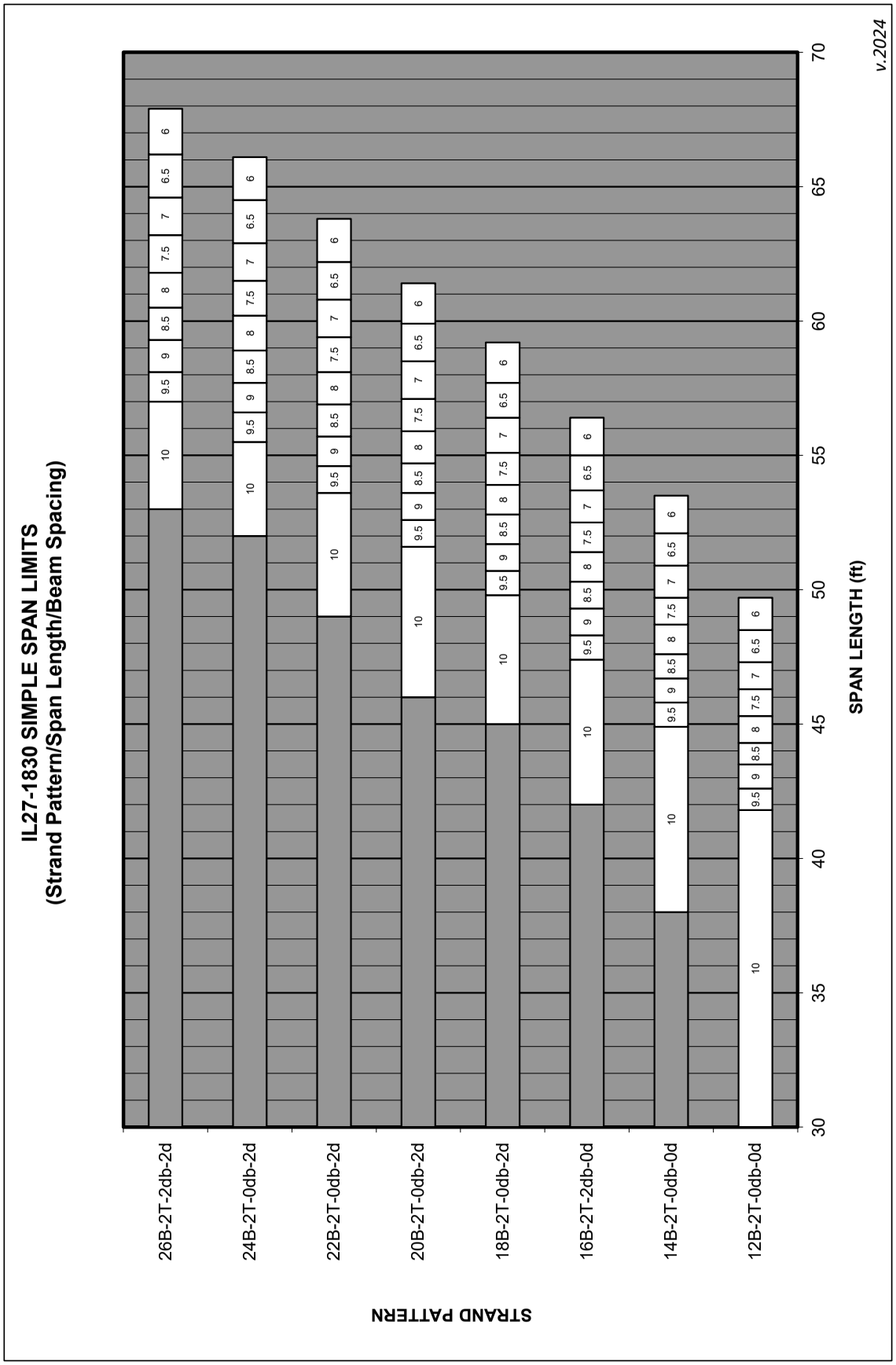


Figure 2.3.6.1.3-15

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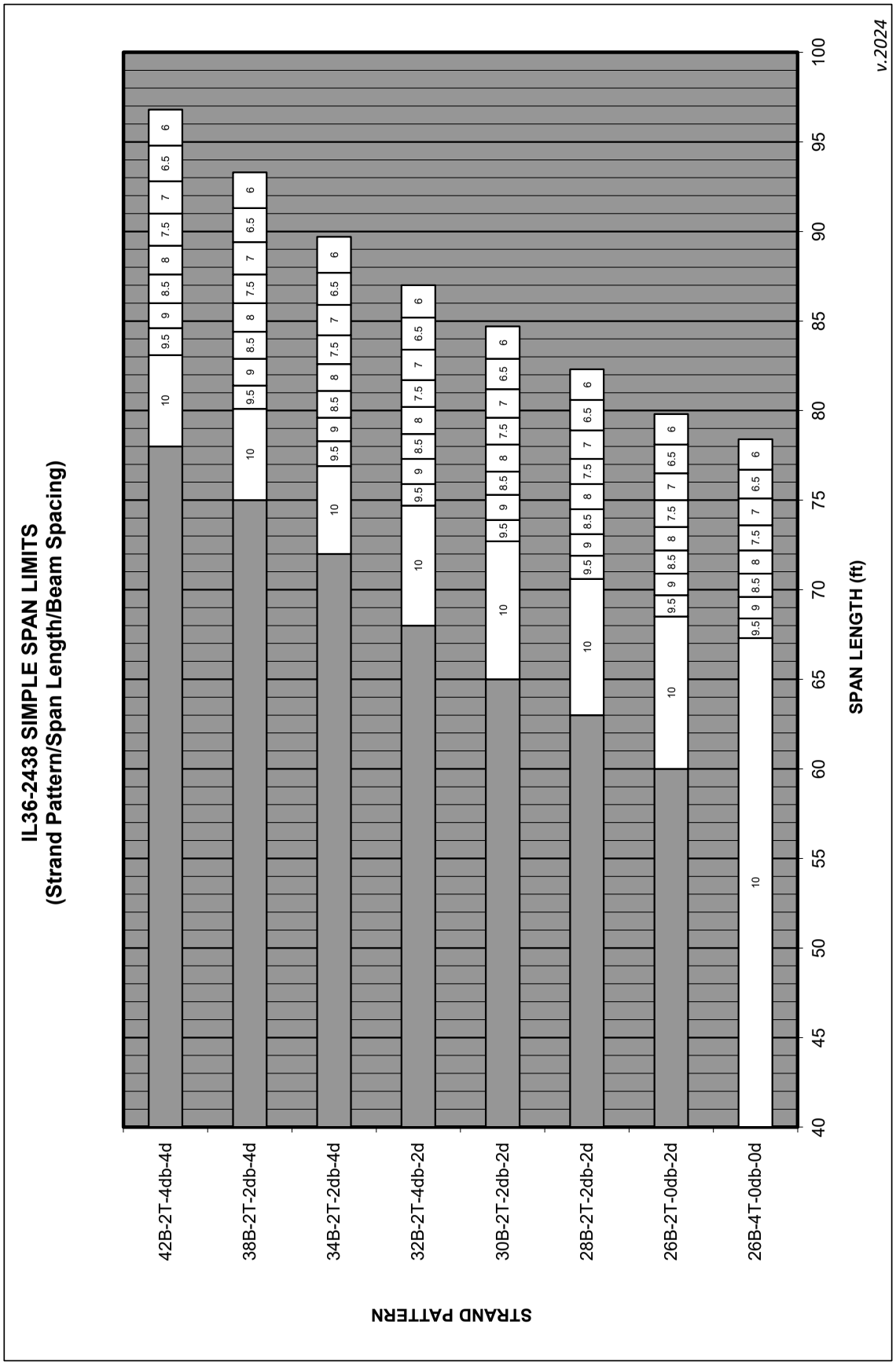


Figure 2.3.6.1.3-16

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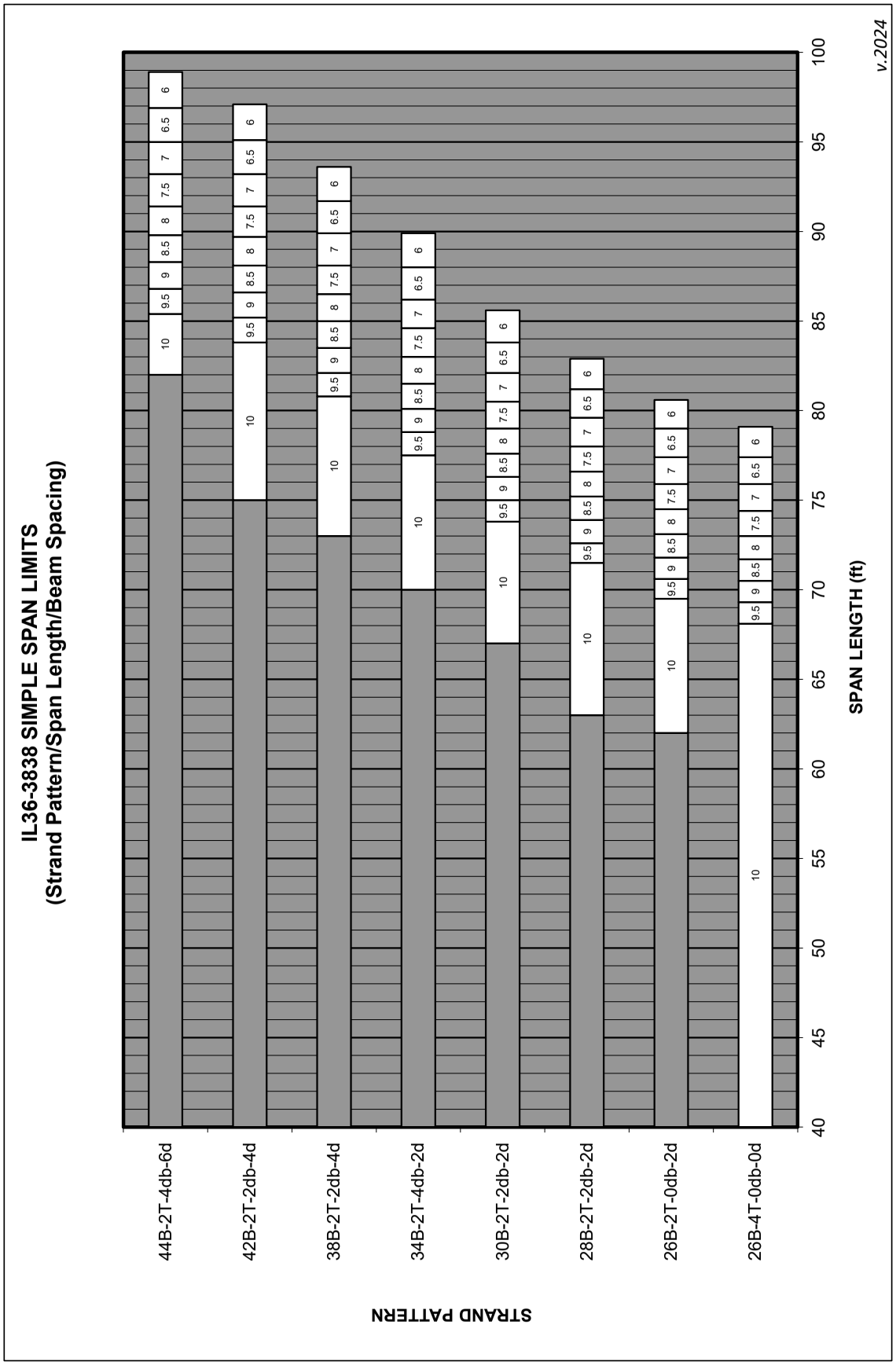


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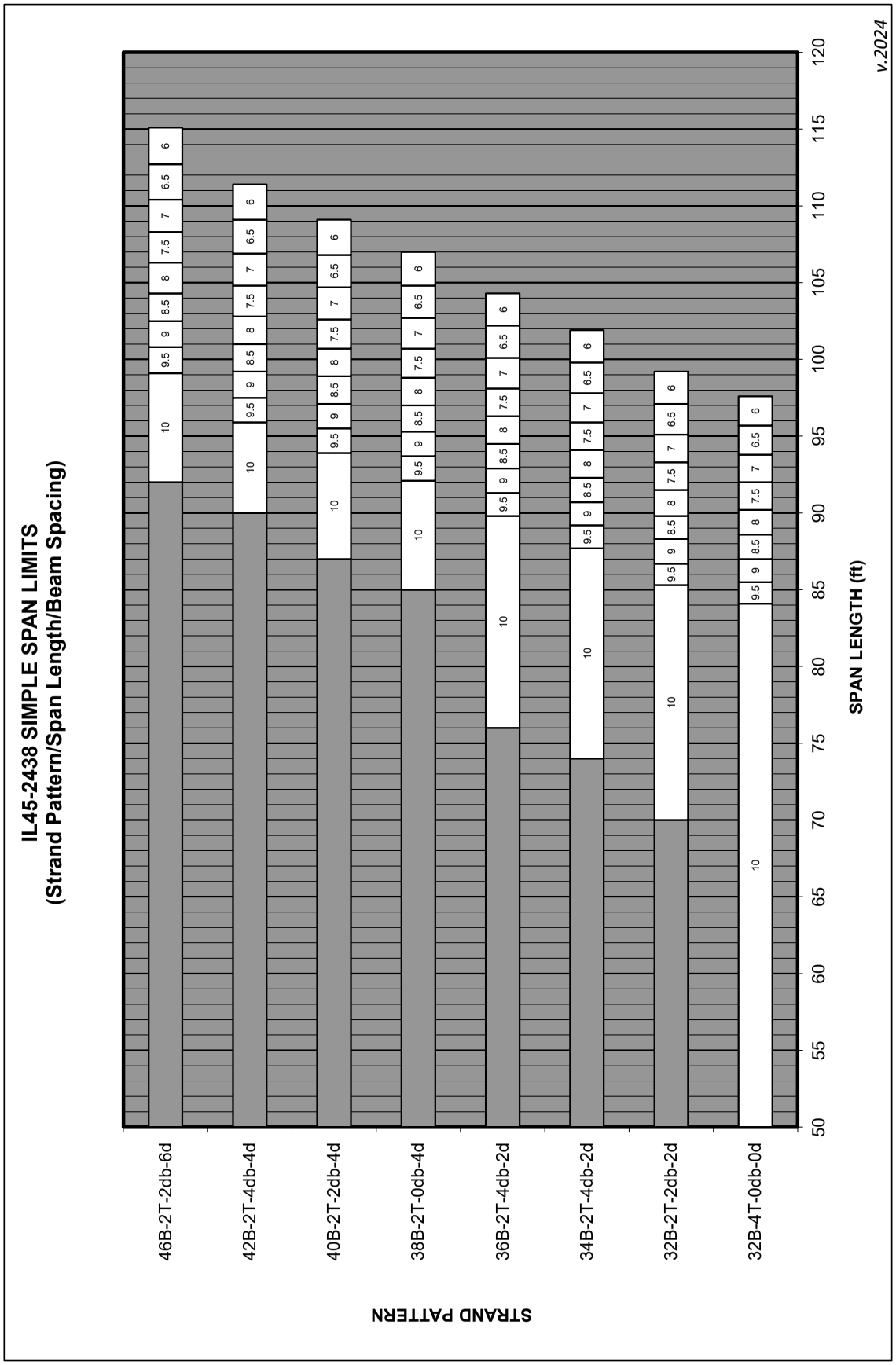


Figure 2.3.6.1.3-18

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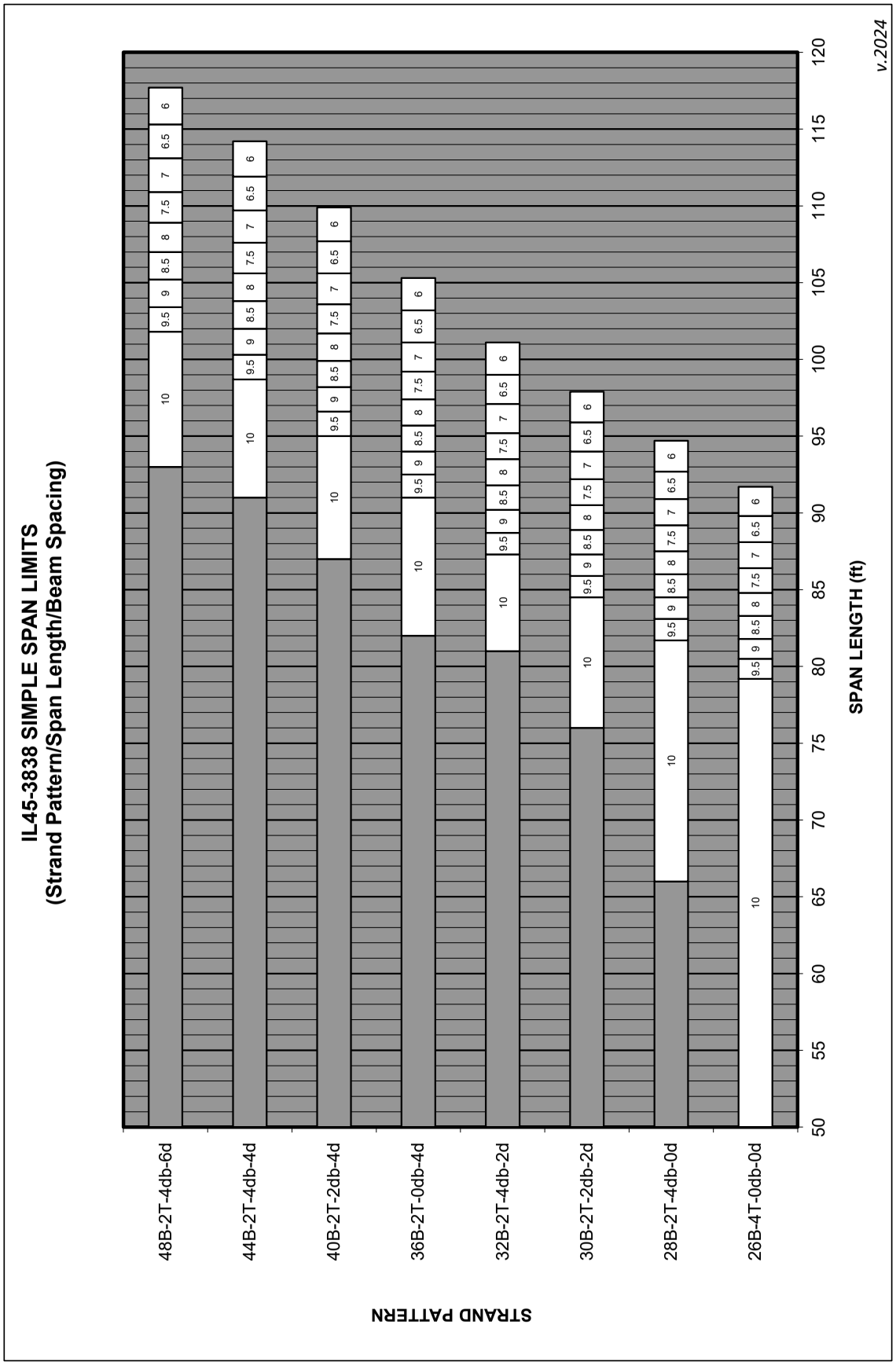


Figure 2.3.6.1.3-19

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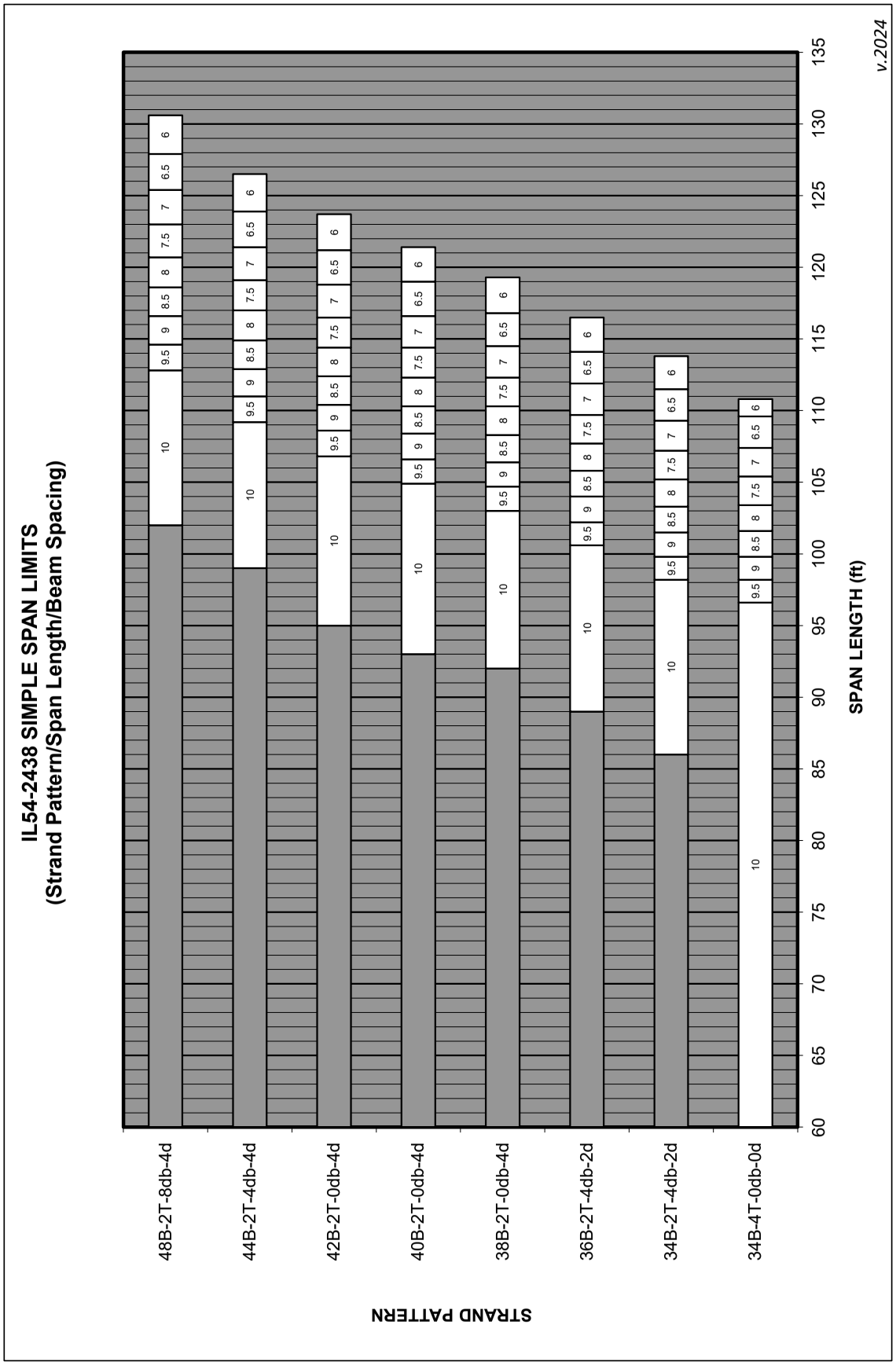


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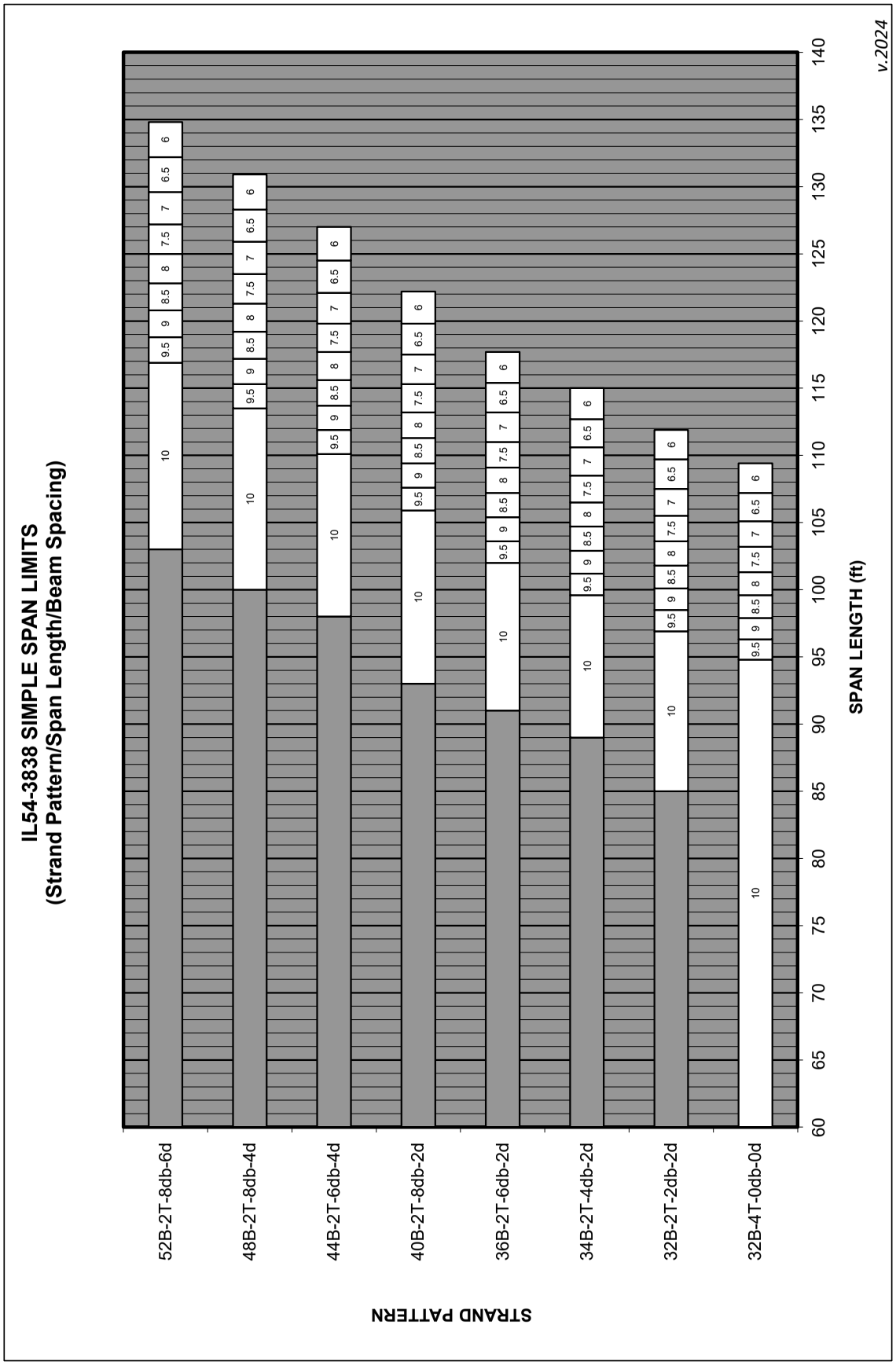


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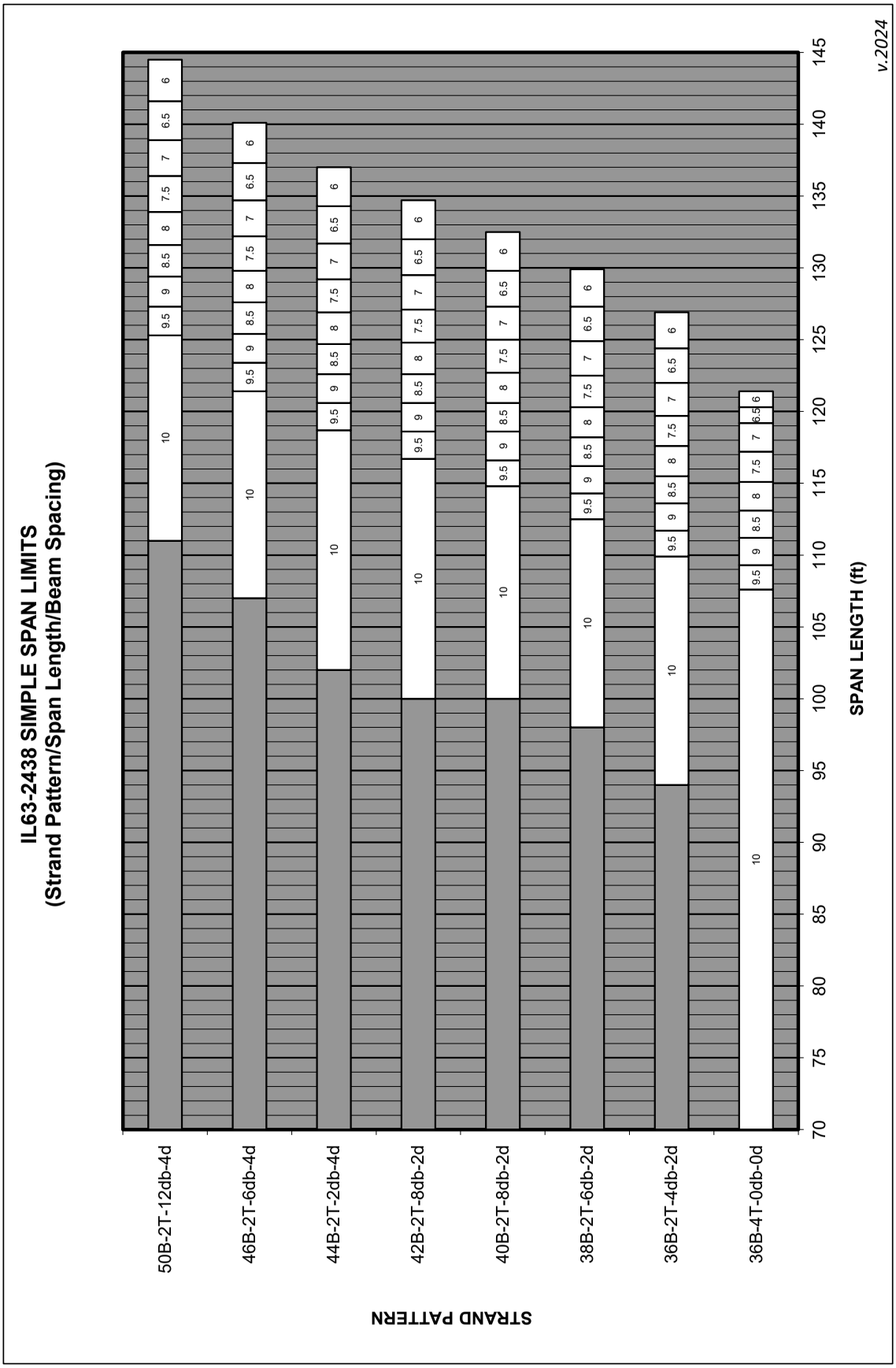


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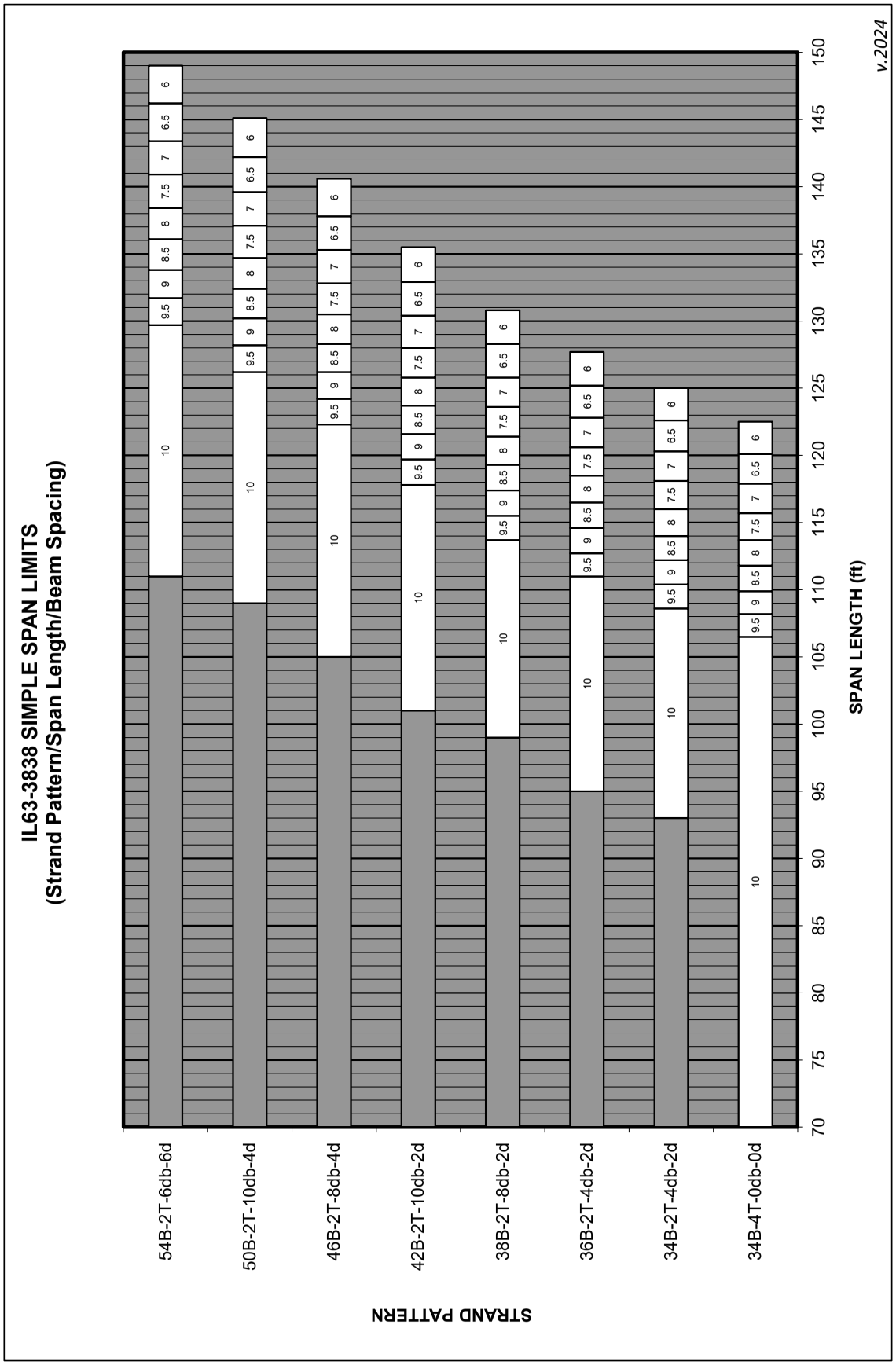


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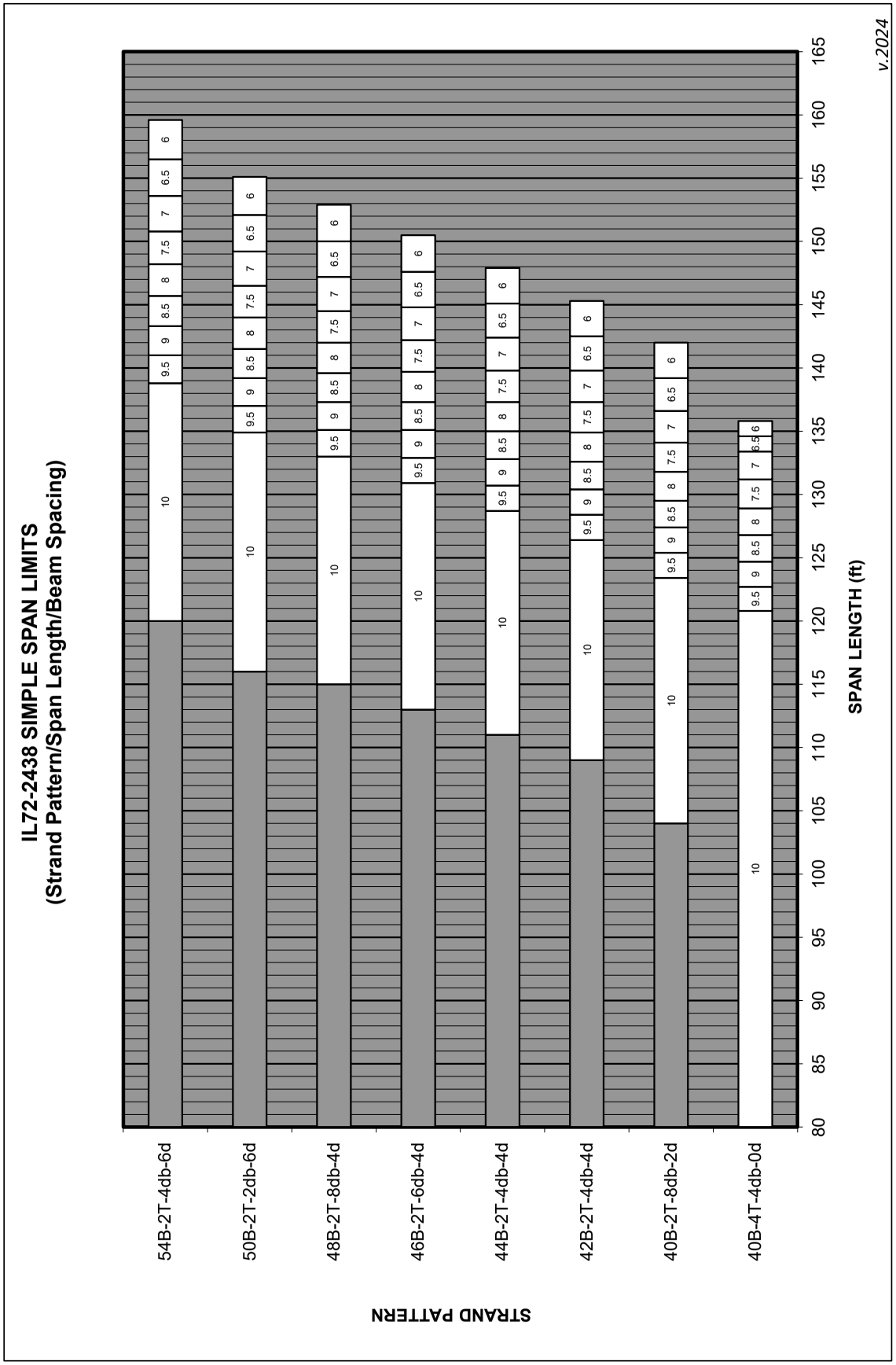


Figure 2.3.6.1.3-24

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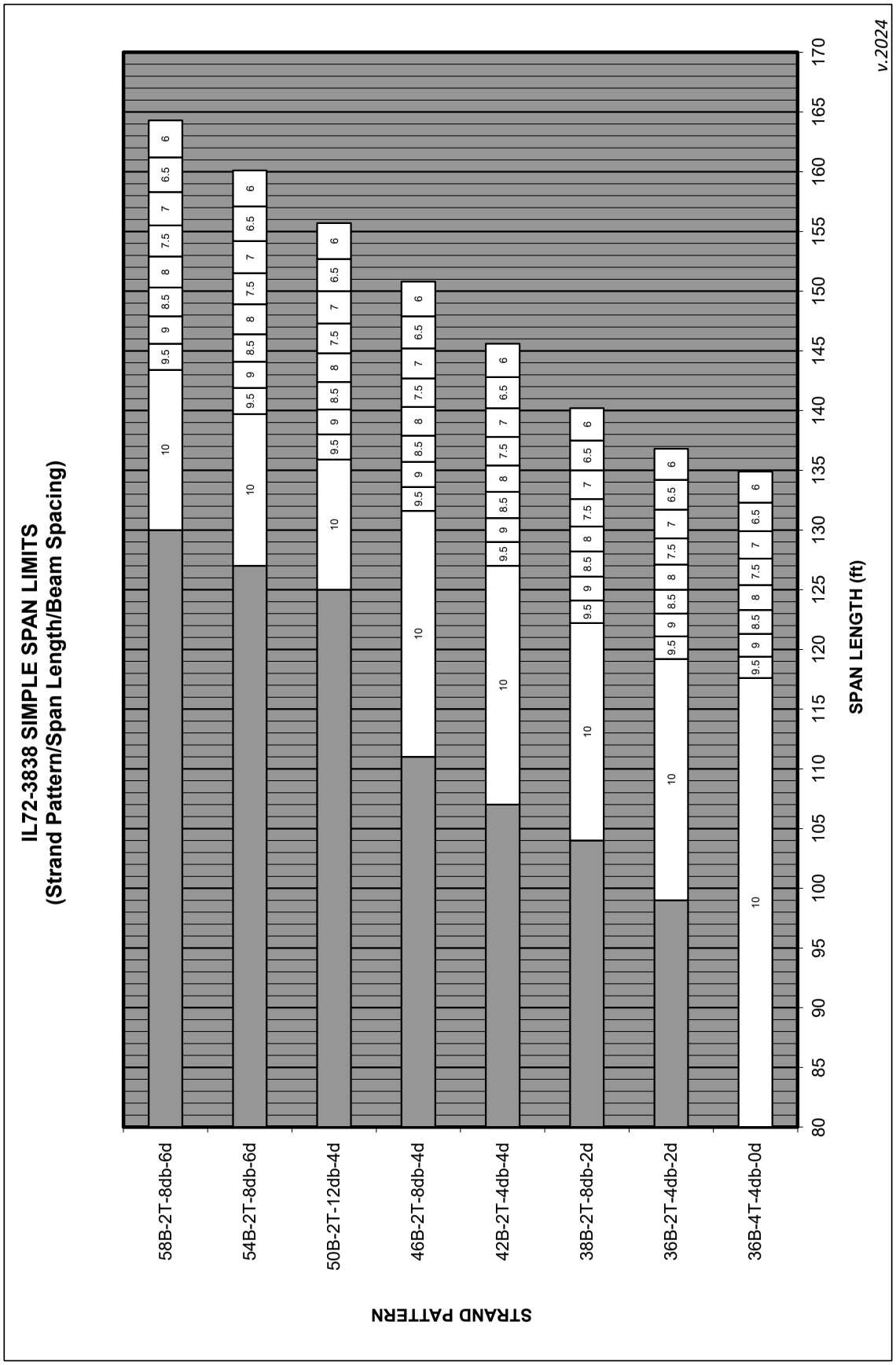


Figure 2.3.6.1.3-25

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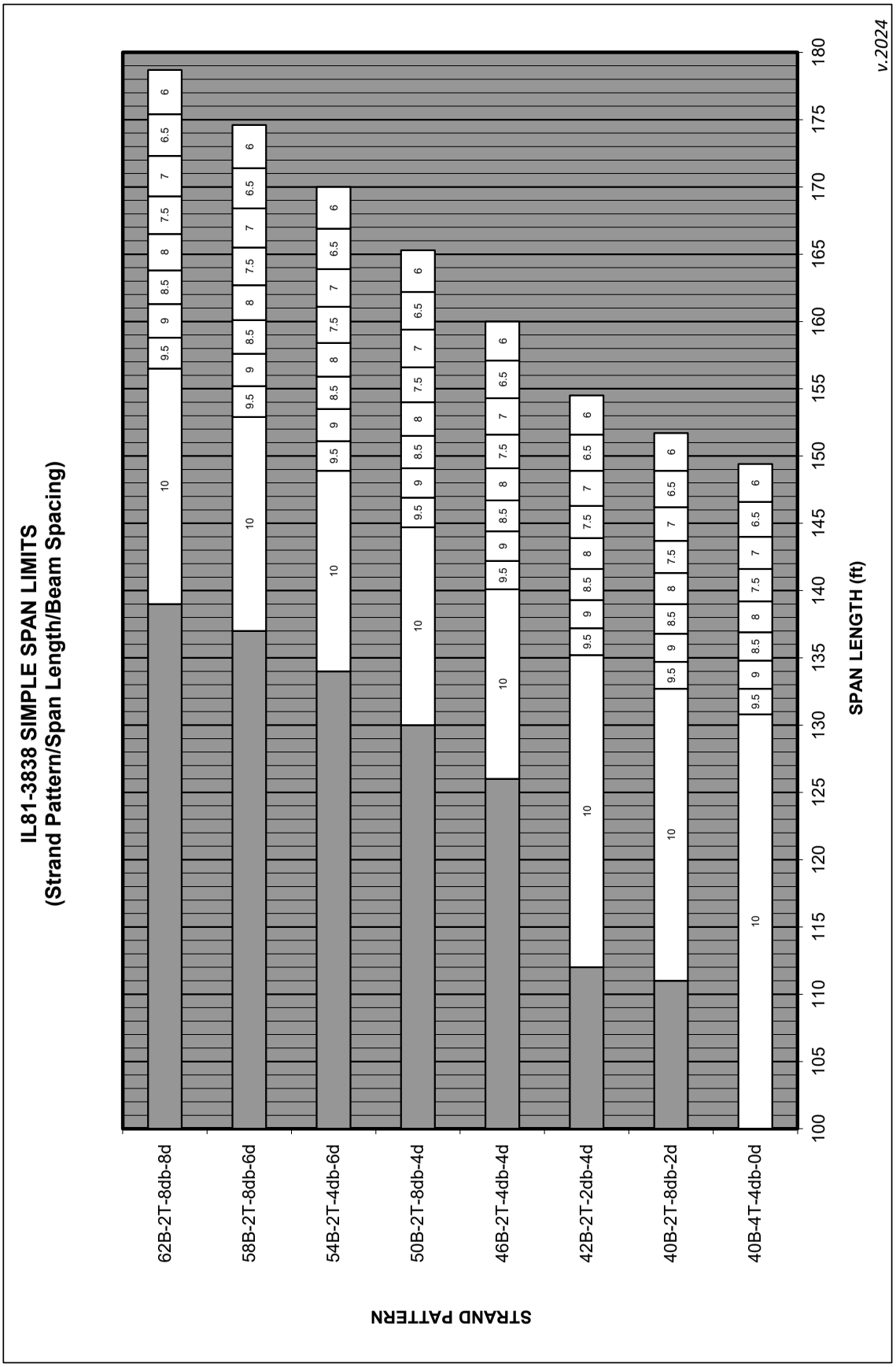


Figure 2.3.6.1.3-26

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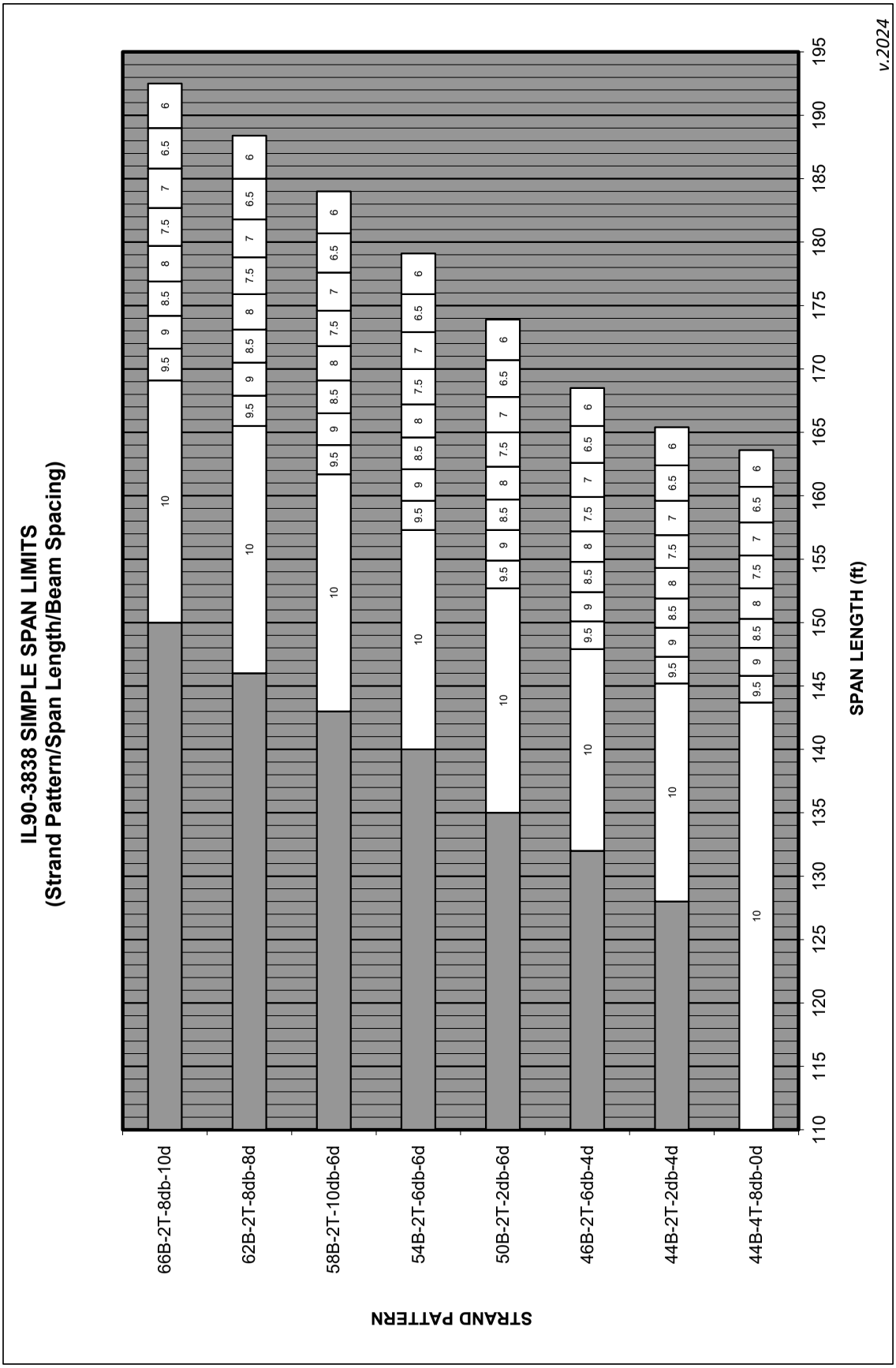


Figure 2.3.6.1.3-27

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2.3.6.1.4 Slab Bridges

Use of slab bridges will be limited to a maximum span length of 40 ft. The slab thickness shall be based on design requirements and not on the minimum slab thickness tables found in the LRFD specifications. See Section 3.2.11 for more information.

2.3.6.1.5 Bearing Type

The Department generally specifies three primary types of bearings. These are elastomeric expansion, fixed, and HLMR (pot or disc) bearings. Seismic isolation bearings have also been employed by the Department in selected cases. When HLMR, or seismic isolation bearings are under consideration, the Bureau of Bridges & Structures should be consulted to verify their acceptability.

Section 3.7 contains technical details, policies and procedures for the design of bridge bearings. Most of the typical bearing details for PPC I-beam, IL-Beams, Bulb T, and PPC deck beam bridges are depicted on the prestressed base sheets. Sections 3.7 and 3.15 contain seismic provisions, details, and policies for bearing design.

The Department typically uses standardized elastomeric bearings in conjunction with standard fixed bearings for non-integral abutment bridges when the expansion length is less than 500 ft. See Section 3.7 for guidance and for illustrations of the standard fixed steel bearing used in conjunction with elastomeric bearings. Longer expansion lengths usually call for the use of HLMR bearings. Structures designed for curvature shall have HLMR bearings at all locations.

Standard fixed bearings and elastomeric bearings are also used on bridges in Illinois which have integral abutments. There also is a standard fixed bearing detail used at integral abutments with steel beams which is presented in Section 3.7. Generally, if a pier is rigid and the expansion length is long, elastomeric bearings may be considered at the piers of integral abutment bridges. As a minimum for multi-span bridges, one pier should be fixed for construction stability and longitudinal forces; additionally, one rigid pier should be considered on long integral bridges. Otherwise, standard fixed bearings are usually specified.

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2.3.6.1.6 Bridge Deck Expansion Joints

All expansion joints in decks shall prevent deck drainage from penetrating the bridge deck expansion joint openings. The preferred types of expansion joints are strip seals, and fingerplates with troughs. Modular joints may be used in lieu of fingerplates or when the limits for the use of fingerplates are exceeded. Preformed joint seals and neoprene joints have been phased out. The use of the preformed joint seals and neoprene joints should be limited to short-term repairs; refer to the Structural Services Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Repairs) for additional guidance.

Section 3.6 contains technical details, policies and procedures for the design of expansion joints. A guide for selection of expansion joint type is presented in Figure 2.3.6.1.6-1. The joint type in the guide is primarily a function of contributing expansion length and skew. Contributing expansion length at a pier shall be defined as the distance between fixed bearings measured along the bridge. At an abutment (non-integral), the expansion length shall be the distance from the abutment joint to the nearest fixed pier bearing.

Expansion Joint Limitations

The use and limitations of the various expansion joint devices used in Illinois are as shown in Figure 2.3.6.1.6-1. Strip seals shall be used for bridges with contributing expansion lengths less than or equal to 305 ft. with skews between 0 and 60 degrees with maximum contributing expansion lengths reduced as shown in the figure. Strip seals can accommodate small amounts of curvature as well, but calculations should be made to ensure the strip seal can accommodate differential expansions due to skew and curvature before using them on curved structures. Beyond the limits for strip seals, fingerplates with troughs or modular joints shall be used. A hybrid (swivel) modular joint system designed to accommodate differential non-parallel longitudinal movements shall be used for bridges which are subjected to large lateral loads, are designed for the effects of curvature, and/or have skews greater than 60 degrees.

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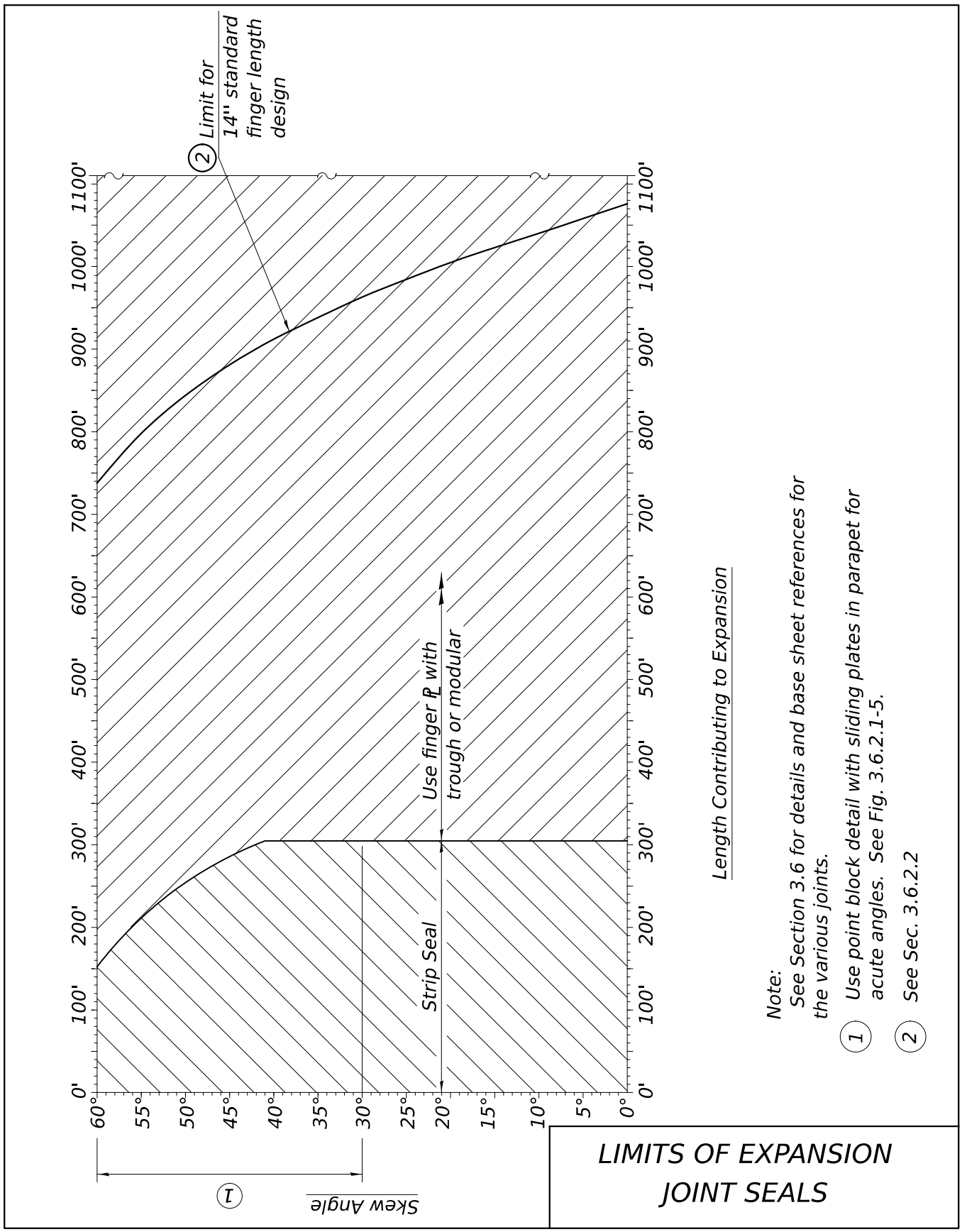


Figure 2.3.6.1.6-1

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2.3.6.1.7 Bridge Railing

All new bridge railing configurations shall be shown to be structurally and geometrically crashworthy according to the appropriate Test Level of Section 13 of the AASHTO LRFD Bridge Design Specifications and shall demonstrate acceptable performance through a full scale crash test. The owner is responsible for determining the Test Level necessary for each application. Railings on all new or rehabilitated bridges on Federal and State routes shall satisfy the crash standards of the 2016 Manual for Assessing Safety Hardware (MASH) with a minimum crashworthy Test Level of TL-4. Railing Criteria, such as the MASH Test Level, maximum post spacing and other design requirements for each railing are located on the Base Sheets. Some railing systems are designated with the Test Level for the previous NCHRP 350 safety report. These railings may continue to be used until their MASH evaluation and revisions are completed. The Bureau of Local Roads & Streets Manual contains policy regarding the use of railings on the local system. Lower test level railings applicable for only the local system are available in the "Local Only" Base Sheets at the primary IDOT CADD page as they become available.

Railing Base Sheets shall not be altered in order to maintain their crash worthiness; however, minor changes may be approved by the Bureau of Bridges & Structures for certain details and cases.

When possible, all bridge railings and parapets shall be extended 15 feet onto the approach pavement. Special exceptions may be necessary due to the close proximity of an entrance or unique geometric requirements such as a curved roadway on a straight bridge. See the "Traffic Barriers" discussion in the "Bridge Geometry and Layout" Section 2.3 herein and Section 3.2.

The preferred MASH TL-4 railing system is the 39" Constant Slope parapet detailed in Figure 3.2.4-1. This parapet is structurally and geometrically crashworthy according to Section 13 of the AASHTO LRFD Bridge Design Specifications and has been crash tested for a MASH Test Level 4 (TL-4) through a full-scale crash test.

A 44 in. Constant-Slope parapet is detailed in Figure 3.2.4-4. This railing is crashworthy for test level TL-5 and should be used when at least one of the following scenarios occurs:

1. Structures with a future DHV (one way) × % trucks greater than 250.
2. Structures located in areas with high incidences of truck rollover accidents.
3. Structures with a radius of 1000 ft. or less with truck traffic.

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The Constant-Slope 39-inch-tall and 44-inch-tall parapets are suitable for both wet cast and slip form construction as permitted in Section 3.2.4.2.

Traffic Barrier Terminal (TBT), Type 6 (Highway Standard 631031) is required at all entrance ends for constant slope parapets, and where applicable at the exit ends. For some cases, the Traffic Barrier Terminal (TBT), Type 5 (Highway Standard 631026) may be warranted at exit ends.

The Bureau of Design & Environment Manual (available at the primary Highways Page) gives guidelines as to when glare screens shall be added to parapets, and the Department's Highway Standards Manual (also at the primary Highways Page) provides details for both metal and concrete glare screens. The addition of either glare screen to a parapet shall not be considered as improving or reducing the designated crash worthiness of a parapet.

Steel railings may be requested by the District or the owner; however, steel railings shall be approved by the Bureau of Bridges & Structures (BBS) for structures on Federal or State routes. Railing posts shall be spaced at equal or nearly equal spaces when possible and shall miss all parapet and deck joints. See Tables 2.3.6.1.7-1 through 2.3.6.1.7-5 for a summary of the steel railing library.

The preferred steel railing is the Type IL-OH side mounted steel bridge railing which is depicted on the R-40 Base Sheet series with five different applications: R-40BD (Bridge Deck); R-40CWS (Deck Beam with initial Concrete Wearing Surface); R-40HMAWS (Deck Beam with initial Hot Mix Asphalt Wearing Surface); R-40 HMAWS-AS (Deck Beam with initial Hot Mix Asphalt Wearing Surface and Approach Slab); and R-40NWS (Deck Beam with no wearing surface). This railing series has been successfully crash tested for MASH 2016 Test Level 4 (TL-4) through a full-scale crash test. The failure mechanism from impact is yielding in the post above the anchorage connection. This design feature prevents damage to the post anchorage and the bridge deck or deck beam superstructure, allowing for quicker and less expensive repairs after impact. These railings are side mounted with a spacer tube such that the railing is flush with the edge of the bridge deck to maximize bridge width. The railing is also curbless, which mitigates potential ponding problems. This railing should not be considered on grade separations over other roadways or railroads. See Section 2.3.6.1.8 for more information on drainage. Traffic Barrier Terminal Type 13 (Highway Standard 631061) is required at all ends of the Type IL-OH railing. The post spacing from the last bridge post to the first driven Traffic Barrier Terminal post = 9'-0". The post spacing from the last bridge post to the first post on the approach slab is 8'-0". The first and last IL-OH railing post mounted on the bridge deck shall always be located at 3'-0" from the end of the bridge deck or deck beam as illustrated on the base sheets.

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Another MASH 2016 crash tested TL-4 steel railing is the Type CO-10 railing which is a top-mounted curb railing. This railing is detailed and applicable for new and existing conditions and is suitable for grade separations over other highways with regards to drainage. Additionally, this railing may be used for culverts or on bridge decks as approved by the Bureau of Bridges & Structures. On new structures, the minimum concrete compressive strength, f'c, of the concrete deck, approach slab, or precast concrete beam that the Type CO-10 railing is connected to, shall be 4,500 psi. Railing replacement projects on existing structures utilizing the Type CO-10 railing may waive this concrete compressive strength requirement because the drilled and grouted anchor rods for the new railing shall be extended into the existing concrete to a level below that of the top mat of the existing reinforcement. Traffic Barrier Terminal (TBT), Type 14 (Highway Standard 631066) is required at all ends of the Type CO-10 railing.

The Type SMX, R-41 Base Sheet series, is a Conditional MASH 2016 TL-3 side mounted steel railing. It passes all of the computational and geometrical requirements but is awaiting the physical crash test to be completed. Therefore, it is a Conditional MASH TL-3 railing until the completion of this test. A passing test is anticipated so it may be used on all IDOT projects where a MASH TL-3 is applicable. Base sheet versions of this railing include: R-41CWS (Deck Beam with Initial Concrete Wearing Surface); R-41CWSC (Deck Beam with Initial Concrete Wearing Surface and Curb); and R-41HMAWS (Deck Beam or Bridge Deck with Initial Hot Mix Asphalt Wearing Surface). The minimum concrete compressive strength, f'c, of the concrete deck, approach slab, or precast concrete beam connecting to the Type SMX railing, shall be 5,000 psi. Traffic Barrier Terminal (TBT), Type 6A (Highway Standard 631032) is required at all ends of the Type SMX railing. See Section 3.2 for proper bridge deck geometry and reinforcement to accommodate the Type SMX side mounted connection. Details are also provided for post spacing and anchor geometry requirements affected by skew. An approach slab is required for skews greater than 15 degrees for the Type SMX railing.

The Type SM, R-34 Base Sheet series, is a MASH 2016 TL-2 side mounted steel railing. Base sheet versions of this railing include: R-34CWS (Deck Beam with Initial Concrete Wearing Surface); R-34CWSC (Deck Beam with Initial Concrete Wearing Surface and Curb); and R-34HMAWS (Deck Beam or Bridge Deck with Initial Hot Mix Asphalt Wearing Surface). The minimum concrete compressive strength, f'c, of the concrete deck, approach slab, or precast concrete beam connecting to the Type SM railing, shall be 5,000 psi. Traffic Barrier Terminal (TBT), Type 6A (Highway Standard 631032) is required at all ends of the Type SM railing. See Section 3.2 for proper bridge deck geometry and reinforcement to accommodate the Type SM side mounted connection. Details are also provided for post spacing and anchor geometry requirements affected by skew. An approach slab is required for skews greater than 24 degrees for the Type SM railing.

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Railing options for traffic structures with sidewalks and a posted speed limit greater than 45 mph are depicted in Base Sheets R-29 and R-33. These railings should be utilized on all bridges where provisions are made for the specific operation of bicycles. The Parapet Railing mounted on the concrete parapet on these base sheets is crash tested for MASH 2016 TL-4 (see Base Sheets R-29 and R-33). Railing options for traffic structures with sidewalks and a posted speed limit less than or equal to 45 mph shall typically use the standard sidewalk section shown in Figure 3.2.4-11 and 3.2.4-12. The Type L railing portion of the sidewalk section shown in Figure 3.2.4-11 is detailed on Base Sheet R-20. This combination railing is crashworthy for NCHRP 350 TL-4. Base Sheets R-28 and R-32 are additional sidewalk applications on structures with a posted speed limit less than or equal to 45 mph and they depict protective fencing with straight or curved Bridge Fence Railing. These combination railings may be utilized upon approval by the District where it is anticipated that a problem with debris or litter being thrown from a structure could cause a hazard to traffic or pedestrian movements below. Both railings are crashworthy for NCHRP 350 TL-4.

The Concrete Bridge Railing, Base Sheet R-38, is a MASH 2016 TL-4 railing and is considered an aesthetic railing developed from Texas railing details. It is typically utilized on old Route 66 structures and other locations as directed by the District offices. See Section 3.2 for bridge deck connection requirement details and geometric parameters for panel and end post lengths.

Base Sheet R-27 depicts permissible anchorage details for bridge structures with staged construction utilizing the Temporary Concrete Barrier (Highway Standard 704001).

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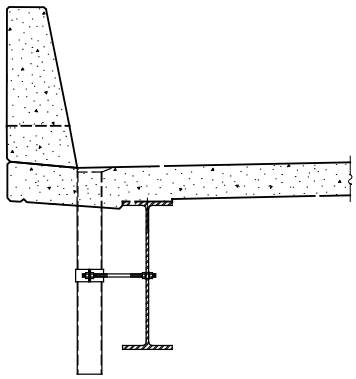
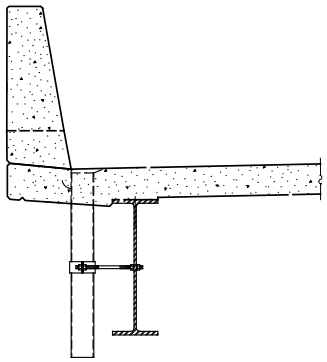
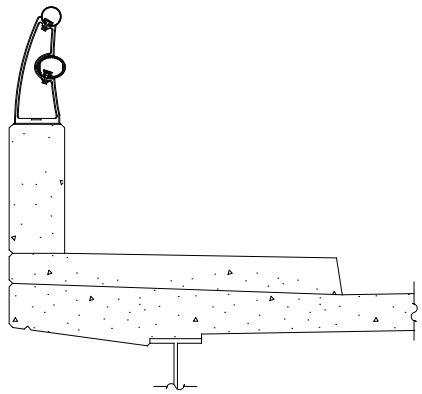
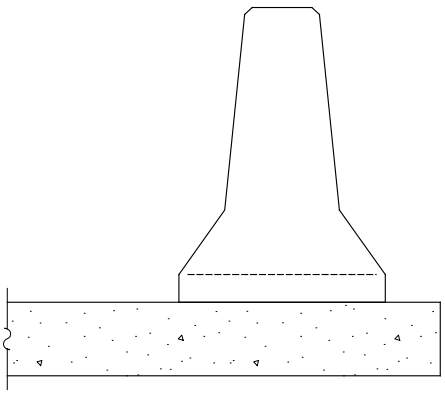
Detail	Description	Crash Test Designation	Max. Post Spacing	Traffic Barrier Terminal
	39” Constant-Slope Concrete Parapet See “Concrete Superstructure” Base Sheets	MASH TL-4	N.A.	H.S. 631031
	44” Constant-Slope Concrete Parapet See “Concrete Superstructure” Base Sheets	MASH TL-5	N.A.	H.S. 631031
	Base Sheet R-20 Aluminum Railing, Type L	NCHRP 350 TL-4	7’-0” – 10’-0”	Concrete stub wall Or Roll railing elements to parapet wall as shown
	Base Sheet R-27 Temporary Concrete Barrier Highway Standard 704001	MASH TL-3	N.A.	N.A.

Table 2.3.6.1.7-1

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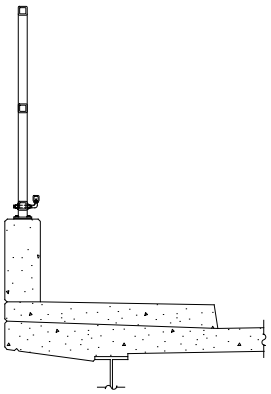
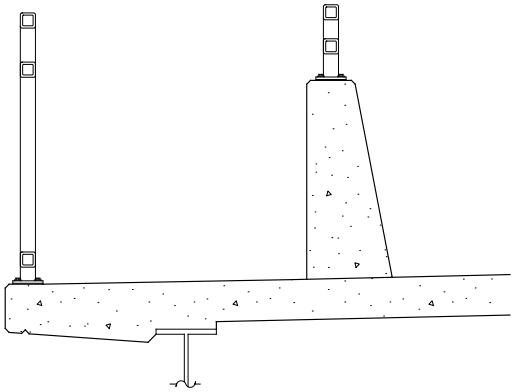
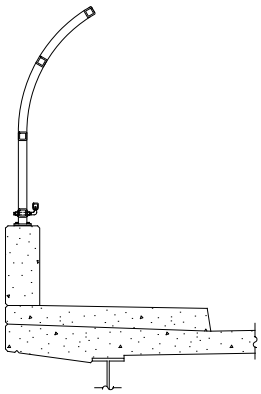
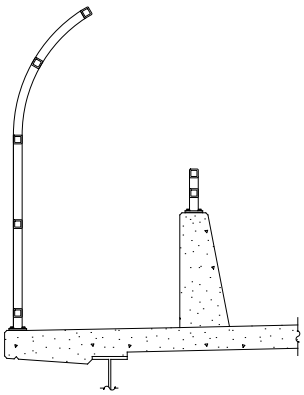
Detail	Description	Crash Test Designation	Max. Post Spacing	Traffic Barrier Terminal
	Base Sheet R-28 Bridge Fence Railing	NCHRP 350 TL-4	10’-0”	See Chapter 38 of the BDE Manual
	Base Sheet R-29 Bicycle Railing and Parapet Railing	MASH TL-4	10’-0”	See Chapter 38 of the BDE Manual
	Base Sheet R-32 Bridge Fence Railing, Curved	NCHRP 350 TL-4	10’-0”	See Chapter 38 of the BDE Manual
	Base Sheet R-33 Bicycle Railing, Curved and Parapet Railing	MASH TL-4	10’-0”	See Chapter 38 of the BDE Manual

Table 2.3.6.1.7-2

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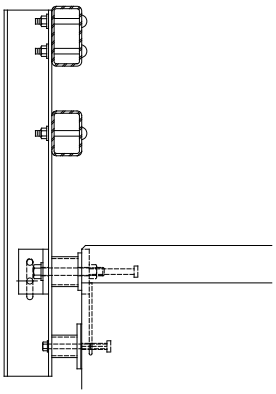
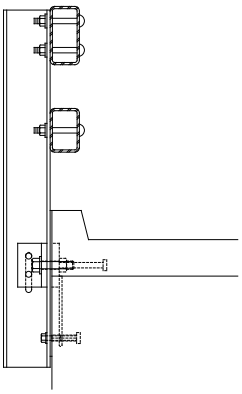
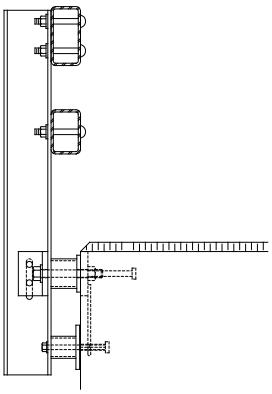
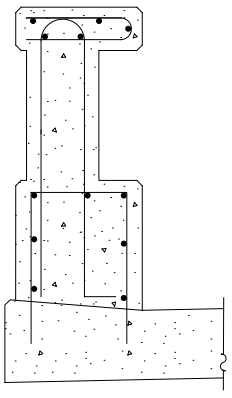
Detail	Description	Crash Test Designation	Max. Post Spacing	Traffic Barrier Terminal
	Base Sheet R-34CWS Steel Railing, Type SM with Concrete Wearing Surface	MASH TL-2	6’-3”	H.S. 631032
	Base Sheet R-34CWSC Steel Railing, Type SM with Concrete Wearing Surface, and Curb	MASH TL-2	6’-3”	H.S. 631032
	Base Sheet R-34HMAWS Steel Railing, Type SM with HMA Wearing Surface	MASH TL-2	6’-3”	H.S. 631032
	Base Sheet R-38 Concrete Bridge Railing	MASH TL-4	N.A.	See Chapter 38 of the BDE Manual

Table 2.3.6.1.7-3

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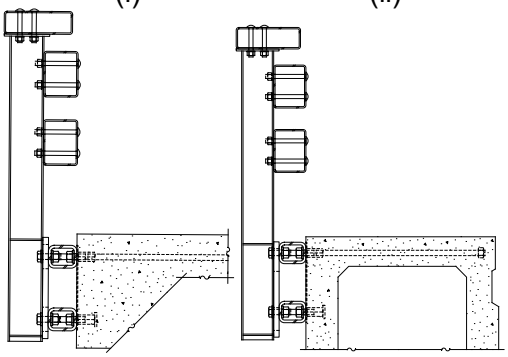
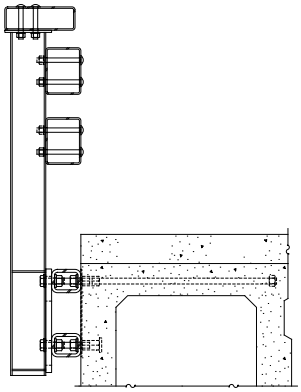
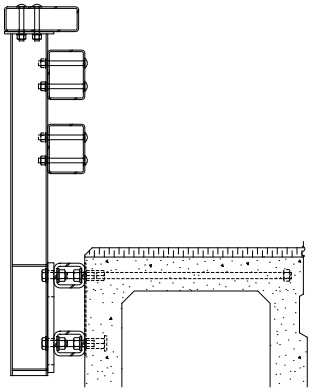
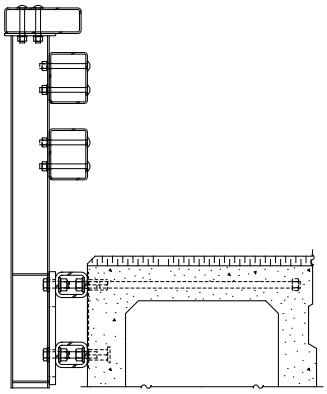
Detail	Description	Crash Test Designation	Max. Post Spacing	Traffic Barrier Terminal
(i) (ii) 	(i) Base Sheet R-40BD Steel Railing, Type IL-OH Bridge Deck (ii) Base Sheet R-40NWS Steel Railing, Type IL-OH Bare Deck Beams	MASH TL-4	8’-0”	H.S. 631061
	Base Sheet R-40CWS Steel Railing, Type IL-OH Concrete Wearing Surface	MASH TL-4	8’-0”	H.S. 631061
	Base Sheet R-40HMAWS Steel Railing, Type IL-OH HMA Wearing Surface	MASH TL-4	8’-0”	H.S. 631061
	Base Sheet R-40HMAWSAS Steel Railing, Type IL-OH HMA Wearing Surface With Approach Slab	MASH TL-4	8’-0”	H.S. 631061

Table 2.3.6.1.7-4

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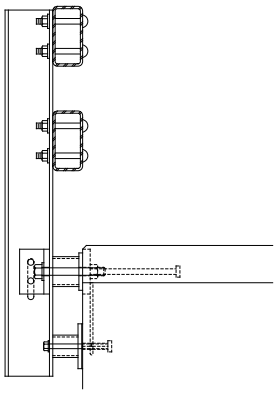
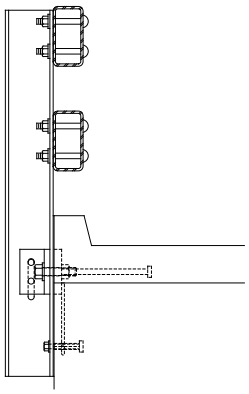
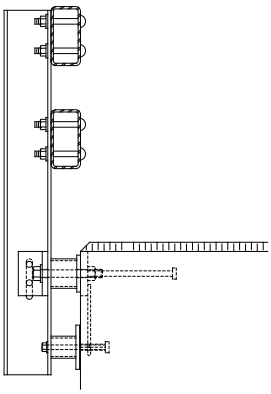
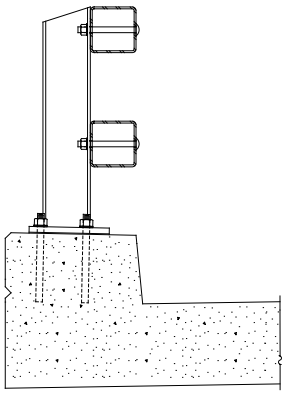
Detail	Description	Crash Test Designation	Max. Post Spacing	Traffic Barrier Terminal
	Base Sheet R-41CWS Steel Railing, Type SMX Concrete Wearing Surface	MASH TL-3 Conditional	6’-3”	H.S. 631032
	Base Sheet R-41CWSC Steel Railing, Type SMX Concrete Wearing Surface with Curb	MASH TL-3 Conditional	6’-3”	H.S. 631032
	Base Sheet R-41HMAWS Steel Railing, Type SMX HMA Wearing Surface	MASH TL-3 Conditional	6’-3”	H.S.631032
	Base Sheet R-42 Steel Railing, Type CO-10 Curb Mounted – New or Existing	MASH TL-4	10’-0”	H.S. 631066

Table 2.3.6.1.7-5

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2.3.6.1.8 Bridge Deck Drainage

Bridge deck drainage should be considered when establishing the profile grade across a structure and when establishing superelevation transition locations.

Bridge drainage scuppers and floor drains can have deterioration effects on superstructure elements. The Department's philosophy for bridge deck drainage is consistent with AASHTO Article 2.6.6. As such, bridge drainage scuppers and floor drains should only be provided on a structure when the off-bridge drainage cannot be contained or captured within the traffic lane encroachment limits. This indicates bridge decks, when feasible, should be watertight (without deck drains) with drainage runoff carried to the ends of the bridge approach slabs. To accomplish this in a safe manner, the drainage runoff shall be:

- contained within the traffic lane encroachment limits (see *Drainage Scupper Location by Hydraulic Analysis* section herein);
- captured with off-bridge drainage features to prevent erosion at the bridge approach ends.

The Engineer responsible for the TSL will coordinate the off-bridge drainage and roadway plan needs with the District.

Table 2.3.6.1.8-1 should be added to the TSL to show the amount of bypass flowrate draining to each quadrant at the end of the bridge approach slab.

BYPASS FLOWRATE FROM BRIDGE LIMITS (END OF APPROACH SLAB) TO ROADWAY

	NW Curbline	SW Curbline	NE Curbline	SE Curbline
Q (C.F.S.)				

Table 2.3.6.1.8-1

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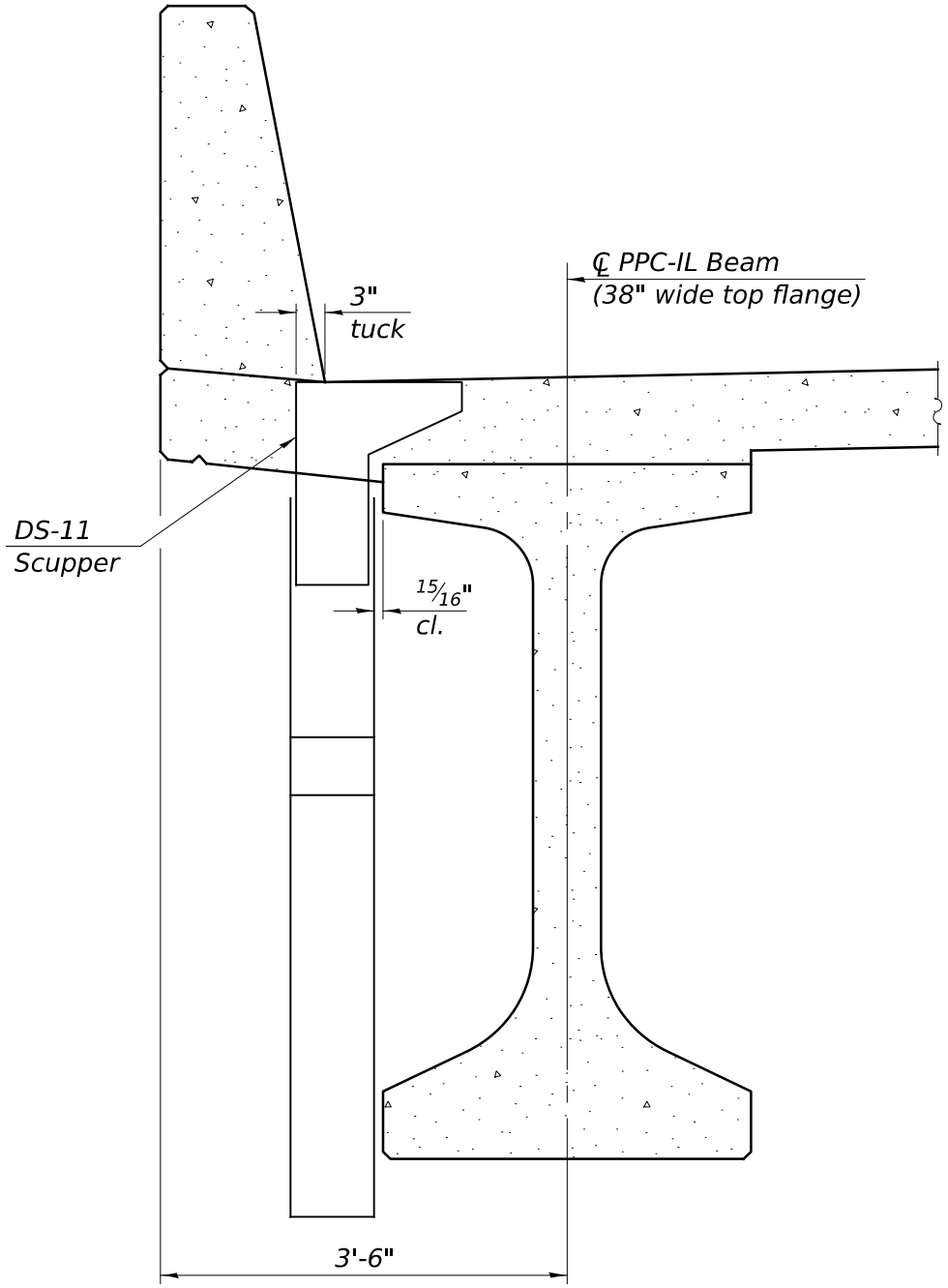
The hierarchy for addressing bridge deck drainage shall be:

1. Provide off-bridge drainage features to capture drainage runoff
    a. Install a combination of curbed pavement connector shoulders, gutters, inlets, and drainage appurtenances (e.g. Hwy. Std. 610001 and others in the 606 series)
    b. Construct surface riprap apron down the embankment (at bridge approach ends).

2. Provide drainage scuppers on the bridge deck (see *Bridge Drainage Scuppers* section below) when there is adequate clearance beyond the fascia beam; see Table 3.2.9-1.

3. When drainage scuppers do not fit between the fascia girder flange and the bridge parapet front face per Table 3.2.9-1:
    a. Revise top flange width (steel beams) or select a narrow top flange (concrete beams), and/or increase deck overhang in combination with a kinked drain pipe (see Fig. 3.2.9-2)
    b. Tuck the scupper beneath the parapet per Figures 2.3.6.1.8-1 through 2.3.6.1.8-2 (see Section 3.2.9)
    c. Notch/cope/core the top flange of PPC beams per Figure 2.3.6.1.8-3 (after the beam has been cast) as allowed by beam capacity
    d. Increase the deck overhang beyond the 3'-6" maximum and design for the exterior beam (requires BBS approval)

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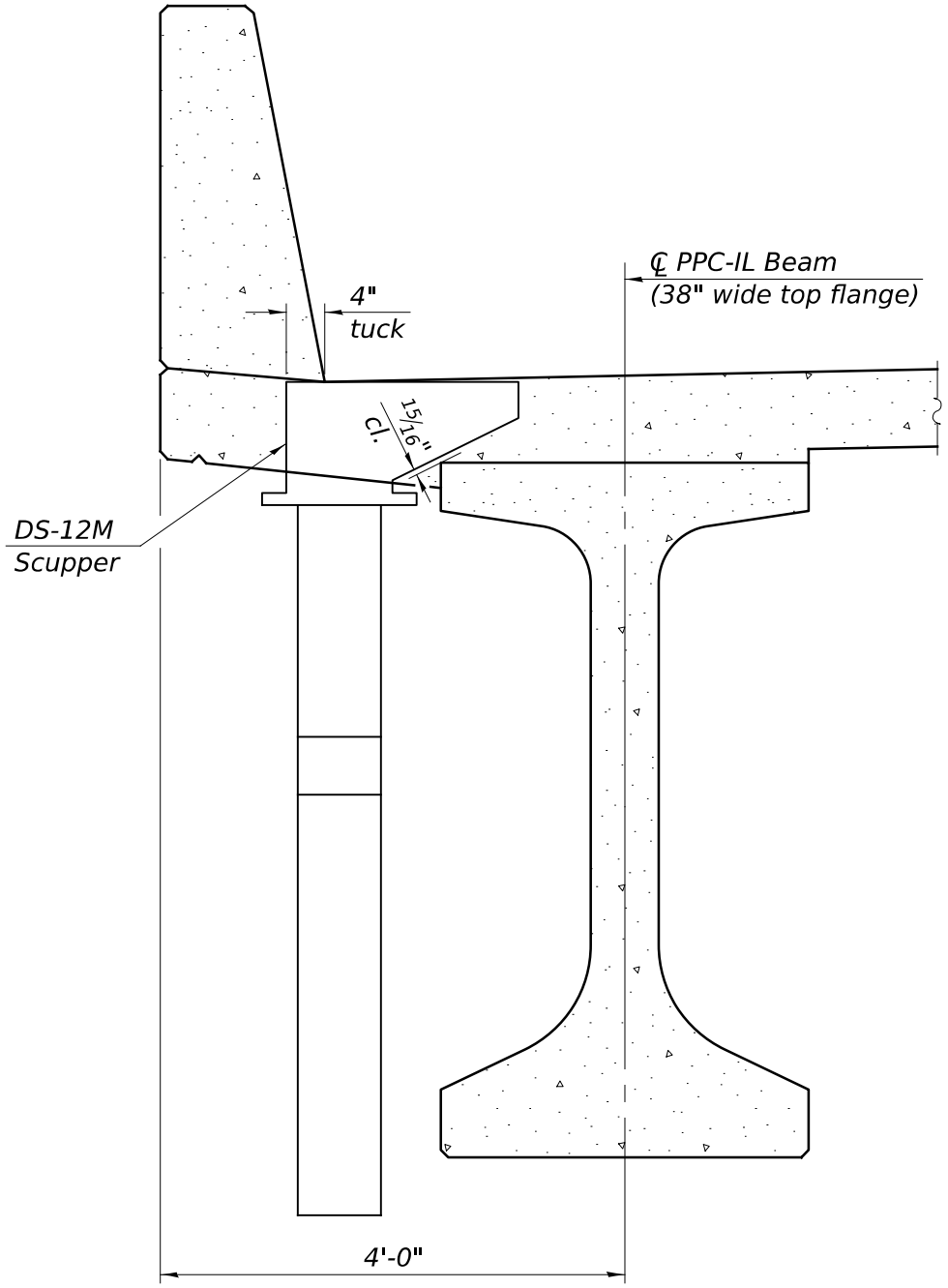
ELEVATION

Note:
BBS approval required for 3'-9" overhang (with no tuck).

ALTERNATE SCUPPER POSITIONING FOR PPC-IL DS-11

Figure 2.3.6.1.8-1

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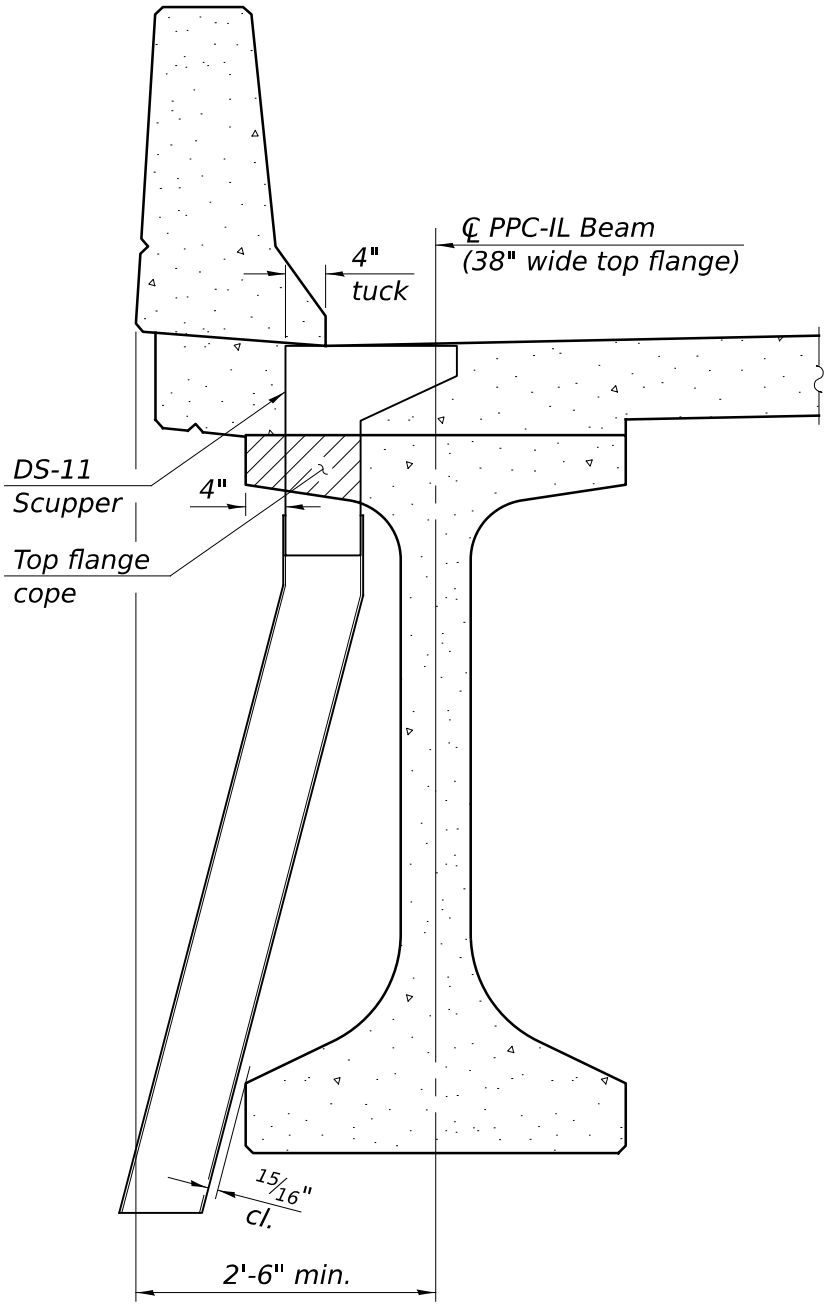
ELEVATION

Note:
DS-12M shown, DS-12 similar.
BBS approval required for use of DS-12M or DS-12 with 4'-0" overhang (with shown tuck) or larger (with smaller tuck).

ALTERNATE SCUPPER POSITIONING FOR PPC-IL DS-12M

Figure 2.3.6.1.8-2

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ELEVATION

Note:
Rehabilitation for F-shape parapet shown.
Tuck may vary for other railing types and overhang widths.

ALTERNATE SCUPPER POSITIONING FOR PPC-IL DS-11 (REHABILITATION)

Figure 2.3.6.1.8-3

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It is desirable that profile grades be established such that the longitudinal grade on a bridge is not less than 0.5%. In certain circumstances, such as near the crest of vertical curves, grades less than 0.5% may not be avoidable; however, efforts should be made to minimize these areas.

Profile grades of less than 0.5% are particularly discouraged for precast prestressed concrete deck beam superstructures with curbs or parapets. These structures typically do not have drainage systems, and ponding of runoff results in premature deterioration of the keyways and beams. However, railings on Base Sheets R-34CWS and R-34HMAWS are curbless and allow flow to run off the side of the bridges. While this method of bridge drainage may be unacceptable in some urban settings or over railroads, it is satisfactory for deck beam bridges in most rural applications.

Typically, the minimum cross slope should be 1.5%. At superelevation transitions where the cross slope reverses from full crown to full superelevation, care should be exercised to avoid impoundments and to eliminate cross road flow.

See Section 3.2.9 for details of floor drains and drainage scuppers, minimum deck overhang limitation, and drain pipe recommendations.

Bridge Drainage Scuppers

Drainage scuppers are required on bridge decks wherever needed to prevent gutter flow spread from exceeding traffic lane encroachment limitations. The spread of gutter flow, under a rainfall intensity of 7 in. per hour, shall not encroach on the traveled way more than 1 ft. when the design speed is 50 mph or greater, nor more than 3 ft. when less than 50 mph.

A drainage scupper shall be provided at a distance D_1 from the high point of the bridge deck and subsequent drainage scuppers shall be spaced at distances D_2, D_3, etc. Theoretical values of D_1, D_2, D_3, etc. should be determined with the equations shown in *Drainage Scupper Location by Hydraulic Analysis* section herein and in accordance with the methods presented in the "Bridge Scupper Placement" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design.

Drainage scuppers are also required at the bottom of any sag vertical curve and to prevent significant flow from crossing the deck immediately ahead of any superelevation transition. In addition, it is desirable to locate a drainage scupper immediately upgrade from a transverse deck expansion joint at piers and abutments.

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Free fall drainage scuppers should not be located within 10 ft. from the faces of substructure elements. Where discharge from the drainage scuppers cannot be allowed to fall free to underlying areas, the drainage scuppers should be attached to downspouts or an enclosed drainage system. Direct downspouts are preferred over a lengthy enclosed drainage system when either is feasible. See Figures 2.3.6.1.8-1 through 2.3.6.1.8-3 for beam size and deck overhang constraints based on scupper type.

For cases when the scupper or drain downspout extension below the fascia beam bottom flange may be impacted by high water or debris, the designer shall replace the fiberglass downspouts with cast iron downspouts to help avoid downspout damage during high water events.

The preferred drainage scuppers hierarchy is as follows:

1. DS-12: This scupper type shall be used whenever feasible. The first digit represents the Department's preference rank, and the second digit represents the scupper width (along gutter flow spread), in feet.
2. DS-12M10: This is a modified DS-12 scupper, hence the "M" within the scupper name. This drain has a larger footprint and steeper slope compared to the DS-12. The two digits after the "M" correspond to the pipe diameter, in inches.
3. DS-11: This scupper type is suitable for limited shoulder width and limited space between girder flange and parapet. This scupper type has a reduced grate length when compared to DS-12 scuppers, thus more scuppers in series are required.
4. DS-33: This scupper type should be avoided, when possible, due to its poor performance and clogging potential. This scupper type is suitable when deck overhang constraints prevent the use of other scupper types. The use of this scupper type requires BBS approval.

Scuppers should ideally be oriented with the downspout outside of the fascia beam for best efficiency. Orientation with the downspout inside of the fascia beam (to accommodate unique situations; such as small deck overhangs) is allowed but not preferred due to reversal of the downspout slope. See Figure 3.2.9-6, Figure 3.2.9-8 and Figure 3.2.9-10.

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Floor Drains

Bridge decks or portions thereof on vertical tangent grades of less than 0.5% should be provided with standard free fall floor drains spaced at 15 ft. centers. Similar provisions should be made on crest vertical curves with K-values of 167 or greater over the portion of the curve having a grade of 0.3% or less. Crest vertical curves with K less than 167 need not be provided with floor drains. (See equation in the *Drainage Scupper Location by Hydraulic Analysis* section herein for the definition of K.)

Free fall floor drains should not be located within 10 ft. from the faces of substructure elements. Where free fall drains are not permitted (i.e. over roadways, railroads, etc.), a special investigation should be conducted to determine whether to provide special drainage scuppers attached to an enclosed drainage system, to re-space the drains, or to omit the drains.

The Department's preferred floor drain is a 6" diameter drain. Alternate floor drains (8" diameter, or 4"x12" rectangular) are available.

Off Bridge Drainage

To address runoff directed to the end of the bridge approach slab, off-bridge drainage features may be required to control roadway and berm slope erosion. Off-bridges drainage features shall

- be designed by the roadway designer for the bypass flowrates on Table 2.3.6.1.8-1 (as a general rule, off-bridge velocity rates less than 1 ft/sec may not require off bridge drainage features)
- be specified on the roadway plans
- consist of inlets placed in the curbed roadway pavement connector (See Hwy. Std. 420401 and 610001), surface riprap aprons placed at the end of the bridge approach slab with uncurbed roadway pavement connector (where aesthetics is not a concern), or a combination of both.

For bridges with expansion joints, bridge approach shoulder inlets (Highway Standard 606001 or 606006) are acceptable, if needed.

At bridges along curbed roadway sections, the gutter flow entering the bridge should be intercepted by an inlet in the roadway pavement connector immediately ahead of the bridge. Also, at bridges along curbed roadway sections, with off-bridge runoff, the drainage inlets shall be spaced and detailed within the roadway plans.

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Drainage Scupper Location by Hydraulic Analysis

The number and spacing of drainage scuppers on a bridge deck should be computed from the following formulae. These formulae are applicable to flow in triangular channels. The "Bridge Scupper Placement" Design Guide (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design) presents example calculations.

$$d_{max} = \frac{T_{max}}{z}$$

$$Q_{Di} = \frac{0.56z}{n} S_i^{1/2} d_i^{8/3}$$

$$D_i = \frac{43560}{CIW} Q_{Di}$$

$$d_i = \frac{T_i}{z}$$

$$d_{bi} = d_i\left(1 - \frac{w_d}{T_i}\right) = d_i - \frac{w_d}{z}$$

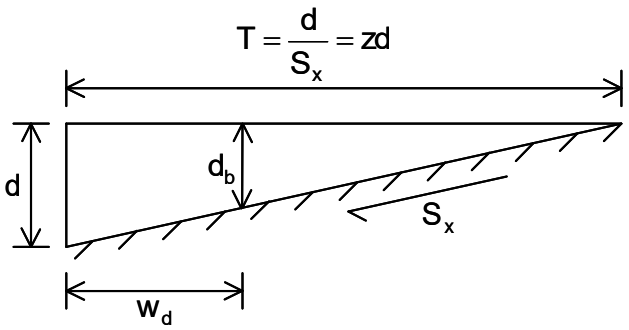
$$V_i = \frac{1.12}{n} S_i^{1/2} S_x^{2/3} T_i^{2/3}$$

$$R_{fi} = 1 - 0.09(V_i - V_0) \le 1.0$$

$$Q_{bi} = \frac{0.56z}{n} S_i^{1/2} d_i^{8/3}(1 - R_{fi}) + \frac{0.56z}{n} S_i^{1/2} d_{bi}^{8/3} R_{fi}$$

$$Q_{TOT\,i} = Q_{Di} + Q_{b(i-1)}$$

$$K = \frac{L}{|g_2 - g_1|}$$



Triangular Channel Section

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Where:

C	=	runoff coefficient = 0.95
d_{bi}	=	depth of bypass flow, i = 1 to no. of inlets-1 (ft.)
d_i	=	actual depth of flow at curb face, i = 1 to no. of inlets (ft.)
d_{max}	=	maximum allowable depth of flow at the curb face (ft.)
D_i	=	distance from high point on bridge to location of first inlet or distance between inlets, i = 1 to no. of inlets (ft.)
g_1	=	grade of initial tangent (%)
g_2	=	grade of final tangent (%)
I	=	rainfall intensity = 7 (in./hr.)
K	=	vertical curve length coefficient
L	=	length of vertical curve (ft.)
n	=	Manning's roughness coefficient = 0.013 (concrete surface)
Q_{bi}	=	flowrate bypassing preceding inlet, i = 1 to no. of inlets-1 (cfs)
Q_{Di}	=	flowrate from drainage area at inlet, i = 1 to no. of inlets (cfs)
R_{fi}	=	frontal flow capture fraction (equals 1 if $V_0 > V$)
S_i	=	longitudinal slope at inlet*, i = 1 to no. of inlets (ft./ft.)
S_x	=	cross slope (ft./ft.)
T_i	=	actual gutter flow spread at inlets, i = 1 to no. of inlets (ft.)
T_{max}	=	maximum gutter spread (ft.)
		shoulder width + 1 ft. for speeds ≥ 50 mph
		shoulder width + 3 ft. for speeds < 50 mph
V_i	=	actual gutter velocity at inlet, i = 1 to no. of inlets (ft./sec.)
V_0	=	grate splash over velocity (ft./sec.)
	=	2.8 for DS 11, DS 12, and DS 33
		5.8 for DS12M10
w_d	=	width of scupper (ft.)
W	=	width of deck drained (ft.)
z	=	reciprocal of cross slope

*Portions of decks where the longitudinal grade is less than 0.5% shall be assumed to have a grade of 0.5%.

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2.3.6.2 Substructure Component Selection

2.3.6.2.1 Abutment Type

Common abutment types fall into four categories: open, closed, MSE, and vaulted. Historically, open abutments have also been referred to as pile bent or spill through. Open abutments include integral, semi-integral and stub. Closed abutments are of two types, restrained top and bottom or cantilever. Vaulted abutments are of two types, filled and unfilled. Section 3.8 contains technical details, policies and procedures for the design of abutments. See also the "Bridge Length" portion of the "TSL Guidelines" Section 2.3 herein for additional information.

Individual pile encasement shall be provided for steel piles and pile reinforcement shall be provided for metal shell piles at all semi-integral and stub abutments. For integral abutments, pile encasements are omitted in order to increase abutment flexibility and better accommodate thermal displacements. See Base Sheets F-MS and F-HP in the "Piles" IDOT CADD Bridge Cell Library for details.

Integral Abutment

The preferred open abutment type is integral. Use of integral abutments on structures beyond the limitations below require approval of the BBS. Typically, this entails detailed soil/structure interaction studies which prove the acceptability of a proposed design.

Expansion joints and other structural releases allow a structure to expand and contract freely with changing temperatures. Integral abutment bridges eliminate expansion joints in the bridge decks by accounting for thermal movement with expansion joints at the end of the approach slabs. This reduces the initial construction cost as well as subsequent maintenance costs. The use of integral abutment structures is permitted within the following limitations:

1. Maximum skew is 45 degrees.
2. Total structure length is 610 ft. maximum. The maximum simple span length is 170 feet. The maximum end span for in a multi-span structure is 200 feet.
3. All structures shall be built on a tangent alignment or built on a tangent (no curved girders).
4. Abutments and piers shall be parallel.
5. Standard cap widths are:
    a. Slabs: 3'-0"
    b. Steel Beams: 3'-8"
    c. Concrete Beams: 3'-11" (May be increased due to profile grade and camber)

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6. Standard centerline of bearings are:
    a. End of all steel beams to centerline brg.: 7”
    b. End of all concrete beams to centerline brg.: 7 ½”
    c. End of all steel beams to centerline brg. (on semi-integral abutment structures with elastomeric bearings): 5 ½”
    d. End of all concrete beams to centerline brg. (on semi-integral abutment structures with elastomeric bearings): 8”

An “Integral Abutment Feasibility Analysis” spreadsheet is available to verify the use of an integral abutment. The spreadsheet and the design guide for the spreadsheet are saved under the “Integral Abutment Pile Selection” at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical.

Standard integral abutment detailing is illustrated in Section 3.8.3. Integral abutment depths can be made deeper to shorten the structure, comply with slope wall layout, or span length limitations. Integral abutment depth limitations are shown in Figure 2.3.6.3.3-5 and Figure 3.8.3-3 thru Figure 3.8.3-5.

When integral abutment foundation limitations are exceeded, semi-integral or stub abutments should be used. These can be supported by either spread footings, drilled shafts or piles below a concrete cap block. See Sections 3.8.4 and 3.8.5 for details.

For repair scope of converting existing abutments into an Integral Abutment type, see Section 2.5.8 of the latest Structural Services Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Repairs).

Semi-Integral Abutment

Semi-integral abutments may be applicable for new construction when piles are battered, set in rock, or have multiple rows. Other cases where this type could be appropriate include use of drilled shaft foundations and those supported by spread footings. All of these foundation types prohibit the use of integral abutments. Generally, a total bridge length greater than 130 ft. should be planned with similar abutment types on both ends. These projects should be considered on a case-by-case basis, and the Bureau of Bridges & Structures should be contacted for approval.

See Section 3.8.4, 3.8.5, and the “Semi-Integral Abutments” discussion in the “Evaluation Process” Section 2.4 for detailed information concerning semi-integral, semi-integral conversion, and other open abutment types such as stub. The skew for a semi-integral structure is only limited by the permissible

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skew of the joint located at the end of the approach slab. For joint limitations, see Fig. 2.3.6.1.6-1. For repair projects converting existing abutments into a semi-integral abutment type, see Section 2.5.8 of the latest Structural Services Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Repairs). For rehabilitation or reconstruction projects converting existing abutments into a semi-integral Abutment type, see Section 2.4.2.3.

Open abutments with or without MSE walls are not required to be designed for vehicle collisions.

Closed Abutment

Closed abutments are typically cost prohibitive, particularly in stream crossing situations. Also, closed abutments can be a safety concern when placed within roadway clear-zone limits. Therefore, their use should be documented with an economic analysis. There are instances where closed abutments are feasible such as railroad bridges and urban areas where right-of-way is limited for end slopes, thus ruling out open abutments.

MSE Abutment

MSE abutments are typically used when the right-of-way is limited at grade separations; they are not permitted adjacent to waterways. See Section 3.8.9 for details, and see Tables 3.8.9.5-1 and 3.8.9.5-2 for wrap-around configurations and limitations.

Vaulted Abutment

Use of this abutment type shall be coordinated with the BBS. See Section 3.8.14 for details.

2.3.6.2.2 Pier and Bent Type

The number, type and location of piers shall be determined to produce optimum bridge economy and safety within the constraints of vertical and horizontal clearance requirements, stream flow requirements and aesthetics. Bridge piers and bents are generally separated into three main groups.

1. Footing Supported Piers
2. Individual Encased Pile or Drilled Shaft Column Bents
3. Solid Wall Encased Pile or Drilled Shaft Bents

Individually encased pile bents consist of a single row of piles in which each pile is individually encased in a column of concrete and supports a pier cap beam. Individual drilled shaft column bents appear and act in a similar manner as individually encased pile bents. Where required for hydraulic purposes,

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individual piles or shafts may be encased in a solid wall of concrete (referred to as a solid wall encased bent). A web walled drilled shaft bent pier is more economical and should be substituted for an encased drilled shaft bent pier unless a smooth pier face is required. Footing supported piers may be composed of multiple rows of piling, one or more rows of drilled shafts or spread footings on rock or soil. Pier stems extending from the footing may be solid walls, web walls, walls with cantilever extensions (hammerheads), or may consist of a multi-column frame mounted on a "crash wall" which supports a pier cap beam.

Special considerations should be given to pier type in regions with moderate to high seismicity (See the "Seismic Issues" Section 2.3 herein and the Seismic Manual). For footing supported piers, multiple round columns are preferred and optimal for the resistance of earthquake loadings. The footings are typically supported with steel piles, metal shell piles, or drilled shafts. Multiple round column bents are considered optimal for design which considers extreme lateral forces in two orthogonal directions. Solid wall pier bents are also allowed in moderate seismic areas.

Sections 3.9 and 3.10 contain technical details, policies and procedures for the design of piers. Detailed seismic considerations for pier design and analysis are presented in Section 3.15.

Footing Supported Piers

General proportions for grade separation piers are shown in Figures 2.3.6.2.2-1, 2.3.6.2.2-2 and 2.3.6.2.2-3. The ratios given in Figure 2.3.6.2.2-2 should be used with caution for any extreme heights. In all cases, a scale drawing should be made so the pier's true proportion can be visualized. Crash walls typically have rounded ends but may be made square to accommodate issues such as aesthetics or guard rail attachments. The piers in Figure 2.3.6.2.2-1 should be used with open abutment types. Figure 2.3.6.2.2-2 should be used when an aesthetic option is needed at a particular location. The piers in Figure 2.3.6.2.2-3 should be used with vaulted abutments.

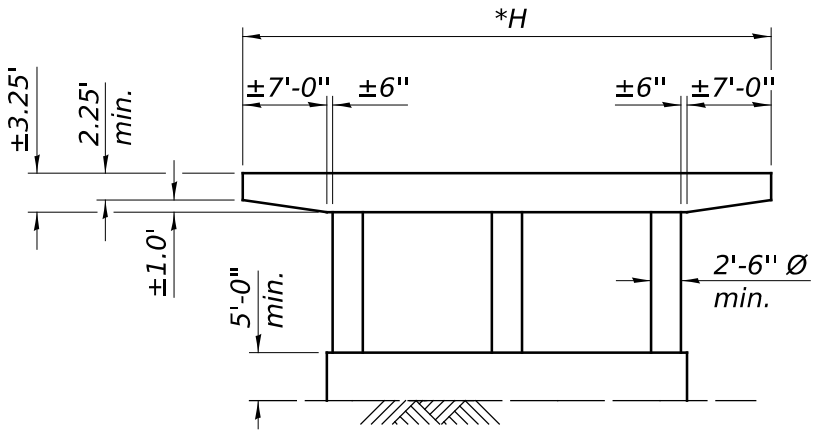
On typical stream crossings, the solid piers shown on Base Sheets P-1, PB-1, and PC-1 of the "Piers" IDOT CADD Bridge Cell Library can be used. The sides of solid piers shall be straight, except, when required by design, the sides shall be battered. The minimum width at the top of a solid pier shall be 2 ft. – 0 in. If the bearing seat requirements are such that more than 2 ft. – 0 in. in width is needed, consideration should be given to the use of a hammerhead grade separation pier or a modified hammerhead pier such as that shown on Base Sheets P-10 of the "Piers" IDOT CADD Bridge Cell Library. The ends of pier stems shall be rounded when located in a main stream. Different pier types should be considered at bridge sites which lend themselves to special architectural treatment.

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All piers within the clear zone of vehicular traffic shall have a crash wall which extends a minimum of 5 ft. – 0 in. above the finished ground line at the pier. The top of the crash wall shall run continuously level. Crash walls typically have rounded ends but may be made square to accommodate aesthetics or guard rail attachments. Figure 2.3.6.2.2-4 illustrates the crash wall criteria for railroad crossings and grade separations.

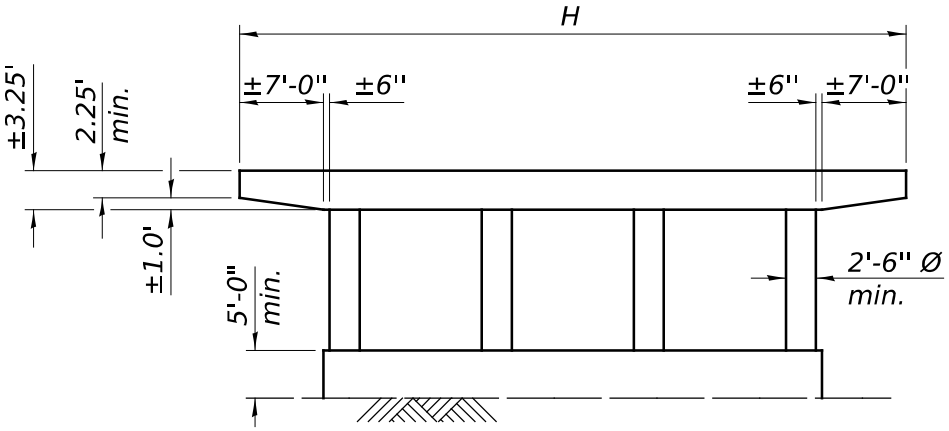
Section 3.9.3.7 contains policies and procedures for the design of new grade separation piers subject to vehicle collisions. The vehicle collision requirements of AASHTO LRFD Bridge Design Specifications Article 3.6.5 must be applied to new grade separation piers classified as critical or essential unless the piers are protected as specified therein. The BBS recommends a cost benefit analysis prior to implementing either option. Existing piers are not required to meet the vehicle collision requirements of AASHTO LRFD Bridge Design Specifications Article 3.6.5.

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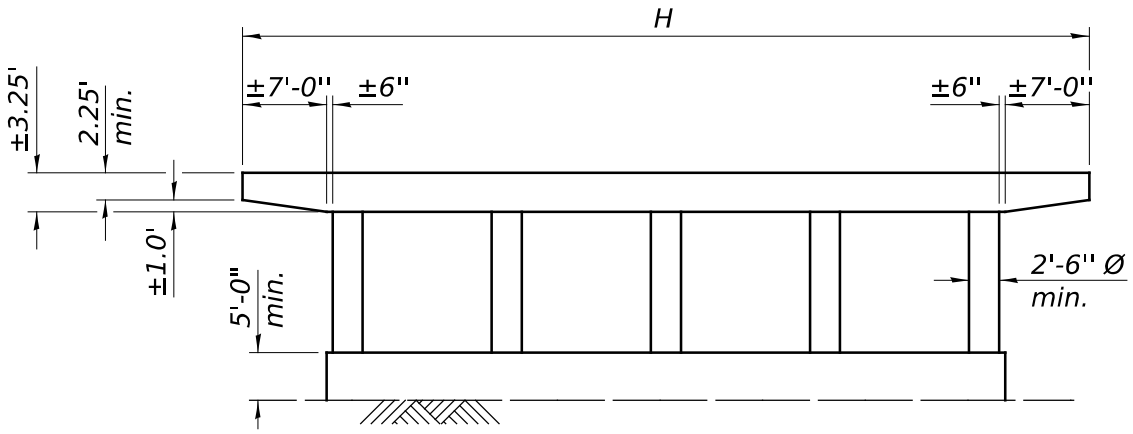
MULTIPLE ROUND COLUMN (3)

Use H = Up to 48'



MULTIPLE ROUND COLUMN (4)

Use H = 48' to 63'



MULTIPLE ROUND COLUMN (5)

Use H = 63' to 78'

Note:
Minimum number of columns shown for ranges of "H".
**See BDE Figure 49-3.J for bridge widths as small as 32'-0" when using 3R criteria.*

GRADE SEPARATION PIERS

Figure 2.3.6.2.2-1

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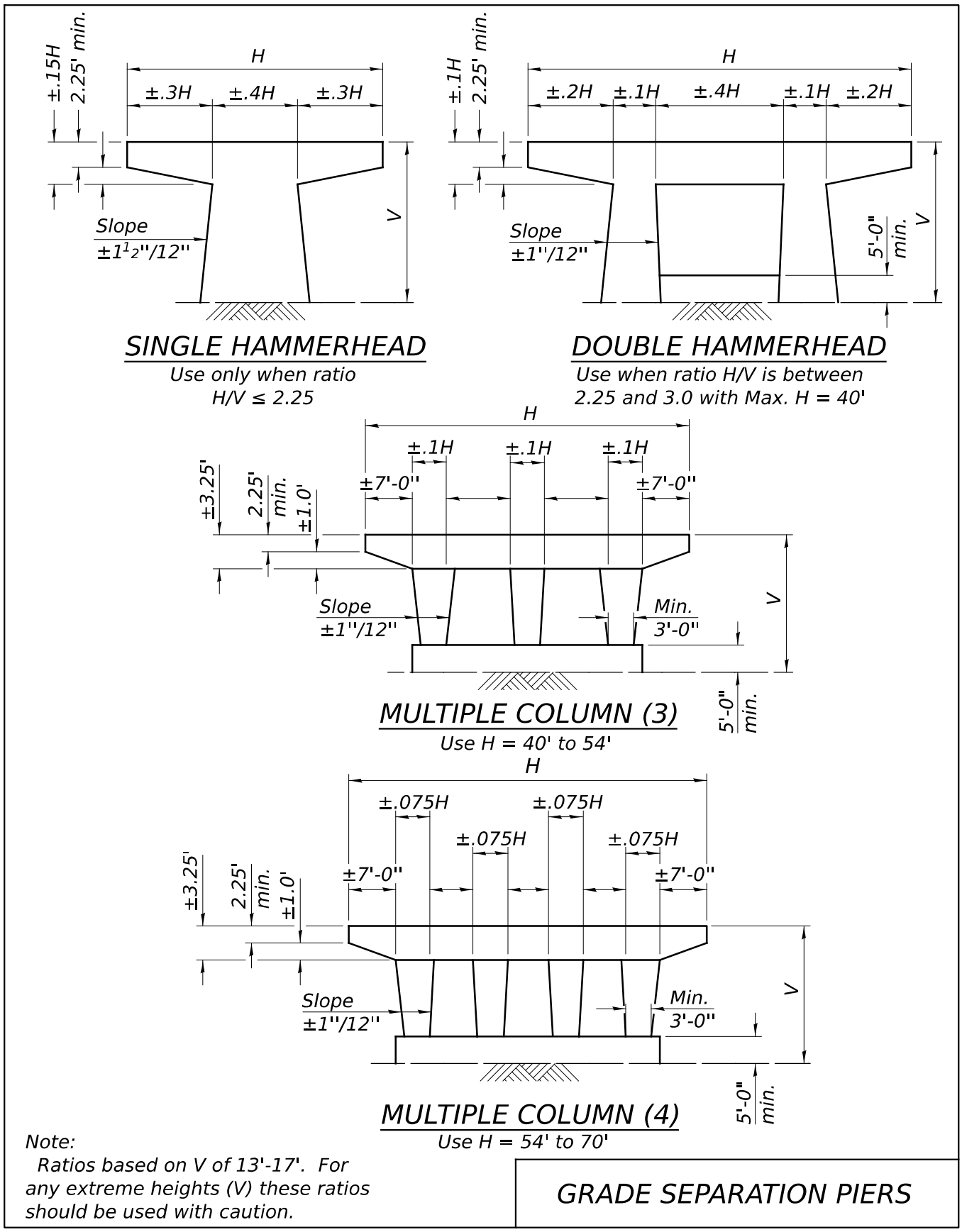
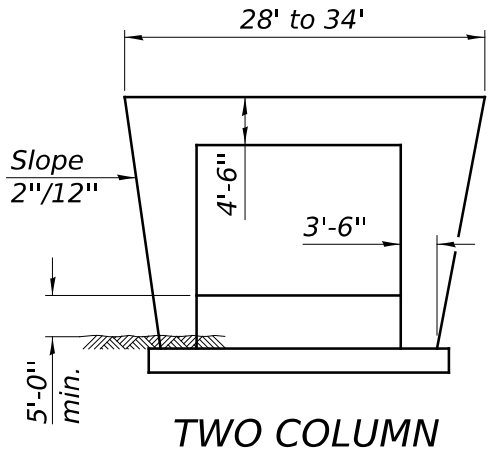
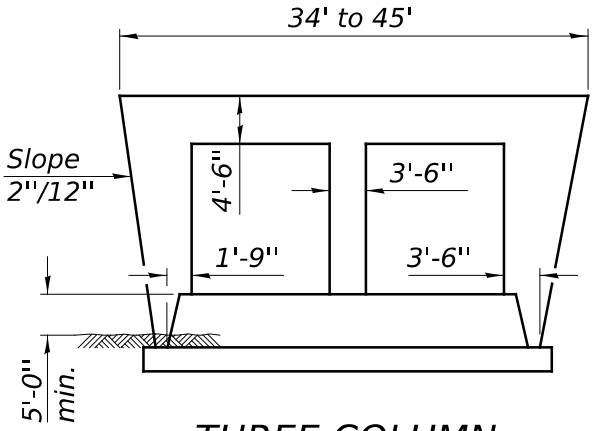


Figure 2.3.6.2.2-2

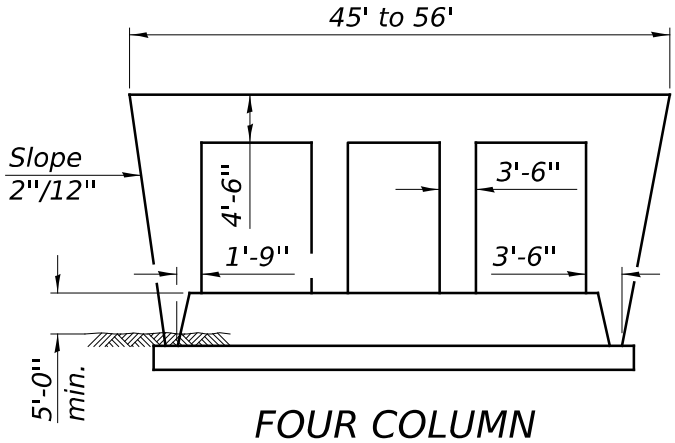
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TWO COLUMN



THREE COLUMN



FOUR COLUMN

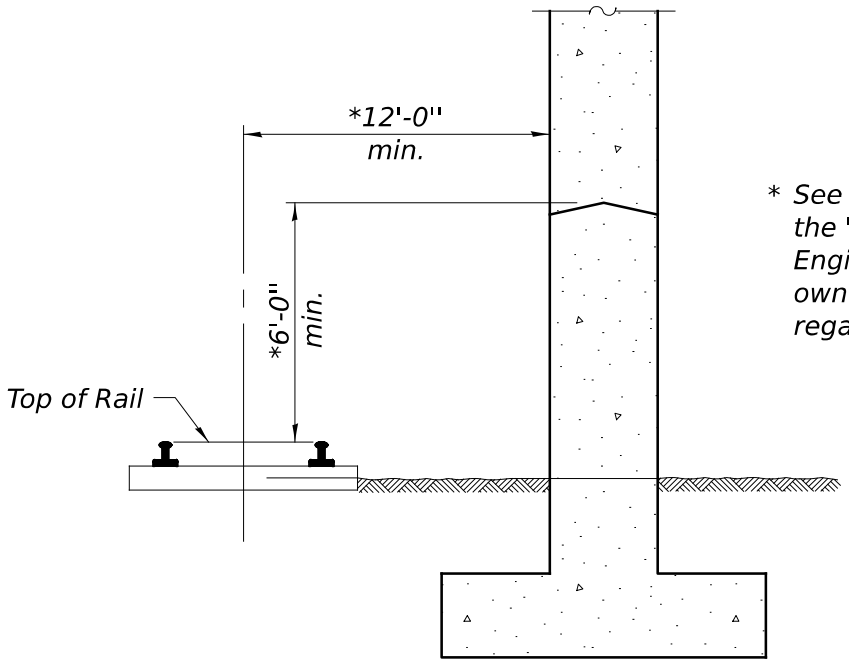
TRAPEZOIDAL PIERS

Note:
For widths greater than 56 feet add columns as needed.

GRADE SEPARATION PIERS

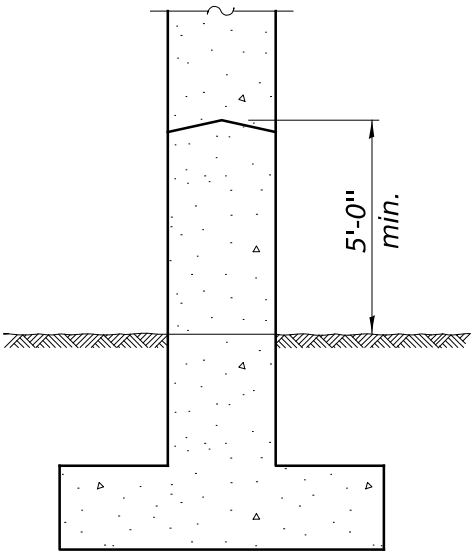
Figure 2.3.6.2.2-3

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* See chapter 8, Article 2.1.5 of the "AREMA Manual for Railway Engineering". Contact Railway owner for additional policies regarding crashwall height.

CASE I
ADJACENT TO RR TRACKS



CASE II
GRADE SEPARATIONS

HEIGHT OF CRASHWALL
OF MULTI-COLUMN PIERS

Figure 2.3.6.2.2-4

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Individually Encased Bent Piers

Individually encased bent piers are primarily used at stream crossings where the potential for the collection of debris and ice is not a concern. Individual pile encasements are primarily intended to provide corrosion protection of steel piles and not to prevent pile damage and waterway blockage due to the collection of debris and ice. The District should be consulted prior to the use of this pier type. For individually encased bent piers, individual pile encasements within channel bank limits shall extend 2.5 ft. below the streambed elevation. Beyond channel bank limits, individual pile encasements for individually encased bent piers shall extend 2.5 ft. below the ground line. Non-encased piles may only be used under special circumstances (such as unmarked routes, etc.) after discussion and concurrence with the Bureau of Bridges & Structures. Example cases for which encasement is not required include when a precast pile is specified, when a corrosion reduced cross section is used in design, or when a corrosion protection system such as paint, galvanization, etc. is utilized. This pier type can also be used at locations without water such as overbank piers.

Individual column drilled shaft bent piers typically provide the most economical alternative when small single row drilled shafts foundations are recommended. They are commonly used at stream crossings where debris collection is not a concern. The top of the drilled shaft shall be shown on the TSL to be located 1 ft. above the Estimated Water Surface Elevation (EWSE) (see the “Cofferdams” portion of the “Temporary Construction Works” section 2.3 herein). If aesthetics allow, permanent casing may be specified to simplify construction of the shaft through the water to above the EWSE. If the appearance of permanent casing is undesirable, construction of the shaft through the water to above the EWSE can be completed with a removable form system. If the removable forms system exceeds 10 ft, a permanent casing shall be specified to make up the difference. Although the pier Base Sheet P-DS in the “Piers” IDOT CADD Bridge Cell Library is detailed for a pier located in water with no permanent casing, permanent casing can be added, or the Base Sheet modified for use at piers without concerns for water such as overbank piers or grade separations where a crash wall is not required.

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Solid Wall Encased Bent Piers

Generally, solid wall encased bent piers are utilized at stream crossings to prevent pile damage and waterway blockage due to the collection of debris and ice. The following guidelines shall be followed for using solid wall encased pile bent piers on stream crossing structures.

Unless otherwise approved by the District, piers located within 25 ft. of the channel bank limits, including piers within the channel itself, shall have a solid wall encasement which extends to 2.5 ft. below the current stream bed elevation. Since the concern for debris collection is minimal outside of these limits, any remaining piers located beyond these limits should be individually encased bent piers with the individual encasements extending 2.5 ft. below the ground surface. Pile bent piers, with the exception of drilled shaft pile bent piers (with web walls), shall not be used on the major river crossings listed below:

1. The Mississippi River
2. The Ohio River
3. The Illinois River
4. The Wabash River
5. The Rock River
6. The Navigable reaches of the Des Plaines River

(See Section 2.3.9.3 for navigable waterways.)

Solid wall encased drilled shaft bent piers may be used at locations requiring a solid wall encasement pier when the use of piles or spread footing foundations is not feasible or economical. This pier type uses small diameter drilled shafts with permanent casing which will be covered by the solid wall encasement. The drilled shaft diameter shall be shown at least 1 ft. less than the encasement width to accommodate shaft construction tolerances. Since the encasement width limits the shaft diameter, more shafts are normally required, which causes this shaft supported pier type to be more costly than the web wall drilled shaft bent pier discussed below. When the EWSE indicates water is expected to contact substructure concrete, the use of a cofferdam may be warranted. See the “Cofferdams” portion of the “Temporary Construction Works” section 2.3 herein for descriptions of the types of cofferdams to be used. Base Sheet P-DSSW (in the “Piers” IDOT CADD Bridge Cell Library) provides a construction sequence and other pertinent information.

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Column Web-Wall Drilled Shaft Bent Pier

Column web-wall drilled shaft bent piers are preferred in lieu of solid wall encased shafts where possible. This pier type is less expensive than the solid wall encased drilled shaft bent pier because it involves less concrete, labor and time to construct. The lower web wall is only connected to the upper web-wall (not the shafts). Consequently, the upper web wall should extend to 5 ft. above the lower web wall or to the project's Design High Water Elevation, whichever is greater. The use of this pier type without cofferdams should be limited to locations where:

- soil conditions are adequate to seal against the shafts and streambed for a tremie or non-tremie pour,
- the total web wall height and thickness does not exceed the capacities of the formwork and bar splicers to form and to hang the web wall.

Base Sheet P-DSWW (in the "Piers" IDOT CADD Bridge Cell Library) provides a construction sequence and other information.

Transfer Beam Drilled Shaft Bent Pier

Transfer beam drilled shaft bent piers are most suitable when the design loading (vessel impact, ice, seismic, etc.) requires more strength, stiffness, and redundancy along the axis of the pier. The transfer beam also provides additional construction tolerances to facilitate incorporation of out-of-plan location shafts which are more likely in deep water shaft installations. Permanent casing can avoid the need for a cofferdam, provide a form through deeper water sites and add protection against stream abrasion. However, since the casings will remain below the beam, aesthetics and debris collection may require other pier types or special modifications to address these issues. Base Sheet P-DSTB (in the "Piers" IDOT CADD Bridge Cell Library) provides more details on this pier type.

Crash Wall Drilled Shaft Bent Pier

Crash wall drilled shaft bent piers are normally used at grade separations where the proximity of the adjacent roadway or railroad traffic dictates the use of a crash wall. Since the crash wall is not acting as a footing, it extends 2 ft. below the finished grade. The crash wall pier can also be used in locations requiring added strength, stiffness, and redundancy along the axis of the pier. In cases where the shaft diameter causes the crash wall width to increase excessively, a wider grade beam may be located below the thinner crash wall to connect it to the larger shafts. Reference Base Sheet P-DSCW (in the "Piers" IDOT CADD Bridge Cell Library) for more information.

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2.3.6.3 Foundation Component Selection

2.3.6.3.1 Foundation Type

There are three basic kinds of foundations used for bridges and structures. These are piles, drilled shafts and spread footings. Piles are the most commonly used foundation type for bridge construction. Piles are usually selected when the foundation soil conditions are not sufficient to support a spread footing and drilled shafts are found to be either too expensive or incompatible for a specific structure.

Drilled shaft foundations are specified to address vertical and lateral load capacity concerns resulting from large scour depths, high seismic loadings, potential liquefaction, low soil strengths and inadequate pile embedment. There are six Departmental Base Sheets for drilled shafts which can be employed for various situations. See Appendix A1.2 for details.

Spread footings can be the most economical foundation type when the soil or rock at a site is sufficient to carry the design loads.

Section 3.10 and the IDOT Geotechnical Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>**Foundations and Geotechnical) should be referenced for detailed technical information concerning the design of foundations. Note, however, that a large portion of the foundation and geotechnical engineering required for a project is contained in the Structure Geotechnical Report (SGR) which is completed during the TSL development phase. The foundation type selected from the three primary categories shall consider the geotechnical issues contained in the SGR, the anticipated cost impact on the project, and structural design feasibility. The most appropriate foundation type shall be shown on the TSL plan.

Geotechnical Issues and Recommendations

A separate SGR shall be completed for each TSL and submitted to the BBS with the TSL plan to assure that all the geotechnical issues have been evaluated and properly addressed. Note that the approval processes for the TSL and SGR are concurrent. The purpose of the SGR is to identify and communicate geotechnical considerations to the planner and provide foundation design recommendations to the designer so they may be incorporated in the contract documents. In some cases, a design phase geotechnical memorandum may be required in addition to an SGR. The IDOT Geotechnical Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical) and AGMU 12.0 (i.e. AGMUs at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical) provide policy and guidance on the content and need for an SGR.

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Piles

When the SGR either recommends or recognizes that a pile supported foundation may be viable, guidance is provided on which pile types are considered feasible, given the soil profile, the range of anticipated axial pile loadings, and embedment necessary to develop fixity. Five basic pile types are allowed: steel H-piles, metal shell, large diameter open ended pipe piles, precast concrete, and timber. Steel H-piles and metal shell piles are, by a large margin, the most commonly used pile types within the five basic pile types. The metal shell, if feasible, is considered the most cost effective option. The engineer responsible for development of the TSL shall utilize the SGR to select the feasible pile types along with their associated lengths, resistances and treatments, conduct preliminary computations to determine the various potential piling-substructure configurations that may be feasible, and perform a cost evaluation to determine the most appropriate pile type to be shown on the TSL plan. Often times, the SGR also contains site specific soil-structure interaction analyses for various pile types considered feasible to assist the planner in preliminary analyses as well as the final designer in assessing the relationship between lateral pile loading, deflection, and developed moment.

Structure or substructure type, or other circumstances may dictate that certain pile types be specified. Examples include integral abutments, pile bent piers, seismic applications, structure demands for ductility, and requirements for combined bending and axial strength. It is also not uncommon that the anticipated vertical loading level may be in a range which eliminates lower capacity pile types.

Metal Shell Piles: Metal shell piles should typically be the preferred pile type option when feasible. They offer many advantages including a relatively low installed cost, availability in several diameters and wall thicknesses, are easily spliced, and allow inspection after driving. The 12 in. Metal Shell with 0.25 in. wall provides a cost effective section which has shown the ability to withstand driving stresses in very-dense or hard layers. In some soil profiles, pre-coring of the pile locations will allow use of this pile size. This pile size can be particularly useful when scour, liquefaction, or downdrag reduce the factored or allowable resistance available to support factored or service loadings applied from the structure.

When the Nominal Required Bearing specified exceeds that available by the 12 in. metal shell piles, a 14 in. Metal Shell pile with 0.25 in. walls can be selected. This larger diameter will also result in shorter pile lengths than 12 in. piles which can provide cost advantages in some cases. In soft/loose soil profiles, the 14 in. may be required to keep the pile length from extending beyond the limits of the subsurface exploration. In stiffer/dense deposits, the 14 in. Metal Shell pile with 0.312 in. walls offers higher resistance to driving damage but this is, to some extent, partially offset by increased end bearing and skin friction resistance which can result in continued risk of either pile installation damage or concerns for inadequate penetrations to develop lateral fixity. This pile type allows the use of a reinforcement cage

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which increases flexural capacity, allows for anchorage of the pile to the substructure or footing, and provides added corrosion protection in addition to the shell.

Metal shell piles have multiple in-state suppliers who can fabricate and deliver piling in a more timely fashion in most cases. On projects where both pile types could be utilized and delays in construction could be problematic, metal shell piles should be given preference over H-piles when selecting TSL pile type.

Steel H-Piles: Steel H-piles (which are almost exclusively HP sections, but may also include other structural shapes) are typically chosen when the nominal required bearing is larger than that of other pile types or when the expected subsurface conditions could cause damage to other pile types. They also have significant lateral load or moment capacity which makes them well suited for applications which require ductility such as for bridges located in moderate to highly seismic regions.

When the estimated tip elevation of the pile is within 20 feet of the bedrock surface, H-piles extended to bedrock and driven to their maximum nominal required bearing are often selected for several reasons. First, the higher available resistance can allow the number of piles to be reduced which results in a net savings despite having to increase the pile length into rock. Second, the risk of driving damage as bedrock is approached is minimized with H-piles. And finally, H-piles driven to bedrock typically require fewer test piles than other pile types which results in cost savings. For conditions where rock depths are close to the ground surface, H-pile “set in rock socket” maybe required as a cost effective alternative.

When bedrock is not an issue, H-piles may not be the most cost effective choice when the loadings and subsurface conditions permit the use of other pile types, such as metal shells. This is because an H-pile has a lower soil volume displacement resulting in a lower total resistance and thus longer pile lengths. Also, required lengths for H-piles are difficult to estimate and cost overruns are common for friction H-piles. However, for piles driven in closely spaced multiple rows, the cumulative soil displacement and densification that might occur in some soils may still require that H-piles be used.

Precast Concrete Piles: This pile type requires special techniques of handing and shipping to avoid damage or excessive stresses. Splicing for additional length or cutting piles to plan elevations can be problematic and, in some cases, is not permitted. However, precast piles do provide some advantages over more common piles and in some applications provide cost savings or structural and aesthetic advantages. They offer excellent corrosion resistance and thus do not require individual pile encasement at pile bent piers or below abutment substructures. Precast-prestressed piles do not require epoxy coated bars and should be selected over precast piles when the subsurface driving conditions are more demanding.

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Timber piles: While cutting this pile type is relatively easy, several concerns exist with regard to driving and splicing timber piles. There is an added need to accurately and conservatively estimate the pile length as well as order lengths correctly. The piles' low maximum Nominal Required Bearing also limits the locations which would permit use of timber piles. Untreated timber piles are not recommended for most permanent IDOT maintained structures, although they may be cost effective for temporary foundation support or for other structures with a short design life. Untreated timber piles have been seen to last many years when they are installed in permanently saturated soils (not subject to wetting and drying cycles). However given the common soil/air/water conditions present at most sites and the added durability of a treated timber pile or other pile type, their use is extremely limited.

TSL Specification: The TSL plan shall specify the general pile type (steel H, metal shell, precast concrete, or timber) to be used at each foundation location. Since it is not possible for the engineer responsible for the development of the TSL to accurately check every load combination using the final pile spacing, size and group configuration for both lateral and vertical loadings, the structural engineer responsible for the final structural plans design will select the final pile size that best satisfies the SGR, TSL and structural design requirements. In cases where project specific conditions mandate further pile specification, the TSL shall further indicate those limitations. These typically included:

1. Indicating HP to eliminate the use of W sections
2. Indicating minimum HP depth (such as Min. HP10 or Min. HP12 etc.) when required by the SGR or by substructure type limitations
3. Specifying Metal Shell Diameter or thickness (such as 14" metal shell, or metal shell with 0.25" walls, etc.) when required by the SGR or by substructure type limitations
4. Indicating Prestressed to eliminate Precast
5. Indicating Treated to eliminate Untreated
6. Indicating "Set in Rock" when driving will not obtain adequate lateral capacity
7. Cofferdams (Type 1 or Type 2), and Seal Coat if required

Drilled Shafts

When the SGR recognizes that a drilled shaft foundation may be viable, the engineer responsible for the TSL development shall compare the various feasible foundation type alternatives to determine if drilled shafts are the most cost effective. Shafts may also be the most appropriate foundation type based on structural feasibility analyses, physical site limitations or subsurface conditions.

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If rock or dense soils prevent driven piles from obtaining sufficient embedment to develop fixity, drilled shafts may be selected to ensure adequate foundation depth into appropriate subsurface materials. When rock is present but too deep for the economical use of spread footings (considering stage construction, R.O.W. excavation support requirements, etc.), drilled shafts extending into rock may be the most cost effective foundation type.

Shafts are also preferred over spread footing foundations when the subsurface information indicates a highly sloping, irregular, or very poorly defined rock surface. In these cases, the plan footing elevation often must be lowered during construction to satisfy the plan minimum footing rock embedment or assure the entire footing bears on uniform non-weathered rock as encountered. Lowering the elevation results in cost increases for rock excavation as well as concrete costs and can delay construction if modified rebar quantities/lengths or redesign is required. The use of drilled shafts at these sites allows for an easier extension or shortening of the shaft rebar cage and results in fewer changes in foundation costs as only the shaft quantity in soil, not the embedment in rock, is typically affected.

Drilled shafts should also be considered if noise caused by driving pile operations has been determined to be unacceptable. Vibrations caused by pile driving can cause damage to buildings or other infrastructure in some cases and necessitates the use of drilled shafts. Limited overhead clearance, proximity of power lines as well as other physical site limitations often causes problems with pile driving, in which case special drilled shaft equipment may be required to address these constraints.

Drilled shaft foundations can be used when concerns exist for potential high loadings, such as earthquake, stream flow/debris and vehicular or vessel impact. They can also be designed to perform well at sites with a lack of resistance due to large anticipated design scour, substantial liquefaction, or very low soil strengths. Compared to piles, drilled shafts provide significantly higher lateral resistance which makes them a viable foundation option when these loading are present.

Piers located near or in deeper stream waters supported by drilled shafts may require a cofferdam (as would other foundation types) for proper construction. However some drilled shaft supported pier types can be constructed with removable forms to avoid the expense of cofferdams when the EWSE is within the limits covered in the “Pier Type” discussion of “Substructure Component” Section 2.3 herein. Permanent casing can also be used to facilitate construction in deeper water and avoid cofferdams, but the added expense and aesthetic impact of exposed steel should be evaluated.

Guidance should generally be provided in the SGR and design phase geotechnical memorandum on side and/or end bearing resistance. When the shafts are to extend to rock, the estimated top of rock elevations should be provided in the SGR so they may be included in both the TSL and Final plans. Although less

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common, shafts may not extend to rock, in which case they utilize both end bearing and side resistance. Shafts located in granular deposits below the water table are least attractive due to the added expense of maintaining shaft excavation support and placing concrete below the water table by tremie or pump. In contrast, shafts terminating in cohesive soils can be more easily drilled, may not require excavation support, and typically can be dewatered to allow concrete placement. When recommended in the SGR and determined to be cost effective, a bell (or enlarged base) may be utilized to maximize the end bearing resistance when cohesive soils exist within the height of the bell. Permanent casing should not be specified as a temporary construction aid, since drilling slurry or temporary casing can be used at the Contractor's option.

As described in Section 3.10 and Appendix A1.2, there are six drilled shaft Base Sheets used for abutment and pier applications.

TSL Specification: The TSL plans shall indicate the foundation type (Drilled Shafts) and include the following information, as appropriate, at each foundation location:

1. Bottom of footing, abutment cap or pier encasement elevation.
2. The "estimated top of rock" elevation (when shafts will extend to rock).
3. Approximate bell or tip elevation (when shaft will not extend to rock)
4. Note "Number, diameter and depth of shafts to be determined in design"
5. Minimum number of shafts per bent (only when required)
6. The estimated water surface elevation (if located at a stream crossing)
7. Permanent Casing, removable forms, or Cofferdam (Type 1 or Type 2).

The designer should not show temporary casing on the TSL or Final plans. The contractor is responsible for using temporary casing, drilling slurry or other systems to maintain the shaft excavation support per the IDOT drilled shaft specifications. The contractor's installation procedure is reviewed and approved by the Department and further adjusted by the contractor to fit the subsurface conditions encountered.

Additional guidance is provided in Section 3.10 on drilled shaft feasibility and design requirements.

Spread Footings

When the SGR recognizes that a spread footing foundation may be viable, the factored bearing resistance, the corresponding footing elevations, and other recommendations should also be provided to aid the engineer responsible for developing the TSL plan in determining if it is the most appropriate foundation type.

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Spread footings are most commonly found to be a cost effective foundation type when fairly level or easily excavated rock is present within a reasonable distance from the existing ground surface. Spread footings may also bear on soils when the calculated resistance and service settlements are acceptable for the applied loadings and structure type. A cost and feasibility analysis shall be conducted using preliminary loadings to verify that the resulting approximate footing size is reasonable and cost effective (considering staging, site excavation constraints and ground water level).

For cost estimates and excavation feasibility evaluations, closed abutment footing widths can be estimated to be between 0.5 and 0.6 times the distance from the crown to the bottom of the footing for which ½ the width is behind and ⅓ is in front of the stem. Bridge pier spread footing widths should be estimated using preliminary loadings and bearing capacity/eccentricity feasibility analyses.

The bottom of spread footings should be located a minimum of 4 ft. below finished grade unless solid rock is encountered. This should, in most cases, preclude concern for frost heave, and provide some tolerance for erosion as well as future utility or other temporary excavations. At stream crossings, spread footings may only be used when embedded in rock and located below the design scour depth.

When selecting an approximate footing elevation, reasonable interpretations and extrapolations between all available boring data are critical such that, upon excavation, the encountered rock or soil deposit will likely have a relatively uniform stiffness throughout the entire footing area. The SGR should provide assistance on selecting an appropriate elevation and bearing resistance.

The IDOT Geotechnical Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical), FHWA-IF-02-054 “Shallow Foundations”, and the AASHTO Standard and LRFD Specifications should be referenced when evaluating the feasibility of spread footings. Bearing capacity, eccentricity limits, sliding, and settlement are the primary geotechnical considerations.

Using possible footing elevations and sizes, the factored (LRFD) equivalent uniform bearing pressures applied to the foundation soils or rock shall be calculated to assess feasibility. The SGR should provide factored bearing pressure resistance values and other footing recommendations. If the bearing resistance values in the SGR are not sufficient to carry the applied bearing loadings, the foundation soils should be evaluated to determine if some type of ground modification can be used to increase the bearing resistance at a reasonable cost. The SGR may contain recommendations on ground modification, and, if not, the geotechnical engineer should be contacted and the SGR modified prior to selecting spread footings as the most appropriate foundation type. When the added cost to improve foundation soils is

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less than the cost of changing to piles or drilled shafts, spread footings may be specified assuming all other geotechnical design considerations can be addressed.

In cases where the bearing resistance is relatively high compared to the applied moments and lateral loading, the eccentricity limitations may control the footing size and thus should be checked during the feasibility analysis. The eccentricity limitations require that the vertical resultant be located at an acceptable offset from the center of the footing as specified in AASHTO. Generally, increasing the footing widths to reduce vertical resultant offset (eccentricity) to within AASHTO limits minimizes footing uplift and assures a reasonable factor of safety against overturning. The criterion changes when the footing is placed on soil as compared to rock.

In most cases, the passive resistance of the soil in front of spread footings is not included when evaluating sliding. Some weak cohesive soils can have problems developing adequate sliding resistance in which case shear keys should be considered. In granular soils however, shear keys are commonly not needed and can be difficult to construct. Spread footings on rock are normally placed some distance below the rock surface to ensure that, upon excavation, the entire footing will bear in competent rock. When added sliding resistance is desired, a minimum embedment in rock can be specified. If a footing is to be placed on shale, a six inch thick “mud slab” or “seal coat” concrete is normally specified to maintain the deposit’s integrity and assist in ensuring sliding resistance.

When spread footings are to be located on rock, settlements do not normally need to be evaluated since they are expected to be less than ½ in. which is tolerable for most bridge configurations used by IDOT. Conversely, spread footings placed in soil deposits shall be evaluated for settlement to assure that the amount of vertical deflection expected is within the tolerance and serviceability of the structure being supported. As a general rule, spread footings may be susceptible to excessive settlement when 1) the footing is located in recently placed cohesive embankment, 2) new fill is being placed adjacent to or above the footing, 3) the moisture content of the foundation soils exceeds 18%, or 4) the equivalent uniform bearing pressure applied exceeds either 1½ times the existing overburden (current vertical loading) soil pressure or 1½ times the unconfined compressive strength. In some cases, ground modification may provide a cost effective means of decreasing settlement to a point that would allow the use of spread footings where piles or drilled shafts would otherwise be necessary.

The approach slab is supported on an approach footing which acts as a spread footing foundation. Thus, the foundation soil conditions should be evaluated for bearing capacity, settlement, etc. during the TSL phase.

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TSL Specification: The TSL plans shall indicate the foundation type (Spread Footing) and include the following information, as appropriate, at each foundation location:

1. Approximate bottom of footing elevation
2. The “estimated top of rock” elevation (when placed in rock)
3. Note “Footing elevation, width, and other proportions to be finalized in design”
4. Any minimum embedment in rock or shear keys proposed.
5. The Estimated Water Surface Elevation or EWSE (if located at a stream crossing)
6. Cofferdams (Type 1 or Type 2), and seal coats if required

Additional guidance is provided in Section 3.10 on spread footing feasibility and design requirements. See also Section 3.11 for the design requirements of CIP and MSE walls which are often supported by spread footing foundation soils.

2.3.6.3.2 Scour Consideration and Design Scour Table

The most common cause of bridge failure is foundation and structural instability resulting from excessive removal of stream bed soils (scour) during major flood flow events. This multidisciplinary concern requires the engineers responsible for hydraulic evaluations, geotechnical/foundation analyses, and structure TSL planning to work together to determine the appropriate design scour depths, strategically locate the substructures and design the foundations to withstand the design flood.

When designing for the Extreme Event II limit state, the check flood scour depth for routine bridges in Illinois shall be determined using the Q200 flood, rather than the Q500 flood referred to in previous editions of the AASHTO LRFD Bridge Design Specifications. The scour depths shall continue to be adjusted for soil or rock type per “*Scour Estimation at Bridges*” below, and the Strength and Service limit state design scour depths shall continue to be determined using the Q100 flood.

While the Q100 and Q200 floods are generally expected to produce the maximum scour depths needed for design, the presence of roadway overtopping may result in lesser floods producing deeper scour depths. When maximum design scour depths are caused by floods less than Q100 and/or Q200, the maximum scour depths shall be used in lieu of the Q100 and/or Q200 scour depths.

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Scour Estimation at Bridges

The Hydraulic Report provides the initial theoretical scour calculations for the 100 year event (Q100) and 200 year event (Q200). These normally consider the cumulative effects of long-term aggradation/degradation, general contraction scour and local pier scour. Analyses may be completed at an early stage in the project using a specific set of assumed parameters including pier width, shape, foundation configuration, soils information, opening area, bridge skew, and others. If these or other key parameters affecting scour are modified during the development of the TSL plan, the hydraulics engineer should be contacted to determine if the calculated scour depths need to be recalculated. Refer to Chapter 10 of the IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) for more details on scour calculation methods.

The Hydraulic Engineering Circular HEC-18 based scour equations contained in the Hydraulic Report are primarily derived from empirical laboratory research in sand. The local scour equations at piers are specifically for live-bed scour in cohesionless sand-bed streams. Consequently, for some cohesive soil or rock deposits, HEC-18 based scour depths may be excessively deep since they do not account for the increased scour resistance which exists in some non-granular streambed conditions. The Structure Geotechnical Report should include the total Q100 and Q200 scour depths and provide any reductions in the final design scour amount when cohesive soils or rock exist. The Department is conducting research and working to develop more accurate methods of making these scour depth reductions. At select sites where Shelby tube soil samples can be obtained near the pier, the Department's Erosion Function Apparatus (EFA) can be used, primarily on an experimental basis, to determine the erosion rate of cohesive soils and the scour depth can be re-calculated using the SRICOS analysis program. Contact the BBS Foundations and Geotechnical Unit or Hydraulics Unit to determine if this testing and analysis is possible or appropriate on a case-by-case basis. In the absence of an EFA/SRICOS cohesive soil scour analysis, the following general guidance has been used by the Department and is provided to assist the geotechnical engineer in making recommendations on reducing the theoretical, predicted scour depth at typical bridge locations with non-granular streambeds.

1. Non-weathered limestone or dolomite is generally not considered susceptible to scour and, in most cases, should be assumed to arrest scour from extending below the non-weathered elevation. (100% reduction in scour depth)
2. Shale and sandstone deposits are more susceptible to erosion depending on their strength and degree of weathering. In most typical deposits, the amount of scour computed to extend into to this rock may be assumed to be only 10% of the predicted value for sand. (90% reduction in scour depth)
3. When stiff to hard (Q_u > 1.5 TSF) cohesive soil layers exist with no sandy or lower strength layers

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present and the boring data is located close to the proposed substructure, the predicted scour depth can be assumed to be only 50% of the predicted value for sand. (50% reduction in scour depth)

4. When soft to stiff (Q_u between 0.5 to 1.5 TSF) cohesive soils are present with no sandy layers or lower strength layers present, the scour can be taken as 75% of that predicted in sand. (25% reduction in scour depth)
5. When lower strength (Q_u < 0.5 TSF) cohesive soils or substantial layers of sands are present, or the boring data is not close to the proposed pier, the scour should be assumed to act as granular, and, as such, the scour should be taken as 100% of that predicted in the Hydraulic Report. (0% reduction in scour depth)

Most sites will not be easily classified into one of the above categories. It is recommended that some interpolation, weighted averaging, and substantial engineering judgment be used to determine if any reduction can be provided in the SGR.

Foundations Design for Scour

The foundations shall be designed to provide full factored resistance available to resist strength limit state loadings during the Q100 event but shall also provide full factored resistance available to resist Extreme Event II loadings during the Q200 event using a resistance factor or factor of safety equal to 1.0. The geotechnical engineer should discuss the impact of site-specific soils with the engineer responsible for the TSL and determine the Q100 and Q200 scour elevations to be used in the SGR and TSL. See Table 2.3.6.3.2-1 below.

Piers are of primary concern for damage from scour. The TSL engineer should compare the cost and feasibility of designing the piers to withstand the design scour with other alternatives such as relocating the pier or changing foundation type. The TSL engineer may also employ structural countermeasures (examples: sheet piling around foundation, deeper footings) within the development of pier alternatives. These alternatives should be compared utilizing essentially equivalent design scour conditions. There is also the option of improving scour conditions (i.e. reducing estimated scour) by enlarging the waterway opening if the above alternatives prove too costly or infeasible.

A widely employed tactic that is no longer recommended for new structures is the use of “hydraulic countermeasures” such as riprap or gabion baskets to armor the pier at the streambed interface. Per HEC-18 and FHWA hydraulic policy directives, “hydraulic countermeasures” intended to protect the pier or stabilize channel alignment cannot be considered absolute safeguards against scour. It is unrealistic to expect these “countermeasures” to remain stable and in-place throughout the service life of a structure.

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Consequently, the TSL engineer should consider alternatives to ensure the foundation is structurally stable for design scour without the use of riprap, gabions, or some other type of revetment intended to reduce or mitigate estimated scour. Use of riprap at piers is allowed if additional alternatives are also employed. This is employed on an infrequent basis, typically at the request of District or BBS Hydraulics. See Chapter 11 of the IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) for hydraulic scour countermeasure direction. For bridge rehabilitation projects, existing riprap may be considered a "countermeasure" in some cases, with the approval of the district bridge hydraulic engineer and bridge maintenance engineer.

Unlike riprap or other revetments at piers, armored embankments (slope walls) are considered to be scour deterrents for typical IDOT bridge abutments. The combination of an open, "spill-through" abutment configuration set back away from the channel and positioned behind a 1:2 (V:H) embankment lined with Class A4 or A5 stone riprap is considered to be an adequate level of scour protection for open abutment foundations. With this waterway opening geometry and revetment in place, potential damage from a single event is minimized.

As opposed to the relatively immediate and potentially catastrophic development of scour at piers, damaging scour at the abutment slope wall generally results from multiple flood events over a period of time. This rate of scour development and the relative ease of observing scour at abutments (in comparison to piers) generally allows inspectors more time to identify the loss of riprap or embankment material. The primary exception to this generality occurs at bridges where the abutment is not set back from the channel and the slope wall is in proximity to the channel bank. Another example arises when channels have the potential to migrate. At locations where the abutment is not set back from the channel or could become impacted by channel migration or other flow conditions, the TSL engineer should consult with the project hydraulic engineer. Ideally, design recommendations that address this atypical issue by upgrading the slope wall armoring should originate in the Hydraulic Report. The upgrade can consist of larger stone, more rigid revetment (such as slope mattress), or river-training measures in the vicinity of the bridge (such as a bendway weir) to stabilize potential channel migration.

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Design Scour Table for Bridges

In addition to design, inspection of the actual streambed and structure conditions throughout the life of the bridge is required for maintaining public safety through the assurance of design assumptions. In a joint National Bridge Inspection Standards (NBIS) review with IDOT and the FHWA, it was recommended that the design scour elevations be provided on all new bridge plans over waterways to assist the bridge maintenance engineers during their inspections. This will allow a better assessment of the severity of any changes in the streambed surface over time or to quickly identify problem scour conditions that require remediation.

For all state and local agency maintained structures over waterways with the exception of closed bottom box culverts, the Design Scour Elevation Table in Table 2.3.6.3.2-1 below shall be placed near the Waterway Information Table on the TSL plan and on the general plan and elevation sheet of the final design plans for all bridges over waterways. This table documents the soil adjusted calculated scour, the design and check scour as well as the Illinois Structure Information System (ISIS) Item 113 rating.

Event/Limit State	Design Scour Elevations (ft.)				Item 113
	W. Abut.	Pier 1	Pier 2	E. Abut.	
Q100					
Q200					
Design					
Check					

Table 2.3.6.3.2-1

ISIS Item 113 rating is based on the relationship between the Q100 scour elevation and the foundation. The bridge planner shall determine the ISIS Item 113 rating for the proposed structure. The bridge designer shall verify the Item 113 rating after the design is complete. Any required changes to the approved rating shall be reported to the BBS Bridge Planning Unit or Local Bridge Unit for verification. Additional ISIS Item 113 Rating guidance is noted below:

1. Item 113 Rating Table A, B and C (with Table 2.3.6.3.2-A, Table 2.3.6.3.2-B, and Table 2.3.6.3.2-C) have been included herein to provide guidance on determining the appropriate design and check scour elevations, as well as the appropriate Item 113 rating for new or total replacement structures, rehabilitation of existing structures not designed using LRFD, and rehabilitation of existing structures designed using LRFD.
2. While foundation elevations should be located in accordance with Section 2.3.6.2, consideration may be given to lowering a foundation elevation if scour, economics, constructability or environmental

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concerns exist. Unless otherwise noted, ISIS Item 113B – Scour Critical Evaluation Method shall be A – Analytical method.

3. For new or total replacement structures, only ratings of 5, 8 or 9 are allowed. For projects reusing existing substructures, typically only ratings of 5, 7, 8 or 9 are allowed. Consult the Bureau of Bridges & Structures in circumstances where this cannot be met.
4. Submit a completed copy of the Scour Critical Evaluation Coding Report, Form BBS SCE, to DOT.BBS.BridgeMgmt@illinois.gov after the TSL is stamped approved. Determination of Item 113 shall follow the guidance provided in the Rating Tables herein.
5. For all piers where significant debris collection is expected, countermeasures should be considered as part of the project, regardless of the Item 113 rating. For all structures where the Design Flood provides less than one foot of vertical clearance or where the Q100 flood inundates the low beam, countermeasures should be considered as part of the project.

The design scour elevations at each substructure unit should be selected to document the amount of tolerable soil loss at a substructure unit while maintaining the specified factored resistance available. At open abutments (integral, semi-integral, stub) protected with riprap, design scour is typically set not at predicted scour, but at the bottom of the abutment. At piers, the elevation should be taken from the SGR which provides the necessary scour reduction (due to cohesive soil or rock deposits) to the original scour elevation provided in the Hydraulic Report. The adjusted scour elevations shall not be raised any higher than the elevation given in the SGR but may be lowered if the footing is located below these elevations. When the footing elevations extend below the predicted scour, the design scour elevations are typically set at the bottom of the footing. Also as mentioned above, significant revisions during TSL development to the hydraulic design recommendations and approved waterway opening (such as relocating piers, changing low beam clearance, or changing the opening size) may significantly impact the design scour elevations at piers. When this occurs, the TSL engineer should use hydraulic and geotechnical input to determine if the calculated and adjusted scour depths should be revisited.

If ice forces are considered in the design of piers per Section 3.9.3.5, the structure shall also satisfy the Extreme Event II limit state with IC - Ice Load. The scour depth for this condition shall be equal to the lower of the design scour elevation or the midpoint between the finished ground line and the check scour elevation.

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Item 113 Rating - Table A: Design and Check Scour Elevations for New Structures

All new structures over waterways shall be planned such that the Item 113 rating is 5, 8 or 9. For single span bridges with closed abutments and for multiple span bridges, the lowest Item 113 rating of the individual substructure units shall be used for the structure rating. Unless otherwise noted, ISIS Item 113B – Scour Critical Evaluation Method shall be A – Analytical method.

When maximum design scour depths are caused by overtopping floods less than Q100 and/or Q200, the maximum scour depths shall be used in place of the Q100 and/or Q200 scour depths.

Spill Through Abutments

Spill through abutments shall be protected by riprap or slope wall in accordance with Section 2.3.6.3. The design and check scour elevations shall be the bottom of the cap. The Item 113 rating shall be 8. The Item 113B rating shall be B (rational analysis).

Spread Footings

Spread footings shall have the design and check scour elevations located at the bottom of footing elevation. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

The use of spread footings in non-scour resistant material is not recommended and requires prior approval from the Bureau of Bridges & Structures. If allowed, the bottom of footing shall be located at least 1 foot below the calculated Q200 scour elevation.

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Pile Supported Footings

For pile supported footings, the design scour elevation shall be the lower of the bottom of the footing and the calculated Q100 scour elevation. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

The check scour elevation shall be the lower of the bottom of the footing and the calculated Q200 scour elevation.

Foundation design for scour and countermeasures designed for the Q200 flood are required if the calculated Q100 scour elevation is below the bottom of the footing unless approved otherwise by the Bureau of Bridges & Structures.

Drilled Shaft Supported Footings

For drilled shaft supported footings, the design scour elevation shall be the lower of the bottom of the footing and the calculated Q100 scour elevation. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

The check scour elevation shall be the lower of the bottom of the footing and the calculated Q200 scour elevation.

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Pile Bent Piers and Solid Wall Encased Drilled Shaft Bent Piers

For pile bent piers, the design scour elevation shall be the lower of the bottom of the pile encasement and the calculated Q100 scour elevation. If the calculated Q100 scour elevation is within 6 feet of the finished ground line, the Item 113 rating shall be 8. If the calculated Q100 scour elevation is 6 feet or greater below the finished ground line, the Item 113 rating shall be 5.

The check scour elevation shall be the lower of the bottom of the pile encasement and the calculated Q200 scour elevation.

Foundation design for scour is required and countermeasures for the Q200 flood should be considered if the calculated Q100 scour elevation is below the bottom of pile encasement or 6 feet or greater below finished ground line.

Individual Drilled Shaft Bent Piers

For drilled shaft bent piers, the design scour elevation shall be the calculated Q100 scour elevation. If the calculated Q100 scour elevation is less than 6 feet below the finished ground line, the Item 113 rating shall be 8. If the calculated Q100 scour elevation is 6 feet or greater below the finished ground line, the Item 113 rating shall be 5.

For drilled shaft bent piers, the check scour elevation shall be the calculated Q200 scour elevation.

Footings and pile encasements should not be lowered solely for the purpose of achieving an Item 113 rating of 8. Engineering judgment and economics should be used to determine if a foundation elevation lower than current Bridge Manual policy is warranted.

If the required riprap size for the Q200 flood is larger than A5, the designer shall contact the Bureau of Bridges & Structures for further guidance.

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Table A - Design and Check Scour Elevations for New or Total Replacement Structures					
Foundation Type	Calculated Q100 Scour Elev.	Design Scour Elev.	Check Scour Elev.	Item 113	Notes*
Spread Footing in scour resistant material (rock)	Above top of footing	Bottom of footing	Bottom of footing	8	
	Within footing thickness			5	
Spread Footing in non-scour resistant material	Above top of footing	Bottom of footing	Bottom of footing	8	A
	Within footing thickness			5	
Pile supported footing	Above top of footing	Bottom of footing	Lower of calculated Q200 scour elevation and bottom of footing	8	
	Below top of footing	Lower of the calculated Q100 scour elevation and bottom of footing		5	B
Drilled shaft supported footing	Above top of footing	Bottom of footing	Lower of calculated Q200 scour elevation and bottom of footing	8	
	Below top of footing	Lower of the calculated Q100 scour elevation and bottom of footing		5	
Pile bent piers	Less than 6 feet below the finished ground line	Lower of the calculated Q100 scour elevation and bottom of encasement	Lower of the Q200 scour elevation and bottom of encasement	8	C
	6 feet or greater below the finished ground line			5	
Drilled shaft bent piers	Less than 6 feet below the finished ground line	Calculated Q100 scour elevation	Calculated Q200 scour elevation	8	
	6 feet or greater below the finished ground line			5	
Spill thru abutments	Not calculated	Bottom of cap	Bottom of cap	8	D, E

Table 2.3.6.3.2-A

*Table A Notes:

A) Spread footings in non-scour resistant material shall be located such that the bottom of the footing is at least 1 foot below the calculated Q200 scour. Engineered scour countermeasures are required and shall be designed for the Q200 flood. The use of spread footings in non-scour resistant material requires prior approval from the Bureau of Bridges & Structures.

B) Foundation design for scour and countermeasures are required and shall be designed for the Q200 flood unless otherwise approved by the Bureau of Bridges & Structures.

C) Foundation design for scour is required and countermeasures for the Q200 flood should be considered if the Q100 scour is below the bottom of pile encasement or greater than 6 feet below finished ground line.

D) Bridge abutments with embankment cones shall be protected by slope wall or engineered countermeasures.

E) ISIS Item 113B Scour Evaluation Rating Method shall be B – Rational Analysis

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Item 113 Rating - Table B: Design Scour Elevations for Existing Structures Not Designed Using LRFD

All structure projects over a waterway incorporating existing substructures not designed using LRFD methods and requiring a Type, Size and Location plan or a Preliminary Bridge Design Hydraulic Report shall be evaluated for scour due to the Q100 flood using current hydraulic methods. Older reference data including the "check flood" and the requirement to satisfy the extreme event do not apply. The projects shall be planned such that the Item 113 rating is 5, 7, 8 or 9 in the final condition. Under special circumstances and with concurrence of the Bureau of Bridges & Structures, scour critical substructure units may be allowed to remain scour critical subject to the requirements herein.

Previous hydraulic analyses may not reflect current hydraulic data or modeling methods, and the current practice of using soil type adjustment factors in the scour depth calculation may not have been used in older ratings. Consequently, existing structures may have Item 113 ratings inconsistent with current practice. Bridge planners should not assume the current recorded ISIS Item 113 rating is accurate.

The initial evaluation of the structure shall ignore existing scour countermeasures, if present. If the initial evaluation indicates the structure is not adequate for the Q100 flood, the design / construction of existing scour countermeasures, if present, shall be evaluated. If the countermeasures are satisfactory for the Q100 flood, the Item 113 rating shall be 7 and no remediation is required. Countermeasures that are not satisfactory for the Q100 flood shall be retrofitted to mitigate the Q200 flood unless otherwise approved by the Bureau of Bridges & Structures. If retrofitted, the Item 113 rating shall be 7. If not retrofitted, the Item 113 rating shall be 3 and documentation of acceptable past performance, a scour plan of action, and monitoring within the Department's BridgeWatch system is required.

Unless otherwise noted, ISIS Item 113B – Scour Critical Evaluation Method shall be A – Analytical method.

When maximum design scour depths are caused by overtopping floods less than Q100 and/or Q200, the maximum scour depths shall be used in place of the Q100 and/or Q200 scour depths.

If the required riprap size to mitigate the Q200 flood is larger than A5, the designer shall contact the Bureau of Bridges & Structures for further guidance.

Spill Through Abutments

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If a spill through abutment is protected by satisfactorily performing riprap or slope wall, the design scour elevation shall be the bottom of the cap. The Item 113 rating shall be 8. The Item 113B Scour Critical Evaluation Method shall be B (rational analysis).

Spread Footings

Spread footings on scour resistant material (rock) shall have the design scour elevation at the bottom of footing. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

Spread footings on non-scour resistant material having a calculated Q100 scour elevation above the bottom of the footing shall have the design scour elevation at the bottom of footing. Engineered scour countermeasures designed for the Q200 flood are required. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

Spread footings on non-scour resistant material having a calculated Q100 scour elevation below the bottom of footing shall have a design scour elevation at the bottom of footing. Engineered scour countermeasures designed for the Q200 flood are required. The Item 113 rating shall be 7.

Pile and Drilled Shaft Supported Footings

If the calculated Q100 scour elevation is above the top of the footing, the design scour elevation shall be the bottom of the footing. The Item 113 rating shall be 8.

If the calculated Q100 scour elevation is within the limits of the footing, the design scour elevation shall be the bottom of the footing. The Item 113 rating shall be 5.

If the calculated Q100 scour elevation is below the bottom of the footing and the Group I load combination is satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the calculated Q100 scour elevation. Engineered scour countermeasures designed to satisfy the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures. Documentation of acceptable past performance may be used to justify not providing countermeasures. The Item 113 rating shall be 5.

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If the calculated Q100 scour elevation is below the bottom of the footing and the Group I load combination is not satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the bottom of footing. Existing countermeasures, if present, shall be evaluated for the Q100 flood. If the countermeasures satisfy the Q100 flood, no remediation is required. Countermeasures that do not satisfy the Q100 flood shall be retrofitted to satisfy the Q200 flood. The Item 113 rating shall be 7.

All new countermeasures and extensions of existing countermeasures required for widened substructures shall be designed to mitigate the Q200 flood.

Bent Piers with Individually Encased Piles or Solid Wall Encased Piles or Drilled Shafts

If the calculated Q100 scour depth is less than 6 feet below the finished ground line and the Group I Load Combination is satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the lower of the calculated Q100 scour elevation and the bottom of encasement. The Item 113 rating shall be 8.

If the calculated Q100 scour depth is 6 feet or greater below the finished ground line and the Group I Load Combination is satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the lower of the calculated Q100 scour elevation and the bottom of encasement. Engineered scour countermeasures designed for the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures. Documentation of acceptable past performance may be used to justify not providing countermeasures. The Item 113 rating shall be 5.

If the calculated Q100 scour depth is 6 feet or greater below the finished ground line and the Group I load combination is not satisfied with all soil above the calculated Q100 scour elevation removed, the design scour shall be set at the lower of the finished ground line and the bottom of encasement. Existing countermeasures, if present, shall be evaluated for the Q100 flood. If the countermeasures satisfy the Q100 flood, no remediation is required. Countermeasures that do not satisfy the Q100 flood shall be retrofitted to satisfy the Q200 flood. The Item 113 rating shall be 7.

All new countermeasures and extensions of existing countermeasures required for widened substructures shall be designed to mitigate the Q200 flood.

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Bent Piers with Individual Drilled Shafts or Exposed Piles

If the calculated Q100 scour depth is less than 6 feet and the Group I load combination is satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the calculated Q100 scour elevation. The Item 113 rating shall be 8.

If the calculated Q100 scour depth is 6 feet or greater and the Group I Load Combination is satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the calculated Q100 scour elevation. Engineered scour countermeasures designed for the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures. Documentation of acceptable past performance may be used to justify not providing countermeasures. The Item 113 rating shall be 5.

If the Group I Load Combination is not satisfied with all soil above the calculated Q100 scour elevation removed, the design scour elevation shall be the finished ground line. Engineered scour countermeasures designed to satisfy the Q200 flood are required. Existing countermeasures, if present, shall be evaluated for the Q100 flood. If the countermeasures satisfy the Q100 flood, no remediation is required. Countermeasures that do not satisfy the Q100 flood shall be retrofitted to satisfy the Q200 flood. The Item 113 rating shall be 7.

All new countermeasures and extensions of existing countermeasures required for widened substructures shall be designed to mitigate the Q200 flood.

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Table B - Design Scour Elevations for Existing Structures Not Designed Using LRFD				
Foundation Type	Calculated Q100 Scour Elevation	Design Scour Elevation	Item 113	Notes*
Spread Footing in scour resistant material (rock)	Above top of footing	Bottom of footing	8	
	Within footing thickness		5	
Spread Footing in non-scour resistant material	Above top of footing	Bottom of footing	8	A
	Within footing thickness		5	
	Below bottom of footing		7	A
Pile and drilled shaft supported footings	Above top of footing	Bottom of footing	8	
	Within footing thickness		5	
	Below bottom of footing (Group I load combination satisfied)	Q100 scour elevation	5	B
	Below bottom of footing (Group I load combination not satisfied)	Bottom of footing	7	C
Bent piers with individually encased piles or solid wall encased piles or drilled shafts	Less than 6 feet below the finished ground line (Group I load combination satisfied)	Lower of Q100 scour elevation or bottom of encasement	8	
	6 feet or greater below the finished ground line (Group I load combination satisfied)		5	B
	Below bottom of encasement (Group I load combination not satisfied)	Bottom of encasement	7	C
Bent piers with individual drilled shafts or exposed piles	Less than 6 feet below the finished ground line (Group I load combination satisfied)	Q100 scour elevation	8	
	6 feet or greater below the finished ground line (Group I load combination satisfied)		5	B
	Any depth (Group I load combination not satisfied)	Finished ground line	7	C
Spill thru abutments	Not calculated	Bottom of cap	8	D,E

Table 2.3.6.3.2-B

*Table B Notes:

A) Engineered scour countermeasures designed for the Q200 flood are required.

B) Engineered scour countermeasures designed for the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures.

C) Engineered scour countermeasures required to mitigate the Q200 flood are required if countermeasures are not present. If countermeasures are present and mitigate the Q100 flood, no additional countermeasures are required. If countermeasures are present and do not mitigate the Q100 flood, the countermeasures shall be retrofitted to mitigate the Q200 flood.

D) Bridge abutments with embankment cones shall be protected by satisfactorily performing countermeasures.

E) ISIS Item 113B Scour Evaluation Rating Method shall be B – Rational Analysis

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Item 113 Rating - Table C: Design and Check Scour Elevations for Existing Structures Designed Using LRFD

All structure projects over a waterway incorporating existing substructures designed using LRFD and requiring a Type, Size and Location plan or Preliminary Bridge Design Hydraulic Report shall be evaluated for scour due to the design flood and check floods using current hydraulic methods. The design flood shall be the Q100 flood and the check flood shall be the Q200 flood. The projects shall be planned such that the Item 113 rating is 5, 7, 8 or 9 in the final condition. Under special circumstances and with concurrence of the Bureau of Bridges & Structures, scour critical substructure units may be allowed to remain scour critical subject to the requirements herein.

Previous hydraulic analyses may not reflect current hydraulic data or modeling methods, and the current practice of using soil type adjustment factors in the scour depth calculation may not have been used in older ratings. Consequently, existing structures may have Item 113 ratings inconsistent with current practice. Bridge planners should not assume the current recorded ISIS Item 113 rating is accurate.

The initial evaluation of the structure shall ignore existing scour countermeasures, if present. If the initial evaluation indicates the structure is not adequate for the Q100 flood, the design / construction of existing scour countermeasures, if present, shall be evaluated. If the existing countermeasures are satisfactory for the Q100 flood, the Item 113 rating shall be 7 and no remediation is required. Countermeasures that are not satisfactory for the Q100 flood shall be retrofitted to mitigate the Q200 flood unless otherwise approved by the Bureau of Bridges & Structures. If retrofitted, the Item 113 rating shall be 7. If not retrofitted, the Item 113 rating shall be 3 and documentation of acceptable past performance, a scour plan of action, and monitoring within the Department's BridgeWatch system is required.

If the required riprap size to mitigate the Q200 flood is larger than A5, the designer shall contact the Bureau of Bridges & Structures for further guidance.

Unless otherwise noted, ISIS Item 113B – Scour Critical Evaluation Method shall be A – Analytical method.

When maximum design scour depths are caused by overtopping floods less than Q100 and/or Q200, the maximum scour depths shall be used in place of the Q100 and/or Q200 scour depths.
Spill Through Abutments

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If a spill through abutment is protected by satisfactorily performing riprap or slope wall, the design and check scour elevations shall be the bottom of the cap. The Item 113 rating shall be 8. The Item 113B Scour Critical Evaluation Method shall be B (rational analysis).

Spread Footings

Spread footings in scour resistant material (rock) shall have the design and check scour elevations located at the bottom of footing elevation. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

Spread footings in non-scour resistant material having a calculated Q100 scour elevation above the top of the footing and the calculated Q200 scour elevation above the bottom of the footing shall have the design scour and check scour elevations located at the bottom of footing. Engineered scour countermeasures designed for the Q200 flood are required. The Item 113 rating shall be 8 if the calculated Q100 scour elevation is above the top of the footing or 5 if the calculated Q100 scour elevation is below the top of the footing.

Spread footings in non-scour resistant material having either a calculated Q100 or Q200 scour elevation below the bottom of the footing shall have the design scour and check scour elevations located at the bottom of footing. Engineered scour countermeasures designed for the Q200 flood are required. The Item 113 rating shall be 7.

Pile and Drilled Shaft Supported Footings

If the calculated Q100 scour elevation is above the top of the footing and the appropriate limit states are satisfied for both the design and check events with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be located at the bottom of the footing. The check scour elevation shall be the lower of the calculated Q200 scour elevation and the bottom of footing. The Item 113 rating shall be 8.

If the calculated Q100 scour elevation is within the limits of the footing and the appropriate limit states are satisfied for both the design and check events with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be located at the bottom of the footing. The check scour elevation shall be the lower of the calculated Q200 scour elevation and the bottom of footing. The Item 113 rating shall be 5.

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If the calculated Q100 scour elevation is below the bottom of the footing and the appropriate limit states are satisfied for both the design and check events with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be located at the calculated Q100 scour elevation. The check scour elevation shall be located at the calculated Q200 scour elevation. Engineered scour countermeasures designed to satisfy the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures. Documentation of acceptable past performance may be used to justify not providing countermeasures. The Item 113 rating shall be 5.

If the calculated Q100 scour elevation is below the bottom of the footing and the appropriate limit states are not satisfied with all soil above the calculated Q100/200 scour elevation removed, the design scour and check scour elevations shall be located at the bottom of footing. Existing countermeasures, if present, shall be evaluated for the Q100 flood. If the countermeasures satisfy the Q100 flood, no remediation is required. Countermeasures that do not satisfy the Q100 flood shall be retrofitted to satisfy the Q200 flood. The Item 113 rating shall be 7.

All new countermeasures and extensions of existing countermeasures required for widened substructures shall be designed to mitigate the Q200 flood.

Bent Piers with Individually Encased Piles or Solid Wall Encased Piles or Drilled Shafts

If the calculated Q100 scour depth is less than 6 feet below the finished ground line and the appropriate limit states are satisfied for both the design and check events with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be the lower of the calculated Q100 scour elevation and the bottom of encasement. The check scour elevation shall be the lower of the calculated Q200 scour elevation and the bottom of the encasement. The Item 113 rating shall be 8.

If the calculated Q100 scour depth is 6 feet or greater below the finished ground line and the appropriate limit states are satisfied with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be the lower of the calculated Q100 scour elevation and the bottom of encasement. The check scour elevation shall be the lower of the calculated Q200 scour elevation and the bottom of encasement. Engineered scour countermeasures designed for the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures. Documentation of acceptable past performance may be used to justify not providing countermeasures. The Item 113 rating shall be 5.

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If the calculated Q100 scour depth is 6 feet or greater below the finished ground line and the appropriate limit states are not satisfied with all soil above the calculated Q100/200 scour elevation removed, the design and check scour shall be set at the lower of the finished ground line and the bottom of encasement. Existing countermeasures, if present, shall be evaluated for the Q100 flood. If the countermeasures satisfy the Q100 flood, no remediation is required. Countermeasures that do not satisfy the Q100 flood shall be retrofitted to satisfy the Q200 flood. The Item 113 rating shall be 7.

All new countermeasures and extensions of existing countermeasures required for widened substructures shall be designed to mitigate the Q200 flood.

Bent Piers with Individual Drilled Shafts or Exposed Piles

If the calculated Q100 scour depth is less than 6 feet below the finished ground line and the appropriate limit states are satisfied for both the design and check events with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be located at the calculated Q100 scour elevation. The check scour elevation shall be located at the calculated Q200 scour elevation. The Item 113 rating shall be 8.

If the calculated Q100 scour depth is 6 feet or greater below the finished ground line and the appropriate limit states are satisfied for both the design and check events with all soil above the calculated Q100/200 scour elevation removed, the design scour elevation shall be the calculated Q100 scour elevation. The check scour elevation shall be the calculated Q200 scour elevation. Engineered scour countermeasures designed for the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures. Documentation of acceptable past performance may be used to justify not providing countermeasures. The Item 113 rating shall be 5.

If the appropriate limit states are not satisfied with all soil above the calculated Q100/200 scour elevation removed, the design and check scour elevations shall be located at the finished ground line. Existing countermeasures, if present, shall be evaluated for the Q100 flood. If the existing countermeasures satisfy the Q100 flood, no remediation is required. Countermeasures that do not satisfy the Q100 flood shall be retrofitted to satisfy the Q200 flood. The Item 113 rating shall be 7.

All new countermeasures and extensions of existing countermeasures required for widened substructures shall be designed to mitigate the Q200 flood.

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Table C - Design and Check Scour Elevations for Existing Structures Designed Using LRFD

Foundation Type	Calculated Q100 Scour Elev.	Calculated Q200 Scour Elev.	Design Scour Elev.	Check Scour Elev.	Item 113	Notes*
Spread Footing in scour resistant material (rock)	Above top of footing	Above bottom of footing	Bottom of footing	Bottom of footing	8	
	Below top of footing				5	
Spread Footing in non-scour resistant material	Above top of footing	Above bottom of footing	Bottom of footing	Bottom of footing	8	A
	Within footing thickness				5	
	Below bottom of footing OR	Below bottom of footing			7	A
Pile and drilled shaft supported footings	Above top of footing (Strength I and Service I load combination satisfied)	Any depth (Extreme Event II load combination satisfied)	Lower of bottom of footing or Q100 scour elevation	Lower of bottom of footing or Q200 scour elevation	8	
	Within footing limits (Strength I and Service I load combination satisfied)				5	
	Below bottom of footing (Strength I and Service I load combination satisfied)				5	B
	Below bottom of footing (Strength I and Service I load combination not satisfied) OR	Below bottom of footing (Extreme Event II load combination not satisfied)	Bottom of footing	Bottom of footing	7	C
Bent piers with individually encased piles or solid wall encase piles or drilled shafts	Less than 6 feet below the finished ground line (Strength I and Service I load combination satisfied)	Any depth (Extreme Event II load combination satisfied)	Lower of bottom of encasement or Q100 scour elevation	Lower of bottom of encasement or Q200 scour elevation	8	
	6 feet or greater below the finished ground line (Strength I and Service I load combination satisfied)				5	B
	Below bottom of encasement (Strength I and Service I load combination not satisfied) OR	Below bottom of encasement (Extreme Event II load combination not satisfied)	Bottom of encasement	Bottom of encasement	7	C

*Continued on the next page

Table 2.3.6.3.2-C

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Table C (continued)- Design and Check Scour Elevations for Existing Structures Designed Using LRFD

Foundation Type	Calculated Q100 Scour Elev.	Calculated Q200 Scour Elev.	Design Scour Elev.	Check Scour Elev.	Item 113	Notes*
Bent piers with individual drilled shafts or exposed piles	Less than 6 feet below the finished ground line (Strength I and Service I load combination satisfied)	Any depth (Extreme Event II load combination satisfied)	Q100 scour elevation	Q200 scour elevation	8	
	6 feet or greater below the finished ground line (Strength I and Service I load combination satisfied)				5	B
	Any depth (Strength I and Service I load combination not satisfied) OR	Any depth (Extreme Event II load combination not satisfied)	Finished ground line	Finished ground line	7	C
Spill thru abutments	Not calculated	Not calculated	Bottom of cap	Bottom of cap	8	D,E

Table 2.3.6.3.2-C

*Table C Notes:

A) Engineered scour countermeasures designed for the Q200 flood are required.

B) Engineered scour countermeasures designed for the Q200 flood are required unless otherwise approved by the Bureau of Bridges & Structures.

C) Engineered scour countermeasures required to mitigate the Q200 flood are required if countermeasures are not present. If countermeasures are present and mitigate the Q100 flood, no additional countermeasures are required. If countermeasures are present and do not mitigate the Q100 event, the countermeasures shall be retrofitted to mitigate the Q200 flood.

D) Bridge abutments with embankment cones shall be protected by satisfactorily performing countermeasures.

E) ISIS Item 113B Scour Evaluation Rating Method shall be B – Rational Analysis

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Item 113 Rating - Table D: Design and Check Scour Elevations for Three Sided Structures Designed Using LRFD

See Sections 2.3.11.2.2 and 2.3.11.2.4 for scour remediation Methods 1, 2 and 3 details and further guidance.

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Table D - Design and Check Scour Elevations for New Three-Sided Precast Concrete Structures Designed Using LRFD					
Foundation Type	Calculated Q100 Scour Elevation	Design Scour Elevation **	Check Scour Elev. ***	Item 113	Notes
Spread footing or pile supported footing with top of footing at or below Check Scour Elev. in scourable material. (Method 1)	Above Top of Footing	Top of Footing	Top of Footing	8	
Spread footing set min. of 6” into non- scourable material. (Method 1)	Above Top of Footing	Top of Rock	Top of Rock	8	
	Within Footing			5	
Spread footing or pile supported footing protected by bulkhead (Method 2)	< 6’ Below Streambed	Lower of Design Scour (Strength I) or Check Scour	Lower of Design Scour (Strength I) or Check Scour	8	
	> 6’ Below Streambed			5	
Individual spread footing or pile supported footing protected with closed bottom (Method 3)	Above Top of Footing	Top of Concrete Slab	Top of Concrete Slab	8	A, B
	Within or Below Footing			5	
Raft footing (Method 3)	Above Top of Footing	Top of Concrete Slab	Top of Concrete Slab	8	A, B
	Within or Below Footing			5	

Table 2.3.6.3.2-D

*Table D Notes:

A) Closed bottom or Raft footing shall be 3” minimum below the streambed elevation.

B) 4’ cutoff walls measured from the top of bottom slab are required at each end of slab.

** Design Scour is based on the Strength I load group

*** Check scour is based on the Extreme Event load group and includes contraction scour, local scour, and long term degradation scour.

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2.3.6.3.3 Slope Protection and Berms

Slope Protection for Stream Crossings

Layout of slope protection systems for stream crossing structures is shown in Figure 2.3.6.3.3-1 and several online approved TSLs referenced in Appendix A1.3 for examples. In each situation, the slope protection system is developed to protect the bridge embankment endslopes and areas where stream bank failure could endanger the structure or its individual components. The detail shown in Figure 2.3.6.3.3-2 shall be shown on the plans when a standalone riprap treatment is used on the pier. Figures 2.3.6.3.3-3 and 2.3.6.3.3-4 indicate the approved treatments for ending a stone riprap embankment protection system. The flank detail shall be used along both the upstream and downstream sides of the riprap treatment. All required riprap treatment details shall be shown or specified on the TSL.

For additional slope wall information and details, see Section 3.14.

Abutment Cap Geometry

Figure 2.3.6.3.3-5 depicts the preferred methods of treating abutment cap geometry for a single structure with or without varying elevations between exterior beams.

These sketches are presented as guides and it is anticipated that situations will occur which will fall outside the limits defined here. These situations will require combinations of the treatments shown or unique solutions to solve specific problems.

Dual structures will normally require individual evaluation to determine the appropriate berm treatment.

Berm Widths

Figures 2.3.6.3.3-6 and 2.3.6.3.3-7 are provided to show the development of berm widths for open abutment structures.

Slope Walls

Section 3.14.4 presents details and Departmental policies for concrete and bituminous slope walls.

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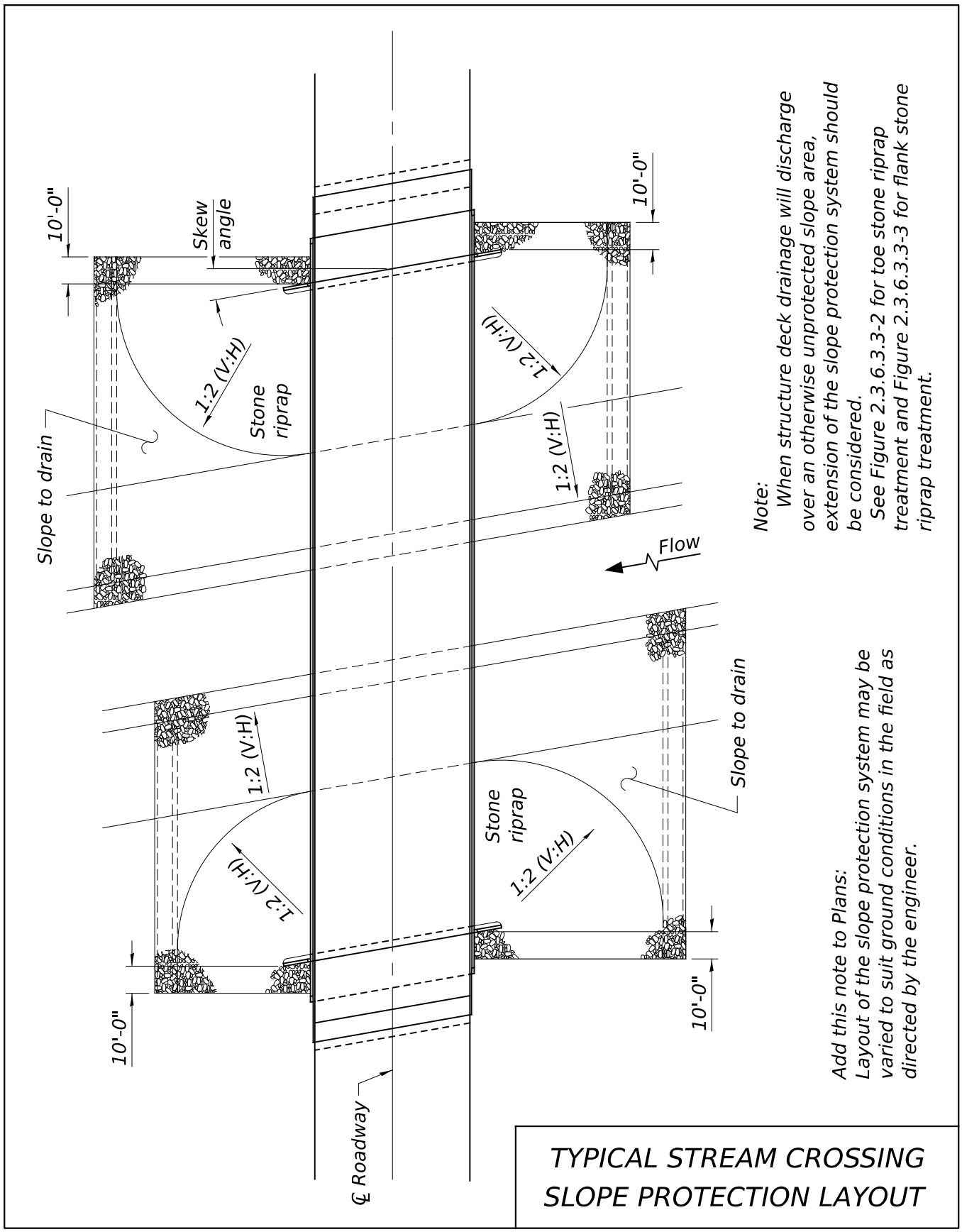


Figure 2.3.6.3.3-1

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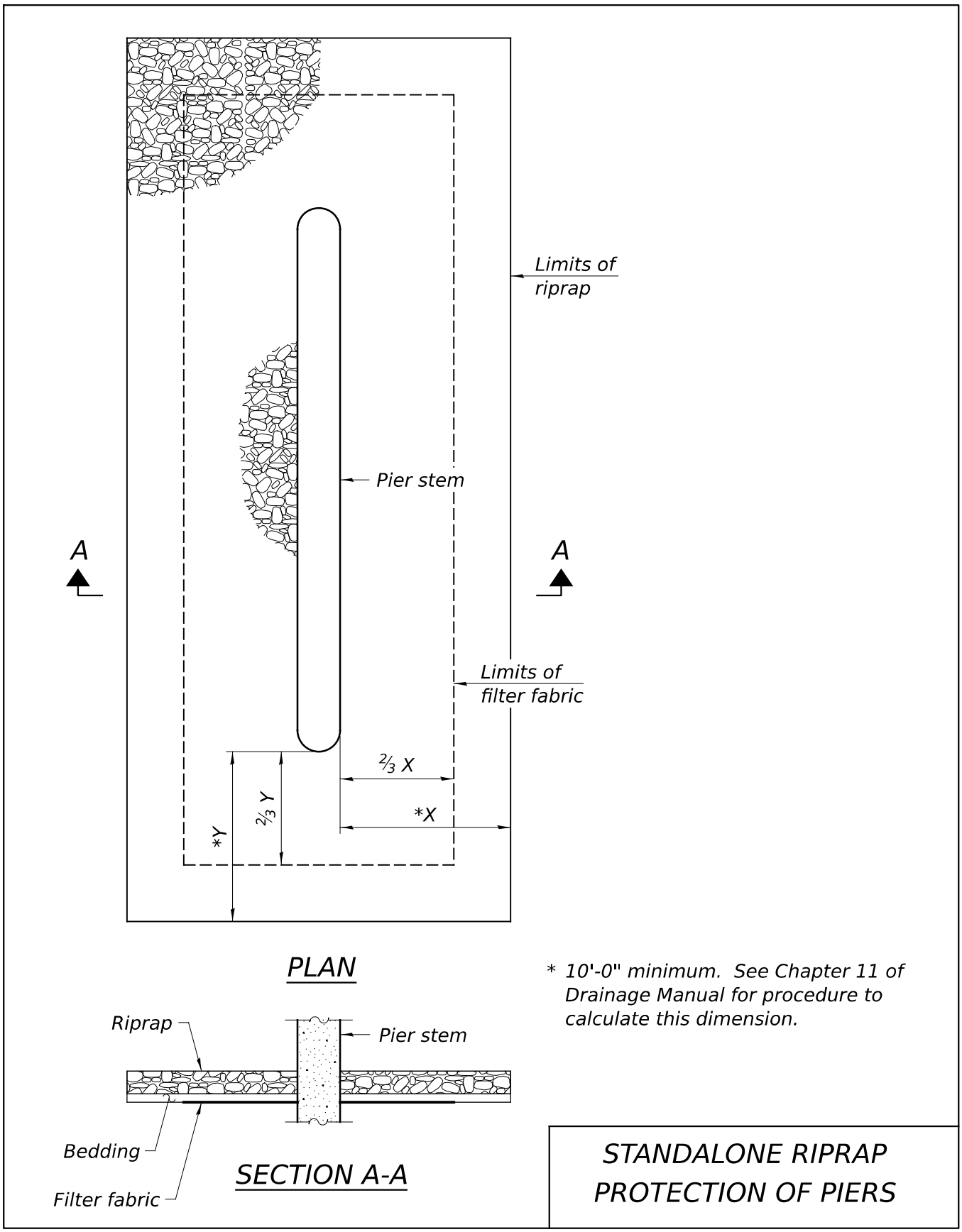
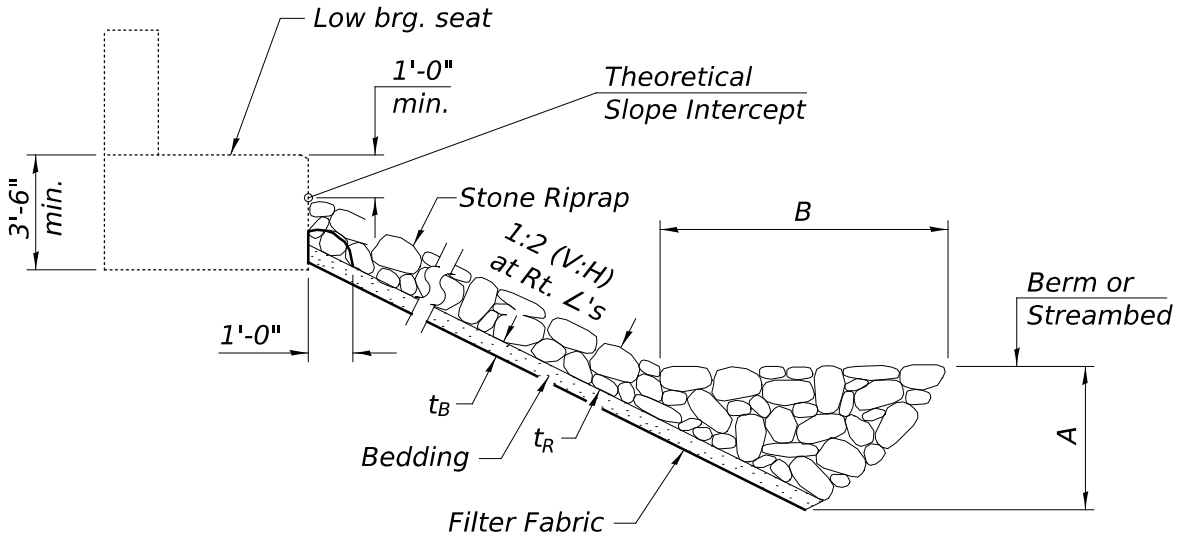


Figure 2.3.6.3.3-2

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Riprap Class	t_R	t_B	A	B	Min. Abutment Cap Height
A4	16"	6"	4'	8'	3'-6"
A5	22"	8"	5'	10'	4'-0"
A6	26"	10"	6'	12'	4'-6"
A7	30"	12"	7'	14'	5'-0"

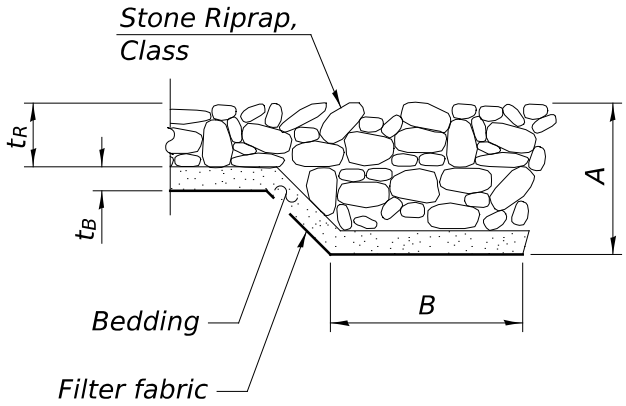


TOE STONE RIPRAP TREATMENT STREAM CROSSINGS

Figure 2.3.6.3.3-3

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Riprap Class	t_R	t_B	A	B
A4	16''	6''	2'-8''	4'-0''
A5	22''	8''	3'-8''	5'-6''
A6	26''	10''	4'-4''	6'-6''
A7	30''	12''	5'-0''	7'-6''



FLANK STONE RIPRAP TREATMENT FOR STREAM CROSSINGS

Figure 2.3.6.3.3-4

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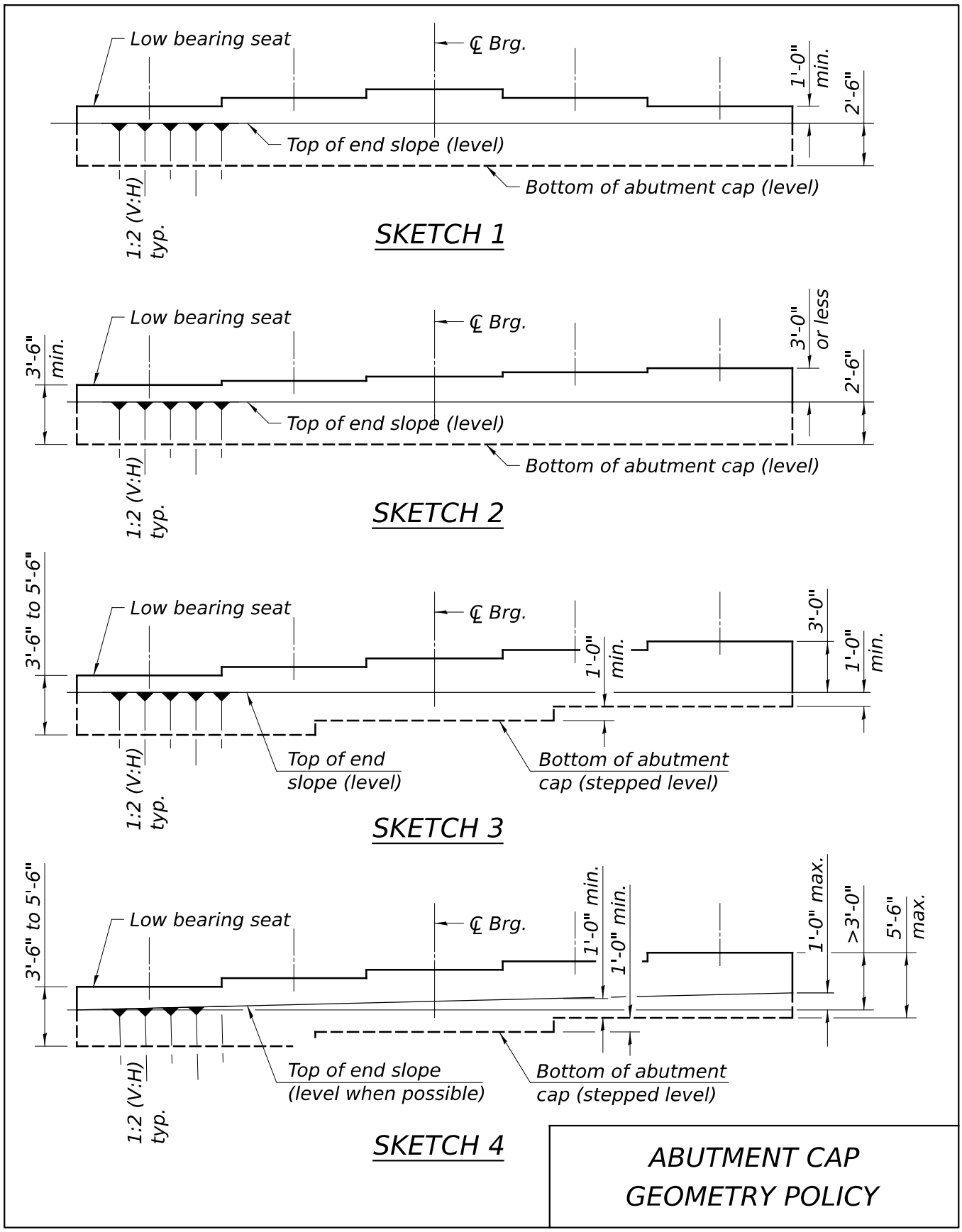


Figure 2.3.6.3.3-5

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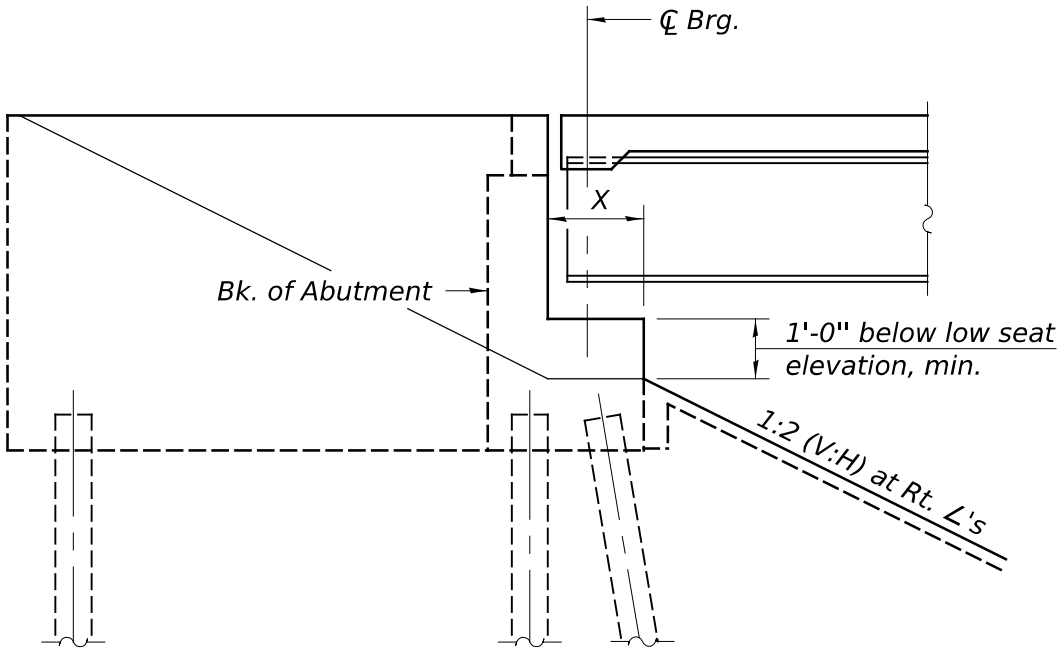


FIGURE 1

Berm width equal to width of seat.
For X ≤ seat width

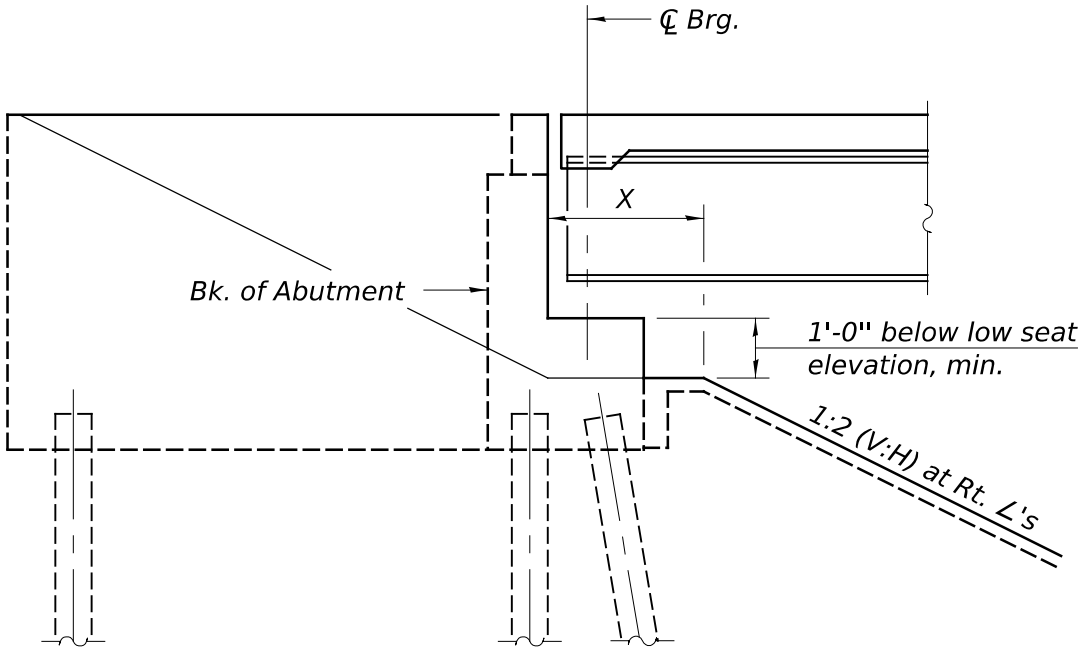


FIGURE 2

Wider berm required for site conditions.
X ≥ seat width

BERM WIDTH ELEVATIONS

Figure 2.3.6.3.3-6

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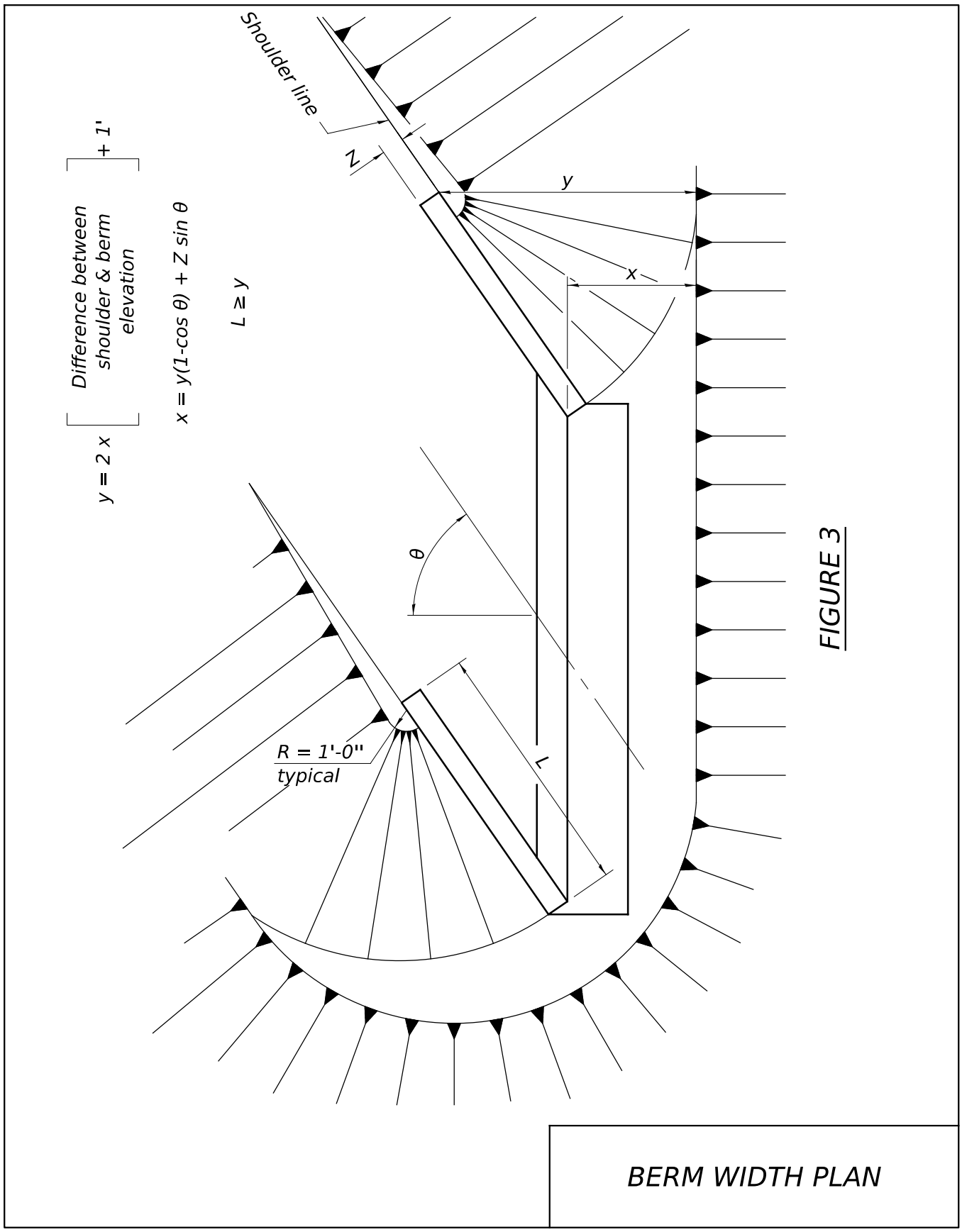


Figure 2.3.6.3.3-7

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2.3.6.4 Temporary Construction Works

2.3.6.4.1 Temporary Sheet Piling and Temporary Soil Retention Systems

During the planning phase, it is important to evaluate the likely temporary excavation slopes necessary to complete the construction being considered. When these excavations extend beyond State ROW, encroach on traffic lanes or other infrastructure, a temporary soil support system of some type will generally be required. The Structure Geotechnical Report will evaluate and identify the proposed temporary construction slopes as being unstable for the soil type/strengths present and recommend some retention or slope flattening. The engineer responsible for the TSL plan preparation should evaluate the added cost of installing temporary retention systems vs. using other substructure types/locations to verify that the most cost-effective structure type, with any necessary temporary retention, is shown on the TSL. It is desirable to explore whether a simple cantilever sheeting piling design may be feasible or if a more elaborate and expensive temporary soil retention system design might be required as this may affect the cost comparison between different structure configurations. IDOT normally provides temporary sheet piling designs in the contract documents using the design charts and methods provided in the "Temporary Sheet Piling Design" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design. At locations where the simplified charts do not work, the pay item "Temporary Soil Retention System" is utilized which allows the contractor to evaluate the exposed retention surface area and heights (provided in the contract documents) and propose a cost-effective wall system design during construction. The TSL plan need only show the locations of temporary sheet piling or a temporary soil retention system and should not show a full design. The final retention area/heights and temporary sheet piling design evaluation are completed during the Final plans phase.

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2.3.6.4.2 Cofferdams

Most structural concrete for substructures should be built in dry conditions, especially those with reinforcement congestion which makes constructability and construction inspection difficult. Dewatering is typically achieved by the use of cofferdams. When the Estimated Water Surface Elevation (EWSE) indicates water is expected above the bottom of the footing or encasement but below the existing ground line, a cofferdam will not be required unless soil conditions exist where reasonable pumping efforts cannot keep the excavation dry. When the EWSE indicates water is expected to be above the ground surface at the substructure location, a cofferdam shall be used. Locations with six feet or less of water above the bottom of the encasement or footing will typically require a Type 1 Cofferdam. Locations with greater than six feet of water will require a Type 2 Cofferdam. The exceptions to this policy are when:

- column-web wall drilled shaft bent piers are used,
- drilled shafts have permanent casing and/or removable forms,
- when individually encased HP pile bents are proposed,
- or where an approved coating for a metal shell pile is used.

See BM Section 2.3.6.2.2. The SGR provides recommendations concerning the need for cofferdams and seal coats. The EWSE is a key value used by both the geotechnical and structural engineer to determine the need and design requirements for cofferdams and seal coats. It is also used to select the most appropriate pier type for the expected construction conditions. The EWSE value is typically determined by a simple procedure described below. When Type 2 Cofferdams are necessary, the Cofferdam Design Water Elevation (CDWE) shall be specified in the Contract plans. The CDWE is 3 ft. above the EWSE. If the foundation soils require the use of a seal coat, the seal thickness design (either the initial designed thickness placed on the Contract plans or a redesigned thickness by the contractor) shall be based on the CDWE. See Section 3.13.3 for more information on these two types of cofferdams. The use of permanent casing on individual column drilled shaft bent piers and transfer beam drilled shaft bent piers, extending to 1 ft. above the EWSE can be used without a cofferdam or seal coat in waters of most any depth.

Many bridge sites will be located in controlled pools, especially on major rivers, where the normal pool elevation established by the United States Corps of Engineers or other agencies will be readily available. Other sites will be located at or near a United States Geological Survey stream gage station, which may be a source of data for determining the EWSE. A controlled pool elevation, gage data or any other information pertinent to EWSE determination should normally be contained within the Hydraulic Report. However, many sites will require an estimate based on hydraulic site surveys. In this case a standard method of finding the EWSE is presented below:

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1. From Hydraulic Report stream survey, find the *existing water surface elevation*, as provided per the IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) 2-602.02 & Fig. 2-602.02 b, (or low flow) at the bridge site and the month that this elevation was surveyed. Also, find the top of bank elevation from the stream cross sections at the bridge.
2. The *existing water surface elevation* is assumed to be a "typical low flow", in any year, for the month taken. April is assumed to be the typical "high" month for water surface elevations and September is assumed to be the typical "low" month. The following table may be used to adjust the *existing water surface elevation* to the month of April.

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
+1.5	+1.5	+0.75	0	+0.75	+1.5	+2.25	+3.0	+3.75	+3.0	+2.25	+1.5

3. The maximum elevation to be used is 75% of the difference from the typical September low flow elevation to the top of bank elevation added to the September low flow elevation. The September elevation may be assumed to be 3.75 feet below the April elevation but not lower than one foot above the streambed elevation.
4. The Estimated Water Surface Elevation is the lower elevation from step 2 or step 3.
5. The EWSE calculated from this procedure should be tested for reasonableness and the District Hydraulic Engineer and/or the District Bridge Maintenance Engineer should be consulted if there is any question about the validity of the elevation.

The following provides an example EWSE computation:

Step 1: Collect Data

From Hydraulic Report stream cross section or profile:

Existing water surface elevation = 606.1 at bridge site

Top of bank elevation = 611.3 at bridge site

Streambed elevation = 602.2 at bridge site

Month of survey is November

Step 2: Adjust existing water surface elevation to an assumed April Value

606.1 + 2.25 = 608.35

Step 3: Check maximum water elevation

Assumed September elevation: 608.35 – 3.75 = 604.6

One foot above streambed elevation: 602.2 + 1.0 = 603.2

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604.6 > 603.2, therefore use 604.6 as September elevation

75% of difference between September elevation and top of bank elevation
0.75(611.3 – 604.6) + 604.6 = 609.6

Step 4: Select preliminary EWSE

608.35 < 609.6, therefore use EWSE = 608.35

Step 5: Verify with District if this calculated value is reasonable

The engineer responsible for the TSL plan should consider the added expense of using cofferdams as this may affect the cost comparison between different structure configurations. For more information on cofferdams and seal coats refer to Section 3.13.3.

2.3.6.4.3 Temporary MSE Walls and Temporary Geotextile Retaining Walls

In fill construction, sheet piling may not provide the most cost effective retention system. Sheeting in taller fill retention applications can result in excessive deflections caused by the higher than active compaction efforts against the wall which may not be desirable. IDOT has found the use of temporary MSE or temporary geotextile walls to be cost effective in many fill conditions. For large retention heights, critical retention applications (such as high ADT, Interstate, tight staging alignment, etc.) and/or a relatively large quantity of wall surface area, temporary MSE walls are recommended. These walls are designed and contracted similar to “temporary soil retention systems” in that the retention surface area is provided in the Final plans and the contractor provides a design from a qualified MSE vendor. Smaller retention applications can utilize a temporary geotextile wall system, designed and provided to the contractor in the Final plans. Common applications include stage construction fills, fill retention on top of and adjacent to box culverts, and fill retention where the foundation soils or rock will not allow the penetration of sheet piling. The TSL plan will normally call out either temporary MSE wall or temporary geotextile wall when they are determined to be cost effective and when they are to be further developed in the Final plans phase. The SGR should provide recommendations on the use and feasibility (bearing pressure, settlement, etc.) of using these systems. For more information on the design and plan requirements for these walls, refer to Section 3.13.2.

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2.3.7 Bridge Geometry and Layout

Bridge geometric policy is the application of highway geometric design policies to the configuration of bridges, and generally defines the relationship between the physical limits of a structure, the supported roadway and the obstruction or obstructions bridged.

Since good bridge geometric design is intrinsic to the development of aesthetic, economic and safe structures, the following policies have been developed to facilitate the preparation of TSL plans along these lines.

Any deviation from these policies shall receive prior approval from the Engineer of Bridges & Structures.

2.3.7.1 Overall Length

Overall length of a bridge is generally determined by required horizontal and vertical clearances. For grade separation bridges, the minimum horizontal and vertical clearances for each roadway classification may be found in Chapters 38, 39, and 44 through 50 of the Bureau of Design & Environment Manual available at the primary Highways Page. For bridges over stream crossings, the minimum freeboard requirement and the approved waterway opening can be obtained from the Hydraulic Report. For bridges over navigable waterways requiring a permit from the United States Coast Guard (USCG), in addition to the Hydraulic Report, other clearance requirements may be obtained from the USCG publication "Bridge Permit Application Guide." The minimum vertical clearance required by IDOT policy for bridges over waterways is 2 ft. For bridges over railroads, the minimum horizontal and vertical clearances may be found in Chapter 39 of the Bureau of Design & Environment Manual.

2.3.7.1.1 Horizontal Clearance

The minimum horizontal clearance shall be provided from any obstruction such as piers, abutments, MSE walls, etc. for the safety of the traveling public. The minimum horizontal clearance is defined as the clear horizontal distance from the edge of pavement to the face of pier or abutment. Reduced horizontal clearances may be provided; however, all reduced clearances shall be economically justified with barrier protection provided and subject to approval by the District and BDE and, if Federally funded, the FHWA. Note that the vehicle collision force provisions discussed in Section 3.9 shall not be applicable to the base of MSE walls at grade separations as discussed in AASHTO LRFD Bridge Design Specifications Article 3.6.5. Therefore, an additional barrier is not required by the Department in front of the MSE for these situations.

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2.3.7.1.2 Vertical Clearance

Vertical clearance is defined as the clear vertical distance between the low superstructure and the highest roadway elevation (including shoulders), the design natural high water elevation, or the highest railroad track elevation (9 ft. from each side of centerline). Typically, at stream crossings structures, this determination is made at the abutment. However, for longer bridges or for bridges on substantial vertical grade where the beam elevation may vary by several feet over the length of the bridge, the vertical clearance may be over the span of the stream channel (when needed for multi-span bridges). Please refer to the IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) for further reference. The reference location of the PVC and the actual minimum vertical dimension shall be shown on all TSL plans.

For reconstruction/rehabilitation projects over streams, with substandard minimum vertical clearance/freeboard and the established profile grades remain unchanged, the District shall secure a policy waiver from the Bureau of Bridges & Structures. Each waiver of vertical clearance/freeboard will require that the District Hydraulic Engineer provide proper justification and documentation and when needed submit to the Bureau of Bridges & Structures Hydraulics Unit. For projects where the District Hydraulic Engineer has approval authority for the Hydraulic Report, the District has the authority to determine if a policy waiver should be made to the clearance/freeboard criteria. In that instance, the District is still required to justify and document the waiver, but BBS approval of the waiver is not required.

2.3.7.2 Bridge Width

Rural bridge width on a rehabilitation or reconstruction project is required to be addressed in the BCR and is a function of traffic, design speed, existing roadway features and the proposed roadway improvement. It should be verified that the bridge width shown on the TSL plan follows that recommended by the BCR. If there is no BCR, detailed guidelines on required bridge widths can be found in Chapters 39 and 44 through 50 of the Bureau of Design & Environment Manual available at the primary Highways Page. Urban bridge widths for rehabilitation/reconstruction projects generally match the approach roadway template.

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2.3.7.3 Skew Angle

The relationship between two or more intersecting elements (skew) of a roadway shall be shown on all TSL plans. See Appendix A1.3 regarding approved TSLs available online for proper examples and application of this requirement.

The accuracy of the skew angle required to accommodate either stream crossings, roadways or railroads shall be limited to the nearest second with the exception of standard bridges which have been developed utilizing skew increments of 5 degrees. Bridges over waterways are typically skewed to better align the waterway opening with the stream channel at the upstream face of structures. The Hydraulic Report provides a skew angle that best accommodates flood conditions, however this recommendation is subject to refinement during TSL plan development.

For bridge types with limited allowable skew such as structures with integral abutments per Section 2.3.6.2 (where skew angle ≤ 45 degrees), the skew angle of the substructures does not necessarily need to match the roadway or stream crossing skew. Contact the Bureau of Bridges & Structures if the planned skew angles between substructures and roadway or stream crossing skew differ by more than 10° for a particular project.

Wrap-around MSE abutments contain limitations on skew angles at their acute corners. See Section 3.8.9.5 for discussion and figures.

2.3.7.4 Cross Slopes

2.3.7.4.1 Tangent Sections

Figure 2.3.7.4.1-1 indicates the deck cross slopes for structures with various combinations of lanes and medians. These slopes are appropriate for all new bridge superstructures. Cross slopes for deck replacement projects should be considered on an individual basis to avoid excessive fillets and undesirable additional dead loads.

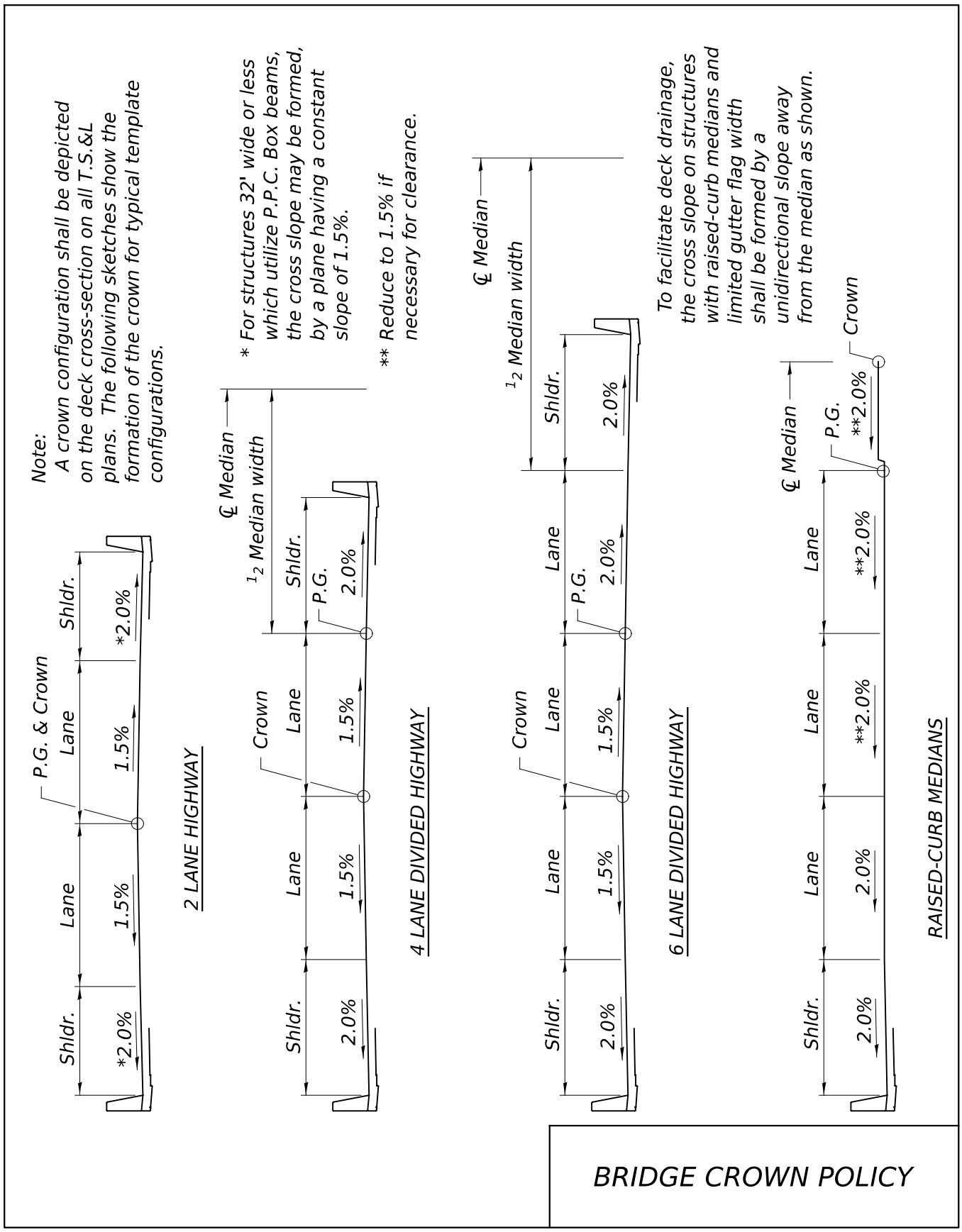


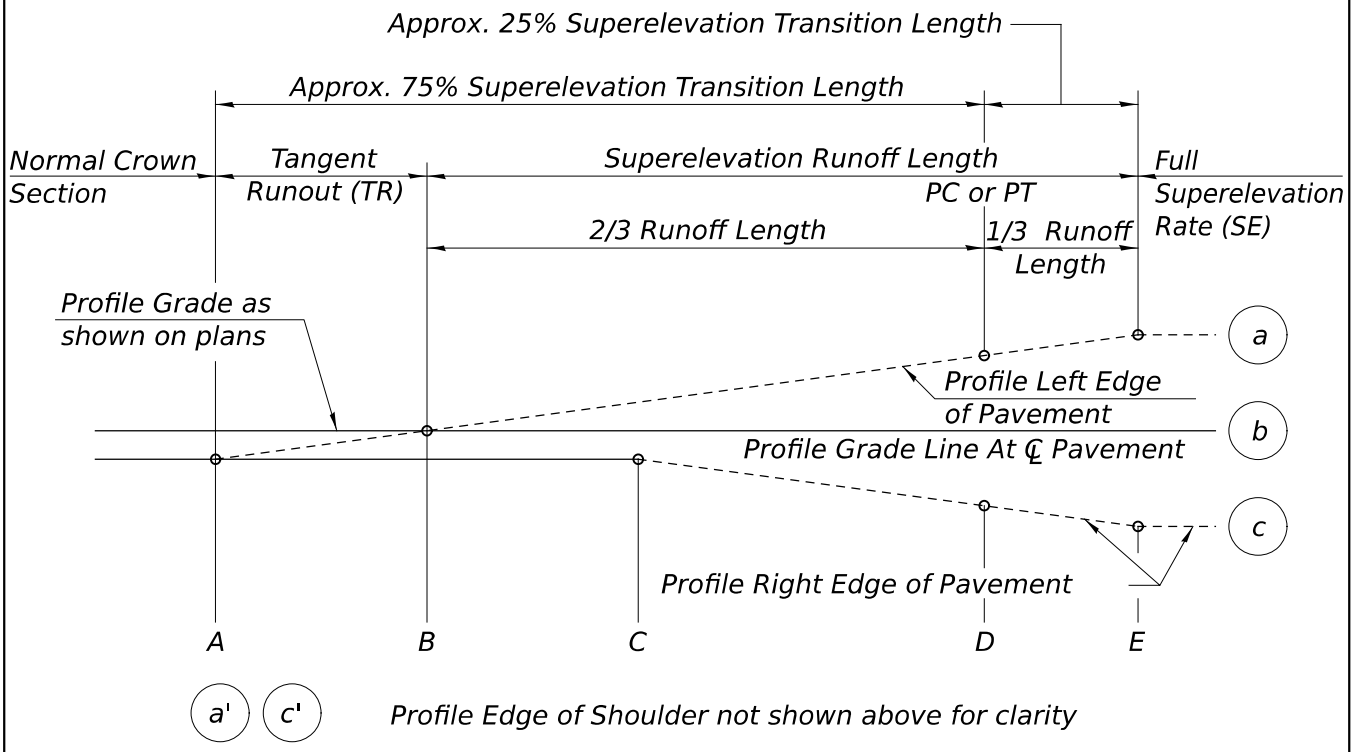
Figure 2.3.7.4.1-1

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2.3.7.4.2 Superelevation Development

The approved procedure for developing superelevation is shown in Figure 2.3.7.4.2-1 and 2.3.7.4.2-2. The layout of a structure located within a horizontally curved section of highway is shown in Appendix A1.3 via approved TSLs. The appropriate offset treatment is described in Section 2.3.7.6.

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Note:
See Chapter 32 of the BDE Manual.

METHOD OF ATTAINING SUPERELEVATION (LOOKING AT RT. ∠'S TO ROADWAY)

Figure 2.3.7.4.2-1

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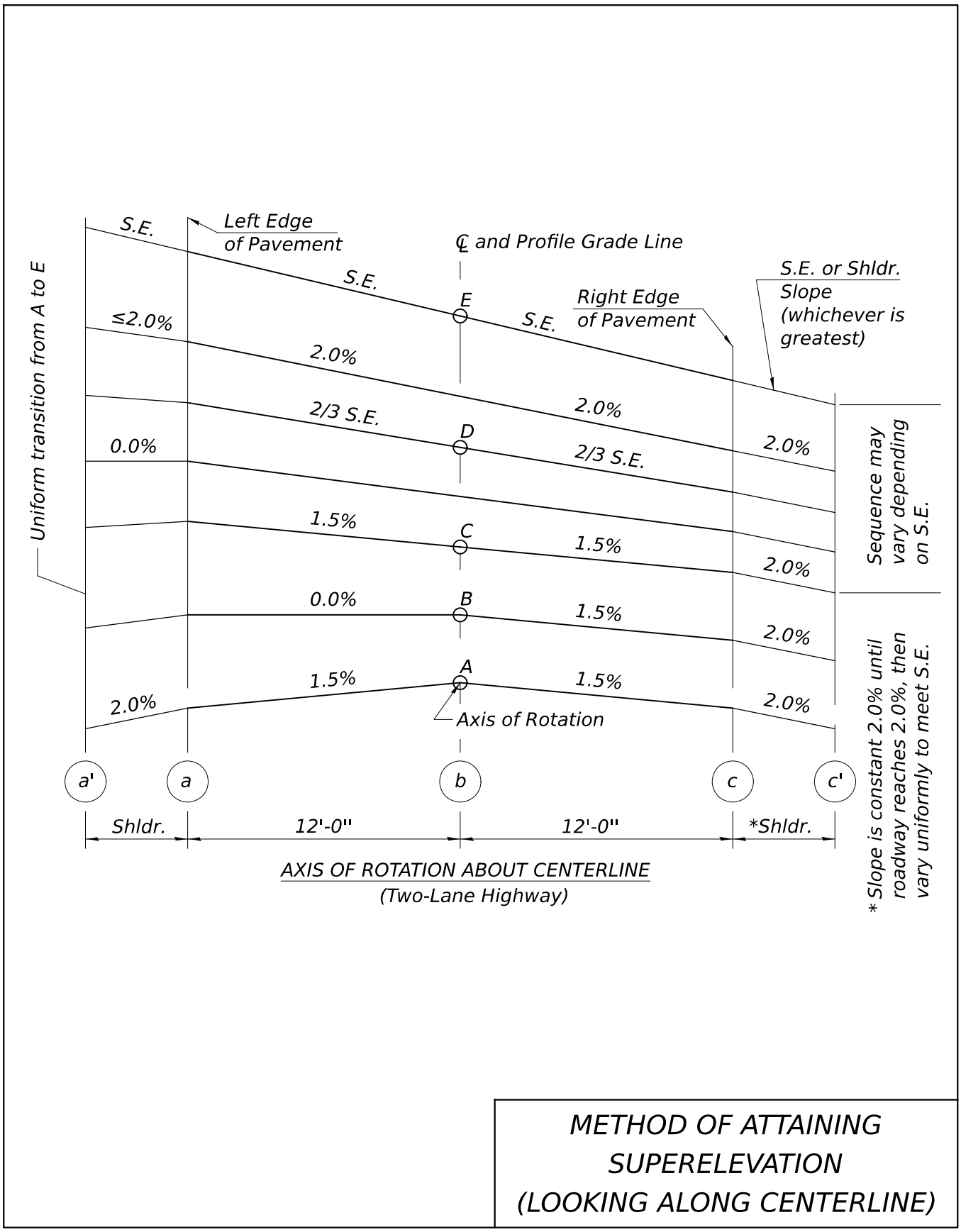


Figure 2.3.7.4.2-2

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2.3.7.5 Sidewalks and Bikeways

The general procedure for new construction of sidewalks and bikeways on bridges is to slope the surface transversely away from concrete parapets. This will avoid the need for any surface drainage through concrete parapets. The typical cross slope is 1.67%. See Base Sheets R-28, R-29, R-32, and R-33 in the "Railings" IDOT CADD Bridge Cell Library and Section 3.2 for detailed configurations.

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2.3.7.6 Curved Alignment

Bridges located on horizontally curved alignments present special problems in layout, design and construction. Because of this, the effect of curvature on a bridge should receive careful consideration in the planning stages to assure a problem free structure that is economically and structurally justifiable. An increase in the degree of curvature increases the magnitude of torsional forces which results in a reduction in the direct bending capacity (stress) of a beam. Other factors affecting the stresses that should be accounted for are uplift for sharply skewed structures, stiffness analysis and effect of forces on shear center.

To reduce cost, bridges along curved roadway alignments should be straight where practical. The following treatments are recommended when straight bridges along curved roadway alignments are practical:

- Split the offset for bridges with offsets between 1 ft. and 2 ft. (from abutments). Interstate bridges must provide the minimum shoulder width for the entire length of the structure (i.e., no split offset).
- Widen bridges when the offset is ≤ 3 ft. (from abutments).
- For offsets ≥ 3 ft, perform an economic evaluation and shoulder transition study and consult the Bureau of Bridges & Structures with the proposed structural layout.

For straight bridges with curved roadway alignments, the girders shall be straight, and overhangs shall be constant, except when there is limited space between dual structures. For that specific case, the girders should be straight; however, the overhangs should vary.

Straight bridges with a curved roadway may be constructed with a crown or superelevation. The crown is typically placed at the centerline of roadway which follows the stripped curved alignment. Optionally, the crown (and cross slope changes) may be constructed parallel to the deck edge or tangent line, which does not follow the stripped curved alignment. Placing the crown parallel to the tangent line or along the centerline of roadway should be evaluated based on ease of constructability, possible driving issues, stage construction and difficulty in computing the bridge elevations. A superelevated structure does not have a similar issue.

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2.3.7.7 Traffic Barriers

Traffic Barrier Terminals, Type 6 and 6A are crash tested and approved by the FHWA for connection of Steel Plate Beam Guardrail to the bridge approach or ends of bridges. These terminals shall be implemented on applicable projects. The Type 6 (Highway Standard 631031) is required at all entrance ends for constant slope parapets, and where applicable at the exit ends. For some cases, Type 5 (Highway Standard 631026) may be warranted at exit ends.

Bridge superstructure parapets or steel railings shall be extended 15 feet onto approaches. This 15 foot parapet or steel railing continuation requirement may be omitted for special cases, such as sight distance requirements for adjacent roads. For those cases, the bridge approach slab curb shall extend for the entirety of the slab or straight bridges on curved roadway alignments to minimize the bridge width increase and to avoid a possible kink in the traffic barrier terminal connection. For cases where the parapet is omitted from the approach, use a curbed approach the entire length of the approach.

Details of special treatments for bridges with sidewalks can be found in the "Details-Planning" IDOT CADD Bridge Cell Library.

For bridges with expansion joints, the standard parallel wingwall as shown in Section 3.8.5 shall be utilized. The foundation support for the wingwalls (i.e. piles, drilled shafts or spread footings) shall normally be the same as that for the abutment. Some structures with expansion joints may have stub abutments with dog-ear wingwalls in which case the wingwall is typically moved 6 in. toward the face of the abutment.

2.3.7.8 Bridge Deck Grinding and Smoothness Criteria

At the request of a District, a bridge project may have special grinding and smoothness criteria for the deck and approach slab. See also Section 3.1.9 and Section 3.2.1.2 for additional information.

> Smoothness grinding of PCC pavement has been used successfully throughout the state and some Districts have extended the merits of this to profile and smoothness grinding of bridge decks also. A recent IDOT/FHWA process review on bridge deck construction recognized the improved ride quality resulting from smoothness grinding of bridge decks and formed a committee to further study it and develop a consistent policy throughout the state.

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Bridge decks are currently finished according to Article 503.16 of the Standard Specifications and the Supplemental Specifications. These requirements seek to achieve quality concrete with a smooth riding surface. However, excessive finishing tends to create an overworked and weak laitance surface and also delays curing, all of which can adversely affect the riding surface and the concrete quality.

Therefore, the Department implements the "Planning Guidelines for Selecting Bridge Deck Smoothness Grinding" (I-VI on the following pages) to achieve a smoother riding surface on selected bridge decks while keeping in mind economics and time to project completion. It also improves the concrete quality of the bridge deck by separating the deck casting and curing process from the deck surface finishing process.

Smoothness and profile grinding is particularly beneficial on high speed, high volume routes where the smoother ride benefits derived by the traveling public offsets the additional cost of this work. Projects utilizing smoothness grinding for bridge decks shall be coordinated during the TSL phase with the District and the BBS. Projects utilizing smoothness grinding for concrete overlays shall be coordinated during repair plan development with the District and the BBS. Exemptions from this policy shall be coordinated through the District and the BBS. An additional "Design Guidelines for Smoothness Grinding of Bridge Decks" in Section 3.2 assists the designer in properly preparing bridge plans for structures selected for smoothness grinding.

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Guidelines for Selecting Bridge Deck Smoothness Grinding

I. Decisions regarding implementation of Bridge Deck Smoothness Grinding shall be made in the Bridge Planning (TSL) phase of the project. Exemptions shall be coordinated through the District and the Bureau of Bridges & Structures.

II. Bridge Deck Smoothness Grinding may only be considered for bridges with new concrete decks, slabs, or new concrete overlays.

III. Required Bridge Deck Smoothness Grinding Applications:

1. Interstate (mainline) bridges satisfying both of the following criteria:
   a. Bridge length ≥ 150'
   b. ADT (current) > 10,000

IV. Recommended Bridge Deck Smoothness Grinding Applications:

1. All State route bridges satisfying all of the following criteria:
   a. Bridge length ≥ 150'
   b. ADT (current) > 10,000
   c. Posted Speed >45 MPH
2. Interstate ramp structures with ADT (current) > 10,000
3. As further defined by District Policy

V. Grooving Applications:

All bridge decks with smoothness grinding shall be longitudinally grooved.

VI. Applicable Special Provisions

GBSP 59 "Diamond Grinding and Surface Testing Bridge Sections"
GBSP 78 "Bridge Deck Construction"
GBSP 79 "Bridge Deck Grooving (Longitudinal)"

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GBSP 78 permits elimination of several bridge deck finishing requirements for structures where smoothness grinding is specified.

GBSP 59 provides equipment and construction requirements and defines a bridge section for grinding as the bridge deck plus the approach slabs and the connector pavements. The grinding shall be diminished over the length of the connector pavement to provide a smooth transition to the roadway pavement. The grinding machine geometry, the cutting speed, and the down pressure maintain the profile grade while smoothing the transition between high and low spots. The grinding process may remove up to ¼″ of concrete and therefore the bridge deck, approach slabs and connector pavements shall be constructed ¼″ thicker on projects selected for smoothness grinding. The contractor is required to satisfy a minimum average smoothness of 25.0 in./mile or less per lane and this is recorded in the BC 2450 form in GBSP 59.

Projects selected for bridge deck smoothness grinding will be longitudinally grooved, in lieu of the typical transverse grooving. Longitudinal grooving provides a quieter ride and it is appropriate to incorporate it with the better ride quality provided by smoothness grinding. Longitudinal grooving also requires fewer passes than transverse grooving which saves time and will be less likely to reduce the improvement of smoothness grinding. GBSP 79 shall be included for this work.

2.3.8 Maintenance of Traffic

When staged construction has been determined to be the most cost effective alternate to provide for traffic flow during the reconstruction process, staging sequences shall be shown on the TSL plan. Several online approved TSLs (see Appendix A1.3) are available for examples which illustrate typical staging plans. The deck stage construction joint should be located within the center half of the slab span between beams/girders. Where a wide-load detour is not available, the minimum lane width for a single lane staged roadway shall be 14 ft. – 0 in. If a separate wide-load detour is provided, a minimum lane width of 10 ft. – 0 in. may be provided. The minimum lane width for multiple lane widths shall be provided in increments of 10 ft. – 0 in. Each of the above lane widths should be considered as minimums and additional width should be provided whenever practical. The recommended lane width is 12 ft. – 0 in.

To separate traffic from construction areas during staging, a temporary concrete barrier shall be provided when it can be safely supported by the existing structure. See Base Sheet R-27 in the "Railings" IDOT CADD Bridge Cell Library for the appropriate details.

All stage traffic over deck-girder superstructures shall be supported by at least three girders. New deck-girder superstructures which may not be initially staged should consider the number and arrangement of

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girders in order to provide at least three girders for possible future staging. This requirement may be waived if traffic can be detoured during future reconstruction or if approval is obtained from the Bureau of Bridges & Structures. Special attention should be given to stage construction of concrete bridge decks on longer span structures when large deflections or cambers may cause construction problems in the final deck pour. Alternate beam sections or a third stage closure pour should be considered when differential dead load deflections of 2 in. or larger are anticipated along a stage construction line.

2.3.9 Hydraulic Issues

In addition to all criteria in Section 2.3.6.3.2, all structure replacements over waterways shall meet the applicable regulatory criteria established by the Illinois Department of Natural Resources Office of Water Resources (IDNR-OWR) Floodway Construction Program. Similarly, projects over navigable waterways shall satisfy requirements of the United States Coast Guard. Please refer to the Chapter 1 of IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) for more detail on these two regulatory agencies and their project requirements. The approved TSL plan is a part of the documentation required by both of these agencies.

2.3.9.1 IDNR-OWR

IDNR-OWR jurisdiction includes all IDOT roadway crossings of watersheds over 1.0 square mile in an urban or urbanizing location. For crossings that are not considered urban or urbanizing, IDNR-OWR jurisdiction includes all crossings with watersheds over 10.0 square miles. The appropriate IDNR-OWR floodway construction permit is either a statewide permit issued in-house by IDOT acting as the agent of IDNR-OWR, or a formal application is made to IDNR-OWR for an individual permit. In the first case, the approved TSL plan is part of the permit documentation. In the latter case, the approved TSL plan accompanies the submittal package from the Bureau of Bridges & Structures Hydraulics Unit to IDNR-OWR.

It is important to note that IDNR-OWR floodway construction permits are granted primarily on the basis of a single criteria, the structure's backwater impact upon the properties that constitute the upstream floodplain. Backwater or created head at a given crossing typically relates most directly to the overall length of the bridge\size of the culvert; i.e., the waterway opening that the structure provides. Consequently, IDNR-OWR does not approve or comment upon hydraulic design features such as beam clearance, pier location, number of culvert cells or scour countermeasures that do not relate directly to backwater impact. The applicant assumes responsibility for sound hydraulic design. In particular, the structural planner should be aware that IDNR-OWR permit issuance is not tied to or contingent upon meeting the 2 ft. low beam clearance or the 3 ft. roadway freeboard policy criteria.

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2.3.9.2 Permit Sketches

As part of the preparation of plans for stream crossing structures, sketches shall be prepared for submittal to the agencies having jurisdiction over the involved waterways. The IDOT Drainage Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Hydraulics) provides additional guidance to the general requirements below.

The Illinois Department of Natural Resources Office of Water Resources requires the submittal of Waterway Sketches and Channel Change Sketches. Samples of these sketches are illustrated in the Drainage Manual.

As applicable, names of waterways shall be shown in the title block of TSL and Final Design plans. See the Drainage Manual Appendix for a list of public waters.

The U.S. Coast Guard requires permit sketches when navigable waters are involved. Figure 2.3.9.2-1 through Figure 2.3.9.2-5 illustrates the proper presentation and requirements to be followed in the preparation of these drawings.

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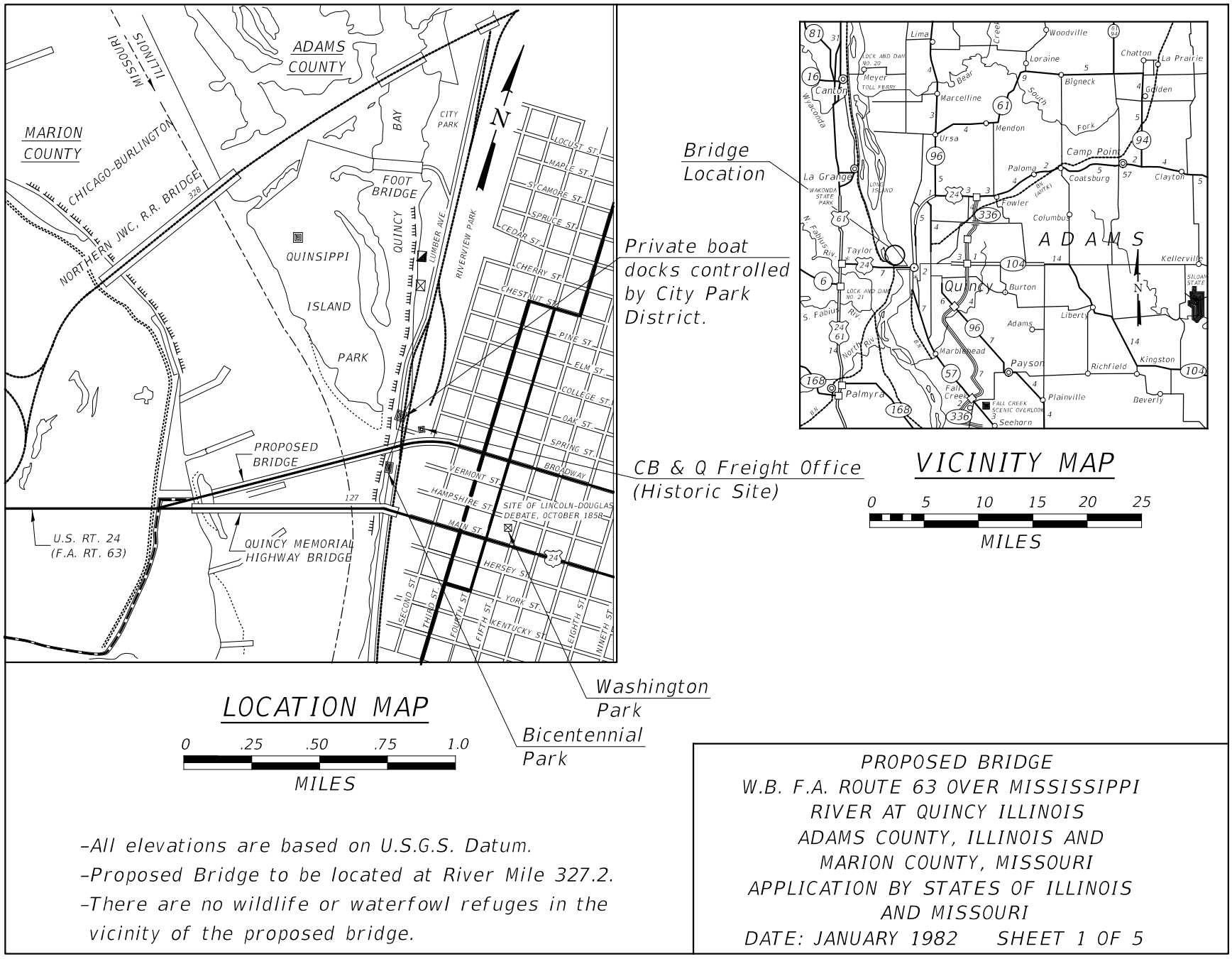


Figure 2.3.9.2-1

-All elevations are based on U.S.G.S. Datum.
-Proposed Bridge to be located at River Mile 327.2.
-There are no wildlife or waterfowl refuges in the vicinity of the proposed bridge.

PROPOSED BRIDGE
W.B. F.A. ROUTE 63 OVER MISSISSIPPI RIVER AT QUINCY ILLINOIS
ADAMS COUNTY, ILLINOIS AND MARION COUNTY, MISSOURI
APPLICATION BY STATES OF ILLINOIS AND MISSOURI
DATE: JANUARY 1982 SHEET 1 OF 5

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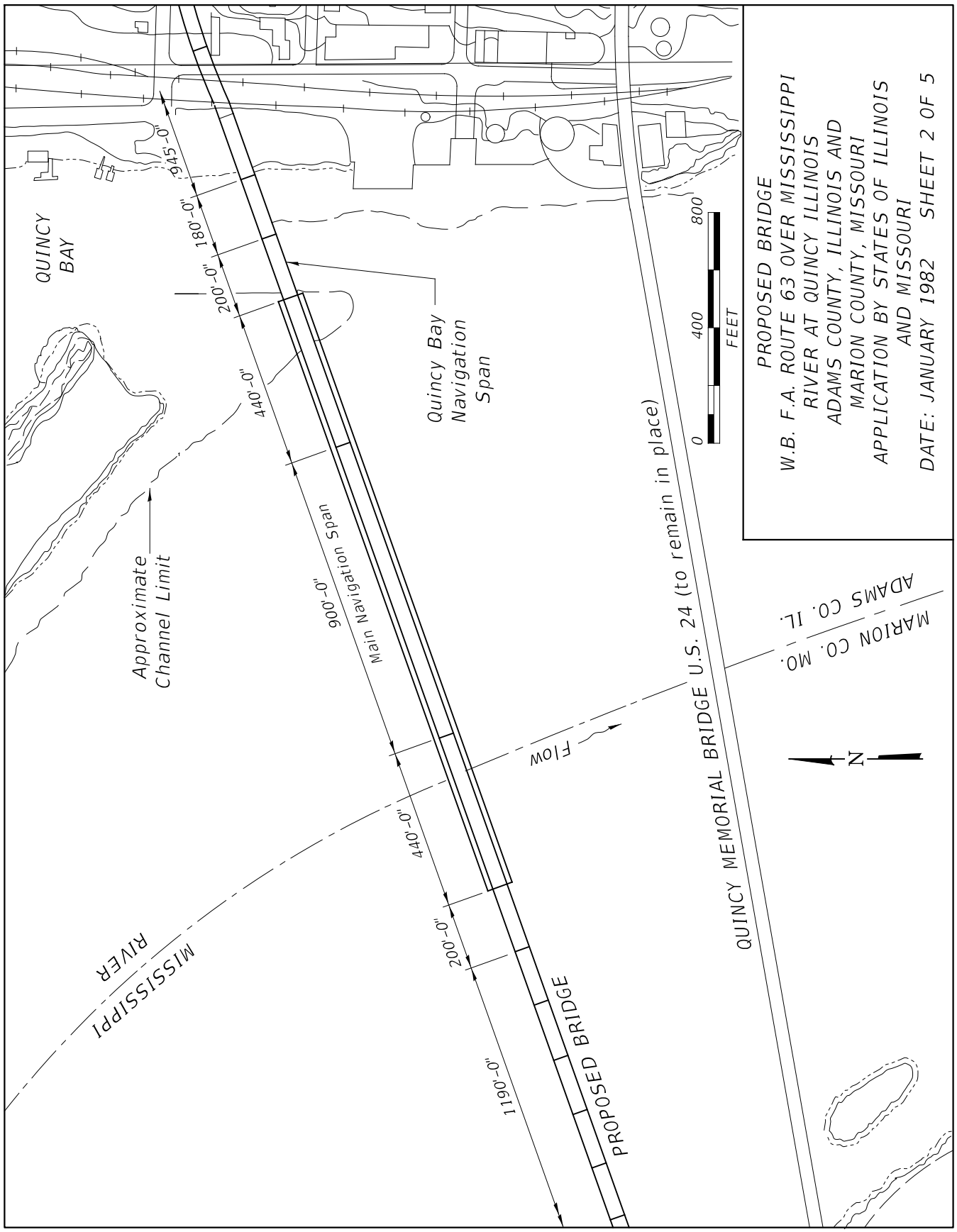


Figure 2.3.9.2-2

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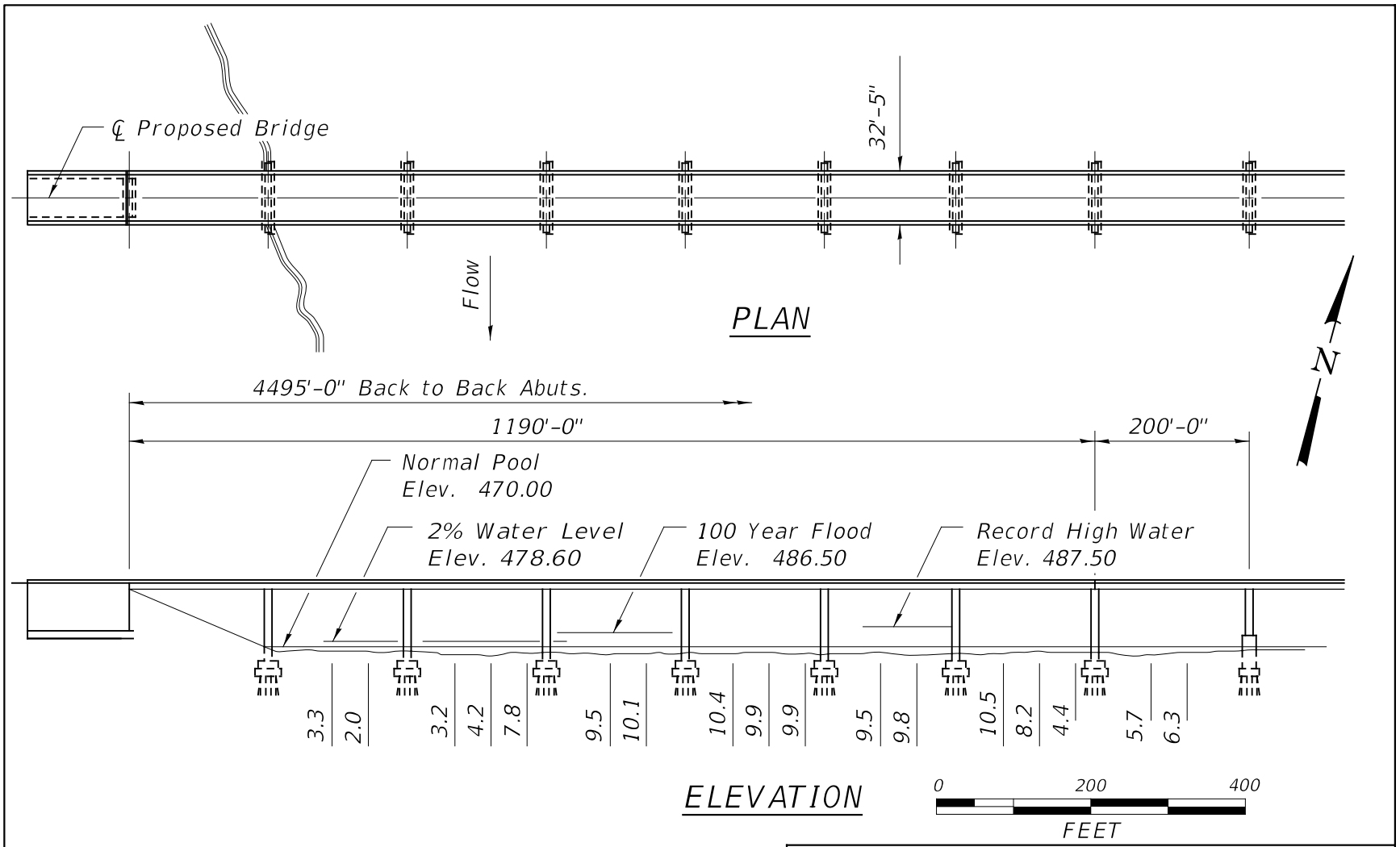


Figure 2.3.9.2-3

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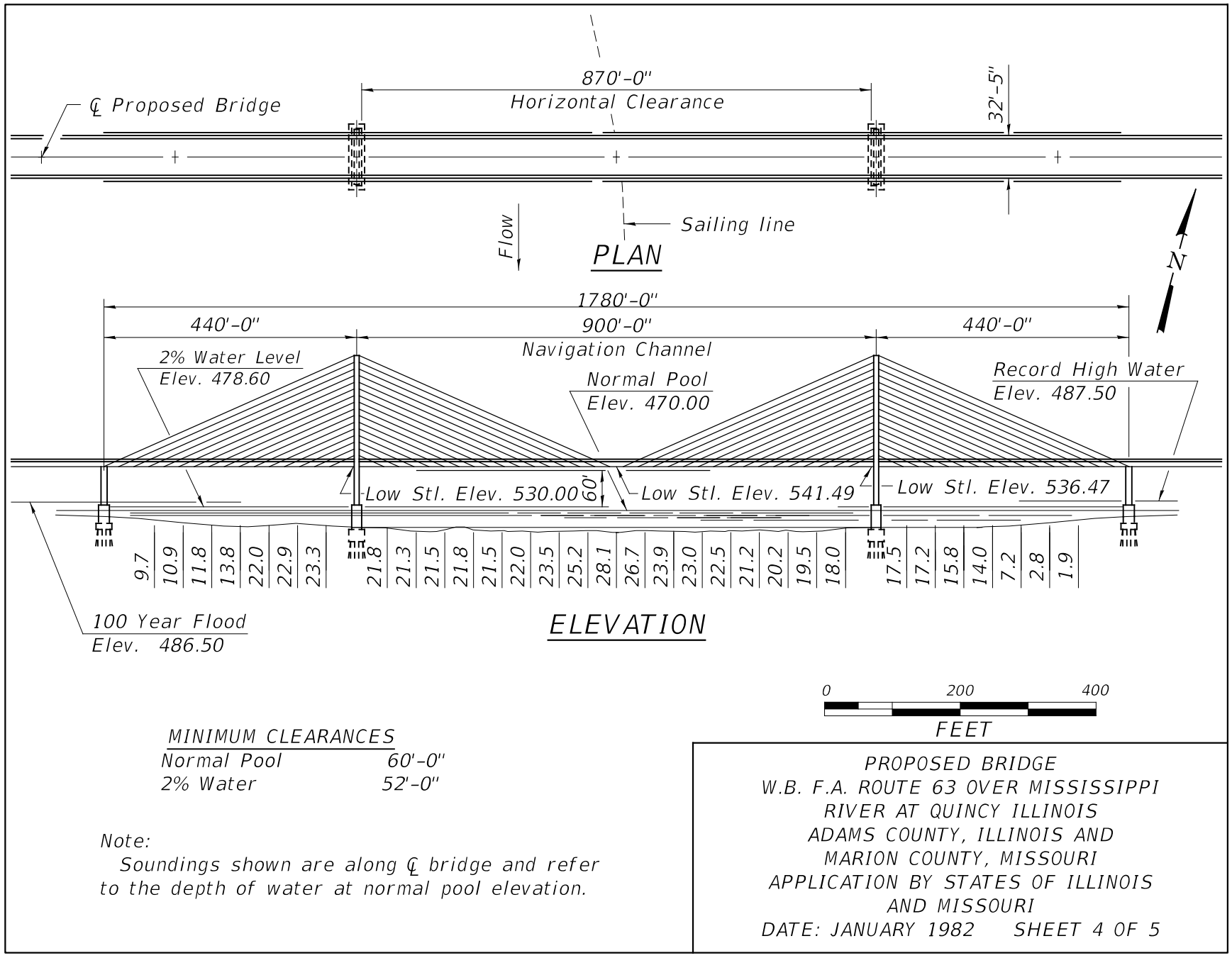


Figure 2.3.9.2-4

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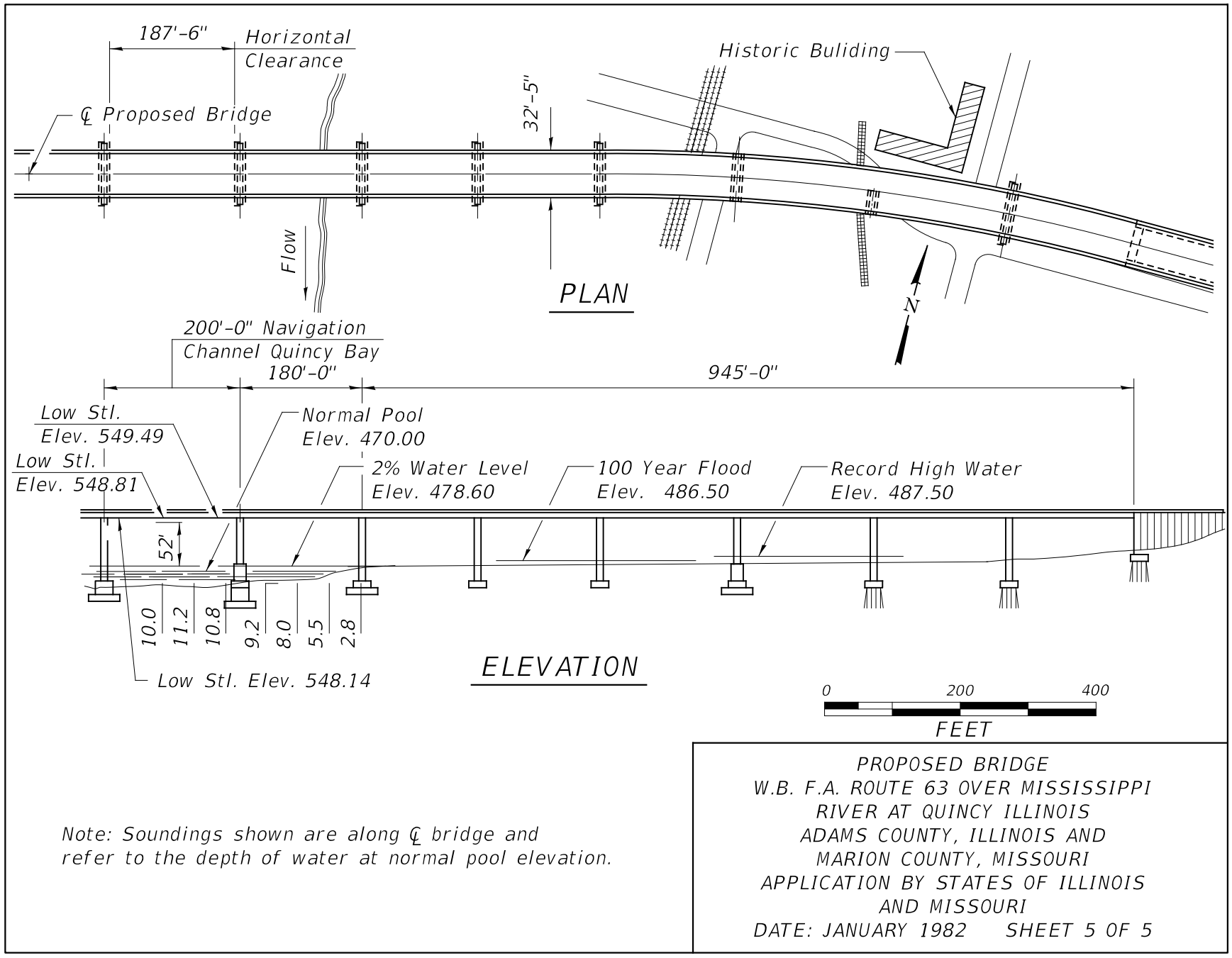


Figure 2.3.9.2-5

Note: Soundings shown are along ℄ bridge and refer to the depth of water at normal pool elevation.

PROPOSED BRIDGE
W.B. F.A. ROUTE 63 OVER MISSISSIPPI RIVER AT QUINCY ILLINOIS
ADAMS COUNTY, ILLINOIS AND MARION COUNTY, MISSOURI
APPLICATION BY STATES OF ILLINOIS AND MISSOURI
DATE: JANUARY 1982 SHEET 5 OF 5

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2.3.9.3 Bridges Over Navigable Waterways

It is the responsibility of the Engineer of Bridges & Structures to obtain from the Commandant, United States Coast Guard (USCG), a permit approving the location and plans for the construction or approval for alteration of any bridge on the State highway system over certain navigable waterways. Alteration in this context means any work that would permanently alter the navigation clearances.

Requirements for navigation lights and vertical clearance gages are established by the USCG and become conditions of the permit.

A USCG Permit is required when a bridge crosses waters which are used or susceptible to use in the natural condition or by reasonable improvement as a means to transport interstate or foreign commerce. The determination of the need for a permit is made by the USCG. The following Table 2.3.9.3-1 lists those waterways that in the past have required permits under the foregoing definition.

U.S.CG. Permit Waterways

Eighth Coast Guard District - St. Louis, Missouri

Waterway	Upper Limit
Big Muddy River	Murphysboro, Illinois, Mile 37.5
Chain of Rocks Canal	In its entirety
Des Plaines River	Mile 291.1
Chicago Sanitary and Ship Canal	S. of 9th St. in Lockport
Illinois River	Confluence Kankakee and Des Plaines River, Mile 273.0
Kaskaskia River	Fayetteville, Illinois, Mile 36.2
Little Wabash River	Mile 39.7
Ohio River	In its entirety
Upper Mississippi River	In its entirety

Table 2.3.9.3-1

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Carr Creek	Mile 2.4
Fountain Creek	Mile 5.75
Massac Creek	Mile 2.2
Big Grande Pierre Creek	Mile 6.0
Mary's River	Mile 14.0
Round Springs	Mile 0.8
Quincy Bay	In its entirety
Chaney Creek	Mile 0.5
Grays Bay	Mile 0.4
Larry Creek	Mile 0.9
Sonora Creek	Mile 0.6
Waggoner Creek	Mile 0.7
Riley Creek	Mile 0.4

Ninth Coast Guard District - Cleveland, Ohio

Waterway	Upper Limit
Waukegan Harbor	In its entirety
Chicago River:	
Main Branch	In its entirety
North Branch and	
North Branch Canal	Mile 7.29 (Addison Street)
South Branch	In its entirety
South Fork of S. Branch	In its entirety
Chicago Sanitary and Ship Canal	N. of 9th St. in Lockport
Calumet - Sag Channel	In its entirety
Little Calumet River	Calumet - Sag Channel
Calumet River	In its entirety
Lake Calumet	In its entirety
Grande Calumet River	State line

Table 2.3.9.3-1 (Cont.)

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All Federally funded bridges over navigable waters which do not meet the above definition are exempt from the USCG permit process. The FHWA will make the exempt status determination in the early coordination phase of project development. Non-Federally funded bridge projects where the permit requirement is not apparent after an investigation into stream navigability shall be referred to the USCG for a permit requirement determination.

In the early stages of project development, the District shall consult with the Bureau of Bridges & Structures, who will assess the need for a Coast Guard permit. When a permit is required, the District should initiate coordination with the USCG at an early stage of project development and provide opportunity for the USCG to be involved throughout the environmental review process in accordance with Title 23 CFR, Part 771. The Bureau of Design & Environment should be consulted for coordination procedures and requirements.

2.3.10 Seismic Issues

The planning guidance herein shall be applicable to all structures being designed using an LRFD force-based design approach. When the Department incorporates alternate seismic design approaches, refer to the latest seismic memorandums and manuals found at the primary Bridges & Structures Page.

Displacement-Based Seismic Design

Refer to the IDOT Seismic Manual available at the primary Bridges & Structures Page.

Force-Based Seismic Design (when approved by Department)

Regions with moderate to high seismicity (generally about the Southern ½ to ⅓ of the State depending on soil conditions) require additional earthquake loading consideration for the design of new bridges and retrofitting of existing bridges. Regardless of region, seismic data shall be provided on TSL's for all structures except most walls and buried structures. Three sided precast concrete structures are considered buried structures. However, seismic data is required on the TSL in order to satisfy the detailing needs of the special provision. TSLs for retaining walls shall have seismic data only when the consequences of their failure during a seismic event could cause loss of life as determined by the Bureau of Bridges & Structures, or the Design Engineer of Record for Local Agency Projects.

The design earthquake return period in the AASHTO LRFD Bridge Design Specifications increased from 500 yrs. to 1000 yrs. beginning with the 2008 interims. Seismic Performance Zones (SPZ) in LRFD are analogous in the sense that they represent differing levels of accelerations and requirements a structure shall be designed for.

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The method for determining the design acceleration and SPZ for a structure also changed significantly in 2008. See Sections 3.15.2 of this Manual and Section 3 of the AASHTO LRFD Bridge Design Specifications for more information. In previous AASHTO codes, the SPZ was only a function of the horizontal bedrock acceleration coefficient at a period of zero seconds unmodified for soil conditions at a project site. Soil type or Site Class (A through F), and the Spectral Acceleration on rock at a period of 1.0 sec (S_1) are now employed to determine the SPZ. Figure 2.3.10-1 through Figure 2.3.10-4 indicates the extent of each SPZ, assuming various soil site classes found in Illinois which assist the engineer in estimating the SPZ for preliminary planning. The final SPZ shall be determined using the AASHTO LRFD Bridge Design Specifications, shown in the Structural Geotechnical Report (SGR), and documented on the TSL seismic data.

The potential for liquefaction shall be evaluated for all projects according to the requirements or their SPZ.

2.3.10.1 Seismic Design of New Bridges

Displacement-Based Seismic Design

Refer to the IDOT Seismic Manual available at the primary Bridges & Structures Page.

Force-Based Seismic Design (when approved by Department)

All new (non-major) bridge construction on the State system and, as applicable, new bridges on the Local System shall be designed for the 1000 yr. design return period seismic event according to the AASHTO LRFD Bridge Design Specifications and the Department's Earthquake Resisting System (ERS) strategy. A "flexible" approach for the 1000 yr. design return period seismic event using the LRFD Code and the Department's ERS strategy may also be permitted for some local bridges in primarily rural and/or low ADT areas. See Sections 3.7, 3.10, 3.15 and the "Seismic Design" Design Guide (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design) for more detailed information.

For significant or critical bridges, e.g. major river crossings, it is likely that a much longer design return period (2500 years) will be warranted along with more sophisticated design methods than those in the current AASHTO LRFD Bridge Design Specifications. The Bureau of Bridges & Structures will make the determination of applicable seismic design criteria for major bridges on a case-by-case basis.

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The selection of PPC I-beam and bulb T-beam superstructures for bridges in LRFD SPZ 3 and 4 should be carefully considered by the engineer responsible for the TSL plan. Due to their higher mass (and therefore increased seismic design forces) relative to bridges with steel beam superstructures, PPC I-beam and bulb T-beam superstructures may not be the optimal choice.

Piles in regions of high seismicity (LRFD SPZ 3 and 4) should not be battered. For bridges in LRFD SPZ 2, the specification of pile batter should be considered on a case-by-case basis.

2.3.10.2 Retrofitting of Existing Bridges

Displacement-Based Seismic Design

Refer to the IDOT Seismic Manual available at the primary Bridges & Structures Page.

Force-Based Seismic Design (when approved by Department)

All existing bridges on the state and local system which are undergoing rehabilitation (redecking, widening, superstructure replacement, etc.), or on occasion repair projects on existing non-LRFD substructures and foundations, should meet the requirements of the 1000 yr. design return period seismic event according to the FHWA Seismic Retrofitting Manual for Highway Structures (2006) and the Department's ERS strategy. For questions regarding specific interpretations of these documents, contact the Bureau of Bridges & Structures.

Significant retrofitting measures may include: adequate seat widths, "equivalent seat widths" through the use of longitudinal and/or transverse restraint devices, retrofitting columns and foundations, and isolation bearings. A minor overstress of the existing substructure for seismic loading may be considered on a case-by-case basis when in good condition. Factors which the engineer responsible for the BCR or TSL should consider include budgeted funds, ADT, bridge operational category (formerly "importance"), bridge condition, remaining life, and retrofit vs. replacement cost. For detailed guidelines regarding seismic evaluations of existing structures, please refer to the "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning.

Bridges along (or over) Earthquake Response Routes should be carefully evaluated during the planning phase for their operational category and condition. See the "Emergency Routes for Preparation of TSLs" guide (also known as Section VI of the IDOT Earthquake Preparedness and Response Plan) at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning. An easy detour around a bridge may lower its importance to life safety which should

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be coordinated by the district. On a project-by-project basis at the discretion of the BBS, significant or critical bridges along Earthquake Response Routes may be retrofitted according to the 1000 yr. return period. In these cases, the FHWA Seismic Retrofitting Manual for Highway Structures and the Department's ERS strategy should be used.

2.3.10.3 Seismic Data

When required, the following data shall be given on the TSL (Examples of how to fill in the data are shown in *[brackets]*:

Displacement-Based Seismic Design

Refer to the IDOT Seismic Manual available at the primary Bridges & Structures Page.

Force-Based Seismic Design (previous policy when approved by Department)

SEISMIC DATA
Seismic Performance Zone (SPZ) = _ [1, 2, 3, 4]
Design Spectral Acceleration at 1.0 sec. (S_{D1}) = _._ _ _ g [0.085g, etc.]
Design Spectral Acceleration at 0.2 sec. (S_{DS}) = _._ _ _ g [0.150g, etc.]
Soil Site Class = _ [A, B, C, D]

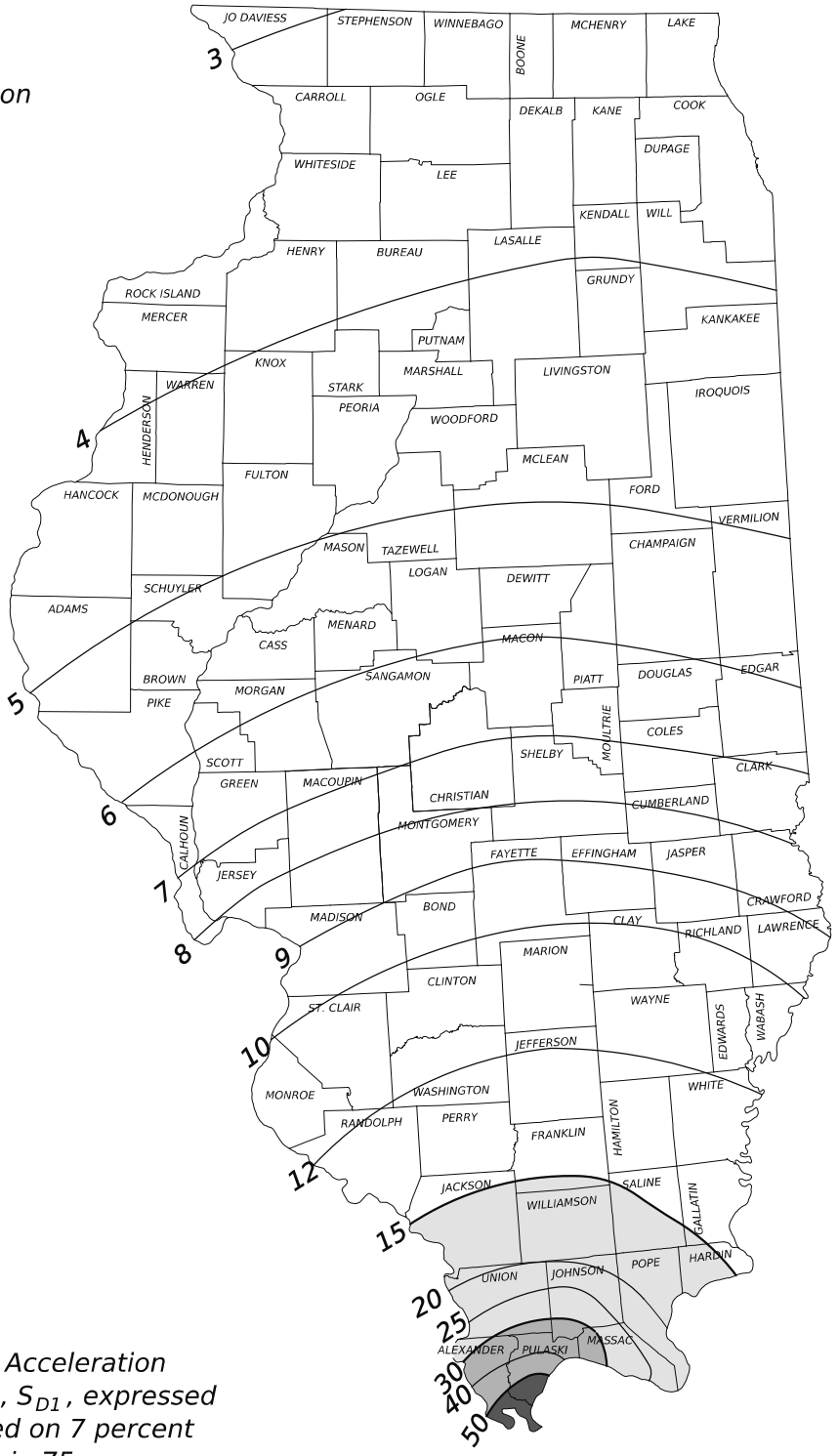
Retrofit Seismic Design

SEISMIC DATA
Seismic Retrofit Category (SRC) = _ [A, B, C, D]
Design Spectral Acceleration at 1.0 sec. (SD1) = _._ _ _ g [0.085g, etc.]
Design Spectral Acceleration at 0.2 sec. (SDS) = _._ _ _ g [0.150g, etc.]
Soil Site Class = _ [A, B, C, D]
Performance Level (PL) = PL_ [PL0, PL1, PL2, PL3]

See Appendix A1.3 for examples of seismic design data specification on approved TSL plans. See Section 3.15 for force-based seismic design and retrofit seismic designs, and "Bridge Condition Report Procedures and Practices" Section 3.7. For Displacement-Based Seismic design see the IDOT Seismic Manual.

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Note:
To be used for preliminary project scoping. Refine acceleration during TSL phase using procedures outlined in Section 3 of the AASHTO LRFD Bridge Design Specifications.



Map of Horizontal Ground Acceleration at a period (T) of 1.0 sec., S_{D1}, expressed as percent of gravity based on 7 percent probability of exceedance in 75 years. (1000 Year Return Period)

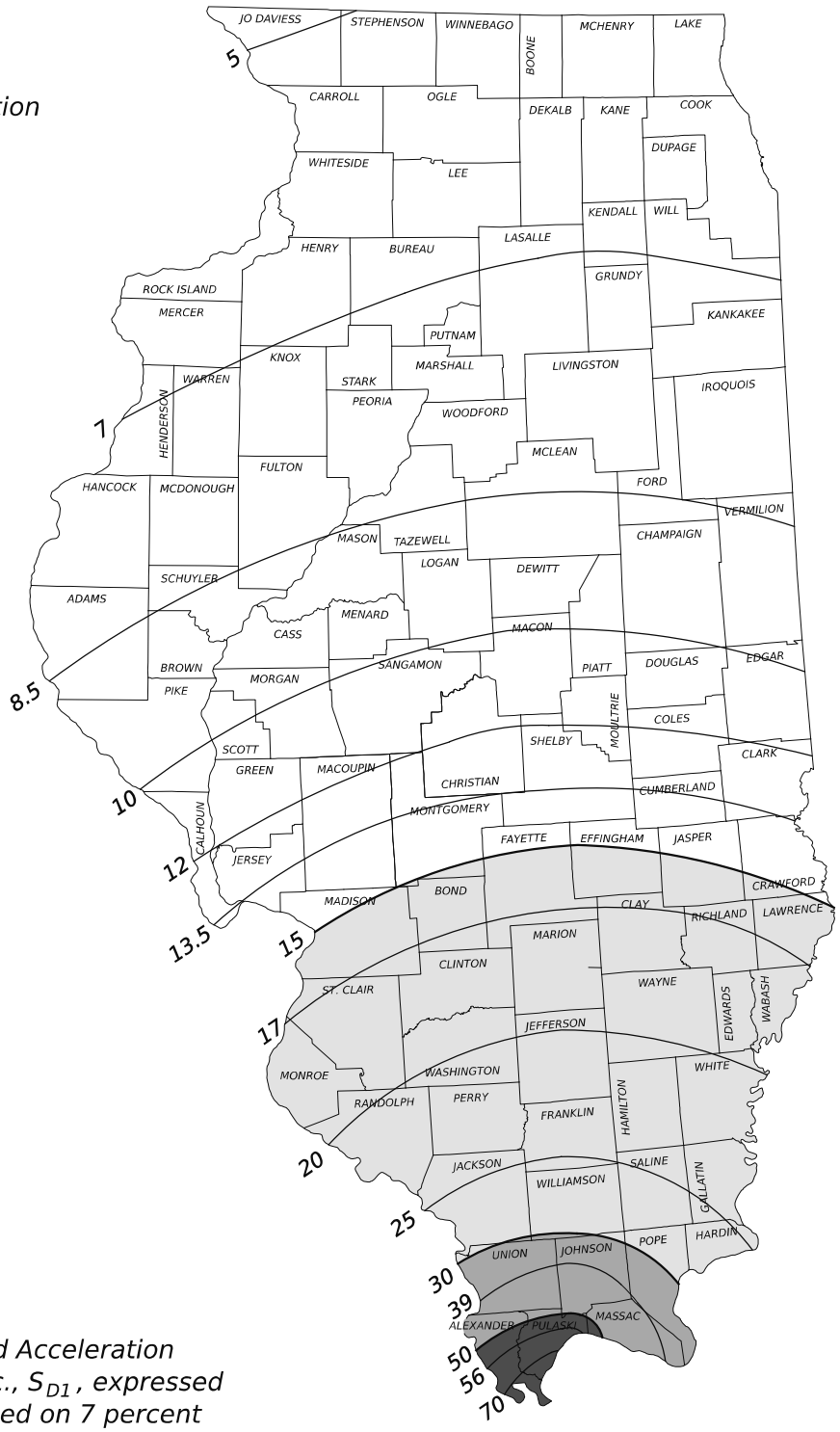
Based on the U.S. Geological Survey 2007 maps for the 2008 AASHTO LRFD Bridge Design Specifications.

SPECTRAL ACCELERATION AT T=1.0 SEC. (S_{D1}) FOR SITE CLASS B SOIL (LRFD)

Figure 2.3.10-1

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Note:
To be used for preliminary project scoping. Refine acceleration during TSL phase using procedures outlined in Section 3 of the AASHTO LRFD Bridge Design Specifications.



SPZ 1
SPZ 2
SPZ 3
SPZ 4

Map of Horizontal Ground Acceleration at a period (T) of 1.0 sec., S_{D1}, expressed as percent of gravity based on 7 percent probability of exceedance in 75 years. (1000 Year Return Period)

Based on the U.S. Geological Survey 2007 maps for the 2008 AASHTO LRFD Bridge Design Specifications.

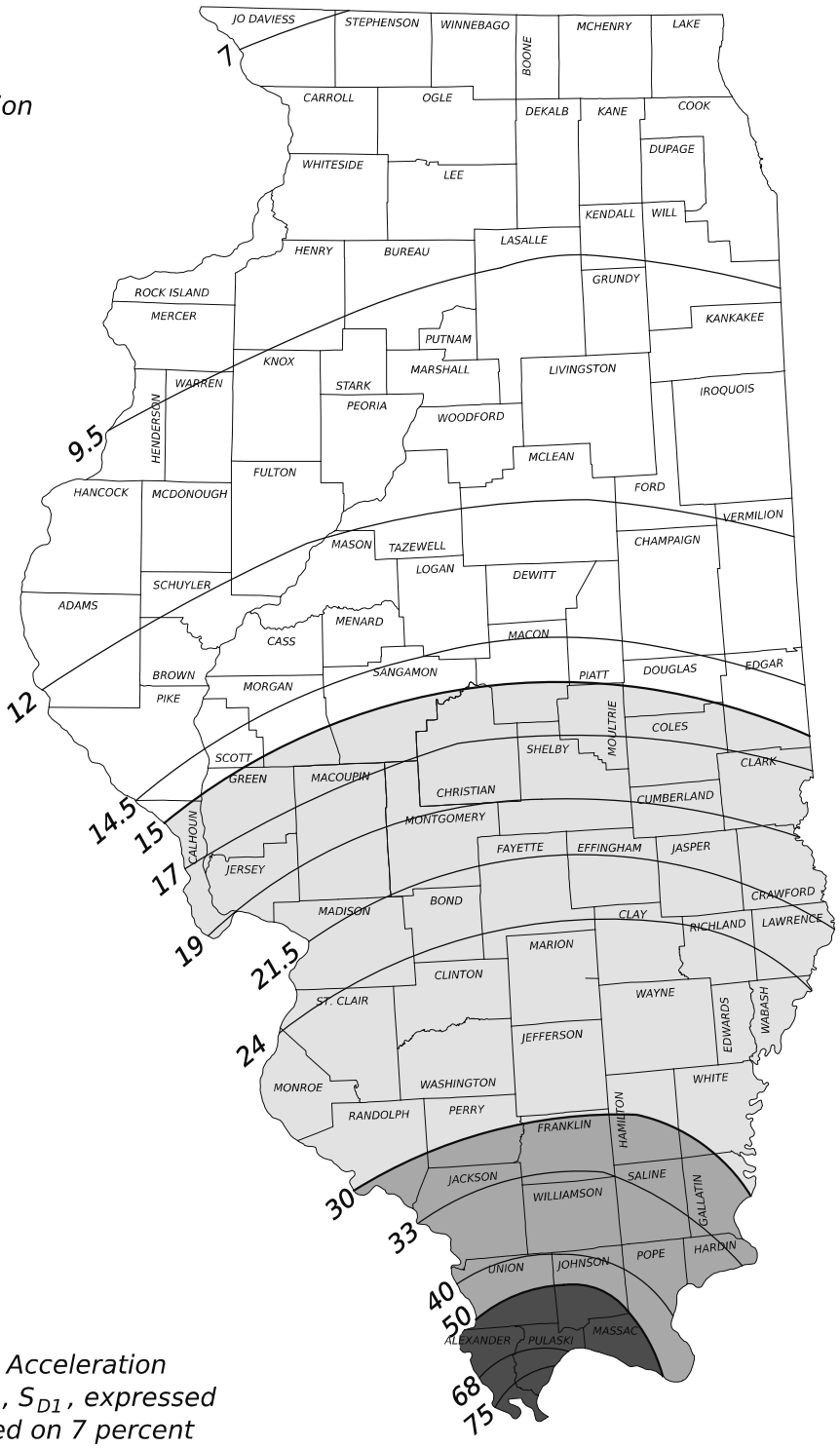
DENSE SOIL (N>50, Q_u >2^{tsf})

SPECTRAL ACCELERATION AT T=1.0 SEC. (S_{D1}) FOR SITE CLASS C SOIL (LRFD)

Figure 2.3.10-2

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Note:
To be used for preliminary project scoping. Refine acceleration during TSL phase using procedures outlined in Section 3 of the AASHTO LRFD Bridge Design Specifications.



SPZ 1
SPZ 2
SPZ 3
SPZ 4

Map of Horizontal Ground Acceleration at a period (T) of 1.0 sec., S_{D1}, expressed as percent of gravity based on 7 percent probability of exceedance in 75 years. (1000 Year Return Period)

Based on the U.S. Geological Survey 2007 maps for the 2008 AASHTO LRFD Bridge Design Specifications.

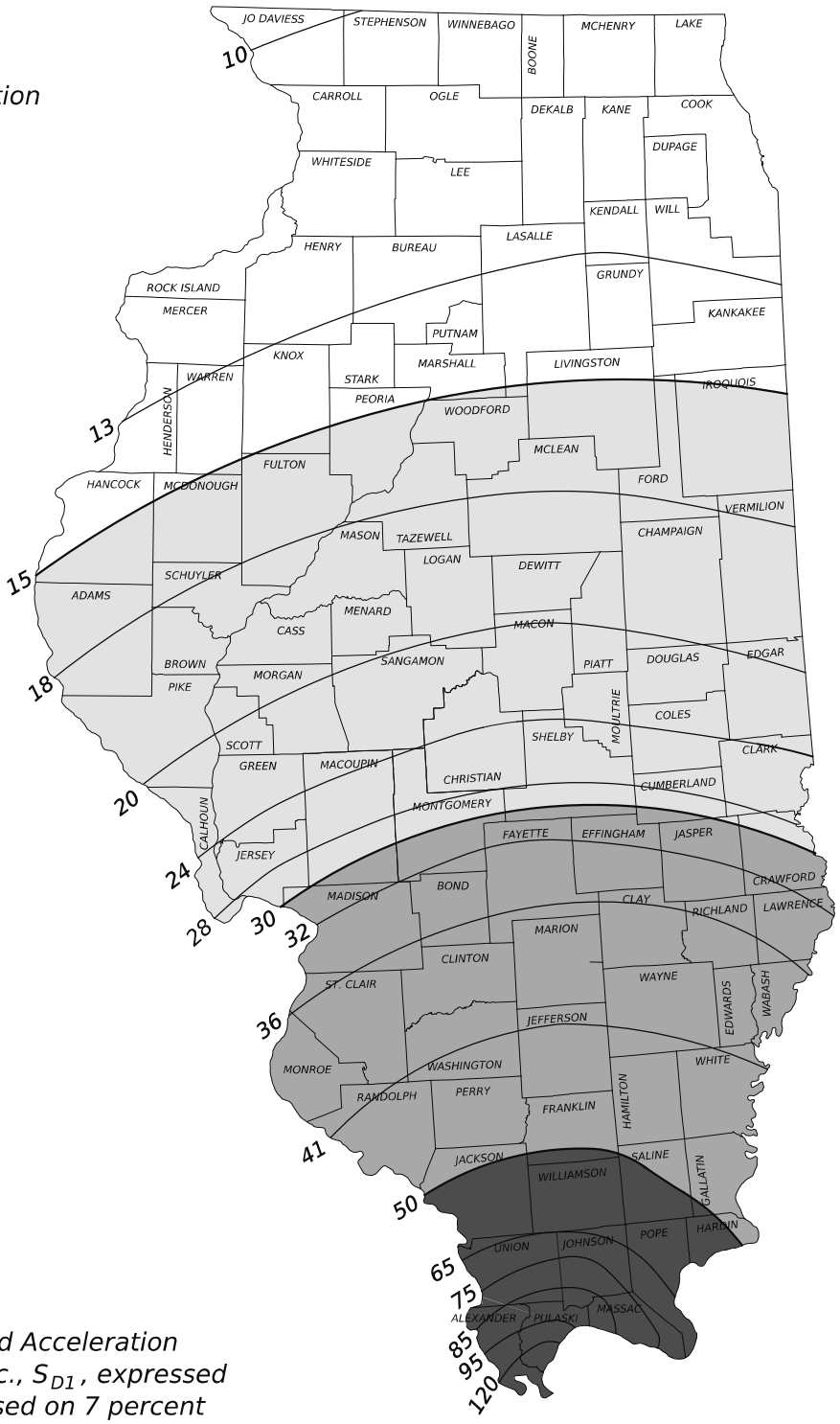
STIFF SOIL ($15<N<50$, $1^{tsf}<Q_u<2^{tsf}$)

SPECTRAL ACCELERATION AT T=1.0 SEC. (S_{D1}) FOR SITE CLASS D SOIL (LRFD)

Figure 2.3.10-3

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Note:
To be used for preliminary project scoping. Refine acceleration during TSL phase using procedures outlined in Section 3 of the AASHTO LRFD Bridge Design Specifications.



SPZ 1
SPZ 2
SPZ 3
SPZ 4

Map of Horizontal Ground Acceleration at a period (T) of 1.0 sec., S_{D1}, expressed as percent of gravity based on 7 percent probability of exceedance in 75 years. (1000 Year Return Period)

Based on the U.S. Geological Survey 2007 maps for the 2008 AASHTO LRFD Bridge Design Specifications.

SOFT OR LOOSE SOIL ($N<15$, $Q_u<1tsf$)

SPECTRAL ACCELERATION AT T=1.0 SEC. (S_{D1}) FOR SITE CLASS E SOIL (LRFD)

Figure 2.3.10-4

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2.3.11 Culvert and Three Sided Precast Concrete Structure Selection Process

The selection of whether a structure over a waterway should be a culvert, a three sided precast concrete structure or a bridge is heavily influenced by the hydraulic opening. As the hydraulic opening becomes larger, the selection process for structure type progresses from culvert to three sided precast concrete structure to bridge. Initial cost, future maintenance, profile grade, staging, skew, soil conditions and alignment are also important variables which should be considered. Culverts generally have low future maintenance; however, culverts should not be considered for waterways with a history or potential of debris to avoid channel cleanout maintenance. In these cases, a three sided precast concrete structure may be more appropriate. Three sided precast concrete structures have the advantage of larger single and multiple openings, ease of construction, and low future maintenance costs.

Precast culverts and three sided precast concrete structures are acceptable options for pedestrian tunnels. The joints shall be sealed and the barrel covered with a full waterproofing membrane system. To provide for drainage, geocomposite wall drains shall be used in lieu of weep holes.

2.3.11.1 Culverts

The plan preparation and structural design of cast-in-place (CIP) multiple cell box culverts remains with qualified consultants or the BBS in conformance with current plan development procedures. Additional details and guidelines can be found in the Culvert Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design) and the TSL checklist for culverts given in Section 2.3.13. However, precast multiple cell box culverts meeting the limitations described below may be undertaken by the Districts at their discretion. The development of Design plans for CIP single box culverts remains with the Districts with assistance from the Culvert Manual and the BBS. The option of a precast or a CIP multiple box culvert should be evaluated and determined prior to the TSL plan phase. If a CIP multiple box culvert is required or preferred by the District, a note on the TSL plan disallowing the precast option shall be provided.

The following guidelines are provided when using precast multiple cell box culverts:

2.3.11.1.1 General

Relatively small projects may not be economical if end sections, wingwalls/aprons are cast-in-place (CIP). For all culverts, especially short culverts (under 2 lane roadways), the following guidelines shall be considered:

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1. Use precast end sections whenever possible, subject to hydraulic acceptability.
2. Use CIP wingwalls with aprons when precast ends are not feasible.
3. Avoid using CIP end barrels with CIP wingwalls as much as possible.
4. For skewed culverts, lengthen the culvert as required (even if additional right-of-way is needed) to allow the use of precast end sections whenever possible.

2.3.11.1.2 Use Limitations

1. Cambering the box will not be allowed for precast concrete box culverts.
2. A minimum cover of 6 in., measured from the bottom of the pavement surface to the top of the top slab taken at the roadway edge, shall be provided for precast concrete box culverts. Cast-in-place (CIP) concrete box culverts may be used in zero-fill situations. In these cases, approach slabs shall also be used and connected to the box culvert with a 1'-0" corbel on the exterior culvert walls.
3. All headwalls for multiple cell precast concrete box culverts shall be collared around the end of the precast sections. Because of the size and weight of these units, it is anticipated that headwalls for multiple cell precast concrete box culverts will be CIP similar to the details shown in Figures 2.3.4-3 and 2.3.4-4 of the Culvert Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design). Individual precast end sections similar to those detailed in Section 2.3 of the Culvert Manual may be used if hydraulically acceptable.
4. Precast box culvert designs shall provide hydraulic equivalence to conventional CIP designs. This may occasionally require a larger precast culvert size to compensate for the additional inlet losses and the adjustment to standard sizes.
5. The use of multiple cell precast concrete box culverts is not recommended under the following conditions:
    a. Where high settlement could be anticipated.
    b. Where design flood velocity and stream bed soils raise concern for scour.
    c. Where clogging from debris or sedimentation is a concern.
6. The use of multiple cell precast concrete box culverts under the conditions listed below should be avoided. Consultation with the BBS before use is strongly recommended.
    a. For special designs such as when set directly on rock.
    b. Conditions where pile foundations would be required.
    c. Culverts along Interstate

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2.3.11.1.3 Processing Requirements

1. Consistent with current processing procedures for all bridge and multiple cell culvert projects, submittal of a BCR to the BBS for review and approval is required. The BCR for multiple cell precast concrete box culverts, will, however, be the tool with which the BBS documents its approval or disapproval of that structure type for a specific location. In order to make these determinations, structure boring data will be required during the BCR submittal.
2. The processing of TSL plans or Final plans for multiple cell precast concrete box culverts to the BBS for review and approval will not be required except for structures on the interstate system.

Installation, Method of Measurement and Basis of Payment for Precast Concrete Box Culverts are included in the Standard Specifications. The Standard Specifications also requires joints between units to be sealed to assure no embankment material is allowed to pass through.

The BBS is available to assist the District in resolving problems that may arise during plan development and clarifications of any questions relative to the interpretation of these requirements.

2.3.11.2 Three Sided Precast Concrete Structures

Three sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The ease and short duration of construction make them an attractive alternative which may be considered on certain projects. A TSL preparation checklist for three sided precast concrete structures is provided in Section 2.3.13.

Three sided precast concrete structures are proprietary systems where the primary structural unit is designed after the contract is awarded. There are several systems approved for use in Illinois the contractor may choose from and they may be found in GBSP 90. Each of these systems has unique design limitations detailed on their web sites and the planner should carefully consider these limitations when determining whether a three sided precast concrete structure would work on their project. The FHWA requires at least two independent systems capable of satisfying the project needs in order to utilize a three sided precast concrete system.

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2.3.11.2.1 Use Limitations

In addition to the requirements of GBSP 90, the following structure limitations shall be considered by the Planner when considering whether a three sided precast concrete structure is appropriate for a project.

1. Skew. A zero degree skew is preferable but skews may be accommodated in a variety of ways. Skews should be limited to 5 degree increments when practical. The range of skew is dependent on the design span and the fabrication limitations of each proprietary system. Some systems are capable of fabricating a skewed segment up to a maximum of 45 degrees. Other systems accommodate skew by fabricating a special trapezoidal segment. If adequate right-of-way is available, skewed projects may be built with all right angle segments provided the angle of the wingwalls are adjusted accordingly. The planner shall consider the lay out of the traffic lanes on staged construction projects when determining whether a particular three sided precast concrete structure system is suitable.
2. Span. The maximum clear span permitted by the Department is 60 feet measured at right angles from the inside face of sidewalls. If a railing is required to be attached to the structure:
   - an independent anchorage slab to isolate the railing loads from the three sided structure are required for (i) structures with spans greater than 42 feet (measured along the centerline of roadway) and (ii) fill depths requiring rail posts to be attached to the structure;
   - the engineer shall investigate whether all design requirements can be satisfied before specifying a large clear span.
3. Rise. The maximum rise of an individual segment is 13 feet. This limit is based on the fabrication forms and transportation. The maximum rise of the segment may also be limited by the combination of the skew involved because this affects transportation on the truck. Certain rise and skew combinations may still be possible but special permits may be required for transportation. The planner should verify this with each proprietary system under consideration. The overall rise of the three sided structure should not be a limitation when satisfying the opening requirements of the structure because the footing is permitted to extend above the ground to meet the bottom of the three sided segment.
4. Design Fill. The minimum design fill is equal to the summation of the pavement structure plus the cover, measured at the edge of pavement. The cover may be permitted to be zero. Approach slabs are not required.
5. Embedment. A 4 foot minimum embedment measured from bottom of footing to streambed elevation is required.
6. Waterproofing. The entire top surface and the top one foot of the side walls shall be waterproofed on structures with three feet of fill, or less to improve longevity of the structure. Structures which serve as pedestrian tunnels shall have the entire top surface and full height of side walls waterproofed, regardless of fill height.

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2.3.11.2.2 Planner/Designer Responsibilities

1. For each project, the above limitations and combinations thereof shall be verified through the approved manufacturers that are listed in the special provisions. In addition, the cost of any temporary soil retention system shall be included in the economic evaluation. Complex soil retention systems due to stage construction may negate the cost effectiveness of staging a three sided precast concrete structure.

2. Foundation borings and an SGR are required. See "Foundation Issues" in Section 2.3.

3. The actual design of the three sided precast concrete structure is the responsibility of the supplier. Shop Drawings for the three sided precast concrete structure sections and all other precast elements along with formal structural calculations, shall be submitted to the BBS for approval. Shop Drawings shall be certified by the supplier as being designed in accordance with the applicable AASHTO specifications. The supplier shall also indicate any additional backfilling requirements that shall be met beyond those found in the Standard Specifications and shall show the limits of those backfilling requirements.

4. Hydraulic and waterway opening requirements shall be handled similarly to any other project. A scour analysis shall be performed. See Sections 2.3.11.2.4 and 2.3.6.3.2, Table D for additional guidance. The Structural Engineer preparing the contract plans shall select Method 1, 2, or 3 (below) based on economic and other factors and show appropriate details on the plans. The Contractor shall have the option of using the method shown on the contract plans or revising the scour protection design to one of the other methods. If the Contractor chooses to deviate from the contract plans, the Contractor shall be responsible for the plan revisions, subject to review and approval by the Department (or by the Owner for Local Agency projects).

    - **Method 1** Place the footing below the check scour elevation, including contraction scour, local scour, and long term degradation. For foundations in scourable material, the top of the footing shall be at or below the check scour elevation. For foundations in non-scourable material, the footing shall be keyed into the non-scourable material. The distance between the faces of walls or the stub walls (if present) shall be used for the scour calculations. See Figure 2.3.11.2.2-1 for an illustration.

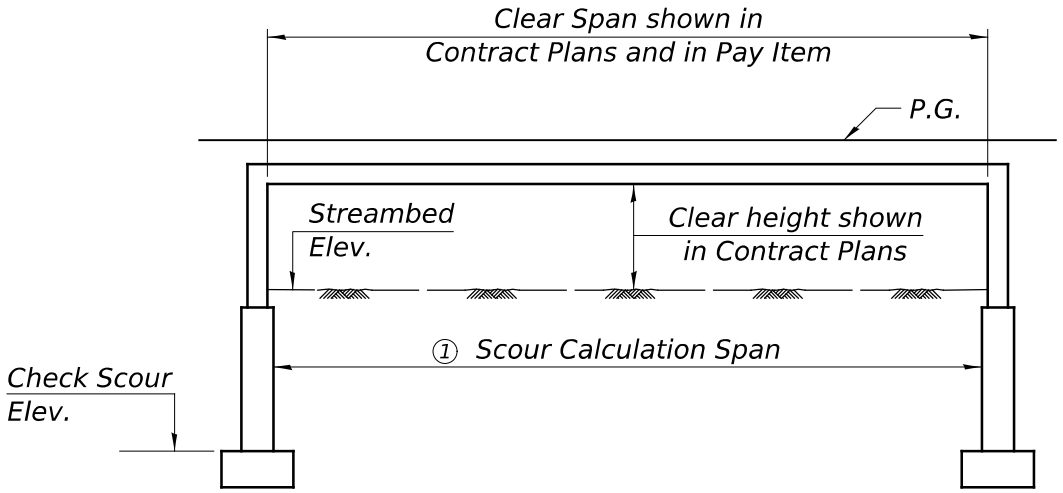
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- **Method 2** Protect and retain the soils below the footings with a sheet pile bulkhead. The bulkhead shall be designed by the Contractor, assuming all material on the stream side of the sheets has been removed to the design and/or check scour elevation shown on the contract plans. The distance between the faces of the bulkheads shall be used for the scour calculations. Wales and tiebacks may be needed in some cases and may intrude into this space. Struts spanning between sheeting faces to support the wales will not be allowed. The footing shall be positively attached to the sheets and any exposed soil between the sheets and the footing sealed with grout or cast in place (CIP) concrete. The bottom of footing shall be located as required by normal design considerations such as frost heave, bearing pressure, etc. See Figure 2.3.11.2.2-2 for an illustration.

- **Method 3** Allow the three sided structure to be treated as a closed bottom structure for scour evaluation by using a raft foundation or by installing a slab spanning the width of the opening and positively attached to the footings or walls. The top of the slab shall be a minimum of three inches below the streambed elevation, similar to culverts. Cutoff walls at each end of the bottom slab are also required. The bottom of footing elevation shall be the elevation required by normal design considerations such as frost heave, bearing pressure, etc. Only cast in place slabs or precast slabs with tongue in groove joints between individual segments will be considered to “close” the bottom.

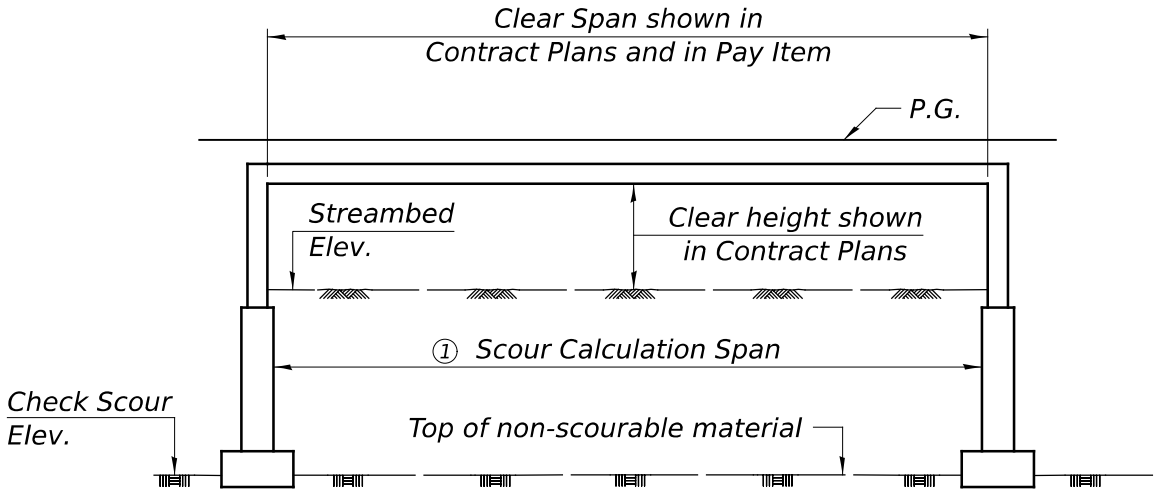
  A closed bottom structure may not always be allowed, due to regulatory requirements. When not permitted it shall be indicated on the plans. On a case-by-case basis, the top of slab elevation may be required to be more than three inches below the streambed elevation, due to regulatory requirements. See Figure 2.3.11.2.2-3 for an illustration.

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FOOTING IN SCOURABLE MATERIAL

(Top of footing at or below Check Scour Elev.)



FOOTING IN NON-SCOURABLE MATERIAL

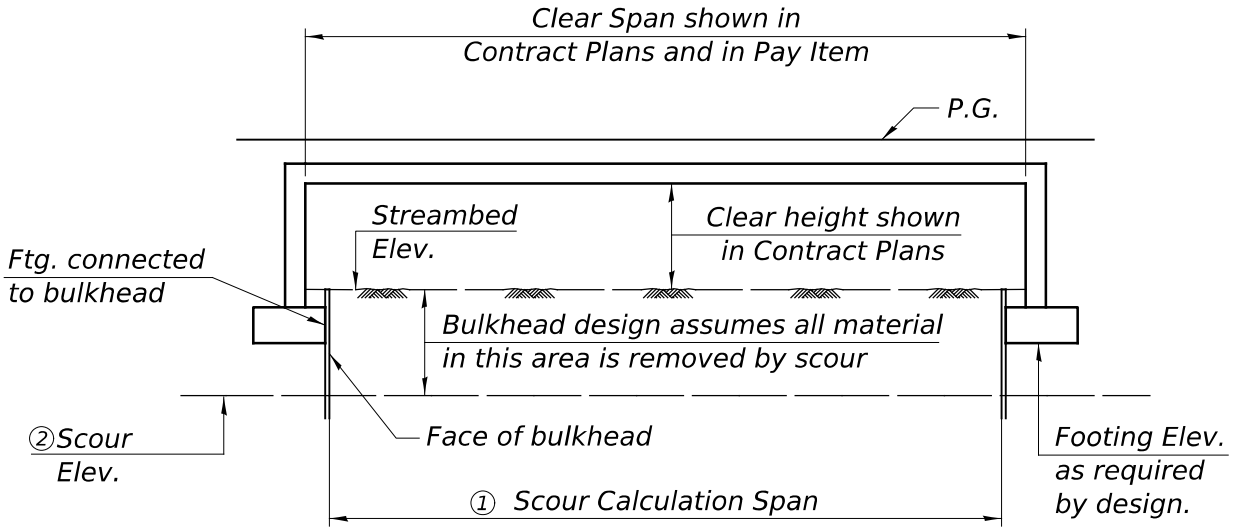
(Bottom of footing a minimum of 6" into non-scourable material)

① *Assumed to be Clear Span minus 1 foot.*

METHOD 1
SCOUR DETAILS FOR
THREE SIDED STRUCTURES

Figure 2.3.11.2.2-1

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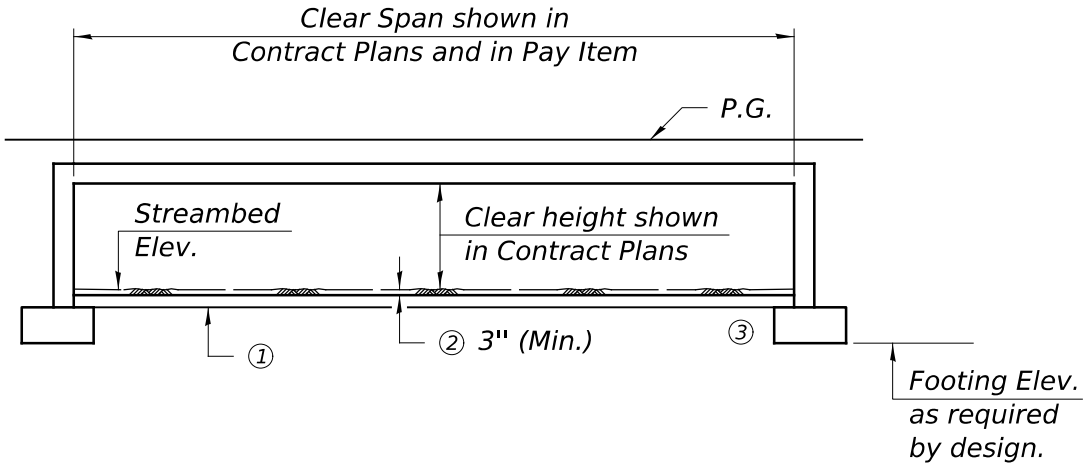
① *Assumed to be Clear Span minus 4 foot.*

② *Design Scour - Strength I*
Check Scour - Extreme Event

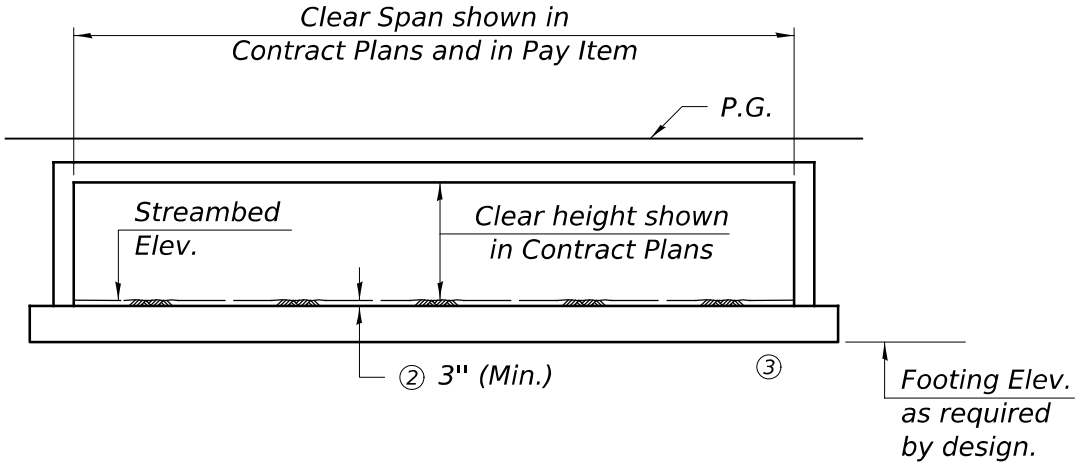
METHOD 2
SCOUR DETAILS FOR
THREE SIDED STRUCTURES

Figure 2.3.11.2.2-2

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INDIVIDUAL FOOTINGS WITH CLOSED BOTTOM



RAFT FOOTING

① *Slab shall be designed to span opening; attached to wall or footing; and have a minimum thickness of 4". Slab may be cast in place or precast with tongue in grove joints.*

② *Greater than 3" may be required for environmental reasons.*

③ *4' cutoff walls measured from top of bottom slab are required at each end of slab.*

METHOD 3
SCOUR DETAILS FOR
THREE SIDED STRUCTURES

Figure 2.3.11.2.2-3

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2.3.11.2.3 Site Limitations

Three sided precast concrete structures may be impacted by the following conditions:

1. Flowline is underlain by scour susceptible sandy soils. A scour evaluation is required and protective measures, if necessary and appropriate, shall be provided.
2. High seismicity areas, unless special foundation treatments and/or anchoring devices can be provided effectively and economically.
3. Weak soil conditions which would require pile foundations.

When the above conditions would impose relatively high additional costs, a cost comparison is required to justify a three sided precast concrete structure compared to other bridge/culvert alternatives.

2.3.11.2.4 Plan Processing Procedures

The contract plans shall show a generic three sided structure with a rectangular shape and level top slab, and a suggested foundation type (spread footing, pile supported footing, etc.) with a suggested maximum bottom of footing elevation.

The clear span shown in the plans shall be the minimum horizontal opening allowed rounded up to the nearest foot, measured at right angles to the centerline of the three sided structure and shall be used in the pay item. The minimum or lowest allowable elevation of the bottom of the top slab shall be determined from the Hydraulic Report. The Contractor's proposed three sided structure and foundation design shall provide at least the minimum clear span for the full height from the bottom of the top slab elevation to the check scour elevation, for Method 1 and Method 2.

The clear height shall be shown on the plans and shall be defined as the distance from the bottom of the top slab at the centerline of the span to the stream bed elevation at the upstream end of the structure. This height shall no longer be specified in the pay item.

If either Method 1 or Method 2 is shown on the contract plans, a Design Scour Elevation Table shall be shown on the plans. The Design Scour Elevation and Check Scour Elevation shall include scour due to contraction scour, local scour, and long term degradation. The NCHRP 24-20 equations in HEC18 should be used for the local scour calculations. Three sided structure design and construction should maintain the shape and location of the natural channel beneath and throughout the structure, preserving the natural flow characteristics in the channel to the extent feasible.

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If Method 3 is shown on the contract plans, the Q100 and Q200 calculated scour elevations shall be provided in the General Notes.

If a railing or parapet is required, it shall be designed and detailed on the contract plans. If an anchorage slab is required to isolate the railing loads from the rest of the structure, it shall be designed and detailed on the contract plans.

If waterproofing is required, it shall be designed and detailed on the contract plans and GBSP 81 shall be included in the contract.

The Contractors shall utilize the services of a pre-qualified design engineering firm pre-qualified by the Department in the category of "Structures - Highway: Typical" to coordinate the development of all construction documents necessary to design and construct the structure.

The Contractor's construction documents (i.e. shop plans, design calculations, working drawings etc.) shall include the design of the three sided structure, the design of the foundation, wing walls, and the design of any structural component required for the method chosen to address potential scour. The plans shall also include a revised waterway information table with the actual opening provided for all events, and any revisions to the scour table (if necessary) to account for the actual structure installed. The remaining information in the waterway information table shall match the waterway information table shown in the contract plans. The soil borings and Structure Geotechnical Report (SGR) will be available to the Contractor at the time of the bulletin.

With these changes, the Structural Engineer(s) sealing the drawings for the Contractor/supplier shall be the engineer(s) of record for the design of the three sided structure, as well as the foundations, wing walls, and scour protection method utilized. In summary:

1. Prequalification, materials, design, submittals, and construction for all projects for all projects utilizing a three sided precast concrete structure shall follow GBSP 90.
2. TSL and Final plans for three sided precast concrete structures shall identify the size (span x rise), length, and skew angle (in 1 degree increments) of the bridge.
3. A detailed design of three sided precast concrete structures with precast headwalls and precast wingwalls is not required on the Final plans. However, final plans shall include all geometric dimensions and a detailed design for all cast-in-place foundation units and cast-in-place headwalls and wingwalls. In addition, all General Notes in Section 3.1.3 related to three sided precast concrete structures shall be shown on the plans.
4. Final plans shall include the pay item Three Sided Precast Concrete Structures (Special) of the

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clear span specified. (Final plans using older provisions that separately measure and pay for cast-in-place headwalls, wingwalls, and footings shall include the pay item Three Sided Precast Concrete Structures, *Span x Rise* and applicable pay items for the remainder of the substructure elements).

5. Final plans, along with any revised waterway information table for the actual structure to be installed, shall be submitted along with all pertinent special provisions to the Department for review and approval.
6. Shop Drawings of all precast elements including the detailed design of the three sided precast concrete structure shall be prepared and submitted according to GBSP 90 for review and approval. The Shop Drawings will be incorporated as part of the As-Built plans.

To facilitate the initiation of this type of project, the BBS is available to assist the Districts/Consultants in working out problems which may arise during plan development.

2.3.11.3 Hydraulic Issues

The invert elevations of all culverts at stream crossing locations shall be set a minimum of 3 in. below the lowest point in the stream cross section. This will ensure that culvert inverts will not become a barrier to fish migration during low water. The size of the culvert opening does not need to be increased to compensate for lowering the invert 3 in. Locations which may warrant lower invert elevations shall override this policy.

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2.3.11.4 Foundation Issues

Many of the foundation type issues discussed under Foundation Component Selection in Section 2.3.6.3 for bridges are also applicable to foundations for box culverts and three sided structures. The SGR should provide planning and design recommendations to determine the most cost effective and feasible foundation treatment to be used on the TSL plans.

2.3.11.4.1 Culverts

Box Culverts have some unique geotechnical issues that should be evaluated to ensure adequate long term performance of both the box and the wingwalls.

The most common issues affecting the box portion of a culvert structure are mitigating differential settlement and ensuring constructability of the bottom slab. Boxes are often located in existing stream channels where the new loading from a culvert and fill above will likely generate some settlement. It should be noted that the theoretical new loading at the base of the box is not as large as the new full height of soil fill loading adjacent to the box which can result in differential settlement along the roadway alignment. Since portions of the new box alignment are often located on previously unloaded channel sediments while other segments may be placed through preloaded existing embankment, concern for differential settlement along the box alignment should also be considered. Consequently, it is critical that the designer evaluate the variation in applied loadings as well as the changes in foundation soil conditions to determine if any ground modification is necessary. Cast-in-place boxes have some tolerance to bridge across settlement prone areas but can crack when the differential foundation support is excessive.

As an alternative to ground modification, a box can be designed and constructed in non-continuous segments which are jointed by collars to allow articulation and prevent overstress. Known as segmenting and cambering, the collar joints are placed at locations where changes in surcharge loading or foundation stiffness occur, and constructed at an elevation which will settle into the desired location. The most common configuration involves dividing the box into three segments with the center segment located directly below the level portion of the embankment. The center segment and its collars are detailed to be constructed above the desired flow line by 90 to 110% of the amount of estimated settlement, while the remaining outer segments are shown sloping from the collar to the head wall which is normally located above the flow line by about 10 to 25% of the estimated settlement amount.

Precast boxes are articulated and can handle some differential settlement. However, when excessive movement is expected, the joints between the box sections can separate, causing the geotextile joint

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fabric to fail and allow soil to enter the box. In some cases, joint separation may decrease, causing contact and damage to the precast units. For these reasons, precast box culverts are normally only used at sites where minor amounts of settlement are expected or at locations where the foundation soils have been modified to mitigate settlement.

Where inadequate soil conditions are present to support the proposed loadings of either precast or cast-in-place boxes, removal of these soils and replacement with a coarse aggregate can be an economical treatment which provides the required stiffness or uniformity in foundation support. The cost effectiveness of this solution versus other ground modifications or structure type changes should be verified. It is normally used when removal depths are not excessive, since concerns over cut slope stability or feasibility of stage construction soil retention can necessitate the use of other options. Removal and replacement also typically requires some field verification and adjustment to plan limits in order to address local problem zones or areas of uncertainty between borings. This may mean reduced cost if the engineer finds the encountered conditions to be better than that indicated in the boring data. The designer should determine and show in the plan view the horizontal limits (stations and offsets) of the removal at the base of the excavation. The elevation view should show the elevations at the base of the removal. The plan and elevation removal limits should closely correspond to the boring data so the inspector can determine the material the designer intends to be removed and what can remain. Since conditions encountered upon excavation can differ, the Geotechnical and Field Construction Engineers may need to extend or reduce the limits to address the as encountered conditions. Along with the plan and elevation limits, the following note should be included.

> *The limits and quantities of removal and replacement shown are based on the boring data and may be modified by the District Geotechnical and Field Engineers for variable subsurface conditions encountered in the field.*

Excavation of unsuitable material shall be paid for as “Removal and Disposal of Unsuitable Material for Structures”. The replacement material and capping requirements are dependent upon the application, considering the anticipated loading, placement conditions, structure settlement tolerance, and cost of the replacement material. The pay item “Rock Fill - Foundation” is commonly used. Consult the BBS for the appropriate Special Provision. Figure 2.3.11.4.1-1 gives an example of elevation and plan view details for removal and replacement.

For conversion between weight and volume, Rock Fill may be assumed as 1.8 tons/cu yd for estimates of quantities.

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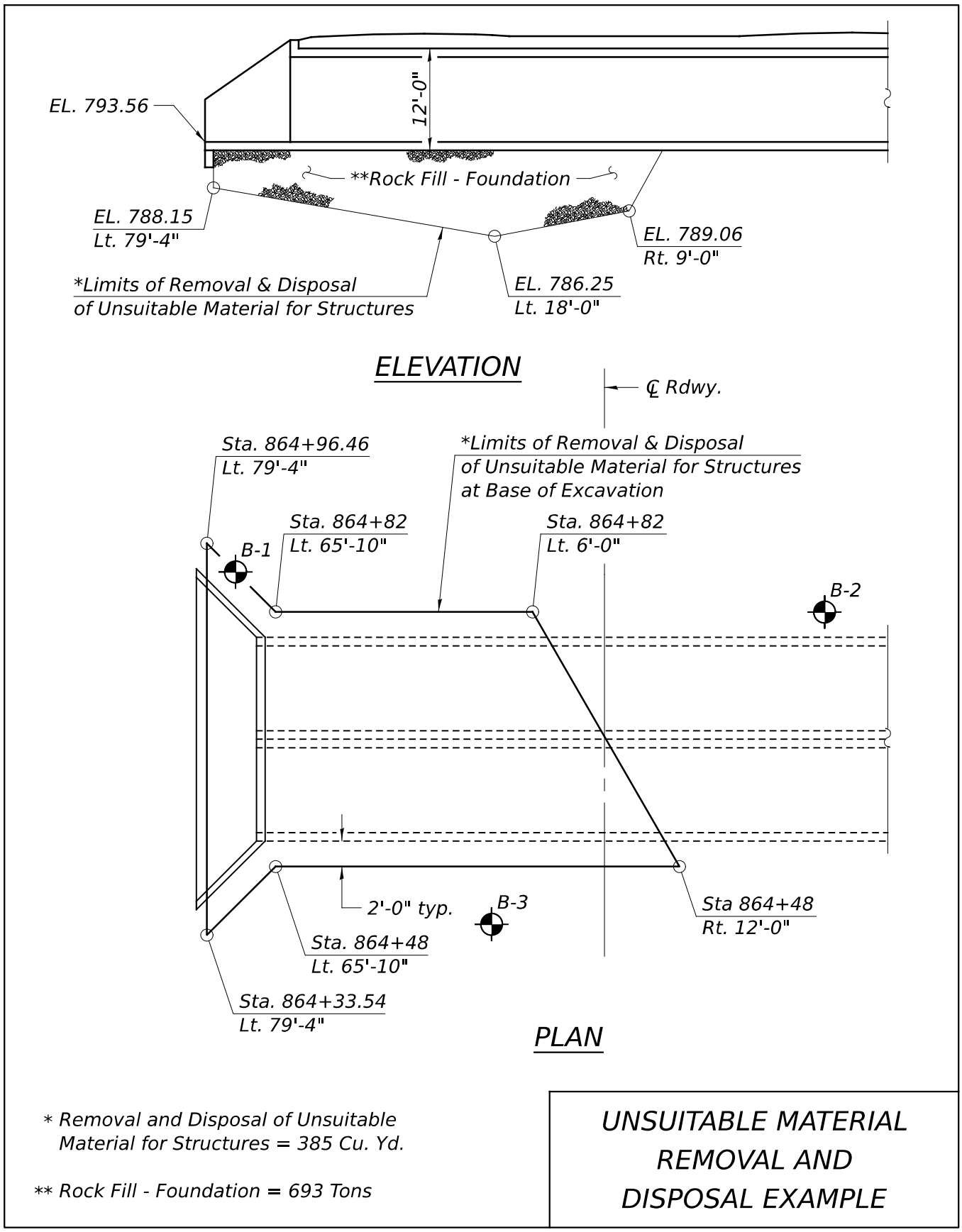


Figure 2.3.11.4.1-1

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When no removal and replacement is required, the foundation soils can become pumpy and unstable due to construction equipment loadings during excavation, rebar placement, forming and concrete placement. In these cases, the Contractor may need a "working platform" to properly construct the culvert bottom slab. The need for such platforms is dependent on the type, thickness and strength of the soils encountered, the method of water diversion selected by the Contractor, precipitation, construction sequence, and the time of the year the box is constructed. Since the borings are often taken in locations that do not give the designer accurate information about the sediment in the channel and considering the many factors cited above that affect the need for a platform, the designer is usually not in a position to specify its use and thus should not in the plans. The Field Engineer or District Geotechnical Engineer should make the determination that a "working platform" is required during excavation based on the field conditions. Thus, any removal on the plans is understood to be related to the long term foundation performance of the box and embankment, rather than a tool to facilitate construction.

When the estimated water surface elevation of the stream water exceeds 4 ft., construction of a water diversion system may be very difficult. Maintaining dry conditions for bottom slab construction can be problematic in granular foundation soils below the water table, as their permeability may not allow normal pumping to keep up with the water inflow through these soils.

In addition to the geotechnical issues discussed above affecting the box portion of the culvert, the culvert wingwalls are the other important element that is heavily influenced by the foundation soils and wall backfill. In most cases, a horizontal wing is the most economical and preferred wall type. They are supported by the box rather than the foundation soils and thus, their feasibility evaluation is structural rather than geotechnical. In cases where the culvert height and/or wing length/skew will not permit the use of horizontal wings, L-type wings provide an excellent alternative. The L-type wing is structurally connected to the box at the cutoff wall and via the wing footing/bottom slab connection but is not connected above the invert. Thus, the foundation soils, particularly toward the end of the wing, should assist in providing vertical and lateral support. The standard designs provided in the Culvert Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design) can be used when the factored applied bearing pressures do not exceed the factored bearing resistance of the foundation soils. When the resistance is not adequate, or the structural limits shown in the Culvert Manual are exceeded, or if precast boxes are used, other soil dependent/box independent wings should be used. These wings include MSE, T-type, gabion, sheet piling, soldier piling, apron supported, and precast modular. MSE is normally not economical due to the small quantity and raises concerns in some hydraulic applications about loss of granular backfill or foundation soils. T-type wings are fairly common as their aesthetics, alignment and foundation design can be modified to accommodate most any application. However, the resulting foundation expense, particularly when either a cofferdam or piles are required, may suggest that another wing type may be more appropriate. Gabion wings can be specified to follow a wide range

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of curved alignments and face batters. They can be placed through limited depths of water, but should be supported on reasonably good foundations soils to resist overturning and bearing pressures. Sheet pile walls also allow installation through open water and at locations where bearing capacity may not be adequate for gravity walls. Soldier piles are used where sheeting cannot be driven because H-piles can penetrate farther or can be drilled when required. However, they require either a cast-in-place or other facing system. Cast-in-place aprons are often used with precast boxes and should be analyzed like a "reverse L" wall design as the apron and cutoff wall provides the foundation. The apron's lack of embedment and soil weight makes them difficult to design and should be used where proper foundation soils (sliding and bearing pressure) are present and where the skew angle is not excessive. Various precast modular wingwall systems have also been used, most commonly with precast boxes and three sided structures to make the entire structure precast.

2.3.11.4.2 Three Sided Precast Concrete Structures

The GBSP 90 for three sided precast concrete structures permits the contractor to elect to provide an alternate method of scour protection according to Section 2.3.11.2.2 so long as hydraulic equivalency is provided. Three sided structures should be provided with adequate foundation support to satisfy the design assumptions permitting their relatively thin concrete section. These foundations are designed and provided in the plans using the worst case loadings which are available from approved pre-cast vendors. Spread footing foundations are most commonly used since they prove cost effective when rock or scour resistant soils are present with adequate bearing and sliding resistance. The use of precast spread footings shall be controlled by the planner and shall only be allowed when soil conditions permit and shall not be allowed to bear directly on rock or when rock is within 2 feet of the bottom of the proposed footing. When lower strength soils are present, or scour depths become large, a piles supported footing shall be used. The lateral loading design of the foundation is important because deflection of the pile or footing should not exceed the manufacturers' recommendations to preclude cracks developing.

In addition to foundation type, the wingwall type shall be provided on the TSL and Final plans. Wing selection issues are similar to those discussed in the culvert section above. The restrictions on the use of cast in place or precast wings and headwalls shall be based on site conditions and the preferences of the owning District. These restrictions shall be noted on the TSL and final plans.

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2.3.11.5 Culvert Nesting Ledge Issues

Multiple box culverts with a clear height of 4 ft. – 0 in. and greater shall be provided with 1 in. ledges, 4 ft. – 0 in. long on each side of all interior walls near the downstream end when these walls contain a single plane of reinforcement bars located at the wall center. These ledges provide suitable nesting sites for certain bird species (phoebes and barn swallows) that tend to nest in man-made shelters. The ledge detail is depicted in Figure 2.3.11.5-1.

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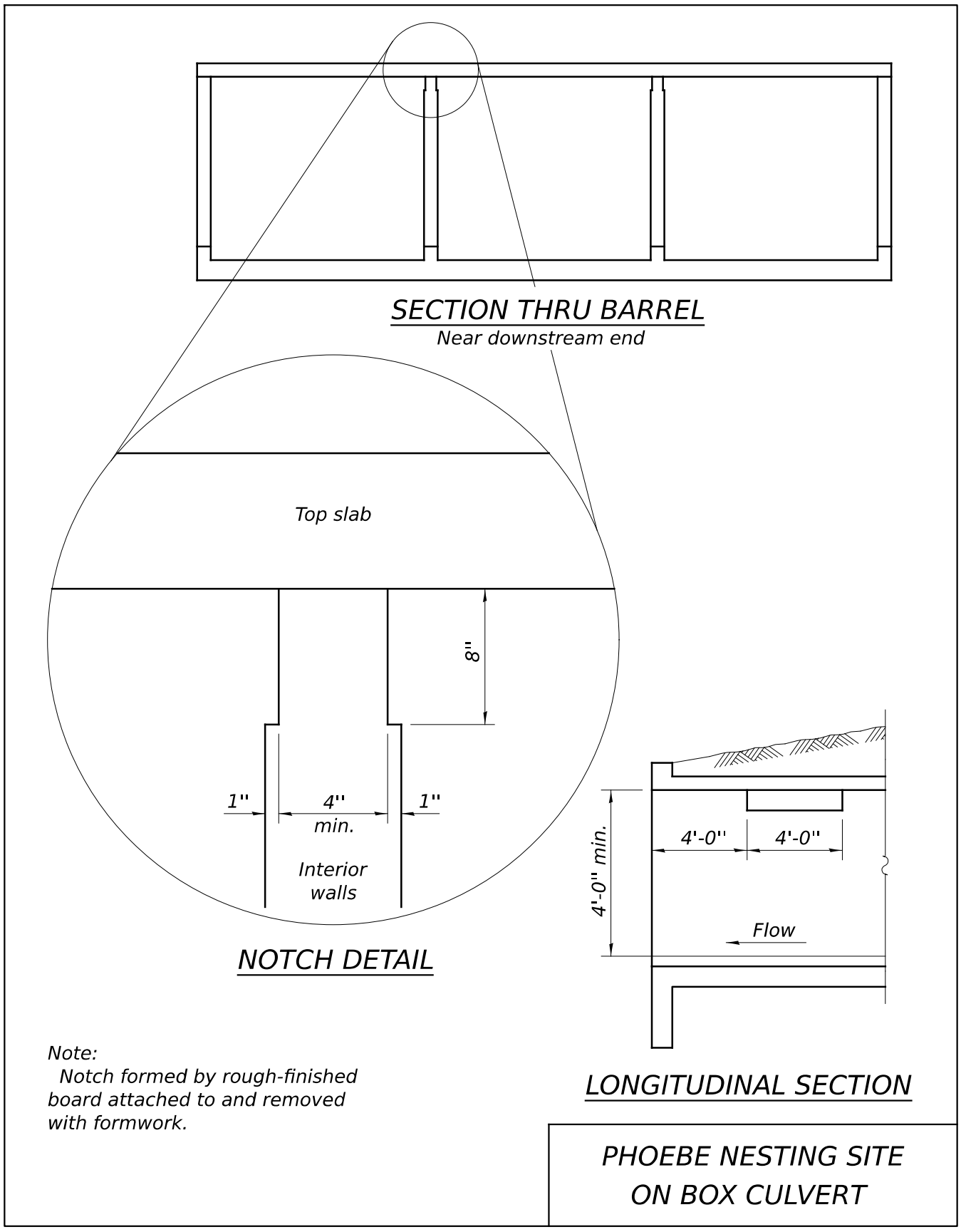


Figure 2.3.11.5-1

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2.3.12 Retaining Wall Selection Process

Many factors such as geometric limitations, geotechnical site conditions, aesthetics, structural feasibility, construction equipment access, and traffic staging affect retaining wall type selection. The engineer responsible for the TSL plan should identify and evaluate these issues, and conduct feasibility analyses which leads to a retaining system selection that is cost effective.

To reduce both initial and long term maintenance expenses, efforts should be made to minimize overall wall length and exposed height. This can be accomplished by utilizing slopes either in front or behind the wall as well as evaluating various wall locations/alignments. During this process, there are a number of right-of-way, roadway cross section, drainage, utility, and construction limitations that should be considered.

The exposed retention height for a wall is established from finish grade elevations found within the roadway cross sections. The engineer responsible for TSL development should work closely with the roadway engineer when determining finish grade cross sections along the length of the wall. The bottom of wall elevations should be determined to accommodate any drainage/utility excavation proposed in front of the wall and satisfy the minimum embedment necessary for the wall type selected. The final top of wall elevations should be established to satisfy the cross section retention requirements while forming an aesthetic top of wall profile. Where the wall face is visible to traffic or commonly viewed by the public, the use of form liners or other wall face texturing should be strongly considered. The need for coping, traffic barrier, noise wall/sight screen, railing or fencing mounted on the top of wall should also be determined and coordinated with the District.

In addition to establishing the required wall retention geometry and other site design constraints, geotechnical issues have a substantial impact on the wall type section process. The majority of loadings applied to the structure as well as the capacity of the foundation are controlled by the soil conditions present. The SGR generally contains all geotechnical analyses, foundation and wall type recommendations, and design parameters required to assist in wall type feasibility and cost analyses. Substantial coordination between the structural and geotechnical engineer during the SGR development process is necessary to ensure appropriate design parameters and recommendations are provided. Section 2.3.6.3 should also be referenced when evaluating foundation type options. Section 3.11 and the IDOT Geotechnical Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical) should be referenced for detailed technical information concerning the design and subsurface investigation of retaining walls.

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There are four retaining wall types commonly built in Illinois. These are mechanically stabilized earth (MSE), cast-in-place concrete T-type, soldier pile, and permanent sheet pile. Soil nailed, precast modular, segmental block, gabions, and other specialized wall systems have been utilized by the Department on a limited basis when their unique wall properties lend themselves to specific project conditions. MSE and cast-in-place walls are most economical and better suited for use in "fill" sections, while soldier piles and permanent sheet pile walls are generally most advantageous in "cut" section retention applications.

Typical cross section and details should be shown on the TSL plan. Policy and procedures for the design of retaining walls are given in Section 3.11.

2.3.12.1 MSE Walls

MSE walls provide one of the most cost effective and durable wall structures available. The walls internal stability is designed by the wall vendor during construction to provide over 75 years of design life. The cost savings advantage is most prevalent on projects with large bid quantities or on structures where the maximum wall height is relatively tall. Locations with short wall heights or lengths often lend themselves to other wall types.

Precast panels avoid the typical cracking that occurs on CIP walls and provide superior aesthetics due to their articulated panel pattern. Panels can be cast with a smooth face or form liners can be specified to produce a variety of cast patterns. MSE walls can also be constructed along curved alignments. Both design and construction time is reduced when an MSE wall is selected.

The TSL shall provide the "top of exposed panel line", the "finished grade line at front face of wall" and the "theoretical top of leveling pad" which is normally set 3 ft. – 6 in. below the finished grade unless there are other geotechnical or geometric limitations. Using these wall heights, the reinforced mass should be assumed to be 0.7 of the height for feasibility and economic analyses. The engineer responsible for the TSL, assisted by the recommendations contained in the SGR, is responsible verifying external stability and foundation soil adequacy. The reinforced mass is considered rigid and shall resist overturning, sliding, and bearing pressure, as well as satisfy settlement and global stability considerations similar to those for spread footings. See Section 3.10 for more information on the design of spread footing foundations. These walls have substantial weight and some settlement is almost certain to occur (some during construction and some after). Although MSE walls are well suited to handle settlements due to panel articulation and have the ability to adjust while maintaining stability because of the semi-rigid reinforced mass, the engineer responsible for the TSL, using the SGR, should assess the magnitude, horizontal limits, and time of settlement as well as the effects of settlement on the wall and any

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infrastructure placed on top of the reinforced volume. When settlement or bearing capacity of the foundation soils is not adequate, ground modification should be evaluated prior to using other wall types. Removal and replacement of unsuitable soils, use of lightweight fill, wick drains, aggregate columns ground improvement, or longer soil reinforcement (wider than 0.7xH), etc. should be considered.

When abutments are placed on MSE walls, piles are most commonly used although spread footings and drilled shafts have also been employed. Spread footings are more cost effective and avoid the approach slab "bump". However, foundation bearing soils below the wall should have superior capacity to carry both the bridge and wall loadings. When piles are selected, pile corrosion and negative skin friction should be considered. Most pile supported abutments also require soil reinforcement to be attached to the abutment backwall since battered piles should not be used to resist lateral loadings. Settlement of walls with abutments becomes a more critical concern since excessive long term settlement with a spread footing abutment can cause distress in a bridge structure. Piles may develop negative skin friction, but not settle, causing an approach slab bump.

The vehicle collision force provisions discussed in Section 3.9 shall not be applicable to the base of MSE walls at grade separations as discussed in AASHTO LRFD Bridge Design Specifications Article 3.6.5. Therefore, an additional barrier is not required by the Department in front of the MSE for these situations.

Inspection of structure elements (including bridge, if applicable) shall be considered for abutments on MSE walls. For structures built over or adjacent to railroads there may be significant lead times required for permits for inspections.

2.3.12.2 Cast-In-Place T-Type Walls

Cast-in-place T-type walls comprise the large majority of wall inventory currently in service in Illinois. For wall comparison purposes, cast-in-place T-Type walls are considered to have a design life of 75 years. It is often deemed a feasible wall type since it can be structurally designed to perform with limited geotechnical input for a wide variety of conditions. However, this wall type is often inappropriately selected on "fast track" projects or based on inadequate geotechnical or cost data only to be value engineered during construction.

T-type walls commonly require a smaller excavated width than MSE walls. The heel of T-type walls typically extends 25% to 30% of the wall height behind the face while MSE walls require an excavation width of at least 70%. This produces cost savings and can help avoid temporary sheeting for construction of T-type walls. Traffic barriers, parapets, noise walls, and other structures can easily be attached to the stems of T-type walls. Their configuration also allows for easy utility installation and future utility

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excavation behind the wall stem. In some locations, the cost of footing de-watering, cofferdams, seal coats, as well as ground modification, piles, or drilled shafts should be taken into account.

Feasibility evaluations consist of evaluating the adequacy in bearing of spread footings, settlement considerations, etc., or whether piles or a drilled shaft foundation should be specified. The lateral earth pressures to be used in design and feasibility analyses should consider backslope angle, height, and retained soil properties. The SGR and/or geotechnical engineer should be consulted to develop appropriate design values. Regardless of foundation type selected, CIP T-type walls normally have their footings located at least 4 ft. below finished grade.

2.3.12.3 Soldier Pile Walls

Soldier pile walls are most suited for “cut” situations, particularly when continuous undisturbed lateral support is required to be maintained adjacent to existing ground and infrastructure. Soldier pile walls can also be used to retain new fill at locations with moderate retained heights, adequate foundation soils, or tolerance for deflection. However, since other feasible wall types often provide a longer design life with less concern for wall deflection (resulting from fill compaction and passive pressure mobilization), soldier pile walls see more limited use in fill applications.

Various wall facing treatments can be used depending on aesthetics and costs. Locations that are rural or rustic in nature, or are hidden from public view can utilize an exposed treated timber lagging which provides the least expensive facing. An exposed lagging wall typically includes a CIP concrete cap to cover the top of the soldier piles and lagging. More commonly, though, concerns for similarity in wall aesthetics, and maintenance of exposed timber and solder piles dictates that a CIP concrete facing be used. The use of a CIP concrete facing allows some variation in rear face alignment to hide out-of-alignment soldier piles, pile deflections and lagging deflection. Precast lagging and precast panels have been used, but the casting, shipping, handling and installation expense can be excessive for most locations in the State. In most cases, the wall facing is specified on the TSL to be extended 2 ft. below the finished grade at the wall face.

Soldier piles can be driven to the required tip elevation or placed in drilled holes and encased in concrete. Driving piles is normally less expensive but the designs are limited to H-pile and small W-sections. Drilled soldier piles can utilize larger W-sections, built up plate sections or multiple W-sections to accommodate tie back connections. The larger lateral bearing area of drilled soldier piles generally allows a wider spacing or shorter embedment. Drilled piles also provide added corrosion protection and, where exposed treated timber lagging or precast panels are used, alignment can be more carefully controlled than driven

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piles. Where vibration, noise, or driving feasibility (close bedrock, cobbles/till, overhead clearance interference, etc.) are concerns, drilled soldier piles should be specified.

When evaluating feasibility, the spacing of soldier piles should typically be assumed to be from 6 to 8 ft., although spacings up to 14 ft. have been used in portions of walls with shorter retained heights. For special applications, where bending or deflection requirements are severe, soldier piles can be drilled nearly tangent (adjacent) to each other to address these issues. When the wall height and loadings are not uniform along the length of the wall, the pile spacing or pile section size or both can vary along the length of the wall to produce consistent deflections and maximum design economy.

Soldier pile walls are not considered as durable as MSE walls. They can be estimated to have a 50 yr. design life. The TSL shall specify items such as facing type, soldier pile type (driven or drilled), estimated top of rock, and the tip elevation/spacing anticipated from the preliminary feasibility analyses, noting that the final spacing and tip elevation to be determined in design.

2.3.12.4 Permanent Sheet Piling

Permanent sheet piling is best suited for sites where the soil has little or no cohesive binder (predominantly granular conditions) and is unlikely to temporarily arch for lagging placement. Permanent sheet piling is also commonly used in conditions where the excavation is to go below the water table and/or where water retention is required. In sandy conditions, maintaining a drilled excavation can be difficult and expensive, and may require casing, over-sizing the holes, and tremie concrete placement methods. As such, sheet piling should be considered.

Depending on the required aesthetics of the location, a cast-in-place concrete facing can be added or the sheeting can be left exposed and capped with a steel channel section, making it a cost-effective alternative to other wall types.

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2.3.12.5 Anchors

Some locations where a soldier pile or sheet pile wall is selected may require wall anchors to satisfy deflection or strength issues. In these cases, the TSL plans should indicate the anchor type determined to be most cost effective and feasible given ROW and geotechnical design constraints. Deadman anchors are normally the least expensive but usually offer a lower capacity than other anchor types. Helical anchors can be installed with less disturbance than deadmen, but are limited to use in locations where the soil strength will not prevent installation. Permanent ground anchors provide the highest confidence and capacity but are more expensive.

2.3.13 Type Size and Location Presentation

The checklists below are provided as an aid to the planner when completing a TSL plan. They may not be all-inclusive for any particular project.

See also Appendix A1.3 for approved TSL plans which are available online for example reference.

2.3.13.1 Checklist for Bridges

General

1. Review Bridge Condition Report, Structure Geotechnical Report, and Hydraulic Report to see that the TSL plan agrees with the listed reports and that the structure fits the site conditions.
2. Verify the district has reviewed and agreed to all information on the Structure Reports prepared by consultants.
3. Verify the information on the Plan & Profile sheet (North arrow, stream flow, profile grade, horizontal curve, riprap, guardrail, etc.) matches the TSL plan.
4. Consultants should provide company name on TSL plan.
5. For deck or superstructure replacement projects, provide brief Scope of Work listing main items to be replaced or repaired.
6. For existing concrete slab bridges, with condition rating 4 or less, review concrete cores to determine stage construction feasibility (See “Bridge Condition Report Procedures and Practices” Section 3.6.3)

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Title Block

1. Label the page as “General Plan and Elevation”
2. List the following data:
    a. Roadway name/marked route over feature crossed.
    b. If the structure is over a waterway listed as a navigable public body of water, provide the term “Public Water”.
    c. Designated funding route and section number.
    d. County.
    e. Station at the center of the bridge of main route or intersecting survey lines.
    f. Structure number. (New structure numbers are issued for bridges that do not reuse any part of the existing structure.)

Location Sketch

1. Provide a sketch that shows four sections of the township.
2. Label the range, township and principle meridian.
3. Provide a north arrow.
4. Call out the bridge location.
5. Recheck names of major features on sketch.

Highway Classification

1. List the following data for each route over and under a structure:
    a. Designated funding route and roadway name/marked route.
    b. Functional Class from the Illinois Structure Information System.
    c. ADT – Present and Future.
    d. DHV – Future.
    e. ADTT (including single and multiple unit trucks).
    f. Design speed.
    g. Posted speed.
    h. Indicate the traffic count as one-way or two-way.
    i. Provide the Directional Distribution for two-way traffic.

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Loading (truck)

1. Provide the correct truck loading based on the LRFD design specification.
2. Show the allowance for Future Wearing Surface (FWS).
3. On rehabilitation projects, verify the beams and existing substructure ("Bridge Condition Report Procedures and Practices" Section 3.5.7) can support the proposed future wearing surface and meet the required loading (LRFD, or the design specifications of the existing structure when approved by the BBS).
4. Provide the live load deflection criteria for bridges with sidewalks and/or bicycle lanes.

Design Specifications

1. Provide the applicable bridge design specifications.
2. Include any additional applicable specifications (e.g. seismic, seismic retrofit, curved girders, etc.)

Design Stresses

1. Provide the design stresses for Field, Precast, and Precast Prestressed units as required.
2. Provide a separate table for design stresses of existing elements that will be incorporated into new construction.
3. Verify the correct design stresses are shown for the proposed design method.
4. Indicate the design stresses and type of protection for primary steel girders.

Seismic Data

1. Provide the seismic data for the applicable design specification.

Upper Left Hand Corner Data

1. Provide a benchmark that matches the structure report and survey data.
2. Provide the existing structure number with the construction year and project name.
3. Provide a brief description of the existing superstructure and substructure that includes the length and width of existing structure.
4. Indicate the proposed method of traffic control for the proposed bridge construction.
5. Indicate if any items of the existing bridge construction will be salvaged for future IDOT use.

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Waterway Information Table

1. Verify the numbers match the approved hydraulic waterway information table.
2. Verify the design high water elevation matches the elevation shown in the elevation view.
3. Verify the proposed hydraulic opening equals or exceeds the existing opening, and reduces the created heads.
4. Below the waterway information table, provide the existing and proposed velocities.

Design Scour Elevation Table

1. For stream and river crossings, provide a Q_{100} and Q_{200} design scour elevation for each substructure unit.

Offset Sketch

1. Provide an offset sketch for curved roadway alignments. Show the x and y offset (at each substructure unit and end of approach, if applicable) from centerline of curved roadway to a straight reference line. See Appendix A1.3 for examples.

Profile Grade

1. Provide a profile grade that extends beyond the bridge approach slabs.
2. Show grade slopes, curve length, elevation and stations of PVC, PVT, & PVI.
3. Verify the profile grade matches the plan and profile sheet.
4. Indicate the roadway name and location of the profile grade line.
5. Provide profile grade of roadway under the structure to verify vertical clearance.
6. Check for negative or excessive fillets on rehabilitation projects with field survey data.
7. For structures with bridge deck smoothness grinding, place the following note beneath Profile Grade Figure: *The Profile Grade shows the final grade after grinding*.

Horizontal Curve Data

1. Provide horizontal curve data including the PI station, Δ, D, R, L, T, E, PC station, PT station, SE, TR, and runoff length.
2. Indicate superelevation and/or normal crown transition stations if transition occurs between approach slabs.

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Cross Section

1. Verify the bridge width is correct for the roadway classification and consistent with the approved BCR, if applicable.
2. Indicate the roadway centerline and profile grade location.
3. Provide out-to-out bridge width, including lane, shoulder, sidewalk and parapet dimensions.
4. Provide deck cross slopes and label the crown location, if applicable.
5. Provide slab thickness and check on diamond grinding criteria. For cast-in-place concrete slab bridges, provide thickness and indicate “subject to refinement during the design phase”.
6. Show beam depth. For a plate girder, indicate web depth only.
7. Provide the composite note, if applicable.
8. Verify the fillet is shown correctly.
9. Provide the barrier type and vertical dimension.
10. Provide median and sidewalk dimensions, if applicable.
11. Show a longitudinal open joint if required.
12. Locate the stage construction line and stage removal line.
13. For curved roadway alignments, provide distance from stage construction line to the nearest beam.
14. For each stage sequence, indicate the location of traffic lanes, limits of removal, limits of construction, and location of temporary barrier.
15. Show local tangent, offset, radial and varying dimensions for curved alignments. Include end of approach offset for curved alignments with straight beams. See Appendix A1.3 for examples .
16. Provide clearance diagram for all Railroad Bridges.
17. Provide construction diagram for all bridges over railroads.
18. Verify the clearance between the stage removal line and the stage construction line can accommodate temporary sheet piling, if required.
19. Verify whether the depth of differential dead load deflection at the stage construction line is acceptable. If not, investigate stiffer beam, beam spacing and closure pour options to satisfy deflection criteria.
20. Evaluate the condition of the existing superstructure and substructure to determine proper lane usage for Stage I traffic. Evaluate Load Rating Inspection (LRI), if available. Evaluate concrete cores, if applicable.
21. Provide spacing and label the floor drains and scuppers and verify bridge drainage is provided, as necessary. Also, verify if off-bridge drainage is required.
22. Indicate enclosed drainage system, if necessary.
23. Provide an outline of the cross section of the existing structure without dimensions.
24. Locate any utilities below the superstructure or conduits in the concrete parapets.
25. Show beam spacing and overhang dimensions.

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Abutment Section

1. Verify integral and semi-integral abutments meet limitation requirements.
2. Specify the type of expansion joint and verify it fits the bridge geometry.
3. Show the bridge approach slab.
4. Show the back of abutment location.
5. Show the vertical clearance to berm/end of slope wall.
6. Dimension the abutment width, depth, piling location, and pile type. For stub abutments, provide minimum width, subject to refinement in design.
7. Provide the dimension from the back of the abutment to the centerline of bearing.
8. Show the bearing type.
9. Dimension the approach slab seat width and backwall thickness.
10. Provide appropriate backfill and drainage details for selected abutment type.
11. For skewed bridges, indicate horizontal dimensions are at right angles.
12. Verify riprap bedding to above bottom of abutment cap.

Slope Details

1. Show appropriate slope treatment.
2. Provide a detailed section through the slope wall (or riprap) and corresponding ditch/anchor detail, and label any toe stone treatment as “Berm” or “Streambed” long with elevation as applicable.
3. Provide a slope wall flank detail and indicate layout distances in plan view.
4. Verify the riprap size is consistent with the stream velocity, if applicable.
5. Provide stone riprap flank details, if applicable.

Pier Sketch

1. Verify proper pier type configuration.
2. Show the actual number of columns for multi-column piers.
3. Verify the correct crash wall heights for bridges over railroads.
4. Provide dimension from ground line to top of crash wall.
5. Show ground elevations for each pier.
6. Provide foundation type and related elevations/details.
7. Provide section thru pier with an expansion joint, if applicable
8. Label expansion joint type, bearing types and dimensions from centerline of pier to centerline of bearings, if applicable.
9. Show open joints in caps and construction joints in base wall according to policy.
10. Indicate number and spacing of piles as required in design for pile or drilled shaft group footings.

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Elevation View

1. Show fixity and expansion conditions at all substructure elements.
2. Show vertical and horizontal clearances.
3. Vertical clearance for a bridge over a railroad should be shown in accordance with AREMA.
4. Vertical clearance for a bridge over a sag roadway should be determined based on form OPER 1306 (available at the primary Forms Page).
5. Show approach traffic barrier terminal types.
6. Show bridge approach slab, parapet, curb, and sleeper slab footing.
7. Show bottom of footing, abutment, or encasement wall elevations.
8. Show the existing superstructure, piers and abutments. Avoid using elevations from existing plans for rehabilitation projects.
9. Show foundation type and required elevations.
10. Show beam type. If steel, add note “Composite full width”.
11. Show slope treatment and indicate rise and run.
12. Show pipe culverts through embankment if required at grade separations.
13. Plot the existing ground line (if different than proposed).
14. Show 60’-0” construction berm with 1V:6H slopes for new and widening bridge projects (see Figure 3.14.1-1 through Figure 3.14.1-3).
15. Show ground elevations at piers.
16. Show ground elevation at base of berm slope.
17. Show streambed elevation.
18. Show design highwater elevation and EWSE.
19. Provide vertical clearance dimension to design high water elevation.
20. Indicate the type of cofferdams (Type 1 or 2) at each substructure unit where applicable.
21. Show location of light poles, if required.
22. Show navigation obstruction lighting, architectural lighting or other electrical systems, as required.
23. Show cross slopes for the roadway and shoulders below the bridge.
24. For structures over railroads, add a note indicating “No freefall deck drains will be permitted in the span over the tracks or within 10 ft. of cross arms of a railroad pole line”.

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Plan View

1. Show span lengths, distances from back of abutment to centerline of bearing for abutments with expansion joints, and back-to-back of abutment length. For integral abutments exclude the centerline of bearing.
2. Ensure the above dimensions match the stationing distances for tangent alignments.
3. Show the skew angle at a substructure unit.
4. Show approach roadway template, i.e. lane and shoulder widths, curb and gutter type, etc.
5. Show the bridge widths and out-to-out dimensions.
6. Show stations and elevations along profile grade at substructure units.
7. Show station equation for intersecting reference lines on roadways.
8. Show stations and offsets to roadway's tapers that are under or across structure.
9. Ensure bridges are shown with stationing increasing to the right.
10. Show stationing/flow direction under roadway
11. Show lane and shoulder dimensions under roadway.
12. Indicate the type of traffic protection on substructures for under-passing roadways.
13. Show channel width at right angles to stream.
14. Locate point of minimum vertical clearance on the bridge. For railroad bridges, the minimum vertical clearance should be shown in accordance with AREMA .
15. Indicate and check horizontal clearances.
16. Show stage construction line.
17. Show temporary construction requirements (sheet piling, geotextile wall, etc.) when applicable.
18. Plot the boring locations.
19. Show proper layout of slope wall or riprap configuration.
20. Show slope wall slope at right angles to stream.
21. Show pipe culverts and local drainage near structure.
22. Show bridge approach slab with parapet.
23. Show guardrail.
24. Show expansion joint accurately at bridge ends.
25. Show railroad mile post information.
26. Verify handicap ramps are shown on sidewalks at intersections.
27. Show north arrow.
28. Provide estimated light pole foundation locations, when present.
29. Show limits of existing structure.
30. Show floor drain/scupper spacing and type.
31. Show bridge approach shoulder drains when applicable.
32. Provide details for any utilities that may significantly affect the bridge design and construction.

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2.3.13.2 Checklist for Culverts and Three Sided Structures

General

1. Review Bridge Condition Report, Structure Geotechnical Report, and Hydraulic Report to see that the TSL plan agrees with the listed reports and ensure the structure fits the site conditions.
2. Verify the district has reviewed and agreed to all information in the Structure Reports prepared by consultants.
3. Verify the information on Plan & Profile sheet (North arrow, stream flow, profile grade, horizontal curve, riprap, guardrail, etc.) matches the TSL plan.
4. Consultants should provide company name on TSL plan.
5. For existing concrete box culverts, with condition rating 4 or less, review concrete cores to determine stage construction feasibility (See “Bridge Condition Report Procedures and Practices” Section 3.6.3).

Title Block

1. Label the page as “General Plan and Elevation”
2. List the following data:
    a. Roadway name/marked route over feature.
    b. Designated funding route and section number.
    c. County.
    d. Station at the center of the structure.
    e. Structure number. (New structure numbers are issued for bridges that do not reuse any part of the existing structure.)

Location Sketch

1. Provide a sketch that shows four sections of the township.
2. Label the range, township and principle meridian.
3. Provide a north arrow.
4. Call out the structure location.
5. Recheck names of major features on sketch.

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Highway Classification

1. List the following data:
    a. Designated funding route and roadway name/marked route.
    b. Functional Class from the Illinois Structure Information System.
    c. ADT – Present and Future.
    d. DHV – Future.
    e. ADTT (including single and multiple unit trucks).
    f. Design speed.
    g. Posted speed.

Loading (truck)

1. Provide the correct truck loading based on the design specification.
2. Show an allowance for Future Wearing Surface (FWS).
3. On rehabilitation projects, verify the structure can support the proposed future wearing surface and meet the required rating.

Design Specifications

1. Provide the applicable bridge design specifications.

Design Stresses

1. Provide the design stresses for Field and Precast units.
2. Provide a separate table for design stresses of existing elements that will be incorporated into new construction.
3. Verify the correct design stresses are shown for the proposed design method.

Design Earth Cover

1. Indicate the depth of cover (in feet) for all precast culverts.

Seismic Data

1. Provide the seismic data based on the applicable design specification (three-sided structures only).

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Upper Left Hand Corner Data

1. Provide a benchmark that matches the structure report and survey data.
2. Provide the existing structure number with the construction year and project name.
3. Provide a brief description of the existing structure.
4. Indicate the proposed method of traffic control for the proposed structure construction.
5. Indicate if any items of the existing bridge construction will be salvaged for future IDOT use.
6. Add a note stating “Precast alternate is not allowed” if site conditions require a cast-in-place culvert.

Waterway Information Table

1. Verify the numbers match the approved hydraulic waterway information table.
2. Verify the design high water elevation matches the elevation shown in the elevation view.
3. Verify the proposed hydraulic opening equals or exceeds the existing opening, and reduces the created heads.
4. Below the waterway information table, provide the existing and proposed velocities.

Profile Grade

1. Provide a profile grade that extends beyond the limits of the structure.
2. Show grade slopes, curve length, elevation and stations of PVC, PVT, & PVI.
3. Verify the profile grade matches the plan and profile sheet.
4. Indicate the centerline of the roadway and location of the profile grade line.

Horizontal Curve Data

1. Provide horizontal curve data including PI station, Δ, D, R, L, T, E, PC station, PT station, and SE.
2. Indicate superelevation and/or normal crown transition stations if the structure is located within a transition.

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Section Through Barrel of Structure

1. Show size of barrel opening.
2. Show thickness of walls.
3. Show thickness of top slab.
4. Show bottom culvert slab 1 in. thicker than top slab for cast-in-place culverts.
5. Indicate culvert top and bottom slab thickness is subject to refinement during final design.
6. If there is no fill on the CIP culvert, provide corbels.
7. Show construction joints 6 in. above the top of bottom slab for CIP culverts.
8. Show construction joints between walls and top slab for CIP culverts.
9. Indicate Phoebe nesting sites at downstream end of interior walls on CIP culverts.
10. For three sided structures, indicate slab and wall thickness and shape may vary as per manufacturer.
11. The top and bottom slabs of multiple cell box culvert extensions should be designed as continuous members.
12. Indicate membrane waterproofing system as necessary.

Longitudinal Section

1. Show lane, shoulder, median, barrier, and sidewalk widths.
2. Show roadway cross slopes.
3. Show profile grade location.
4. Show exit and entrance guardrail (if required)
5. Show height of barriers and pedestrian rails.
6. Show upstream and downstream flow-line and invert elevations. Set invert 3 in. below flow-line (or based on USACE Regulation, 12 in. below flow-line).
7. Show design high-water elevation (at upstream end of culvert) and EWSE.
8. Show height and width of headwall.
9. Show stage traffic widths.
10. Show stage construction widths.
11. Verify stage construction is consistent with condition of existing bridge (and the District's desire).
12. Show temporary concrete barrier.
13. Show top and bottom slab thickness.
14. Verify the need for an edge beam on the top slab (of cast-in-place culverts) at the stage construction joint. (Note, an edge beam is typically not required if stage traffic is located further than half of the design live load distribution width from the stage construction joint.)
15. Show cutoff walls depth.
16. Show buried utilities
17. Plot natural ground line.
18. Show foundation type.

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Plan View

1. Show dimension from out-to-out of headwalls (i.e. length along walls).
2. Show controlling culvert dimensions perpendicular to barrels.
3. Show approach roadway template.
4. Give skew angle.
5. Show width of headwall.
6. Show typical value of side-slopes in vicinity of culvert wings.
7. Show station and elevation on profile grade at CL of culvert.
8. Show culverts with stationing increasing to the top (typically, 3-sided precast structures are laid out like culverts).
9. Show flow direction under roadway.
10. Show centerline of culvert.
11. Indicate and check important horizontal clearances.
12. Show stage construction line and locate on roadway.
13. Show temporary construction details (sheet piling, geotextile wall, etc.) when applicable.
14. Show limits of existing structure.
15. Plot boring locations.
16. Show pipe culverts and local drainage near structure.
17. Show bridge approach slab if there is no fill on the culvert.
18. Show guardrail.
19. Show type of curb and gutter.
20. Show north arrow.

Design Scour Elevation Table

1. For 3-sided structures only, provide a design scour elevation table at streams and river crossings.

Stream Protection Details

1. Verify the need for stream protection (i.e. riprap, aprons, etc.) and show stream protection details as required.

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2.3.13.3 Checklist for Retaining Walls

General

1. Review Structure Geotechnical Report and any other reports to check the TSL plan agrees with the listed reports and that the structure fits the site conditions.
2. Verify the district has reviewed and agreed to all information on Structure Reports prepared by consultants.
3. Verify the information on Plan & Profile sheet (North arrow, stream flow, profile grade, horizontal curve, riprap, guardrail, etc.) matches the TSL plan.
4. Consultants should provide company name on TSL plan.

Title Block

1. Label the page as “General Plan and Elevation”
2. List the following data:
    a. Retaining Wall along Roadway name/marked route.
    b. Designated funding route and section number.
    c. County.
    d. Beginning and ending stations of the wall.
    e. Structure number. (New structure numbers are issued for walls that do not reuse any part of the existing structure.)

Location Sketch

1. Provide a sketch that shows four sections of the township.
2. Label the range, township and principle meridian.
3. Provide a north arrow.
4. Call out the structure location.
5. Recheck names of major features on sketch.

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Highway Classification

1. List the following data:
    a. Designated funding route and roadway name/marked route.
    b. Functional Class from the Illinois Structure Information System.
    c. ADT – Present and Future.
    d. DHV – Future.
    e. ADTT (including single and multiple unit trucks).
    f. Design speed.
    g. Posted speed.

Loading

1. Provide a noise wall loading, if applicable.

Design Specifications

1. Provide the applicable design specifications.

Design Stresses

1. Provide the design stresses for Field and Precast units.
2. Provide a separate table for design stresses of existing elements that will be incorporated into new construction.
3. Verify the correct design stresses are shown for the proposed design method.

Upper Left Hand Corner Data

1. Provide a benchmark that matches the structure report and survey data.
2. Provide the existing structure number with the construction year and project name when applicable.
3. Provide a brief description of the existing structure.
4. Indicate the proposed method of traffic control for the proposed construction.
5. Indicate if any items of the existing construction will be salvaged for future IDOT use.

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Profile Grade

1. Provide a profile grade that extends beyond the limits of the structure.
2. Show grade slopes, curve length, elevation and stations of PVC, PVT, & PVI.
3. Verify the profile grade matches the plan and profile sheet.
4. Indicate the centerline of roadway and location of the profile grade line.

Horizontal Curve Data

1. Provide horizontal curve data including PI station, Δ, D, R, L, T, E, PC station, PT station and SE.
2. Indicate superelevation and/or normal crown transition stations if the structure is located within a transition.

Elevation View

1. Show the view looking at the front (exposed) face of the wall.
2. Show the total length of the wall.
3. Provide stations and elevations at the beginning of wall, end of wall and intermediate control points along the top of the wall. (Intermediate control points include points of curvature, kink points, slope break points, expansion joints, wall type transition stations, etc.)
4. Plot the existing and proposed ground line along the front face of the wall.
5. Provide invert elevations and sizes for drainage structures.
6. Indicate the following features for MSE walls:
    a. Top of exposed panel line.
    b. Theoretical top of leveling pad.
    c. Top of coping or traffic barrier.
    d. Generic panel lines if panels are precast.
    e. Ground improvements if applicable.
7. Indicate the following features for cast-in-place (T-type) walls:
    a. Bottom of footing elevations.
    b. Foundation type.
    c. Construction and expansion joint locations.
8. Indicate the following features for soldier pile and permanent sheet pile walls:
    a. Bottom of concrete facing, if applicable.
    b. Transition station between cantilevered and anchored wall, if applicable.
    c. Construction and expansion joints, if applicable.
    d. Driven or drilled soldier pile or sheeting foundation type.
9. Show rustification limits, if applicable.

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Plan View

1. Show a plan layout of the wall that includes the surrounding roadways, utilities, buildings, etc.
2. Provide stations and offsets at the beginning of wall, end of wall, and all intermediate control points.
3. Specify location offsets are referenced to (i.e. front face of wall).
4. Show stationing/flow direction of adjacent roadway/stream.
5. Label and dimension adjacent roadway and topography features.
6. Show ground slopes.
7. Show and label any traffic barriers.
8. Show proposed wall element's approximate layout limits (including footings, soil reinforcement, ground anchors, soil nails, deadmen, etc.).
9. Show temporary construction requirements (sheet piling, soil retention system, etc.), if required for construction.
10. Show R.O.W limits.
11. Plot the boring locations.
12. Show north arrow.
13. Show limits of existing wall, if applicable.

Section Through Wall

1. Provide a typical wall section for each wall type used.
2. Show existing ground line.
3. Show proposed ground lines on the front and back side of the wall.
4. Show existing and proposed utilities.
5. Show R.O.W limits, if a constraint.
6. Show any adjacent surface drainage ditch or gutter.
7. Show maximum exposed and design wall height "H".
8. Major elements of wall system should be shown and labeled.
9. Show temporary construction details (sheet piling, soil retention system, etc.), if required.
10. Label and detail any protection (parapet, fence, etc.) at the top of the wall.
11. Show adjacent roadways with centerline location, PG location, cross slopes, etc.
12. Show the point on the proposed wall that offsets are referenced to (i.e. F.F. of wall).
13. Show existing wall, if applicable.
14. Show aesthetic facing treatment, if applicable.

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15. Indicate the following feature for MSE walls:
    a. Show soil reinforcement.
    b. Show select fill.
    c. Show top of leveling pad 3 ft. – 6 in. min. below finished ground line.
    d. Show limits of reinforced soil mass.
    e. Show “top of exposed panel line”.
    f. Show precast panels or cast-in-place facing.
    g. If cast-in-place facing is utilized, show applicable wall drainage details.
16. Indicate the following features for cast-in-place (T-type) walls:
    a. Show footing 4 ft. min. below finished grade.
    b. Show stem thickness subject to refinement during final design.
    c. Indicate backfill material.
    d. Show applicable wall drainage details.
    e. Show footing width subject to refinement during final design.
17. Indicate the following features for Soldier Pile and Permanent Sheet Pile walls:
    a. If both anchored and cantilevered walls are utilized, provide a typical section for both wall types.
    b. Show concrete facing 2 ft. typical below finished ground line , if applicable.
    c. Show facing thickness, if applicable.
    d. Show timber lagging, if applicable.
    e. Show soldier pile encasement concrete, if applicable (drilled soldier piles).
    f. Show “controlled low strength material”, if applicable (drilled soldier piles in cut situations).
    g. Show applicable wall drainage details.
    h. Show anchor types, if applicable.

Special Considerations

1. Provide any unique construction sequence notes or staging details.
2. Identify and locate all design and construction constraints such as overhead power lines and existing structures.

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2.3.14 Approved Type Size and Location Plans

Approved TSL plans which may contain a range of grade separation and stream crossing structures, as well as retaining walls are a quick reference for bridge planning policy and presentation methods. See Appendix A1.3 for further discussion.

2.3.15 Deliverables

The process of completing and delivering the final complete TSL plan and "agreement" works somewhat differently for a BBS prepared project as opposed to when a District hires a consultant. There are also variations within the processes depending on the type of structure involved (for example there are differences between highway and railway bridges). The delivery process for each main scenario (i.e. BBS or consultant) is given in the following two sections.

2.3.15.1 BBS Prepared TSL Plans

The TSL plan is the means of obtaining agreement from all interested parties on the general bridge configuration prior to the development of the Final plans. To obtain this agreement, the plan should normally be processed as follows:

a. District submits Structure Report and corresponding attachments to BBS in order to initiate TSL plan and SGR preparation.

b. After completion by the Bridge Planning Section, the TSL will be transmitted to the Regional Engineer for review and approval. The plan shall be reviewed for agreement with the Project Report, the development of the Roadway plans and other project requirements. The Regional Engineer's approval assures that agreement. An approved copy, marked with revisions as necessary, shall be returned to the Bridge Planning Section.

c. When levees or Federal civil work projects are affected, the District shall obtain the Corps of Engineers and/or Levee Authority's approval of the plan and so notify the Bridge Planning Section.

d. For railroad and railroad grade separation projects, the revised TSL plans will be transmitted by the district railroad coordinator to obtain the railroad company approval.

e. Stamped approved TSL plan shall be transmitted to the District as the basis for the preparation of the Final design plans and to the Federal Highway Administration for review and approval.

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f. Concurrently with the TSL plan processing, the Bridge Planning Section will obtain construction permits, when required, from the Department of Natural Resources and the U.S. Coast Guard.

g. After approval by the Regional Engineer and other parties with interest, and after all required revisions are made, the Engineer of Bridges & Structures will approve the TSL plan as the basis for the preparation of Final plans and direct that preparation.

h. The Planner shall submit the Scour Evaluation Coding Report, for structures within waterways to: DOT.BBS.BridgeMgmt@illinois.gov.

2.3.15.2 Consultant Prepared TSL Plans

The Regional Engineer is responsible to direct and supervise work performed by consultants. TSL plans developed by consultants are normally processed as follows:

a. The Consultant provides the completed TSL plan to the District where it is reviewed for agreement with the Project Report and other project requirements.

b. At the District's discretion, the TSL plan along with the Structure Report, SGR and corresponding attachments can be transmitted by the Consultant concurrently with the above submittal or separately by the District to the BBS (to DOT.BBS.Planning@illinois.gov). The Bridge Planning Section may approve the TSL with comments or reject the TSL request to re-submittal.

c. For railroad and railroad grade separation projects, after Bridge Planning Section review, the revised TSL will be transmitted by the District railroad coordinator to obtain the railroad company approval.

d. The Consultant shall submit the revised TSL with a disposition of comments to the BBS (to DOT.BBS.Planning@illinois.gov) and after Railroad approval (if required), for stamped approval of the TSL. The stamped approved TSL will be transmitted by the BBS to the Consultant and District as the basis for the preparation of the Final design plans, and FHWA for review and approval.

e. The Regional Engineer may then direct the Consultant to proceed with the final design.

f. The Regional Engineer may wish to delay directing the Consultant to proceed with the Final plans until FHWA approval has been obtained - depending on the degree of complexity or controversy involved in the proposed design.

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g. The consultant shall submit the Scour Evaluation Coding Report, for structures within waterways to: DOT.BBS.BridgeMgmt@illinois.gov

TSL plans prepared by railroad companies or their consultants for railroad structures overpassing highways shall be processed as above except for Item c.

In instances where TSL plans are to be prepared by the State for consultant final design, the plans will be processed according to the previous “BBS Prepared TSL Plans” Section.

As with State design, the Bridge Planning Section will apply for construction permits from the Department of Natural Resources or the U.S. Coast Guard; however, the permit drawings should generally be provided by the Consultant.

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2.4 Structure Reconstruction/Rehabilitation

When the scope-of-work for a structure is rehabilitation/reconstruction as opposed to bridge replacement, there are some special considerations during the planning process not fully covered in previous articles of this section.

2.4.1 Definition and Submittal Requirements

Bridge rehabilitation, such as redecking and widenings, and bridge reconstruction defined as superstructure replacement, require TSL plans but an SGR is not required.

2.4.2 Evaluation Process

The following articles give general guidelines on special evaluation processes for rehabilitation and reconstruction projects during TSL plan development.

2.4.2.1 Abutment Widening

Abutment caps and/or walls are typically widened by either extension in-kind or by providing a cap cantilevered from the existing abutment. The use of a cantilevered cap is normally a function of the structural capacity of the existing abutment, imbalance of the deck system during construction staging, or the structural limitations of the cantilever itself. Since the cantilever method of extension is normally more economical than extension in-kind, a cantilever should be used whenever it is both structurally and economically feasible.

The extension of the abutment cap beyond the existing embankment width by cantilever on a closed abutment leaves the wingwalls inadequate and creates a void between the existing and proposed shoulders adjacent to the abutment cap. An approach shoulder beam treatment was developed to retain the widened embankment. The approach shoulder beam, typically a precast channel section, rests on the modified abutment at one end and a pedestal at the other, and, for additional stability, the approach beam is normally tied into the adjacent slab. The beam length is determined by computing the distance necessary to provide a 2:1 slope from the end of the existing wingwall to the shoulder edge at the end of approach beam. The minimum beam length to be used should be 19 ft. – 11 in. and when additional length is required it should be increased in increments of 4 ft. – 0 in.

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2.4.2.2 Pier Widening

The widening of solid piers is typically accomplished by one of three methods. The first method utilizes cantilever construction and is accomplished by rebuilding the pier cap to the required width by extending cantilever arms out past the pier shaft. This method is limited by the capacity of the pier to accept the additional dead load and by the length of the cantilever arm.

The second method utilizes open or encased pile bent construction to extend the pier to the necessary width. This method is limited by the capacity of the pile bent construction.

The third method for pier extension is an extension of the pier with the same type of construction as used on the original pier. This method is the costliest of the three options and is generally used when the other options prove unacceptable for either structural or physical reasons.

Other pier types, such as drilled shaft or open column bents, are typically widened with in-kind construction. Crash walls and footings shall be connected to the existing crash walls and footings in all cases. On framed piers with cantilevered caps, the pier caps shall not be attached and shall be designed accordingly. All other pier types shall have a full connection at the cap. The normal distribution of dead and live loads to the beams or girders of both the existing and widened portions may be assumed when following these criteria.

The intended design approach consistent with the planning review shall be stated on the TSL plan.

2.4.2.3 Semi-Integral Abutments

A semi-integral jointless abutment may be appropriate for rehabilitation and reconstruction projects which incorporate an existing abutment not originally designed to be semi-integral. For guidance related to existing abutment conversion to semi-integral configuration for rehabilitation or reconstruction projects, refer to Section 3.8.4.1. The rehabilitated abutment may continue to use expansion bearings, but the beams can be integral with the backwall, eliminating the deck's expansion joint at the abutment. The bridge expansion is provided for as in a typical integral abutment at the ends of the approach slab away from the bridge. The backwall shall be partially removed and the removal line should be located as indicated in Fig. 3.8.4-1, Fig. 3.8.4-3, Fig. 3.8.4-4 or Fig. 3.8.4-7. Loads transferred from the bridge approach slab to the substructure shall be considered to act through the bearing, not the backwall. Existing abutments should not be considered for this type of rehabilitation when they have a history of inward rotational problems, or when partial removal of the existing backwall is either unfeasible or undesirable. Existing abutments which have been built in a manner that have demonstrated stability with

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no known vulnerabilities, such as a standard open abutment or stub abutments on spread footings, may be considered for a semi-integral abutment application.

2.4.2.4 Retrofit of Existing Welded Cover Plates

Existing steel girder bridges having either positive moment or negative moment welded cover plates shall use the following retrofitting policy when evaluating the reuse of the existing steel girders for deck repair or deck replacement projects.

Deck Repair Projects

Retrofitting the coverplate ends subject to fatigue is not required unless known fatigue cracks exist.

Deck Replacement Projects

Fatigue prone regions identified during bridge inspection or Bridge Condition Report preparation shall be investigated for retrofitting. Otherwise, retrofit of existing cover plate ends subject to fatigue shall be evaluated especially on bridges located on a route that is identified as carrying unusually heavy truck loads (e.g. quarry loads, landfills, etc.) or a significant number of permit loads. The designer should contact the District Bureau of Operations for guidance concerning these routes.

Consideration to retrofit all coverplate ends subject to fatigue shall be given on all bridges located on a route that has an ADTT (Average Daily Truck Traffic) exceeding 1500 trucks. For bridges meeting this criterion, a fatigue evaluation shall be completed to calculate the remaining mean fatigue life utilizing Section 7 (Fatigue Evaluation of Steel Bridges) of the latest AASHTO “Manual for Bridge Evaluation” and interims, and the following cases a-c shall be investigated:

a. When the calculations indicate the mean fatigue life is 50 years or greater, retrofitting is not required.
b. When the calculations indicate the mean fatigue life is less than 50 years, the fatigue life should be re-evaluated using composite action on the entire steel girder length.
c. If (b.) still provides a mean fatigue life less than 50 years, the coverplate ends subject to a fatigue life of less than 50 years shall be retrofitted. An economic analysis should be completed comparing the cost of utilizing the existing steel girder with all associated girder repairs, retrofitting and painting considered, versus superstructure replacement to justify the decision to retrofit. Contact the BBS Estimates Engineer for the current estimated cost of “Structural Steel Repair”.

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Once the decision to retrofit the coverplate ends has been made, the design of the bolted plate thickness and number of bolts shall be based on providing 100% of the existing girder flange area. Details in Section 2.12 of the Structural Services Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Repairs) shall be used. See Section 3.3 for further information on cover plate design.

2.4.2.5 Distortion-Induced Fatigue Evaluation

Distortion-induced fatigue has initiated failures in some existing bridges. Rehabilitation projects should evaluate deficient details and potential associated fracturing, and retrofit or replace material to eliminate defects and mitigate highly susceptible conditions, such as cross-frame connection plates not positively attached to flanges. Detailing and design procedures to avoid distortion-induced fatigue in new structures are mandated in AASHTO LRFD 6.6.1. For existing tension flanges, bolted rather than welded connections may be required to satisfy fatigue stress range limits.

2.4.2.6 Re-Decking, Widenings and New Superstructures on Existing Substructure Units

See Section 2.1 for the appropriate design specification to be used. For further guidance, analysis and requirements, see Section III of "Bridge Condition Report Procedures and Practices" available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning.

2.4.3 TSL Plan Preparation and Approved TSL Plans

Some Approved TSL plans for rehabilitation projects are included in the list of online TSLs discussed in Appendix A1.3.

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2.5 Utility Attachments

The policy on the "Accommodation of Utilities on Right of Way" of the Illinois State Highway System, 92 Illinois Administrative Code 530, governs the attachment to and assessment for utility installations to bridges and to traffic structures under the jurisdiction of the Department. A copy of the most recent publication of this policy can be obtained from the Central Bureau of Operations.

All utility companies, whether private, cooperative or municipally-owned, who wish to attach their facilities to bridges or to traffic structures under the jurisdiction of the Department are subject to assessment charges. The administration and regulation of utility attachments are functions of the Bureau of Operations and of the Regional Engineer.

If the Regional Engineer approves of the proposed attachment to the structure, an application for a permit for utility attachment to a bridge or structure shall be submitted to the Central Bureau of Operations for review of compliance with policy and method of attachment. If approved by the Central Bureau of Operations, the permit will be forwarded to the Bridge Planning Section of the Bureau of Bridges & Structures for structural analysis and computation of assessment charges. Copies of the Computation of Cost Assessment will then be sent directly to the Regional Engineer with a copy of the letter of transmittal to the Bureau of Operations and, in the case of new structures, to the Bureau of Construction for further processing.

In instances where it is desirable to attach a utility to a proposed new structure undergoing design or reattach to a proposed bridge reconstruction, a permit and assessment should be processed as above; however, the plans and details of the attachments should be transmitted to the Bureau of Bridges & Structures early in the design stage so that provision for the attachment may be incorporated into the Contract plans. Such provisions will generally be limited to:

1. Including the weight of the attachment in the design loads (e.g. pounds per lineal foot, total pounds for isolated attachments, etc.).
2. Providing concrete inserts for anchor bolts.
3. Providing openings or passageway thru or around structural elements where structurally practical.
4. Providing material type of brackets and connections to verify incompatibility with galvanized or metallized bridge components (e.g. steel versus aluminum brackets, etc.)

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2.5.1 Computation of Cost Assessment

A utility company whose facility is to be carried on a structure will be assessed an amount equal to the product of the ratio of the weight of facility to the live load for which the structure was designed and the cost of applicable structural items which contribute to the longitudinal carrying capacity of the structure. Assessment will be made for the full capacity of attachment; for example, if six telephone ducts are installed, assessment will be made for all six ducts, even though initially only two ducts might be utilized.

The weight of facility shall include all conduits, cables and pipes, completely filled, and all material necessary for attachment to structure.

The live load for which the structure is designed is either present-day loading or any condition of loading previously used in the design of the existing structure. If sidewalk loading was or is incorporated in the design of the structure, it shall be included in arriving at a proper design live load ratio. Also, whenever the weight of a utility attachment is included in the design of a structure, the utility weight shall be included in the design live load ratio.

All items that contribute to the longitudinal carrying capacity of the structure element shall be included.

Superstructure:

All items of the superstructure exclusive of the roadway deck slab, sidewalks and railing shall be included. However, when composite action is utilized in the design, the roadway concrete (deck slab and reinforcement) shall be included.

Substructure:

All material in the piers and abutments, i.e., concrete, reinforcing, foundations and piling shall be included. Wingwalls shall also be included when they are tied to the main abutment wall and/or are supported on a monolithic footing. Seal coats, metal shoes and cofferdams necessary to facilitate foundation construction shall not be included.

Whenever possible, cost assessments shall be based on final quantities and actual contract prices. When final quantities and actual contract prices are not available, the plan quantities and present-day estimated prices shall be used. When present-day estimated prices are used, the cost of applicable structural items shall be prorated to time of actual construction by application of documented domestic

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cost indices. Also, when estimated prices are used, 10% of the cost of applicable structural items shall be included in the total cost of structure to cover engineering contingencies. The 1913 Cost Index shall be used if the structure was built prior to the year 1913.

In no case shall the assessment to utility companies to support their facilities be less than $300.

When contract prices are used in the cost assessment computations, the cost assessment formula is as follows:

$$\text{Assessment} = \frac{a}{b} \times c$$

When estimated prices are used in the cost assessment computations, the cost assessment formula is as follows:

$$\text{Assessment} = \frac{a}{b} \times \frac{e}{f} \times (c + d)$$

Where:

a = Weight of Utility
b = Design Live Load
c = Cost of Applicable Structural Items
d = 10% for Engineering and Contingencies
e = Cost Index - Year of Construction
f = Cost Index - Present Day

2.6 Permits Projects

Permits Projects occur when a municipality, county, city, private developer, etc. would like to encroach on, build on, or build underneath an existing IDOT structure or right-of-way (ROW). Typically, the primary approval for a Permit Project is given through the Region or District. For most Permits Projects, a TSL is required. The Bureau of Bridges & Structures is responsible for verifying that the TSL meets all Departmental policies for structures. The actual process by which Permits Projects are finally approved varies from District to District. As such, the District should initially be contacted for approval coordination of these projects during their initial phases.

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Section 3 Design

3.1 General

This Bridge Manual provides structure design policies, details and policies for the design of structures using the current AASHTO LRFD Bridge Design Specifications.

3.1.1 Plan Presentation

The Design plans and Specifications are the means of communication between the design engineer and the contractor. For the structure to be built in accordance with the structural design and governing policies, the plans should be both accurate and explicit.

Electronic 11 in. x 17 in. plans with electronic signature and seal are archived for permanent record. The Computer Aided Design, Drafting, Modeling & Deliverables Manual (available at the primary IDOT CADD Page) shall be referenced for proper font, font size, line weights and general presentation of plans.

Multiple-structure contracts shall not have bridge plan sheets combined into a single set; each individual structure shall have its content (as described in Section 3.1.2) grouped separately with no combined tables and details. For items constructed between adjacent structures (i.e. slope protection systems), clear dimensions and boundaries on the plans shall be used to associate pay items to the appropriate structure. Dual adjacent structures typically are combined into a single set of plans, share a variety of details and utilize double dimensioning when necessary.

3.1.2 Content of Bridge Plans

Bridge Plans are composed of sheets covering the following aspects of a structure and are usually presented in the order given below on IDOT projects:

1. General Plan and Elevation (GP&E)
2. General Data (except for simple structures)
3. Footing Layout (if required)
4. Stage Construction Details (if required)
5. Temporary Barrier or Temporary Railing Details (if required)
6. Top of Slab Elevations
7. Top of Approach Slab Elevations

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8. Superstructure (Plan and Cross Section)
9. Superstructure Details
10. Diaphragm Details
11. Bridge Approach Slab Details
12. Bridge Railing Details
13. Expansion Joint Details
14. Drainage Scuppers (if required)
15. Girder and Framing Details (Steel or Concrete)
16. Bearing Details
17. Abutment Details
18. Pier Details
19. Foundation Details (includes piles)
20. Concrete Parapet Slipforming Option (if required)
21. Bar Splicer Assembly / Mechanical Splicer Details
22. Boring Logs or Subsurface Data Profile Plot

3.1.3 General Notes

The following plan notes are required, when applicable, to supplement the Standard Specifications. The notes marked with an asterisk (*) should be placed near the associated detail as indicated.

For plan notes below with a letter following its designated number set (e.g. 1a, 1b…, or 5a, 5b…), the designer should only include the number/letter combination note(s) that apply to the specific project (e.g. a project may need note 1a but not 1b nor 1c).

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No	*Note*		*Application*
1a.	All new structural steel shall be AASHTO M270 Grade XXW (except expansion joints and expansion bearings which shall be AASHTO M270 Grade XX).		When structural steel is unpainted AASHTO M270 Grade 50W or HPS70W weathering steel.
1b.	All new structural steel shall be galvanized. See Special Provision for "Hot Dip Galvanizing for Structural Steel".		All structures with galvanized steel, when all structural steel is to be Hot-Dip Galvanized.
1c.	All new structural steel shall be metallized. See Special Provision for "Metallizing of Structural Steel."		When all structural steel is to be metallized.
2.	Calculated weight of Structural Steel =		For steel structures. If multiple steel grades are used, show quantity for each. Show quantity to nearest 10 lbs.
3.	No field welding is permitted except as specified in the contract documents.		All structures with primary steel members.
4.	Load carrying components designated "CVN" shall conform to the Charpy-V-Notch Impact Energy Requirements, Zone 2.	*	Components designed for tensile stress require at least a minimum toughness to avoid crack propagation. These components include wide flange beams, tension flanges and webs of plate girders, all splice plate material except fill plates, and bracing designed for live load in curved or highly skewed (> 60 degrees) structures. Place note on sheet detailing structural steel.
5a.	Fasteners shall be ASTM F 3125 Grade A325 Type 1, mechanically galvanized bolts in painted areas. Bolts ___in. diameter, holes ___in. diameter, unless otherwise noted.		All structures with painted steel. If ASTM F3125 Grade 490 bolts are required, preface note with "Except as otherwise specified" and add notes where applicable.
5b.	Fasteners shall be ASTM F 3125 Grade A325 Type 1, mechanically galvanized bolts in painted or coated metallized areas. Fasteners shall be ASTM F 3125 Grade A325 Type 1, hot-dip galvanized in uncoated metallized areas. Fasteners shall be ASTM F3125 Grade A325 Type 3 weathering steel bolts in unpainted weathering steel areas. Bolts ___in. diameter, holes ___in. diameter, unless otherwise noted.		All structures with weathering steel or metallized areas. If ASTM F3125 Grade 490 bolts are required, preface note with "Except as otherwise specified" and add notes where applicable. Mechanically galvanized bolts are required in metallized areas where the bolts will be coated with paint or sealer. Hot-dip galvanized bolts are required in metallized areas where the bolts will remain uncoated.

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5c.	Fasteners shall be ASTM F 3125 Grade A325 Type 1, hot-dip galvanized. See Special Provision for "Hot Dip Galvanizing for Structural Steel." Bolts ___in. diameter, holes ___in. diameter, unless otherwise noted.	All structures with galvanized steel. If ASTM F3125 Grade 490 bolts are required, preface note with "Except as otherwise specified" and add notes where applicable.
5d.	Fasteners shall be ASTM F 3125 Grade A325 Type 1, hot-dip galvanized bolts in uncoated metallized areas. Bolts ___in. diameter, holes ___in. diameter, unless otherwise noted. See special provision for "Metallizing of Structural Steel".	All structures with metallized steel. If ASTM F3125 Grade 490 bolts are required, preface note with "Except as otherwise specified" and add notes where applicable.
6.	Prior to pouring the new concrete deck, all heavy or loose rust, loose mill scale, and other loose detrimental foreign material shall be removed from the surfaces in contact with concrete (SSPC – SP3 standards). Tightly adhered paint may remain unless otherwise noted. Removal shall be accomplished by methods that will not damage the steel and the cost will be paid for according to Article 109.04 of the Standard Specifications. As directed by the Engineer, existing construction accessories welded to the top flange of beams and girders shall be removed. The weld areas shall be ground flush and inspected for cracks using magnetic particle testing (MT) or dye penetrant testing (PT) by qualified personnel approved by the Engineer. Any cracks that cannot be removed by grinding ¼ in. deep shall be identified and reported to the Bureau of Bridges & Structures for further disposition. The cost of removing welded accessories, grinding and inspecting weld areas and grinding cracks will be paid for according to Article 109.04 of the Standard Specifications.	For bridge rehabilitation projects where the complete or partial removal of existing concrete deck is specified.

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7.	The Contractor shall test the existing welds by non-destructive methods within 2 ft. of the end of the existing cover plates for cracks after removal of the existing concrete deck. Dye penetrant (PT), magnetic particle (MT), or other approved testing method shall be performed by qualified personnel approved by the Engineer. If cracks are found, report them to the Bureau of Bridges & Structures for disposition. The cost of testing is included in Removal of Existing Concrete Deck. The cost of crack repair, if necessary, will be paid for according to Article 109.04 of the Standard Specifications.	For existing steel beams or girders with welded cover plates that are to remain and be reused after partial or complete deck replacement.
8.	All (cross-frames or diaphragms) shall be installed as steel is erected and secured with erection pins and bolts except as otherwise noted. Individual (cross-frames or diaphragms) at supports may be temporarily disconnected to install bearing anchor bolts.	* Straight steel girder bridges. “Except as otherwise noted” typically includes stage construction region with differential deflection. Place note with framing plan or appropriate bracing details. (Use “cross-frames" or “diaphragms” as applicable.)
9.	All (cross-frames or diaphragms) between beams or girders shall be installed with erection pins and bolts in accordance with the erection plan approved by the Engineer. Individual (cross-frames or diaphragms) at supports may be temporarily disconnected to install bearing anchor bolts.	* Horizontally curved steel bridges, including those considered equivalently straight as per Article 4.6.1.2.4b of the AASHTO LRFD Bridge Design Specifications, and as applicable for high skews, flares, cross girders, etc. that require an erection plan prepared by a licensed structural engineer. Place note with framing plan or appropriate bracing details. (Use “cross-frames" or “diaphragms” as applicable.)
10.	Reinforcement bars designated (E) shall be epoxy coated.	Place in General Notes only.
11.	Reserved.	

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12. If the Contractor elects to use cantilever forming brackets on the exterior beams or girders, the brackets shall be placed at the same locations as required for the hardwood blocks in Article 503.06(b) of the Standard Specifications. If additional cantilever forming brackets are required, hardwood blocking shall be wedged between the exterior and first interior beam at each of these additional bracket locations.

    When the exterior beam overhang on a steel girder exceeds 3 ft. – 3 in. or when the overhang exceeds half the beam spacing.

13. The finishing machine rails shall be placed on the top of the top flange of the exterior beams within the deck pour. Beam blocks shall be placed between beams at all tie locations in each bay for the full width of the deck pour.

    When the total beam depth is less than 30 inches.

14. The concrete for bridge decks finished according to Article 503.16(a) of the Standard Specifications shall be placed and compacted parallel to the skew in uniform increments along centerline of bridge. The machine used for finishing shall be set parallel to the skew for striking off and screeding the concrete.

    For all decks on steel or concrete girder structures with skew angle 45 degrees or greater or structures with skew angle exceeding 30 degrees and the ratio of the width of deck pour (out-to-out deck or between longitudinal bonded joints) to the span length exceeding 0.8.

15. When the deck pour is stopped for the day at one or more of the transverse bonded construction joints in the deck pouring sequence as shown, the next pour shall not be made until both of the following are met:

    1. At least 72 hours shall have elapsed from the end of the previous pour.

    2. The concrete strength shall have attained a minimum flexural strength of 675 psi or a minimum compressive strength of 4000 psi.

    * When a deck pouring sequence is shown on the plans. Place note on sheet with deck pouring sequence.

    When the design compressive stress for the bridge deck is 5000 psi (typically due to Type SM railing) then part 2 of this note shall read "The concrete strength shall have attained a minimum flexural strength of 800 psi or a minimum compressive strength of 5000 psi."

16. Up to ¼ inch to be ground off the bridge deck and the bridge approach slabs. The Profile Grade shows the final grade after grinding.

    * For structures with bridge deck smoothness grinding. Place note beneath Profile Grade Figure on GPE.

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17.	The Contractor shall make allowance for the deflection of forms, shrinkage, and settlement of falsework, in addition to allowance for dead load deflection. Forms for deck slab shall be removed prior to placement of bridge approach slab.		R.C. Slab bridges.
18.	Protective coat shall not be applied to surfaces to which Waterproofing Membrane System is applied.		When waterproofing membrane is specified.
19.	Slipforming of the parapets is not allowed.		When the structure does not meet slipforming criteria or slipforming is not allowed by the District.
20.	(Finger plate or Modular) expansion joints shall be assembled in their final relative position with the ends in place for shop inspection and acceptance.	*	Finger plate and modular expansion devices. Insert the item(s) within the ( ) that apply to the project. Place note with finger plate or modular joint details.
21.	The structural steel plates of the Bearing Assembly shall conform to the requirements of AASHTO M270 Grade XX (AASHTO M270 Grade XXW.)	*	The designer is alerted that Grade 36W does not exist, therefore should not be specified. Insert appropriate Grade for XX. Place note on applicable HLMR (pot or disc) or elastomeric bearing sheet.
22.	Bearing seat surfaces shall be constructed or adjusted to the designated elevations within a tolerance of ⅛ in. (0.01 ft.). Adjustment shall be made either by grinding the surface or by shimming the bearings.		All continuous steel beam structures.
23.	Two ⅛ in. adjusting shims shall be provided for each bearing in addition to all other plates or shims and placed as shown on bearing details.	*	All continuous steel beam structures. For Type I elastomeric bearings, shims should be detailed between the bearing and the flange, and not extend beyond their mutual contact area. Place note on applicable bearing detail sheet.

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24.	All (embedded and separate) bearing plates, side retainers, anchor bolts, nuts, washers and pintles shall be galvanized according to AASHTO M111 or M232 as applicable.	* -When there are bearings for structures with metallized or galvanized steel beams (Scenario 1: for metalized steel beams, metallize fixed bearings components, and galvanize all other components; Scenario 2: for galvanized steel beams, galvanize all components), or -For all PPC I and Bulb T-beam structures with bearings that have metal parts (Scenario 3: for PPC I/IL/Bulb-T beams, galvanize the embedded planes and any separate external plates). Place note on applicable bearing detail sheet.
25.	H.S. bolts in bearing assembly shall be galvanized according to ASTM B 695 Class 50.	* For all PPC I, Bulb T-beam and IL Beam structures with H.S. bolts in bearing assemblies and when specified for structures with steel beams. Place note on applicable bearing detail sheet.
26.	The anchor bolt sizes and grades shown constitute a calculated seismic structural fuse. Substitution of higher diameter and/or grade anchor bolts will not be allowed.	* When the seismic load case controls the design of anchor bolts in zones other than Seismic Performance Zone 1. Place note on applicable sheet with anchor bolt details.
27.	Concrete Sealer shall be applied to the designated areas of the _____. The Concrete Sealer shall be a “film forming” type for horizontal surfaces.	For new substructures per Sections 3.8.1 and 3.9.1.
28.	Excavation behind existing abutment walls shall be performed to balance front and back soil pressure before removing the existing superstructure. For staged structures, the Contractor shall sawcut the upper portion of the existing abutment at the stage removal line before Stage I removal to ensure the remaining portion will not be prematurely damaged.	Removal of existing closed abutment structures (restrained top and bottom).
29.	Backfill shall be placed behind the abutment after the superstructure has been poured and falsework removed. See Article 502.10 of the Standard Specifications.	* Closed abutments (restrained top and bottom). Place note on abutment sheet.

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30. The Seal coat design thickness is estimated based on the Cofferdam Design Water Elevation (CDWE) shown. Final cofferdam design, details and `seal coat thickness shall be submitted to the Engineer for approval.

When a cofferdam with seal coat is shown on the plans.

31. Piles shall be driven through _____ diameter precored holes extending to elevation _____ according to Article 512.09(c) of the Standard Specifications. Cost included in Driving Piles.
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* When precoring of pile locations is specified. (Typical locations are but not limited to: new embankment where down drag is not accounted for in design; MSE abutments on integral abutments; or when stiff soil layer needs to be penetrated for pile driving when specified in SGR.) Place note with pile data on appropriate substructure sheet.

32. Piles shall be driven through _____ diameter precored holes extending to elevation _____ according to Article 512.09(c) of the Standard Specifications except that the void space outside the pile shall be filled with bentonite according to the manufacturer's recommendations to achieve a Qu of 1.5 tsf. Cost included in driving piles.

* When precoring of pile locations is specified at integral abutment with a Qu of 3.0 tsf or greater. Place note with Pile Data on appropriate substructure sheet. HP 18 and HP 16 piles shall have a 36" diameter precored hole. HP 14 piles shall have a 30" diameter precored hole. HP 12 piles and smaller shall have a 24" diameter precored hole.

33. Granular Backfill behind the abutments shall be compacted according to Article 205.06 of the Standard Specifications.

* For structure types where: granular backfill behind the abutments has railing or guardrail posts driven into the backfill, or structures requiring backfill compaction for dense-material springs for seismic design.

34. If a portion of the drilled shaft web walls is under water, reinforcement may be placed underwater into forms. Concrete shall be tremied according to Article 503.08 of the Standard Specifications to an elevation of 1'-0" above the water line at the time of construction.

* When a portion of drilled shaft web walls is shown to be below the EWSE. Place note on applicable pier sheet.

35. If the Contractor chooses to alter the temporary cantilevered sheet piling design requirements shown on the plans, a design submittal including plan details and calculations will be required for review and acceptance by the Engineer.

* When a cantilever sheet piling design is shown on the plans. Place note on sheet with sheet piling details.

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36.	The Contractor shall connect the first sheet to the existing abutment wall to ensure stability of sheets driven to the top of the existing footing. This connection shall be reviewed and accepted by the Engineer and included in the cost for Temporary Sheet Piling.	* When a footing interferes with required sheet piling penetration. Place note on sheet with sheet piling details.
37.	A cantilevered sheet piling design does not appear feasible and additional members or other retention systems may be necessary. The Contractor shall submit a temporary soil retention system design including plan details and calculations for review and acceptance by the Engineer.	* When a temporary soil retention system is on the plans. Place note on sheet with temporary soil retention system details.
38.	Reserved.	
39.	Slope wall shall be reinforced with welded wire fabric, 6 in. x 6 in. – W4.0 x W4.0, weighing 58 lbs. per 100 sq. ft.	* For all concrete slope walls. Place note with slope wall details.
40.	Layout of the slope protection system may be varied to suit ground conditions in the field as directed by the Engineer.	Stream crossings only.
41.	The embankment configuration with 60 ft. berm shown shall be the minimum that must be placed and compacted prior to construction of the abutments.	All structures requiring new or widened embankment cones.
42.	The Contractor shall obtain a construction permit from the Illinois Department of Natural Resources (IDNR), Office of Water Resources for any temporary construction activity placed in the water except cofferdams. This shall include the placement of material for run-arounds, causeways, etc. Any permit application by the Contractor shall refer to the IDNR 3704 Floodway Construction permit number allowing permanent construction as shown in the contract plans.	When temporary construction features are placed within public waters. Also place the term “Public Waters” in the title block of the Design plans to alert the Contractor of this additional responsibility as noted.

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43.	Plan dimensions and details relative to the existing structure have been taken from existing plans are subject to nominal construction variations. The Contactor shall field verify existing dimensions and details affecting new construction and make necessary approved adjustments prior to construction or ordering of materials. Such variations shall not be cause for additional compensation for a change in scope of the work, however, the Contractor will be paid for the quantity actually furnished at the unit price bid for the work.	Widening, repair or rehabilitation of existing structures.
44.	Existing reinforcement shall be cleaned, straightened, and incorporated into the new construction. Cost included with Concrete Removal.	* When existing reinforcement is to be reused during rehabilitation projects. Place note on sheet where reinforcement is to be reused.
45.	Existing Name Plate shall be cleaned and relocated next to new Name Plate. Cost included with Name Plates.	* When new name plates are being added to rehabilitated bridges with existing name plates. Place note under name plate detail.
46.	Cleaning and field painting of structural steel shall be done under a separate painting contract.	When painting existing steel to be delayed to a separate paint contract.
47.	The existing structural steel coating contains lead. The Contractor shall take appropriate precautions to address the presence of lead on this project.	Steel structures erected prior to 1986 (or as determined from existing plans) with lead-based primer.
48.	The existing bearings contain lead plates. The Contractor shall take precautions to deal with the presence of lead on this project.	When lead plates under existing bearings are to be removed and disposed of.
49.	The Inorganic Zinc Rich Primer / Acrylic / Acrylic Paint System shall be used for shop and field painting of new structural steel except where otherwise noted. The color of the final finish coat for all interior steel surfaces shall be gray, Munsell No 5B 7/1. The color of the final finish coat for the exterior and bottom flange of the fascia beams shall be (**).	Painting new steel as part of Furnishing and Erecting Structural Steel. This is the preferred paint system when OZ/E/U is not required. **Colors for fascias: 1. Interstate Green, Munsell No. 7.5G 4/8 2. Reddish Brown, Munsell No. 2.5YR 3/4 3. Blue, Munsell No. 10B 3/6 4. Gray, Munsell No. 5B 7/1

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50. The Organic Zinc Rich Primer / Epoxy / Urethane paint system shall be used for painting of new structural steel except where otherwise noted. The entire system shall be shop applied, with the exception that the exterior surfaces and bottom of the bottom flange of the fascia beams, masked off connection surfaces, and field installed fasteners, shall all be touched up and finish coated in the field. The color of the final finish coat for all interior steel surfaces shall be gray, Munsell No. 5B 7/1. The color of the final finish coat for the exterior and bottom flange of the fascia beams shall be (**).

Painting new steel (shop applied paint system) as part of Furnishing and Erecting Structural Steel. This system is required by traffic control considerations or cold weather considerations.

**Colors for fascias:

1. Interstate Green, Munsell No. 7.5G 4/8
2. Reddish Brown, Munsell No. 2.5YR 3/4
3. Blue, Munsell No. 10B 3/6
4. Gray, Munsell No. 5B 7/1

51. Structural steel shall be painted for a distance equal to the depth of the embedment into the concrete cap plus 18 in. Painted areas shall be primed in the shop with a Department-approved zinc rich primer. Field painting will not be required.

New weathering steel with integral or semi-integral abutments.

52. All structural steel and exposed surfaces and bearings within a distance of (*) ft. each way from the deck joints shall be painted using the (**) paint system as specified in Section 506 of the Standard Specifications.

New weathering steel on structures with deck joints.
*The distance shall be 3 times the depth of the beams or girders, not exceeding 10’ and rounded up to the nearest foot.

**Place “Inorganic Zinc-Rich/Waterborne Acrylic” in the general note when the IZ/AC/AC system is required. Place “Organic Zinc-Rich/Epoxy/Urethane” in the general note when the OZ/E/U system is required.

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53a.	The fascia and underside of the exterior beams and their associated splice plates shall be shop painted with field touch up. The color of the final finish coat of paint shall be (*). See Special Provision for "Hot Dip Galvanizing for Structural Steel".	When fascias of galvanized steel structures are to be painted for aesthetic purposes. *Colors for fascias: 1. Interstate Green, Munsell No. 7.5G 4/8 2. Reddish Brown, Munsell No. 2.5YR 3/4 3. Blue, Munsell No. 10B 3/6 4. Gray, Munsell No. 5B 7/1
53b.	Exterior fascia and bottom of bottom flange areas shall be metallized and shop painted with System (*). The color of the final finish coat of paint of fascia areas shall be (**). See special provision for "Metallizing of Structural Steel." (The interior metallized areas shall be painted with System 1.)	When areas are required to be metallized and painted. * 2 or 3 (as determined from special provision and the owner). **Colors for fascias: 1. Interstate Green, Munsell No. 7.5G 4/8 2. Reddish Brown, Munsell No. 2.5YR 3/4 3. Blue, Munsell No. 10B 3/6 4. Gray, Munsell No. 5B 7/1 5. Federal Color Standard 595a 20045 (weathering steel bridges only) Add sentence in parenthesis when applicable.
54	All structural steel girders, cross-frames, and exposed surfaces of bearings within a distance of (*) ft. from the expansion joint shall be metallized. See special provision for "Metallizing of Structural Steel." (**)	When only areas near deck joints are required to be metallized, such as weathering steel bridges with metallized beam ends. Use only when specified by District or BBS. *Calculate the lesser of three times the beam depth or 10 feet and place this number in the general note. Round up to a whole number of feet. (**) Fascia beam areas of weathering steel that have been metallized shall be painted Reddish Brown, Munsell No 2.5YR 3/4 or Federal Color Standard 595a 20045 (weathering steel bridges only).
55.	All new structural steel shall be shop painted with an inorganic zinc rich primer per AASHTO M 300, Type 1.	Painted steel structures with only a small amount of new steel being added. (e.g. diaphragm replacement, additional beam lines, etc.)
56.	Expansion joint plates shall be shop painted with the inorganic zinc rich primer per AASHTO M300, Type 1.	For galvanized steel structures with expansion joints containing steel plates.

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57. Cleaning and painting of the existing and new structural steel shall be as specified in the special provision for "Cleaning and Painting Existing Steel Structures". All existing steel shall be cleaned per Near White Blast Cleaning – SSPC-SP10. All existing steel shall be painted according to the requirements of Paint System 1 - OZ/E/U. The color of the final finish coat for all interior steel surfaces shall be Gray, Munsell No 5B 7/1. The color of the final finish coat for the exterior and bottom flange of the fascia beams shall be (**).

Painted steel structures, where additional beam lines are being added.

**Colors for fascias:
1. Interstate Green, Munsell No. 7.5G 4/8
2. Reddish Brown, Munsell No. 2.5YR 3/4
3. Blue, Munsell No. 10B 3/6
4. Gray, Munsell No. 5B 7/1

58. Cleaning and painting of the existing structural steel shall be as specified in the special provision for "Cleaning and Painting Existing Steel Structures". All beams, bearings and other structural steel within (*) ft (measured along the beam) of either side of deck joints shall be cleaned per Near White Blast Cleaning – SSPC- SP10. The exterior surfaces and bottom of the bottom flange of the fascia beams shall be cleaned per Commercial Grade Power Tool Cleaning – SSPC- SP15.

    The designated areas cleaned per Near White Blast Cleaning and per Commercial Grade Power Tool Cleaning shall be painted according to the requirements of (**). The color of the final finish coat for all interior steel surfaces shall be Gray, Munsell No 5B 7/1. The color of the final finish coat for the exterior and bottom flange of the fascia beams shall be (***).

When painting existing steel structures.

*5 ft. minimum or as required to repair damaged coatings

**See flowchart for correct paint system in the All Bridge Designers Memorandum for "Cleaning and Painting Existing Steel Structures."

***Colors for fascias:
1. Interstate Green, Munsell No. 7.5G 4/8
2. Reddish Brown, Munsell No. 2.5YR 3/4
3. Blue, Munsell No. 10B 3/6
4. Gray, Munsell No. 5B 7/1

59. A minimum of (*) air monitor(s) will be required to monitor abrasive blasting operations at this site. See special provision for "Containment and Disposal of Lead Paint Cleaning Residues."

Air monitors are required for all structures with lead abatement that have sensitive receptors within 1000 feet or 5 times the bridge height. See special provision for additional details.

*Number as determined by Department Policy.

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60.	Containment of cleaning residue is required to control nuisance dust. See special provisions.	For structures getting repainted that do not contain lead.
61.	SSPC QP1 (and QP2) Contractor Certification is required for this Contract.	For all paint contracts where the main item of work is painting, IDOT requires SSPC QP1. For structures with existing lead paint, SSPC QP1 and QP2 are required.
62.	The Contractor shall submit calculations and details demonstrating the structural integrity of the bridge is maintained under the additional imposed loads of the containment system. See special provisions.	On bridge units within a structure having a span greater than 200 feet, truss bridges, or moveable bridges with a containment system.

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3.1.4 Reinforcement Presentation

On any plan sheet which presents drawings for a portion of the bridge structure such as a pier, all reinforcement bars pertinent to that pier shall be detailed and billed on that sheet.

In no case shall the same designation be used for reinforcement bars of a different size, length or shape when they are employed in elements of the substructure or superstructure.

If a horizontal reinforcement bar in an abutment carries an “h5” designation and an “h” bar of the same size, length and shape is used in the design of a pier under the same structure, this latter bar shall also carry an “h5” designation unless the structure is of such magnitude as to make this coordination impractical. Bars of like designation (such as “h”) shall be numbered in sequence as h, h_1, h_2, etc.

When detailing lengths of reinforcement bars, consideration shall be given to transportation and handling and, where extreme lengths are contemplated, to availability and special orders.

All bar sizes #3 through #11 are readily available in lengths up to 60 ft. However, sizes #3 and #4 of more than 40 ft. tend to bend in handling and should be avoided. Sizes #5 through #18 in lengths exceeding 60 ft. can be rolled at mills by special order. In any circumstance, 70 ft. should be considered the maximum limit. For shipping and handling convenience, 50 ft. lengths should be considered the practical limit for all conventional structures.

When appropriate for the proposed schedule of letting, BBS encourages the designer to call several IDOT approved qualified producers/suppliers (listed on the IDOT Bureau of Materials website) when specifying #14 bar sizes and larger due to potential supply issues for larger bar sizes/lengths/quantities.

When the location of bar splices is arbitrary, as in the case of the longitudinal reinforcement of deck slabs on stringers, the following lengths are preferred:

#6 bars and up …… 36 ft.
#4 & #5 bars …… 30 ft.

Tension development lengths for reinforcement shall follow the 2024 AASHTO LRFD Bridge Design Specifications (10th Edition). The Department has updated its set of simplified reinforcement bar lap charts in Appendix A3.4 herein based on these specifications for concrete compressive strengths from 3.5 ksi to 8.5 ksi, and for epoxy coated bars and unprotected bars. The simplified bar lap chars should be used whenever possible. AASHTO now uses a single unified equation for straight, hooked, and headed

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reinforcement bars based on the fourth-root dependency on concrete strength modified with λ factors to adjust the basic tension development length. In order to develop simplified charts, A_{tr}, the term accounting for the cross sectional area of transverse confinement reinforcement within the λ_{rc} modification factor, was conservatively assumed equal to zero. The Department also provides the following guidance for development and lap lengths:

- Straight Bars ($F_h = 0$): Superstructure base sheets have been updated to depict the standard lap lengths for the common parapet reinforcement (#4 bars) and bridge deck reinforcement (#5 bars assuming 1 inch of cover). For the Bar Splicer Assembly base sheet, BSD-1, designers shall determine the appropriate lap length for each specified bar size and location based on the new charts and insert them directly on the base sheet chart.
- Hooked Bars ($F_h = d_b R \vee f'_c$): When available geometry cannot accommodate a required full development length, the reinforcement may be detailed terminating in a standard hook. The resulting modified development length for deformed bars in tension, l_{dh}, is calculated with respect to a "critical section" as defined by Article 5.10.8.2.4a of the AASHTO LRFD Bridge Design Specifications. The hook radius R shall be taken from Appendix A3.3 herein.
- Headed Bars ($F_h = 0.5 A_{ht} \vee f'_c$): In lieu of obtaining company-specific dimensions for headed bars, the net area of the head that bears on concrete, A_{ht}, shall be a minimum of four times the bar area in normal weight concrete. The "critical section" shall be taken to the inside bearing face of the head. Refer to Article 5.10.8.2.7 of the AASHTO LRFD Bridge Design Specifications for additional limitations.

If it is necessary to provide varying length reinforcement bars in order to accommodate a flared condition on any part of a structure, do not detail the bars in a table of small increment changes in length; detail the bars in groups of the same length to accommodate the flare by variance of lap. All bars in the same group shall carry the same bar designation. This criterion is not to be construed as applicable to the ends of the deck slab of a skewed structure supported on steel stringers; in this case, the bars shall be cut in the field as described under Section 3.2.3 - Reinforcement (Treatment of Skewed Decks).

On stage construction projects for both superstructure and substructure elements, bar splicer assemblies shall be used to connect reinforcement bars which cross the stage construction line. Bar splicer assemblies are preferred over extending the reinforcement through the forms to make a lap splice because they provide ease of construction and a safer work environment. Quantities for bar splicers should only be shown in the Total Bill of Material and on the Bar Splice base sheet. Bar splicers should be called out using the following note:

"Bar Splicers (E) for #_ a_(E) bars."

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Bars shall be detailed to the closest inch of length and the total weight of reinforcement bars shown in the Bill of Material shall be to the nearest 10 lbs.

Complex bent bars, such as those in parapets, shall be detailed to provide the proper configuration, concrete cover, and development. Therefore, individual component lengths and the total length, are not required to be detailed to the nearest inch. The total bar length presented in the Bill of Material shall be rounded to the nearest inch.

Headed reinforcement shall be dimensioned to the outside of the head and the total length presented in the Bill of Material shall be rounded to the nearest inch. The additional weight of the head shall not be calculated for the total weight but shall be considered included in the cost of Reinforcement Bars, Epoxy Coated.

3.1.5 Reinforcement Designation

To provide uniformity on all bridge plans, the following reinforcement bar designations shall be used:

- a → Transverse Slab and Median Reinforcement
- b → Longitudinal Slab, Sidewalk and Median Reinforcement
- c → Sidewalk and Median Reinforcement (Transverse)
- d → Vertical Reinforcement in Parapet or Dowel bars at any location except Wall to Footing
- e → Longitudinal Reinforcement in Concrete Parapet
- g → Main Reinforcement - Concrete Girder
- h → Substructure Horizontal - Walls
- m → Horizontal Reinforcement - Diaphragm in Integral and Semi-Integral Abutments and P.P.C. I-Beam Structures
- n → Dowel - Wall to Footing
- p → Pile Caps and Pier Caps - Longitudinal
- s → Stirrup Bars
- t → Footing (Transverse)
- u → Ends of Pier Caps, Pile Caps and Pier Walls
- v → Vertical Bars (Substructure)
- w → Footing (Longitudinal)
- x → Cantilevered Deck Slab (Longitudinal)

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Typically, reinforcement bars are epoxy coated and suffixed with the designation "(E)". For example, the fourth bar in the e-series of longitudinal parapet reinforcement would have the designation e3(E). The note below (repeated from Section 3.1.3) shall also be included with the General Notes on the Contract plans. This note is not required on all sheets which have a Bill of Materials which include epoxy coated reinforcement bars.

Note: Reinforcement bars designated (E) shall be epoxy coated.

3.1.6 Total Bill of Material (General Plan & Elevation Sheet)

Regardless of the placement of a coded "Summary of Quantities" on any other sheet, there shall be a "Total Bill of Material" for bridge quantities on the "General Plan and Elevation" sheet if there is enough space or, if not, it shall be placed on the second sheet. This bill should not include code numbers, but it shall be broken down into Superstructure, Substructure and Total. It shall be carefully checked by the designer to reflect the individual quantity totals within the plans. The "Total Bill of Material" should not be broken down by stage. The "Total Bill of Material" for a dual structure should not be down for each structure.

3.1.7 Bill of Material (Individual Elements of Bridge)

There shall be separate Bills of Material on the appropriate sheets for the superstructure and individual elements of the substructure to assist bidding and construction. If the expansion piers under a structure are very similar in dimension and reinforcement, it is permissible to combine the quantities into one Bill of Material as long as it is clearly denoted as such.

The fixed pier(s) under a structure would normally differ from the expansion pier(s) in dimension and reinforcement. In this case, the fixed pier(s) shall be detailed on a separate sheet from the expansion pier(s). If the fixed piers under a structure are very similar in dimension and reinforcement, it is permissible to combine the quantities into one Bill of Material as noted above.

The same general criteria as that described for piers shall be applicable to presentation of "Bills of Material" for abutments.

Judgment shall be used in the presentation of all Bills of Material, keeping in mind that the bill is not prepared for the convenience of the designer, but rather for the use and convenience of those who are bidding and constructing the bridge.

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3.1.8 Basic Geometry & Footing Layout

The basic geometry for the location of the substructure units shall be clearly shown on the plans.

All portions of the structure shall utilize a common longitudinal reference line. When a structure is on a tangent (straight), this line may be designated as either the Centerline Survey, Centerline Roadway or Centerline North (South, East, West) Bound Lanes. When a structure is on a curve, the reference line preferably should be established and designated as either the "Tangent to Centerline Survey (Roadway, Lanes) at Sta____" or a "Parallel to Tangent to Centerline Survey at Sta. ____". When all or most of the stringers for a curved deck are to be continuously straight and parallel, the reference line selected should be parallel to these stringers.

Except for very simple geometry, such as a singly symmetric structure on a tangent, a footing layout should be shown on the plans. The layout may be in the form of a small diagram or occupy an entire sheet, depending on the complexity of the geometry.

Figures 3.1.8-1, 3.1.8-2 and 3.1.8-3 show typical examples of footing layouts.

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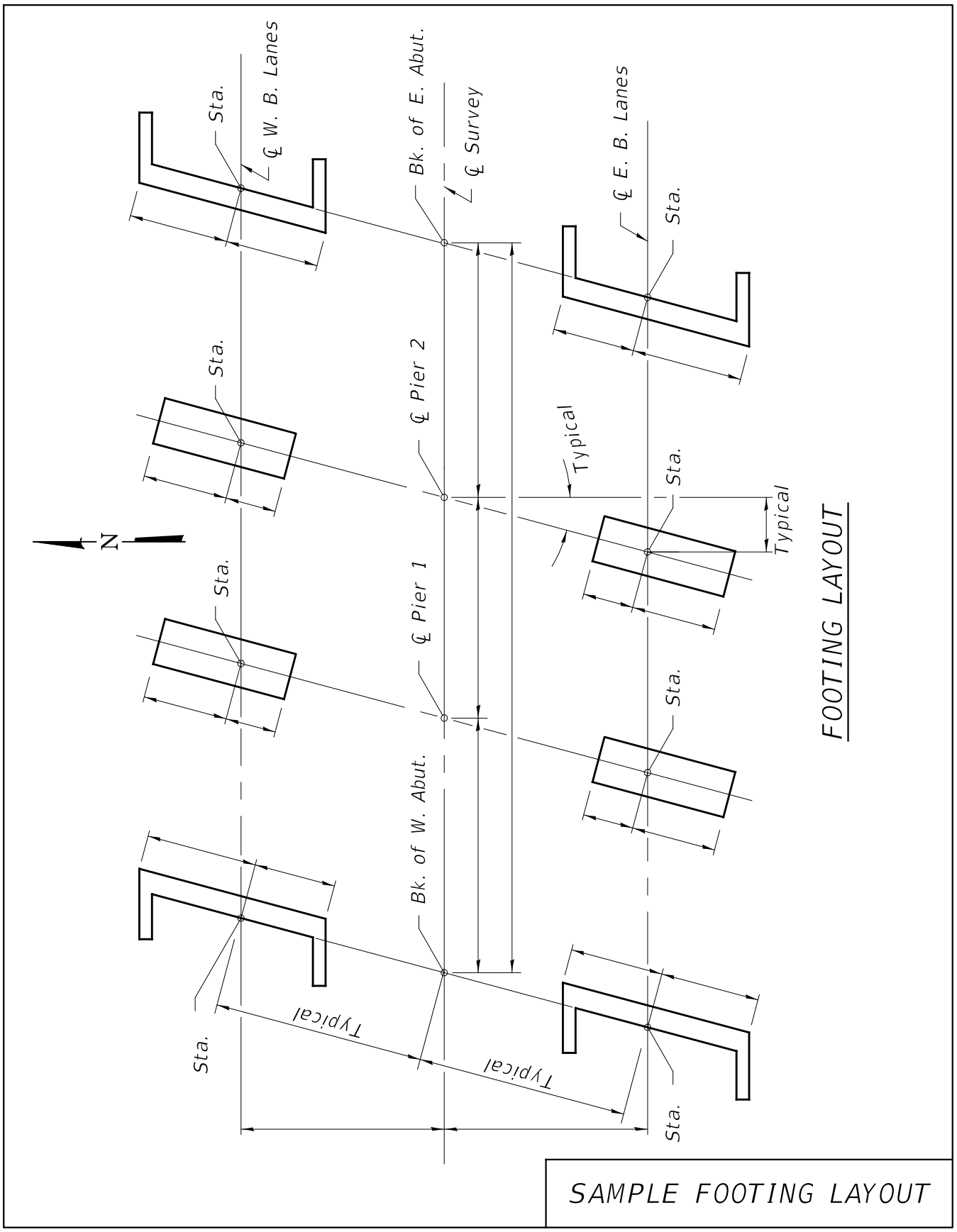


Figure 3.1.8-1

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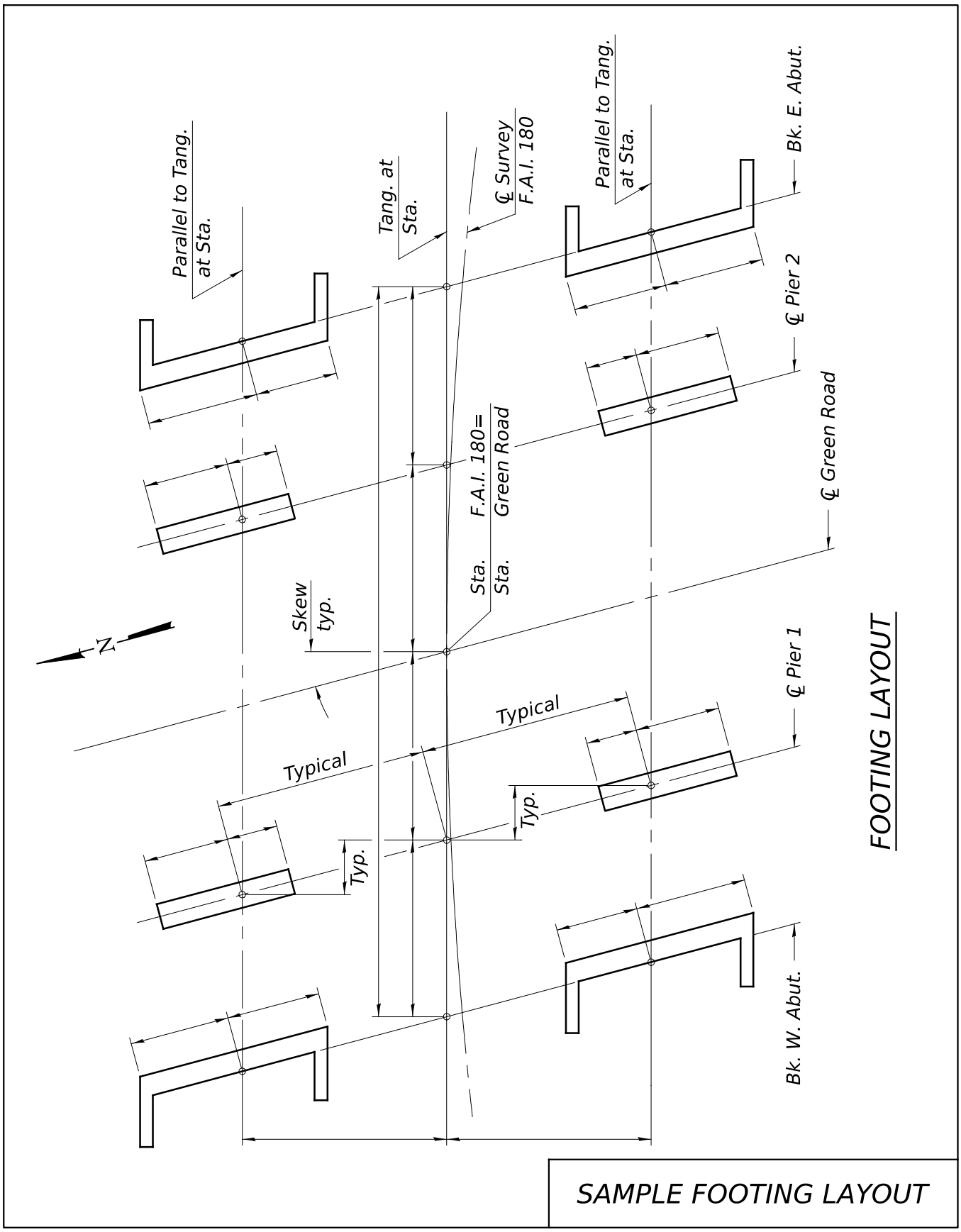


Figure 3.1.8-2

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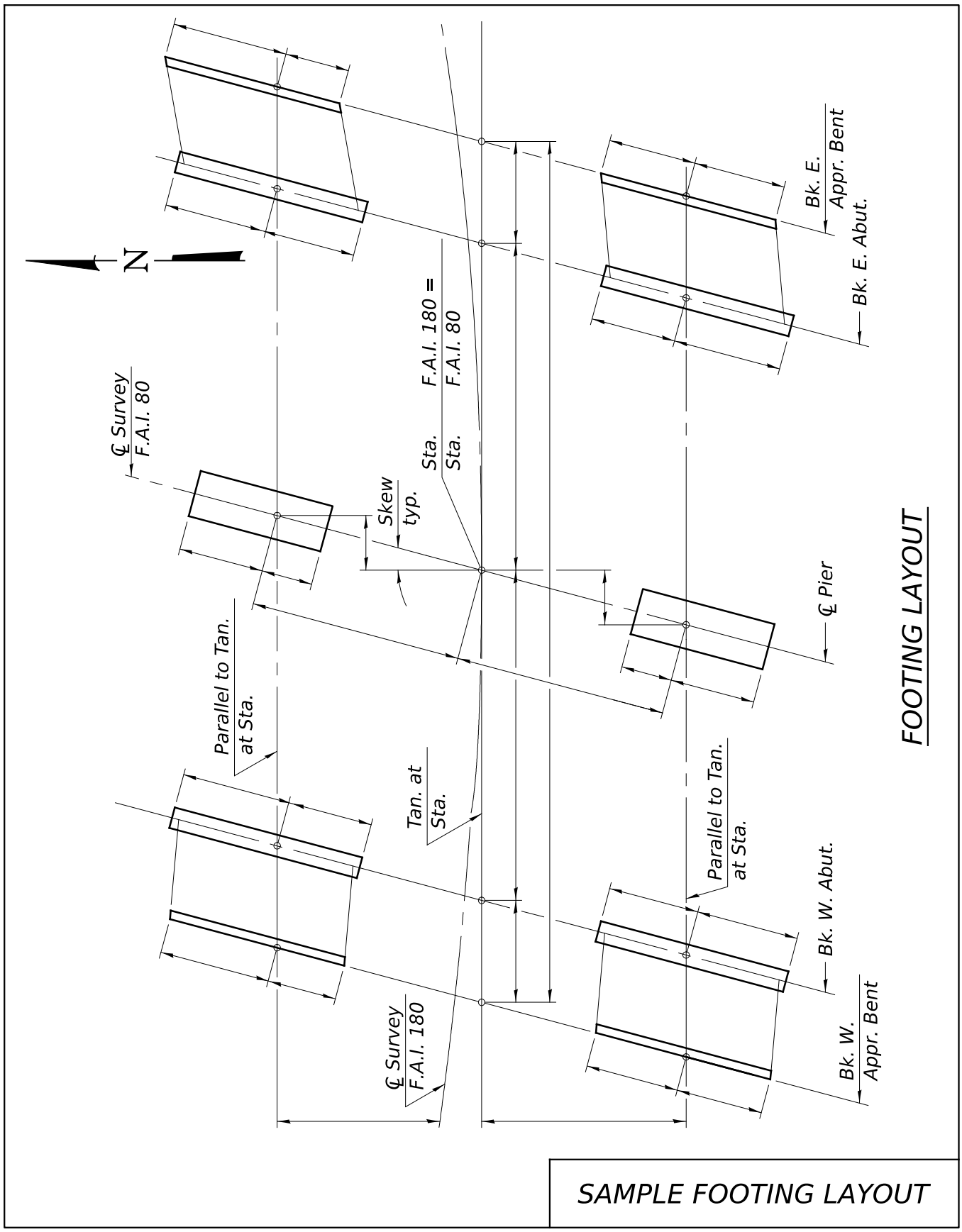


Figure 3.1.8-3

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3.1.9 Top of Slab Elevations

A table showing top of deck slab elevations along the centerline of web for each supporting stringer, each longitudinal bonded and/or stage construction joint, the profile grade, and the crown, if it is not on the profile grade, shall be included on the bridge plans for all structures with steel or prestressed concrete primary beams. This table is usually in the form of tabular computer output on individual plan sheets. Base sheets showing the general layout of top of slab elevation for both bridge decks and approach slabs can be found in the "Base Sheets-General" IDOT CADD Bridge Cell Library. Examples of layout of these elevations are shown in Appendix A3.

Elevations for slab bridges and all Bridge Approach Slabs shall be given along the profile grade, the stage construction joint, edge of roadway, all changes in cross slope, and at the face of parapet or curb line.

If a stringer lies below a curb, sidewalk, or median section, the elevations shall be given for a theoretical top of slab, i.e. the elevation of the top of slab considering that there is no curb, sidewalk or median.

The increments for elevations along each line shall be ten ft. with any odd increment at the end of a span not greater than fifteen ft. (≤ 15 ft.) and not less than five ft. (≥ 5 ft.). A new series of ten ft. increments shall be started at the beginning of each respective span along the structure. In all cases, the increments shall progress in the direction of the stationing on the bridge for the full length of the structure. Additional elevations are required at expansion joint lines for contracts that contain grinding and smoothness criteria.

The top of slab elevations at incremental points shall also be given with adjustments for dead load deflection of the bridge in an additional tabular column which are the finished elevations for construction of the deck slab. These elevations shall be keyed to a diagrammatic plan. Actual dead load deflection (weight of concrete deck and all superimposed dead loads except future wearing surface) diagrams shall be shown on this sheet indicating deflection ordinates at the quarter points and mid-point of all spans for all beams (exterior and interior). However, if the variance in deflection between beams is ⅛ in. or less, one dead load deflection diagram is adequate for all beams. Dead load deflection diagrams indicating deflection ordinates at the quarter points and mid-point of all spans shall be provided for all slab bridges, regardless of whether or not the top of slab elevations are required to be provided.

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3.1.10 Designation of Roadway Crown on Plans

The plans shall clearly show the total crown and how the crown was obtained. For example, if the total crown is 3 in., show this total as well as that it was obtained by 1.5% across a 12 ft. – 0 in. traffic lane and 2.0% across a 3 ft. – 0 in. shoulder.

3.1.11 Boring Logs and Subsurface Data Profile Plot

The boring (and rock core) locations shall be shown on the plan view of the "General Plan and Elevation" sheet and shall be keyed by number to the boring logs such as "Boring No. 1, Boring No. 2, etc." The boring logs, rock core logs and Shelby Tube test results (which are included in the SGR) shall be included in the Final plans. The bottom of footing elevations should, if possible, be indicated on the appropriate boring log and identified as "Bottom of Footing-Pier No. 1, etc." Ground water elevations shown on the boring logs should state "Elevation at time boring was taken." The number of logs per sheet and size of each log should be selected such that a minimum total number of plan sheets are utilized. All lettering and numbers shall not be less than $^{1}/_{10}$ in. in height on a full size plan sheet or $^{1}/_{20}$ in. on a quarter-size plan sheet. As an alternative, a subsurface data profile plot may be provided in place of boring logs and forms. A subsurface data profile plot is required to be provided in the SGR. It contains each soil boring, rock core and laboratory soils test plotted adjacent to each other in a continuous column (from ground surface to the bottom of boring or core) which is vertically to scale in elevation view. To maximize the number of borings per plan sheet, the borings should normally not be plotted to scale horizontally, but should follow the general sequence in station or offset along the longitudinal axis of the structure. When multiple plan sheets are required, the same vertical scale shall be used on each plan sheet.

3.1.12 Load Distribution, Moments, and Shears

3.1.12.1 Distribution of Loads to Beams and Girders

All straight structures with spans 240 feet in length or less with skews from 0 to 60 degrees shall be designed using a beam-slab (line girder) analysis as per Article 4.6.2.2 of the AASHTO LRFD Bridge Design Specifications, modified as stated below.

Curved structures with skewed supports of 10 degrees or less, satisfying the provisions of Article 4.6.1.2.4b of the AASHTO LRFD Bridge Design Specifications, shall be considered equivalently straight and shall be designed using a line girder analysis. Other curved structures, straight structures with skews

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exceeding 60 degrees, and/or structures with spans exceeding 240 feet require a more refined analysis, typically in the form of a grid analysis.

The desire of the Department is to have interior beams govern the design and rating of a bridge. The AASHTO LRFD Bridge Design Specifications provides the live load distribution equations for moment in interior beams in Table 4.6.2.2.2b-1 and the live load distribution equations for moment in exterior beams in Table 4.6.2.2.2d-1. The exterior equations for two or more design lanes loaded suggests that if de is limited to 2'-1", the exterior beam will have less influence than the interior beam. Therefore, to help ensure the exterior beam does not govern, IDOT bridges shall ideally be designed such that the exterior beam overhang is limited to the smaller of de plus the width of a continuous parapet or curb, or half the beam spacing. Structures satisfying these geometric requirements are considered "Typical" bridges and shall utilize the live load distribution equations for moment in interior beams. Figure 3.1.12.1-1 Illustrates the de and the exterior beam overhang for an Illinois parapet and railing. On structures with raised sidewalks the overhang shall be limited to half the beam spacing.

On rare occasions it may be necessary to have an overhang that exceeds half the beam spacing or a de that exceeds 2'-1". These cases are considered "Atypical" bridges and shall utilize the live load distribution equations for moment in exterior beams for two or more design lanes loaded and utilize the lever rule with a multiple presence factor equal to 1.0 for one design lane loaded. The governing live load distribution factor shall be used to design all beams. The interior and exterior beam at a given cross section should ideally be the same size. The effective flange width for the exterior beam shall be taken as no greater than the effective flange width for the interior beam for design calculations. The additional dead load from the wider exterior overhang shall be applied only to the exterior girder as additional DC1. The standard IDOT diaphragms and cross-frames are not considered sufficiently rigid to allow a typical beam slab cross section to rotate and deflect as a rigid cross section and therefore the pile analogy provisions discussed in Article 4.6.2.2.2d for exterior beams are not applicable.

For the strength, extreme event and service limit states, the live load distribution factor for the appropriate number of design lanes loaded should always be calculated. Where there will be only one lane during stage construction, the one design lane loaded distribution factor should also be checked. The maximum value shall govern.

For the fatigue limit state (including the fatigue calculations for stud shear connector design), the one design lane loaded distribution factor should be used for the final cross-section.

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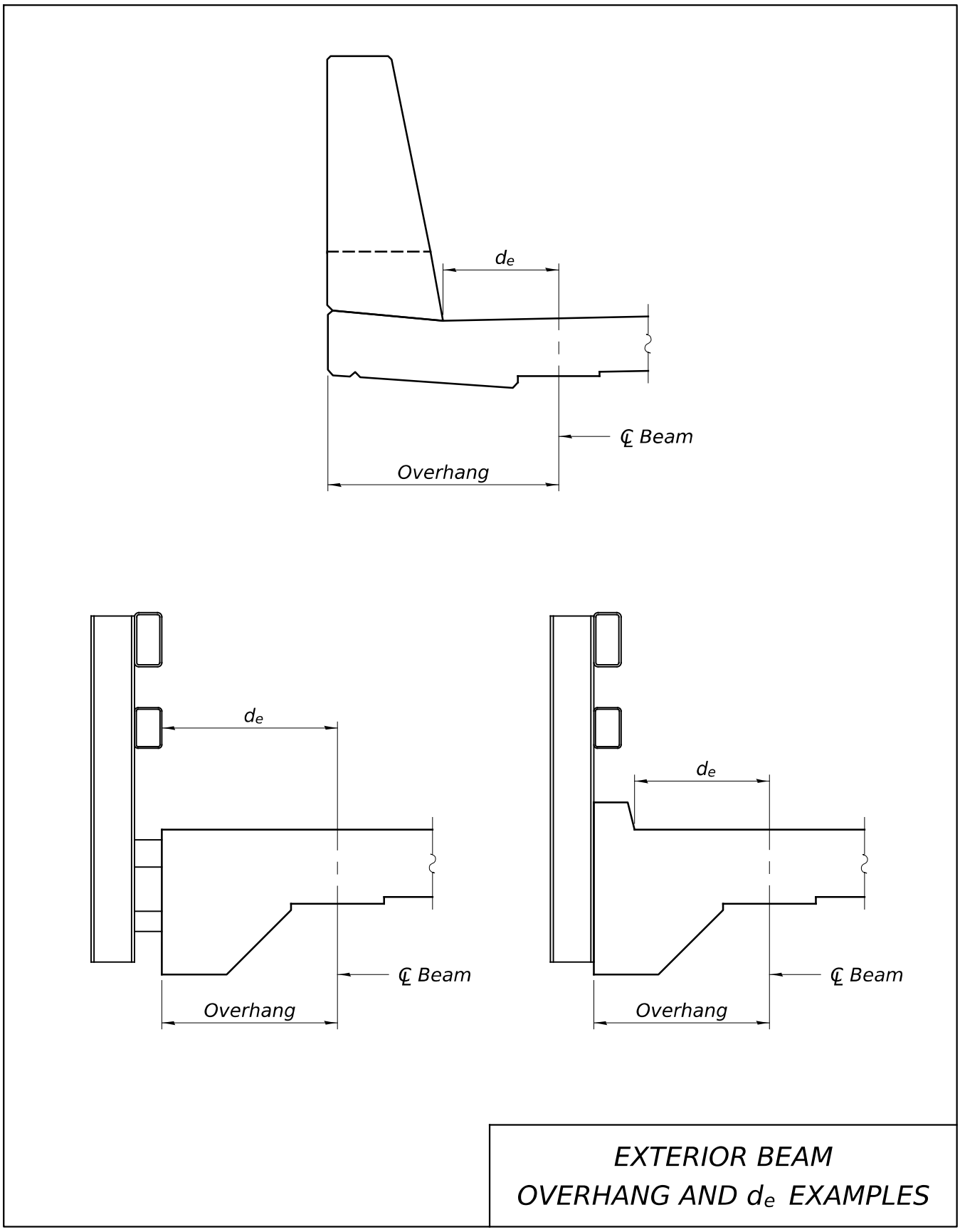


Figure 3.1.12.1-1

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The Bureau of Bridges & Structures has developed simplifications for live load distribution for moments, shears and reactions that may be used for typical IDOT steel bridges. Simplifications for PPC concrete beams should not be used and Table 4.6.2.2.2b-1 of the AASHTO LRFD Bridge Design Specifications shall be used. These simplifications may be used for both interior and exterior beams/girders in lieu of the equations in the AASHTO LRFD Bridge Design Specifications when the following criteria are met:

i.) The cross-section fits case a, e, or k in Table 4.6.2.2.1-1 of the AASHTO LRFD Bridge Design Specifications.
ii.) There are at least 5 beam/girder lines in the final cross-section.
iii.) The beams/girders are straight or considered equivalently straight as defined in Section 3.3.
iv.) The slab thickness is at least 7.5 in.
v.) The beam/girder spacing is between 3.5 ft and 12 ft.
vi.) The span length is between 20 ft and 240 ft.
vii.) The d_e dimension, measured from the centerline of the beam to the inside edge of the parapet, railing, or curb at the exterior beam/girder, is equal to or less than 2'-1".

For moments, the following simplified live load distribution equations may be used for steel structures meeting the criteria above. References are to the AASHTO LRFD Bridge Design Specifications:

$$g_m = 0.075 + C \times \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \quad \text{Multi - Lanes Loaded}$$

$$g_1 = 0.06 + C \times \left(\frac{S}{14.0}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \quad \text{Single - Lane Loaded}$$

Where:

C = 1.02 (Table 4.6.2.2.1-3)
S = Beam spacing in ft.
L = Span length in ft, with L as defined in Table C4.6.2.2.1-1

For fatigue evaluations the following distribution factor equation may be used per Article 3.6.1.1.2:

$$g_1(\text{fatigue}) = \frac{g_1}{m} \quad \text{Single - Lane Loaded}$$

Where:

g_1 = Single - Lane Loaded Distribution Factor
m = Multiple Presence Factor

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The moment reduction factors for highly skewed bridges in Table 4.6.2.2.2e-1 of the AASHTO LRFD Bridge Design Specifications shall not be applied.

For shears and reactions for structures that meet the criteria above, the live load distribution factors for shear from Table 4.6.2.2.3a-1 of the AASHTO LRFD Bridge Design Specifications for the multi-lane loaded and single-lane loaded cases should be used for all beams/girders. For structures that are skewed, these live load distribution factors shall be multiplied by the following simplified obtuse correction factor (OCF) (in lieu of the factors from Table 4.6.2.2.3c-1 of the AASHTO LRFD Bridge Design Specifications):

$$1.0 + 0.20 \tan\theta$$

Where:

θ = Skew angle

Typically, when determining dead load moments and shears using the AASHTO LRFD Bridge Design Specifications for a straight bridge, the deck dead load supported by interior beams shall be the portion center-to-center of the beam spans, and the dead load supported by the exterior beams shall be the portion which is comprised of half the center-to-center distance between beams plus the cantilevered overhanging portion of the deck. The curb section and rail above the mandatory horizontal construction joint, the median (if of the superimposed type), and any superimposed wearing surface (proposed or future) shall be distributed equally to all beams.

There are situations, however, where the simple distribution of superimposed dead loads described above may not be appropriate. These include, but are not limited to, bridges with very wide decks and/or open longitudinal joints, and curved girder bridges. When the equal distribution method appears inappropriate or is not allowed such as for curved girders, engineering judgment may be exercised to distribute superimposed dead load in a non-uniform fashion. For example, if the bridge is straight and has at least 9 beams, the dead load from one parapet can be distributed to the 3 exterior beams. Depending upon the size of a sidewalk or median, the distribution may logically extend over more beams.

The analysis used to determine the composite dead load moments, shears and reactions shall be based on the section modulus with the concrete transformed to steel using a modular ratio of n. The stresses in the section are based off of various section moduli depending upon application. See Design Guide 3.3.4 on the Bridges & Structures page for more information.

Exterior stringers shall typically be of the same section and capacity as the interior stringers even though the design analysis indicates that it could be less. If special cases arise where the design requirements

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of the exterior stringer are greater than the interior, modification of the fascia portion of the structure may be considered. Note that the criteria and simplifications described above for LRFD designs are meant to ensure that an interior beam normally governs the design.

When the total beam depth is less than 30 inches, General Note 13 shall be included in the plans in order to prevent excessive torsion on shallow beams.

3.1.12.2 Table of Moments and Shears – Typical Bridges

To provide the reviewing agencies with a basis for checking the design and to provide ready information for future record or analysis, all detailed bridge plans shall present a table of moments and shears. If possible, this table shall be shown on the structural framing sheet(s) of the plans. If the structure contains a pedestrian sidewalk and/or bicycle lane in the cross section, then the plans shall include a table showing the maximum allowable Service I live load plus impact deflection according to AASHTO LRFD Bridge Design Specifications 9.5.2 (Δ_{ALLOW}) and the calculated value (Δ_{LL+IM}). An example table with its definitions is shown below.

INTERIOR GIRDER DEFLECTION TABLE (NB)		
Δ_{ALLOW}	(in)	3.36
$\Delta_{℄ + IM}$	(in)	1.53

Δ_{ALLOW}: Maximum allowable Service I live load plus impact deflection according to AASHTO LRFD Bridge Design Specifications 9.5.2.
$\Delta_{℄ + IM}$: Calculated value Service I live load plus impact deflection.

Figures 3.1.12.2-1 3.1.12.2-2, and 3.1.12.2-3, present suggested layouts of LRFD tables of moments and shears for “Typical” straight steel beam superstructures and prestressed beam superstructures. Additional location columns shall be added as necessary for structures with more than two cross sections, unsymmetrical spans, and curved girders.

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Symmetrical 2 span

INTERIOR GIRDER MOMENT TABLE			
		0.4 Sp. 1 or 0.6 Sp. 2	*Pier*
I_s	*(in⁴)*		
I_c *(n)*	*(in⁴)*		
I_c *(3n)*	*(in⁴)*		
I_c *(cr)*	*(in⁴)*	———	
S_s	*(in³)*		
S_c *(n)*	*(in³)*		
S_c *(3n)*	*(in³)*		
S_c *(cr)*	*(in³)*	———	
S_x	*(in³)*		
DC1	*(k/')*		
M_{DC1}	*('k)*		
DC2	*(k/')*		
M_{DC2}	*('k)*		
DW	*(k/')*		
M_{DW}	*('k)*		
LLDF			
M_{L+IM}	*('k)*		
f_ℓ *(Strength I)*	*(ksi)*		
$M_u + \frac{1}{3} f_\ell S_x$	*('k)*		
$\phi_f M_n$	*('k)*		
f_s *DC1*	*(ksi)*		
f_s *DC2*	*(ksi)*		
f_s *DW*	*(ksi)*		
f_s *(L+IM)*	*(ksi)*		
f_ℓ *(Service II)*	*(ksi)*		
$f_s + f_\ell / 2$ *(Service II)*	*(ksi)*		
Service II Resistance	*(ksi)*		
$f_s + f_\ell / 3$ *(Strength I)*	*(ksi)*		
$\phi_f F_n$	*(ksi)*		
V_f	*(k)*		

Note:
Within any single vertical column, only a value for $\phi_f M_n$ or a value for $\phi_f F_n$ should be shown, and a dash should be shown in all other adjacent vertical columns (e.g., a value of both $\phi_f M_n$ and a value for $\phi_f F_n$ should not occur in the same vertical column).

LRFD DESIGN MOMENT TABLE FOR TYPICAL STEEL BRIDGES

Figure 3.1.12.2-1

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Definitions for Figure 3.1.12.2-1. All article references are to the AASHTO LRFD Bridge Design Specifications:

I_s, S_s	Non-composite moment of inertia and section modulus of the steel section used for computing f_s (Total-Strength I, and Service II) due to non-composite dead loads (in.^{4} and in.^{3}).
$I_c(n)$, $S_c(n)$	Composite moment of inertia and section modulus of the steel and deck based upon the modular ratio, "n", used for computing f_s (Total-Strength I, and Service II) in uncracked sections due to short-term composite live loads (in.^{4} and in.^{3}).
$I_c(3n)$, $S_c(3n)$	Composite moment of inertia and section modulus of the steel and deck based upon 3 times the modular ratio, "3n", used for computing f_s (Total-Strength I, and Service II) in uncracked sections, due to long-term composite (superimposed) dead loads (in.^{4} and in.^{3}).
$I_c(cr)$, $S_c(cr)$	Composite moment of inertia and section modulus of the steel and longitudinal deck reinforcement, used for computing f_s (Total-Strength I and Service II) in cracked sections, due to both short-term composite live loads and long-term composite (superimposed) dead loads (in.^{4} and in.^{3}).
S_x	Section modulus about the major axis of the section to the controlling flange, tension or compression, taken as yield moment with respect to the controlling flange over the yield strength of the controlling flange (in.^{3}). See Design Guide 3.3.4 on the primary Bridges & Structures Page.
DC1	Un-factored non-composite dead load (kips/ft.).
M_{DC1}	Un-factored moment due to non-composite dead load (kip-ft.).
DC2	Un-factored long-term composite (superimposed excluding future wearing surface) dead load (kips/ft.).
M_{DC2}	Un-factored moment due to long-term composite (superimposed excluding future wearing surface) dead load (kip-ft.).

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DW — Un-factored long-term composite (superimposed future wearing surface only) dead load (kips/ft.).

M_{DW} — Un-factored moment due to long-term composite (superimposed future wearing surface only) dead load (kip-ft.).

LLDF — Live Load Distribution Factor for moment and shear computed according to Article 4.6.2.2 and further IDOT provisions.

M_{LL+IM} — Un-factored live load moment plus dynamic load allowance (impact) (kip-ft.).

M_u — Strength I load combination of factored design moments (kip-ft.).

$1.25(M_{DC1} + M_{DC2}) + 1.5M_{DW} + 1.75M_{LL+IM}$

f_ℓ — Factored calculated flange lateral bending stress as calculated using Article 6.10.1.6 and as further simplified by IDOT provisions (ksi).

$\phi_f M_n$ — Factored nominal flexural resistance of the section determined as specified in Article 6.10.7.1 or A6 applicable (kip-ft.).

f_s DC1: — Un-factored stress at edge of flange for controlling steel flange due to vertical non-composite dead loads as calculated below (ksi):

M_{DC1} / S_s

f_s DC2: — Un-factored stress at edge of flange for controlling steel flange due to vertical composite dead loads as calculated below (ksi):

$M_{DC2} / S_c(3n)$ or $M_{DC2} / S_{c(cr)}$ as applicable

f_s DW: — Un-factored stress at edge of flange for controlling steel flange due to vertical composite future wearing surface loads as calculated below (ksi):

$M_{DW} / S_c(3n)$ or $M_{DW} / S_{c(cr)}$ as applicable

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f_s (LL+IM): Un-factored stress at edge of flange for controlling steel flange due to vertical composite live load plus impact loads as calculated below (ksi):

$M_{LL+IM} / S_c(n)$ or $M_{LL} / S_{c(cr)}$ as applicable

$f_s + f_\ell/2$ (Service II): Sum of stresses as computed below (ksi).

f_s DC1 + f_s DC2 + f_s DW + 1.3 f_s (LL+IM) + $f_\ell/2$

Service II Resistance: Composite ($0.95R_hF_{yf}$) or noncomposite ($0.80R_hF_{yf}$) stress capacity according to Article 6.10.4.2 (ksi).

$f_s + f_\ell/3$ (Strength I): Sum of stresses as computed below on non-compact section (ksi).

$1.25(f_s \text{ DC1} + f_s \text{ DC2}) + 1.5f_s \text{ DW} + 1.75f_s \text{ (LL+IM)} + f_\ell/3$

$\phi_f F_n$: Factored nominal flexural resistance of the section as specified in Article 6.10.7.2 or 6.10.8 as applicable (ksi).

V_f Maximum factored shear range in span computed according to Article 6.10.10.

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GIRDER REACTION TABLE			
		Abut.	Pier
LLDF			
OCF			
R_{DC1}	(k)		
R_{DC2}	(k)		
R_{DW}	(k)		
R_{LL}	(k)		
R_{Im}	(k)		
R_{Total} (Strength I)(Impact)	(k)		
R_{Total} (Strength I)(No Impact)	(k)		

LRFD DESIGN REACTION TABLE
FOR TYPICAL STEEL BRIDGES

Figure 3.1.12.2-2

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Definitions for Figure 3.1.12.2-2. All article references are to the AASHTO LRFD Bridge Design Specifications:

LLDF	Live Load Distribution Factor for moment and shear computed according to Article 4.6.2.2 and further IDOT provisions.
OCF	Obtuse Correction Factor according to Article 4.6.2.2.3c or as further simplified by IDOT provisions.
R_{DC1}	Un-factored reaction due to non-composite dead load (kip).
R_{DC2}	Un-factored reaction due to long-term composite (superimposed excluding future wearing surface) dead load (kip).
R_{DW}	Un-factored reaction due to long-term composite (superimposed future wearing surface only) dead load (kip).
R_{LL}	Un-factored live load reaction (kip).
R_{IM}	Un-factored dynamic load allowance (impact) (kip).
R_{Total} (Strength I): (Impact)	Strength I load combination of factored design reactions (kip). $1.25(R_{DC1} + R_{DC2}) + 1.5\ R_{DW} + 1.75(R_{LL}+R_{IM})$
R_{Total} (Strength I): (No Impact)	Strength I load combination of factored design reactions, not including (kip). dynamic load allowance (Impact) (kip). $1.25(R_{DC1} + R_{DC2}) + 1.5\ R_{DW} + 1.75\ R_{LL}$

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Symmetrical 3 span

INTERIOR BEAM MOMENT TABLE				
		0.4 Sp. 1 0.6 Sp. 3	Pier 1 or 2	0.5 Sp. 2
I	(in^4)			
I'	(in^4)			
S_b	(in^3)			
S_b'	(in^3)			
S_t	(in^3)			
S_t'	(in^3)			
DC1	(k/')			
M_{DC1}	('k)			
DC2	(k/')			
M_{DC2}	('k)			
DW	(k/')			
M_{Dw}	('k)			
LLDF	('k)			
$M_{LL + IM}$	('k)			

	INTERIOR BEAM REACTION TABLE			
		Abutments	Pier 1 Span 1 Pier 2 Span 3	Pier 1 Span 2 Pier 2 Span 2
	LLDF (k)			
	OCF (k)			
	R_{DC1} (k)			
*	R_{DC2} (k)			
*	R_{DW} (k)			
*	R_{LL} (k)			
*	R_{IM} (k)			
	R_{Total} (Strength I)(Impact) (k)			
	R_{Total} (Strength I)(No Impact) (k)			

* *At continuous piers, reactions from composite loads are assumed to be equally distributed to each bearing line.*

LRFD DESIGN DATA TABLES FOR TYPICAL PRESTRESSED CONCRETE BRIDGES

Figure 3.1.12.2-3

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Definitions for Figure 3.1.12.2-3:

I	Non-composite moment of inertia of beam section (in.^{4}).
I'	Composite moment of inertia of beam section (in.^{4}).
S_b	Non-composite section modulus for the bottom fiber of the prestressed beam (in.^{3}).
S_b'	Composite section modulus for the bottom fiber of the prestressed beam (in.^{3}).
S_t	Non-composite section modulus for the top fiber of the prestressed beam (in.^{3}).
S_t'	Composite section modulus for the top fiber of the prestressed beam (in.^{3}).
DC1	Un-factored non-composite dead load (kips/ft.).
M_{DC1}	Un-factored moment due to non-composite dead load (kip-ft.).
DC2	Un-factored long-term composite (superimposed excluding future wearing surface) dead load (kips/ft.).
M_{DC2}	Un-factored moment due to long-term composite (superimposed excluding future wearing surface) dead load (kip-ft.).
DW	Un-factored long-term composite (superimposed future wearing surface only) dead load (kips/ft.).
M_{DW}	Un-factored moment due to long-term composite (superimposed future wearing surface only) dead load (kip-ft.).
LLDF	Live Load Distribution Factor for moment and shear computed according to Article 4.6.2.2 and further IDOT provisions.
M_{LL+IM}	Un-factored live load moment plus dynamic load allowance (impact) (kip-ft.).
OCF	Obtuse Correction Factor according to Article 4.6.2.2.3c or as further simplified by IDOT provisions.

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R_{DC1}	Un-factored reaction due to non-composite dead load (kip).
R_{DC2}	Un-factored reaction due to long-term composite (superimposed excluding future wearing surface) dead load (kip).
R_{DW}	Un-factored reaction due to long-term composite (superimposed future wearing surface only) dead load (kip).
R_{LL}	Un-factored live load reaction (kip).
R_{IM}	Un-factored dynamic load allowance (impact) (kip)
R_{Total} (Strength I): (Impact)	Total factored reaction including dynamic load allowance (Impact) (kip).
R_{Total} (Strength I): (No Impact)	Total factored reaction not including dynamic load allowance (Impact) (kip).

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3.1.12.3 Table of Moments and Shears - Atypical Bridges

As further described in Section 3.1.12.2, to provide the reviewing agencies with a basis for checking the design and to provide ready information for future record or analysis, all detailed bridge plans shall present a table of moments and shears. If possible, this table shall be shown on the structural framing sheet(s) of the plans. Figures 3.1.12.3-1, 3.1.12.3-2, and 3.1.12.3-3, present suggested layouts of LRFD tables of moments and shears for “Atypical” straight steel beam superstructures and prestressed beam superstructures. The definitions for these figures are the same as the comparable figures in Section 3.1.12.2.

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Symmetrical 2 span

GIRDER MOMENT TABLE					
		0.4 Span 1 or 0.6 Span 2		Pier	
		Interior	Exterior	Interior	Exterior
I_s	(in^4)				
I_c (n)	(in^4)				
I_c (3n)	(in^4)				
I_c (cr)	(in^4)	———			
S_s	(in^3)				
S_c (n)	(in^3)				
S_c (3n)	(in^3)				
S_c (cr)	(in^3)	———			
S_x	(in^3)				
DC1	(k/')				
M_{DC1}	('k)				
DC2	(k/')				
M_{DC2}	('k)				
DW	(k/')				
M_{DW}	('k)				
LLDF					
$M_{Ł + IM}$	('k)				
$f_ł$ (Strength I)	(ksi)				
$M_u + \frac{1}{3} f_ł S_x$	('k)				
$\phi_f M_n$	('k)				
f_s DC1	(ksi)				
f_s DC2	(ksi)				
f_s DW	(ksi)				
f_s (Ł+IM)	(ksi)				
$f_ł$ (Service II)	(ksi)				
$f_s + f_ł / 2$ (Service II)	(ksi)				
Service II Resistance	(ksi)				
$f_s + f_ł / 3$ (Strength I)	(ksi)				
$\phi_f F_n$	(ksi)				
V_f	(k)				

Note:
Within any single vertical column, only a value for $\phi_f M_n$ or a value for $\phi_f F_n$ should be shown, and a dash should be shown in all other adjacent vertical columns (e.g., a value of both $\phi_f M_n$ and a value for $\phi_f F_n$ should not occur in the same vertical column).

LRFD DESIGN MOMENT TABLE FOR ATYPICAL STEEL BRIDGES

Figure 3.1.12.3-1

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GIRDER REACTION TABLE					
		Abut.		Pier	
		Interior	Exterior	Interior	Exterior
LLDF					
OCF					
R_{DC1}	(k)				
R_{DC2}	(k)				
R_{DW}	(k)				
R_{LL}	(k)				
R_{Im}	(k)				
R_{Total} (Strength I)(Impact)	(k)				
R_{Total} (Strength I)(No Impact)	(k)				

LRFD DESIGN REACTION TABLE
FOR ATYPICAL STEEL BRIDGES

Figure 3.1.12.3-2

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Symmetrical 3 span

INTERIOR BEAM MOMENT TABLE							
		0.4 Sp. 10.6 Sp. 3		Pier 1 or 2		0.5 Sp. 2	
		Interior	Exterior	Interior	Exterior	Interior	Exterior
I	(in^4)						
I'	(in^4)						
S_b	(in^3)						
S_b'	(in^3)						
S_t	(in^3)						
S_t'	(in^3)						
DC1	(k/')						
M_{DC1}	('k)						
DC2	(k/')						
M_{DC2}	('k)						
DW	(k/')						
M_{DW}	('k)						
LLDF	('k)						
M_{LL+IM}	('k)						

	INTERIOR BEAM REACTION TABLE							
			Abutments		Pier 1 Span 1 Pier 2 Span 3		Pier 1 Span 2 Pier 2 Span 2	
			Interior	Exterior	Interior	Exterior	Interior	Exterior
	LLDF	(k)						
	OCF	(k)						
	R_{DC1}	(k)						
*	R_{DC2}	(k)						
*	R_{DW}	(k)						
*	R_{LL}	(k)						
*	R_{IM}	(k)						
	R_{Total} (Strength I) (Impact)	(k)						
	R_{Total} (Strength I) (No Impact)	(k)						

* *At continuous piers, reactions from composite loads are assumed to be equally distributed to each bearing line.*

LRFD DESIGN DATA TABLES FOR ATYPICAL PRESTRESSED CONCRETE BRIDGES

Figure 3.1.12.3-3

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3.1.13 Plan Development Outline

In order to facilitate a more efficient and timely review and approval of Final plans, a "Plan Development Outline" (PDO) shall be prepared and submitted for each project involving a State structure where TSL and final plan submittals are required. PDO's are not required in the plan development for Local Agency structures. This outline shall be submitted directly to the Bureau of Bridges & Structures for review and approval after the TSL has been approved and early in the Final Design and Plan preparation phase. Figure 3.1.13-1 gives an example format for a "Plan Development Outline". The items listed in the figure are considered the minimum required by the Department.

The intent to classify members as System Redundant Members, Internally Redundant Members, or Nonredundant Steel Tension Members should be communicated as early as possible in the planning and design phase and shall be included in the Plan Development Outline (PDO). The PDO shall include identification of the members to be classified as SRMs, IRMs or NSTMs. See section 3.3.4.2 for additional information.

After reviewing the PDO, the Department will decide either (a) to have a meeting with the design consultant for an "Interim Plan-Review" or (b) to notify the consultant to proceed with the finalization of the final plans, subject to review comments, without a meeting. The comments will include the name and phone number of a contact person within the Bureau of Bridges & Structures. The PDO does not relieve the consultant of their responsibility to submit Final plans which are 100% complete, devoid of errors, and sealed by an Illinois Licensed Structural Engineer. Errors not discussed or commented on by the Department shall be the sole responsibility of the consultant.

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1. **Project Data:** [Provide cover sheet with easily identifiable project data.]

Plan Development Outline
for
[Route]
[Feature Carried and Crossed]
[Section #]
[County]
[Structure #]
[Contract #]
[IDOT Job D#]
[Scheduled Letting Date]

Prepared by: [Consultant Co. Name]
[PTB # / Item #]
[Current Date]

2. **Procedures for Quality Control and Quality Assurance:** [Brief description of the consultant's procedures for quality control and assurance including the names of the project coordinator, QC/QA reviewer, and structural engineer sealing the Final Structure Plans.]
3. **Project Description:** [Brief description of the structure project including the structure type, location, staging, and major items of the project scope-of-work.]
4. **Scope of Services:** [Brief description of the scope of services to be provided by the consultant including BCR and TSL preparation, structural design and Final plan preparation, services as a sub-consultant to (prime consultant), and Fabrication Shop Plan review.]
5. **Schedule:** [Brief outline of proposed schedule for the submittal of Final Plans for review and PS&E submittal.]
6. **Analysis and Design Procedures:** [Brief outline of proposed methods of analysis. Also list the dated version of the AASHTO Specifications, the Standard Specification, and the Supplemental Specifications and Recurring Special Provisions used.]
7. **Special Checks:** [Brief description of special checks that will be needed such as: fatigue analysis, seismic, ice loads, curved girder analysis, etc.]
8. **Constructability:** [Discussion of any issues regarding constructability including: erection, deck pour sequence, staging, slip-forming parapets, etc.]
9. **Non-Conventional Details:** [Note any non-conventional details or concerns that need to be addressed.]
10. **Foundations:** [Brief Summary of foundation treatment based on the Structure Geotechnical Report.]
11. **Preliminary List of Pay Items:** [List all anticipated structure pay items with units.]
12. **Preliminary List of Plan Sheets:** [List all anticipated Structure plan sheets with brief description of contents and Base Sheet designations.]
13. **Preliminary List of General Notes:** [Write out all anticipated Structure plan general notes. See Section 2.3.6.1 for paint system and District for Munsell color.]
14. **Special Provisions:** [List all anticipated structure Special Provisions and applicable Guide Bridge Special Provisions (GBSP's).]
15. **Additional Comments or Coordination Items:** [If needed]
16. **TSL:** [Include an 8 ½ in. by 11 in. copy of the approved TSL.]

Figure 3.1.13-1

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3.1.13.1 "Interim Plan-Review" Meeting Requirements

The consultant engineer responsible for sealing the plans may be required to attend a meeting in the Central Office with personnel from the Bridge Design Section of the Bureau of Bridges & Structures. After receiving comments on the “Plan Development Outline” document, the consultant shall schedule a mutually acceptable date for this meeting as required. The consultant shall respond within 10 calendar days of the first notification date to set the "Interim Plan-Review" meeting date. The list of any specific problems that the consultant is facing or anticipates as well as all pertinent special provisions should be made available to facilitate discussions.

3.1.14 Checklist for Final Plan Preparation

The checklist below is provided as an aid to the designer when completing a set of Contract plans. The checklist of items below is provided for guidance and is not all inclusive. The designer and checker shall supplement this checklist with additional material appropriate for the specific project in order to ensure quality plans and contract documents. Referenced figures and sections are included in the Bridge Manual unless otherwise indicated.

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3.1.14.1 General

1. The Plans, Standard Specifications, Supplemental Specifications, Recurring Special Provisions and contract-specific Special Provisions form the contract documents. It is the responsibility of the designer to produce structure plans and contract documents which are free from errors, omissions, and ambiguities which may result in misinterpretation.
2. Plans depict the scope of work graphically. Generally, they define overall extent, locations, dimensions, materials, methods and quantity considerations. Specifications describe the quality and capacity of material and equipment, the installation methods and techniques and the results to be achieved. Requirements should be defined in only one place. If special emphasis is necessary, item requirements may be defined by Special Provisions and shown on the plans.
3. To become familiar with the project, the designer should review the previous pertinent correspondence, Structure Report, Hydraulic Report, and Structure Geotechnical Report (SGR). When portions of an existing structure will be incorporated and/or construction will be staged, the Bridge Condition Report (BCR), existing Shop Drawings, if any, and the existing Structure plans should also be reviewed.
4. For each project, the consultant's performance will be evaluated for timeliness in meeting Departmental schedules, cooperation, coordination, quality and adequacy of structural design; quality, clarity and accuracy of Structure plans and Special Provisions; extent of corrections noted by Department reviewers or in construction; and initiative in identifying and addressing special and/or key design issues.
5. For Structure plans prepared by consultants, the design consultant firm name should be included on all sheets.
6. The contract number should be included in the lower right corner of all plan sheets in the contract. The structure number should be included in the title block of all Structure plan sheets.

3.1.14.2 General Plan and Elevation (GP&E)

1. Check conformance with the approved TSL plan and verify that all pertinent information on the TSL is included on the GP&E sheet except the following shall be excluded:
    a. Cross section, pier sketch, etc.
    b. Roadway data (highway classification).
2. Confirm indication of the Bridge Approach Slab and label the type.
3. Show name plate lettering and location of name plate on structure (two if dual structures). If it is a rehabilitation project where a portion of the existing structure is being reused, add General Note 45. For bridge widening or partial bridge replacement project requiring new name plates, the name plate shall read “RE-BUILT.” See Appendix A2.

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4. Include all applicable general notes as found in Section 3.1.3, and as required for the specific project.
    a. Coordinate with District and specify appropriate type of paint system and Munsell number.
    b. Specify method of cleaning existing structural steel and applicable cautions for associated hazards.
    c. Specify methods and/or material for concrete repair (cast-in-place or prestressed) and address quantity and/or location variations. For further guidance, refer to the Repairs section of the IDOT Structural Services Manual on the Bridges & Structures page.
5. Total Bill of Material:
    a. Coded pay items shall be used wherever possible.
    b. Verify that non-coded pay items are properly covered by Special Provisions.
    c. Verify that all needed pay items are in the Bill of Materials and add any which are missing.
    d. Any contract-required services or items not included in a pay item description shall be indicated as included in another specific pay item.
    e. Do not make an item “incidental to contract”.
    f. Roadway items not detailed in Structure plans should be listed in Roadway Bill of Materials and not the Structure Bill of Materials.
6. Show slope wall details. See Section 3.14.
7. Show riprap placement details.
8. Location of temporary sheet piling or soil retention system shall be shown (details should be on stage construction detail sheet).
9. Show Stage Construction Line, if applicable. Note which is Stage I Construction and which is Stage II Construction.
10. Show foundation type and required elevations.
11. Do not show structure excavation.
12. Index of sheets.
13. Affix structural seal with signature and expiration date on GP&E sheet of all structures and leave room for the State Bridge Engineer’s approval stamp and signature in lower portion of sheet.
14. Include design stresses. For steel structures give F_y of steel grade(s) specified and indicate the grade(s) used for the primary structural members.
15. Show location, type and owner of utilities and drainage structures (in-ground, on existing structure or overhead) that directly affect construction. Include existing sewers, culverts, etc.
16. Show waterway information table, if applicable.
17. Show design scour elevation table, if applicable.
18. If the GP&E sheet becomes too crowded, provide a “General Data” sheet for General Notes, Total Bill of Materials, and other miscellaneous details as needed. See Section 3.1.6 for guidance.

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3.1.14.3 Footing Layout, Stage Construction Details, Etc.

1. For dual structures, structures crossing navigable waterways, structures on curved alignments, or other unusual situations, show footing layout. See Figures 3.1.8-1 through 3.1.8-3.
2. Include sketches showing the stage removal and stage construction of the structure, and limits of removal of substructure elements not reconstructed. Generally, show elevation views for each removal and construction stage. For example: Stage I Removal; Stage I Construction; Stage II Removal; Stage II Construction. If the stage removal and stage construction lines are different for the superstructure and substructure, add a note on this sheet to alert the contractor.
3. Show location of temporary concrete barrier and temporary bridge rail. Use of temporary bridge rail should be limited to existing portion of the deck. Utilize the standard base sheets.
4. Indicate how the cost of non-standard items (e.g. removal of existing bridge rail, temporary support of existing utilities, or removal and delivery of items salvaged for the Department) is accounted for by notes or Special Provisions.
5. Show detail of partial and full depth deck patching where applicable. See also IDOT document Bridge Condition Report Procedures and Practices at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning.
6. Show a suggested sequence of construction when necessary.
7. Show the limits of soil retention system or temporary sheet piling. See Section 3.13.

3.1.14.4 Deck Elevations

1. Include fillet (haunch) details and associated notes. See Section 3.2.
2. Dead load deflection diagram includes deflections due to dead load of the concrete deck and all initial superimposed dead loads, i.e. parapet, sidewalk, median. It does not include girders, structural steel bracing, temporary forms, or future wearing surface. Show calculated deflections to the nearest ⅛ in. (0.01 ft.). See Section 3.1.9.
3. Layout of elevation lines and substructure lines as shown in tables for bridges with steel or PPC I-beams. See Section 3.1.9.
4. Layout of elevation lines for slab bridges according to Section 3.1.9, if required.
5. Layout for bridge approach slabs. See Section 3.1.9.
6. Show stationing, top of deck elevations and offsets to the nearest 0.01 ft.
7. Use similar lettering size (match the smallest as a minimum) for numbers in elevation tables as is used for lettering on elevation sheets.

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3.1.14.5 Deck Details

1. Cross Section.
    a. Show the cross sectional dimensions and bar locations as in the Standard Base Sheets.
    b. Show location of longitudinal construction joints, if applicable.
    c. Show crown, total drop, bar clearance, slab thickness, location of profile grade line, and stage construction line/joint, if applicable.
2. Plan.
    a. Design the area of the main reinforcement and distribution of steel in accordance with Section 3.2.
    b. Top and bottom mat reinforcement shall be lapped at different locations. See Sections 3.1.4 and 3.2.
    c. Specify minimum lap lengths for specific bars or by bar size, as applicable.
    d. Show top transverse bars in the slab under the curb or parapet lapped with transverse bars at the top of the slab. See Section 3.2.
    e. The section location(s) for expansion joint details at the end(s) of the slab shall be shown.
    f. Show North arrow.
    g. Locate parapet joints.
    h. Locate drains and scuppers and additional reinforcement.
    i. Locate light poles on structure, if applicable.
3. See Section 3.2 for deck pouring sequence. If required, the pouring sequence shall be shown on the plans.
4. Show expansion joint opening requirements. See Section 3.6. Expansion joint details are shown on separate sheets.
5. Show floor drains and/or drainage scupper details. See Section 3.2, and corresponding Base Sheets.
6. On stage construction projects, check dimensions, stage designations, bar call-outs, bar splicers, etc. See Base Sheet BSD-1 in the “Base Sheets-General” IDOT CADD Bridge Cell Library.

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3.1.14.6 Approach Slab Details

1. Plan.
    a. Show North arrow
    b. Show centerline of roadway and local tangent.
    c. Show stations and offsets at beginning and end of approach slab.
    d. Show transverse bar splicers for stage construction in both the slab and footing.
2. Show correct details for flexible vs. rigid pavement connector.
3. Cross Section.
    a. Show the cross sectional dimensions and bar locations as in the Standard Base Sheets.
    b. Show location of longitudinal construction joints, if applicable.
    c. Show crown, total drop, bar clearance, slab thickness, location of profile grade line, and stage construction line/joint, if applicable.
4. Show parapet joint details, if applicable.
5. Show end of parapet treatment, if applicable.
6. Show top and bottom elevations of approach slab footing.

3.1.14.7 Bridge Railing Details

1. Show parapet details. See Section 3.2.
    a. Show Mandatory Construction Joint at top of slab.
    b. Show aluminum plate and cork joint filler parapet joint details and locations.
    c. Show end of parapet details for expansion joint treatment or Traffic Barrier Terminal attachment.

3.1.14.8 Framing Plan and Beam/Girder Details

1. A framing plan for steel layout shall include:
    a. Beam/girder numbering, spacing, and lengths.
    b. Diaphragm/cross-frame type and locations. See Section 3.3.
    c. If curved, a local tangent, table of layout dimensions and offset diagram explaining offsets. See Section 3.3.
    d. Location of field splice(s).
    e. North arrow.

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2. The steel beams/girders shall be designed per the requirements of the AASHTO LRFD Bridge Design Specifications and Section 3.3 of the Bridge Manual.
    a. Beam or girder design shall be based on strength, economy, durability, maintainability and constructability.
    b. Show Moment and Reaction Tables and Definitions. See Section 3.1.12.
    c. Show weld sizes, flange transition locations*, shear stud layout and details, impact energy or fracture critical notations, diaphragm/cross-frame details, bolted field splice design and details. (*welded flange transitions are detailed on the base sheets and are additionally covered by the AASHTO/AWS D1.5 Bridge Welding Code.)
    d. Steel other than Grade 36 shall be identified by label or note on each steel details sheet. The material normally selected for primary steel members is Grade 50. For diaphragms, cross-frames, and connecting plates or angles on straight structures, the default is Grade 36 when non-weathering Grade 50 steel is used for primary members. When the material selected for primary steel members is Grade 50 weathering steel, diaphragms, cross-frames, and connecting plates or angles shall also be Grade 50 weathering steel. See Sections 2.3.6.1.1 and 3.3.4.
    e. Provide a table of Top of Web (or Top of Beam for WF) elevations. Add note under table: “For fabrication only”. These are used for shop-drilling splices with the steel supported, so they include no dead load deflection.
    f. When a girder is cambered, a diagram is required. See Section 3.3.12. A girder segment (between field splices or between a splice and a free end) with a maximum potential calculated camber less than ¾ in. should be detailed straight, and associated substructure seat elevations shall be calculated based on no camber. Rolled beams are not cambered except for unusual profile grade requirements and with the prior consent of the Bureau of Bridges & Structures.
    g. Show bearing stiffener details. Stiffeners acting as cross-frame or diaphragm connection plates shall be welded to both flanges, with minimum fillet weld size based on the thicker plate joined. A “finish to bear” (includes “mill to bear” and “grind to bear”) fit is required at the bottom flange. Do not specify complete joint penetration welds of stiffeners to flanges. See Section 3.3.
    h. If intermediate or longitudinal stiffeners are shown and not otherwise needed as bracing connection plates, investigate the feasibility of an incremental increase in web thickness to eliminate most or all of them.
    i. Show bearing details. See Section 3.7.
    j. Show designation of CVN for applicable rolled shapes, plates and other elements which carry calculated tensile stresses. (See Section 3.1.3)

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k. Designers shall submit a signed and sealed stand-alone written summary of the redundancy analyses along with the Final Structure Plans for acceptance and records. The summary shall include:

i. Identification of the members subjected to a redundancy analysis.

ii. Member designations (i.e., SRM, IRM, NSTM) based on redundancy analysis results.

iii. Nationally recognized methods used to evaluate redundancy. Any modifications to the nationally recognized methods shall be described.

iv. In-service inspection requirements for those members designated as IRMs or NSTMs, as applicable. Inspection requirements for load path redundant members and system redundant members are only subject to routine inspection requirements. This does not need to be added to the submittal.

l. **Plan Details for SRMs:**

Members in new designs that have demonstrated system redundancy through analysis in accordance with the *SRM Guide Specifications* shall be labeled as SRM in applicable design and in-service inspection documents. SRMs are required to be fabricated in accordance with NSTM fabrication requirements. The following notes shall be included in the design plans for newly designed yet to be constructed bridges containing SRMs:

Note	**Application**
Members designated as “SRM” are System Redundant Members	All SRMs
Members designated as “SRM” shall be fabricated to satisfy the provisions of Clause 12 of the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.	All SRMs with welds.
Fabricated plate components of members noted as “SRM” shall satisfy the Fracture-Critical Tension Component Impact Test Requirements, zone 2 specified in AASHTO M 270M/M270 (ASTM A709/A709M).	All load-carrying fabricated plate components of SRMs.
Rolled section components of members noted as “SRM” shall satisfy the Non-Fracture-Critical Tension Component Impact Test Requirements, zone 2 specified in AASHTO M 270M/M270 (ASTM A709/A709M).	All load-carrying rolled section components of SRMs.

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IRMs: Members in new designs that have demonstrated internal redundancy through analysis in accordance with the *IRM Guide Specifications* shall be labeled as IRMs in applicable design and in-service inspection documents. IRMs are required to be fabricated in accordance with NSTM fabrication requirements. The following notes shall be included in the design plans for newly designed yet to be constructed bridges containing IRMs:

Note	Application
Members designated as "IRM" are Internally Redundant Members	All IRMs
Fabricated plate components of members noted as "IRM" shall satisfy the Fracture-Critical Tension Component Impact Test Requirements, zone 2 specified in AASHTO M 270M/M270 (ASTM A709/A709M).	All load-carrying fabricated plate components of IRMs.
Rolled section components of members noted as "IRM" shall satisfy the Non-Fracture-Critical Tension Component Impact Test Requirements, zone 2 specified in AASHTO M 270M/M270 (ASTM A709/A709M).	All load-carrying rolled section components of IRMs.

NSTMs: Members in new designs that have not been established as system or internally redundant through analysis shall be labeled as NTSMs in applicable design and in-service inspection documents. The following notes shall be included in the design plans for newly designed yet to be constructed bridges containing NTSMs:

Note	Application
Members designated as "NSTM" are Nonredundant Steel Tension Members.	All NSTMs
Members noted as "NSTM" shall be fabricated to satisfy the provisions of Clause 12 of the AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.	All NSTMs with welds.
Fabricated plate and rolled section components of members noted as "NSTM" shall satisfy the Fracture-Critical Tension Component Impact Test Requirements, zone 2 specified in AASHTO M 270M/M270 (ASTM A709/A709M).	All NSTMs.

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3. A framing plan for Precast Prestressed Concrete I-beams, Bulb T-beams and IL beams shall be shown.
    a. Show beam spacing and lengths.
    b. North arrow is required.
    c. If non-parallel substructure units, variable spacing or other factors require different beam lengths and/or strand patterns, beams shall be numbered and individually detailed elsewhere in the plans.
4. PPC I-beams, Bulb-T beams and IL beams shall be designed for strength and serviceability in accordance with Section 3.4, applicable ABD Memos (available in the "Bridge Manual" section on the primary Bridges & Structures Page), and the AASHTO LRFD Bridge Design Specifications.
    a. Beam size and strand pattern shall be evaluated for economical design. Minor deviations in the number of strands or strand patterns shall be avoided in similar beams to facilitate fabrication.
    b. Show strand layout, draping details, lifting loop details, bursting steel details, and drain insert details. See tables, charts, details and Base Sheets in Section 3.4, online, and ABD Memos as applicable.
    c. Show bar list, bar details, notes and Bill of Material.
    d. Show required f'_{ci} and other pertinent design stresses, etc. on each beam sheet.
    e. Show bearing details. See Section 3.7.
    f. Show concrete diaphragm details. See Section 3.4.
    g. Show Moment and Reaction Tables and Definitions. See Section 3.1.12.
    h. Show permanent bracing steel diaphragm locations.
5. A framing plan for PPC deck beams is not required.
6. PPC deck beams shall be designed for strength and serviceability in accordance with Section 3.5, applicable ABD Memos, and the AASHTO LRFD Bridge Design Specifications .
    a. Beam size and strand pattern shall be evaluated for economical design. Minor deviations in the number of strands or strand patterns shall be avoided in similar beams to facilitate fabrication.
    b. Show strand layout, transverse tie layout, end block geometry, void tube layout, lifting loop details, and drain insert details. See tables, charts, details and Base Sheets in Section 3.5, and ABD Memos as applicable.
    c. Show bar list, bar details, notes and Bill of Material.
    d. Show required f'_{ci} and other pertinent design stresses, etc. on each beam sheet.

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3.1.14.9 Abutment Details

1. All abutments should, in general, follow layout as shown in Bridge Manual standards and Base Sheets. Show and detail stage construction, if applicable.
2. Integral and Semi-Integral Abutments.
    a. Design of piles and shafts shall include impact.
    b. Step heights shall be greater than or equal to ¾ in. Otherwise, shim plates shall be specified for steel members. Otherwise, for PPC beams, set PPC I-beams, IL beams, or Bulb T-beams on common seats and vary concrete haunch or fillet. See Section 3.8.
    c. Steps 4 in. or larger shall be reinforced. See Section 3.8.
    d. Show dimensions and details of cap. Min. main reinforcement is #7 bars. Check min. shear reinforcement requirements in the cap.
    e. Show wingwall details.
    f. Show step and bottom of cap elevations.
    g. Show step height dimensions.
    h. All elevations shall be shown to nearest 0.01 ft., and all dimensions to the nearest ⅛ in.
    i. Check to see that the proper pile corrosion protection is provided if piles are used. See Section 3.10.
    j. Show anchor bolt locations orthogonal to centerline of bearings.
    k. Show typical section thru abutment.
    l. For semi-integral abutments, add fabric reinforced elastomeric mat details. See Section 3.8.
3. Stub Abutments.
    a. Step heights shall be greater than or equal to ¾ in. Otherwise, shim plates shall be specified for steel members. Otherwise, for PPC beams, set PPC I-beams, IL-beams, or Bulb T-beams on common seats and vary concrete haunch or fillet. See Section 3.8.
    b. Steps 4 in. or taller shall be reinforced. See Section 3.8.
    c. Min. main reinforcement is #7 bars as shown in the Base Sheets. Check min. shear reinforcement requirements also.
    d. Show wingwall details.
    e. Show step and bottom of cap elevations.
    f. Show step height dimensions.
    g. All elevations shall be shown to nearest 0.01 ft., and all dimensions to the nearest ⅛ in.
    h. Show anchor bolt locations orthogonal to centerline of bearings.
    i. Show typical section thru abutment.

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4. Vaulted Abutments.
    a. Show footing dimensions and reinforcement.
    b. Detail "n" bar to provide full development lengths.
    c. Step heights shall be greater than or equal to ¾ in. Otherwise, shim plates shall be specified for steel members. Otherwise, for PPC beams, set PPC I-beams, IL-beams, or Bulb T-beams on common seats and vary concrete haunch or fillet. See Section 3.8.
    d. Steps 4 in. or taller shall be reinforced. See Section 3.8.
    e. Show wingwall details
    f. Show step and bottom of cap elevations.
    g. Show step height dimensions.
    h. All elevations shall be shown to nearest 0.01 ft., and all dimensions to the nearest ⅛ in.
    i. Design and detail vault slab.
    j. Show anchor bolt locations orthogonal to centerline of bearings.
    k. Show typical section thru abutment including drainage details.
    l. Show walls designed as beam lines for vault.
5. Details of foundation type (pile, drilled shaft or spread footing) shall be checked against geotechnical recommendations on file (SGR). Show all foundation design data required per Bridge Manual Section 3.10.
6. Quantity for structure excavation shall be shown for each abutment.
7. Specify Concrete Sealer where applicable.
8. Verify seismic design requirements are met. See Sections 3.7 and 3.15.

3.1.14.10 Pier Details

1. All piers should, in general, follow layout as shown in Bridge Manual standards and Base Sheets. Show and detail stage construction, if applicable.
2. Show anchor bolt locations orthogonal to centerline of bearings.
3. Column Piers/Frame Piers/Drilled Shaft Piers/Pile Bent Piers/Wall and Hammerhead Piers.
    a. Provide reinforcement details in columns, walls, hammerheads, shafts and cap, as required by design.
    b. Check step requirements. See Stub Abutment step requirements noted above as applicable to piers.
    c. Show step and bottom of cap elevations.
    d. Show step height dimensions.
    e. Elevations for all steps and footings shall be shown to the nearest 0.01 ft. and all dimensions to the nearest ⅛ in.

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f. Check that the need for cofferdams in stream crossing or high water table situations has been investigated properly. Check if current criteria and details are being utilized when there is need for cofferdams. Check the SGR for cofferdam and seal coat recommendations.
g. Details of foundation type (pile, drilled shaft or spread footing) shall be checked against geotechnical recommendations on file (SGR). Show all foundation design data required per Bridge Manual Section 3.10.
h. Quantity of structure excavation or cofferdam excavation, as applicable, shall be shown for each pier.
i. Specify Concrete Sealer where applicable.
j. Verify seismic design requirements are met. See Sections 3.7 and 3.15.

3.1.14.11 Boring Logs

1. All Boring log sheets shall be included in the plans or a subsurface data profile plot shall be provided. Boring log stations, offsets and elevations shall be tied to the stationing and elevations of the structure. This can be accomplished through a note below the boring logs. See Section 3.1.11.
2. Show all boring locations on GP&E sheet.

3.1.14.12 Standard Details

1. Ensure that all current applicable Standard Base Sheets, such as temporary concrete barrier, expansion joints, drainage scuppers, bar splicer, pile standards, slip forming, etc. are included. If modification of Standard Base Sheet details or notes is required or made, remove Base Sheet designation from the sheet indicating that it is no longer a Standard Base Sheet.

3.1.14.13 Special Provisions

1. Do not provide Special Provisions for items included in the Standard Specifications.
2. Non-standard pay items shall be adequately covered by plan notes or Special Provisions, including their basis of measurement and payment.
3. Include all applicable Special Provisions. The District may be contacted for currently available Special Provisions. Guide Bridge Special Provisions are also available online.

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3.1.14.15 Structural Design Calculations and Pay Item Worksheet

1. Submit a copy of the Final Structure Design Computations and the Pay Item Work Sheets directly to the BBS at the time that final PS&E's are submitted to the District.

3.1.15 Submittals and Timelines for Final Plans

A Plan Development Outline (PDO) shall be prepared in accordance with Section 3.1.13 for each structure requiring a TSL and Final Structure Plans for review and approval. Three (3) copies of the PDO, including copies of the approved TSL, shall be submitted directly to the BBS no later than 9 to 12 months prior to letting, and at least 30 days prior to the anticipated submittal of Final Plans for review.

Final Structure Plans shall be prepared in accordance with Sections 1.3, 1.4 and Section 3, and shall be submitted to BBS for review and approval. Final Structure Plans shall be submitted directly to the BBS no later than 6 to 9 months prior to letting. The Final Structure Plan submittal for review shall include: three (3) sets of 11 x 17 prints of the structure plans and related Special Provisions, a completed Quality Verification Statement and documentation in accordance with Section 1.4.1.

Final Contract Plans, Specifications, & Estimates (PS&E's) for Letting, including structure plans, shall be prepared and submitted directly to the District per the District's schedule requirements (typically, no less than 4 months prior to letting). Final PS&E's shall not be submitted to the District for processing for Letting without the required BBS review and approval of the structure plans in the contract. Final PS&E's shall incorporate all necessary revisions resulting from BBS or District reviews of previous submittals. At the time that Final PS&E's are submitted to the District, one (1) copy of the Pay Item Work Sheet and one (1) copy of the Structure Design Computations shall be submitted directly to the BBS.

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3.2 Deck

This section covers the Department's policies related to bridge deck design and detailing to support loads and elements specified by the AASHTO LRFD Bridge Design Specifications. Normal weight concrete reinforced with steel bars and meeting the material requirements of the Department comprise the standard basis for design.

3.2.1 Concrete Deck Slabs on Stringers

Non-prestressed concrete used for bridge decks shall be designed to have a specified compressive strength of 4.0 ksi, unless otherwise noted on the TSL.

The design of concrete deck slabs on steel or prestressed concrete stringers is located in Sections 3, 4, 5 and 9 of the AASHTO LRFD Bridge Design Specifications. Figures 3.2.1-1 through 3.2.1-2 are based on the Approximate Elastic or "Strip" Method as outlined in Article 9.6.1 and may be used in lieu of complete computations for the standard 8 in. thick slab. These figures are applicable to the design of slabs on steel or prestressed concrete stringers and also to the transverse design of the slab (flange portion) of reinforced concrete deck girder (T-beam) superstructures.

Figure 3.2.1-2 defines the design span and reinforcement clearances. Design stresses are shown in Figures 3.2.1-1. An allowance of 50 lbs. per sq. ft. for future wearing surface is included in the criteria for Figure 3.2.1-1. All supporting elements of new structures shall be designed using an allowance of 50 lbs. per sq. ft. If an allowance of 25 lbs. per sq. ft. for future wearing surface or no allowance for future wearing surface is specified (e.g. due to capacity of existing substructures and/or foundations) for a bridge, the use of Figure 3.2.1-1 is still permitted since the chart is intended for future wearing surfaces "up to 50 lbs. per sq. ft.".

Bridge projects may require or be recommended for smoothness grinding and grooving applications on the bridge deck and approach slabs as specified in Section 2.3.7.8. See Section 3.2.1.2 for design guidelines when smoothness grinding and grooving are applicable.

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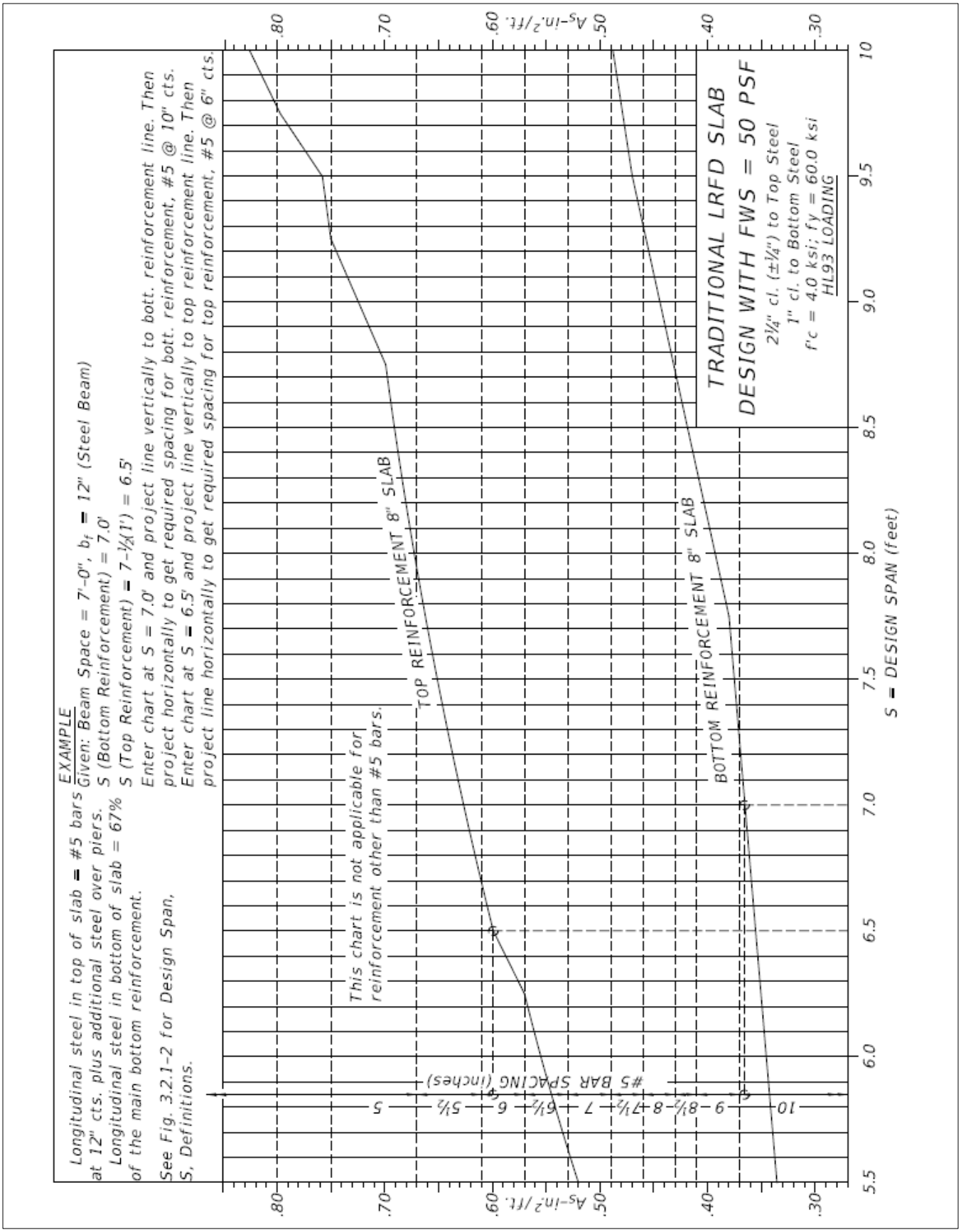


Figure 3.2.1-1

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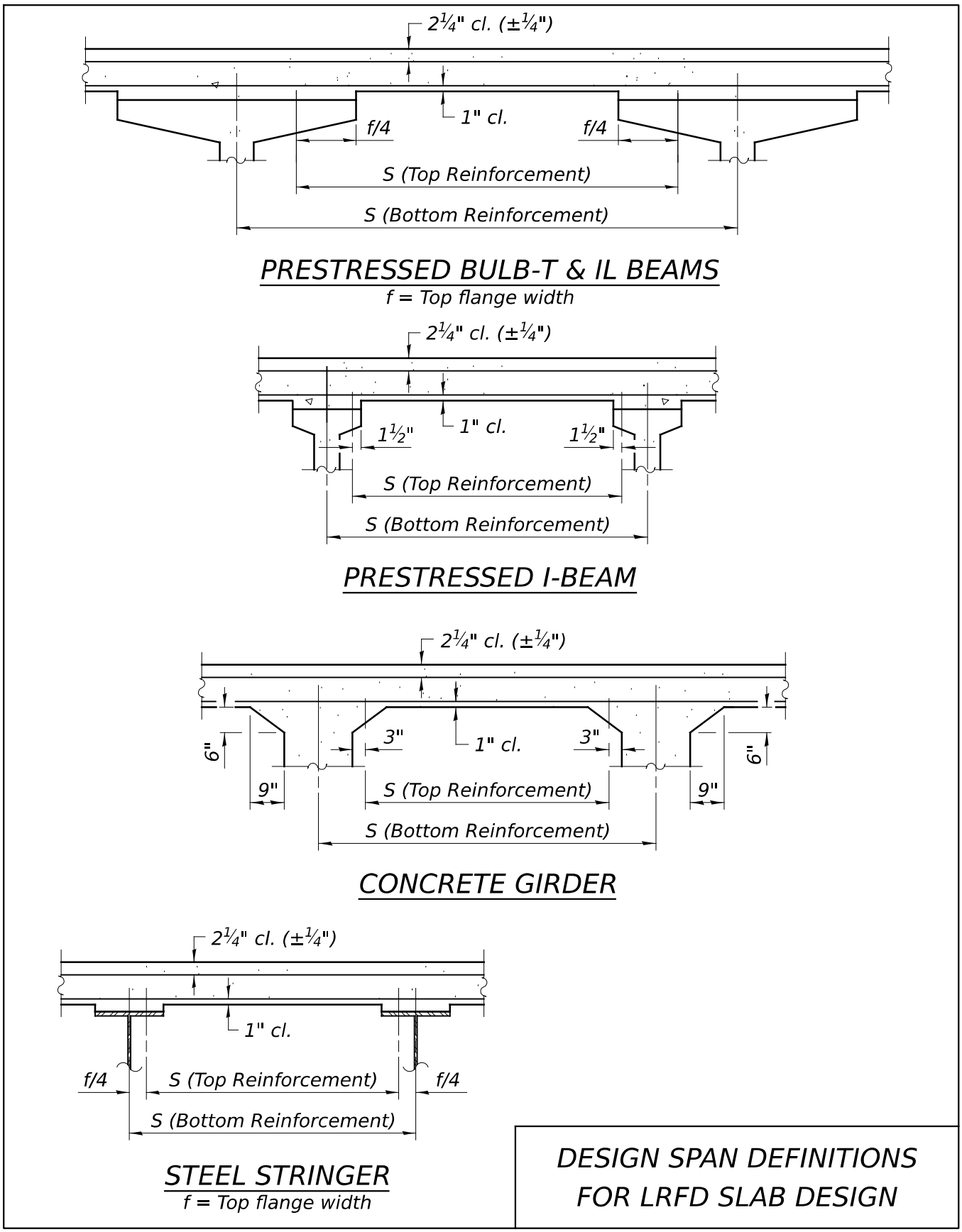


Figure 3.2.1-2

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Figures 3.2.1-1 through 3.2.1-2 were developed based upon the design criteria described in Sections 3.2.1.1 and 3.1.12.1. Designs which fall outside of these parameters require separate computations.

3.2.1.1 Design Criteria Overview

An overview of the design criteria and equations used to develop Figure 3.2.1-1 is given below. As previously stated; design spans, loads, reinforcement bars (#5's), etc. outside the ranges covered by these figures require individual computation with a similar process. More complete references to specification equations, variable definitions, etc. can be found in Design Guide 3.2.1 on the Bridges & Structures page. Article references below are to the AASHTO LRFD Bridge Design Specifications.

Design Stresses:

Concrete $f'_c = 4{,}000 psi$

Reinforcement $f_y = 60{,}000 psi$

Design Thickness:

Slab Thickness 8 in.

Dead Load:

Slab

Future Wearing Surface (FWS)

Live Load: HL-93

Unfactored Moments (Including Impact):

Live Load

$$M_{LL+IM} = \text{Appendix A 4}$$

Dead Load

$$M_{DC \text{ or } DW} = \frac{W_{DC \text{ or } DW} S^2}{10}$$

Where:

S = Design Span

W_{DC} = Slab

W_{DW} = FWS

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Factored Design Moments:

$$M_{LF} = 1.25M_{DC} + 1.5M_{DW} + 1.75M_{LL+IM} \quad (3.4.1\text{-}1)$$

Ultimate Strength Requirements:

$$M_r = \phi\left[A_s f_y\left(d_s - \frac{a}{2}\right)\right] \geq M_{LF} \quad (5.6.3.2.2\text{-}1)$$

Distribution of Reinforcement (Crack Control):

$$f_s \leq \frac{700\,\gamma_e}{\beta_s(s + 2d_c)} \quad (5.6.7\text{-}1)$$

Where:
γ_e = 0.75 for Class 2 exposure condition

Maximum Reinforcement for Over-Reinforced Slabs

Article 5.5.4.2 of the AASHTO LRFD Bridge Design Specifications contains provisions for over-reinforced concrete slabs and beams. When specified reinforcement is excessive, the normal ϕ factor of 0.9 should be reduced.

3.2.1.2 Design Guidelines for Projects with Smoothness Grinding

Planning for grinding and smoothness criteria shall first be investigated per Section 2.3.7.8 prior to implementation of the following guidelines (I-XII) which have been established to assist the designer in preparing bridge plans for structures selected for smoothness grinding. Smoothness grinding shall only be utilized when it has been specified on the approved TSL.

I. Profile Grade

1. The P.G. shown on the approved TSL is the profile grade after the grinding is complete.
2. This P.G. shall be shown on the final plans along with a note stating:

The profile grade shows the final elevations after grinding.

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II. Note on GP&E Sheet

1. To clarify the intent of the plans and for information for future users of the plans, add the following note to the bridge GP&E plan sheet:

   Up to ¼ inch to be ground off the bridge deck and the bridge approach slabs. The Profile Grade shows the final grade after grinding.

III. Design Computations

1. Use the Bridge Manual charts in Section 3.2.1 for 8 inch deck thickness for the design of the bridge deck. Using the charts means that the deck dead load and the deck capacity will both be based on the thinner deck.
2. Use the 8 inch deck thickness for capacity calculations of composite beams.
3. Use the 8 ¼ inch deck thickness for determining the dead load for the design of the beams/girders, the bearings, and the substructure.

IV. Pay Items and Quantities

1. Include the pay item “Diamond Grinding (Bridge Section)”, Sq. Yd. for the smoothness grinding and surface testing work. Calculate the quantity as: (Length from the beginning of one connector pavement to end of other connector pavement) x (inside face to inside face of curbs or parapets minus 4 ft.). See GBSP 59.
2. Include the pay item for “Bridge Deck Grooving (Longitudinal)”, Sq. Yds. Calculate the quantity of grooving as: (Length from beginning of one approach slab to end of other approach slab) x (full width of traffic lanes between shoulders).
3. Calculate the quantity of Concrete Superstructure and Concrete Superstructure (Approach Slab) with the ¼ inch thicker slab.
4. Include pay item for “Protective Coat”, Sq. Yd. on bridge and approach slabs.

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V. Top of Deck Elevation Data

Bridge Deck

1. Use base sheet E-S1.
2. Use the Profile Grade shown on the approved TSL for the calculation of the “Theoretical Grade Elevations”.
3. Calculate the data for the fifth column in the tables, “Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding”. Take the Theoretical Grade Elevations (based on the Profile Grade) and add the deflection that would be caused by an 8 inch deck thickness (and the concrete curb or parapet) and then add an additional ¼ inch to all the values.
4. Include the location of the CL of any bridge joints in the Top of Slab Elevations data tables.
5. When calculating the fillet height using the "Fillet Heights" detail on the base sheet, be sure to subtract the deck thickness prior to grinding. (i.e., 8 ¼ inch for an 8 inch final deck thickness).

Bridge Approach Slab

1. Use the appropriate cast-in-place or precast approach slab base sheet (for example, base sheet E-AS1).
2. Use the Profile Grade shown on the approved TSL for the calculation of the “Theoretical Grade Elevations”.
3. Calculate the “Theoretical Grade Elevations Adjusted for Grinding” by adding an additional ¼ inch to all the values of the Theoretical Grade Elevations (based on the Profile Grade). There is no deflection to be considered here since the approach is either a slab poured on grade or precast (not prestressed) concrete deck beams that are joined with shear keys and a structural concrete overlay.

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VI. Plan Details

1. The top of slab elevations given in plan details for construction (e.g., elevations along the top of the hatched block of a stub abutment) should be based on the elevations before grinding and should have an asterisk with the following note:*"*Prior to grinding".*
2. Dimensions that include the ¼ inch that may be ground off later should have an asterisk with the following note: "*Prior to grinding".
3. The following dimensions should include the ¼ inch that may eventually be ground off:
    a. The bridge deck thickness (add ¼ inch to the normal deck thickness).
    b. The bridge approach slab thickness (add ¼ inch to: CIP approach slab thickness, and/or concrete wearing surface thickness for partial depth precast approach slabs).
    c. The clearance between the top of the deck and the top transverse bars in the bridge deck and in the approach slab (change dimension from 2 ¼ inch cl. (± ¼ inch) to 2 ½ inch cl. (±¼ inch)).
    d. The dimension given between the top of the approach seat and the top of the slab on a section through the abutment.

VII. Seat Elevations

1. On the bridge, determine the bearing seat elevations based on the Profile Grade minus the 8 inch deck thickness.
2. On the bridge approach slab, determine the approach seat elevation by subtracting the approach slab thickness (1'-3" for cast in place; 1'-4" for precast) from the P.G.

VIII. Top of Beam/Web Elevations Table and Camber Diagram

1. The Top of Beam/Web Elevations Tables and camber diagrams should be determined based on an 8 inch deck thickness and the Profile Grade.

IX. Scuppers

1. Scupper types should be positioned so they can be installed outside the potential grinding area (i.e., within 2 feet of the inside face of curb or parapet).

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X. Staging

1. There is no need to grind temporary traffic lanes if they are eventually to be covered by a median or sidewalk.
2. Assume grinding may be delayed, similar to grooving, until two adjacent lanes have been completed.

XI. Preformed Joint Strip Seals

1. For new projects the base sheet note covers smoothness grinding. No base sheet revisions necessary.
2. For Repairs projects, revise the parapet cross section to match the existing.

XII. Modular Joints and Finger Plate Joints

1. The base sheet note covers smoothness grinding. No base sheet revisions necessary.

3.2.2 Reinforcement (Concrete Deck Slabs on Stringers)

All bridge deck reinforcement bars on IDOT maintained structures shall be epoxy coated unless an alternate system is specifically approved by the BBS for the project. Alternate systems may be stainless steel, A1035, galvanized, or TEC (Textured Epoxy Coated) rebar. These systems are currently being evaluated on a case-by-case basis and will be proposed by the Department at the TSL or PDO review phase. Evaluation of alternate systems is not required by the bridge designer. In addition, all reinforcement bars in parapets, sidewalks, medians and solid concrete diaphragms shall be epoxy coated.

Epoxy coated bars shall be indicated by suffixing the bar designation with "(E)". For example, bar a4(E). A separate weight for epoxy coated bars shall be computed and billed as "Reinforcement Bars (Epoxy Coated)" on both the deck detail sheet and the Total Bill of Materials.

Truss bars shall not be used in bridge decks. The maximum size bar permitted in the slab for transverse reinforcement is #6. However, as stated above, Figure 3.2.1-1 is based on #5 bars. If #6 bars are used, they shall be designed by computation. Do not mix bar sizes, i.e. providing #5's and #6's for the main reinforcement. The spacing shall be to an even ½ in., i.e. not 5 ¼ in. but 5 in. The maximum spacing for the bottom and top transverse reinforcement shall be 10 in.

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Article 9.7.3.2 of the AASHTO LRFD Bridge Design Specifications presents the criteria for longitudinal distribution reinforcement in the bottom of slabs when the main reinforcement is transverse to the direction of traffic. In effect, these Articles require (for design spans up to and including about 11 ft.) the amount of distribution steel in the bottom of the slab shall be sixty-seven (67) percent of the main bottom (positive moment) reinforcement in the slab. The longitudinal distribution reinforcement shall be #5 bars and the maximum spacing shall be 15 in.

Bottom distribution reinforcement shall be equally spaced between stringers with the first bar approximately 4 in. from the edge of the flange on steel stringers or 12" from the beam centerline on concrete beams. See Figure 3.2.4-7 and refer to Article 6.10.1.7 of the AASHTO LRFD Bridge Design Specifications.

For instance, if 0.61 sq. in. per ft. is required for the main bottom reinforcement in the slab, #5's at 6 in. centers would be satisfactory, and the required distribution steel would be 0.67 x 0.61 = 0.41 sq. in. per ft.

This area could be furnished by #5's at 9 in. centers spaced as described above between the flange edges (for steel stringers) or relative to the beam centerline (for concrete beams). Indicate the bar spacing, for example, as 8 - #5 bars at 9 in. cts. Do not call for "8 bars equally spaced".

The longitudinal bars in the top of the slab shall be #5's at 12 in. centers. They shall be placed across the full width of the superstructure.

Top and bottom longitudinal bars should not be lapped at the same locations in the deck, nor should the top and bottom transverse bars be lapped in the same locations except when staged construction is utilized.

For continuous steel structures, additional #6 bars shall be equally spaced and placed over the piers between inside-to-inside face of the superstructure parapets. This amount shall be verified to satisfy AASHTO LRFD Bridge Design Specifications Article 6.10.1.7. These bars shall extend to the location to where they are no longer required. See Figure 3.2.4-8. The maximum allowable reinforcement size for PPC beams are #9 bars (for top longitudinal reinforcement) and #8 bars (for bottom longitudinal reinforcement).

The Department will continue to design concrete beams as continuous over the intermediate supports for composite dead load and live load plus impact. Figure 3.2.4-9 depicts the guidelines for the bridge deck reinforcement requirements over a pier. The principles of mild reinforced concrete design shall be

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used with the reinforcement in the bridge deck acting as the tensile portion of the resisting moment couple.

For continuous PPC I-beam, bulb T-beam, and IL-beam bridges, the amount of additional reinforcement required over piers may be greater than the #6 bars at 12 in. centers described above, if required for design. These bars shall extend to the location to where they are no longer required.

Also note that to create an adequate continuity connection at the bottom of the IL-Beams, the conventional mechanically spliced #8 bars have been replaced with extended beam strands. The required number, length and configuration are detailed on each IL-Beam base sheet.

3.2.2.1 Reinforcement (Treatment of Decks at End Diaphragms)

Figures 3.2.2.1-1 and 3.2.2.1-2 present the details for concrete edge beams for steel and concrete beam superstructures respectively. The reinforced concrete edge beam shall be designed to resist the entire wheel loads, and its own dead load. The concrete edge beams shall be placed from fascia beam to fascia beam and not on the overhangs of the structure.

The design of transverse edge beams over steel and concrete girders shall be according to AASHTO LRFD Bridge Design Specifications Article 9.7.1.4. The Department has developed a design aid for typical edge beams for both the top and bottom reinforcement as shown in Figure 3.2.2.1-3. The reinforcement consists of four bars on the top and bottom of the edge beam, placed along the skew. These bars shall be placed within the horizontal one foot nine inch dimension found in Figures 3.2.2.1-1 and 3.2.2.1-2. This horizontal one foot nine inch dimension is a minimum and may be increased as necessary to alleviate bar congestion due to high skews, or to ensure that the edge beam is resting on the end diaphragm for steel bridges. This design aid may be used for both top and bottom main reinforcement in edge beams, and for both steel and precast prestressed concrete beams. This design aid is based on the following criteria:

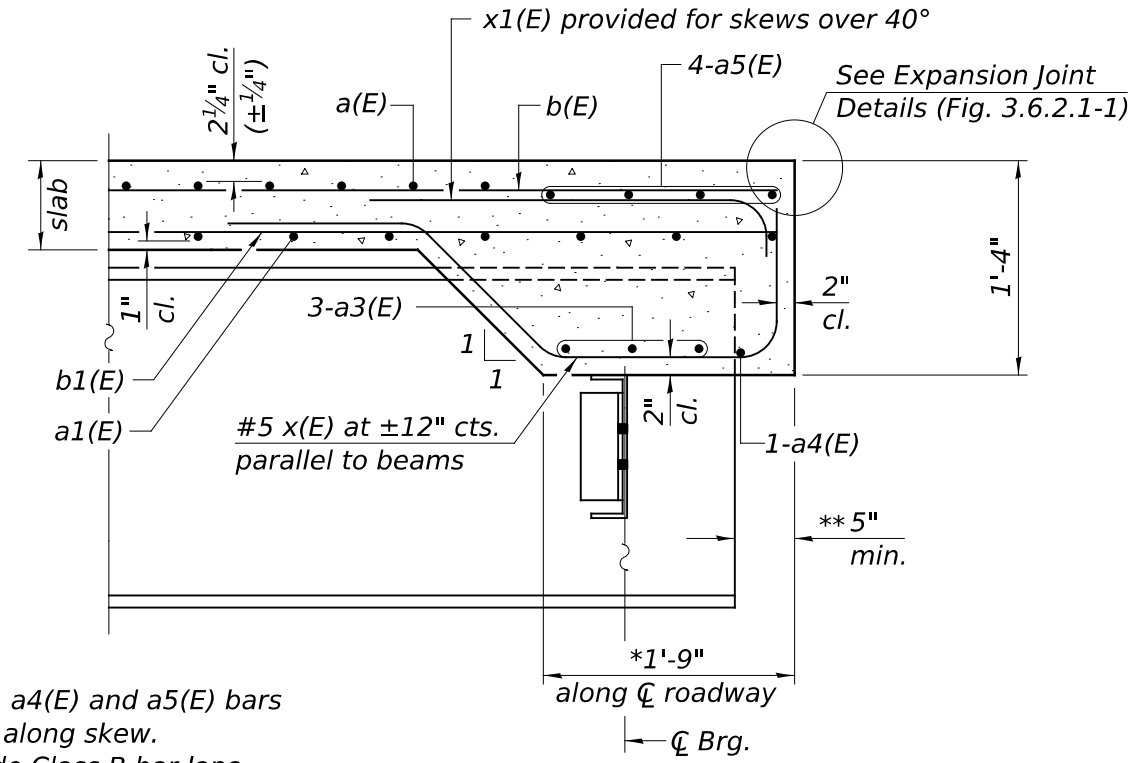
1. f'_c = 4.0 ksi
2. f_y = 60 ksi
3. Effective edge beam width = 21 inches
4. Beam Height = 16 inches
5. Dynamic Load Allowance = 75%
6. Live load moments are derived from AASHTO LRFD Bridge Design Specifications Table A4-1.
7. Maximum design span (normal to the bridge beams) is 9.5 feet.

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To use the design aid, the skew and design span (normal to the beams) are required. Use Figure 3.2.1-3 to determine the design spans for both the positive and negative reinforcement.

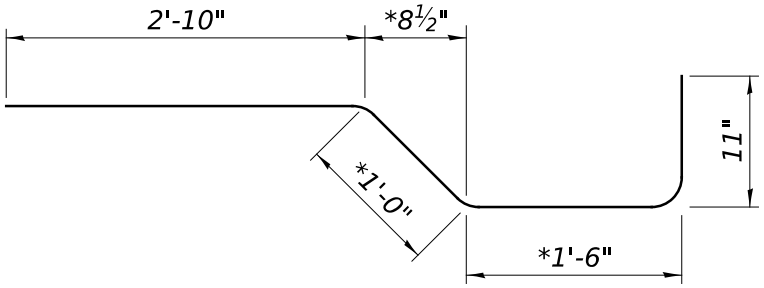
For bridges with a skew greater than 40 degrees, additional #5 x1(E) bars shall be provided to reinforce the cantilevered slab overhanging the end diaphragm or cross-frame. X1(E) bars shall also be provided at the ends of cantilevered deck slabs on both sides of any hinge in a framing plan. The x1(E) bars shall be placed parallel to traffic at ±12 inch centers between beams, within the limits of the end diaphragm.

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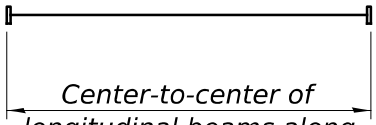


Notes:
a3(E), a4(E) and a5(E) bars placed along skew.
Provide Class B bar laps.

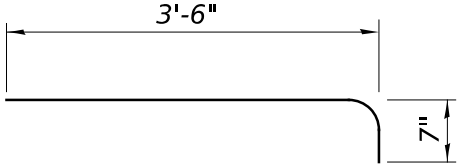
SECTION AT EXPANSION JOINT



BAR x(E)



BAR a3(E)



BAR x1(E)

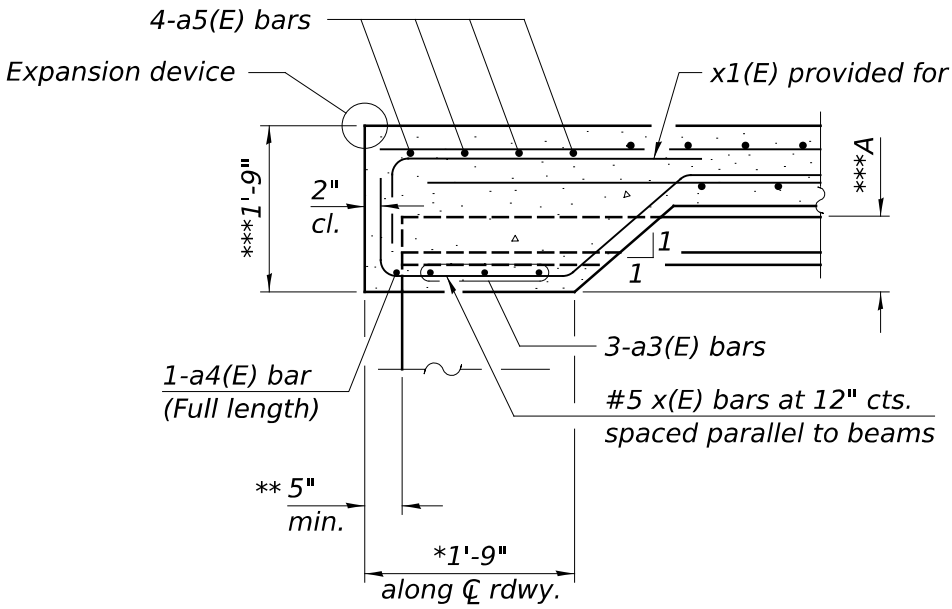
Bar size based on design for beam spacing and skew.

* Minimum effective edge beam width dimension shown. Dimension may be increased to alleviate bar congestion in high-skew scenarios.
** 2" cl. measured at flange for skewed structures.

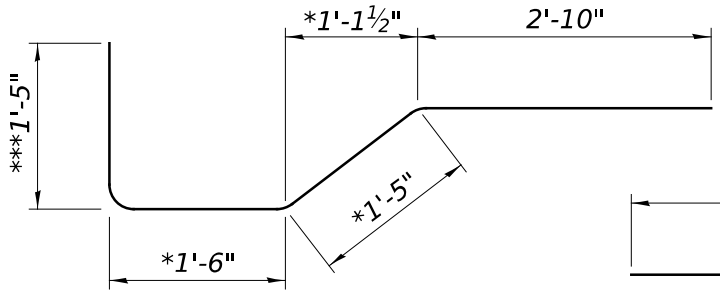
EDGE BEAM AT EXPANSION JOINT FOR STEEL BEAMS

Figure 3.2.2.1-1

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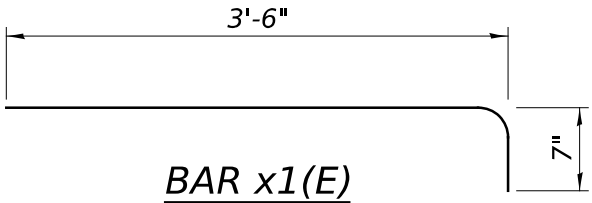


SECTION AT EXPANSION JOINT

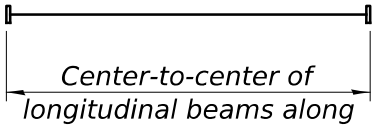


BAR x(E)

Notes:
a3(E), a4(E) and a5(E) bars placed along skew.
Provide Class B bar laps.



BAR x1(E)



BAR a3(E)

Bar size based on design for beam spacing and skew.

* Minimum effective edge beam width dimension shown. Dimension may be increased to alleviate bar congestion in high-skew scenarios.

** 2" cl. measured at flange for skewed structures.

*** Increase as required to accommodate beam camber and to keep a3(E) bars below the top flange of the prestressed beam.

EDGE BEAM AT EXPANSION JOINT FOR PPC BEAMS

Figure 3.2.2.1-2

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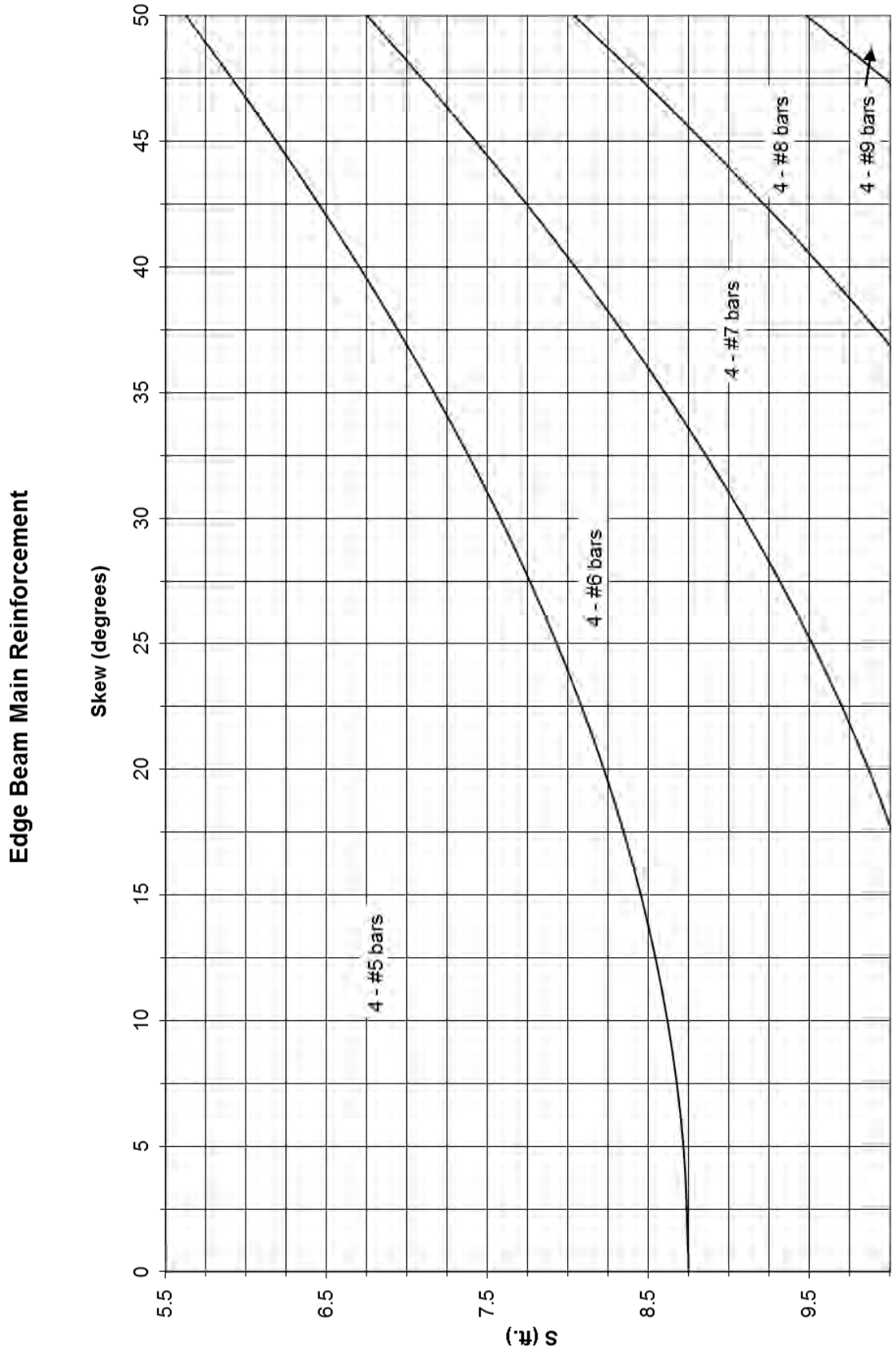


Figure 3.2.2.1-3

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3.2.3 Reinforcement (Treatment of Skewed Decks)

If a bridge is skewed 15 degrees or less and spans less than 50 ft. back-to-back of abutments, the main reinforcement shall be placed parallel to the skew. Design the reinforcement as if it were at right angles to the stringers, and then multiply the required area by the secant of the skew angle squared, i.e. A_s x $Sec^2(\angle)$. The transformed area of steel, results in a bar size and spacing that is at right angles to the main reinforcement, which is parallel to the skew. The transformed (increased) area of steel accounts for increased moments of a longer actual span, which is measured along the reinforcement's skewed orientation. If the bars are to be detailed parallel to the beams on the plans, the perpendicular spacing of the bars should be further multiplied by $Sec(\angle)$. The result is spaced along the beam centerlines. .

If a bridge spans more than 50 ft. back-to-back of the abutments and is skewed, detail the main reinforcement bars for the full length of the slab placed at right angles to the stringers. The "Superstructure-(*Type*)Beams" in the IDOT CADD Bridge Cell Library refers the designer to the "Field Cutting Diagram" presented in the upper-right corner of the "Superstructure Details" (SDI-BT, SDI-I, SDI-IL, or SDI-SB) base sheets to provide the necessary cut line, cut lengths, and bar designations/placement.

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3.2.4 Parapet, Railing, & Sidewalk Sections

Parapet and sidewalk sections are shown in Figures 3.2.4-1 through 3.2.4-15.

The fascia of these sections (that portion of the concrete visible in direct elevation outside of the exterior beam) shall be a constant depth for the full length of the bridge. It shall also afford continuous concealment of the top flange of the exterior beam for steel beam bridges. The dimensions of the vertical surface are standard. The vertical dimension of the sloped undersurface of the section shall be computed for each structure by the designer. To establish this dimension, the designer should accurately estimate the maximum actual depth of slab plus fillet over the top of the outside stringer considering all deflections and camber. Figures 3.2.4-1, 3.2.4-4, and 3.2.4-13 establish ¼ in. as the minimum dimension below the bottom of the top exterior beam flange to the lower edge of the sloping surface for steel beams with standard overhangs. Figure 3.2.4-14 establishes ½ in. as the minimum distance above the bottom of the top exterior beam flange to the lower edge of the sloping surface for prestressed I-Beam, Bulb T-Beams, and IL-Beams. The dimension varies throughout the structure and its maximum and minimum shall be shown on the plans.

Note that the vertical depth of the sloping surface should not exceed 5 in. nor be less than 2 in. as indicated in Figures 3.2.4-1 and 3.2.4-4. In Figure 3.2.4-11, the vertical depth of the sloping surface should range between 5 in. and 3 in.

For structures on horizontal curves with variable overhangs, the depth of the sloping surface should be held close to the 2 in. minimum. The overall depth of the fascia, however, may be different on each side of the structure.

The concrete parapets shown in Figure 3.2.4-1 and 3.2.4-4 shall be used on urban and rural structures where appropriate. See Section 2.3.6.1.7.

See Figure 3.2.4-15 for alternate parapet reinforcement applicable only for interior parapet locations associated with Base Sheets R-29 and R-33.

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3.2.4.1 Parapet Joints (Concrete Deck on Stringers)

To control cracking in the parapets in the negative moment areas, a joint in the parapet which is full height shall be placed over the supports on all continuous structures; in addition, these full height joints are placed at 0.6 the average distance to the points of dead load contraflexure (calculated using only DC1 and DC2 loads) on both sides of the support if either span is more than 50 ft. For bridges with long spans, 0.6 of the average distance to the points of dead load contraflexure may be greater than 20 ft. For these cases, place the joints at 20 ft. The joint consists of an aluminum plate that is full height of the parapet. No reinforcement shall pass through the aluminum plate. The reinforcement bars shall not be cut in the field but shall be properly dimensioned and listed in the Bill of Materials.

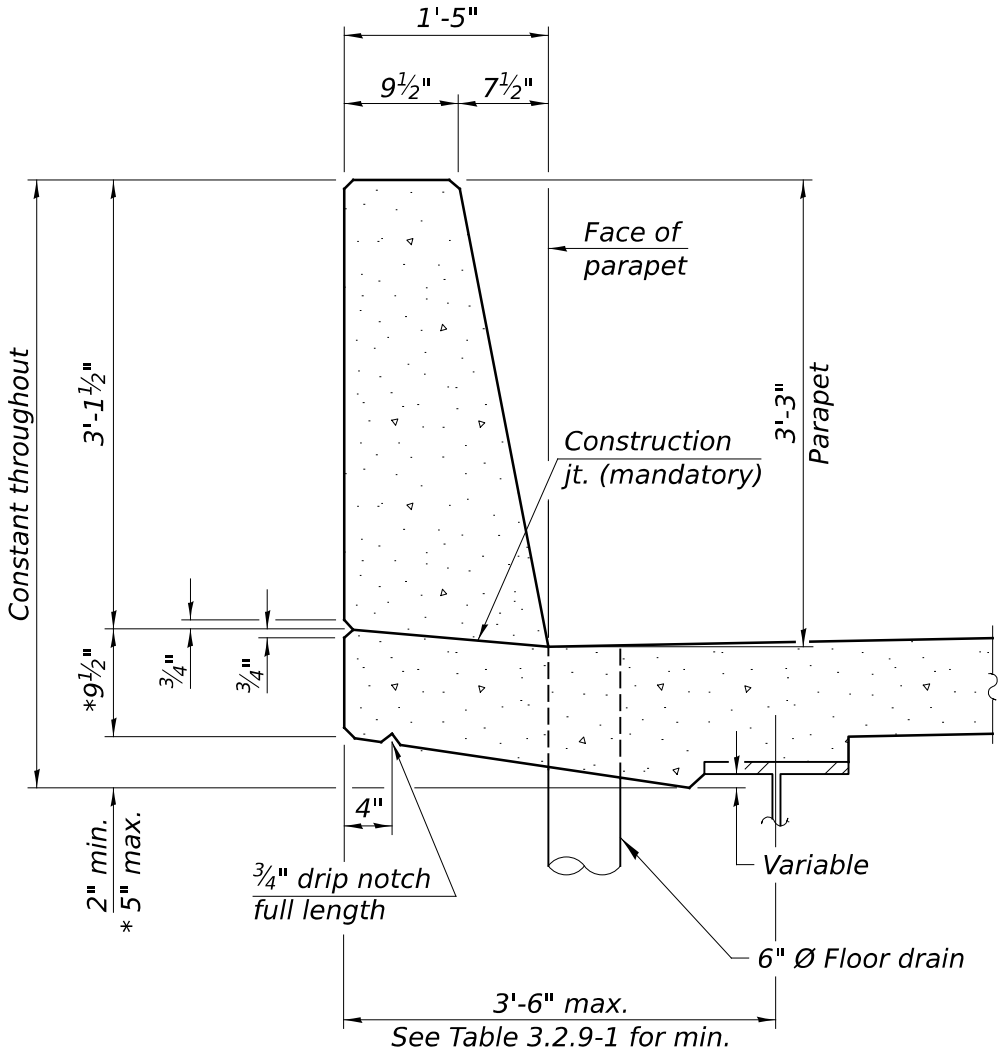
Full height parapet aluminum joints shall also be placed at the back of integral and semi-integral abutments.

Joints in the top portion of the parapet shall also be placed at 14 ft. – 0 in. to 20 ft. – 0 in. intervals outside of interior support regions. These joints shall only consist of cork filler and polyurethane sealant and should be located as uniformly and symmetrically as possible within a span. See Figure 3.2.4-10.

For bridges with light poles, parapet joints shall be placed a minimum of 4 ft. – 9 in. from the center of the light pole support. Parapet joint locations, bridge rail post locations, light pole locations, and sign structure support locations shall be coordinated by the design engineer to avoid conflicts or interferences.

See Figures 3.2.4-1, 3.2.4-4, 3.2.4-8, 3.2.4-9 and 3.2.4-10 for more complete information and details on parapet joints.

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Cross sectional area of parapet = 3.50 ft^2
Protective Coat Area (Top & Front Face) = 4.1 ft^2/LF

** When more than 5" is required, add excess to 9½" dimension.*

39" CONSTANT-SLOPE CONCRETE PARAPET CONFIGURATION

Figure 3.2.4-1

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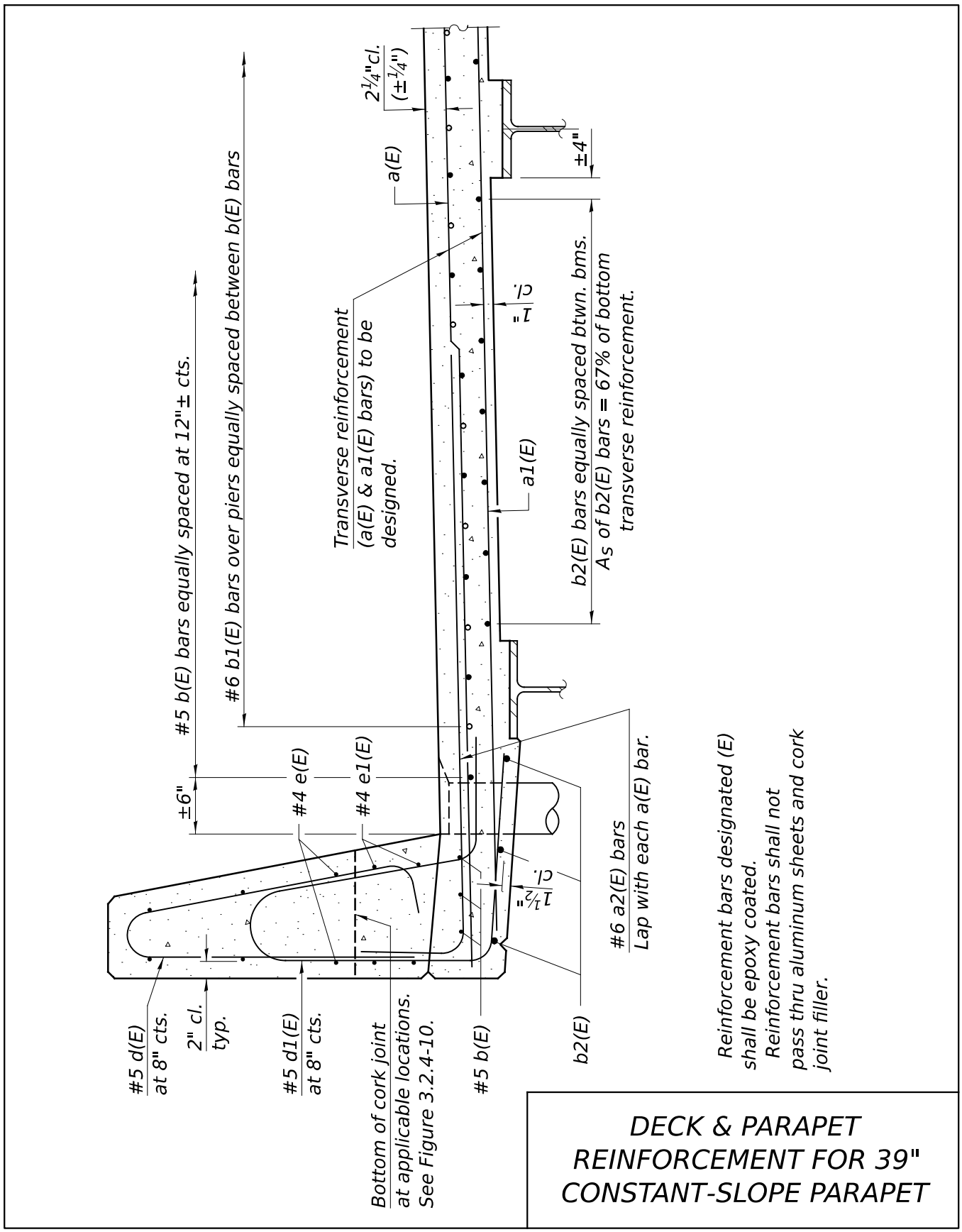
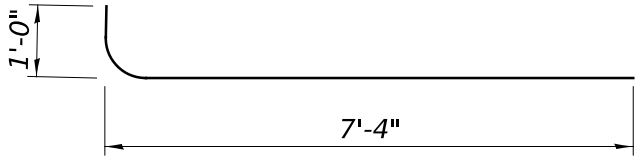
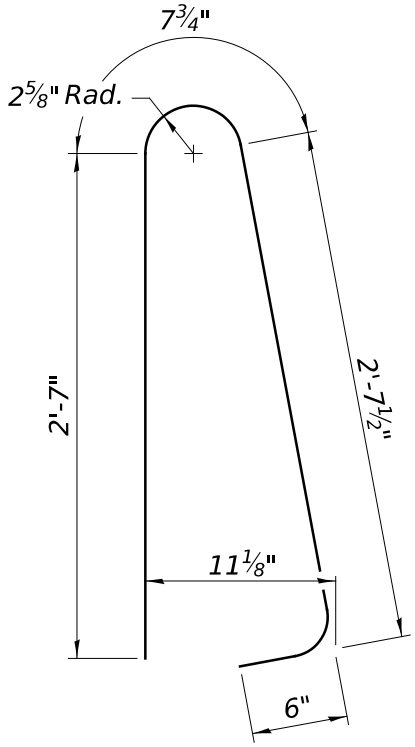


Figure 3.2.4-2

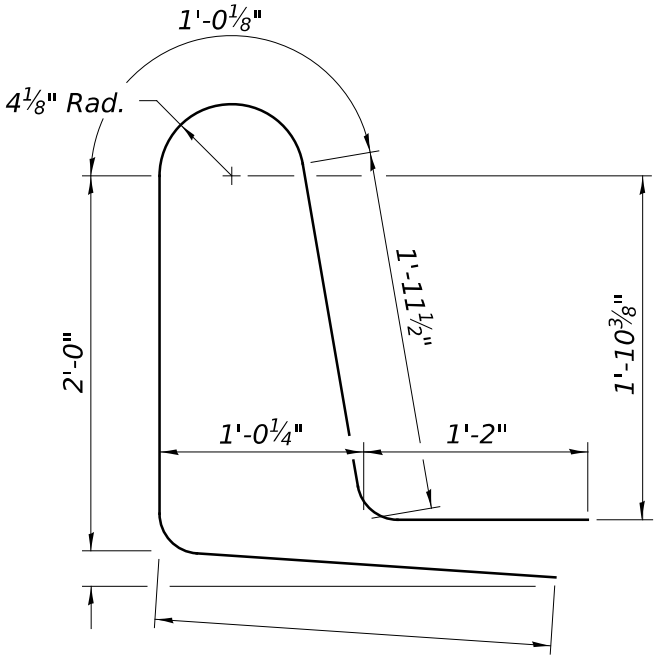
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BAR a2(E)



BAR d(E)

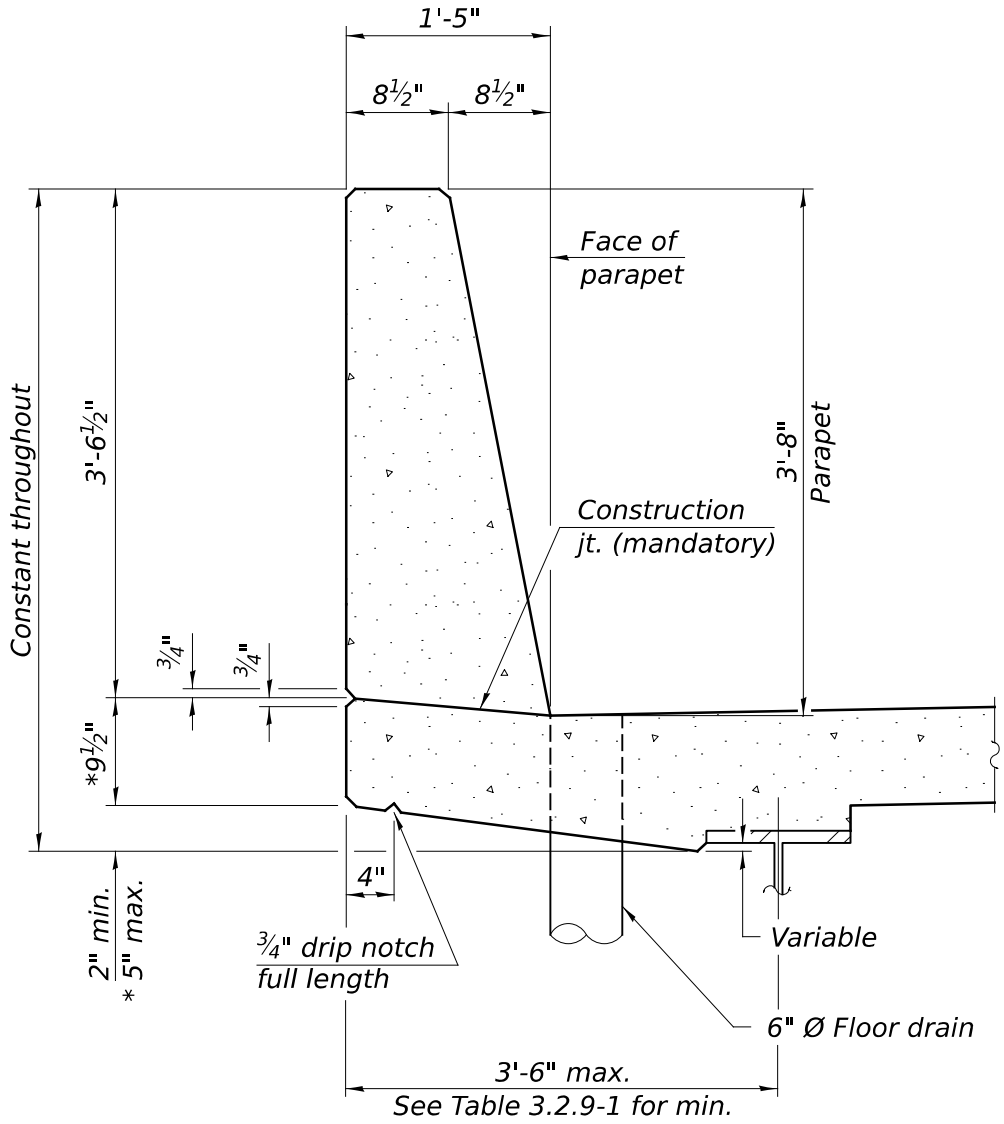


BAR d1(E)

DECK & PARAPET REINFORCEMENT FOR 39" CONSTANT-SLOPE PARAPET

Figure 3.2.4-3

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Cross sectional area of parapet = 3.80 ft²
Protective Coat Area (Top & Front Face) = 4.44 ft² /LF

** When more than 5" is required, add excess to 9½" dimension.*

44" CONSTANT-SLOPE CONCRETE PARAPET CONFIGURATION

Figure 3.2.4-4

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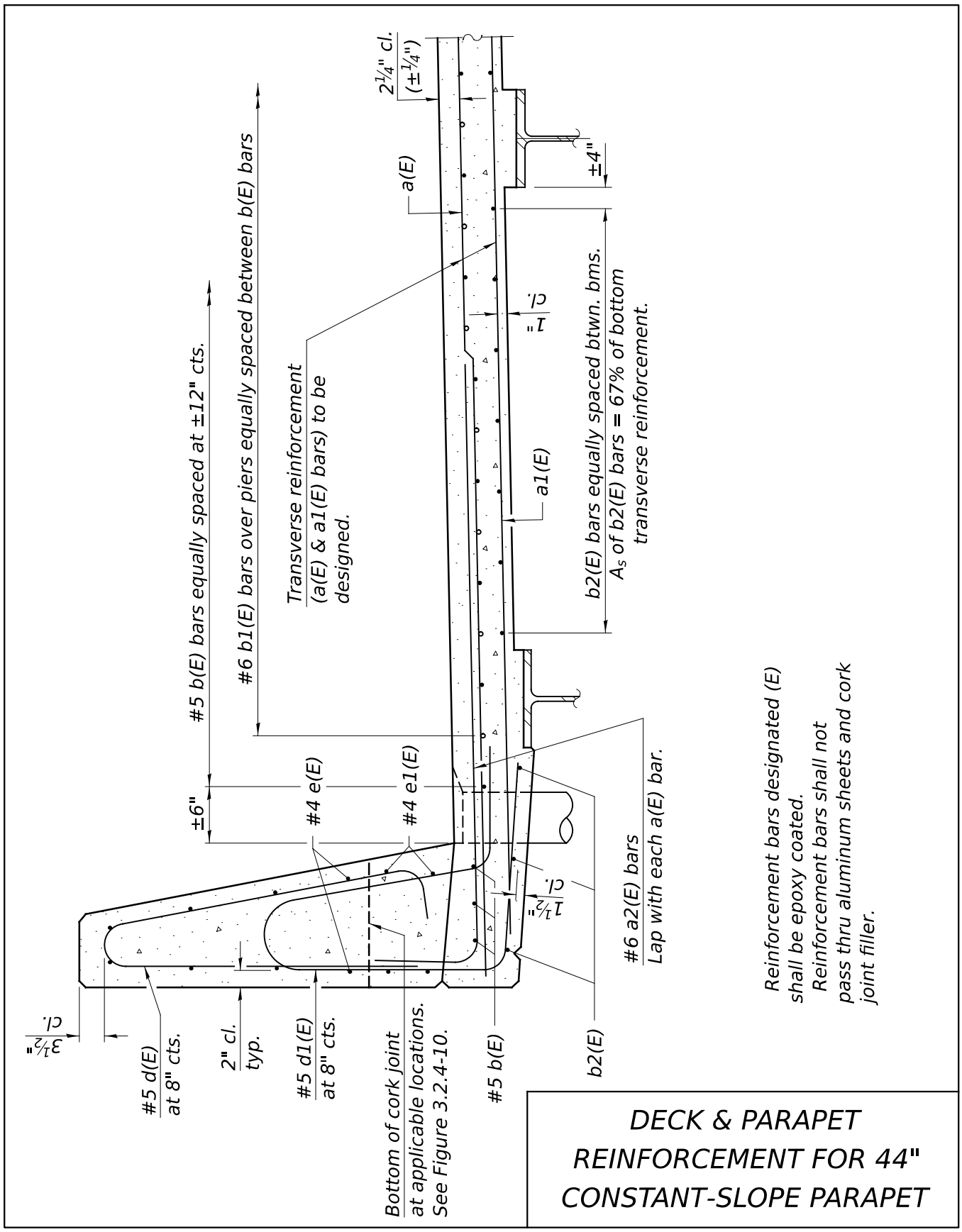
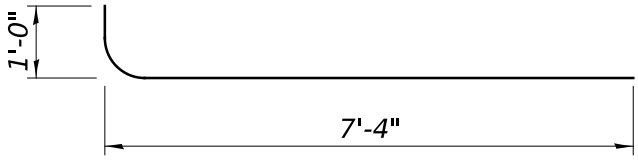
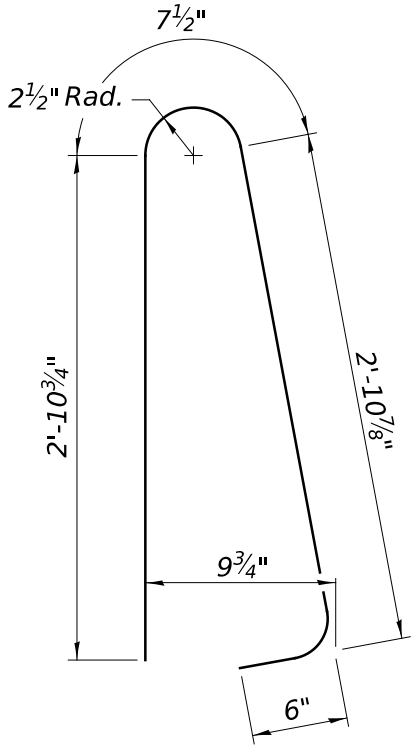


Figure 3.2.4-5

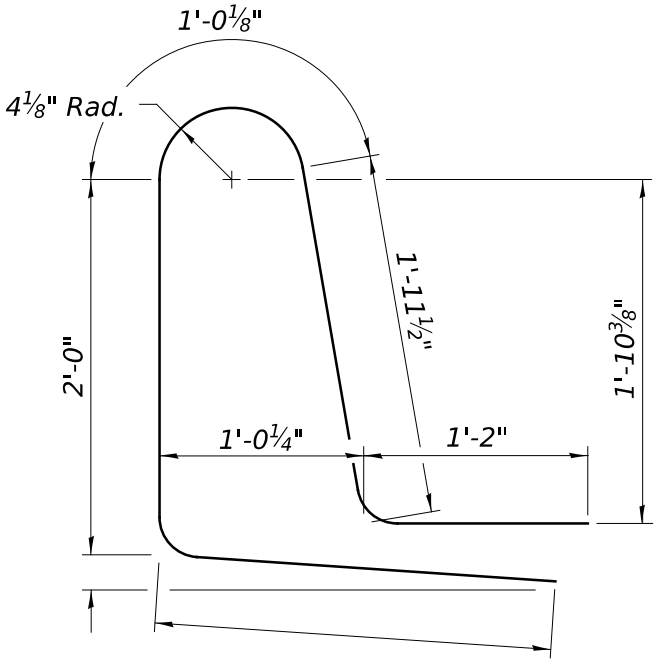
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BAR a2(E)



BAR d(E)

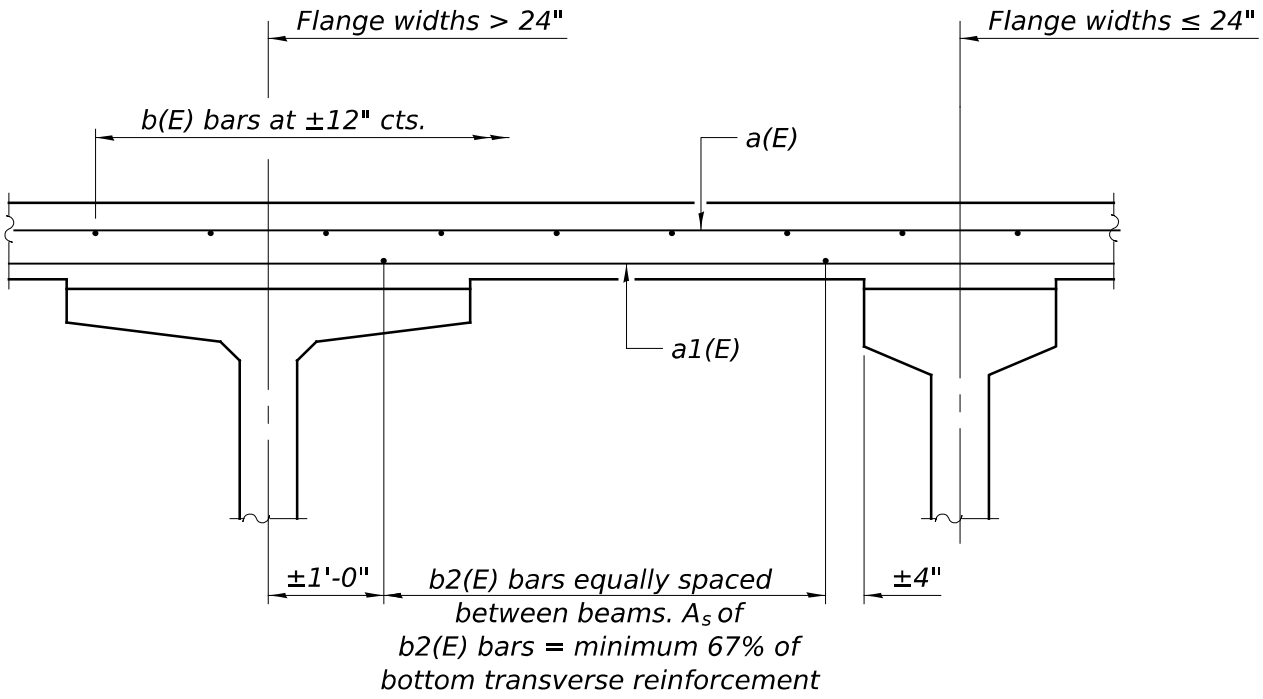


BAR d1(E)

DECK & PARAPET REINFORCEMENT FOR 44" CONSTANT-SLOPE PARAPET

Figure 3.2.4-6

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SECTION NEAR MID-SPAN

Note:
See Design Guide 3.4 for reinforcement details near pier. Bottom longitudinal reinforcement may also be increased to satisfy the longitudinal reinforcement demand near pier as noted. Transition longitudinal reinforcement near pier to longitudinal reinforcement near mid-span as detailed on the base sheet and in Design Guide.

DECK REINFORCEMENT FOR PRESTRESSED BEAMS

Figure 3.2.4-7

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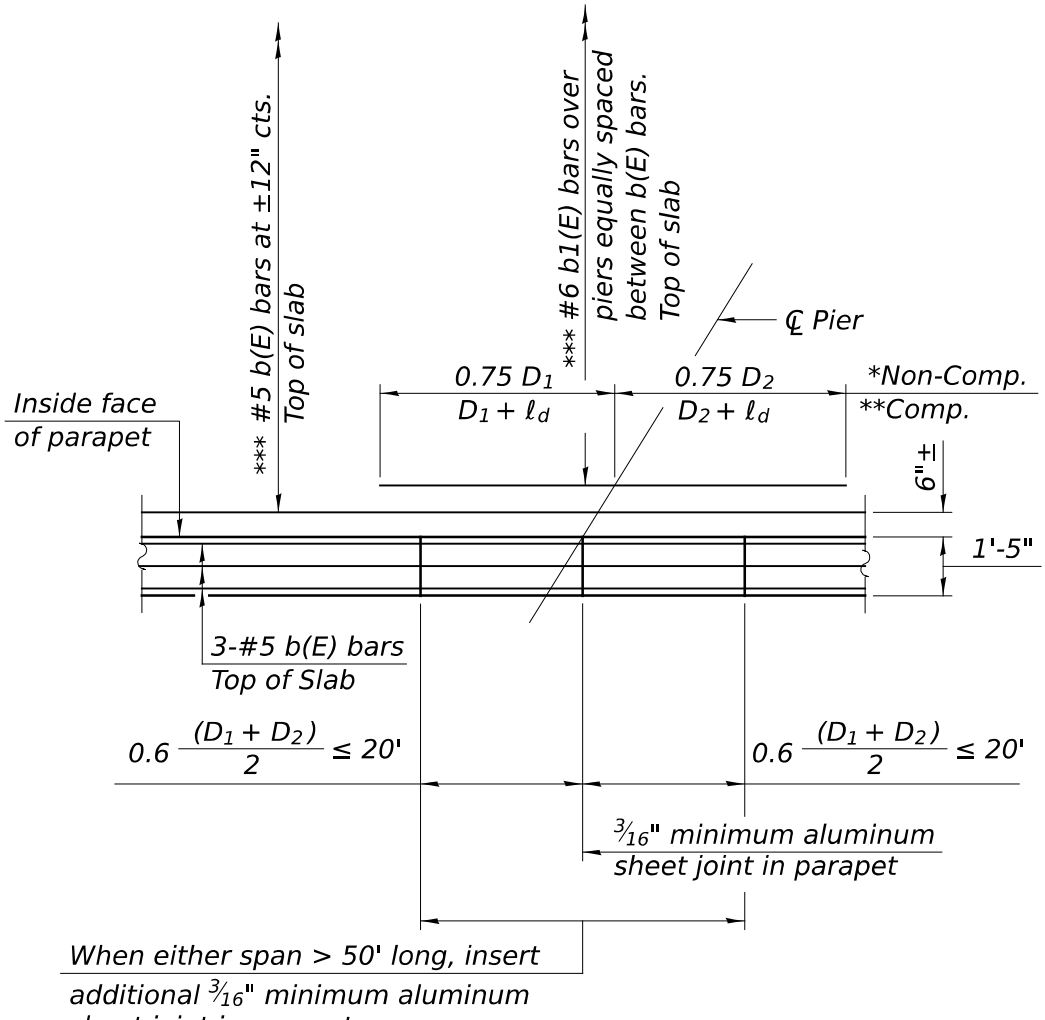
* *For composite stringers when shear connectors are omitted in the negative moment region the reinforcement bars shall extend beyond all additional anchorage connectors at least 40 times the bar diameter.*

** *If splice present at either end of negative moment region, the reinforcement bars shall extend beyond the end of the top splice plate at least one development length.*

*** *Designer to verify that bars satisfy AASHTO 6.10.1.7.*

D = Distance from ℄ Pier to point of dead load contraflexure in span being considered.

D_1 - Span 1
D_2 - Span 2



PARTIAL PLAN AT PIER

DECK REINFORCEMENT & JOINTS IN PARAPET FOR STEEL BEAMS AT PIERS

Figure 3.2.4-8

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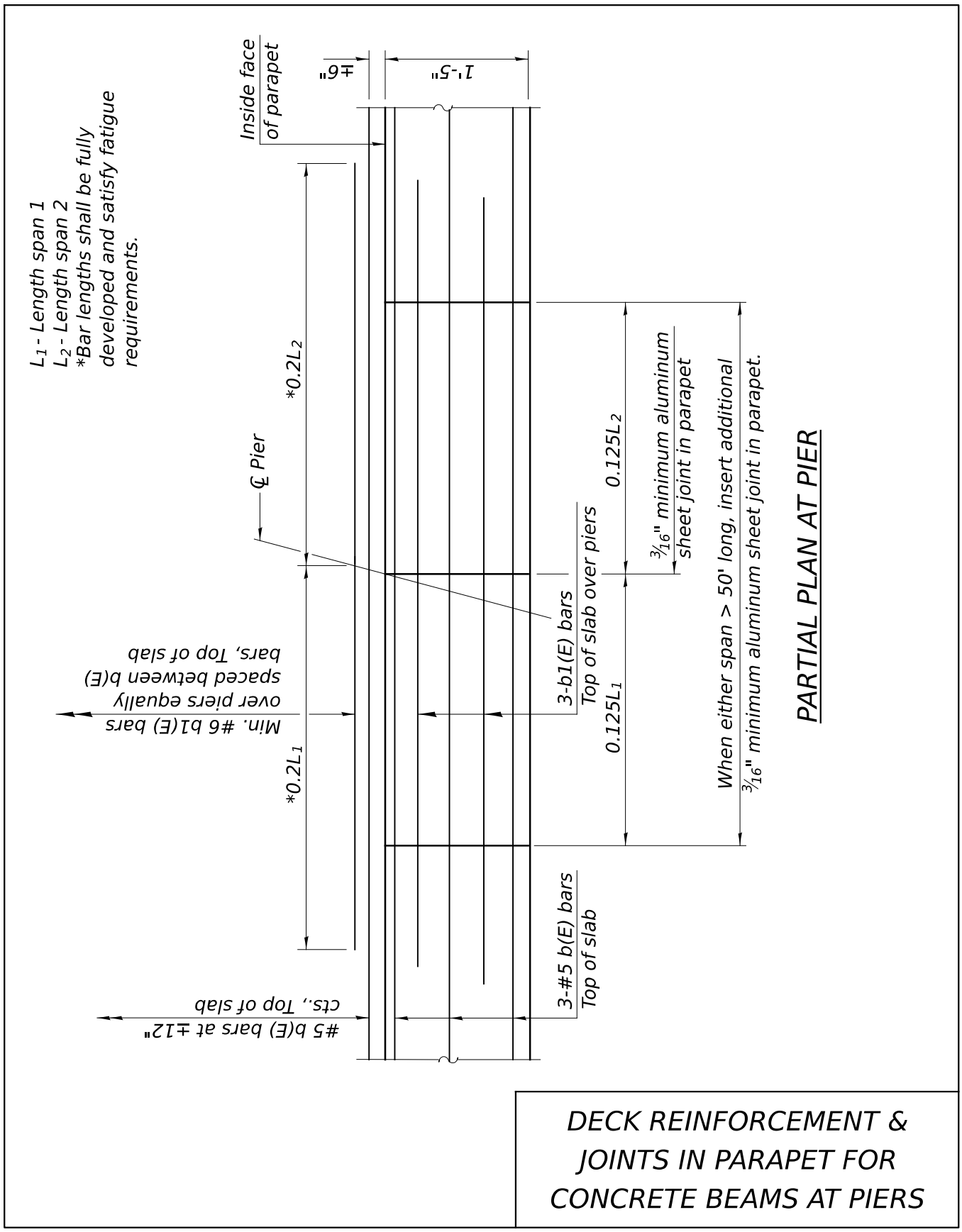
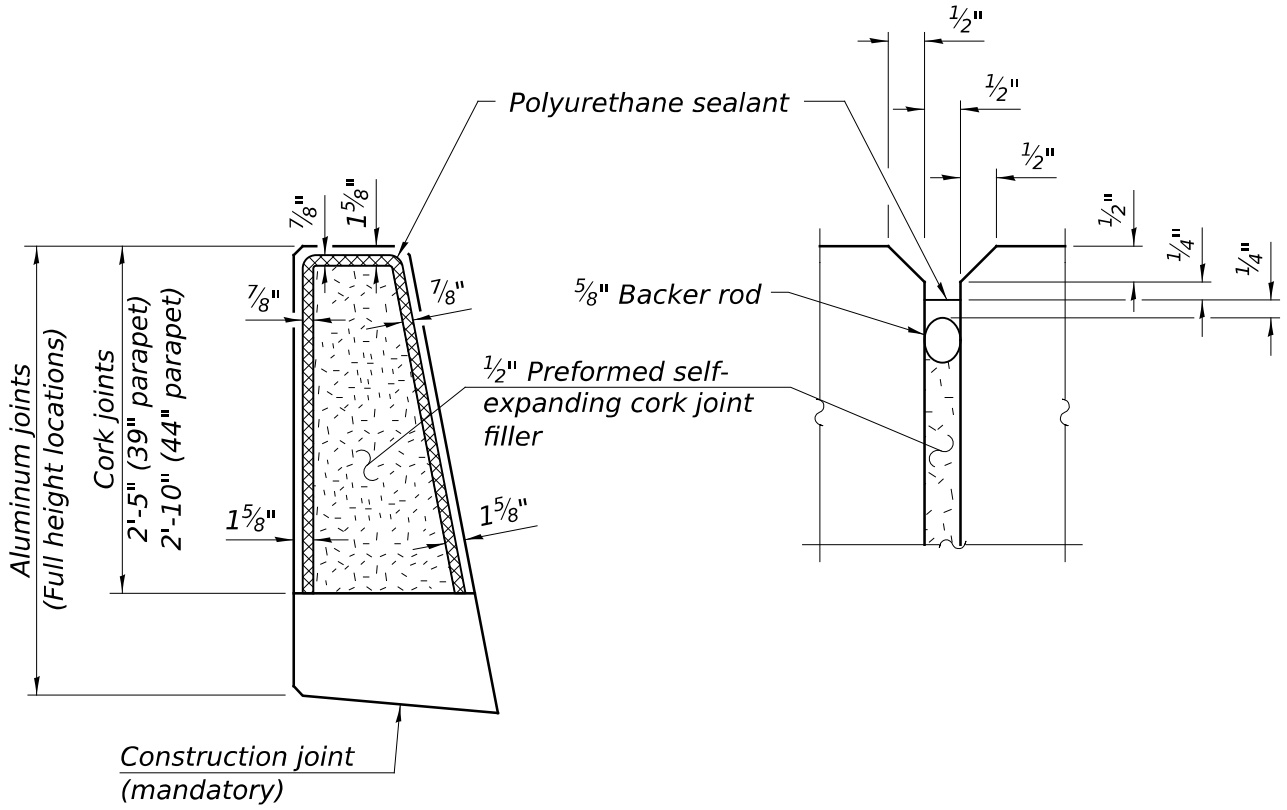


Figure 3.2.4-9

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Note:
The polyurethane sealant shall be according to Article 1050.04 of the Standard Specifications and the color shall be gray.

PARAPET JOINT DETAILS

Figure 3.2.4-10

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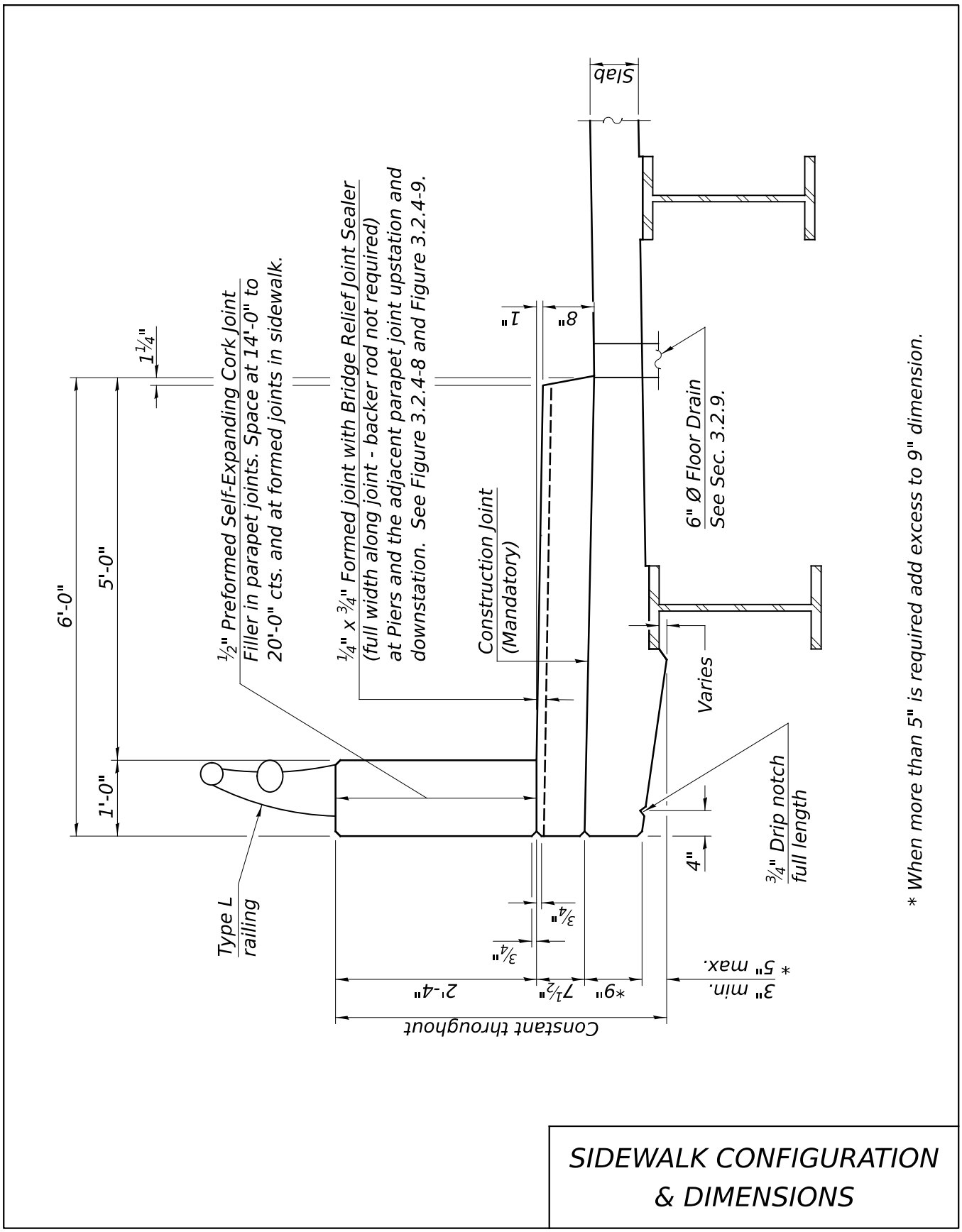


Figure 3.2.4-11

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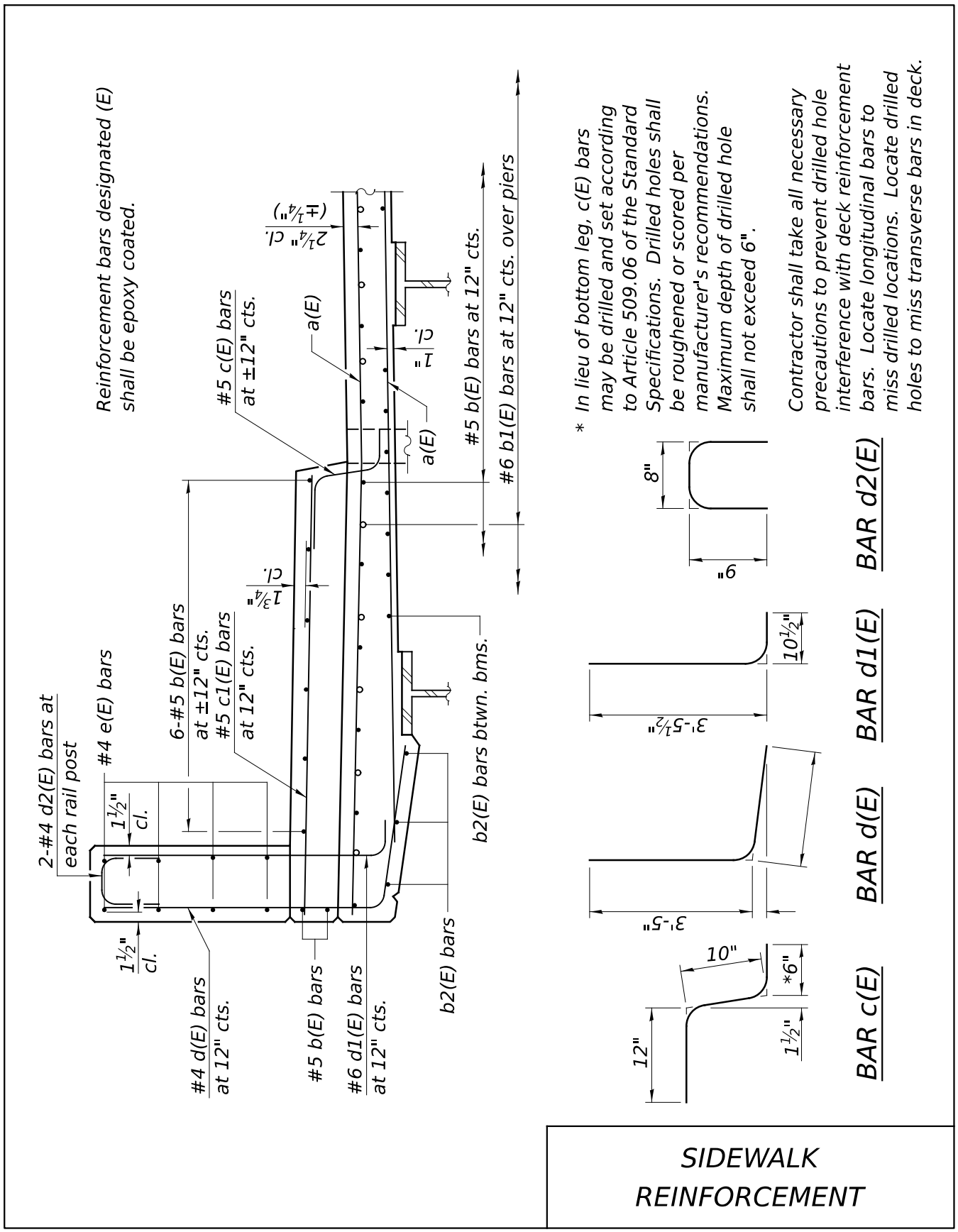


Figure 3.2.4-12

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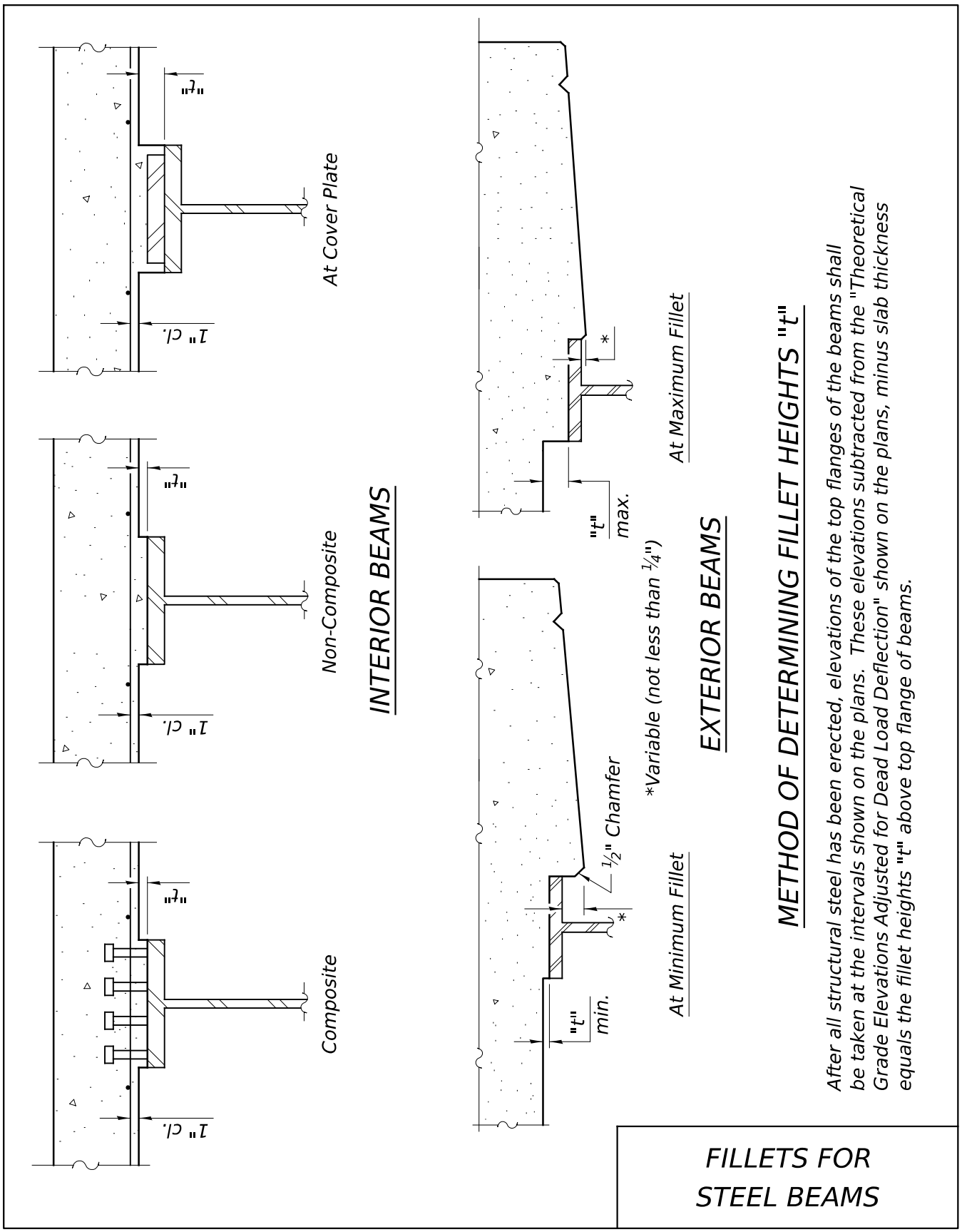


Figure 3.2.4-13

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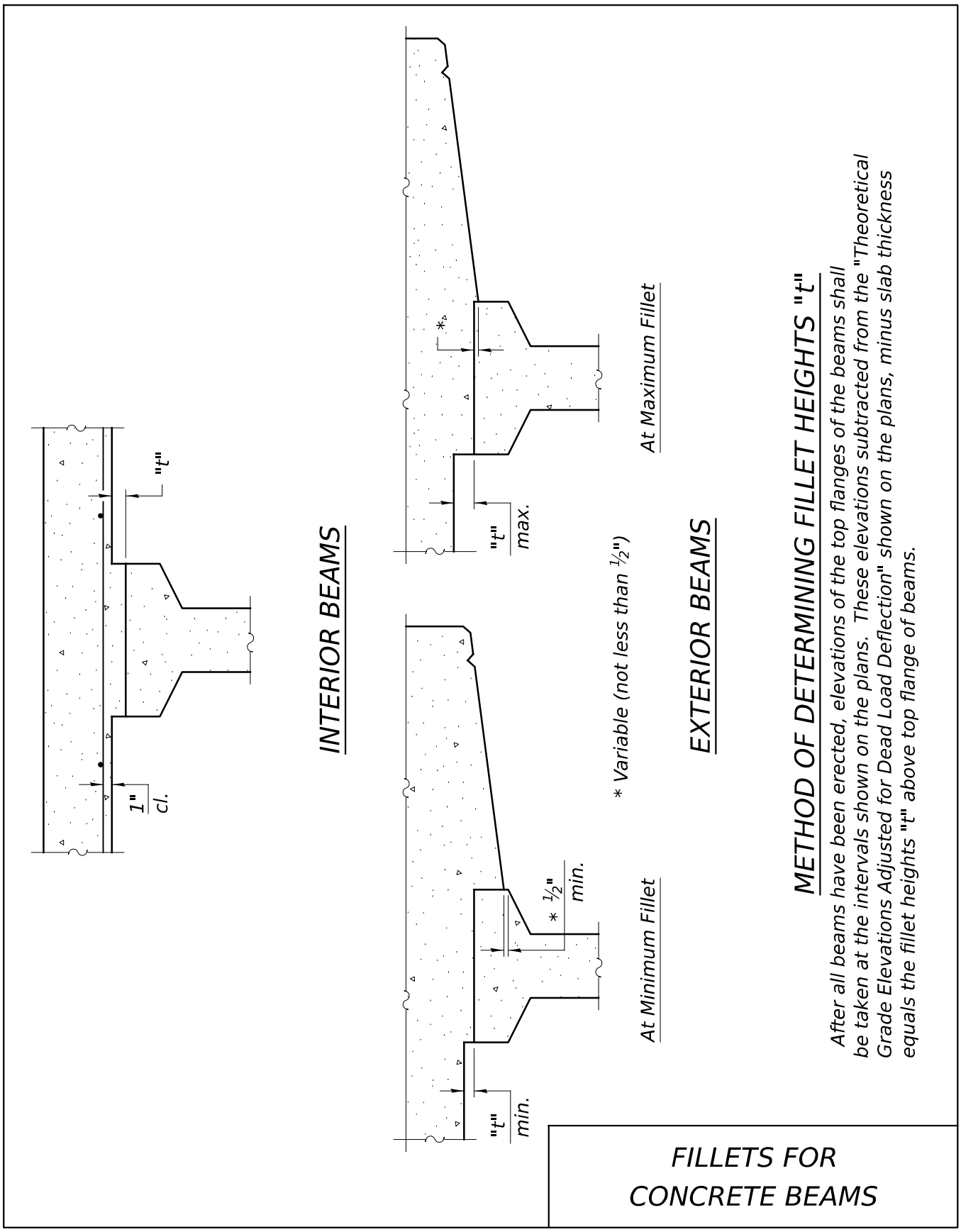


Figure 3.2.4-14

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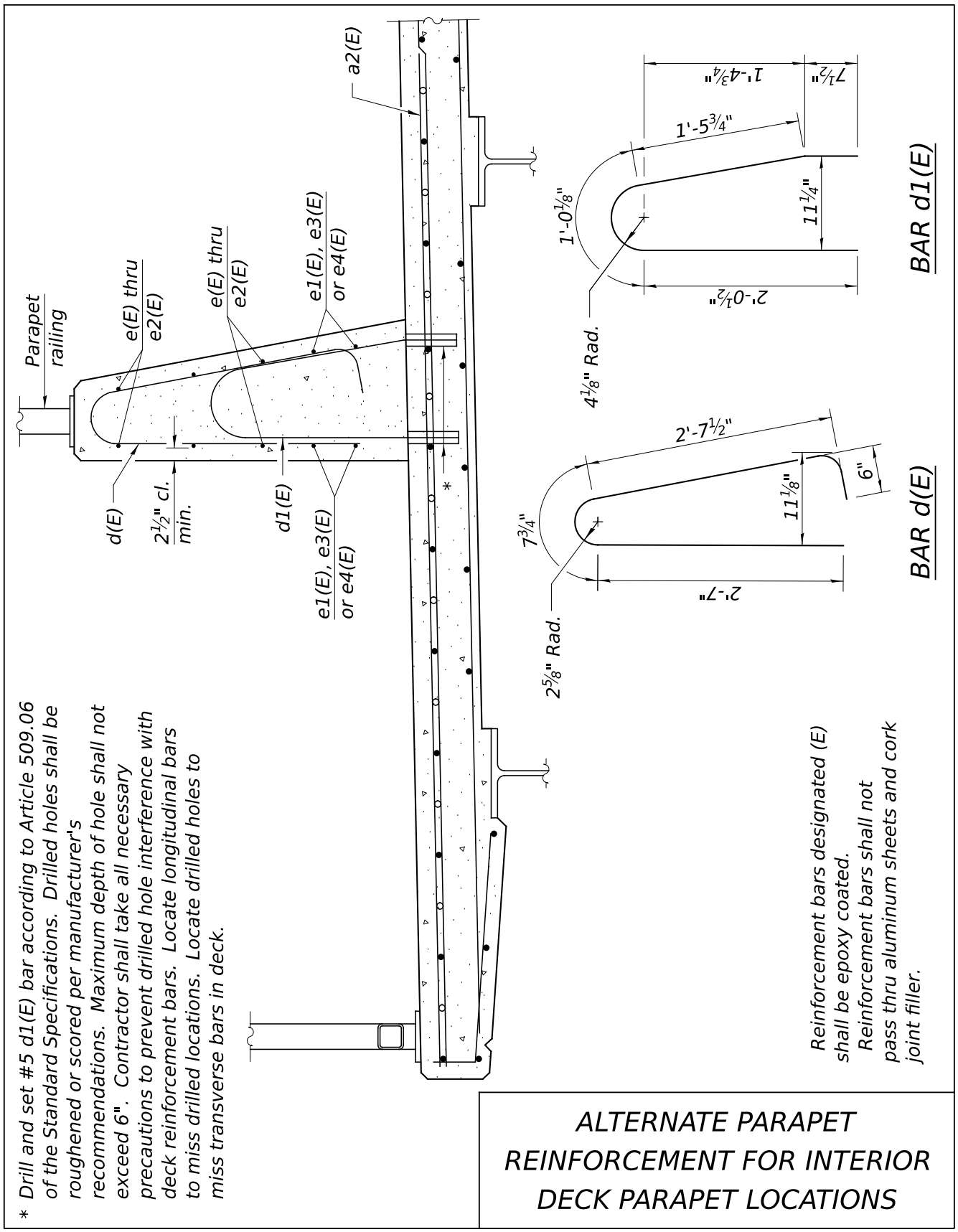


Figure 3.2.4-15

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3.2.4.2 Slipform Parapets

Slipforming of parapets may be allowed with District/Owner approval as determined at the Plan Development Outline (PDO) phase of the project. Projects that meet the following criteria shall not be considered for slipforming:

1. Parapets taller than 44 inches.
2. Back-to-back parapets similar to Figure 3.2.7-2.
3. Parapets with less than 2 feet of horizontal clearance to an obstruction such as an adjacent bridge or other structure.
4. Projects where aesthetic patterns requiring the use of form liners is specified.
5. Projects in regions where there are known problems with either concrete consistency or timely delivery.
6. Curved parapets on a radius of 1500 ft or less

Additional consideration shall be made regarding the following cases:

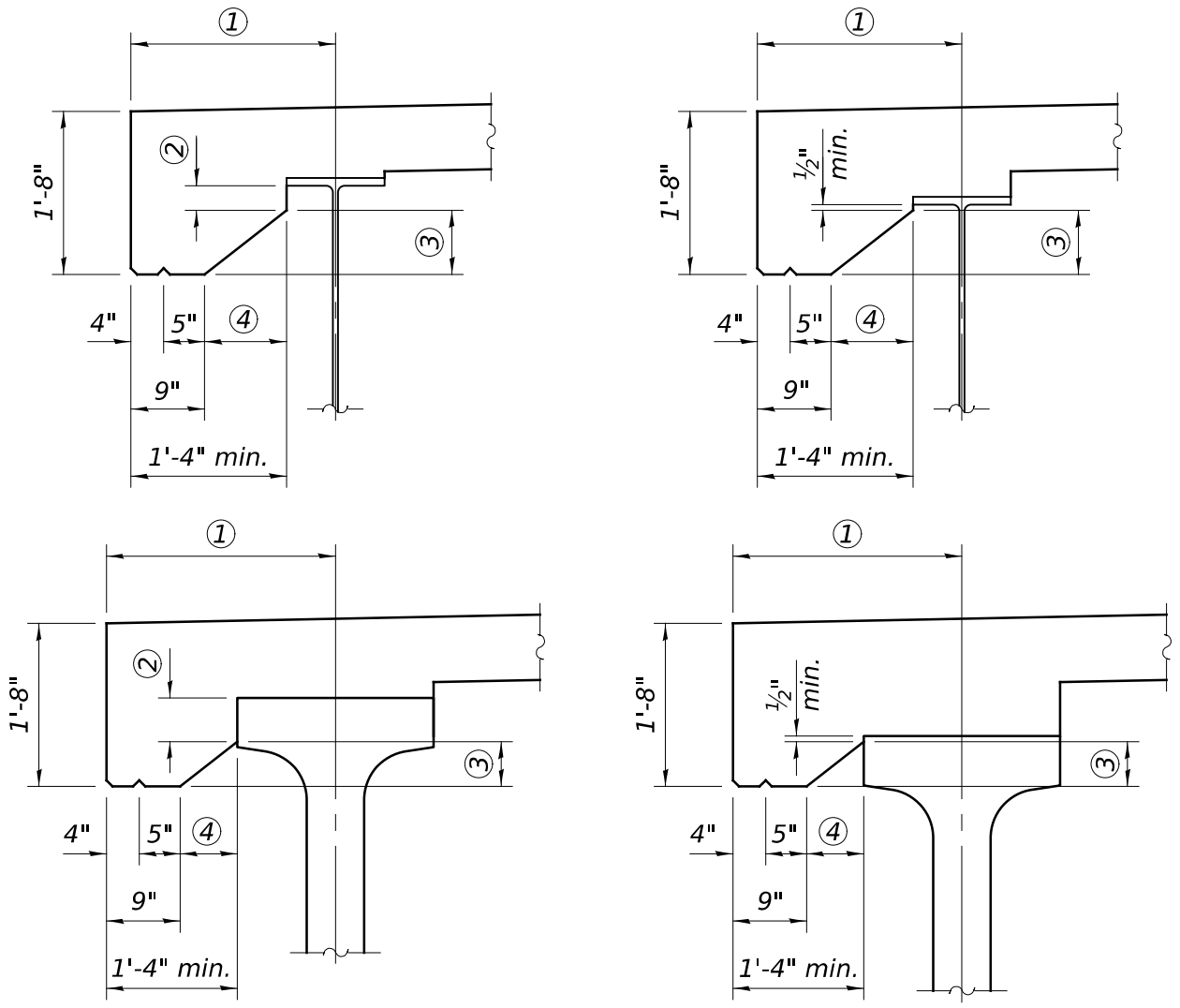
1. Parapets mounted on top of retaining walls or on top of anchorage slabs are acceptable but provisions in the contract documents for an extended portion of the wall or slab shall be provided such that the supporting element extends 1.5 inches beyond the back face of the parapet. Details similar to the SFP base sheet are required.
2. Projects approved for the slipforming option shall include Base Sheet SFP 39-44 and Special Provision GBSP 61 (Slipform Parapet) in the contract documents.

Projects not approved for slipforming shall include the General Note for slip forming from Section 3.1.3. This note may be modified to accommodate projects where the project on one side of the bridge passes the screening, and the other side does not.

3.2.4.3 Steel Railings on Bridge Decks

Structures with side mounted steel railings require a thickened overhang to anchor the railing into. Figures 3.2.4.3-1 through 3.2.4.3-3 show the overhang configuration and additional deck reinforcement required to provide adequate strength for the railing anchorage.

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① *The maximum overhang is 2'-1" if the intent is to have the interior beam design control over the exterior beam design. There will be cases, especially with the prestressed beams, where overhangs greater than 2'-1" will be required.*

② *Maximum fillet minus minimum fillet plus ½". The maximum fillet is determined based on dead load deflection, camber and grade.*

③ *Constant determined from maximum value calculated in note 2.*

④ *Constant determined from overhang width.*

SIDE MOUNT RAILING DECK OVERHANG CONFIGURATION

Figure 3.2.4.3-1

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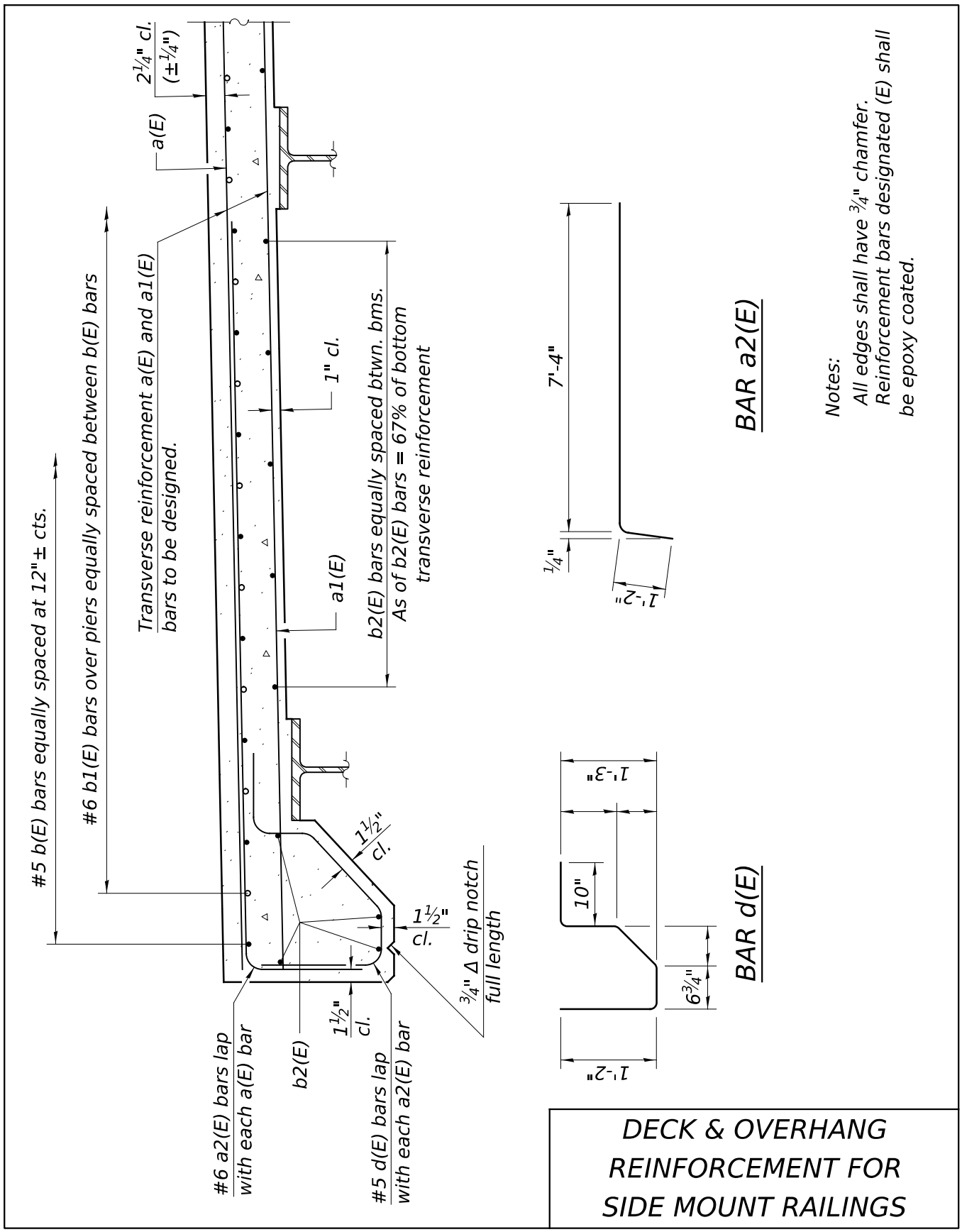
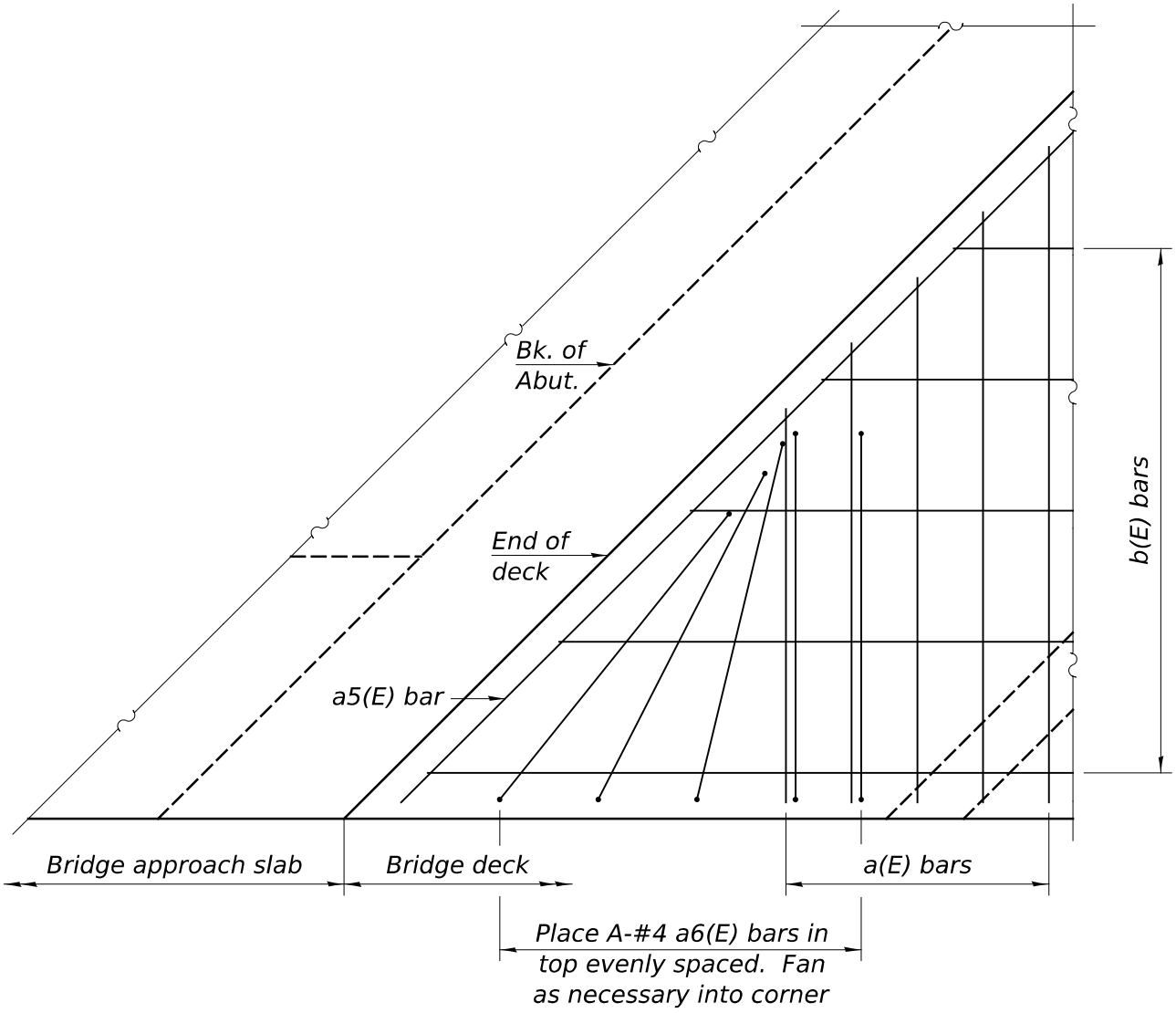


Figure 3.2.4.3-2

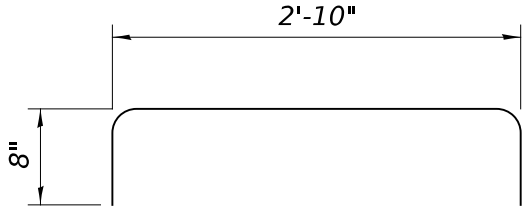
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PLAN

(Showing top reinforcement only)

Skew	A
<15°	0
15°-20°	1
20°-25°	2
25°-35°	4
35°-45°	5



BAR a6(E)

ACUTE CORNER DECK REINFORCEMENT FOR SIDE MOUNTED RAILINGS ON INTEGRAL ABUTMENT BRIDGE DECKS

Figure 3.2.4.3-3

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3.2.4.3.1 Type IL-OH Steel Railing

For deck beam structures with Type IL-OH (R40 series) railing the anchorage shall always be in the deck beam regardless of the wearing surface. Details for this anchorage can be found on the prestressed deck beam base sheets.

The first and last IL-OH railing post mounted on the bridge deck shall always be 3'-0" from the end of the bridge deck or deck beam. The transition to the Traffic Barrier Terminal (TBT), Type 13 or approach slab railing shall be as shown in Figures 3.2.4.3.1-1 through 3.2.4.3.1-3 depending on abutment type and if a bridge approach slab is required.

3.2.4.3.2 Type SM & SMX Steel Railing

For deck beam structures with Type SM or SMX railing the anchorage is allowed in the concrete wearing surface. Details for this anchorage can be found on the prestressed deck beam base sheets. The 4,000 psi concrete deck Figure 3.2.1-1 is permitted for use with the higher f'c, of the concrete elements specified for Type SMX railing in Section 2.3.6.1.7.

The transition between Type SM (R34 series) and Type SMX (R41 series) railing and the Traffic Barrier Terminal, Type 6A or approach railing shall be as shown in Figures 3.2.4.3.2-1 through 3.2.4.3.2-3 depending on if a bridge approach slab is required.

3.2.4.3.3 Type CO-10 Steel Railing

Standard details for the Type CO-10 can be found on the R-42 Railing Base Sheet. The transition to the Traffic Barrier Terminal, Type 13 is according to the applicable Highway Standard and requires no additional modification.

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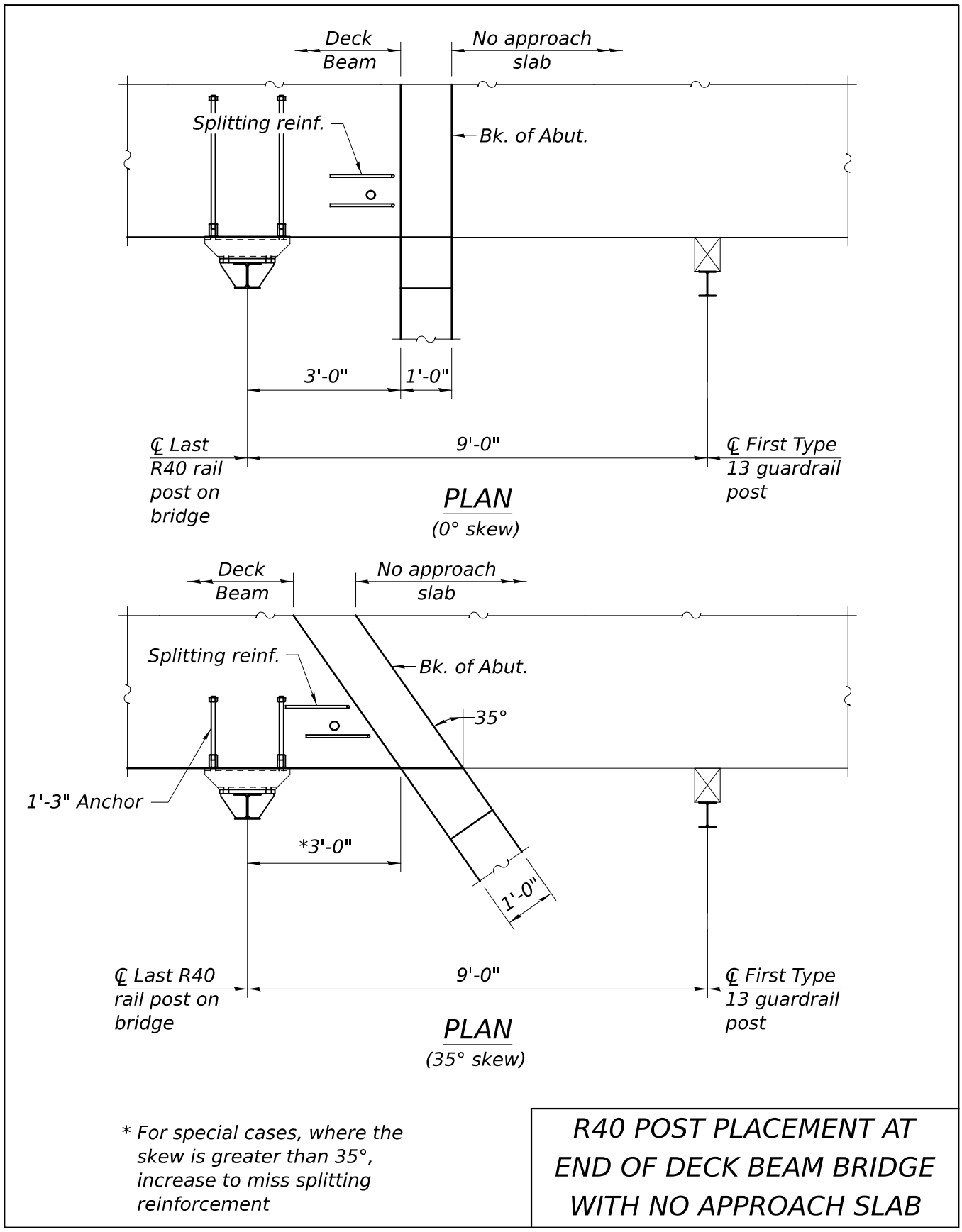
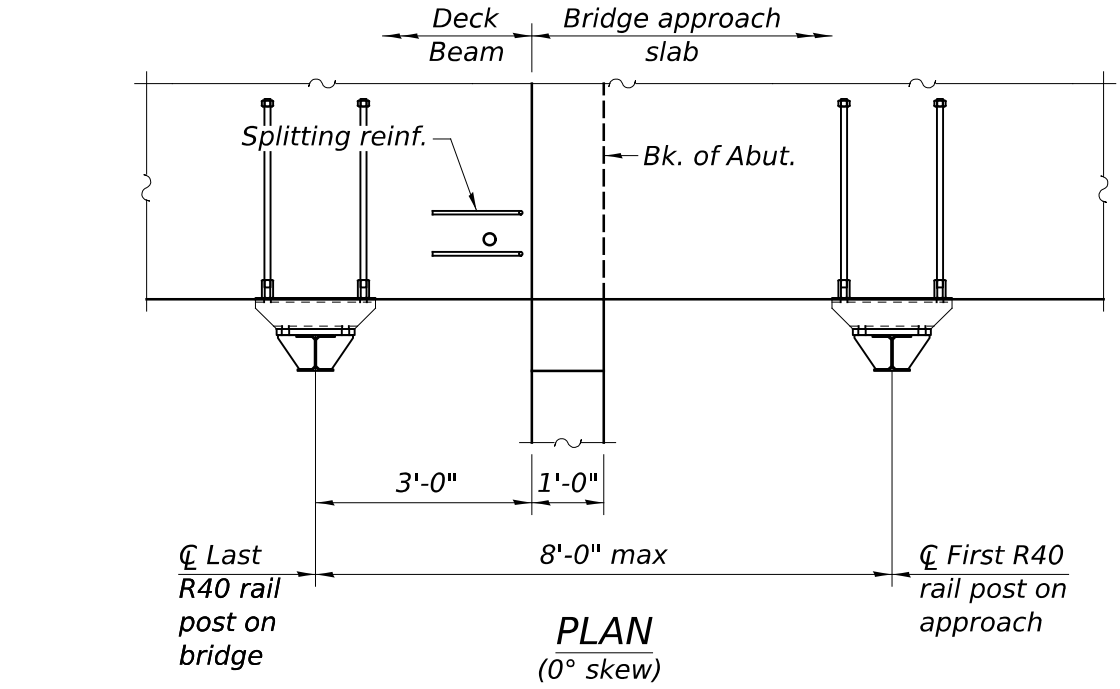
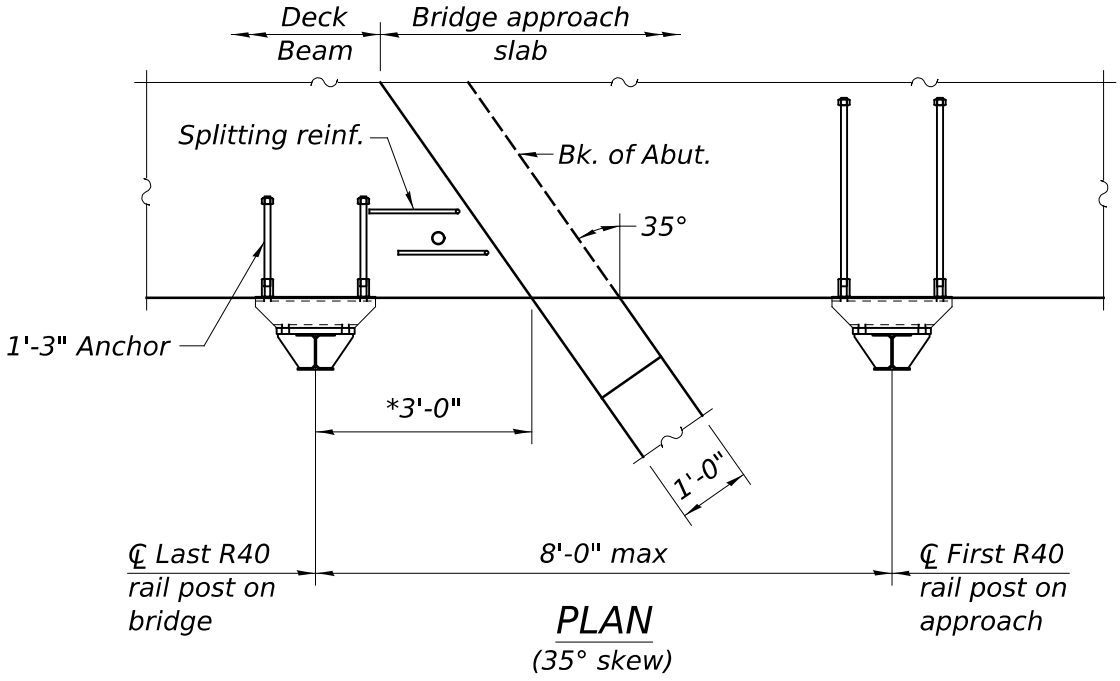


Figure 3.2.4.3.1-1

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* For special cases, where the skew is greater than 35°, increase to miss splitting reinforcement

R40 POST PLACEMENT AT END OF DECK BEAM BRIDGE WITH APPROACH SLAB

Figure 3.2.4.3.1-2

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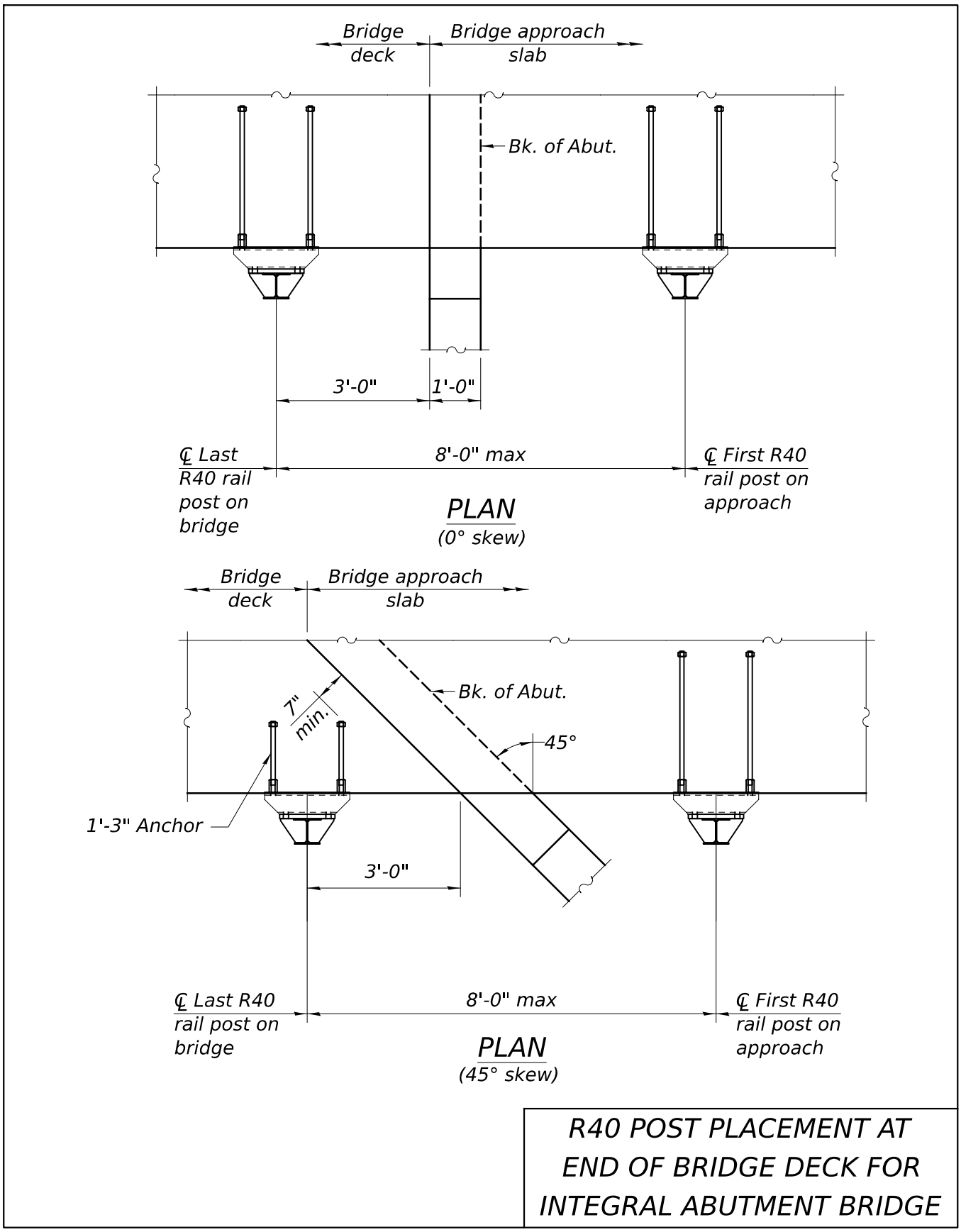
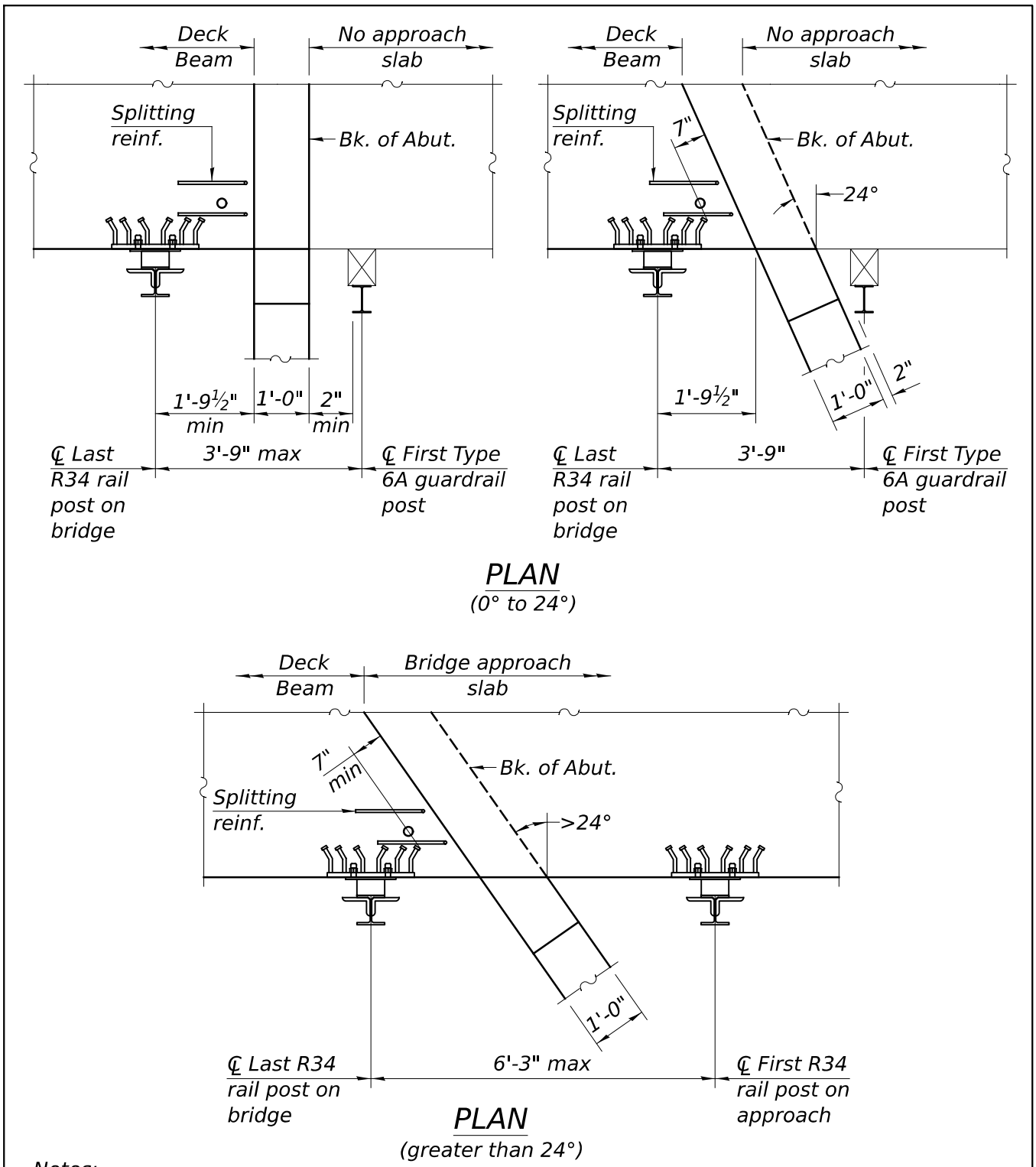


Figure 3.2.4.3.1-3

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Notes:

The R34 railing may be used without an approach slab for skews up to 24 degrees. For skews larger than 24 degrees an approach slab is required and the R34 railing shall continue onto the approach slab. The above geometry applies to integral abutment bridges however an approach slab is always required.

R34 RAIL TREATMENT AT ABUTMENT

Figure 3.2.4.3.2-1

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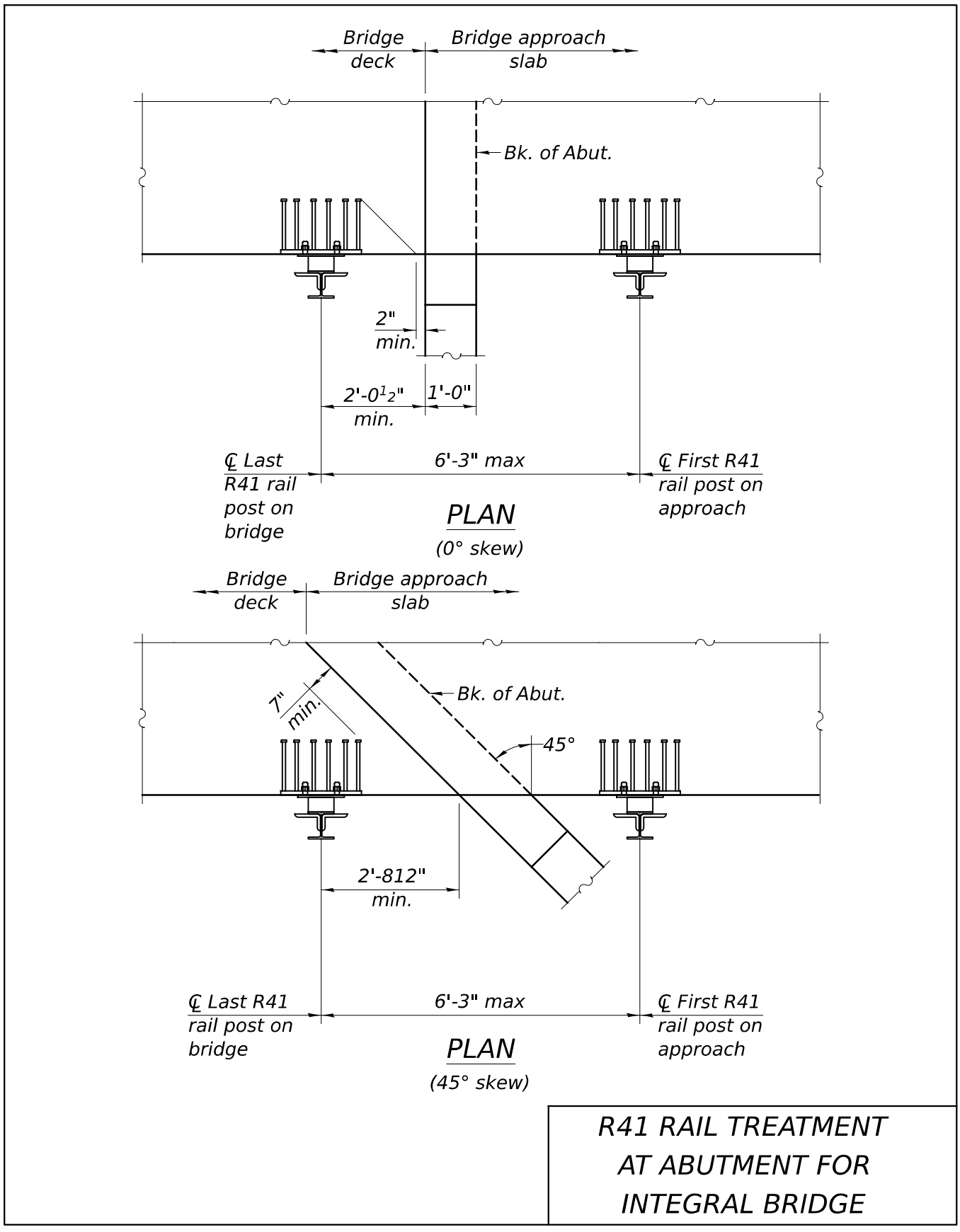
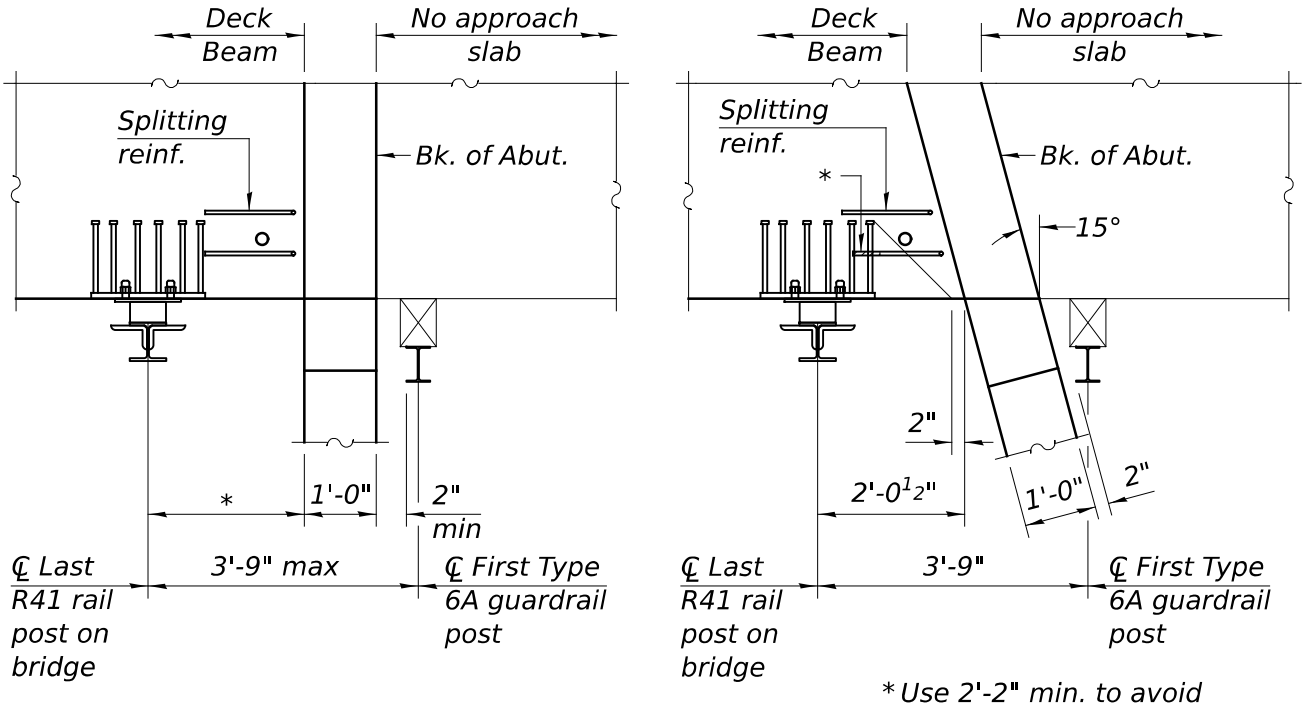


Figure 3.2.4.3.2-2

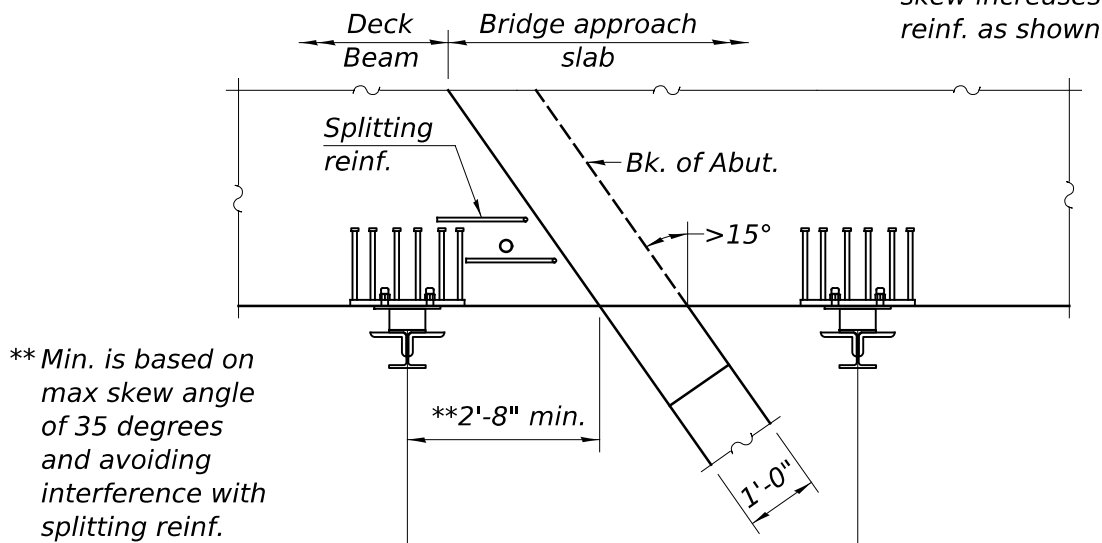
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PLAN

(0° to 15°)

* Use 2'-2" min. to avoid interference with splitting reinf. However as the skew increases cut splitting reinf. as shown.



PLAN

(greater than 15°)

Notes:

The R41 railing may be used without an approach slab for skews up to 15 degrees. For skews larger than 15 degrees an approach slab is required and the R41 railing shall continue onto the approach slab.

R41 RAIL TREATMENT AT ABUTMENT FOR DECK BEAM BRIDGE

Figure 3.2.4.3.2-3

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3.2.4.4 Aesthetic Concrete Bridge Railing

To meet the TL-4 crashworthy requirements for older aesthetic concrete railings, the Department has developed the Concrete Bridge Railing (R-38 series) which is modeled from the Texas C412 railing. It may be used for bridge decks, slab bridges and sidewalk applications. Since the majority of new Illinois structures are integral abutment design, standard details extend this Concrete Bridge Railing 15 feet onto a bridge approach slab with a Type 6 railing connection. Similarly, the Department also developed a superstructure detail sheet, Texas Rail TL-4, to complement the new TL-4 Concrete Bridge Railing for integral structure applications. These base sheets may be modified and used for other structure type applications, provided the typical railing geometry and reinforcement are not modified.

The following design limitations shall be adhered to when utilizing this railing. These limitations are also illustrated in Figure 3.2.4.4-1. The connection of the railing to the superstructure shall be #5 d1(E) bars at 11" cts. The typical panel length shall be a minimum of 10 feet and a maximum of 33 feet. The minimum end post length of a typical panel or a panel adjacent to a pier shall be 1 '-0" and the maximum spacing of the #5 D(E) bars in these end posts shall be 6". The panel length adjacent to a pier shall be determined similar to Figures 3.2.4-8 and 3.2.4-9 except that the minimum panel length shall be 10 feet. The average weight of the railing is 560 pounds per lineal foot.

Base sheets for the railing and detail sheets above are provided in the "Railings" and "Superstructure-Integral Steel Beams" IDOT CADD Bridge Cell Library.

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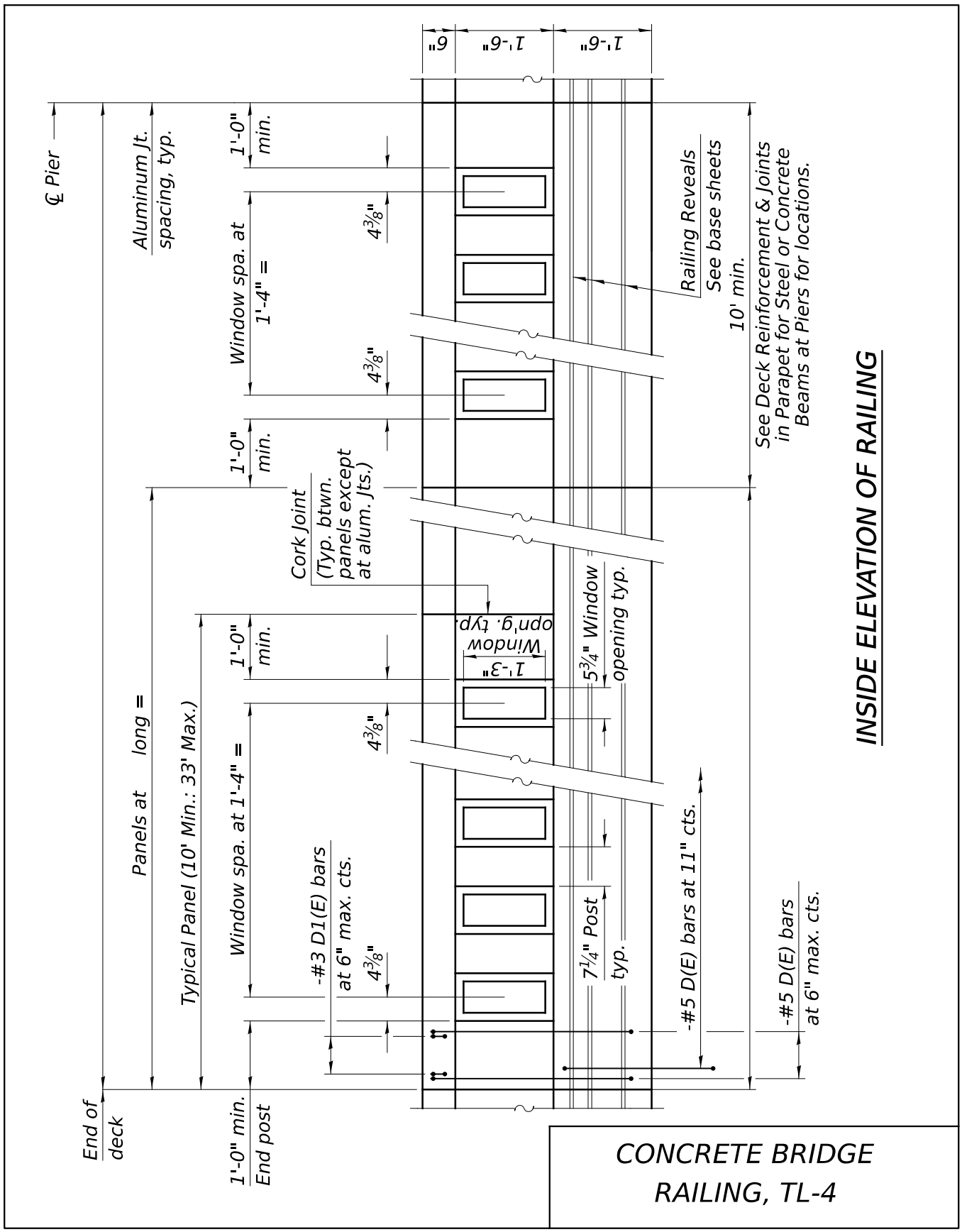


Figure 3.2.4.4-1

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3.2.4.5 Aesthetic Steel Bridge Railing

As discussed in Section 2.3.6.1.7, railings on all new or rehabilitated bridges on Federal and State routes shall satisfy the crash standards of the 2016 Manual for Assessing Safety Hardware (MASH) with a minimum crashworthy Test Level of TL-4.

In specific deck section arrangements, aesthetic treatments to steel railings mounted to the deck (or mounted upon the Department's standard parapets) are sometimes proposed to the Department. One of the primary concerns in MASH testing is related to potential for vehicle snagging and causing excessive damage of the occupant compartment.

Therefore, proposed aesthetic treatments to Department's crash-tested rails may be considered when

1. The rail is set back the same amount from the impact-side top-edge of the single slope concrete barrier as it was in the crash test, and
2. Proposed aesthetic windows in the rails are set within the width of the rails and do not protrude out.

The Designer shall contact the Department prior to incorporating new aesthetic treatments in the final design.

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3.2.5 Raised Curb Medians

Figures 3.2.5-1, 3.2.5-2, and 3.2.5-3 detail two types of raised-curb medians: Superimposed (with or without separation joint), and Voided. Either type may be split with a 1 in. separation joint depending on the overall bridge width. See Section 3.2.7. The type to be used on a particular deck should best balance ease of construction, and economical use of concrete and reinforcement for the required median width and stringer locations. Note that Figures 3.2.5-1 and 3.2.5-2 also provide transverse joint details for superimposed medians.

Raised-curb medians on bridges primarily use barrier curbs as shown in Figures 3.2.5-1, 3.2.5-2, and 3.2.5-3. See Chapter 34 and 39 of the Bureau of Design & Environment Manual available at the primary Highways Page, for additional guidelines on raised-curb medians for bridge decks including those with barrier or mountable curbs.

In general terminology, “Median” is that portion of the deck between the inside edges of the traffic lanes, and the raised portion is this center distance minus the gutter flag that is used in the approach cross-section.

Raised-curb medians on the bridge deck are formed with a radius at the top to match the roadway curb and gutter section. The top of the median surface shall be sloped for drainage.

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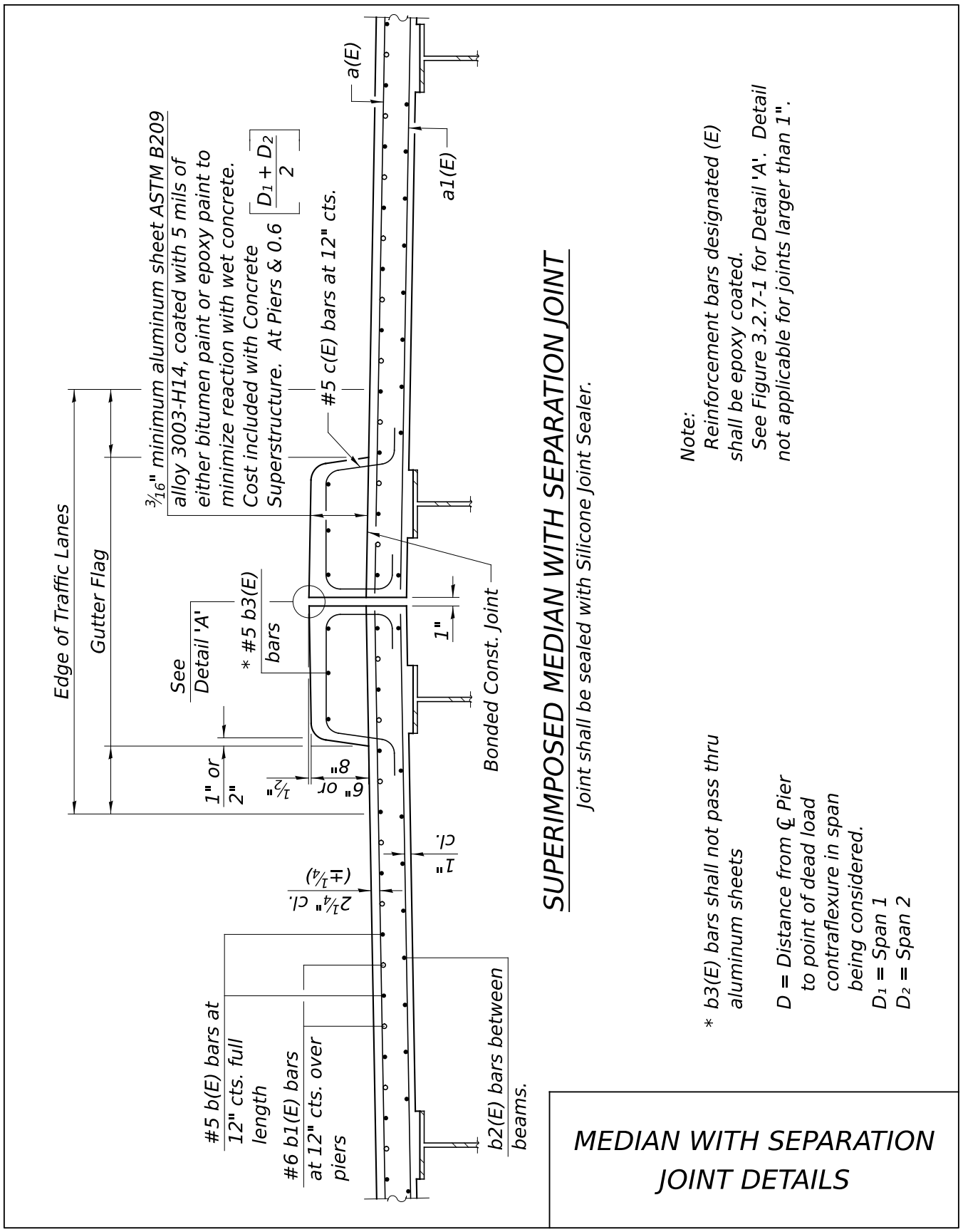


Figure 3.2.5-1

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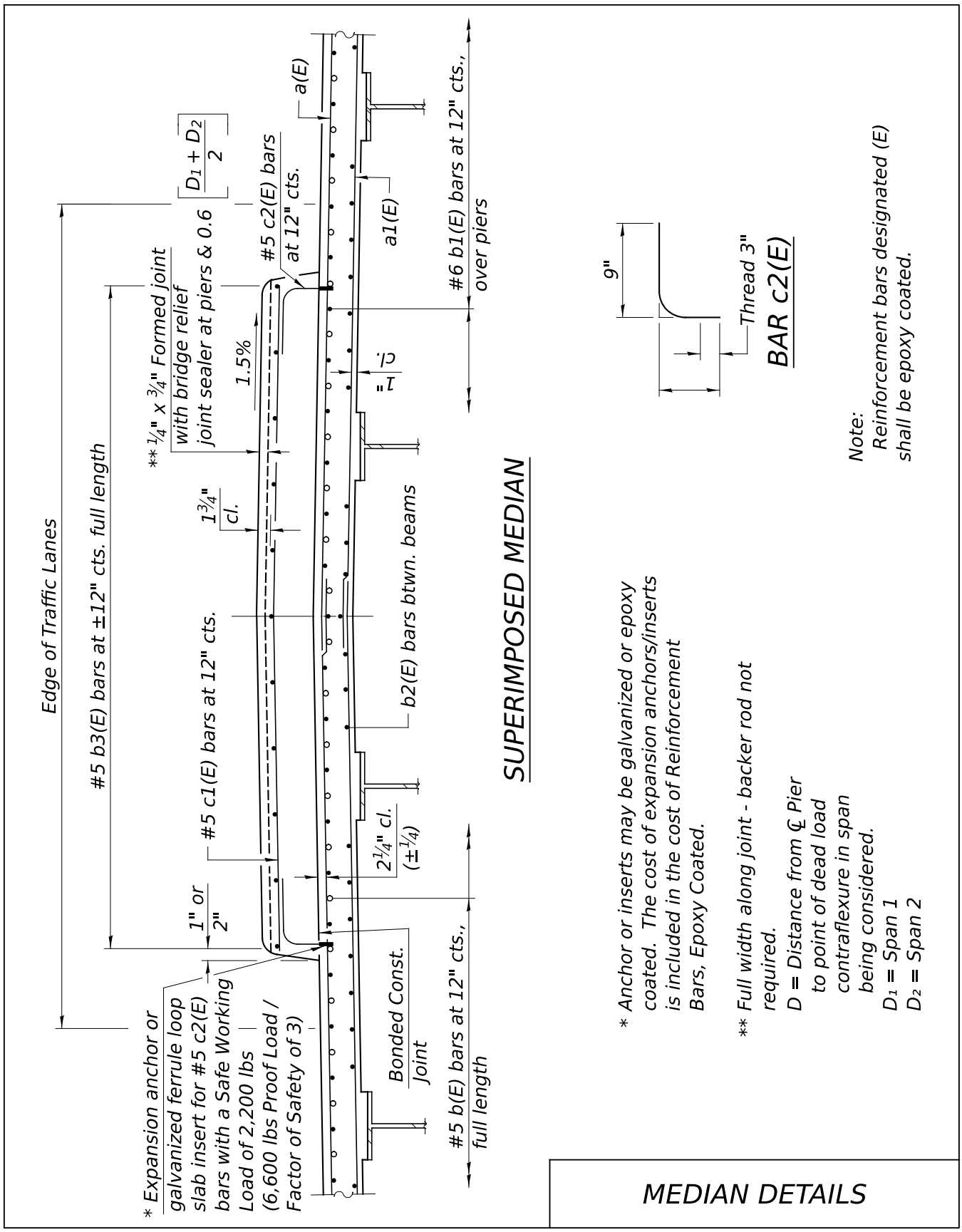


Figure 3.2.5-2

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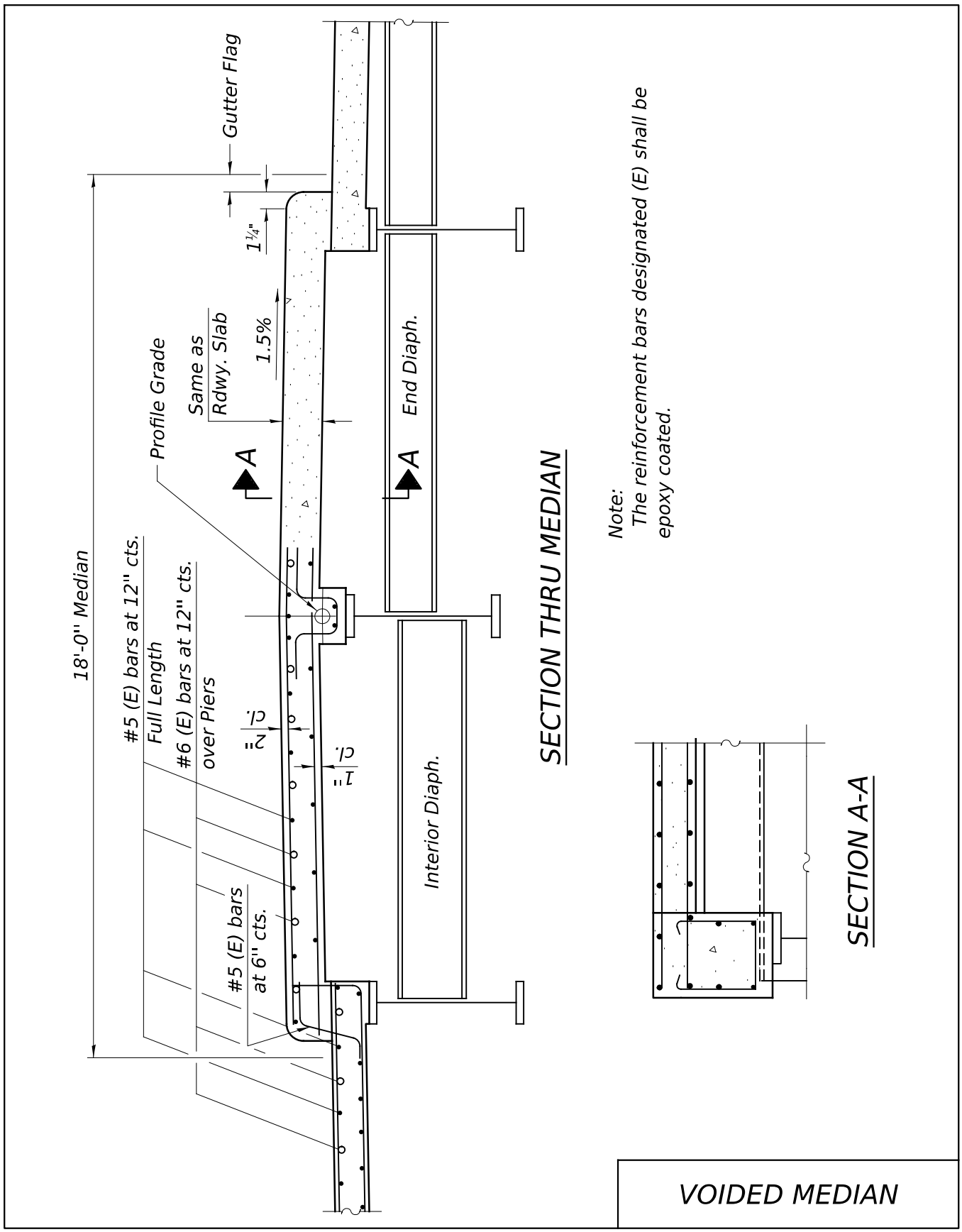


Figure 3.2.5-3

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3.2.6 Longitudinal Bonded Joints (Concrete Deck on Steel Stringers)

The Illinois Department of Transportation allows the use of longitudinal bonded joints in the roadway slab in the middle half of a non-exterior stringer bay of wide decks when construction is not staged (See Figure 3.2.7-1) and, when possible, the longitudinal bonded joint shall line up with the outside edge of the traffic lanes. On wide decks it may be permissible to place a longitudinal bonded joint at the edge of an intermediate traffic lane. No bonded joint shall cross a beam line. If a situation is met which does not appear to permit the use of a longitudinal bonded joint within these limitations, the Engineer of Bridges & Structures should be notified in order that the matter can be resolved.

Longitudinal bonded stage construction joints are used for bridge projects which are staged. Bar splicers shall be required at stage construction joints. See Section 2.3.8 for more information on staged construction. Bar splicers are not required for optional longitudinal bonded construction joints.

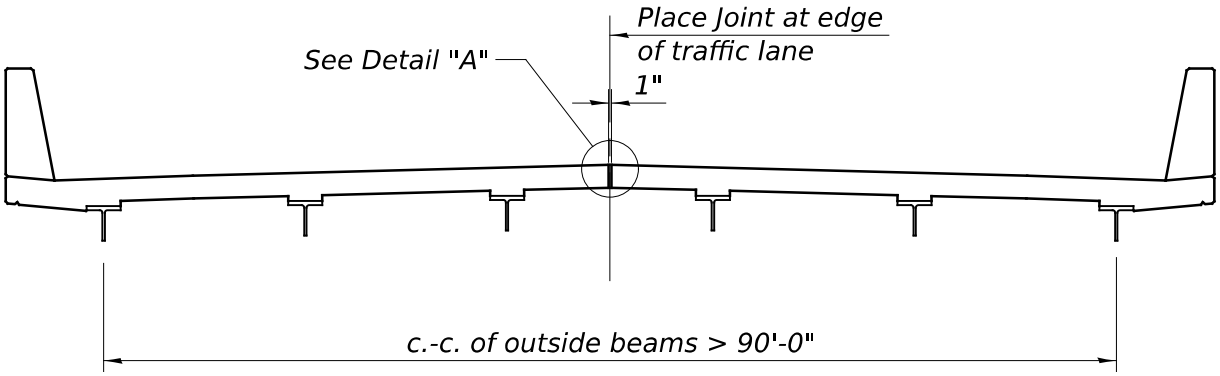
Special consideration should be given to placement of the longitudinal slab steel in relation to the bonded joint on a horizontally curved structure. In extreme cases, it may be necessary to lay out the reinforcement in plan to assure proper placement of the distribution steel.

Note that no longitudinal bonded joint is shown adjacent to the voided median in Figure 3.2.5-3. If the Regional Engineer requests variation from this standard to meet certain construction procedures, the Engineer of Bridges & Structures shall be notified, and the variation will be incorporated into the plans.

3.2.7 Longitudinal Open Joints

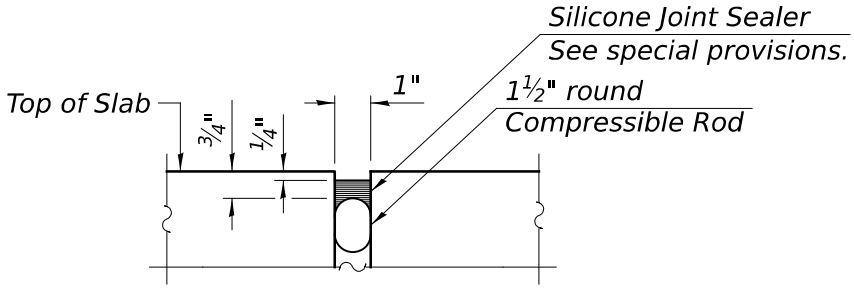
When the distance between the fascia beams is greater than 90 ft. – 0 in., the deck shall be split by means of a one inch (1 in.) open joint. This joint shall be sealed with a silicone sealer and rubber rod designated as “Silicone Joint Sealer” (see Figure 3.2.7-1) on the plans when located in a raised median or in a deck that is not waterproofed and surfaced. The one inch (1 in.) open joint is not required if the deck is stage constructed and the total width of the staged pours is less than or equal to 120 ft. Figure 3.2.7-2 provides details for a longitudinal open joint at median barriers.

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LONGITUDINAL SEPARATION JOINT

To be provided when c.-c. of outside beams is greater than 90 ft and no stage construction. Joint shall be sealed with Silicone Joint Sealer. Joints should be located within the middle half of an interior bay. See Art. 503.16 of the Standard Specifications for additional information.

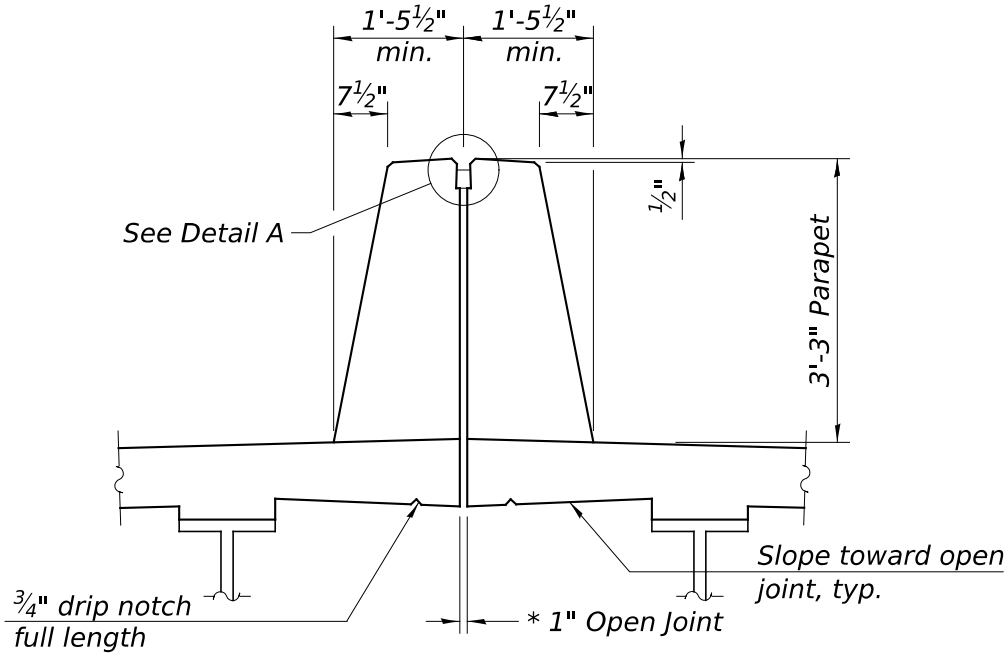


DETAIL "A"

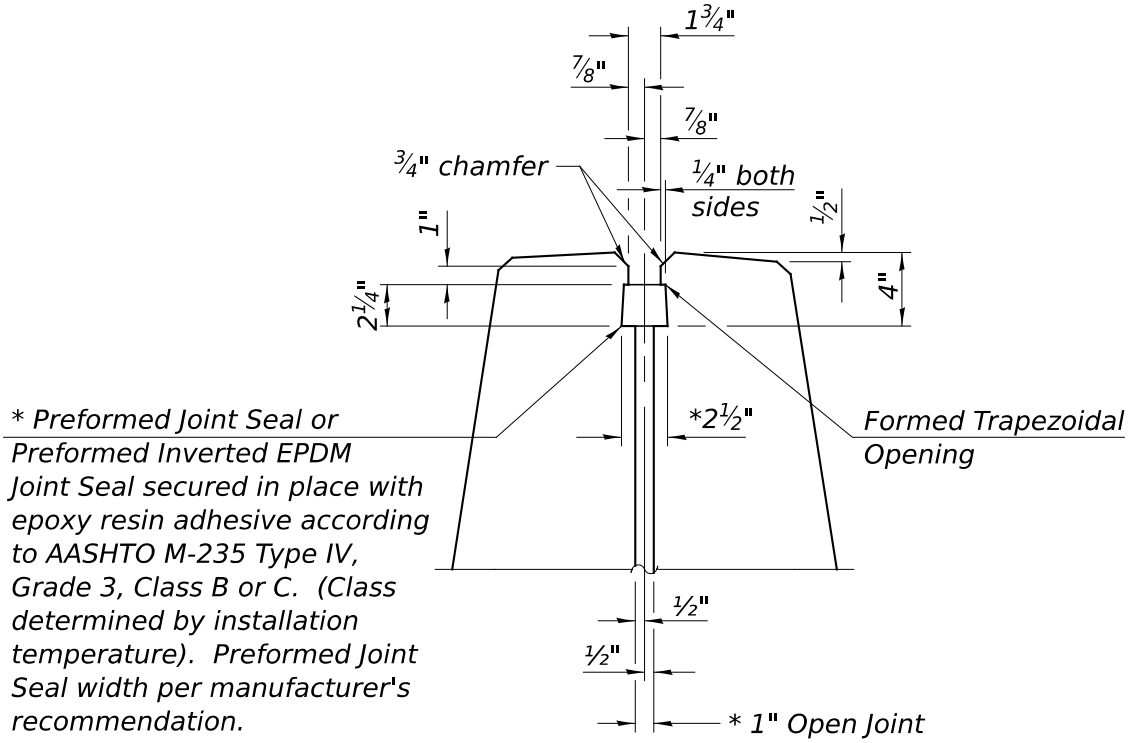
LONGITUDINAL JOINTS
FOR WIDE DECKS

Figure 3.2.7-1

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SECTION AT MEDIAN BARRIER



DETAIL A

* Typical dimension shown (when 2" Open Joint is approved by Planner, increase each dimension by 1").

LONGITUDINAL JOINT SEAL AT MEDIAN BARRIERS

Figure 3.2.7-2

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3.2.8 Transverse Construction Joints

If a bridge deck pour is greater than 400 cu. yds., an "optional transverse construction joint" shall be labeled, and a deck pouring sequence shall be shown on the plans. The location of this optional transverse construction joint shall be near the point of dead load contraflexure (calculated using only DC1 and DC2 loads), with the day's pour terminating at the end of a positive moment area. This requirement applies to superstructures with primary beams which are steel or prestressed concrete. Figure 3.2.8-1 provides an aid for determining when a deck pouring sequence is required. If a deck pouring sequence is required, General Note #15 shall be used.

For continuous steel superstructures with a span or spans exceeding 150 ft., the positive and negative moment areas shall be poured separately, with positive moment areas being poured first. The pouring sequence and location of transverse construction joints shall be determined on an individual basis and shall be shown on the plans.

For PPC I-beam, Bulb T-beam, and IL-beam superstructures, transverse construction joints should be avoided whenever possible. If necessary, a transverse joint should be located with the day's final pour terminating approximately 4 ft. prior to reaching the centerline of a pier.

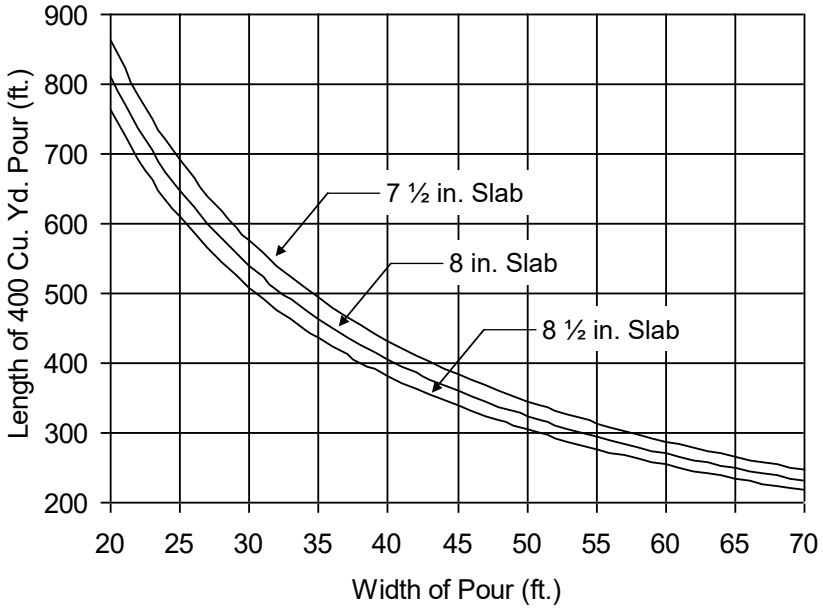


Figure 3.2.8-1

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3.2.9 Deck Floor Drains and Drainage Scuppers

The required minimum deck overhangs for the placement of floor drains or scuppers with respect to fascia-beam-to-parapet-front-face clearance are presented in Table 3.2.9-1. For fascia beams with wide top flanges (steel or prestressed), see hierarchy case 3 within Section 2.3.6.1.8 for alternate layout strategies. Floor drains may be angled or kinked if required, to clear the beam or girder flange as shown in Figure 3.2.9-2. The 4 in. x 12 in. aluminum floor drains shown in Figure 3.2.9-4 may be considered when the 6 in. DIA drains cannot be used. In addition, 8 inch floor drains are allowed when preferred by the District.

The 6 in. DIA and 8 in. DIA floor drains shown in Figures 3.2.9-1, 3.2.9-2, and 3.2.9-3 shall be listed in the Bill of Materials as a pay item, i.e. Floor Drains – each.

If drainage scuppers are used in a bridge deck, they shall be detailed on the plans. Figures 3.2.9-5 to 3.2.9-10 illustrate the control dimensions and additional reinforcement in the slab for drainage scuppers DS-11, DS-12, DS-12M10, and DS-33. Base Sheets for these scuppers can be found online at the IDOT CADD Bridge Cell Library. When necessary, it is acceptable to partially extend/embed these scuppers into the curb of the concrete barrier. A maximum of 4 in. is allowed and shall be detailed on the plans so that removal of the grate will be permitted for cleaning. See hierarchy case 3 within Section 2.3.6.1.8 for alternate layout strategies.

When drainage scuppers are used, the DS-11, DS-12, or DS-12M10 should be used whenever possible. The DS-33 scupper should be used only when the DS-11, DS-12 or DS-12M10 scuppers do not fit (such as for wide flange Bulb-T beams) or are not appropriate for the site.

Provide this note on all plans where applicable:

Drains shall be located clear of all diaphragms.

The color of the floor drains shown in Figures 3.2.9-1 and 3.2.9-4 should match the color of the fascia beam or girder. For treatment of drains adjacent to steel girders, see the notes in Figures 3.2.9-1 and 3.2.9-4. For treatment of drains adjacent to concrete beams, see Figure 3.2.9-3. When weathering steel, metallized steel, or galvanized steel is used or when painting of a structure will be delayed to a separate contract, the note given on Base Sheets SDI-SB-1 and SDI-SB-2 which reads "*The exterior surfaces of the floor drains...*" shall be replaced with the following note:

Floor drains need not be painted.

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Minimum Fascia Beam Overhang				
Type / b_f (in)	6” floor drain	8” floor drain or DS-11 Scupper	DS-12 Scupper	DS-12M10 Scupper
10	2’-5”	2’-7”	3’-2”	3’-0”
12	2’-6”	2’-8”	3’-3”	3’-1”
14	2’-7”	2’-9”	3’-4”	3’-2”
16	2’-8”	2’-10”	3’-5”	3’-3”
18	2’-9”	2’-11”	3’-6”	3’-4”
20	2’-10”	3’-0”	3’-7”	3’-5”
22	2’-11”	3’-1”	3’-8”	3’-6”
24	3’-0”	3’-2”	3’-9”	3’-7”
38	3’-7”	3’-9”	*4’-4”	*4’-2”

Table Notes:

Assumptions:

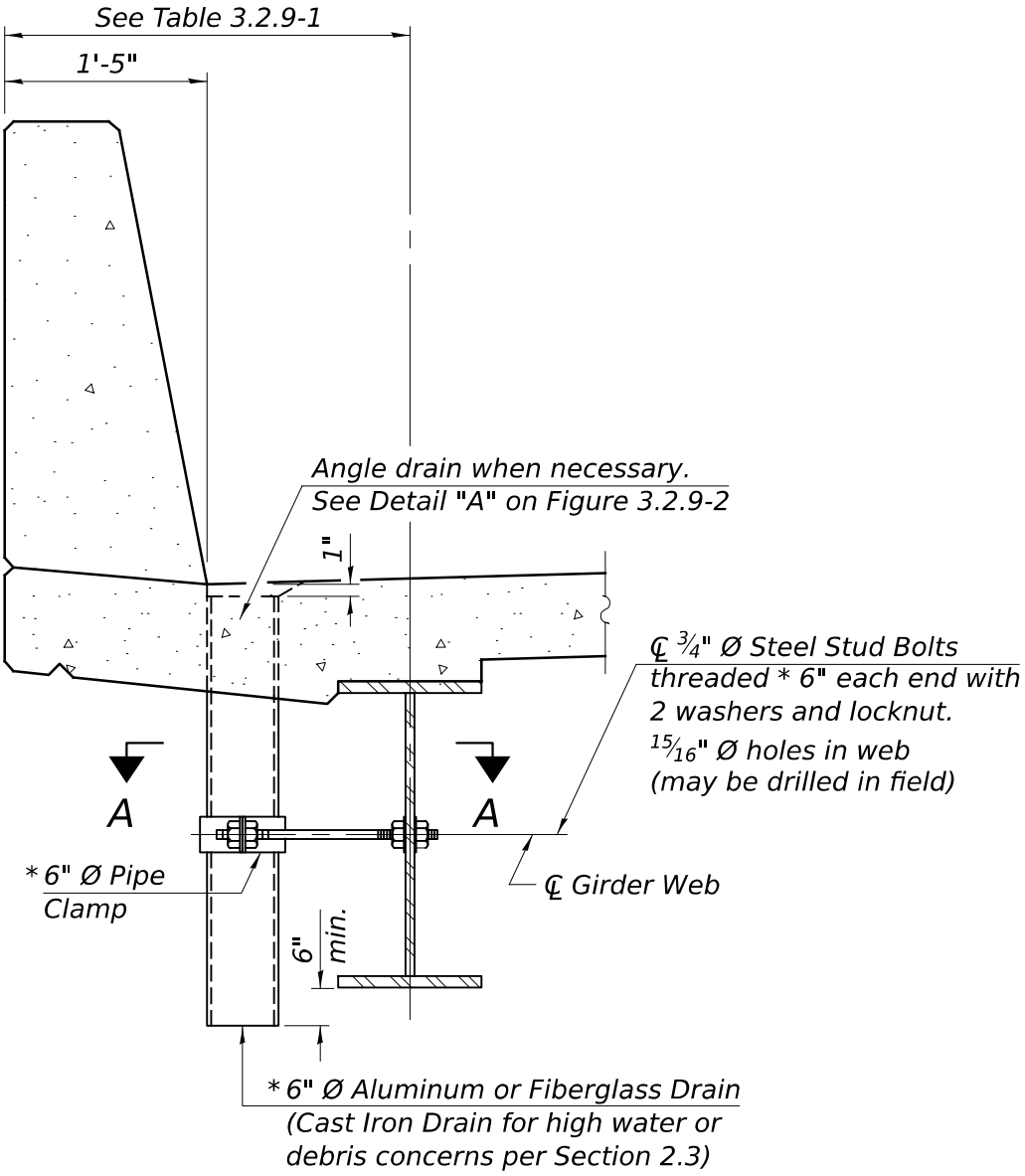
1. 39” Constant-slope concrete parapet configuration (1’-5” wide)
2. 1” plan clearance between drainage element and nearest beam flange
3. 8” reinforced concrete deck with ½“ fillet
4. Drainage element placed at curb line (no tuck) and outside of fascia beam

Shaded areas require overhangs > 3’-6” which cause the exterior beam to govern beam design. Tucking drainage feature up to 4” under parapet is permitted to maintain interior beam design but reduced hydraulic capacity shall be considered.

*Reversing scupper to drain inside of fascia beam may be considered.

Table 3.2.9-1

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SECTION AT PARAPET

* = 6" Drain shown. For 8" Drain, increase by 2".

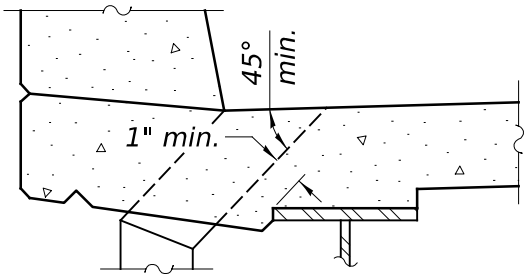
Note:

The exterior surfaces of the floor drains shall be painted with the finish coat as specified in Section 506 of the Standard Specifications. The exterior surfaces of the drains shall be cleaned according to Society of Protective Coating's Spec. SSPC-SP1 prior to painting. Fiberglass pipe shall conform to ASTM D 2996, with short-time rupture strength hoop tensile stress of 30,000 p.s.i. minimum. Galvanize clamping device according to AASHTO M232. Cost of clamping device and galvanizing included with Floor Drains.

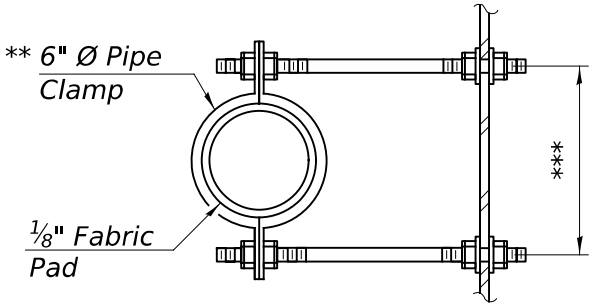
FLOOR DRAIN
FOR STEEL BEAMS

Figure 3.2.9-1

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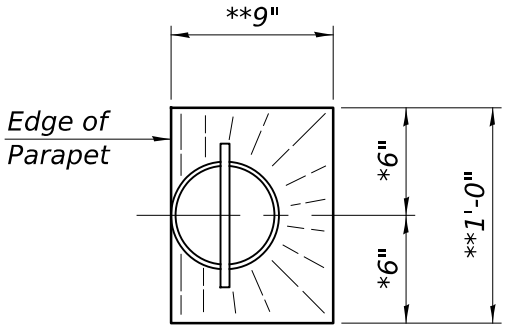


DETAIL "A"

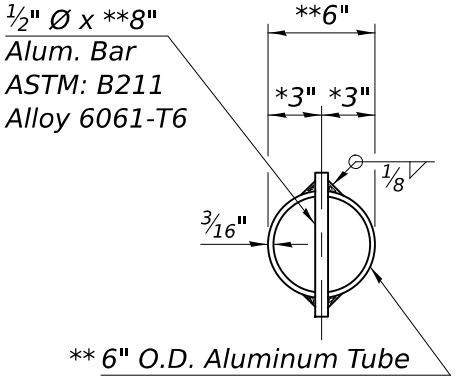


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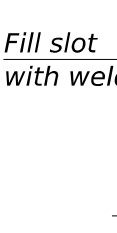
*** Dimension as required by Pipe Clamp.



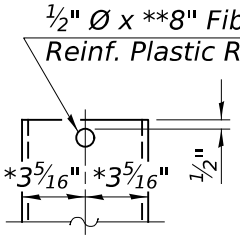
TOP PLAN



TOP PLAN
(Showing Aluminum Tube)



ALUMINUM TUBE



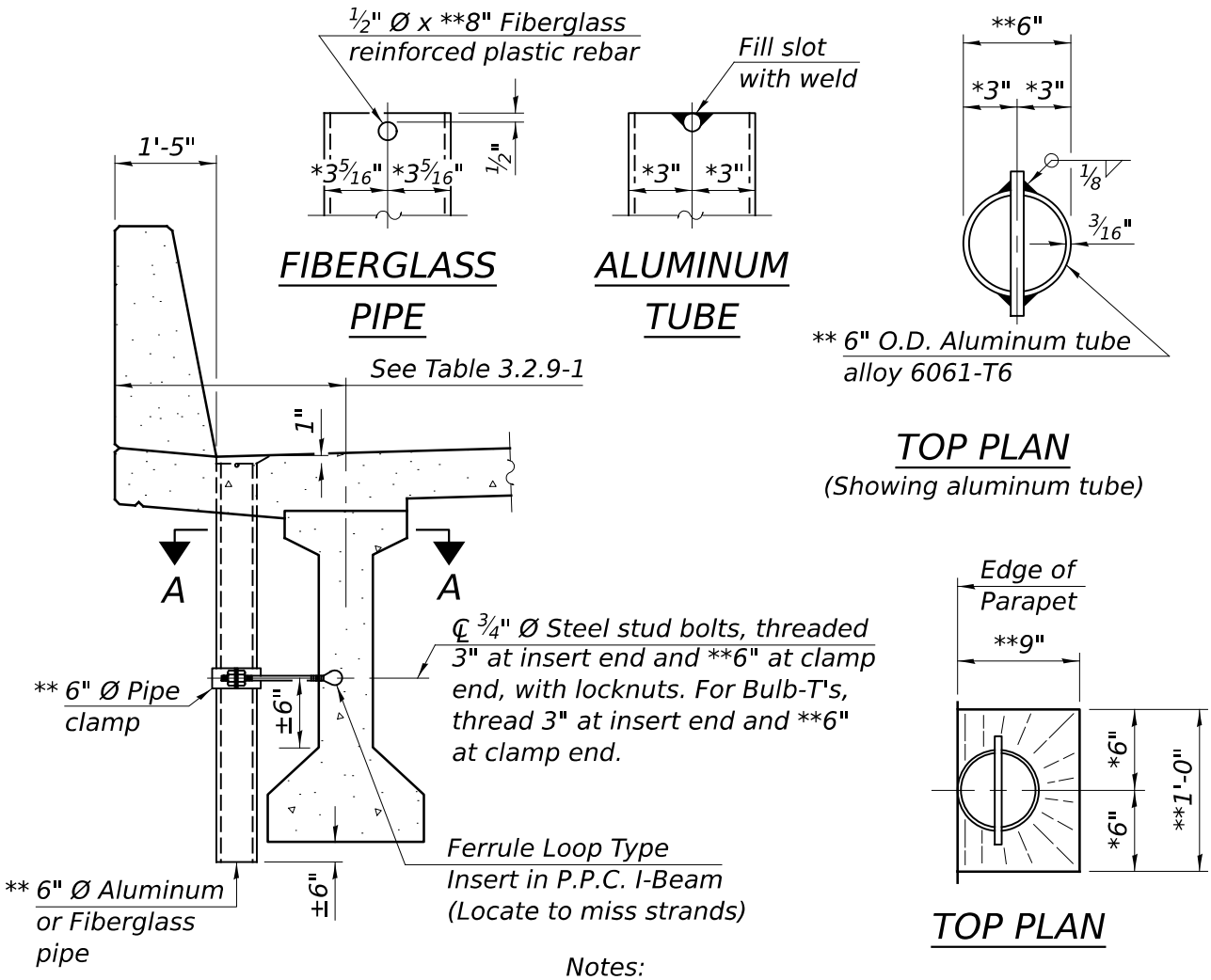
FIBERGLASS PIPE

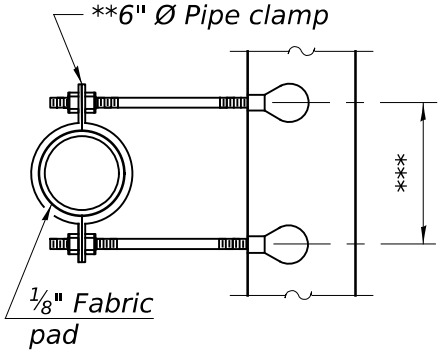
Notes:
6" Drain shown. For 8" Drain:
* = Increase by 1"
** = Increase by 2"

FLOOR DRAIN FOR STEEL BEAMS

Figure 3.2.9-2

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*** Dimension as required by pipe clamp

Notes:

Fiberglass pipe shall conform to ASTM D2996, with short-time rupture strength hoop tensile stress of 30,000 p.s.i. minimum.

The exterior surfaces of the floor drains shall be coated or pigmented by the manufacturer with a color that matches the concrete.

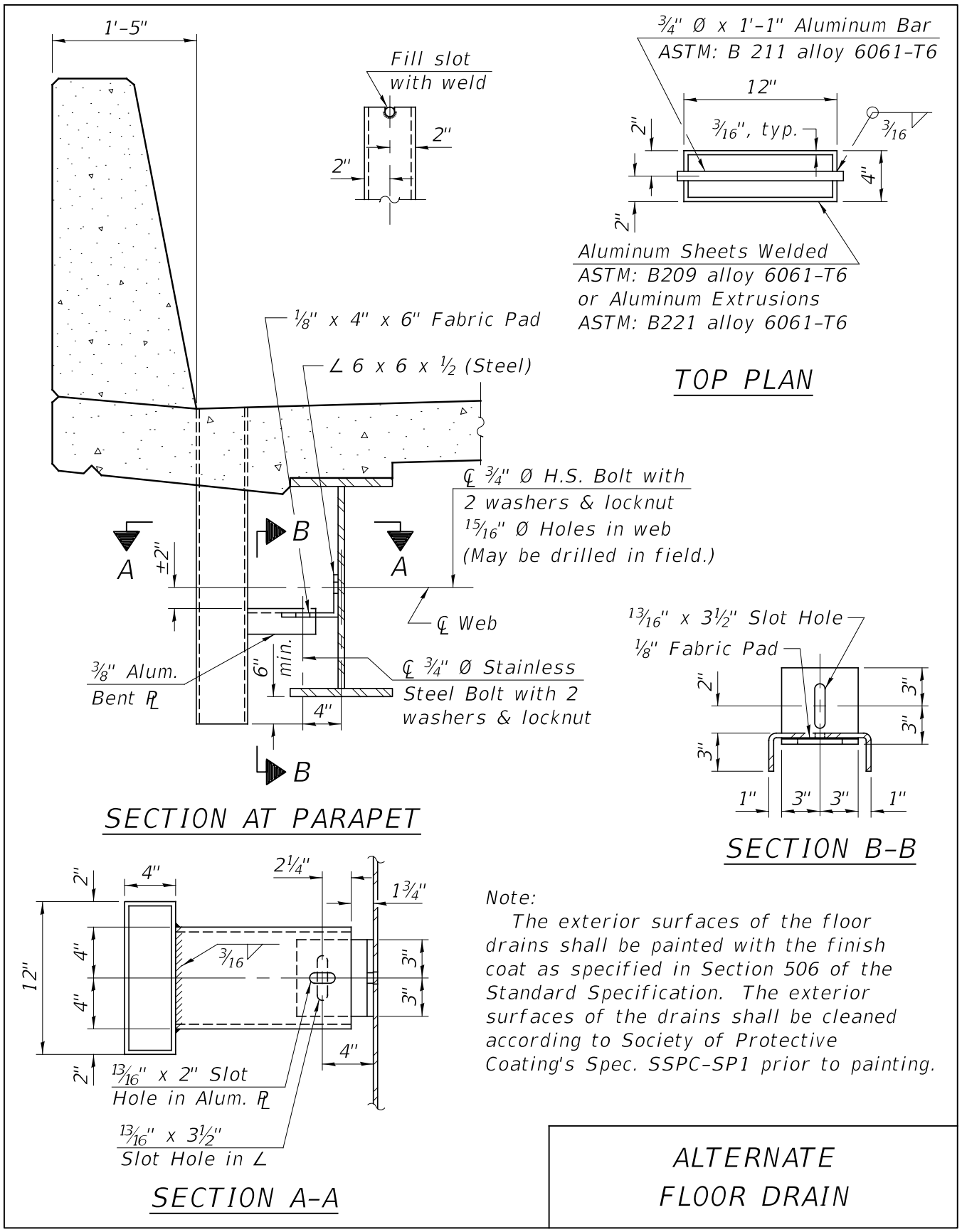
The clamping device and inserts shall be galvanized according to AASHTO M-232. Cost of clamping device and galvanizing included with Floor Drains.

6" Drain shown. For 8" Drain:
** = Increase by 1"*
*** = Increase by 2"*

FLOOR DRAIN FOR CONCRETE BEAMS

Figure 3.2.9-3

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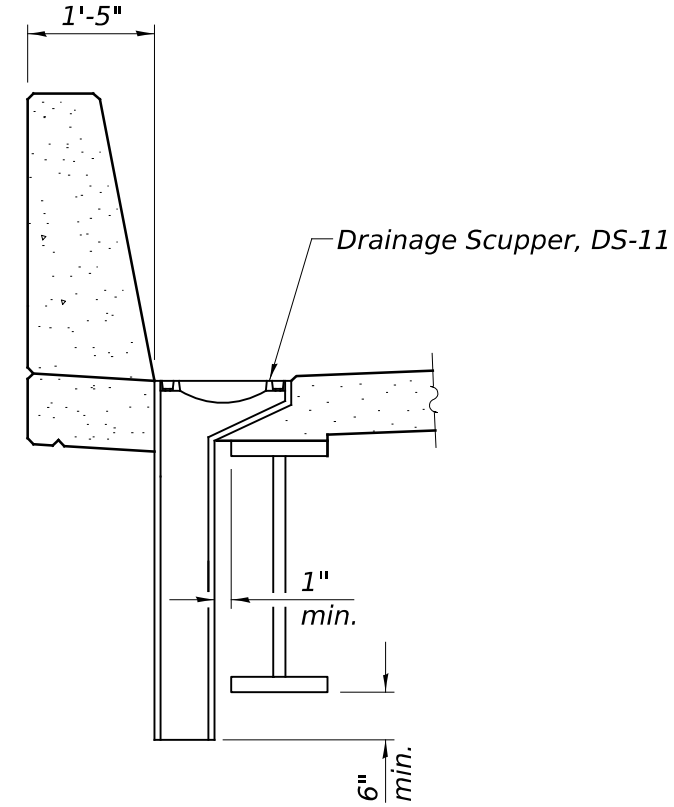


Note:
The exterior surfaces of the floor drains shall be painted with the finish coat as specified in Section 506 of the Standard Specification. The exterior surfaces of the drains shall be cleaned according to Society of Protective Coating's Spec. SSPC-SP1 prior to painting.

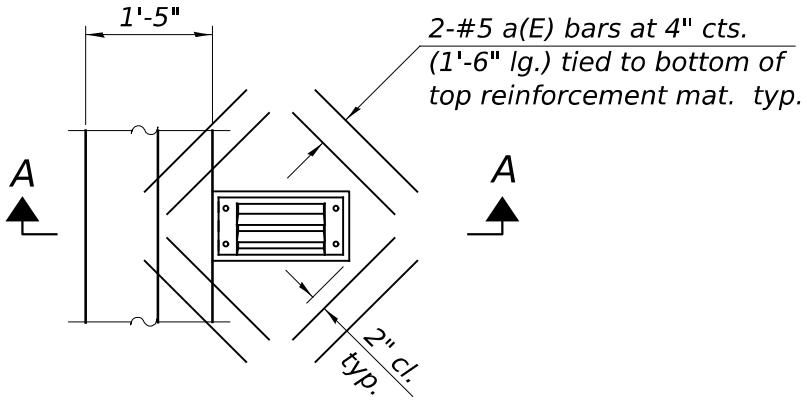
ALTERNATE FLOOR DRAIN

Figure 3.2.9-4

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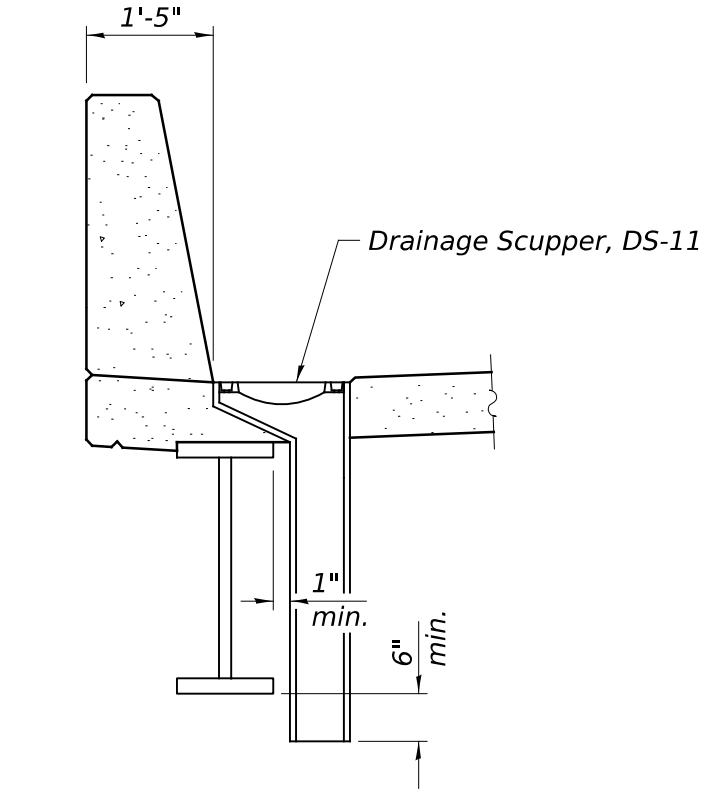
PLAN

Note:
Reinforcement bars designated (E) shall be epoxy coated.
Cut longitudinal reinforcement to clear drainage scuppers.

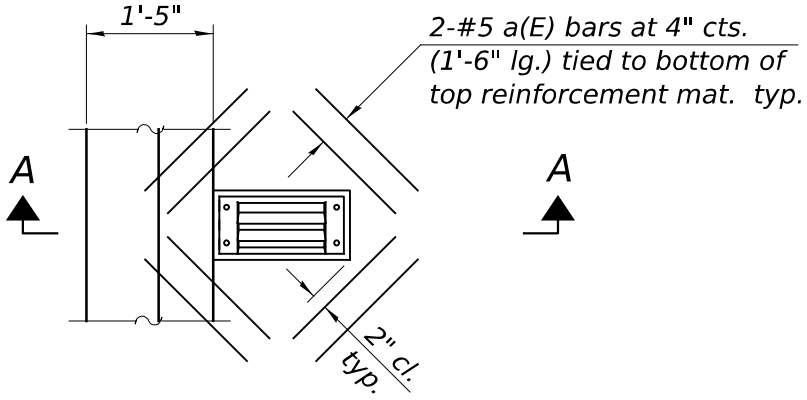
DRAINAGE SCUPPER
DS-11 OUTSIDE FASCIA BEAM

Figure 3.2.9-5

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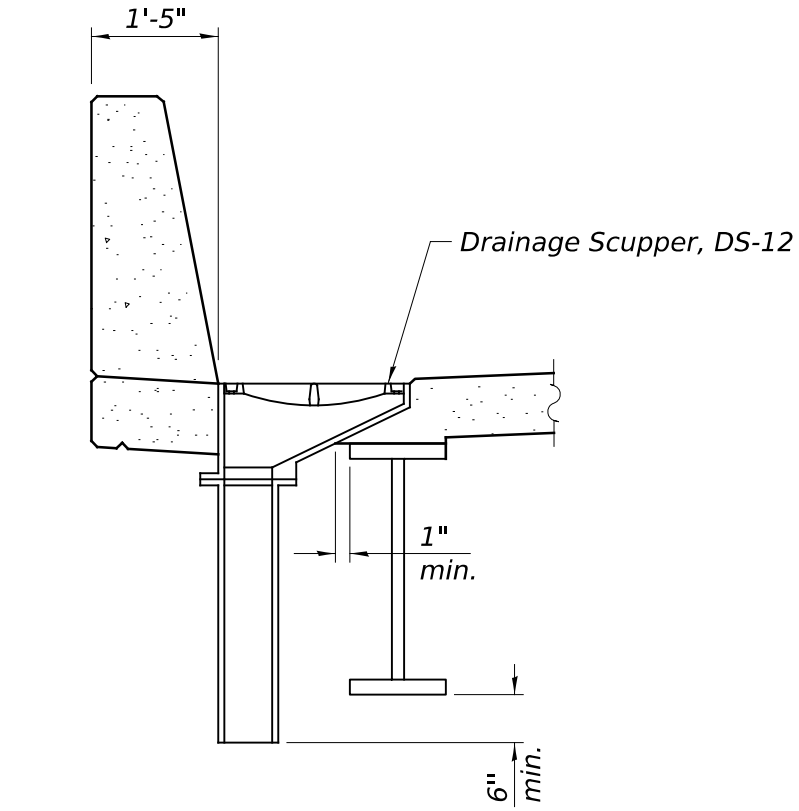
PLAN

Note:
Reinforcement bars designated (E) shall be epoxy coated.
Cut longitudinal reinforcement to clear drainage scuppers.

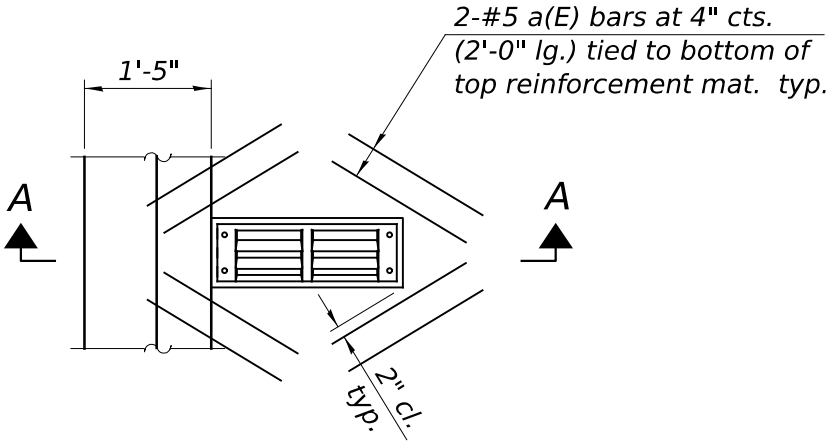
DRAINAGE SCUPPER
DS-11 INSIDE FASCIA BEAM

Figure 3.2.9-6

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SECTION A-A



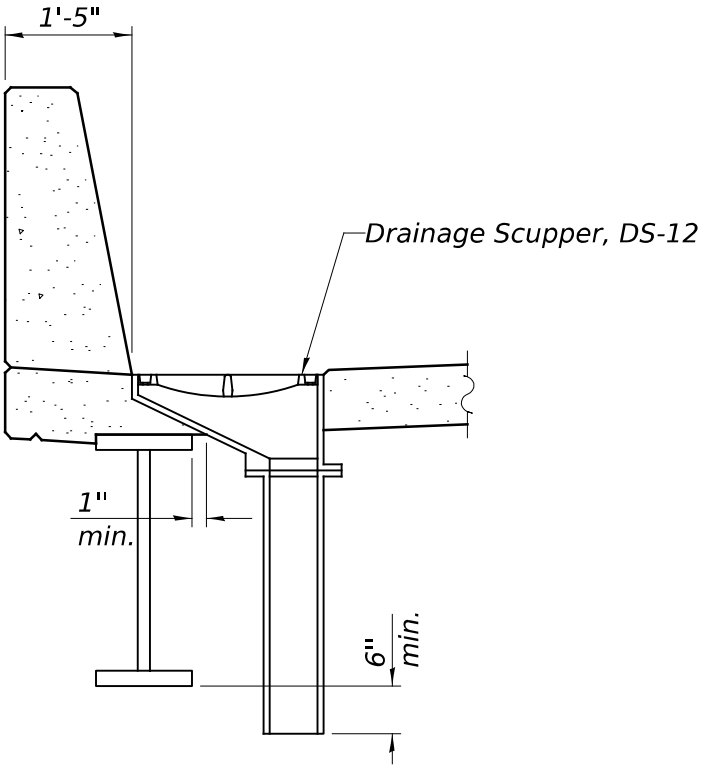
PLAN

Note:
Reinforcement bars designated(E) shall be epoxy coated.
Cut longitudinal reinforcement to clear drainage scuppers.

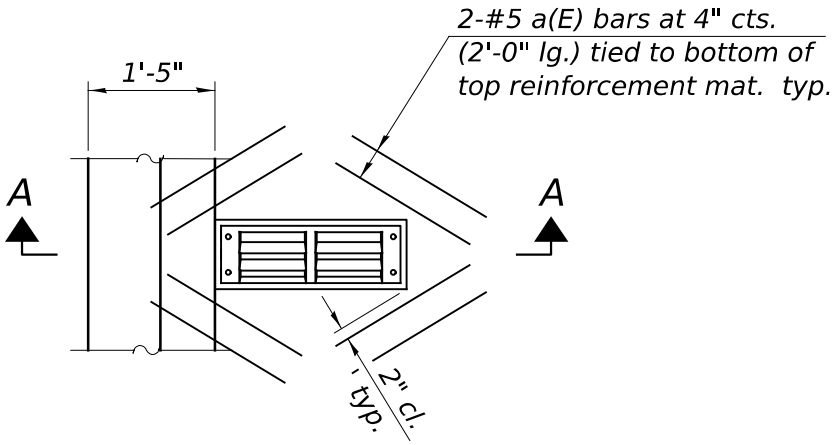
DRAINAGE SCUPPER
DS-12 OUTSIDE FASCIA BEAM

Figure 3.2.9-7

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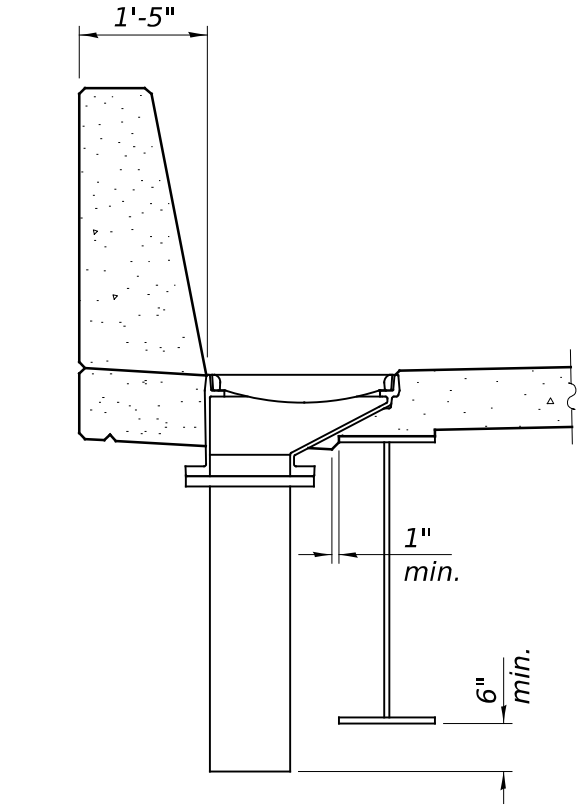
PLAN

Note:
Reinforcement bars designated (E) shall be epoxy coated.
Cut longitudinal reinforcement to clear drainage scuppers.

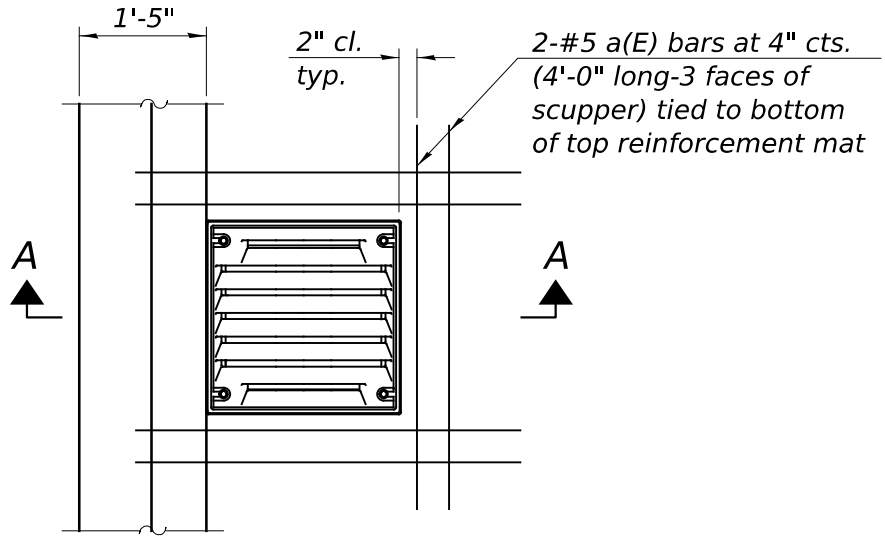
DRAINAGE SCUPPER
DS-12 INSIDE FASCIA BEAM

Figure 3.2.9-8

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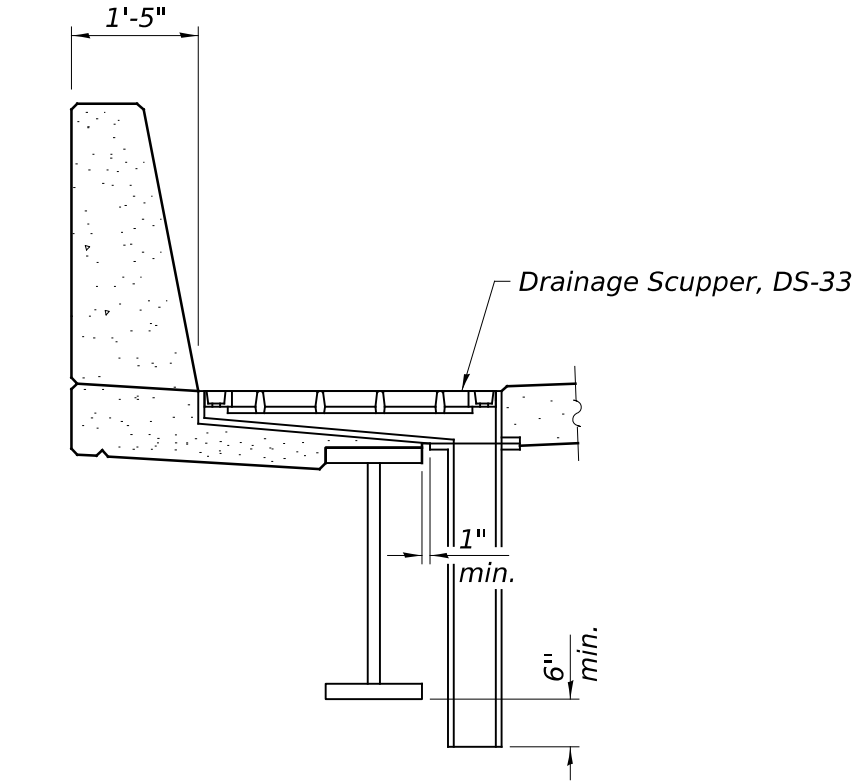
PLAN

Note:
Reinforcement bars designated (E) shall be epoxy coated.
Cut longitudinal reinforcement to clear drainage scuppers.

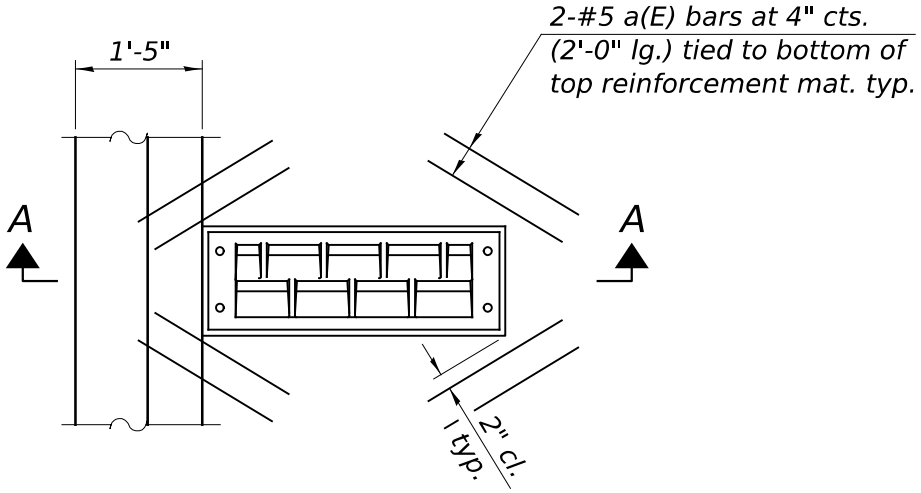
DRAINAGE SCUPPER
DS-12M10

Figure 3.2.9-9

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SECTION A-A



PLAN

Note:
Reinforcement bars designated(E) shall be epoxy coated.
Cut longitudinal reinforcement to clear drainage scuppers.

DRAINAGE SCUPPER
DS-33

Figure 3.2.9-10

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3.2.10 Light Poles and Parapet Conduit

Details for light poles mounted on bridge parapets are depicted in Figures 3.2.10-1 through 3.2.10-5. Light poles mounted on structures which are not subject to live load vibrations, such as retaining walls, will not require vibration isolation pads. In these cases only 5½ in. of anchor rod extension out of the concrete is necessary for the connection. If the anchor rod interferes with the finishing machine, it may be cut, threaded, and reconnected after deck finishing according to the Alternate Anchor Rod detail shown in Figure 3.2.10-5.

The preferred location for conduit is attached to beams or the bottom of the deck. Figures 3.2.10-1 and 3.2.10-2 illustrate details for these situations. An alternate d(E) bar shown in Figure 3.2.10-6 shall be used when conduit is present. If necessary, conduit may be placed in the parapet curb as shown in Figures 3.2.10-3 and 3.2.10-4, provided the following criteria are met:

1. Only conduit accommodating lighting or other traffic related utilities may be placed in the parapets.
2. The conduit shall be PVC pipe, Sch. 40 minimum wall.
3. The maximum single conduit diameter shall be a standard 2 in. conduit (2 3/8 in. O.D.).
4. A maximum of three single conduits may be used.
5. All conduit encased in a parapet shall have a minimum clearance of 1½ in. from all reinforcement.
6. Conduit and other electrical components depicted on the bridge plans should not be included in the bridge Bill of Materials and should not be included in Concrete Superstructure. Rather, these components should be paid for elsewhere in the contract with a note on the bridge plans indicating in what portion of the contract (Electrical, Roadway, etc.) the components will be paid.
7. The conduit shall be placed in the lower portion of the parapet and the following note shall be placed near the details:

   See Highway Standard 812001.

8. Highway Standard 812001 is preferred, but if it not possible the junction box may be placed in the back face of the parapet if the following conditions are met:
   a. The preferred maximum box width is 10”. If larger is required, one d(E) bar may be cut, d2(E) bars shall not be cut. A maximum of one e(E) bar may be cut.
   b. Cut bars only if necessary. Cutting of bars is only allowed on the compression side of wall, on the back face of the parapet.

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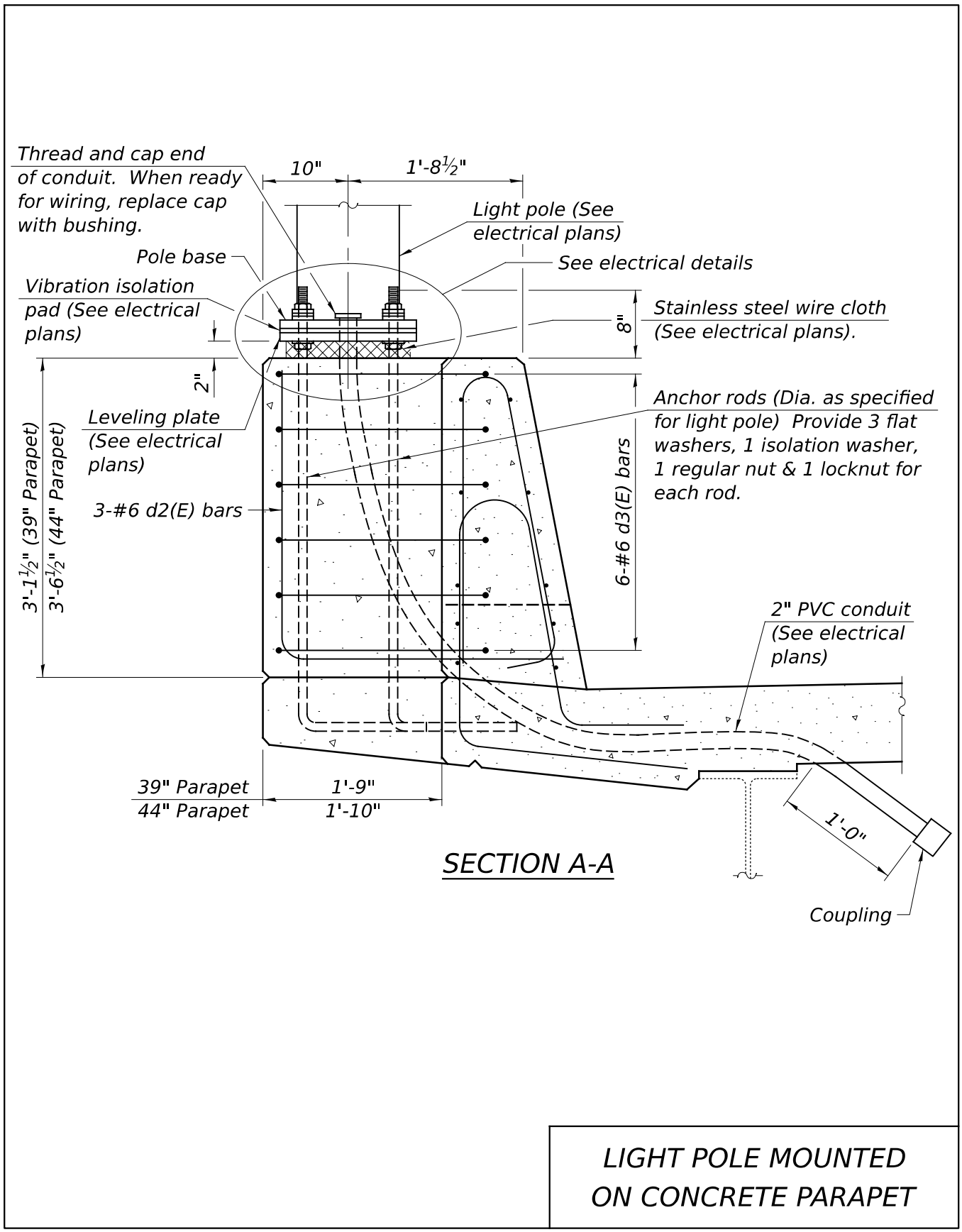
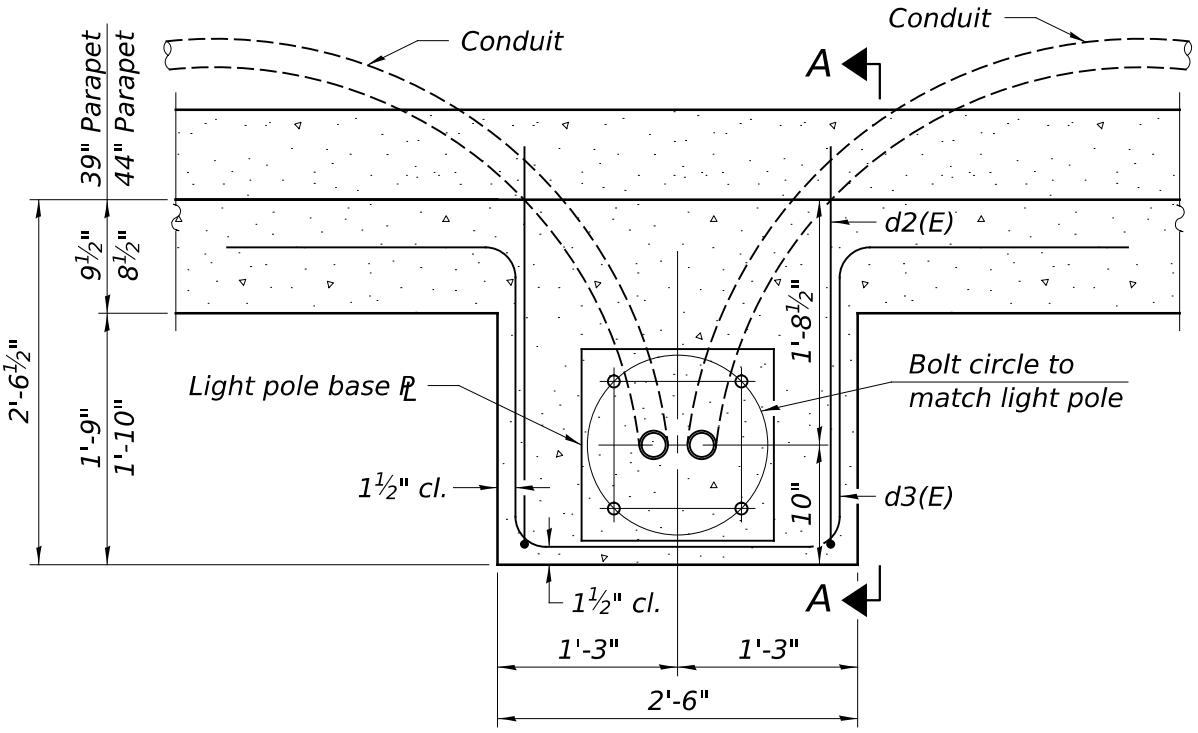


Figure 3.2.10-1

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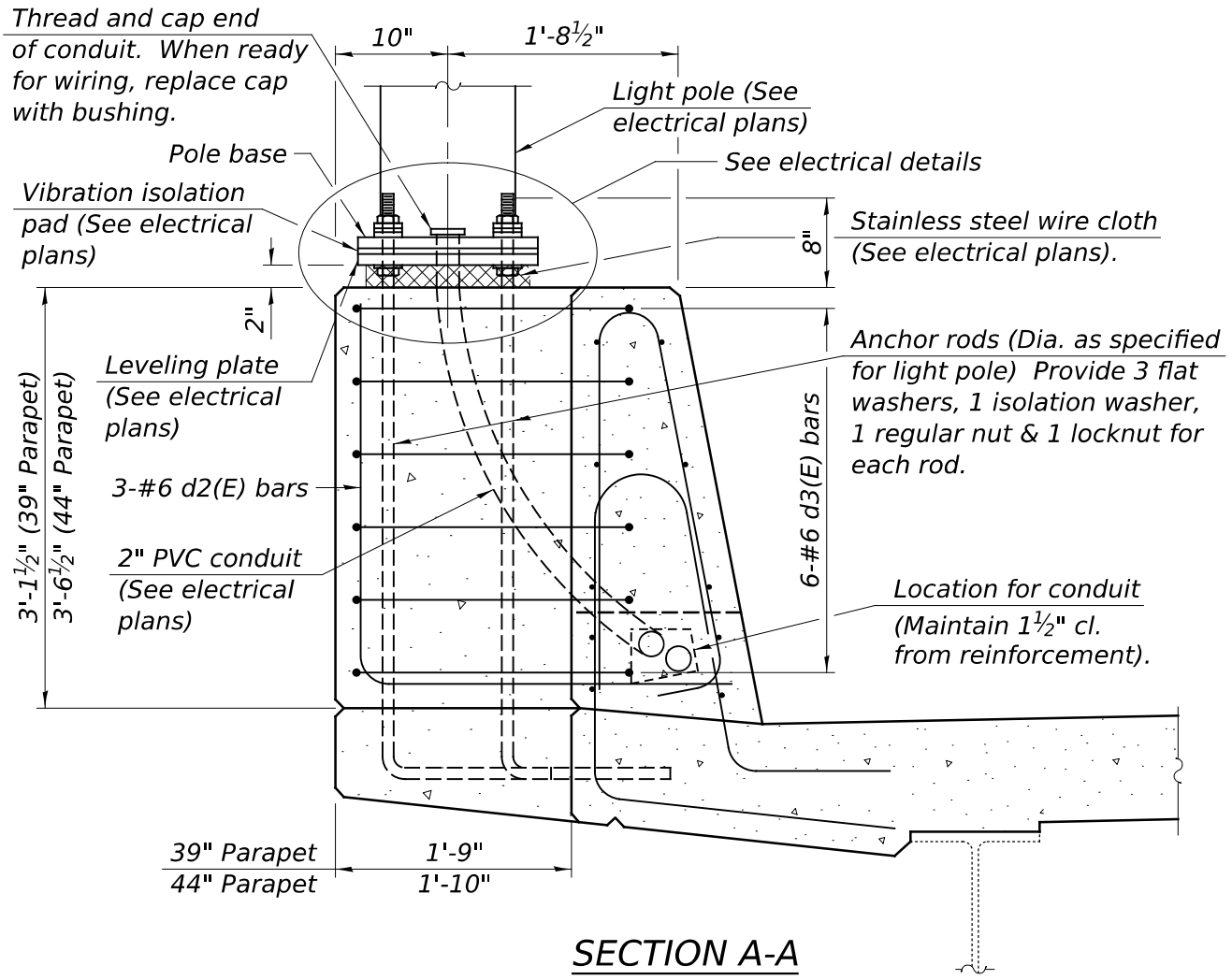
PLAN

Note:
Cost of anchor rods is included with Concrete Superstructure.

LIGHT POLE MOUNTED ON CONCRETE PARAPET

Figure 3.2.10-2

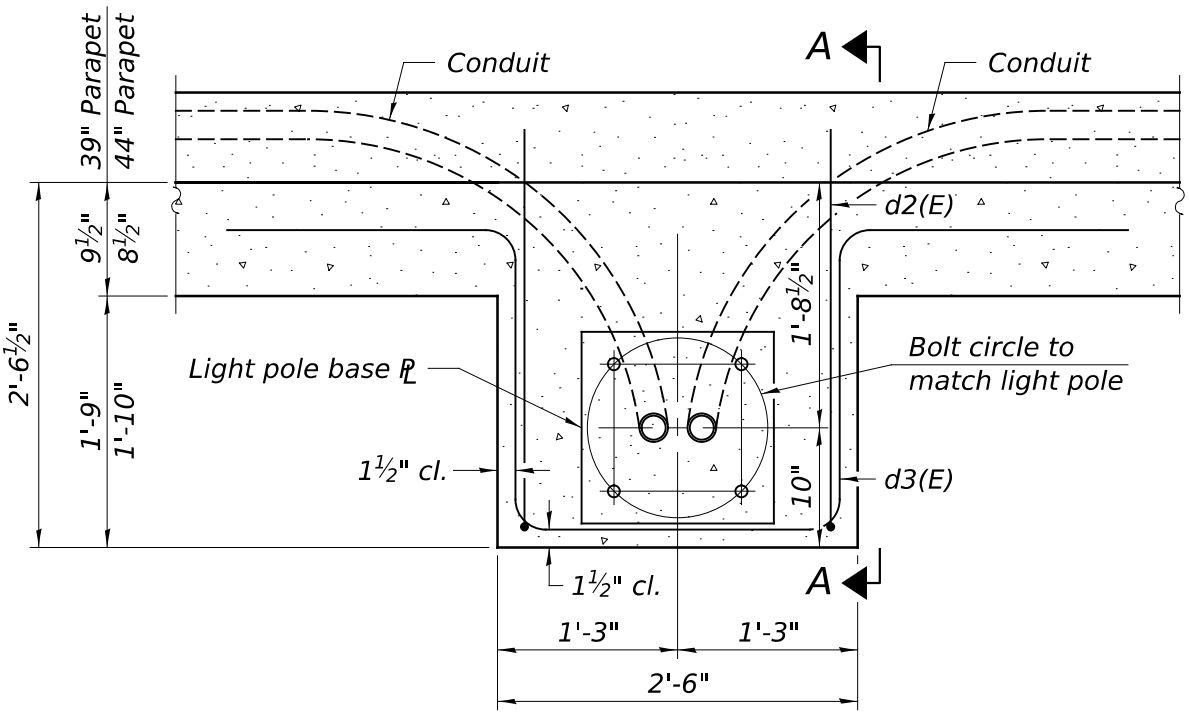
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ALTERNATE
LIGHT POLE MOUNTED
ON CONCRETE PARAPET

Figure 3.2.10-3

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PLAN

Note:
Cost of anchor rods is included with Concrete Superstructure.

ALTERNATE
LIGHT POLE MOUNTED
ON CONCRETE PARAPET

Figure 3.2.10-4

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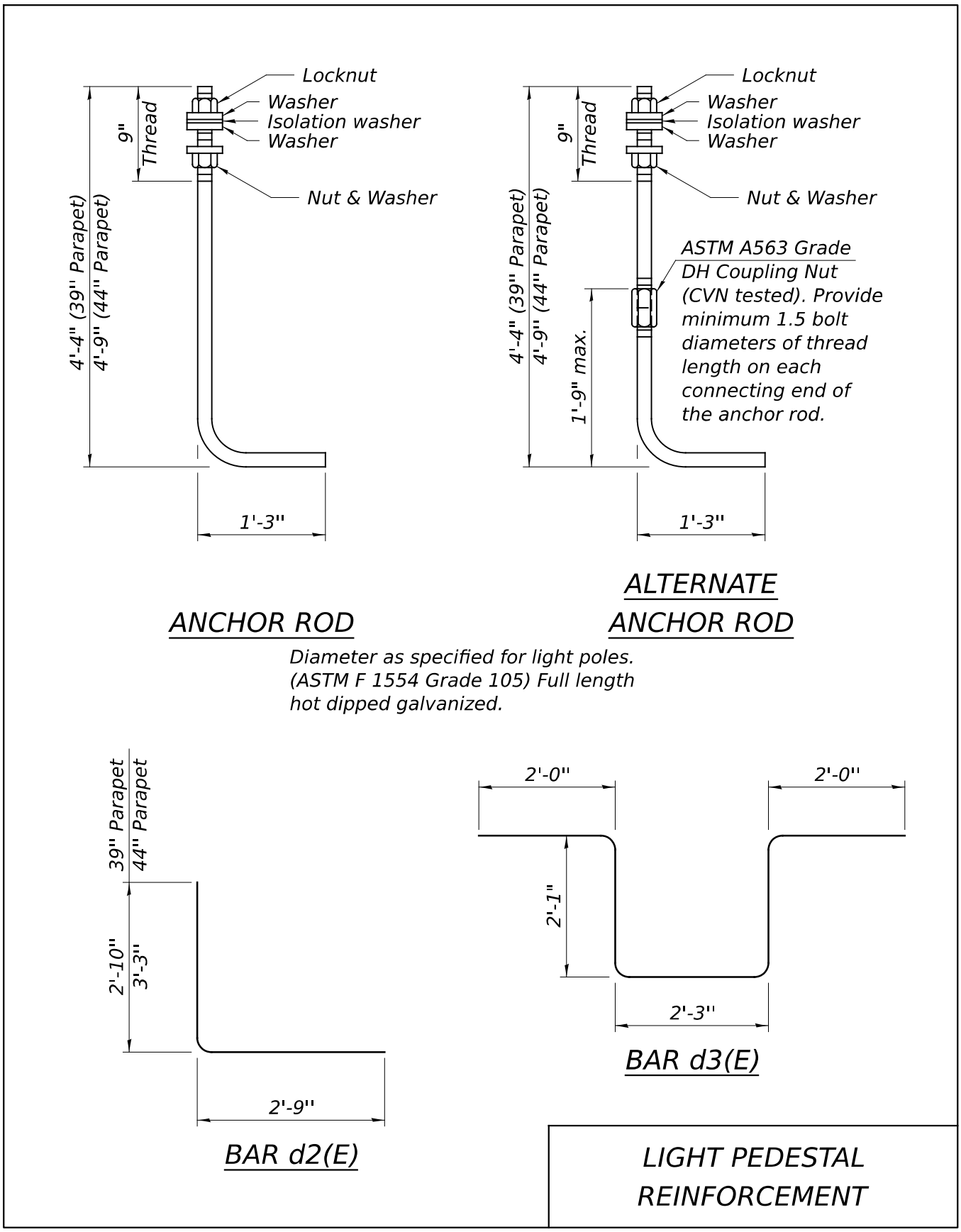
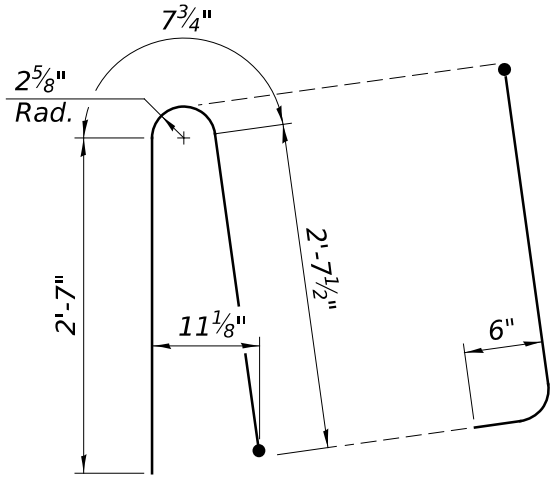


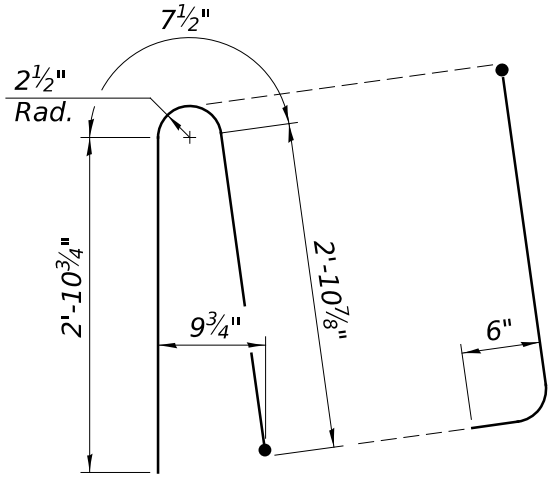
Figure 3.2.10-5

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ALTERNATE BAR d(E)

(For 39" constant slope parapet when conduit is present)



ALTERNATE BAR d(E)

(For 44" constant slope parapet when conduit is present)

ALTERNATE d(E) BAR
WHEN CONDUIT PRESENT

Figure 3.2.10-6

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3.2.11 Slab Bridges (Main Reinforcement Parallel to Traffic)

For slab bridges, the Department utilizes a policy which augments the requirements found in the AASHTO LRFD Bridge Design Specifications, resulting in a more robust design with less chance of cracking. The following equations and recommendations shall be used in addition to the AASHTO LRFD Bride Design Specifications' requirements for reinforced concrete design of slab bridges within the following limits:

- Skew Angle ≤ 50 degrees
- 20 ft. ≤ Span Length (ft.) ≤ 56 ft. *
- 24 ft. ≤ Bridge Width (ft.) ≤ 48 ft.
- 1.0 ≤ Continuous Span Ratio ≤ 1.4
- Transverse reinforcement shall be placed perpendicular to the centerline of bridge.

*Section 2.3.6.1.4 limits slab bridge spans to 40 feet. This limit may only be exceeded for rare circumstances when approved by the Bureau of Bridges & Structures.

3.2.11.1 General

3.2.11.1.1 Primary Longitudinal Reinforcement

The approximate method equivalent strip width, (E), for Slab Type Bridges, equations 4.6.2.3-1 and 4.6.2.3-2 of the AASHTO LRFD Bridge Design Specifications, remains unchanged.

The skew reduction factor (r), equation 4.6.2.3-3 of the AASHTO LRFD Bridge Design Specifications, shall be applied to the live load longitudinal force effect. The equations in this manual have been derived accordingly.

For slab bridges the cover on the bottom longitudinal bars shall be increased from 1" to 1½" to improve detailing and reduce the required lap lengths. As shown on the base sheets, the bottom longitudinal bars in cast-in-place bridge approach slabs have a 2" cover as directed by the AASHTO LRFD Bridge Design Specifications.

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3.2.11.1.2 Transverse Distribution Reinforcement

To allow for higher resistance to cracking, a series of beta factors are used to increase the transverse reinforcement. These factors account for skew, length, width, and continuity as described below. Therefore, in lieu of the AASHTO LRFD Bridge Design Specifications provisions of Article 5.12.2.1 the following equations shall govern the design and placement of transverse distribution reinforcement.

Edge beam reinforcement shall be investigated according to AASHTO LRFD Bridge Design Specifications Article 4.6.2.1.4b but any additional reinforcement determined to be required for the edge beam shall not be considered when determining the percentages of transverse reinforcement.

3.2.11.2 Simple Spans

3.2.11.2.1 Bottom Transverse Distribution Reinforcement

$$A_{s(bot,trans)} = \beta_{total(bot)} \times A_{s(bot,long)}$$

$$\beta_{total(bot)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \leq \beta_{max}$$

where:

$\beta_{total(bot)}$ = factor of main bottom longitudinal reinforcement.
$\beta_{base} = 0.21$
$\beta_{skew} = \tan\theta \times 0.35 (1 + 0.02(L - 20))$
$\beta_{length} = 0.30 - 0.0075L \geq 0.0$
$\beta_{width} = 0.02 \sqrt{(W - 24)} \geq 0.0$
$\beta_{max} = 0.70$
L = span length per AASHTO LRFD Bridge Design Specifications Table 4.6.2.2.1-2 (ft.)
W = physical edge-to-edge final width of bridge (ft.)
θ = skew angle (degrees)

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3.2.11.2.2 Top Transverse and Top Longitudinal Reinforcement

$A_{s(top,trans)} = \beta_{total(top)} \times A_{s(bot,long)}$
$A_{s(top,long)} = \beta_{total(top)} \times A_{s(bot,long)}$

$\beta_{total(top)} = 0.20$

where:

$\beta_{total(top)}$ = factor of main bottom longitudinal reinforcement.

This reinforcement shall not be less than that required for shrinkage and temperature according to AASHTO LRFD Bridge Design Specifications Article 5.10.6.

3.2.11.3 Continuous Spans

3.2.11.3.1 Bottom Transverse Distribution Reinforcement

$A_{s(bot,trans)} = \beta_{total(bot)} \times A_{s(bot,long)}$

$\beta_{total(bot)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \times 1.1 \leq \beta_{max}$

where:

$\beta_{total(bot)}$ = factor of main bottom longitudinal reinforcement.
$\beta_{base} = 0.21$
$\beta_{skew} = \tan\theta \times 0.2 (1 + 0.02(L - 20))$
$\beta_{length} = 0.32 - 0.0055L \geq 0.0$
$\beta_{width} = 0.02 \sqrt{(W - 24)} \geq 0.0$
$\beta_{max} = 0.80$
L = span length per AASHTO LRFD Bridge Design Specifications Table 4.6.2.2.1-2 (ft.)
W = physical edge-to-edge final width of bridge (ft.)
θ = skew angle (degrees)

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3.2.11.3.2 Top Transverse Distribution Reinforcement

$$A_{s(top,trans)} = \beta_{total(top)} \times A_{s(top,long)}$$

See Figures 3.2.11-4 and 3.2.11-5 for application. Note that in top slab regions where no negative moment exists, the minimum top longitudinal reinforcement and the minimum top transverse reinforcement shall be 20% of the main bottom longitudinal reinforcement.

$$\beta_{total(top)} = (\beta_{base} + \beta_{skew} + \beta_{length} + \beta_{width}) \times 1.2 \leq \beta_{max}$$

where:

$\beta_{total(top)}$ = factor of main top longitudinal reinforcement.
$\beta_{base} = 0.24$
$\beta_{skew} = \tan\theta \times 0.55 (1 - 0.013(L - 20)) \geq 0.0$
$\beta_{length} = 0.12 - 0.0025L \geq 0.0$
$\beta_{width} = \sin\theta \times 0.02 \sqrt{(W - 24)} \times (L / 20) \geq 0.0$
$\beta_{max} = 1.00$
L = span length per AASHTO LRFD Bridge Design Specifications Table 4.6.2.2.1-2 (ft.)
W = physical edge-to-edge final width of bridge (ft.)
θ = skew angle (degrees)

Skewed slab structures are generally designed for main reinforcement placed parallel to the centerline of roadway for continuous bridges and when the aspect ratio (width to length) is less than 3 for simple spans. The design span should be taken as the distance along the centerline of the roadway.

Skewed simple spans with an aspect ratio (width to length) greater than or equal to 3 may be designed to span the direct perpendicular distance between supports. When using this method, the main reinforcement shall be initially designed as if it was at right angles to the supports. The resulting required area of reinforcement per ft., if placed parallel to the centerline of roadway, shall then be multiplied by the secant of the skew angle squared $(A_s/ft. \times \sec^2\theta_{skew})$. The spacing of this amplified steel area is perpendicular to the direction of travel. Consequently, a further transformation (see Section 3.5.2 of the Culvert Manual available at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design for similar discussion) is required if the bars are detailed (spacing shown on the plans) parallel to the skew.

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All bridge deck reinforcement bars shall be epoxy coated. In addition, all bars in parapets, sidewalks and medians shall be epoxy coated.

If the out-to-out width of the superstructure exceeds 45 ft. – 0 in., an open longitudinal joint as shown in Figure 3.2.11-1 is required. However, if staged construction is being utilized or additional computations and detailing account for shrinkage, a joint may not be necessary. Consult the Bureau of Bridges & Structures when this situation arises.

Figures 3.2.11-1 through 3.2.11-5 present details and general placement of reinforcement for slab superstructures. References to appropriate articles of the AASHTO LRFD Bridge Design Specifications for slab bridge design are also included. According to AASHTO LRFD Bridge Design Specifications Article 9.7.1.4, edges of slabs shall either be strengthened or be supported by an edge beam which is integral with the slab. As depicted in Figure 3.2.11-1, the #5 d_1 bars which extend from the 39 in. constant slope barrier into the slab qualify as shear reinforcement (strengthening) for the outside edges of slabs. When a 39 in. or 44 in. constant slope barrier (with similar d_1 bars) is used on a slab bridge, its structural adequacy as an edge beam typically only needs to be verified. The barrier should not be considered structural. Additional #6 bars, 7'-4" in length, shall be placed and lapped to each top transverse bar to help satisfy Extreme Event II vehicular collision loading. Edge beam design is required for bridges with open joints and possibly at stage construction lines.

When the main slab reinforcement meets the design requirements for strength, fatigue, crack control, and limits and development of reinforcement; the minimum slab depth criteria specified in AASHTO LRFD Bridge Design Specifications Table 2.5.2.6.3-1 is not applicable.

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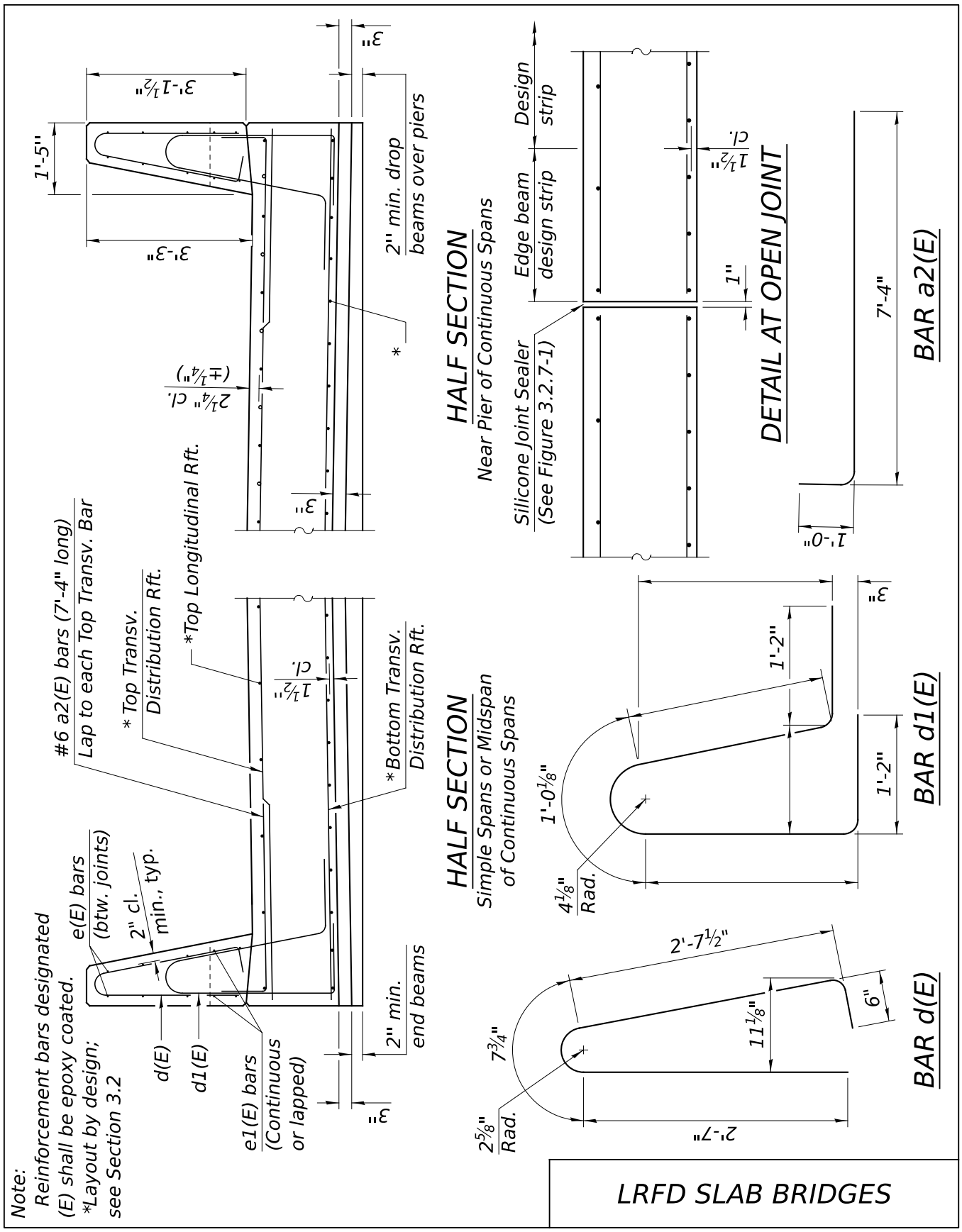
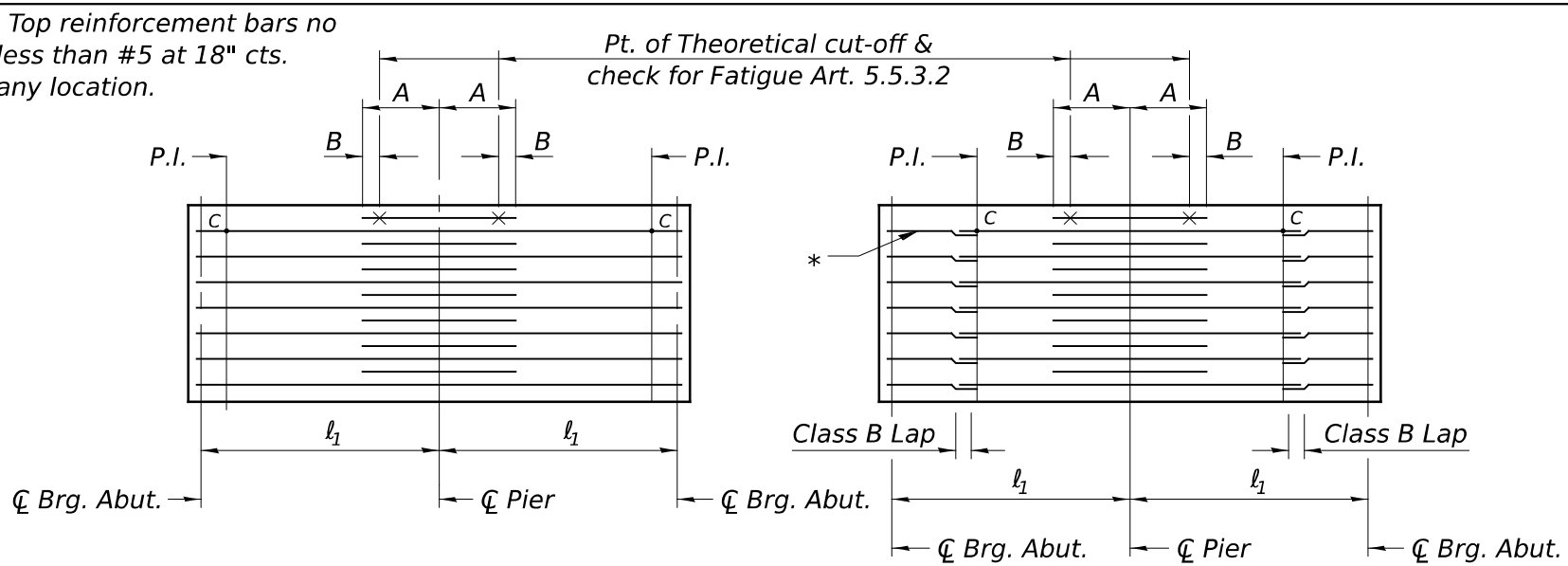


Figure 3.2.11-1

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REINF. IN TOP OF SLAB ($\ell_1 \leq 18'$)

(Use same size bars, or two sizes where long bars are not less than ⅓ A_S. A_S = Total Neg. Moment steel area.)

REINF. IN TOP OF SLAB ($\ell_1 > 18'$)

(Use same size bars, or two sizes where long bars are not less than ⅓ A_S. A_S = Total Neg. Moment steel area.)

REFERENCE TO AASHTO:

A - Article 5.10.8.1.2a & 5.10.8.2
B - 5.10.8.1.2a
C - 5.10.8.1.2c
D - 5.10.8.1.2b

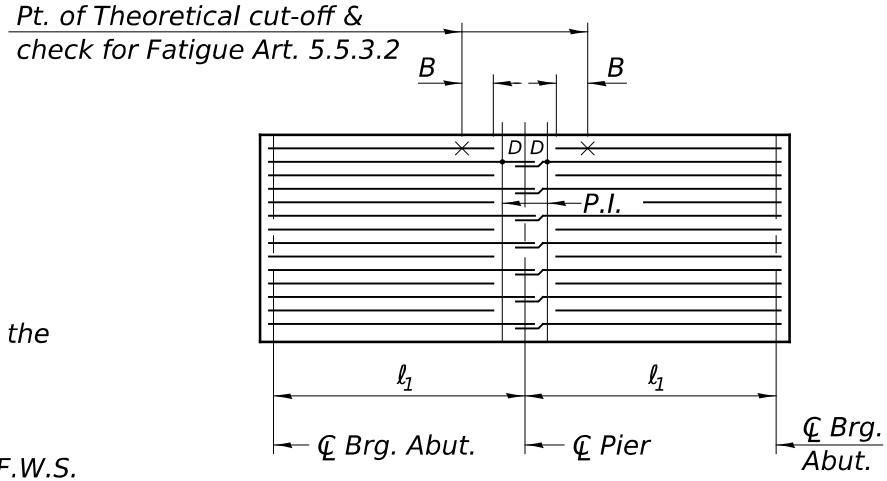
Notes:

P.I. - The point of Inflection as determined from the applied Moment Envelopes of:

D + [-(LL + I)] for Top Reinforcement

D + [+(LL + I)] for Bottom Reinforcement

P.I.'s shall be calculated both with and without F.W.S. The P.I. further from the pier shall be used in detailing bar cutoffs.



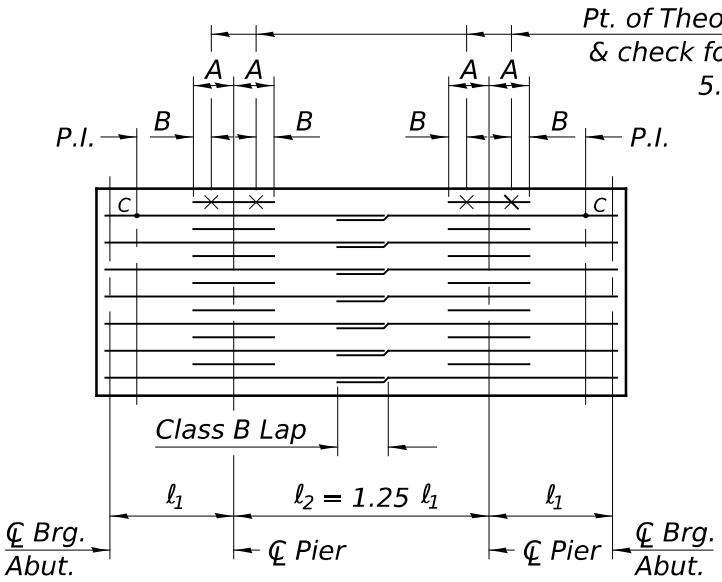
REINF. IN BOTT. OF SLAB

(Use same size bars)

LRFD TWO SPAN SLAB BRIDGES

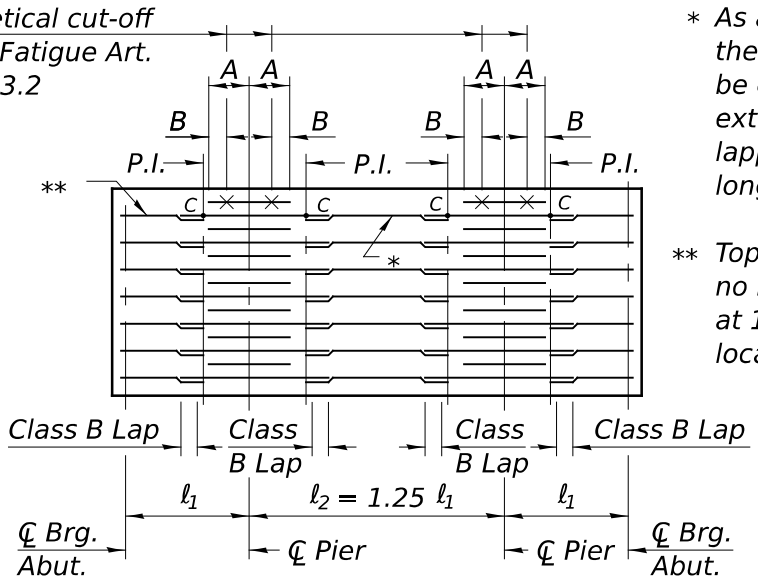
Figure 3.2.11-2

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REINF. IN TOP OF SLAB ($\ell_1 \leq 20'$)

(Use same size bars, or two sizes where long bars are not less than ⅓ A_S. A_S = Total Neg. Moment steel area.)



* *As an alternate, these bars may be omitted by extending and lapping the long bars.*

** *Top reinf. bars no less than #5 at 18" cts. any location.*

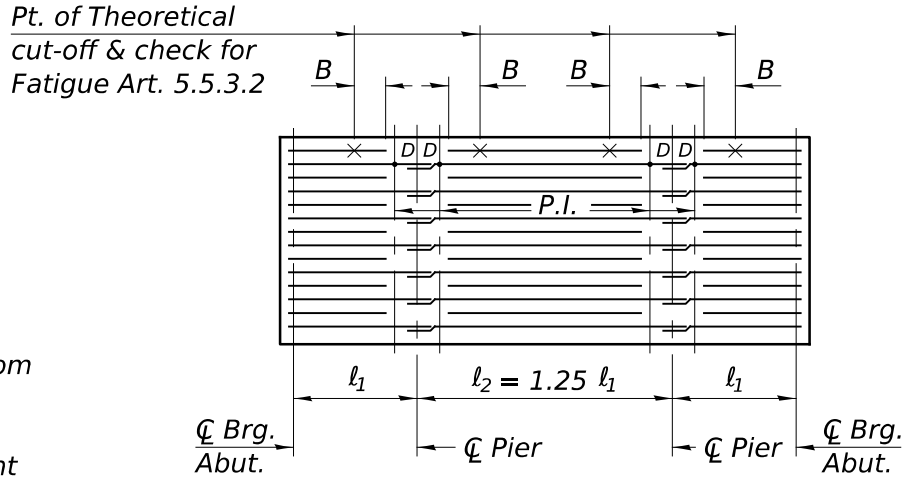
REINF. IN TOP OF SLAB ($\ell_1 > 20'$)

(Use same size bars, or two sizes where long bars are not less than ⅓ A_S. A_S = Total Neg. Moment steel area.)

REFERENCE TO AASHTO:

A - Article 5.10.8.1.2a & 5.10.8.2
B - 5.10.8.1.2a
C - 5.10.8.1.2c
D - 5.10.8.1.2b

Notes:
P.I. - The point of Inflection as determined from the applied Moment Envelopes of:
D + [-(LL + I)] for Top Reinforcement
D + [+(LL + I)] for Bottom Reinforcement
P.I.'s shall be calculated both with and without F.W.S. The P.I. further from the pier shall be used in detailing bar cutoffs.



REINF. IN BOTT. OF SLAB

(Use same size bars)

LRFD THREE SPAN SLAB BRIDGES

Figure 3.2.11-3

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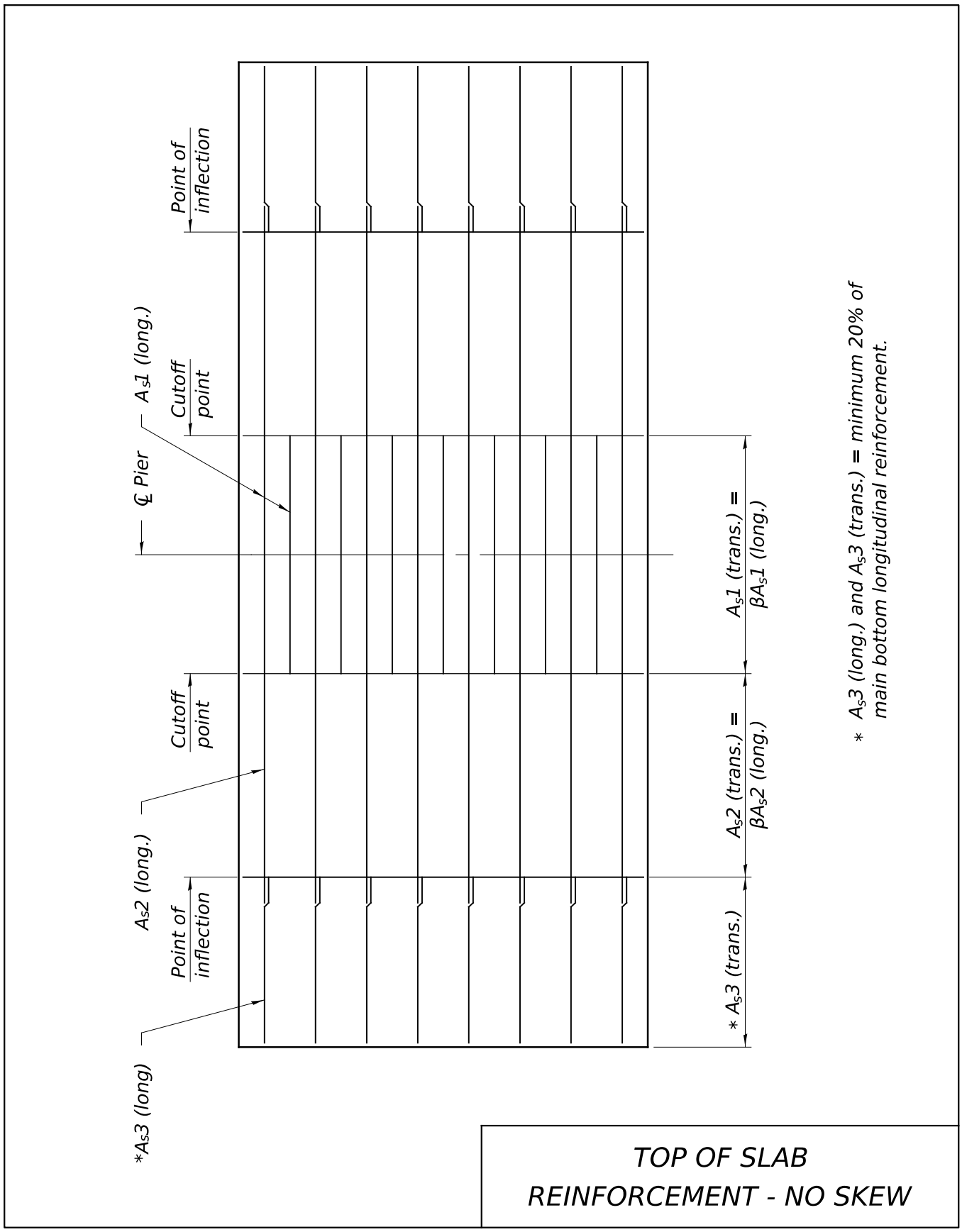


Figure 3.2.11-4

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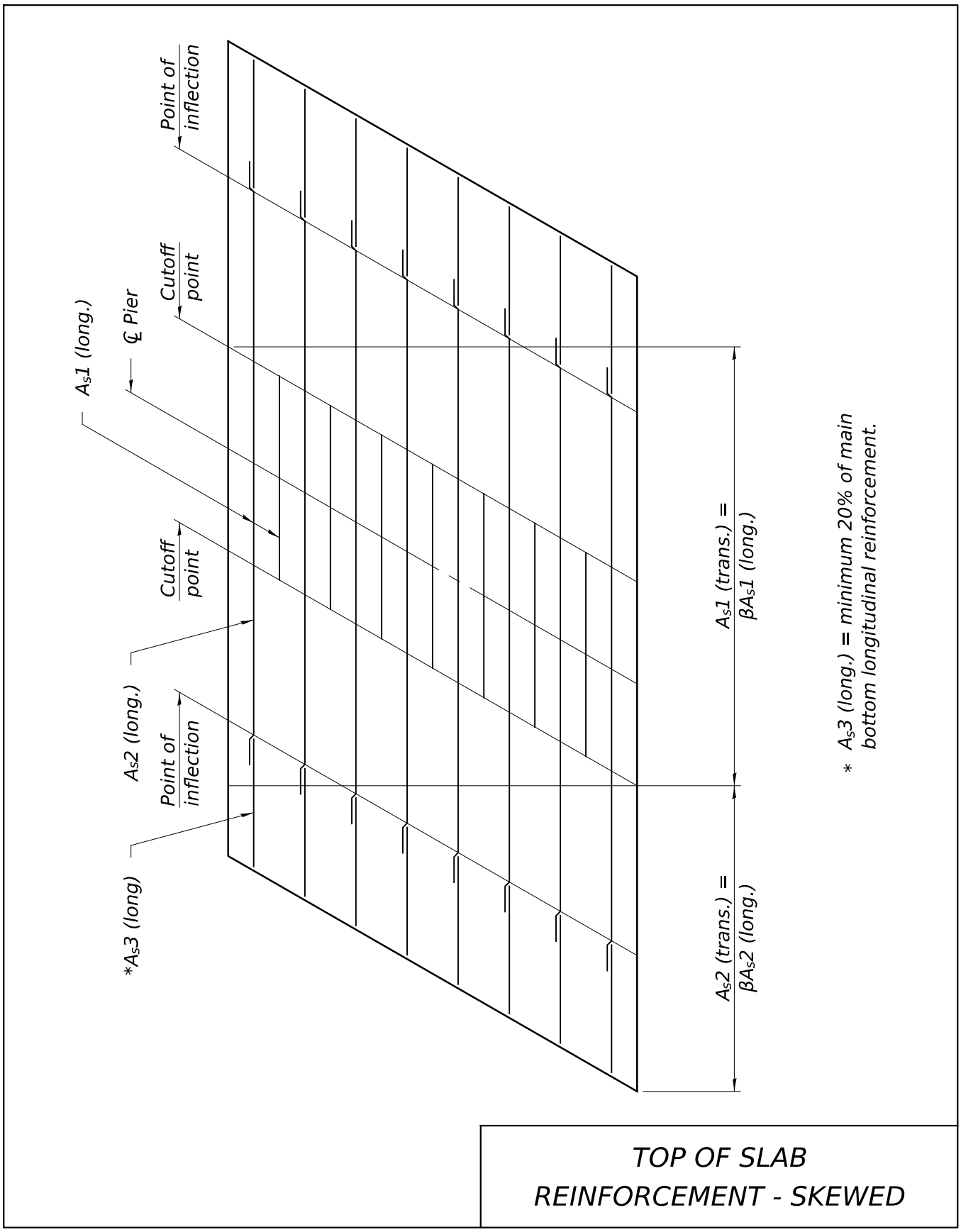


Figure 3.2.11-5

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3.2.12 Bridge Approach Slabs and Approach Footings

Approach slab base sheets, at the IDOT CADD Bridge Cell Library, provide details for IDOT's standard cast-in-place bridge approach slabs and approach footings. The reinforcement in these details has been standardized. Transverse reinforcement is placed parallel to skew to avoid cutting the majority of bars in skewed bridges. Cast-in-place approach slabs shall be used for all stub abutments, and for integral and semi-integral abutments when the expansion length to both abutments does not exceed 130 feet.

Base sheets in the BA-PDP series provide details for IDOT's standard partial depth precast bridge approach slabs and cast-in-place approach footings. These slabs consist of 11 in. depth precast (not prestressed) concrete deck beams, that are placed adjacently and joined with shear keys and a structural overlay. Precast bridge approach slabs shall be used for integral and semi-integral abutments when the expansion length to one or both of the abutments is greater than 130 feet.

Transverse bar splicers shall be used across stage construction joints.

The following assumptions were made in designing the bridge approach slabs and approach footings:

- AASHTO LRFD Bridge Design Specifications 9th Edition, HL-93 Loading
- f'_c = 4.0 ksi
- f_y = 60 ksi
- Approach Slab Span Length = 26 ft. – 1 in.
- Bottom of Slab Clear Cover = 2 in.
- Simply Supported Behavior
- Slab Bridge Live Load Distribution (AASHTO LRFD Bridge Design Specifications Article 4.6.2.3)
- One-Way Slab Behavior in each direction of the Approach Footings
- Flexible foundation model for the Approach Footings, Q_u > 2.0 ksf

Bridge approach slabs shall receive the same surface treatment as the bridges to which they are attached. For example, if the bridge has Bridge Deck Grooving and Protective Coat, the approaches shall receive the same treatment according to Section 503 of the Standard Specifications.

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When utilized with integral abutments, the following design considerations should be made:

1. Bridge approach slabs shall be cast-in-place (CIP) unless the longest distance from the centroid of stiffness of the structure to the back of abutment is greater than 130 feet (for examples of this calculation, refer to the "Integral Abutment Pile Selection" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical). In this case, the structure shall utilize a precast bridge approach slab on both ends of the bridge, regardless if the distance on the other end of the bridge is less than or equal to 130 feet. For further reference, see the "Approach Slabs – Bridge Cast in Place" IDOT CADD Bridge Cell Library.
2. A rigid Pavement Connector (PCC) for Bridge Approach Slab (Highway Std. 420401) is necessary to connect to CIP and Partial Depth Precast Bridge Approach Slabs on all integral structures for proper installation of the expansion joint. If the total integral structure length is less than or equal to 100 feet, the owner may omit the expansion joint at the end of the bridge approach slab and use a flexible Pavement Connector (HMA) for the Bridge Approach Slab (Highway Std. 420406). The designer should coordinate and confirm with the owner.
3. Partial depth precast bridge approach slabs may vary in width from 3'-0" to 6'-0" as necessary to satisfy the location of the stage construction line and edge of pavement. It is preferrable to have an even number of precast slabs to better match the crown of the roadway. The Department recommends utilizing wider precast slabs when possible to minimize the number of longitudinal joints. The reinforcement details on the base sheets are adequate for all widths in this range. Adjacent precast bridge approach slabs shall be separated by a 2" Styrofoam block full length along the bottom of the precast slab. See the "Approach Slabs – Bridge Partial Depth Precast" IDOT CADD Bridge Cell Library. This enables the shear key to be cast with the concrete wearing surface. Partial depth precast bridge approach slabs shall have a 5" concrete wearing surface on top. The pay item for partial depth precast bridge approach slabs is Precast Bridge Approach Slab.

When approach slab parapets are shown on the TSL, they shall be detailed as extending to a distance of 15 feet behind the back wall of the abutment (unless otherwise specified on the TSL). Beyond this point, appropriate guard rail terminals shall be used. Approach slabs shall be detailed as extending full width under parapets, with no parapet overhang. Guard rail terminals shall not be shown on approach slab sheets. When an open joint occurs at an abutment, parapets shall be detailed according to the applicable details in Section 3.6 of the Bridge Manual, including the point block and sliding plate details if necessary. When integral or semi-integral abutments are present, the parapets shall have parapet joints at the back of the abutment according to Figure 3.2.4-10. These joints shall be placed perpendicular to the roadway and include the aluminum joints according to Section 3.2.4. The additional aluminum joint requirements for long spans given in Figures 3.2.4-8 and 3.2.4-9 need not be followed at abutments.

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When side-mounted railings are shown on the TSL, they shall extend onto the approaches for a distance of 15 feet (unless otherwise specified on the TSL). The connection into the approach slab shall be the same as that used on the bridge and as that specified on the applicable railing base sheet. Minor modifications to the applicable railing base sheet may be required to detail the approach slab connection. Beyond the end of railing, guard rail terminals will be used. Guard rail terminals need not be shown on approach slab sheets. When an open joint occurs at an abutment, rail splices shall be detailed at these locations. When integral or semi-integral abutments are present, rails shall be shown as having splices at the backs of abutments.

When parallel wingwalls are present, the wingwalls and parapets shall be the lengths required by Section 2.3 of the Bridge Manual or as shown on the TSL.

When shoulder inlets with curbs (Highway Standard 610001) are required by the TSL plan, they shall be shown on the bridge approach slab sheets with a reference to the roadway plans for quantities.

Bridge approach footings are detailed 10 in. thick and follow the grade. When the roadway is superelevated, the footing shall be placed parallel to the bottom of the bridge approach slab.

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3.2.13 Deck Treatment

Prior to 2023, the Department standard for waterproofing of concrete bridge decks has been the Waterproofing Membrane System specified in Section 581 of the Standard Specifications for Road and Bridge Construction. Since then, the Department developed an alternative system, titled Full Lane Sealant Waterproofing System, which creates a cost-effective, high-performance substitution for the traditional waterproofing membrane system for concrete bridge decks. The system uses a combination of highly polymerized asphalt interlayers of Full Lane Sealant (FLS) with dense, high-quality hot-mix asphalt (HMA) to create a waterproofing system that is easier to construct and prevents the ingress of water and deicing chemicals to protect concrete bridge decks from chloride ingress and subsequent spalling and corrosion.

When used, the details for the waterproofing membrane system and HMA overlay should be replaced with the detail in Figure 3.2.13-1. The pay items used shall be as follows:

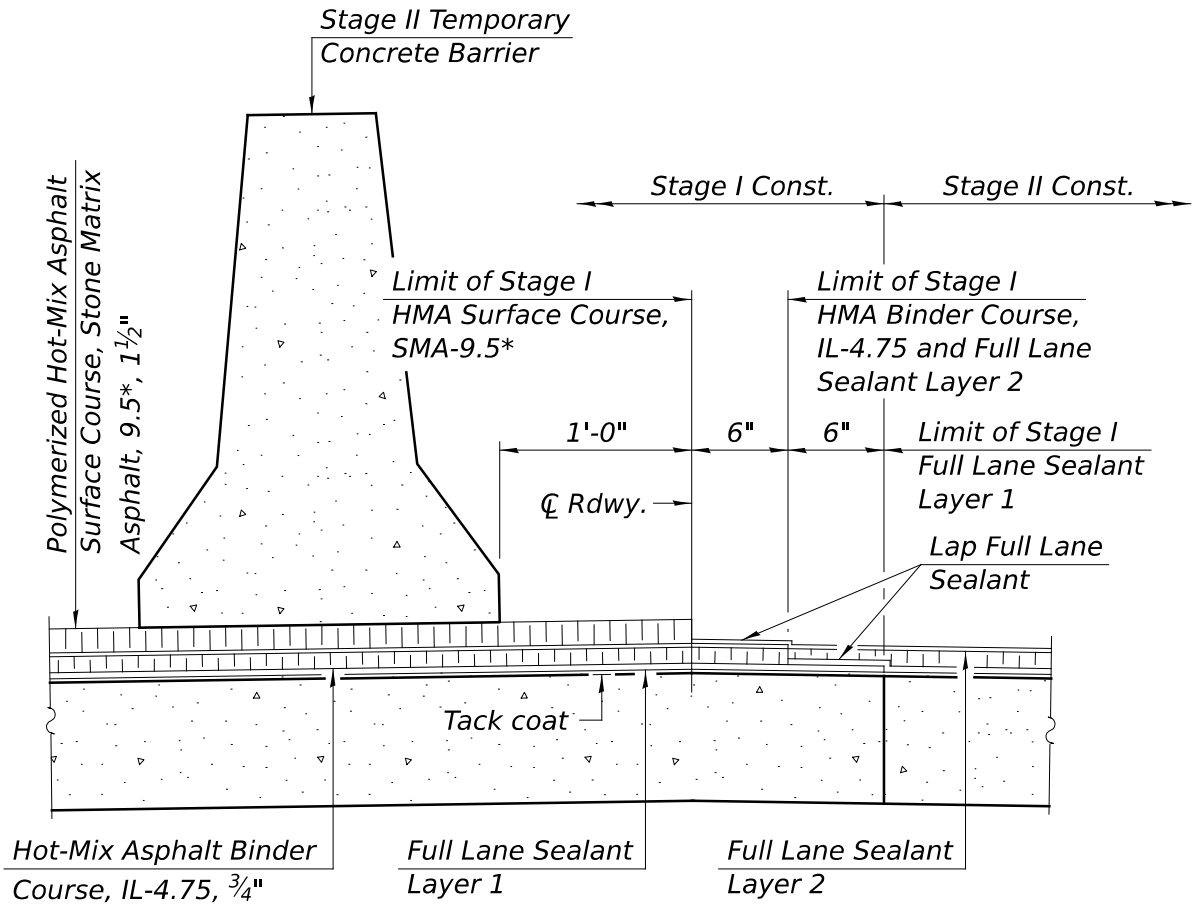
- FULL LANE SEALANT WATERPROOFING SYSTEM, which includes the tack coat, both layers of FLS, and the ¾ in. thick layer of IL-4.75 HMA Binder Course.

- POLYMERIZED HOT-MIX ASPHALT SURFACE COURSE, STONE MATRIX ASPHALT, 9.5, which includes the 1½ in. thick layer of SMA-9.5 Surface Course.

The minimum thickness of the system is 2.25 inches. When required, additional thickness may be added by increasing the thickness of the HMA Binder layer.

The Bureau of Design & Environment maintains a special provision FULL LANE SEALANT WATERPROOFING SYSTEM, which shall be inserted into contracts containing this system.

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WATERPROOFING STAGING

Full Lane Sealant Waterproofing System consists of an initial tack coat to promote bonding, Layer 1 of Full Lane Sealant, a layer of Hot-Mix Asphalt Binder Course, IL-4.75, and Layer 2 of Full Lane Sealant.

** Mix and N-number to be determined in design.*

FULL LANE SEALANT WATERPROOFING SYSTEM

Figure 3.2.13-1

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3.3 Structural Steel

This section covers the Department's policies related to structural steel components, splices, and connections, which are used to support loads and elements specified by the AASHTO LRFD Bridge Design Specifications. Steel components supporting a reinforced concrete deck and that meet the material requirements of the Department comprise the standard basis for design.

3.3.1 Materials

AASHTO LRFD Bridge Design Section 6.4 presents the material designations (for example, *AASHTO M 270M/ M 270 Grade 50*) for various steel and metals used in bridge components. Section 1006 of the Standard Specifications, CADD base sheets, and general notes associated with structural steel components adopt and refer to the "AASHTO" designation unless the "ASTM" designation is the only available standard. For additional information about AASHTO Designation Numbers, refer to the "Standard Specifications for Transportation Materials and Methods of Sampling and Testing and Provisional Standards" and its "Table of Contents" available for download at the primary AASHTO Downloads website (as of December 2024).

The density of structural steel is typically 490 pounds per cubic foot. ASTM further adjusts the steel weight calculation as discussed in Section 3.3.28 for "Computation of Structural Steel Quantities." Specific steel processes, such as galvanizing in Section 3.3.29.3, may also affect the weight of the structural steel. The designer shall consider both adjustments when calculating the dead load and the plan poundage of "Calculated weight of Structural Steel" presented in the plan General Notes.

3.3.2 Limiting Live Load Deflection of Beams and Girders

The limiting ratios of live load deflection to span length for simple or continuous spans, as given in AASHTO LRFD Bridge Design Specifications Article 2.5.2.6.2, shall be applied to all vehicular bridge types with pedestrian sidewalk and/or bicycle lanes.

The live load deflection shall be computed considering all beams acting together and having equal deflection. For composite designs, the stiffness of the design cross section shall not include the effects of railings, parapets, sidewalks, or medians. Impact or the dynamic load allowance shall apply.

The live load deflection distribution factor shall be calculated using the following distribution formula:

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$$g\,(\text{deflection}) = m\frac{N_L}{N_b}$$

Where:
m = Multiple Presence Factor
N_L = Number of lanes
N_b = Number of beams

3.3.3 Uplift at End Reactions

Guidance on how to check for uplift can be found in the commentary of Article 3.4.1 in the AASHTO LRFD Bridge Design Specifications.

Uplift at end reactions resulting in upward beam deflections shall be evaluated using the live load deflection distribution factor above and Service I loading. All lanes shall be loaded simultaneously and impact shall apply. When using standard detailing practices, if uplift is unavoidable then the designer shall make the modifications as necessary to eliminate uplift.

The allowance for future wearing surface should not be included in uplift calculations when it increases the end reactions.

3.3.4 Design of Steel Members- General

All steel stringers, girders, floor beams or sub-stringers shall be of rolled beam or welded plate design. Generally, shop connections shall be welded and field connections shall be made with mechanical fasteners.

The section of a rolled beam stringer of continuous design may be varied at field splices so long as a constant nominal depth is maintained.

Conservation of material in welded plate girders usually requires transitions in flange plate thickness and/or width. Width and/or thickness transitions typically occur at field splices to minimize butt welding requirements. Shop welded flange transitions should be limited to thickness changes only for efficient fabrication. Welded girder segments between field splices may be detailed with a few transverse stiffeners (in addition to cross-frame connection plates) to avoid heavier webs. See Section 3.3.14. Longitudinal stiffeners should only be considered for deep webs, usually over 10 ft. See Section 3.3.20.

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In determining the features of structural steel elements, however, conservation of material should not receive unwarranted emphasis. In welded plate girder design, minimum web thickness shall be $\frac{7}{16}$ in., and a thicker web avoiding extra stiffeners may be more economical for fabrication while providing additional capacity. Simplification and repetition of details, ease of erection, and stability during construction are some of the factors to be considered in design.

AASHTO M270 (ASTM A709) Grade 50 or 50W steel shall be considered the preferred material for designing primary members. ASTM A709 Grade 36 shall be the default material for all diaphragms, cross-frames, and connecting plates or angles on straight structures. For weathering steel (AASHTO M270 Grade 50W), all diaphragms, cross-frames, and connecting plates or angles shall also be AASHTO M270 Grade 50W. See Section 2.3 for further guidance.

Appendix A of Section 6 in the AASHTO LRFD Bridge Design Specifications contains alternative provisions for determining the flexural resistance of straight steel beams in negative moment regions. As permitted (i.e., if a steel beam or girder qualifies), Appendix A should be used for the design of steel beams or girders over piers in straight bridges. See also Section 3.1.12 for additional information.

3.3.4.1 Fatigue Analysis

The fatigue loading frequency is determined as the number of trucks per day in a single lane averaged over the design life ($ADTT_{sl}$) as given in AASHTO LRFD Bridge Design Specifications Article 3.6.1.4. As the equations in this section specify a 75-year design life, the ADTT used to calculate $ADTT_{sl}$ shall be that at 37.5 years. This number may be linearly extrapolated from values of ADTT given on the TSL. See Design Guides 3.3.4 and 3.3.9 on the Bridges & Structures page for examples of calculation of $ADTT_{sl}$.

Distortion-induced fatigue has been observed to be a problem for some existing bridges. Detailing and design procedures for distortion-induced fatigue are given in AASHTO LRFD Bridge Design Specifications Articles 6.6.1.3 and 6.10.5. The rehabilitation of existing bridges should be analyzed for those details. In recent years, distortion-induced fatigue has caused separation of stiffeners from flanges in negative moment areas.

When using the distribution factors contained in Section 4, the final distribution factor shall be divided by 1.2 to eliminate the single-lane multiple-presence factor as given in AASHTO LRFD Bridge Design Specifications Article 3.6.1.1.2.

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3.3.4.2 Nonredundant Steel Tension Members

The Final Structure Plans shall reflect all details and notes associated with the redundancy analysis in accordance with Section 3.1.13, Section 3.1.14.8, and the guidance herein. Bridges with Nonredundant Steel Tension Members (NSTMs) require hands-on inspection by qualified inspectors at an interval defined in the Illinois Department of Transportation (IDOT) *Structural Services Manual* Section 3.4.5 – Fracture Critical Member Inspection Interval. This hands-on inspection often requires additional equipment, such as manlifts and snoopers, and the accompanying lane closures to accommodate said equipment. This results in considerable traffic delays and monetary costs to the motoring public and bridge owners.

To avoid these costs, some members that do not have load path redundancy may be shown to be system redundant or internally redundant, therein removing the NSTM classification and the hands-on inspection requirements. In order to classify a member as redundant, Federal Highway Administration (FHWA) requirements must be met. These requirements are provided in the May 9, 2022, FHWA memorandum titled "Inspection of Nonredundant Steel Tension Members".

The May 9, 2022, FHWA memorandum was written in response to new updates to the National Bridge Inspection Standards (NBIS) in 23 CFR part 650, subpart C (23 CFR 650.C) that became effective June 6, 2022.

To reduce costs associated with NSTM inspections, primary steel tension members without load path redundancy should be designed as system or internally redundant whenever possible.

This section provides guidance on the following:

- Definitions of types of redundancy for steel members
- Classification of member redundancy
- Required submittals and plan details for system redundant, internally redundant, and nonredundant members

The policy herein applies to newly designed yet to be constructed bridges only.

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3.3.4.2.1 Definitions of Types of Redundancy for Steel Members

The first update resulting from the latest NBIS is the introduction of the term Nonredundant Steel Tension Member, defined as a primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse. The term NSTM replaces the term Fracture Critical Member (FCM). The design, fabrication, in-service inspection, and load rating requirements currently in place for FCMs shall apply to NSTMs. The second update resulting from the latest issuance of the NBIS is the formal recognition of the following sources of redundancy:

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

FHWA considers the primary longitudinal load-carrying members in bridges with three or more primary load-carrying members to be load path redundant members, e.g., girders in a three-girder system. A redundancy evaluation is not necessary for the determination of load path redundancy, as it is for system redundancy or internal redundancy.

Primary steel tension members that do NOT have load path redundancy are by default considered NSTMs, unless system redundancy or internal redundancy is shown. The list of members that IDOT considers to NOT have load path redundancy can be found in the IDOT *Structural Services Manual* Section 3.3.5.1 as well as in the coding instructions of Item No. 92A1 in the *Illinois Highway Information System Structure Information and Procedure Manual*. Note that the documents referenced are current as of June 2023 and that IDOT is currently working of updates to both of these documents.

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

To demonstrate system redundancy, it is necessary to evaluate the structure following a hypothetical failure of a primary load carrying member, as the number of primary load carrying members is not sufficient to demonstrate load path redundancy. Therefore, refined structural analysis is typically required to demonstrate system redundancy.

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

Internally redundant members are typically built-up sections, where failure of one component in a section will not result in failure of the entire section.

NSTMs are typically classified as such due to the lack of load path redundancy. If a recognized source of redundancy is demonstrated, NSTM classification may be modified and, therefore, NSTM inspection requirements would no longer apply.

IDOT expects that new bridges will be designed with no NSTMs; however, it is understood that in some instances, it is not possible or practical for all primary steel tension members to be load path redundant. For such instances, IDOT considers system redundancy as the preferred source of redundancy to avoid primary steel members to be classified as NSTMs. Internal redundancy is considered as an alternate source to be considered when system redundancy cannot be demonstrated.

3.3.4.2.2 Classification of Member Redundancy

According to NBIS Section 650.313(f)(1)(i), a State transportation department may choose to demonstrate a member has system or internal redundancy such that it is not considered a NSTM. Current classification of a NSTM may be modified if a recognized source of redundancy is demonstrated. System redundancy and internal redundancy require analysis supplemental to conventional design. Supplemental analysis shall be based on a nationally recognized method.

The following guide specifications are considered nationally recognized methods for System Redundant Member (SRM) and Internally Redundant Member (IRM) analysis.

Herein referred to as the “SRM Guide Specifications”:
AASHTO Guide Specification for Analysis and Identification of Fracture Critical Members and System Redundant Members, First Edition, 2018

Herein referred to as the “IRM Guide Specifications”:
AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members, First Edition, 2018 with 2022 Interims

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These Guide Specifications establish analytical processes and requirements, loading, and performance criteria that a member classified as a NSTM needs to meet to be re-classified as a SRM or an IRM. For a particular bridge redundancy evaluation, modifications to the approaches outlined in the *IRM Guide Specifications* or *SRM Guide Specifications*, or other alternative nationally recognized methods may be accepted by IDOT at its discretion.

Primary steel members that do not have load path redundancy, in which both system and internal redundancy are demonstrated are considered as SRMs for inspection purposes, i.e., they are not subject to the inspection requirements of IRMs. It should be noted that designers are not required nor expected to demonstrate both system redundancy and internal redundancy.

There may be cases where members may not be able to be classified as system or internally redundant. In these cases, the members will be required to be classified as NSTMs.

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3.3.5 Design of Steel Beams and Girders – Lateral Stresses

Section 6 of the AASHTO LRFD Bridge Design Specifications has "unified" the provisions for the design of straight and curved girders. The design equations which shall be satisfied, according to the code, are applicable to both types of steel bridge members. Many of the design equations contain a term for lateral bending stress, f_l. For straight or equivalently straight curved girder bridges (as per AASHTO LRFD Bridge Design Specifications Article 4.6.1.2.4b), including flare geometry, with skews ≤ 45 degrees, this stress term shall be taken as zero (0).

When the skew of a straight girder bridge or equivalently straight curved girder bridge exceeds 45 degrees but is less than or equal to 60 degrees, the factored lateral bending stress shall be taken as 10 ksi, which includes both dead load and live load. For Fatigue load combinations only live load applies, and the factored fatigue lateral stress shall be taken as 5 ksi. AASHTO LRFD Bridge Design Specifications commentary C6.10.1 states that for bridges with skews greater than 20 degrees, lateral bending stresses of 10 ksi may be assumed in lieu of performing a more rigorous analysis. The Department has performed research to determine the accuracy of this value. The research has shown that, for structures with discontinuous diaphragms that utilize Illinois diaphragm/cross-frame connection details, this factored 10 ksi value is accurate or conservative for structures with skews between 45 degrees and 60 degrees.

For curved girder bridges and straight girder bridges with skews greater than 60 degrees, the lateral bending stress is determined using a more refined (e.g. grid) analysis. Most design software will output an envelope of the most conservative values for lateral stresses. NCHRP Report 725: Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Structures may be used as a guide to help produce more accurate lateral force effects should the analysis results seem excessive. Based on this analysis, the Department's standard cross-frame/diaphragm details may be insufficient. Note that diaphragms in straight beam superstructures placed perpendicular to the girders but (staggered) along the skew, as is typical IDOT policy, are considered "continuous" by the Department. The f_l term is not zero for curved girder bridges designed according to LRFD.

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3.3.6 Design of Steel Beams and Girders – Moment Redistribution

Moment Redistribution shall not be used as a structural analysis technique for IDOT bridge design projects. Moment Redistribution is an elastic analysis technique which approximates plastic analysis for continuous steel beams and is allowed under certain conditions according to AASHTO LRFD Bridge Design Specifications Appendix B of Section 6. Generally, for sections which are compact and are sufficiently braced, a fully plastic steel failure is theoretically possible. Plastic analysis of structures is characterized by determining the locations of plastic hinges which form to create a mechanism at failure. The governing mechanism is that which takes the least amount of energy to form. The moments calculated from plastic analysis and its approximation, moment redistribution, are somewhat different than those from conventional elastic analysis, but not to a degree which has appreciable engineering significance for bridges designed according to IDOT policy. It is also Departmental policy to promote a degree of uniformity in bridge design and bridge rating procedures. The employment of moment redistribution does not achieve these aims.

3.3.7 Impact Testing Requirements

The components of main load carrying steel bridge members subject to design tensile stresses shall conform to the Charpy-V-Notch Impact Energy Requirements (Zone 2). These components, including tension flanges, webs and splice plates (excepting fills) shall be designated on the plans by “CVN”, with an explanation of these letters on the sheet. Cross-frame or diaphragm elements and their connecting plates carrying design stresses for curved structures or supporting terminating beam/girder lines shall also be designated “CVN.”

When field splices are near points of dead load contraflexure, bottom flanges on the pier (negative moment) side of the splice and top flanges on the mid-span (positive moment) side of the splice do not require CVN material, even though they may experience slight tensile stresses due to live loads.

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3.3.8 Cover Plates

Cover plates are addressed in Article 6.10.12 of the AASHTO LRFD Bridge Design Specifications. The requirements given below are standard details based on past practices, and the dimensional requirements are either identical or more conservative than those found in the AASHTO LRFD Bridge Design Specifications.

Cover plates should not be used on new bridges or superstructure replacement projects. They may be used for repair projects where cover plated rolled beams are replaced in-kind or for bridge widening jobs to match existing beams. Multiple (stacked) cover plates shall not be used at any location. The maximum thickness shall not be greater than 1 ½ times the thickness of the flange to which the cover plate is attached. (Note that this is conservative relative to the AASHTO LRFD Bridge Design Specifications.) The minimum thickness shall not be less than $\frac{1}{24}$ times the distance between edge welds measured transverse to the direction of stress or ½ in., whichever is greater.

The minimum length of any cover plate shall be twice the depth of the beam plus three (3) ft. The maximum width of any cover plate shall be the flange width minus one inch. If matching an existing condition that does not satisfy these criteria, contact the Bureau of Bridges & Structures.

The designer should be aware that the use of cover plates may not be practical under the current AASHTO LRFD Bridge Design Specifications fatigue requirements, especially for structures subjected to high cyclic stresses. In these structures, the designer should consider the possibility of replacing old members with larger beam sections in lieu of those with cover plates.

The standard end treatment for cover plates is shown in Figure 3.3.8-1.

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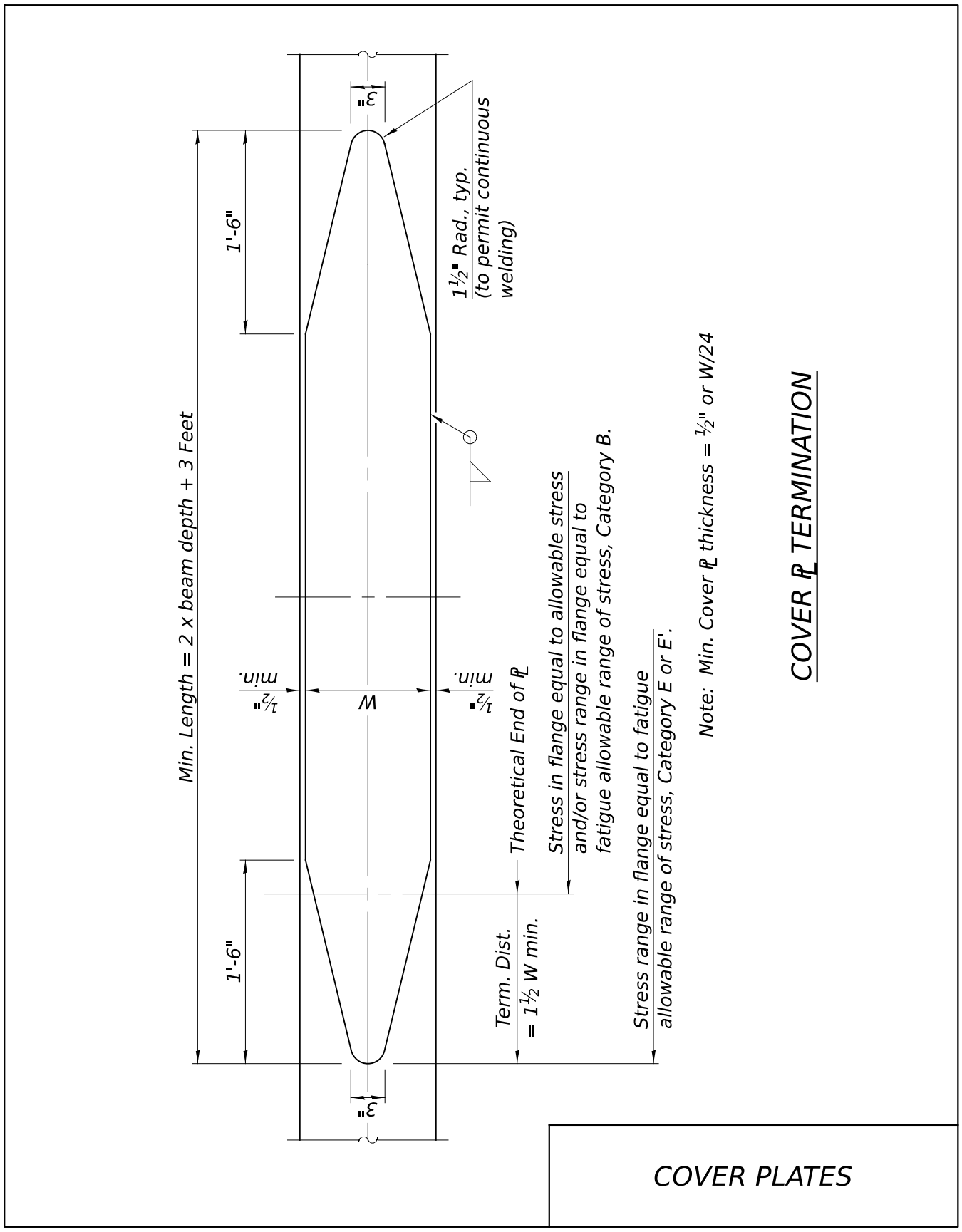


Figure 3.3.8-1

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3.3.9 Composite Beam Design - Shear Connectors

All new structures, whether they are straight, equivalently straight, or curved, shall be designed to be composite for the entire length of the structure. Shear connectors shall be used over the entire bridge length.

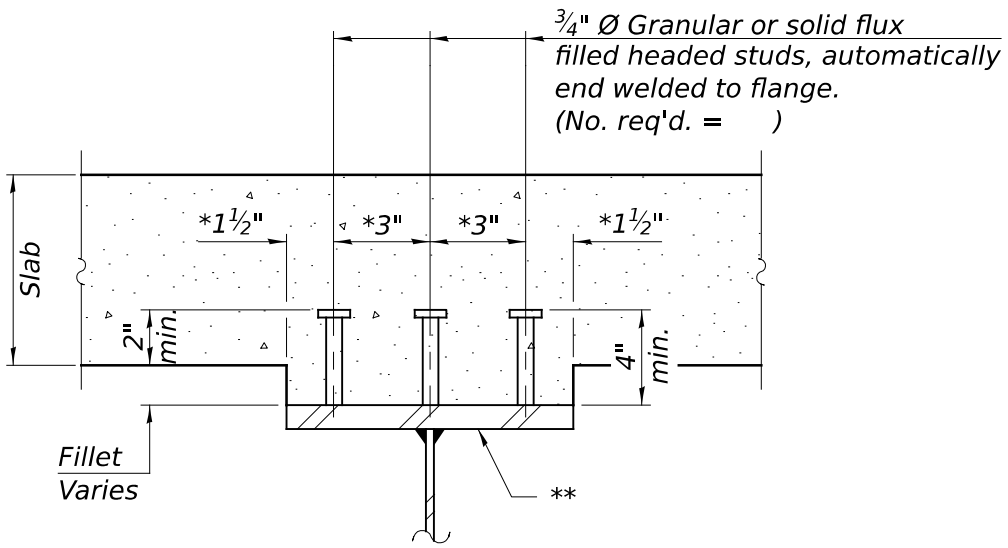
There may be cases, such as redecking or widening of existing structures, where use of a composite section for the entire length of the structure may be problematic for the existing design. If this is the case, the bridge may be designed to be noncomposite in the negative moment regions.

For the usual composite design, ¾ in. diameter stud shear connectors, a minimum of 4 in. long, shall be detailed. Alternatively, ⅞ in. diameter stud shear connectors may be justified for large girders, but these need higher output welding equipment, so approval by the Bureau of Bridges & Structures is required before specifying them. Shear reinforcement shall be provided from the slab to the fillet for deep fillet heights where the top of flange to bottom of deck exceeds 6 inches. See Figure 3.3.9-1. The top of the studs shall be a minimum of 2 in. above the bottom of the deck slab except when fillet areas have shear reinforcement. To allow concrete consolidation without bridging or voids, the minimum distance between centers of stud shear connectors shall be 4 times the nominal diameter of the stud. The distance between the edge of a girder flange and the center of the stud shall not be less than 1 ½ in. Note that these are minimums, and larger spacing is desirable for concrete consolidation. A detail similar to the one given in Figure 3.3.9-1 shall be shown on the plans.

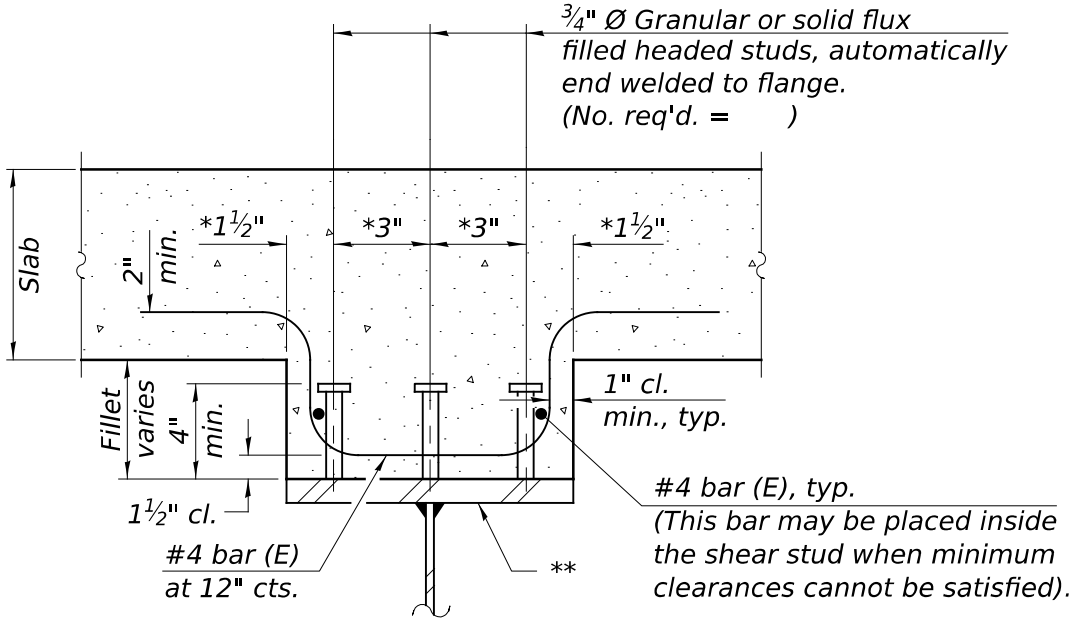
AASHTO LRFD Bridge Design Specifications Article 6.10.10 contain provisions for stud design. Also see Design Guide 3.3.9 on the Bridges & Structures page. It emphasizes the design of shear studs for straight bridges using the AASHTO LRFD Bridge Design Specifications (which has been unified for both straight and curved girder stud design). Note that the definitions for diaphragm continuity and severity of skew outlined in Section 3.3.5 also apply to stud design.

For composite plate girders, the minimum top flange plate width shall be 12 in. and the minimum thickness shall be ¾ in.

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STANDARD FILLET SECTION



DEEP FILLET SECTION

* Minimum

** For plate girders, the minimum flange width is 12 in., so lateral spacings and edge distances should increase (2 in., 4 in., 4 in., 2 in.).
Although 4- ¾" Ø studs could be placed in one row on a 12 in. flange, this should be avoided to facilitate concrete consolidation and to more uniformly distribute studs along the flange. Four studs per row may be appropriate for flanges 16 in. and wider.

TYPICAL DETAILS OF SHEAR CONNECTORS

Figure 3.3.9-1

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3.3.10 Diaphragms and Cross-Frames

Steel Dead Load Fit (SDLF) shall be used whenever applicable for diaphragm and cross-frame design. SDLF is used for all of IDOT's standard cross-frame designs, and these cross-frames are adequate for all straight structures.

The diaphragm placement for curved member bridges becomes more complex as curvature and skew of the supports increase. The Department has developed policies to follow for diaphragm design on curved bridges.

1. For curved bridges which are equivalently straight (as per AASHTO LRFD Bridge Design Specifications Article 4.6.1.2.4b), diaphragms shall be designed and detailed according to Sections 3.3.22 and 3.3.23 (regardless o?" skew).

2. When a curved member bridge is not equivalently straight and the supports are within 20 degrees of radial (≤ 20°), nearby bracing may be placed parallel (not staggered) to the support. Between supports, bracing may remain skewed, become radial, or transition to the next support's skew, depending upon the distance between supports.

3. If supports skew more than 20 degrees from radial and the bridge is not equivalently straight (as per AASHTO LRFD Bridge Design Specifications Article 4.6.1.2.4b); Section 6 of the AASHTO LRFD Bridge Design Specifications and/or the AASHTO/NSBA "Guidelines for Design for Constructability" should be referenced for guidance on placement of bracing.

Diaphragms and cross-frames which are primary members shall have all elements, including connection plates or stiffener plates, so designated and shall use Charpy V-Notch (CVN) material.

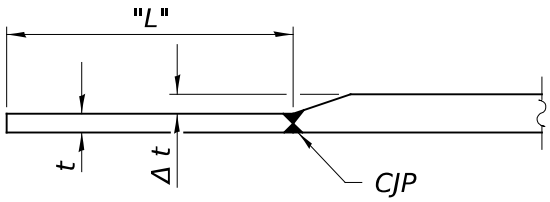
Connection details for bracing on curved structures are addressed in Section 3.3.24.

3.3.11 Welded Girder Flange Transitions

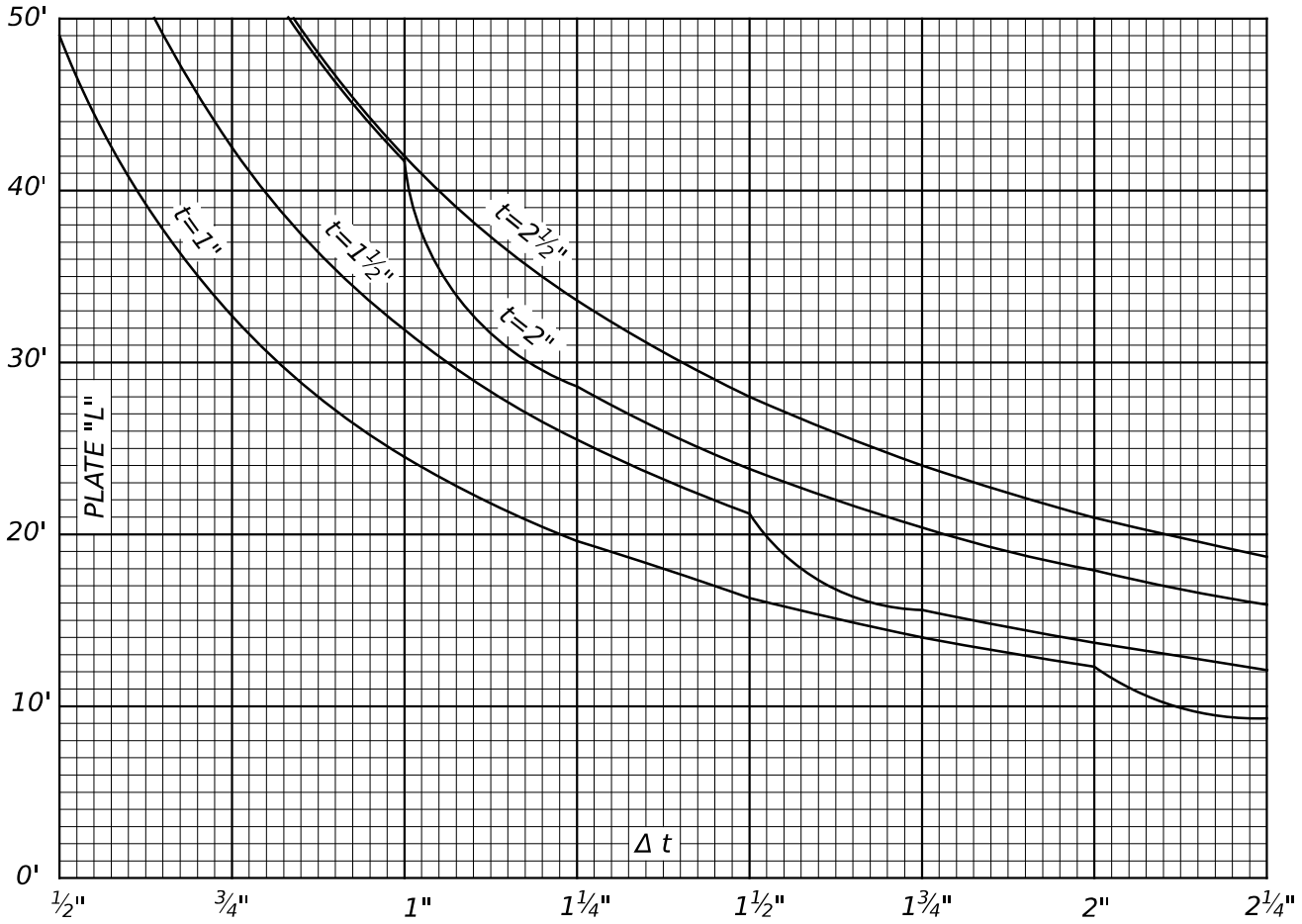
Figures 3.3.11-1 through 3.3.11-4 shall be used to estimate whether a reduction in plate thickness justifies the cost of the butt weld.

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JOINT WIDTH = 12"



"L" = Minimum length of plate with reduction in thickness of Δ t required to justify the cost of the butt weld.

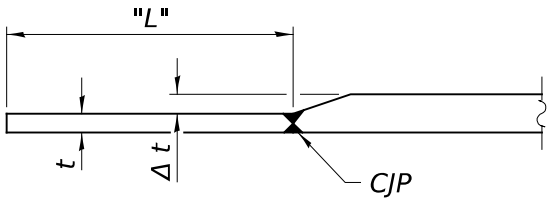


FLANGE PLATE
BUTT WELDS

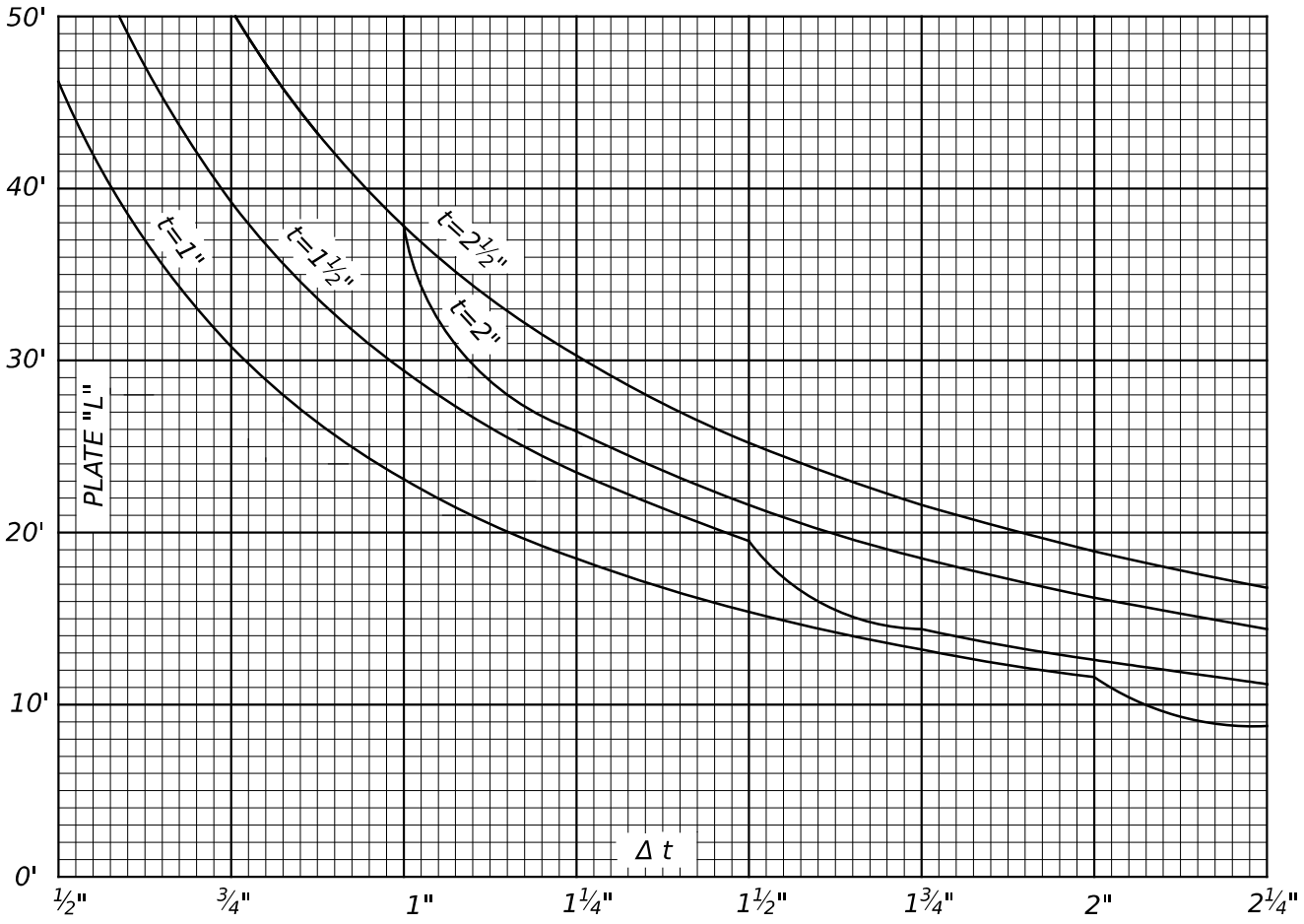
Figure 3.3.11-1

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JOINT WIDTH = 14"



"L" = Minimum length of plate with reduction in thickness of Δ t required to justify the cost of the butt weld.



FLANGE PLATE
BUTT WELDS

Figure 3.3.11-2

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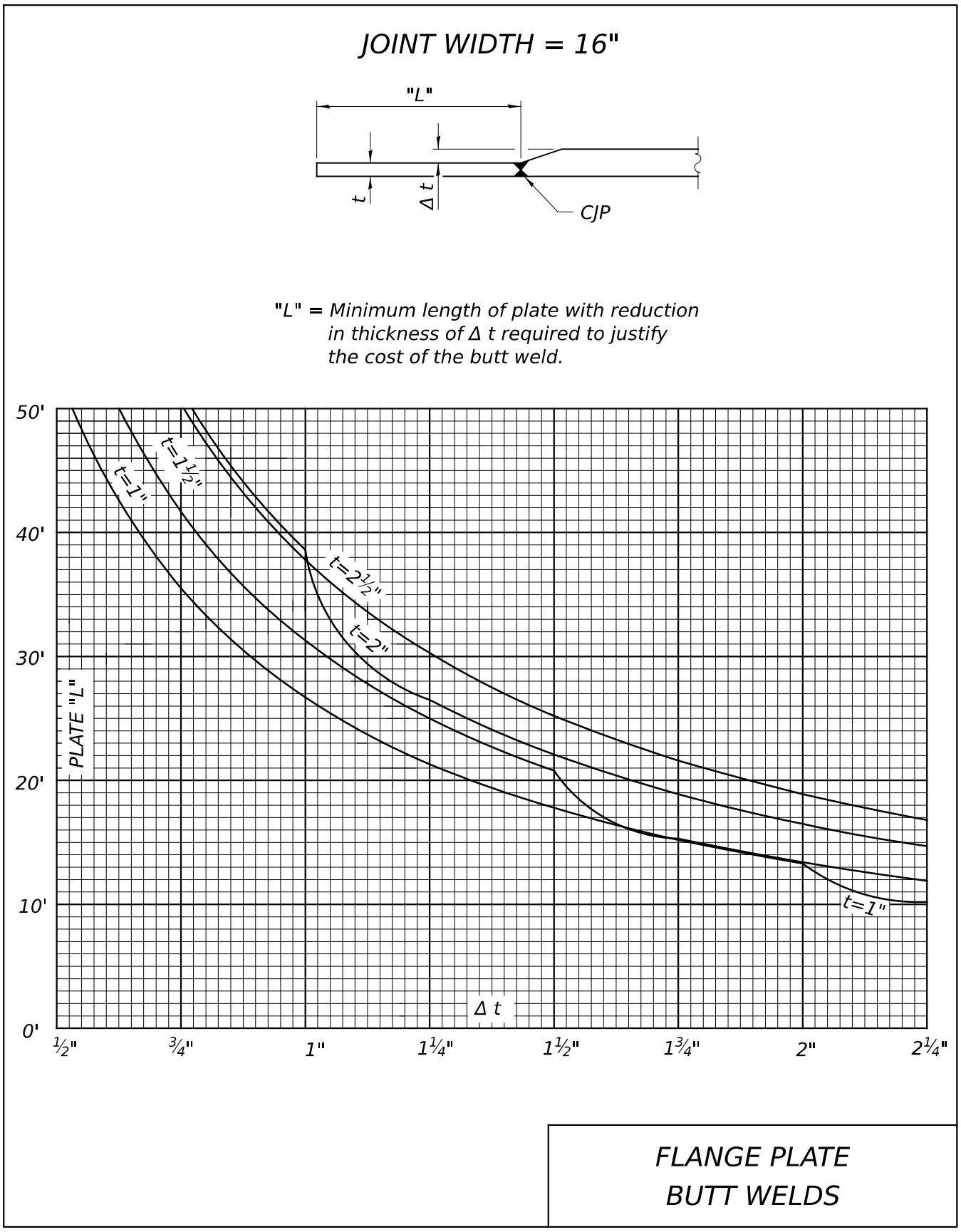


Figure 3.3.11-3

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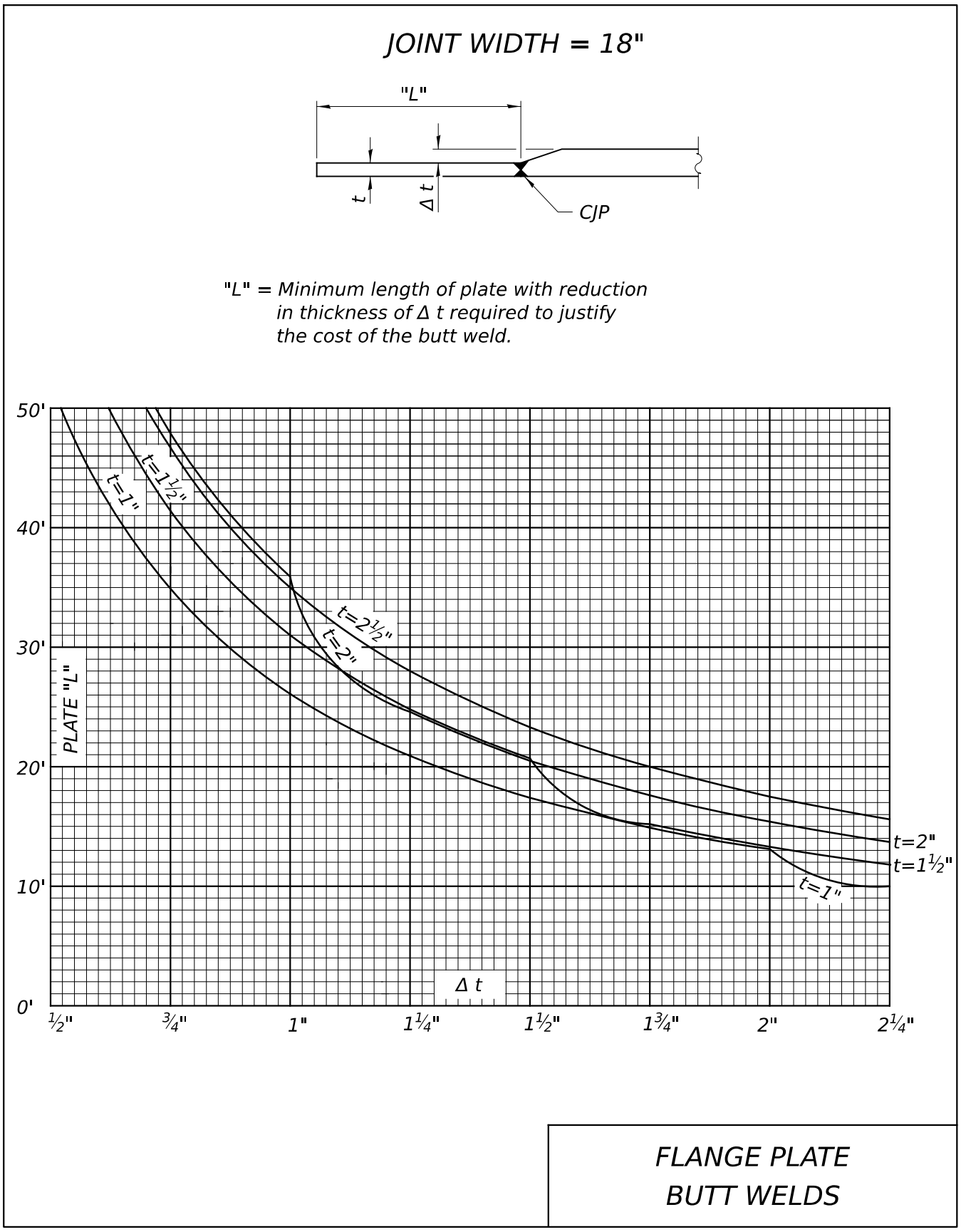


Figure 3.3.11-4

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The thicker plate in the transition shall be limited to approximately twice the thickness of the thinner plate. The flange width between bolted splices should be kept constant for economical fabrication, unless unusual conditions (span ratios, terminating girders, etc.) necessitate a width transition.

Butt welded thickness transitions are to be ground smooth and subject to the applicable fatigue allowable range of stresses for Category B.

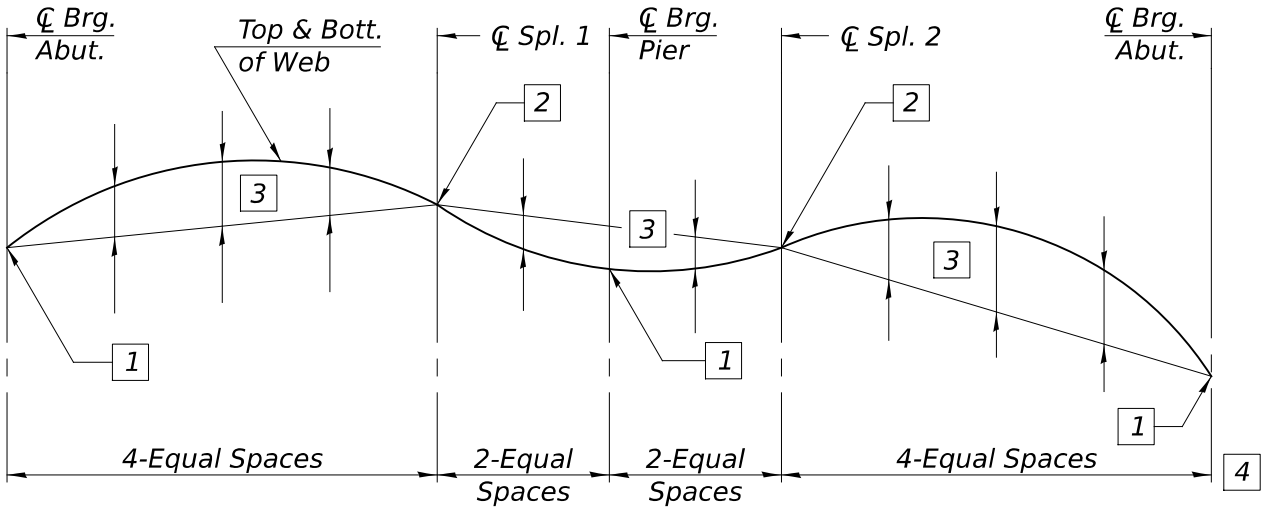
The National Steel Bridge Alliance (NSBA) publication "Guidelines for Design for Constructability" (https://www.aisc.org/nsba/) provides guidance on maximum plate length which also affects butt joint placement.

3.3.12 Camber

Rolled beams shall not be designed with camber unless prior approval is obtained from the Bureau of Bridges & Structures for unusual applications such as a significant vertical curve, clearance problems, architectural requirements, etc. "Hot cambering" (by heat patterns), "cold cambering" (by jacking), or "roll cambering" (by passing between offset rollers) causes significant residual stresses, may negatively affect toughness or ductility, and may be partially lost due to gradual relaxation of internal stresses.

Plate girder webs are typically cambered to reduce the concrete fillet or haunch (the concrete between the top flange and bottom of the structural deck). See Figure 3.2.4-13. Calculated deflection due to the weight of the deck slab and the steel, and the vertical curve are used in computing the camber. The camber shown on the plans shall be the total before any dead load deflection. Fabricators use this to cut webs and verify dimensions with the girders in a "no-load" condition (fully supported with the web horizontal or vertical). Shown below in Figure 3.3.12-1, for guidance, is a schematic of a camber diagram. See also Design Guide 3.3.12 on the Bridges & Structures page.

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CAMBER DIAGRAM

1 *Final top of web elevations to be used by the designer in computing the bearing seat elevations.*

2 *Theoretical elevations before dead load deflection to be determined by the designer.*

3 *The fabrication tolerance for camber is ±¾ in., so if the maximum calculated camber ordinate in a segment is less than ¾ in., the segment is shown on the design as straight (no camber) and cambers for adjacent segments are adjusted accordingly.*

4 *The number of spaces shown is typical for a symmetric, 2-span structure. Curves should be approximately parabolic or circular and avoid kinks or sudden changes in slope. If curvature must change (e.g. become tangent), vertical offsets should be added as needed to define the geometry.*

Figure 3.3.12-1

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3.3.13 Fillet Welds

Per Section 2 of the AASHTO/AWS D1.5 Bridge Welding Code, the minimum size of fillet welds shall be ¼ in. when the thicker plate joined is ¾ in. or less, and $\frac{5}{16}$ in. when the thicker plate exceeds ¾ in. Fillet sizes should increase based on calculated stress requirements, but oversized welds are detrimental. Fillet welds are on both sides of the element joined (stiffener, connection plate) to prevent tension at the root of the weld which is possible with a single sided weld.

3.3.14 Intermediate Vertical Stiffeners

Intermediate vertical stiffeners (also called transverse stiffeners) shall be $\frac{7}{16}$ in. minimum thickness and fillet welded to one side of the web. Intermediate stiffeners not also functioning as bracing connection plates shall be welded to the compression flange, and stopped short of the tension flange. The distance between the end of the stiffeners and the near face of the tension flange shall be a constant dimension of from 4 to 6 times the web thickness plus the flange to web weld size and rounded to the nearest ½ in. When also acting as connecting plates for cross-frames or diaphragms, intermediate vertical stiffeners shall be fillet welded to both flanges and the tension flange stress range shall be investigated for fatigue Category C' in LRFD. If the fatigue allowables are exceeded, either increase the flange thickness or, if only one or two locations exceed it, consider bolted connections at those locations only.

The stiffener plates at the junction of the flanges and the web shall be clipped per Figure 3.3.22-4.

For girders with web depths equal to or smaller than 54 in., avoid transverse stiffeners in addition to cross-frame connection plates. For webs deeper than 54 in., the web thickness may be increased to require only one or two vertical stiffeners per girder segment (between field splices) beyond the cross-frame connection plates.

3.3.15 Cross-Frame and Diaphragm Connection Plates

Connection plates shall be a minimum of $\frac{7}{16}$ in. thick and fillet welded to the web and both flanges. The tension flanges shall be investigated for fatigue (See Section 3.3.14).

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3.3.16 Bearing Stiffeners

Bearing stiffeners are required at beam ends near supports with joints for all steel beam bridges (wide flange and plate girders) and shall be a minimum of 1 in. thick. They shall be placed along the skew for skew angles less than or equal to 20 degrees. Bearing stiffeners shall be finished to bear (ground, milled, etc.) on the bearing end and have a tight fit at the other end. The bearing stiffener plates at the junction of the flanges and the web shall be clipped per Section 3.3.14. They shall be fillet welded to both flanges when used as connecting plates for cross-frames or diaphragms. Complete joint ("full") penetration welds shall not be specified between the stiffener and flange without approval of the Bureau of Bridges & Structures.

3.3.17 Structural Steel Framing

A "concrete fillet" is the concrete between the top of the flange and the bottom of the structural deck. In order to reduce the fillet on steel structures, girders may be cambered and the beam or girder slope may be changed at splices so the top of the top flange stays relatively close to the bottom of the formed deck slab.

On the structural steel sheet, a table showing fabricated top of beam elevations for rolled beams or top of web elevations for plate girders shall be provided. The elevations are prior to steel and concrete induced deflections and shall be given for all beams or girders at their supports and field splices. The table shall be noted as "For Fabrication Only".

For steel structures on a horizontal curve, the uniqueness of the framing plan layout necessitates the inclusion of additional data on the structural steel sheets to facilitate fabrication and erection.

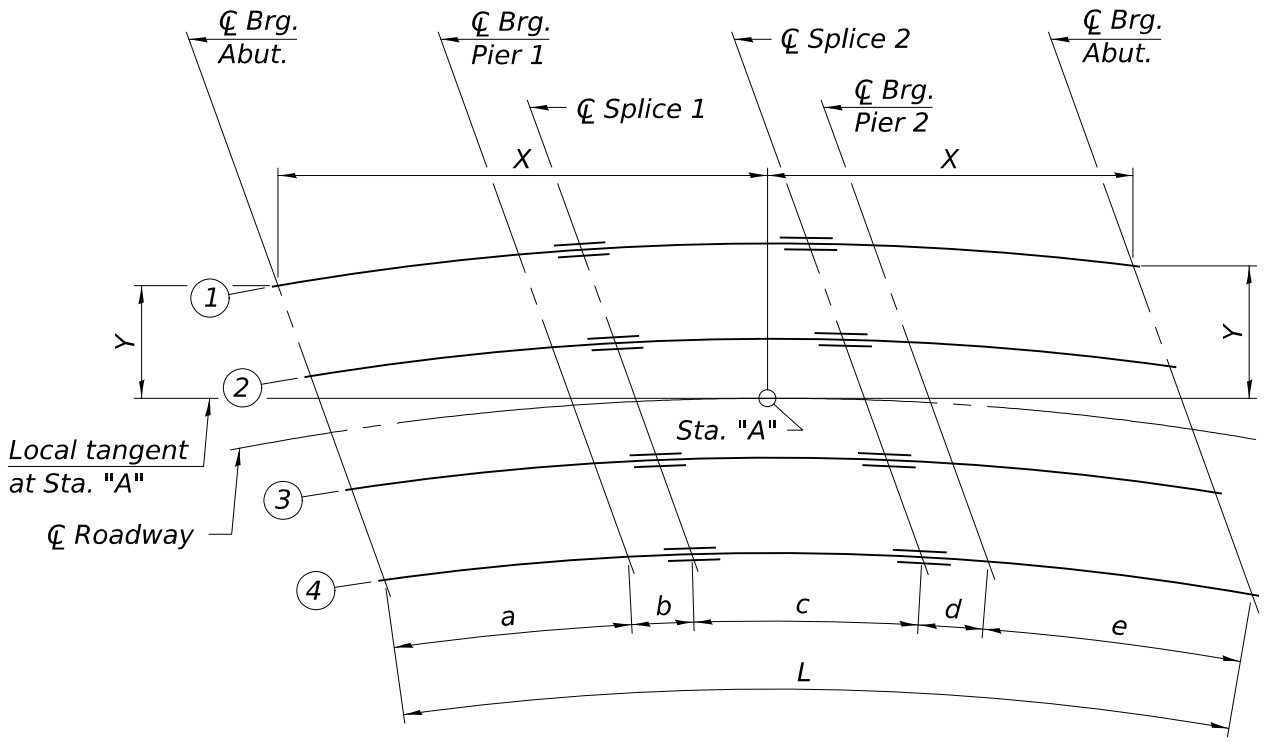
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The following data are considered essential. However, additional data may be provided at the discretion of the designer:

1. The radius of each individual beam or girder line.
2. The length of each individual beam or girder between bearings and splices measured along centerline beam or girder.
3. The total length of each individual beam or girder measured along centerline of beam or girder.
4. The lateral and longitudinal offset at all bearings and splices for each beam measured with reference to the local tangent of the structure.

Shown in Figure 3.3.17-1, for guidance, is a typical example of a proper presentation of the above data.

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BEAM DIMENSIONS (in feet)

Beam	Radius	a	b	c	d	e	L
1							
2							
3							
4							

LAYOUT DIMENSIONS (in feet)

Beam	Abutment		Pier 1		Spl. 1		Spl. 2		Pier 2		Abutment	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
1												
2												
3												
4												

CURVED GIRDER LAYOUT

Figure 3.3.17-1

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3.3.18 Minimum Concrete Fillet Heights

1. Wide Flange Beams:
   Initial top of beam elevations for fabrication shall be established so that after deck placement, all locations on the top of beams, cover plates, and/or bolted splices shall have a positive fillet height of ½ in. or more between the top of steel and deck. Variations due to erection, mill camber, etc. should not result in a "negative fillet" with the flange, cover plate, and/or splice plate entering the deck.

2. Plate Girders
   Initial top of web elevations for fabrication shall be established so that after the deck placement, all locations on the top of girders and/or bolted splices shall have a positive fillet height of at least ¾ in. between the top of steel and deck. Variations due to erection, camber tolerance, etc. should not result in a "negative fillet" with the flange or splice plate entering the deck.

Positive fillets avoid embedment of the top flange or splice into the design structural depth of the deck slab. The minimum fillets for WF beams reflect bearing seat tolerances and "natural camber" in as-received beams. The minimum fillets for plate girders reflect a fabrication camber tolerance of ¾ in. plus or minus. (Note: this is different than the +1 ½ in., -0 in. range permitted by the AASHTO/AWS D1.5 Bridge Welding Code.) See Section 3.3.12 and Figure 3.2.4-13.

3.3.19 Lateral Bracing

The need for lateral bracing in each span shall be investigated in accordance with Article 6.7.5 of the AASHTO LRFD Bridge Design Specifications. On continuous spans, only those spans that require lateral bracing shall have it provided.

Designers shall investigate the need for lateral bracing to prevent overstressing structural members, excessive lateral deflection, and local (girder) or global (system) instability, due to lateral forces on the structure at applicable stages of construction. These stages include: fully-erected steel girders without deck, during placement of concrete deck, and final in-service condition. Temporary or permanent lateral bracing may be necessary for long-span structures or shorter span structures under stage construction.

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3.3.20 Longitudinal Stiffeners

Longitudinal stiffeners may be justified for deep girders, especially at haunches, to avoid very thick webs and/or closely spaced transverse stiffeners. They should end in areas under compression for all anticipated cyclic loading conditions. If they terminate in an area subject to tension, fatigue Category E' or E stress range limits shall be satisfied according to the AASHTO LRFD Bridge Design Specifications. Changing fillet welds connecting a longitudinal stiffener and web into a complete joint penetration weld for the last 6 to 12 inches and radiusing the stiffener end tangent to the face of the web may avoid Cat. E' / E, but this can be expensive and imparts significant residual stresses, so it is not permitted without prior approval by the Bureau of Bridges & Structures.

When possible (e.g. fascia girders), longitudinal stiffeners should be on the opposite side of the web from transverse stiffeners and cross-frame connection plates. If they intersect, the longitudinal stiffeners should be continuous and the transverse stiffeners or connection plates should be fillet welded to both sides of the previously installed longitudinal stiffener in a similar manner to their flange connections.

3.3.21 Bolted Field Splices

Field splices in multi-span continuous structures are generally located near the point of dead load contraflexure (± 5% of span length). (See Section 3.3.9 for guidance on stud shear connector placement.) Splices shall be designed according to Article 6.13 of the AASHTO LRFD Bridge Design Specifications. See Design Guide 3.3.21 on the primary Bridges & Structures Page. When the section changes at the splice location, the splice shall be based upon the smaller section. The connection shall be symmetrical (except for filler plates).

When filler plates are required to connect different size components, they shall not extend beyond the splice plates. If ¼ in. or thicker filler plates are required, a reduction factor shall be applied to the design shear strength of the fasteners passing through the filler, according to AASHTO LRFD Bridge Design Specifications Article 6.13.6.1.4. Compare the fasteners required on the side of the connection with a fill plate to that required by the other side. Both sides shall use the same number of fasteners to ensure a symmetrical splice.

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The following maximum lengths shall be used in determining field splice locations. If greater lengths are considered, the engineer shall investigate transportation and erection feasibility and receive approval from the Engineer of Bridges & Structures before designing the member.

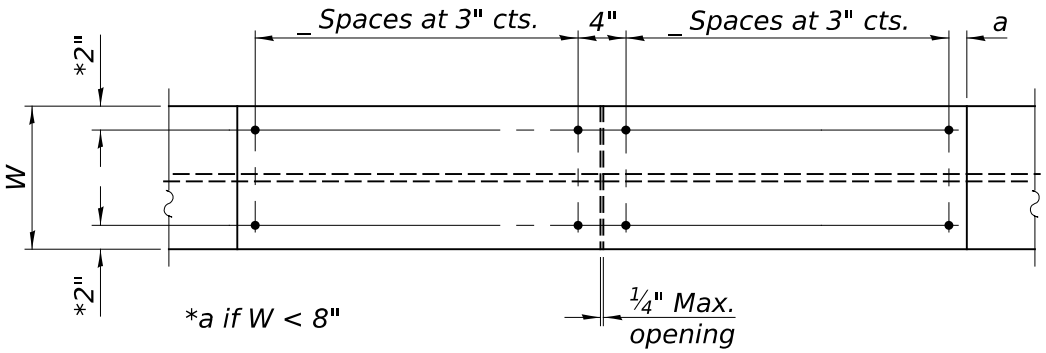
Plate Girders	135 Ft.
W 36 and W 33	90 Ft.
W 30, W 27 and W 24	80 Ft.
W 21 and W 18	70 Ft.

Special considerations should be given to shallow (less than 48 in. web depth), haunched, curved or highly cambered plate girders.

All splice bolts should be, where practical, ASTM F3125 Grade A325 ⅞ in. ϕ with standard size holes for slip-critical connections. For slip resistance, reduction in slip capacity shall follow Section 6.13.2.8 of the AASHTO LRFD Bridge Design Specifications for the given faying surface (e.g. galvanized, metallized, etc.). Interior flange splice plates should be used when geometrically possible to reduce the amount of beam length with shear studs omitted. Note that for beams with narrow flanges, inside plates may not be possible due to the flange being too narrow to accommodate two rows of bolts per side of flange. See Figures 3.3.21-1 through 3.3.21-3 for more details.

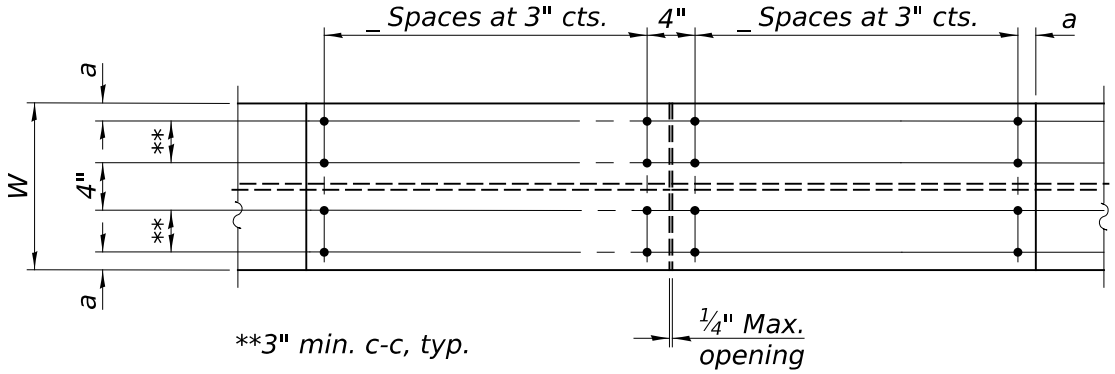
Minimum splice plate thicknesses are ⅜ in. for webs and ½ in. for flanges. For flange splices with inside plates, the two inside plates should optimally use the same material thicknesses as the outside plate while still keeping the areas of the inner and outer plates within 10% of each other. This helps locate the combined centroid of the splice plates near mid-depth of the flange and provides for equal distribution of design forces to the inner and outer plates. Also, use commonly available plate thicknesses, including ⅜, ½, ⅝, ¾, ⅞, 1, 1 ¼, and 1 ½ in. When plate girders require a small number of splices, web and flange thicknesses should be specified to simplify material acquisition. Figures 3.3.21-1 through 3.3.21-3 provides additional geometric detailing guidelines and recommendations for design of splices. See also Design Guide 3.3.21 on the primary Bridges & Structures Page which includes a composite splice design example.

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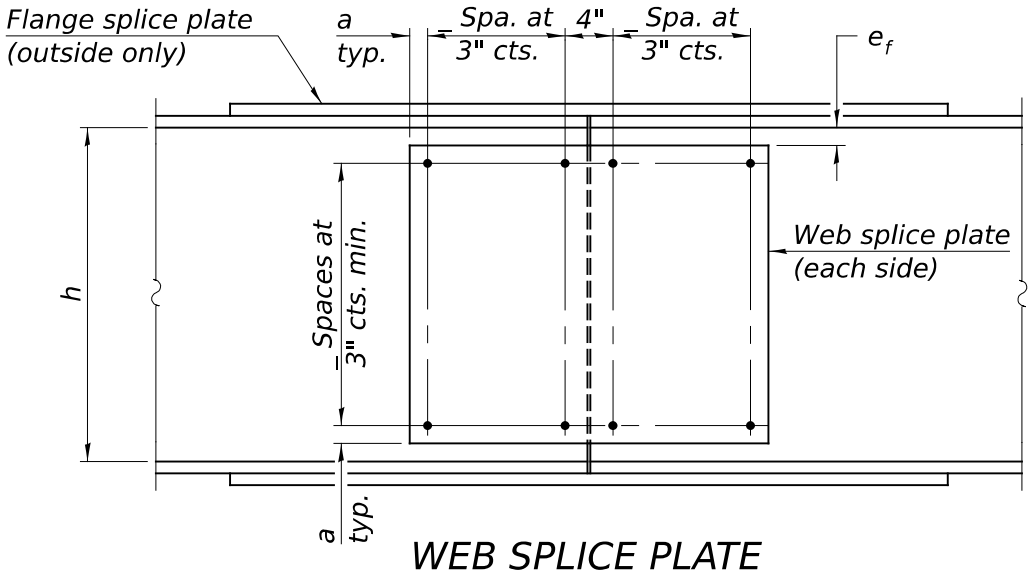
FLANGE SPLICE PLATE

(W ≤ 13")



FLANGE SPLICE PLATE

(W > 13")



WEB SPLICE PLATE

Notes:

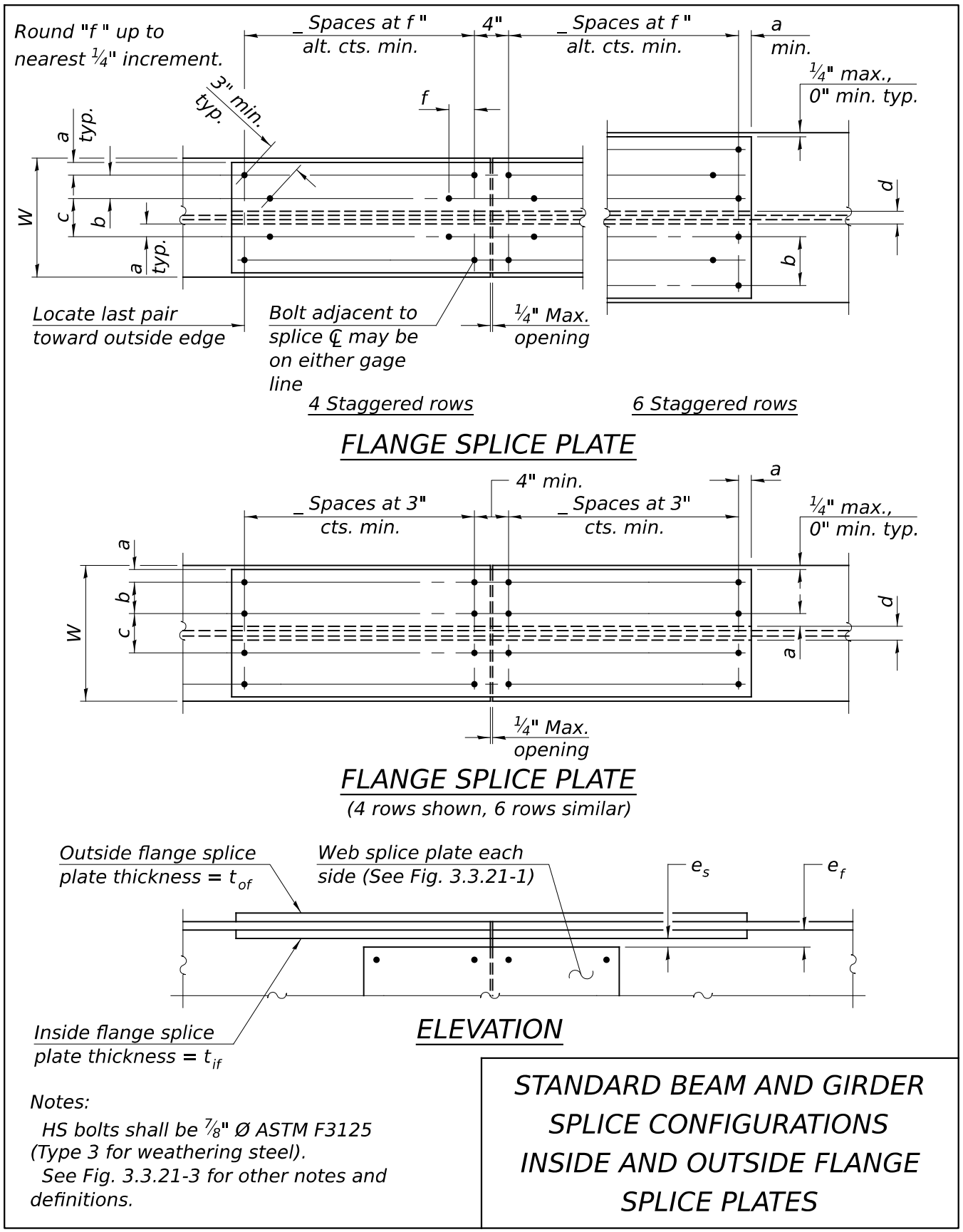
HS bolts shall be ⅞" Ø ASTM F3125 (Type 3 for weathering steel).

See Fig. 3.3.21-3 for other notes and definitions.

STANDARD BEAM AND GIRDER SPLICE CONFIGURATIONS OUTSIDE FLANGE SPLICE PLATES

Figure 3.3.21-1

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Notes:

HS bolts shall be ⅞" Ø ASTM F3125 (Type 3 for weathering steel).

See Fig. 3.3.21-3 for other notes and definitions.

STANDARD BEAM AND GIRDER SPLICE CONFIGURATIONS INSIDE AND OUTSIDE FLANGE SPLICE PLATES

Figure 3.3.21-2

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General Recommendations:
a = edge/end distance: 1¾" recommended, 1½" min.

b:
b < 2": Use 2 parallel rows w/o stagger. (Splices without inside flange PL's only)
2" ≤ b < 3": Use 4 staggered rows
3" ≤ b < 4½": Use 4 parallel rows
4½" ≤ b < 6": Use 6 staggered rows
6" ≤ b: Use 6 parallel rows

c:
5" for PL girders with webs up to ⅝" and flange-web welds up to ⅜"
6" for W-beams with $k_1 \leq 1\frac{1}{8}$" (increase for other cases)
For weathering steel bridges, this dimension may be increased to allow water to flow and reduce pack rust.

d:
(for W beams): $d \geq 2k_1 + \frac{1}{4}$"
(for plate girders): d ≥ web + 2 welds + ¼"

e:
(for W beams): $e_f \geq$ rolled radius + ¼" & $e_f \geq 2 * t_{if} + \frac{1}{4}$" & $e_s \geq t_{if} + \frac{1}{4}$"
(for plate girders): $e_f \geq 1$" & $e_f \geq 2 * t_{if} + \frac{1}{4}$" & $e_s > t_{if} + \frac{1}{4}$"

For flanges ≥ 26" wide, design varies.

$t_{if} \geq t_{of}$: Consider specifying same thickness as other CVN material in project.

Notes:
All splices are symmetrical about ℄ splice except for fills.

STANDARD BEAM AND GIRDER SPLICE CONFIGURATION NOTES AND DEFINITIONS

Figure 3.3.21-3

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3.3.22 Interior Diaphragms and Cross-Frames

Diaphragm or cross-frame bracing shall be placed at or near each support and throughout the span at 25 ft. – 0 in. maximum centers. This is due to constructability, re-constructability, stability concerns, etc. Optimally, place bracing near the maximum positive moment location in a span as well as near field splices (between the splice and the pier) without increasing the total number. At most end supports (abutments with joints, non-continuous piers, seated connections, and pin and link connections), end diaphragms or cross-frames shall be used. See Section 3.3.23. For all steel beams and plate girders on integral and semi-integral abutments, the first interior cross-frame/diaphragm shall be placed 2 ft. into the span from the exterior face of the concrete diaphragm to ensure stability during construction. End cross-frames shall not be used at integral or semi-integral abutments. For skews greater than 20 degrees, intermediate bracing shall be perpendicular to main members. For 20 degrees and smaller skews, bracing shall be placed along the skew line. See Figures 3.3.22-1 and 3.3.22-2 for additional details. Where two adjacent members are not parallel, the bracing shall be shown perpendicular to one of the members.

Special consideration shall be given to the connections between floor beams and main girders to prevent fatigue cracking due to out-of-plane stresses, as described in Article 6.6.1.3 of the AASHTO LRFD Bridge Design Specifications .

On rolled beam spans or for welded plate girders with web depths of less than 40 in., diaphragms shall be used unless curvature effects require cross-frames. The interior diaphragm detail shown in Figures 3.3.22-3 and 3.3.22-4 shall be used these beam types. This connection detail is considered satisfactory for AASHTO LRFD Bridge Design Specifications Article 6.6.1.3.1 and further analysis is unnecessary.

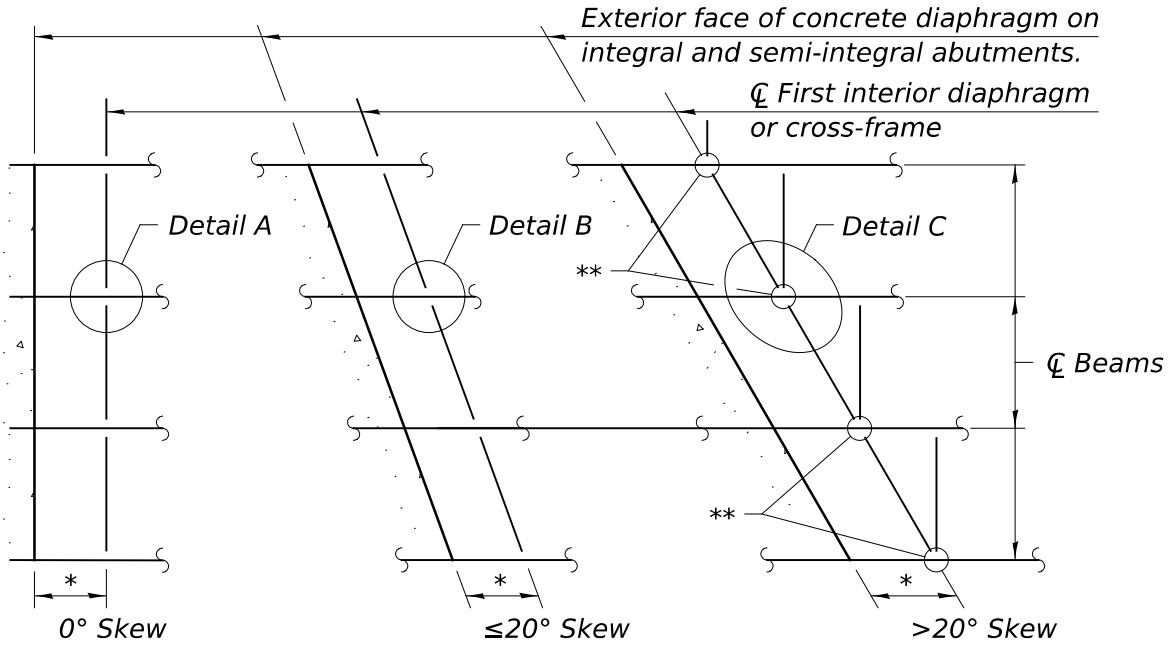
On plate girders with a web depth greater than or equal to 40 in., cross-frames shall be used. Figure 3.3.22-5 illustrates the details of a cross-frame for girders with or without transverse intermediate stiffeners.

The cross-frame and diaphragm details shown in Figures 3.3.22-3, 3.3.22-4, and 3.3.22-5 are generally adequate for girders designed using a line-girder analysis.

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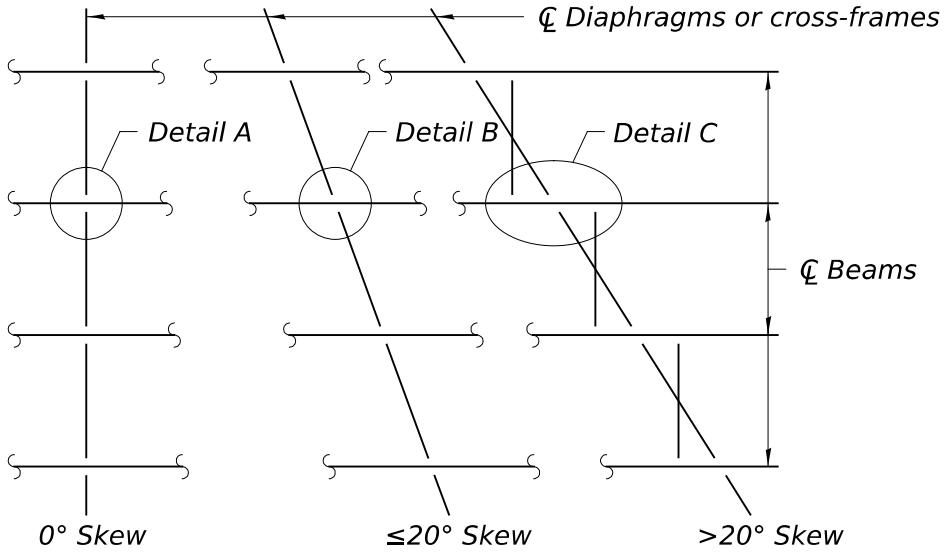
Bracing on girders designed using a grid analysis (significantly curved structures, structures with spans exceeding 240 ft., or structures with skews exceeding 60 degrees) warrant special design as primary members and utilize CVN material. These cross-frames shall include top chords in addition to the diagonals and bottom chords. Girders designed for curvature shall be braced at or within one flange width of its support to prevent lateral twisting of the girder and transverse bearing rotation. Further guidance on the design of bracing for curved girder structures can be found in the code and commentary of Section 6 in the AASHTO LRFD Bridge Design Specifications.

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PARTIAL FRAMING PLAN
(Adjacent to Abutments)

* 2 ft. into span from exterior face of concrete diaphragm.
** 2 ft. measured to the left or right workpoint nearest the concrete diaphragm face.



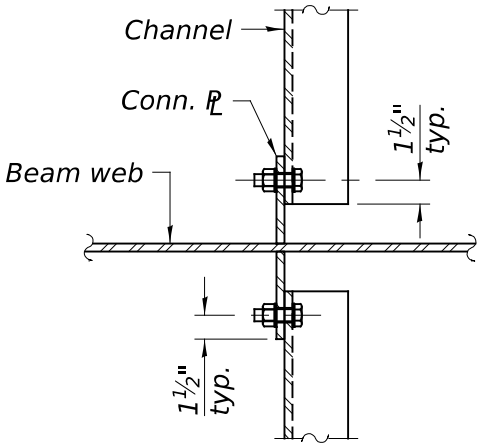
PARTIAL FRAMING PLAN
(Within Span)

Note:
For Details A, B, and C, see Figure 3.3.22-2.

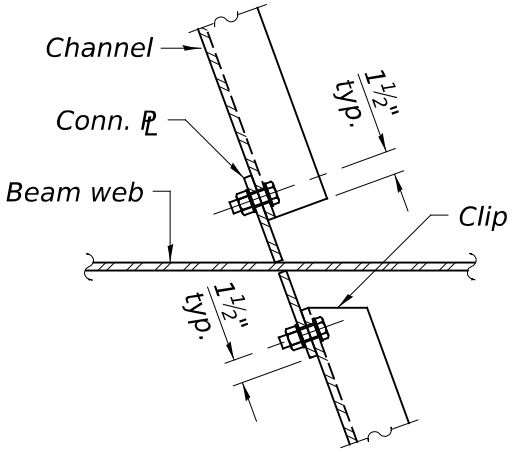
INTERIOR DIAPHRAGM/ CROSS-FRAME PARTIAL FRAMING PLAN

Figure 3.3.22-1

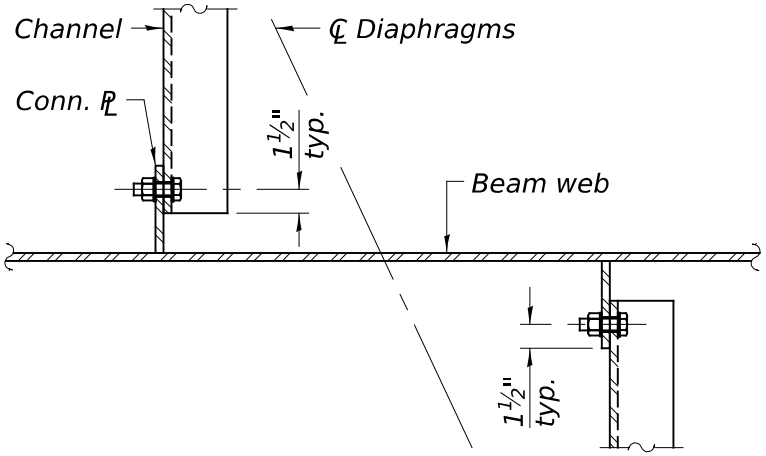
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DETAIL A
(0° Skew)



DETAIL B
(≤20° Skew)



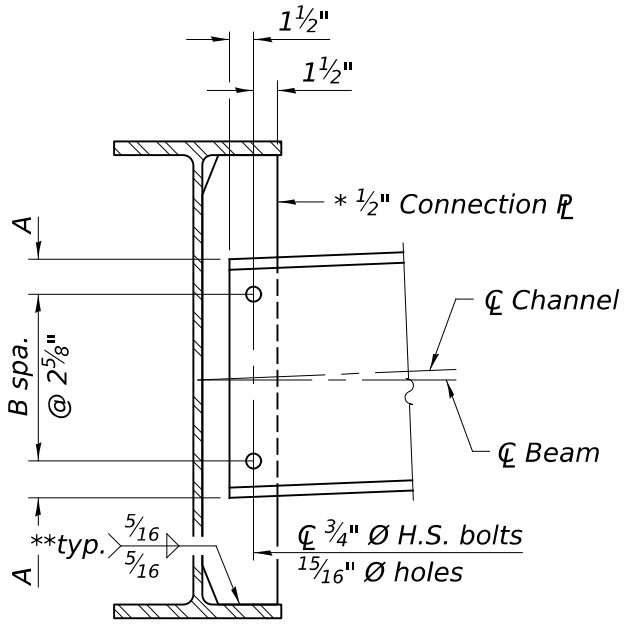
DETAIL C
(>20° Skew)

Notes:
Details A, B and C illustrate channel diaphragms. Cross-frame details similar.
Clip channel as necessary for ease of installation of diaphragms on skews.

INTERIOR DIAPHRAGM/ CROSS-FRAME FRAMING DETAILS

Figure 3.3.22-2

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WIDE FLANGE BEAMS

INTERIOR DIAPHRAGMS

(Wide flange beam shown, PL girder with web depths < 40" similar)

Beam Size	PL Girder Web	Channel	A	B
W21 to W24	< 24"	C12 x 25	±2 1/16"	3
W27 to W30	≥ 24", < 30"	C15 x 40	±2 1/4"	4
W33 to W36	≥ 30", < 40"	MC18 x 42.7	±2 7/16"	5

Notes:

Two hardened washers required for each set of oversized holes.

Alternate channels of equal depth and larger weight are permitted to facilitate material acquisition. Alternate channels, if utilized, shall be provided at no additional cost to the Department.

See Figure 3.3.22-1 and Figure 3.3.22-2 for connection PL orientation.

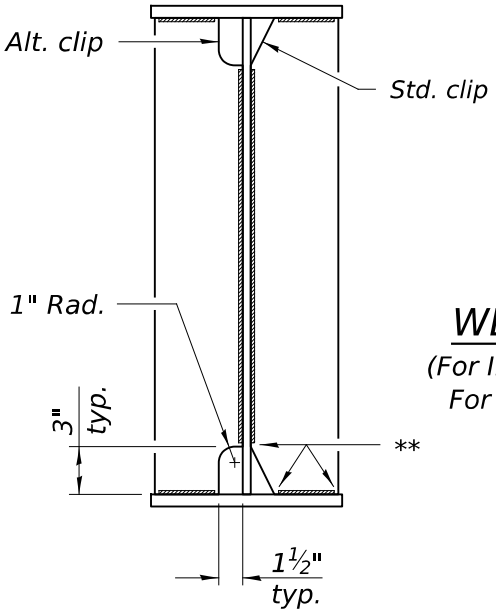
* *May also function as transverse stiffener or bearing stiffener. In these cases, PL thickness and weld size by design.*

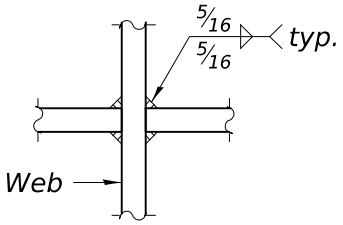
** *3 sides of each stiffener and/or connection PL.*

INTERIOR DIAPHRAGM CONNECTION DETAILS

Figure 3.3.22-3

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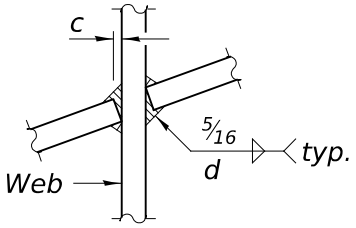


WEB WELD DETAIL (PERPENDICULAR)

(For Interior Diaphragm / Cross-Frames 0° & >20° skew. For End Diaphragm / Cross-Frames 0° & >20° skew.)

WELD LIMITS AND CLIP DETAILS

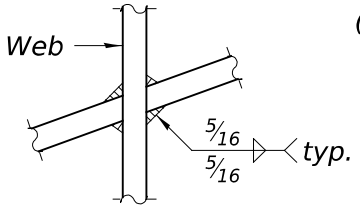
** Stop welds ¼" (±⅛") from edges as shown. Typical.



WEB WELD DETAIL (SKEWED)

$d = \frac{5}{16} + c$

(For Interior Diaphragm / Cross-Frames ≤20° skew. For End Diaphragm / Cross-Frames ≤20° skew.)



ALTERNATE WEB WELD DETAIL (SKEWED)

(For Interior Diaphragm / Cross-Frames ≤20° skew. For End Diaphragm / Cross-Frames ≤20° skew. Bevel connection ℄ or stiffener to allow for smaller weld.)

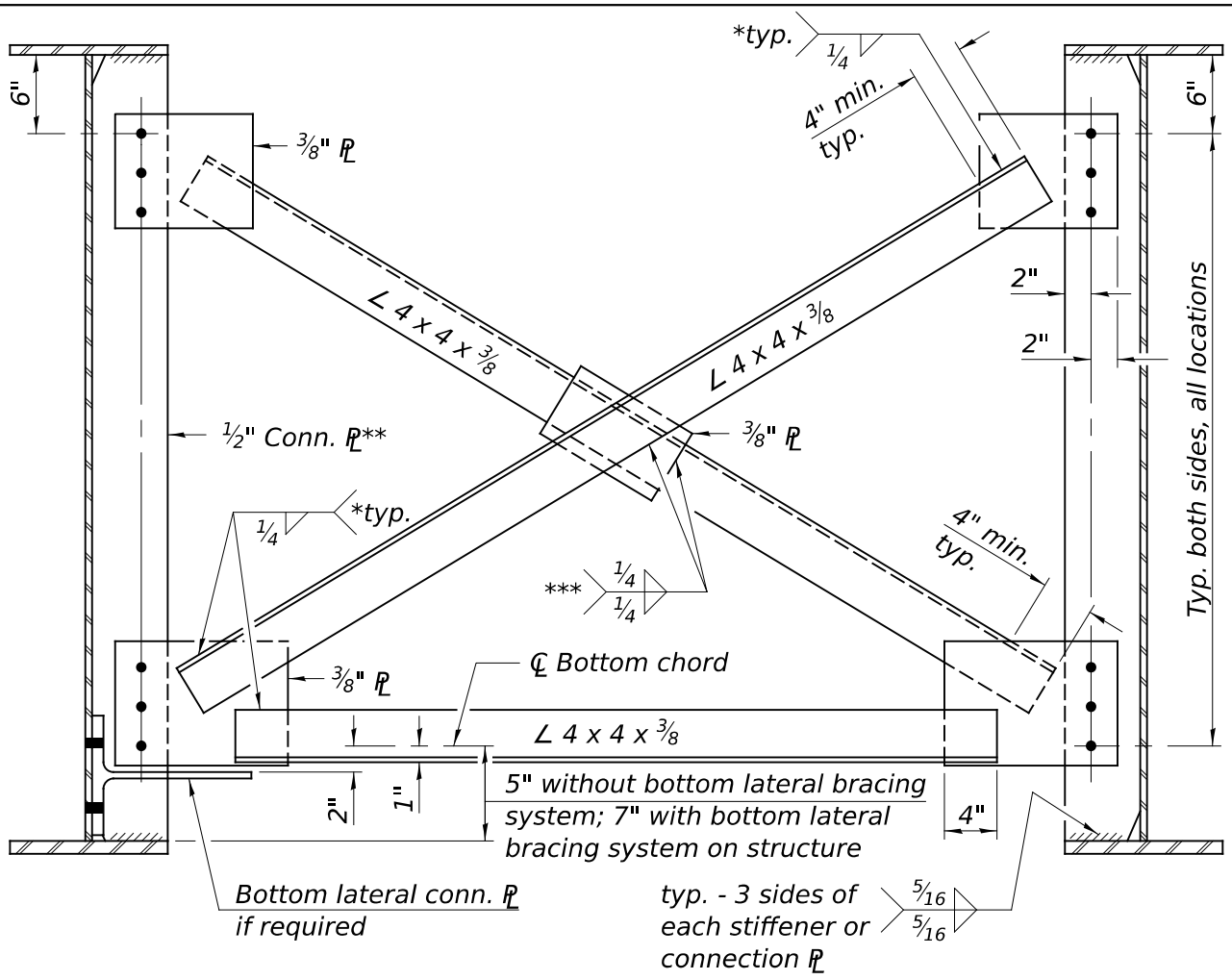
Notes:

Weld sizes are for ½" connection plates. Weld sizes for larger elements or forces shall be by design.

WELD AND CLIP LIMITS AND DETAILS

Figure 3.3.22-4

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TYPICAL INTERIOR CROSS-FRAME

(Detail for PL girders ≥ 40" in depth).

* *Fillet weld angles along 3 sides on one face of gusset PL; however, if cross-frames are galvanized, weld all-around.*

** *May also function as transverse stiffener or bearing stiffener. In these cases, PL thickness and weld size by design.*

*** *If cross-frames are galvanized, weld all-around.*

Notes:

Detail 15/16" Ø holes for all 3/4" Ø bolts (1 1/16" Ø holes for all 7/8" Ø bolts only if required by design). Two hardened washers required for each set of oversized holes.

See Figure 3.3.22-1 for connection PL orientation.

See Figure 3.3.22-4 for weld and clip details.

INTERIOR CROSS-FRAME CONNECTION DETAILS

Figure 3.3.22-5

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3.3.23 End Diaphragms and Cross-Frames

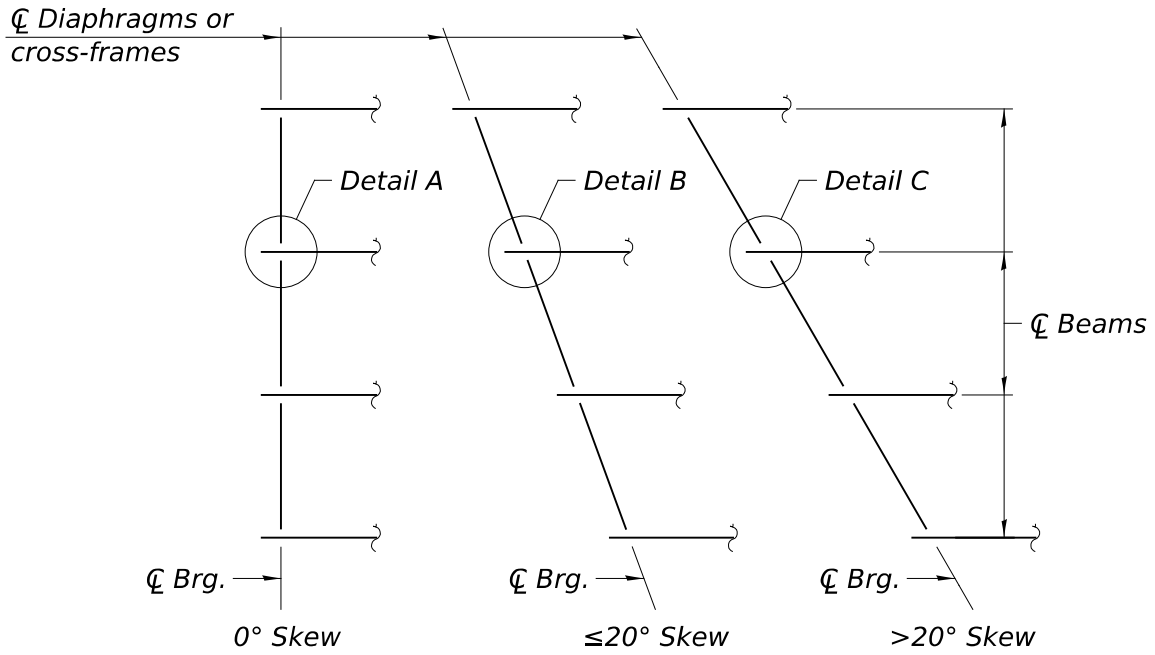
At end supports (non-integral and non-semi-integral abutments, non-continuous piers, seated connections and pin and link connections), diaphragms or cross-frames are normally placed along the centerline of bearings. They should slope from beam to beam with equal vertical clearances from the top flanges for constructability. Figure 3.3.23-1 shows the general layout required for different skew ranges. For skews over 20 degrees, the end diaphragms and cross-frames shall have a continuous centerline of diaphragm that is parallel to and offset from the centerline of bearing. This is achieved by an additional connection plate to the beam as shown in Figure 3.3.23-2. Cross-frames shall have a similar detail up to skews of 45 degrees; refer to the details shown in Figures 3.3.23-6 through 3.3.23-8.

The end diaphragm and cross-frame details given in Figures 3.3.23-1 through 3.3.23-9 utilize a reinforced concrete edge beam which is designed to resist the wheel loads, and steel channel sections (with legs facing away from the abutment backwall) which are designed to only transmit wind loads from the superstructure to the bearings. For design of the concrete edge beam see Section 3.2. The steel channels and reinforced concrete edge beams can be thought of as acting together to form a diaphragm system. The steel diaphragms or cross-frames shall not be stepped at beams. Such steps are difficult to form beyond ends of beams, and the abrupt change in section may induce cracking in the end of the deck slab.

The details given in Figures 3.3.23-1 through 3.3.23-9 are not to be used on bridges which have finger plate or modular joints. End diaphragm and cross-frame details for bridges with finger plate or modular joints are given in Section 3.3.23.1.

End diaphragm details for skews less than 20 degrees are given in Figures 3.3.22-4 and 3.3.23-3-. For skews greater than or equal to 20 degrees, the details are presented in Figures 3.3.22-4 and 3.3.23-4. Cross-frame details for plate girder sections and skews angles less than 45 degrees are given in Figure 3.3.23-5. For skews angles greater than or equal to 45 degrees, details are presented in Figures 3.3.23-6 through 3.3.23-8. Details for end diaphragms on stage construction projects are given in Figure 3.3.23-9.

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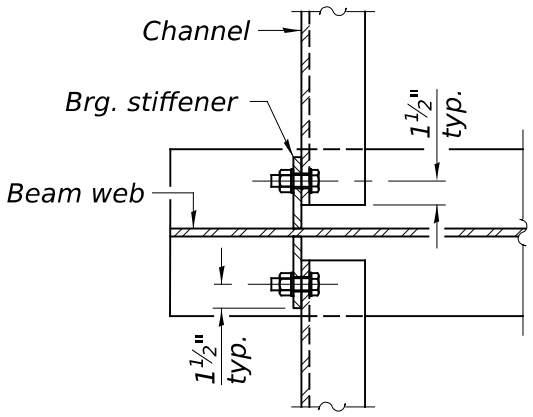
PARTIAL FRAMING PLAN

Note:
For Details A, B, and C, see Figure 3.3.22-2

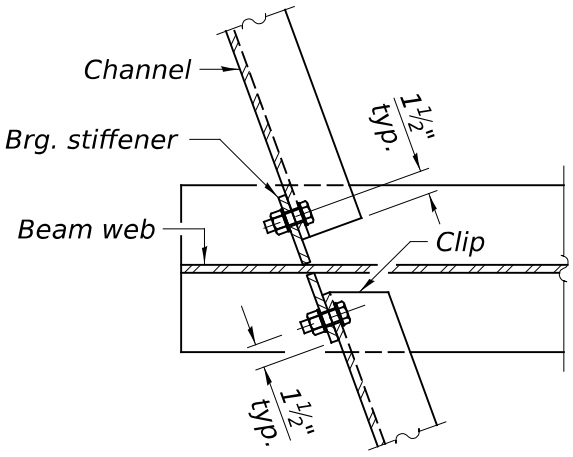
END DIAPHRAGM/
CROSS-FRAME PARTIAL
FRAMING PLAN

Figure 3.3.23-1

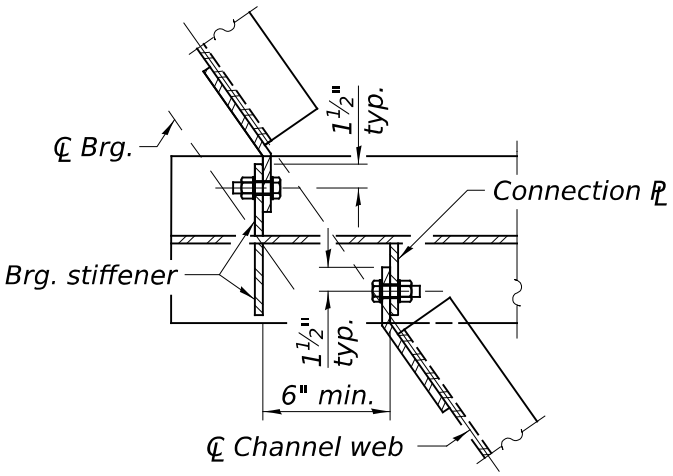
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DETAIL A
(0° Skew)



DETAIL B
(≤20° Skew)



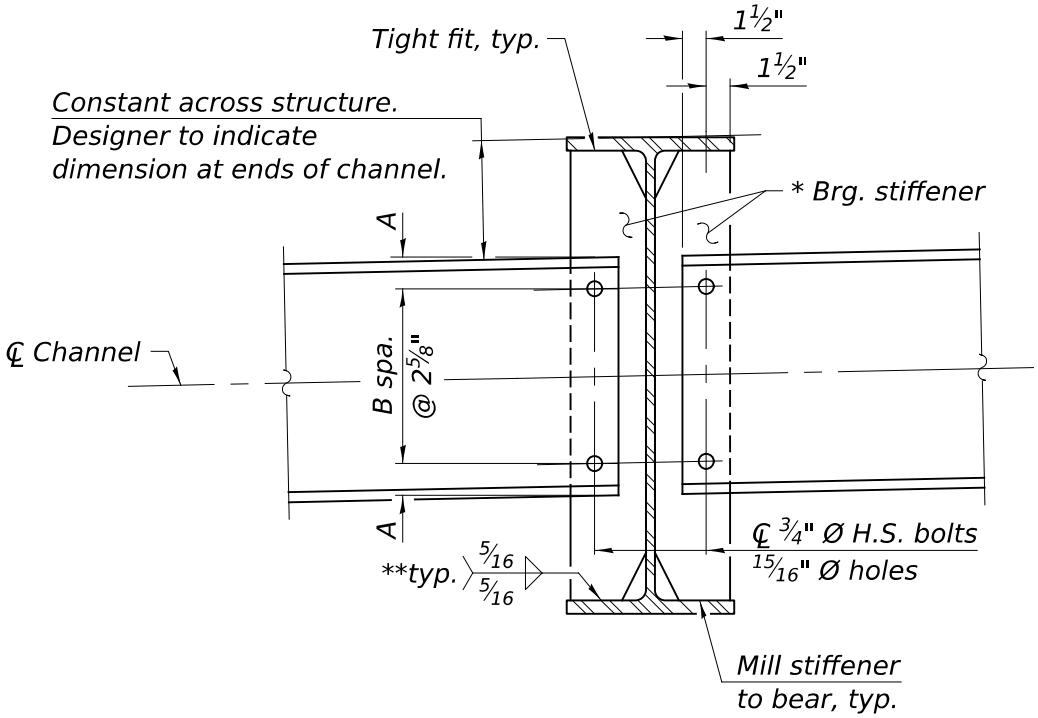
DETAIL C
(20°< Skew <45°)
(See Figure 3.3.23-6 for plate girders with webs ≥ 40" and skews ≥ 45°)

Notes:
Details A, B and C illustrate channel diaphragms. Cross-frame details similar.
Clip channel as necessary for ease of installation of diaphragms on skews.

END DIAPHRAGM/ CROSS-FRAME FRAMING DETAILS

Figure 3.3.23-2

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END DIAPHRAGMS

(Wide flange beam shown, ℄ girder with web depths < 40" similar)

Beam Size	℄ Girder Web	Channel	A	B
W21 to W24	< 24"	C12 x 25	±2 1/16"	3
W27 to W30	≥ 24", < 30"	C15 x 40	±2 ¼"	4
W33 to W36	≥ 30", < 40"	MC18 x 42.7	±2 7/16"	5

* Minimum bearing stiffener thickness shall be 1" or as required by design.

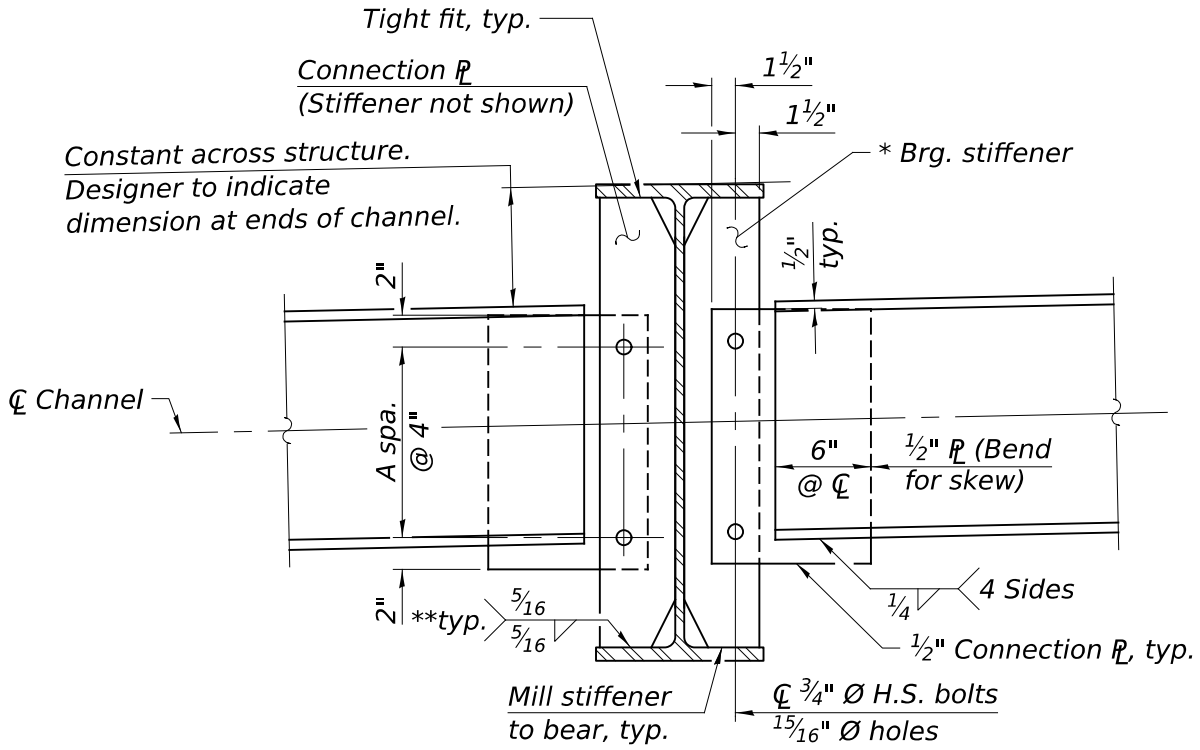
** 3 Sides of each stiffener ℄

Notes:
Two hardened washers required for each set of oversized holes.
Alternate channels of equal depth and larger weight are permitted to facilitate material acquisition. Alternate channels, if utilized, shall be provided at no additional cost to the Department.
See Figure 3.3.23-1 and Figure 3.3.23-2 for connection ℄ orientation.

END DIAPHRAGM CONNECTION DETAIL FOR SKEWS < 20°

Figure 3.3.23-3

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END DIAPHRAGMS

(Wide flange beam shown, PL girder with web depths < 40" similar)

Beam Size	PL Girder Web	Channel	A	B
W21 to W24	< 24"	C12 x 25	±2 1/16"	3
W27 to W30	≥ 24", < 30"	C15 x 40	±2 ¼"	4
W33 to W36	≥ 30", < 40"	MC18 x 42.7	±2 7/16"	5

* Minimum bearing stiffener thickness shall be 1" or as required by design.

** 3 Sides of each stiffener and/or connection PL.

Notes:

Two hardened washers required for each set of oversized holes.

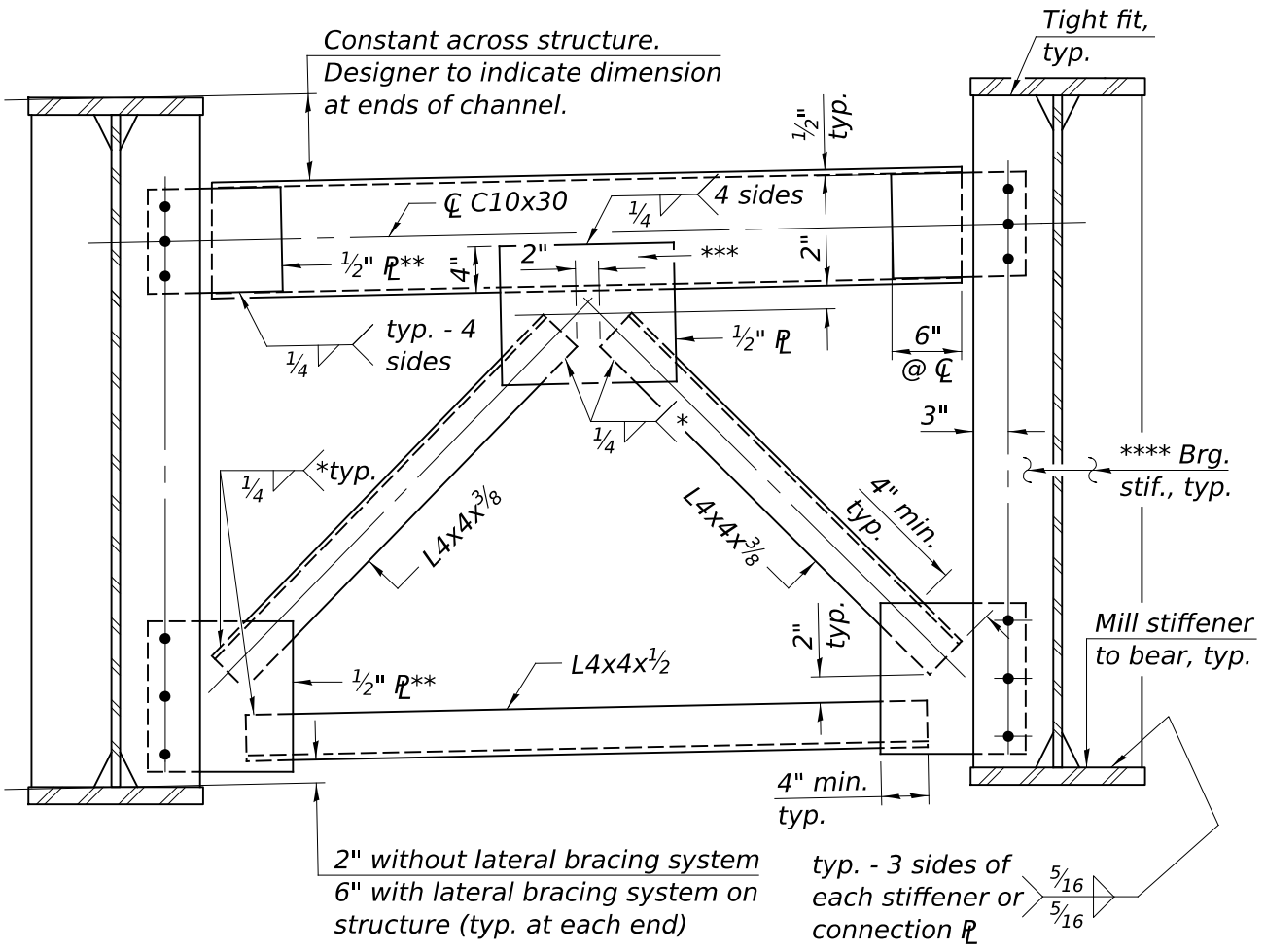
Alternate channels of equal depth and larger weight are permitted to facilitate material acquisition. Alternate channels, if utilized, shall be provided at no additional cost to the Department.

See Figure 3.3.23-1 and Figure 3.3.23-2 for connection PL orientation.

END DIAPHRAGM CONNECTION DETAIL FOR SKEWS ≥ 20°

Figure 3.3.23-4

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TYPICAL END CROSS-FRAME

(Detail for PL girders ≥ 40" web depth).

* *Weld angles along 3 sides on one face of gusset PL; however, if cross-frames are galvanized, weld all-around.*

** *½" PL to be bent for skewed structures.*

*** *Use bolted connection for stage construction, 3 bolts minimum.*

**** *Minimum bearing stiffener thickness shall be 1" or as required by design.*

Notes:

Detail 15/16" Ø holes for all ¾" Ø bolts (1 1/16" Ø holes for all ⅞" Ø bolts only if required by design). Two hardened washers required for each set of oversized holes.

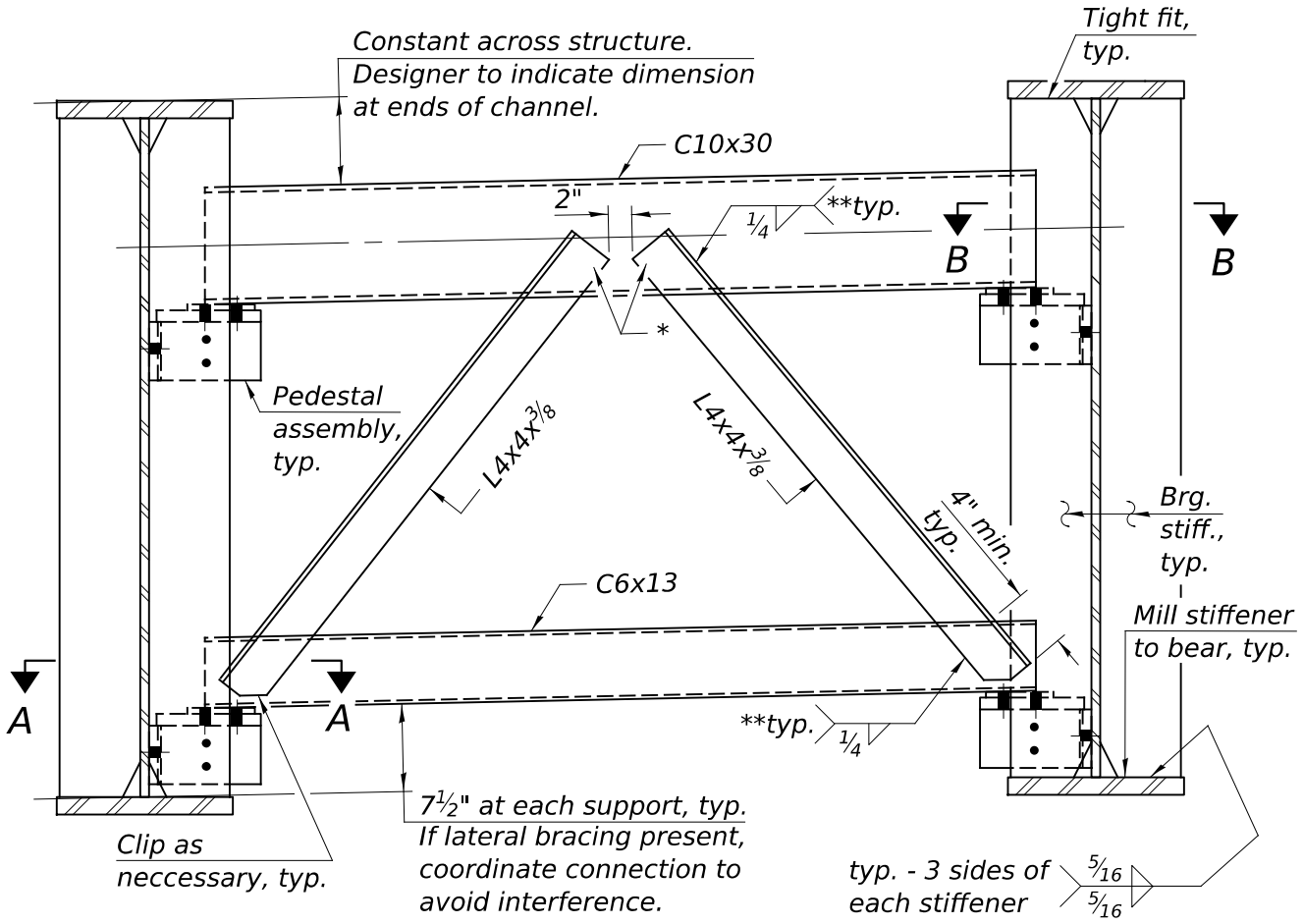
See Figure 3.3.23-1 and Figure 3.3.23-2 for connection PL orientation.

See Figure 3.3.22-4 for weld and clip details.

END CROSS-FRAME CONNECTION DETAIL FOR SKEWS < 45°

Figure 3.3.23-5

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TYPICAL END CROSS-FRAME

(Detail for ℄ girders ≥ 40" web depth)

Notes:

Detail 15/16" Ø holes for all 3/4" Ø bolts (1 1/16" Ø holes for all 7/8" Ø bolts only if required by design).

Two hardened washers required for each set of oversized holes.

Place diaphragm with channel flanges outward from abutment backwall.

For Sections A-A and B-B, see Figure 3.3.23-7.

For weld and clip details, see Figure 3.3.22-4.

For pedestal details, see Figure 3.3.23-8.

* *Use a minimum 2 bolt connection for stage construction.*

** *Weld angles along 3 sides to back face of channel: however, if cross-frames are galvanized, weld all-around.*

END CROSS-FRAME CONNECTION DETAIL FOR SKEWS ≥ 45°

Figure 3.3.23-6

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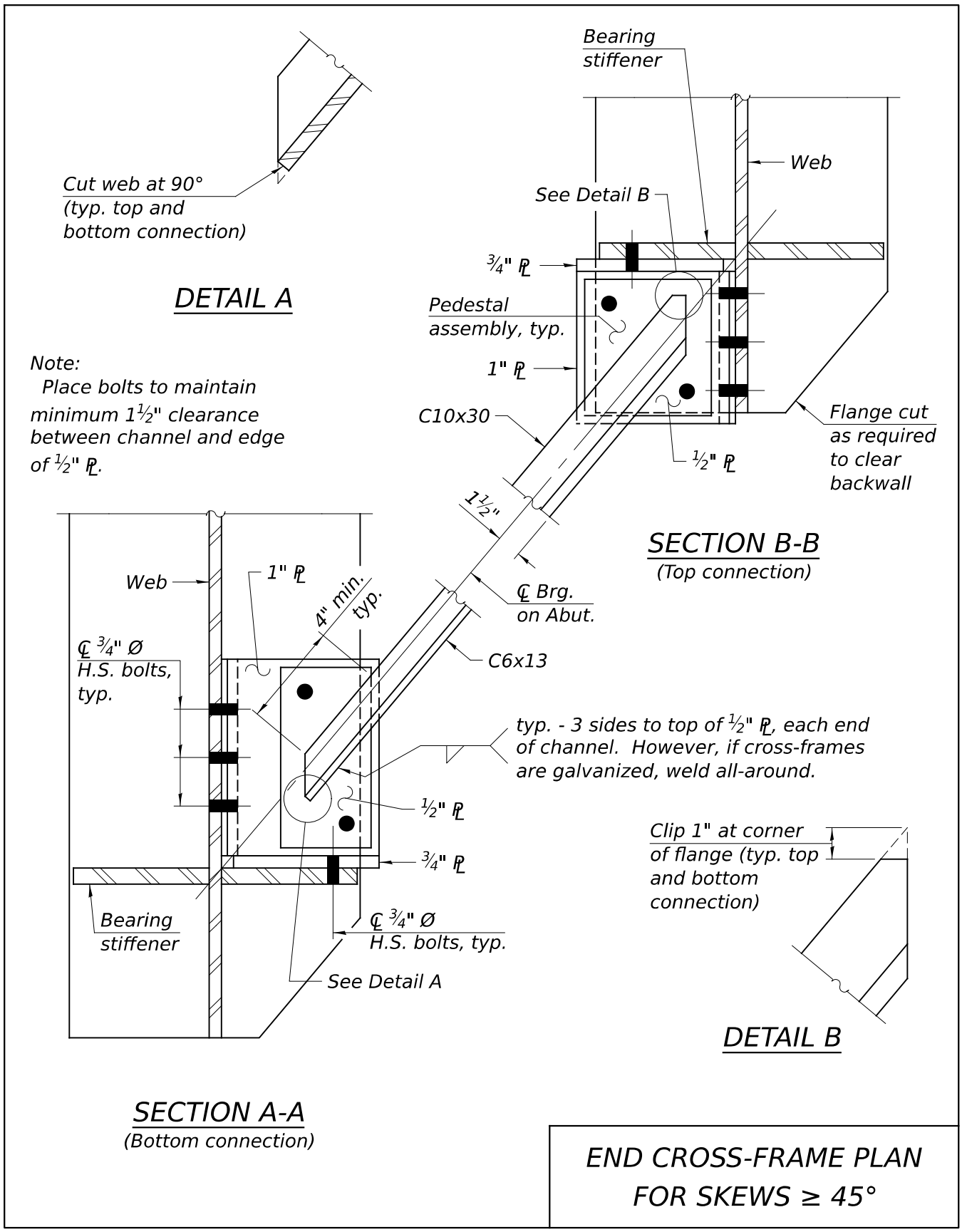
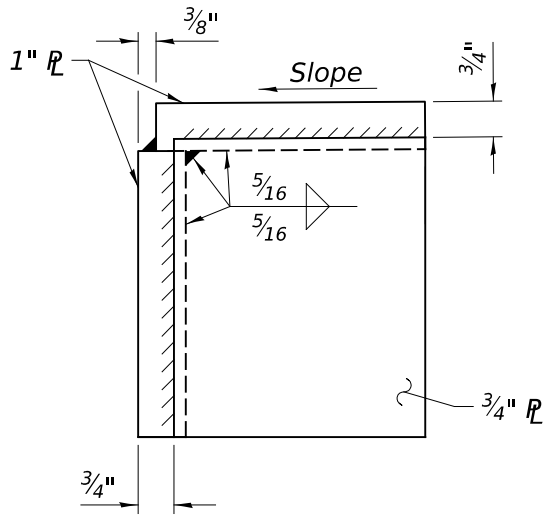
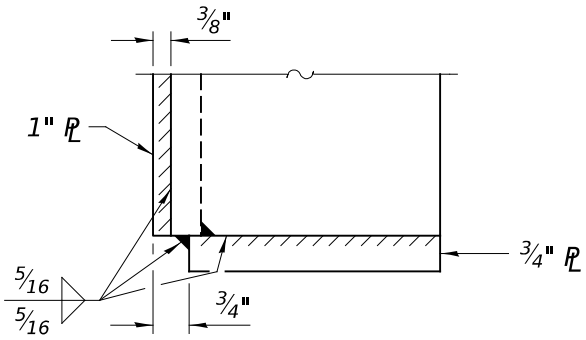


Figure 3.3.23-7

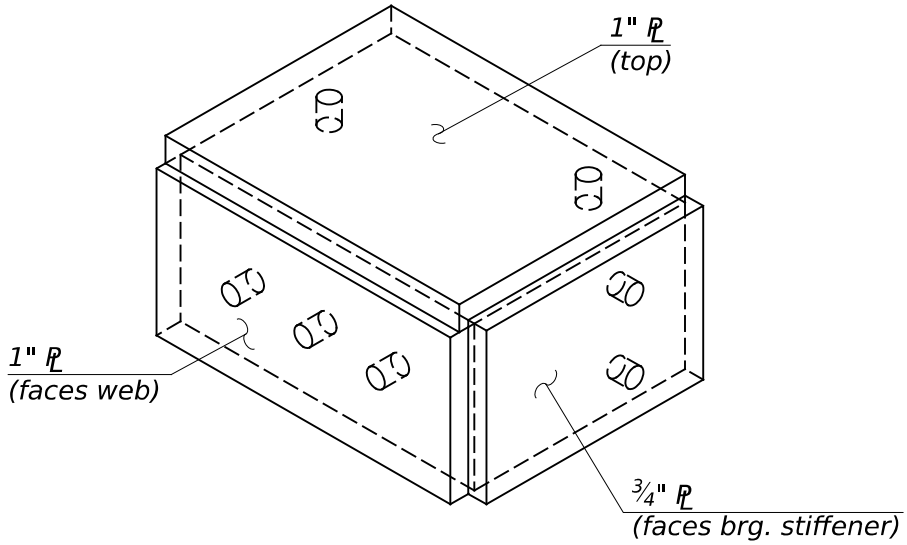
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PEDESTAL ASSEMBLY
ELEVATION



PEDESTAL ASSEMBLY
PARTIAL PLAN



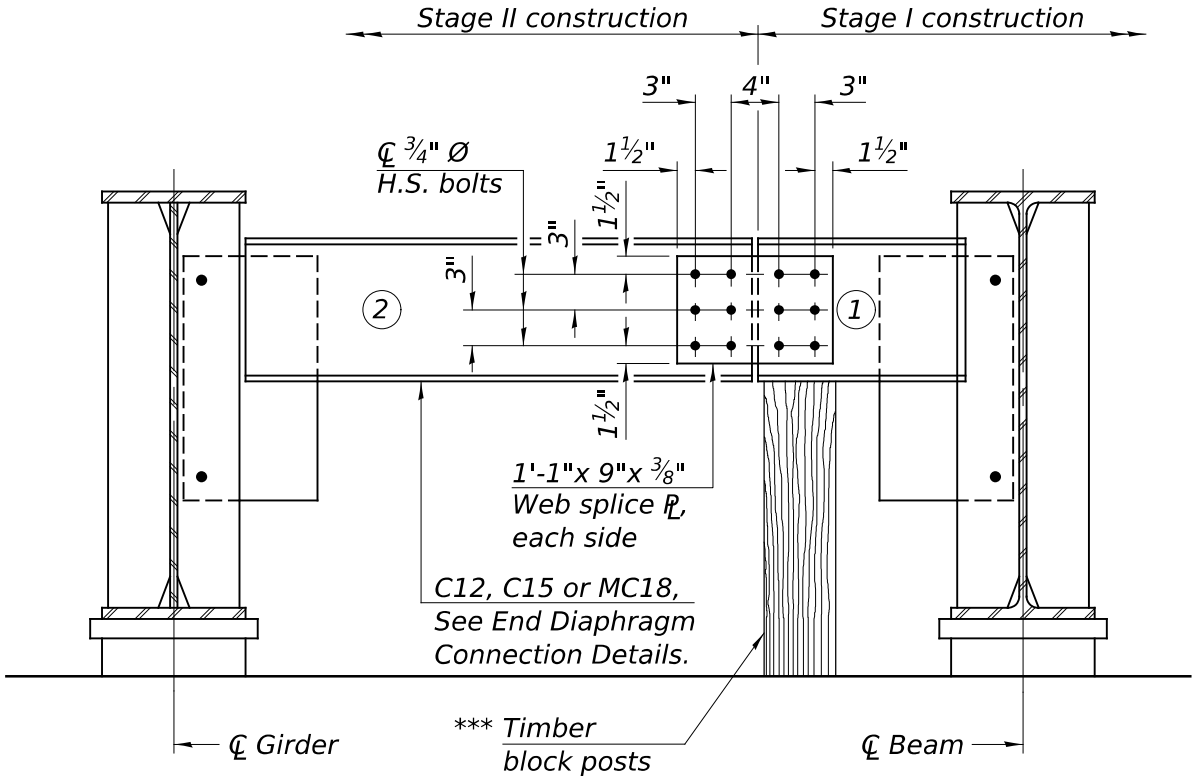
PEDESTAL ASSEMBLY
ISOMETRIC VIEW

PEDESTAL ASSEMBLY DETAILS
FOR END CROSS-FRAME
(SKEWS ≥ 45°)

Figure 3.3.23-8

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*** Cost of timber block posts is included with Structural Steel.



Showing diaphragm for shallow girder

Showing diaphragm for W-beam

END DIAPHRAGM

END DIAPHRAGM STAGE CONSTRUCTION SEQUENCE

1.) Order diaphragm in two sections.
2.) Attach section (1) of diaphragm to beam.
3.) Place timber block posts between section (1) of diaphragm and abutment bearing section.
4.) Attach section (2) of diaphragm to both beam and section (1) of diaphragm during stage II construction with splice PL's.
5.) Remove timber block posts.

Note:
For PL girder ≥ 40" web, use detail above with top C10 only and install lower portion of CF after stage II.

END DIAPHRAGM STAGE CONSTRUCTION SEQUENCE

Figure 3.3.23-9

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3.3.23.1 End Diaphragms and Cross-Frames at Finger Plate and Modular Joints

Cross-frames are the preferred lateral bracing method for plate girder structures with finger plate or modular joints. Cross-frame details for various skews are shown in Figures 3.3.23.1-1 thru 3.3.23.1-5.

The centerline of the diagonal members of the cross-frame are not required to intersect the centerline of the top chord, but it should be as close as practical. When the diagonals of the cross-frame cannot practically have a minimum angle of 30 degrees (as measured from the bottom chord) due to beam spacing, skew, and/or web/cope depth, an alternate bent plate channel is required as shown in Figure 3.3.23.1-6. The bent plate channel is not allowed for skews greater than or equal to 45 degrees. Additional analysis is required if the minimum diagonal angle cannot be satisfied for skews over 45 degrees.

For end cross-frames and end bent plate diaphragms at stage construction joints, see Figures 3.3.23.1-7 and 3.3.23.1-8.

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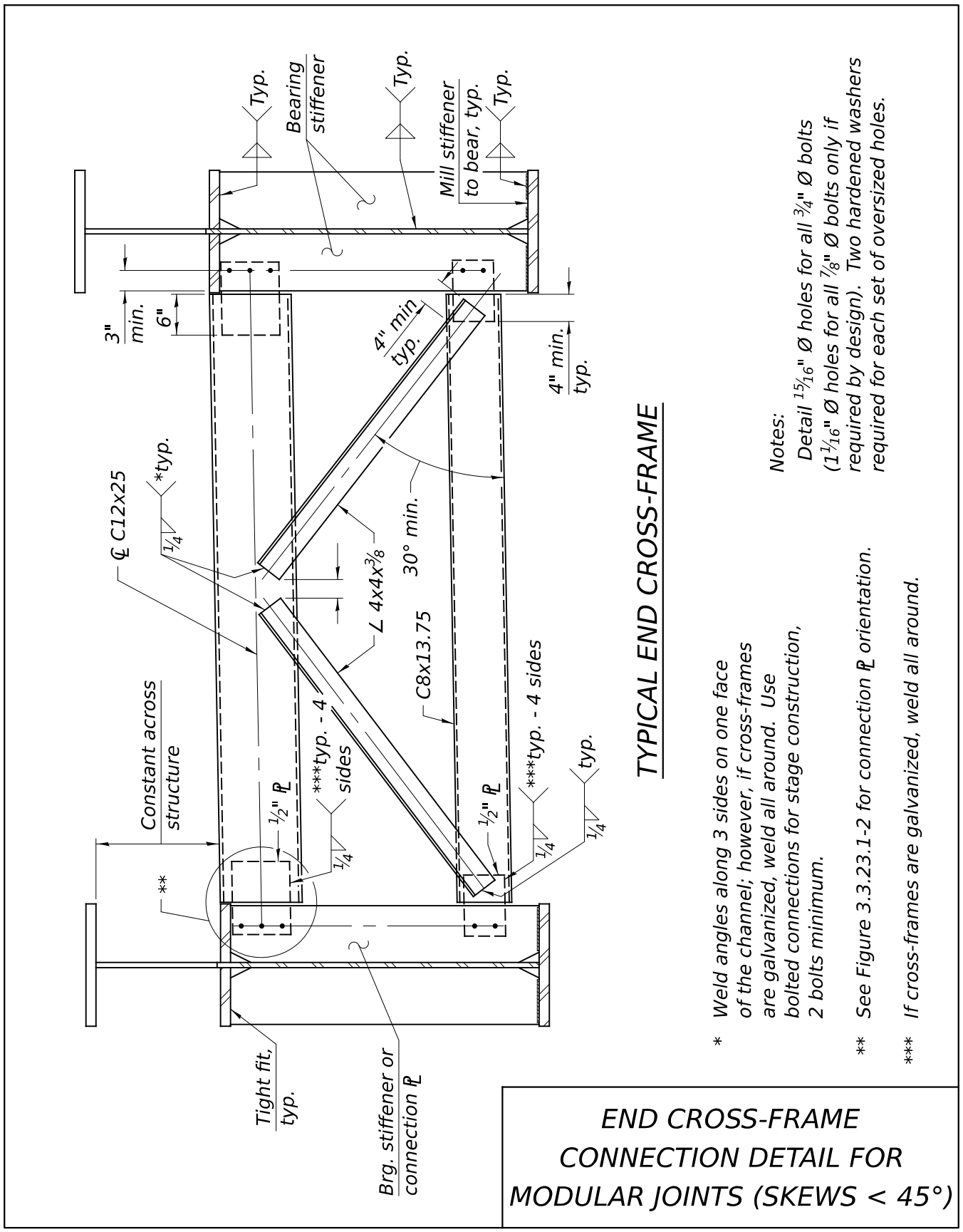
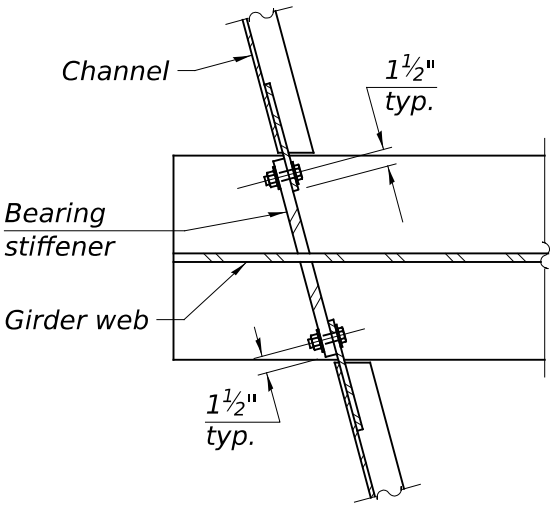
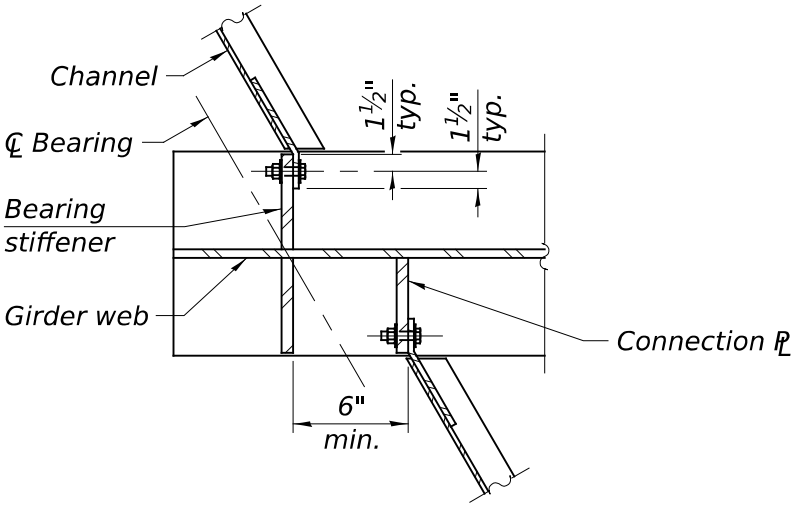


Figure 3.3.23.1-1

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CONNECTION DETAIL
(Skews ≤ 20°)



CONNECTION DETAIL
(20° < Skews < 45°)

END CROSS-FRAME FRAMING DETAILS FOR MODULAR JOINTS (SKEWS < 45°)

Figure 3.3.23.1-2

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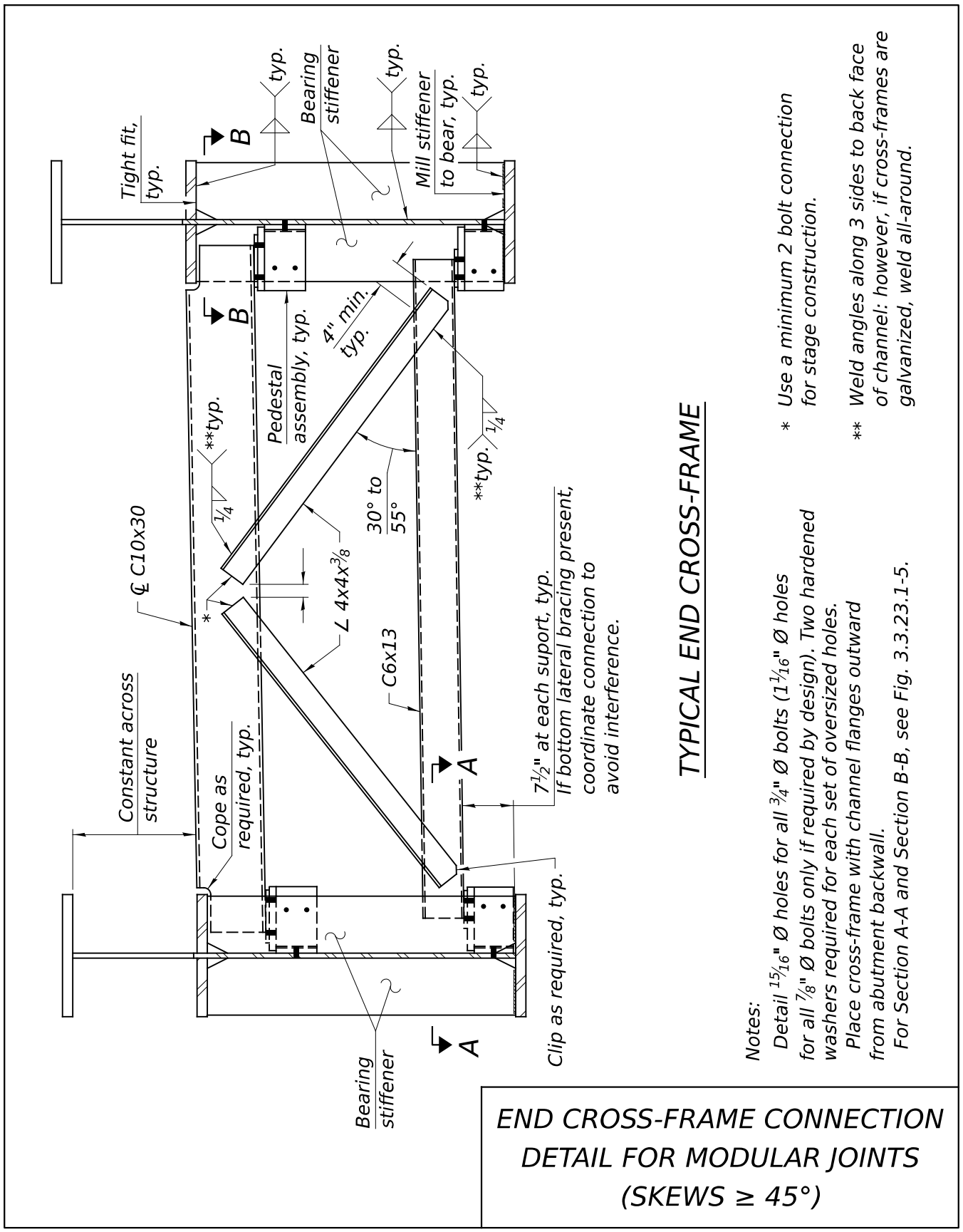


Figure 3.3.23.1-3

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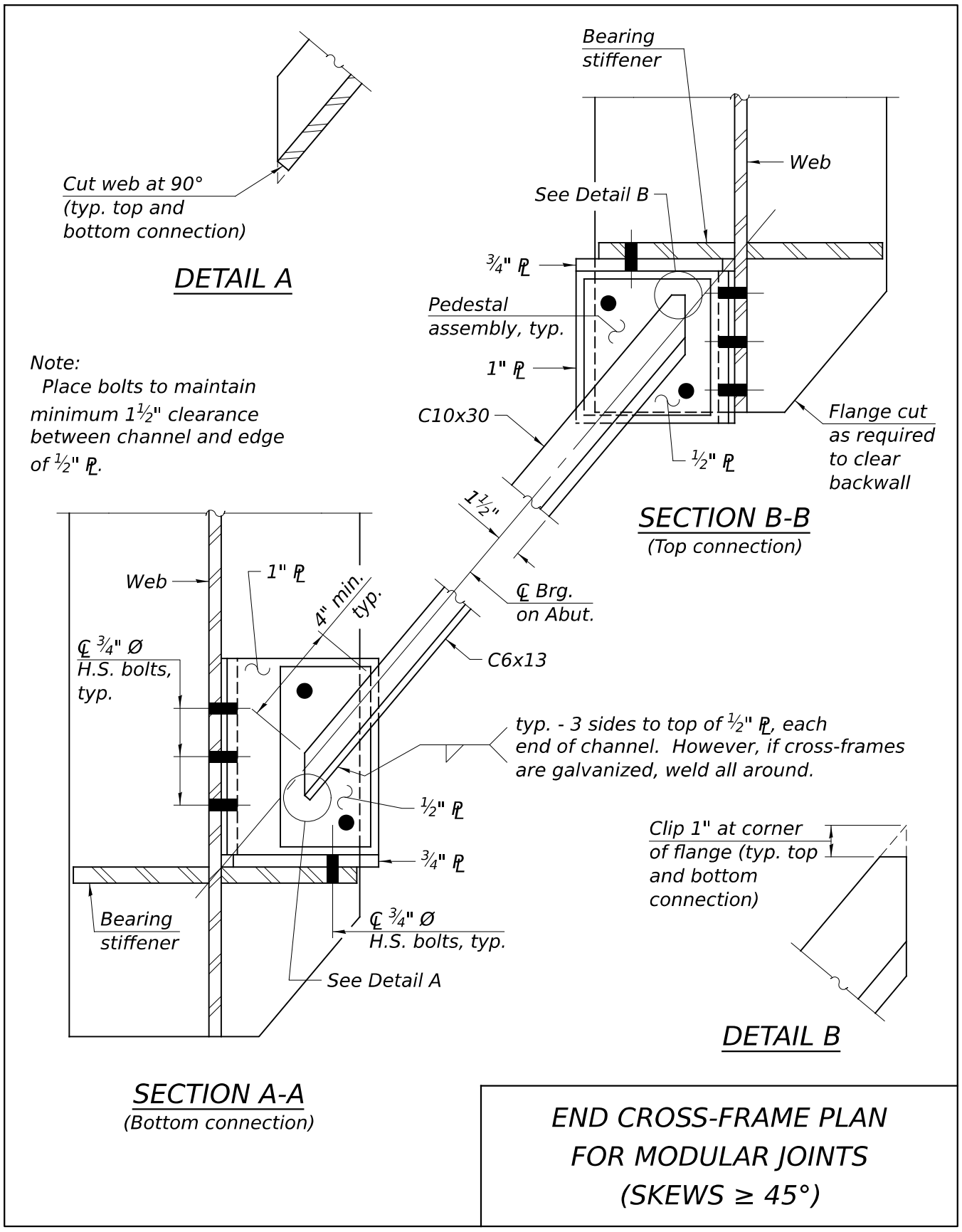
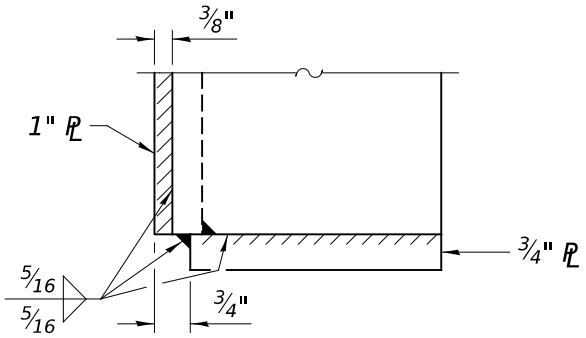
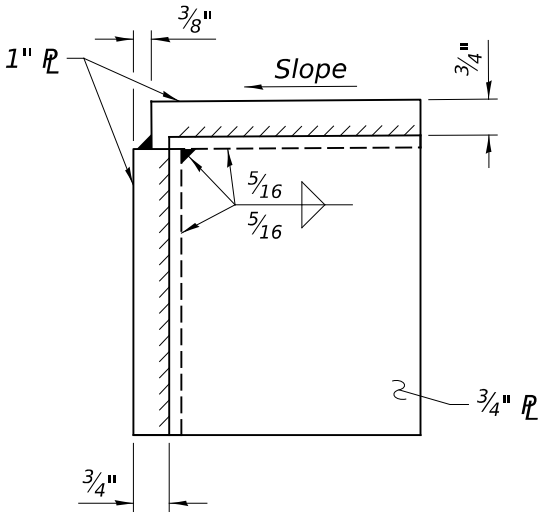


Figure 3.3.23.1-4

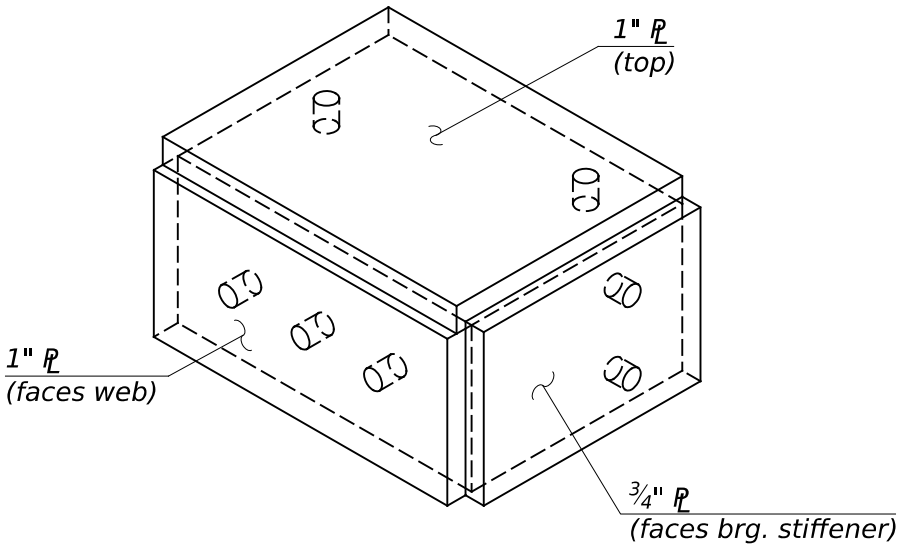
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PEDESTAL ASSEMBLY PARTIAL PLAN



PEDESTAL ASSEMBLY ELEVATION

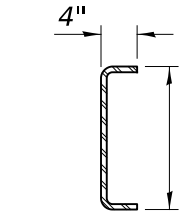


PEDESTAL ASSEMBLY ISOMETRIC VIEW

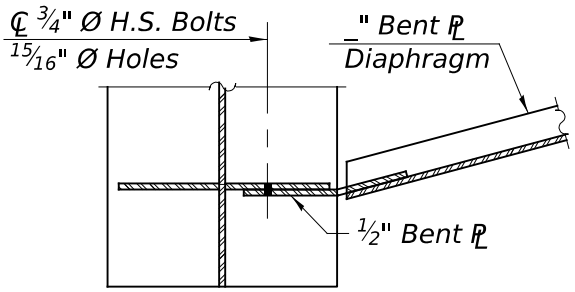
PEDESTAL ASSEMBLY DETAILS FOR MODULAR JOINT END CROSS-FRAME (SKEWS ≥ 45°)

Figure 3.3.23.1-5

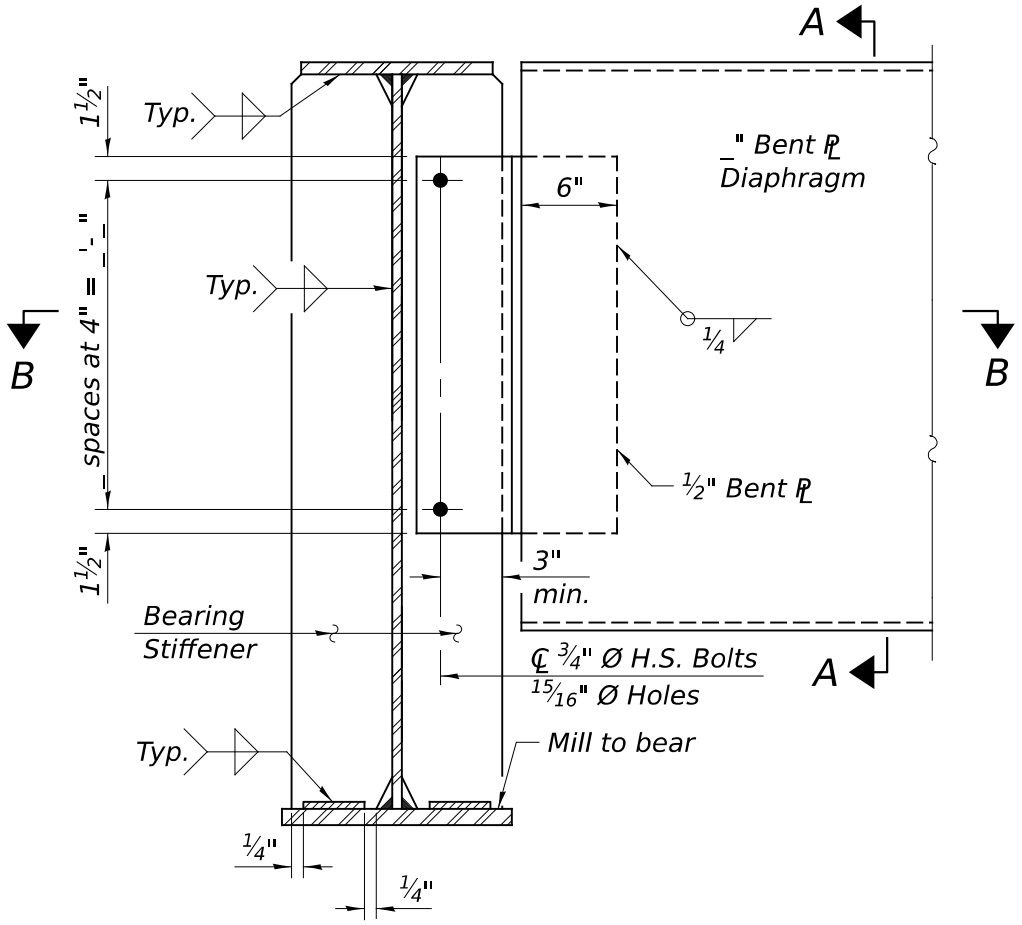
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SECTION A-A



SECTION B-B



END DIAPHRAGM

Notes:

Detail 15⁄16" Ø holes for all ¾" Ø bolts (1 1⁄16" Ø holes for all ⅞" Ø bolts only if required by design).

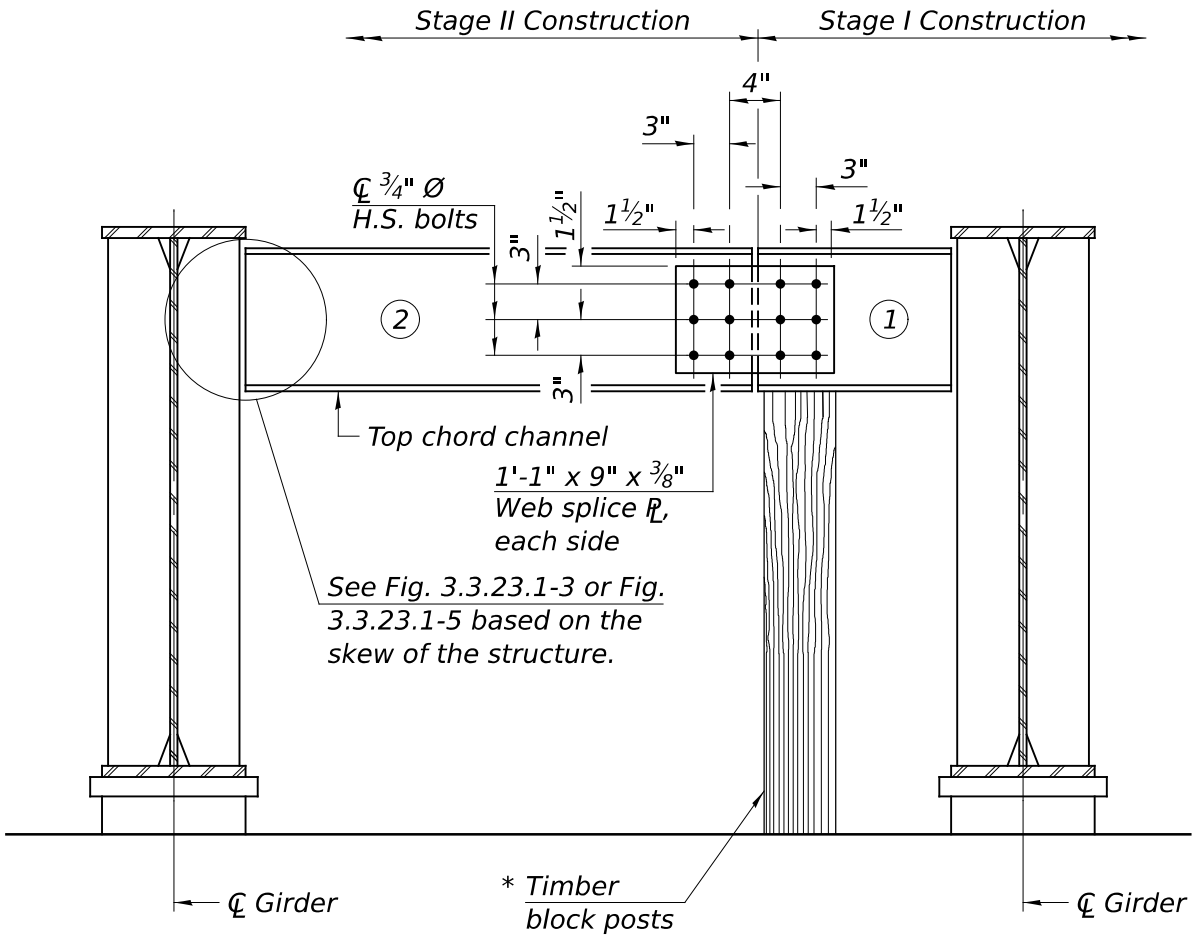
Two hardened washers required for each set of oversized holes.

Place Bent PL diaphragm with flanges outward from abutment backwall.

FINGER PLATE AND MODULAR JOINT DIAPHRAGM (SKEWS ≤ 45°)

Figure 3.3.23.1-6

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STAGE CONSTRUCTION SEQUENCE

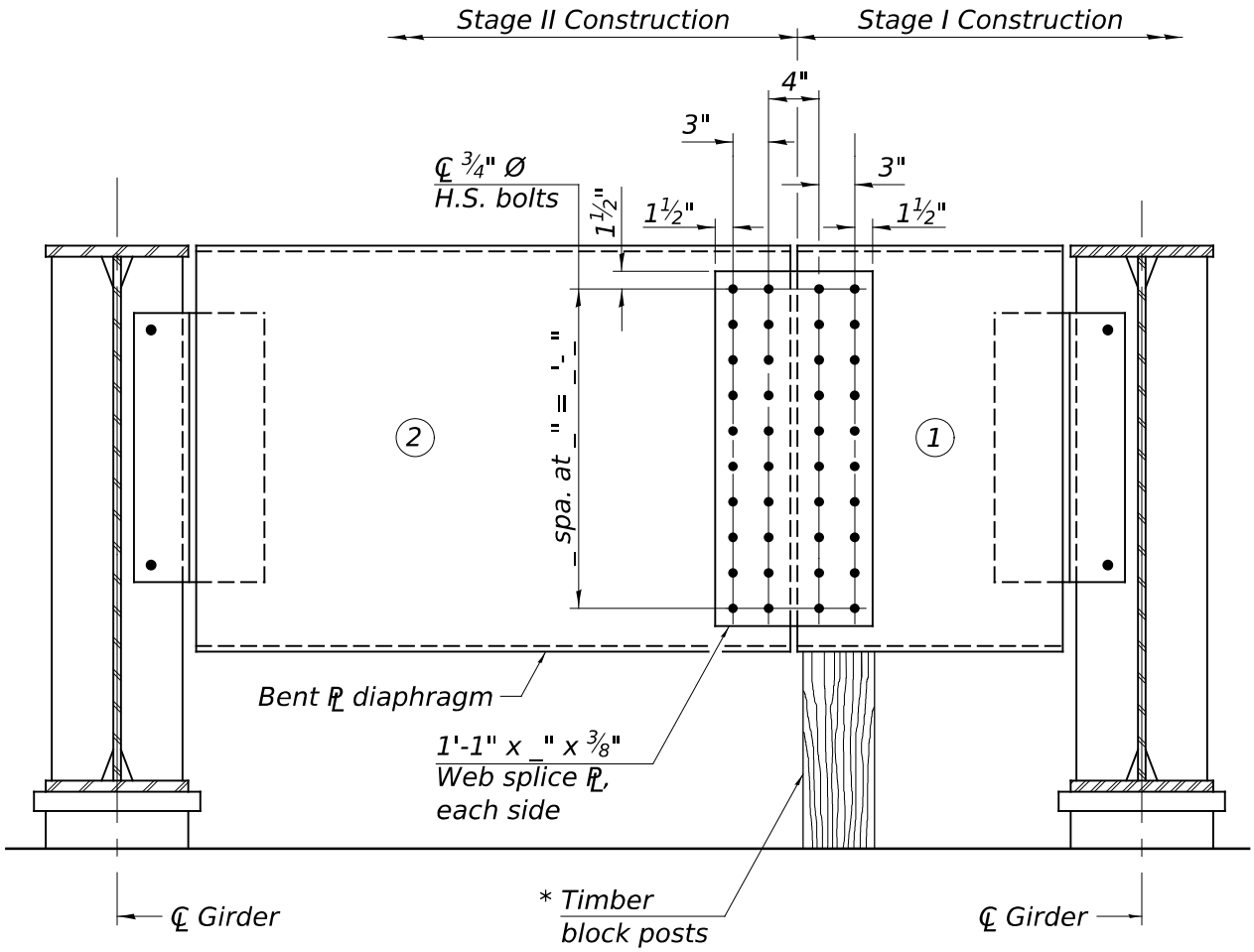
1.) Order top chord in two sections.
2.) Attach section (1) of top chord to girder.
3.) Place timber block posts between section (1) of top chord and abutment bearing section.
4.) Attach section (2) of top chord to both girder and section (1) of top chord during stage II construction with splice PL's.
5.) Attach remaining diagonal truss and bottom chord elements of end cross-frame.
6.) Remove timber block posts.

* Cost of timber block posts is included with Furnishing and Erecting Structural Steel.

END CROSS-FRAME
STAGE CONSTRUCTION SEQUENCE

Figure 3.3.23.1-7

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STAGE CONSTRUCTION SEQUENCE

1.) Order bent ℄ in two sections.
2.) Attach section (1) of bent ℄ to girder.
3.) Place timber block posts between section (1) of bent ℄ and abutment bearing section.
4.) Attach section (2) of bent ℄ to both girder and section (1) of bent ℄ during stage II construction with splice ℄'s.
5.) Remove timber block posts.

* Cost of timber block posts is included with Furnishing and Erecting Structural Steel.

END BENT ℄ DIAPHRAGM STAGE CONSTRUCTION SEQUENCE

Figure 3.3.23.1-8

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3.3.24 Erection and Holes in Diaphragms, Cross-Frames & Lateral Bracing Members

To facilitate erection, bolted connections for steel diaphragms, cross-frames and lateral bracing members in structures not designed for the effects of curvature shall be detailed with oversize holes ($\frac{3}{16}$ in. larger than the nominal bolt diameter) in both plies, as shown in Figures 3.3.22-3 and 3.3.22-5, and Figures 3.3.23-3, 3.3.23-5, and 3.3.23-7. Oversized holes shall have hardened washers according to the Research Council on Structural Connections (RCSC) Specifications for high strength bolt installation.

Standard size holes ($\frac{1}{16}$ in. larger than the nominal bolt size) shall be detailed for all diaphragm and/or cross-frame connections on curved steel structures where the effects of curvature are required to be considered in the determination of major-axis bending moments (i.e. the structure is "significantly curved"). However, the connections for cross-frame and/or diaphragms shall be designed assuming the holes are oversized. The reduced allowable capacity for oversized holes provides an added measure of redundancy should unforeseen issues arise during fabrication and erection. The General Notes (See Section 3.1.3) indicate the default hole and bolt sizes, so only bolt and/or hole diameters differing from the default shall be identified on the plan details.

The Contract plans for significantly curved steel structures shall provide the calculated vertical deflections due to steel self-weight at quarter points along each span for each primary girder/beam in diagrammatic and/or tabular format on the steel framing sheet. The following note shall be associated with the calculated deflections of significantly curved structures on the Contract plans:

> *The calculated deflections of the primary girders/beams under steel self-weight shall be used to detail the diaphragm, cross-frame, and lateral bracing connections, and to erect the structural steel such that the girders/beams will be plumb within a tolerance of ±⅛ in. per vertical ft. throughout when supporting their own weight.*

Field reaming enlarges holes and can adversely affect a significantly curved structure's intended geometry and behavior under design dead, live and/or construction loadings. When field reaming is excessive, reanalysis, revised design, and construction delays may result. Consequently, consideration of constructability should be emphasized by the designer to help avoid potential problems during construction and fabrication. In addition, the following note shall be added to the Contract plans for all significantly curved structures on the steel framing sheet:

> *The Contractor shall either:*
>
> *1. Ream diaphragm and/or cross-frame connection holes during shop assembly, or*

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2. *Provide detailing and fabrication controls acceptable to the Engineer which ensures accuracy such that field reaming will not exceed the amount permitted in Article 505.08(l) of the Standard Specifications.*

For all curved girder/beam structures, the designer shall fully investigate at least one erection sequence to ensure feasibility for construction. The Contract plans and/or Special Provisions shall require the submittal of a comprehensive Steel Erection plan detailing the proposed methods, procedures, and plans for the erection of the structural steel to the desired lines, elevations, and geometry indicated in the Contract plans. Erection plans shall be complete in detail for all phases of the erection process and shall describe the erection procedures, sequences, geometry controls and adjustment procedures, temporary shoring or bracing, bearing and anchor bolt placement, bolt installation and tightening procedures, and shall include any necessary drawings and calculations. The Erection plan shall be prepared and sealed by an Illinois Licensed Structural Engineer and shall be submitted to the Engineer for review and acceptance.

3.3.25 Intermediate Diaphragm or Cross-Frame Connections for Stage Construction

Diaphragms shall not be omitted beneath stage construction lines.

For structures not designed for significant curvature with maximum differential displacements up to 1 in., standard long slots ($^{13}/_{16}$ in. x 1 ⅞ in. for ¾ in. bolts, $^{15}/_{16}$ in. x 2 $^{3}/_{16}$ in. for ⅞ in. bolts) shall be detailed in one end of the bracing and standard oversize holes shall be provided at the other end and in the main member connection plates. For structures with maximum differential displacements greater than 1 in., the designer shall investigate combinations of long slots in both the bracing and main member connection plates to accommodate the differential displacements. Include a note that bolts in slots shall be finger tight until the second stage pour is complete, and position slots so bolts start at one end with no concrete load and finish near the opposite end under deck load, allowing maximum displacement without laterally stressing main members. All holes shall have appropriate hardened or plate washers. Long slots shall not be detailed on webs of members. Structure plans shall clearly detail the initial erected position of slots and bolts.

For straight bridges with differential displacements 2½ in. or larger, a closure pour and articulated temporary bracing may be used to avoid concrete quality problems. The temporary brace has only top and bottom horizontal struts with one bolt at each end through permanent connection holes permitting vertical movement while maintaining lateral spacing. Bolts connecting the temporary bracing should be ⅛ in. larger than permanent bolts ($^{1}/_{16}$ in. smaller than the holes) to better maintain alignment. After the

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differential displacements due to stage construction have been eliminated, temporary braces are replaced by permanent braces with standard oversize holes.

For staged re-decking of existing structures with large differential displacements between stages, or in cases where jacking existing beams or girders with large differential displacements where removal of diaphragms or cross-frames is required, the designer shall evaluate the system for stability and provide details in the structure plans for the removal work and any required temporary bracing.

Structures designed for significant curvature require standard size holes for bracing connections, so if stage construction is necessary, special bracing details shall be developed and accepted by the Department on a case-by-case basis.

3.3.26 Constructability

Designers shall investigate constructability concerns, including strength, local and global stability, and deflections, at applicable stages of construction including, fully erected girders only condition, and during placement of the concrete deck according to plan sequence, in addition to final condition checks.

Constructability requirements are given in Article 6.10.3 of the AASHTO LRFD Bridge Design Specifications. Stability (which includes local and lateral torsional buckling considerations) and the prevention of initial onset of yielding for the beams in the superstructure are the primary considerations during construction of the superstructure. Also see Articles 503.06 and 505.08 of the IDOT Standard Specifications which cover construction of the deck and steel erection, respectively.

The dead loads actually present during each stage of construction should be considered when checking constructability. For typical bridges, this normally entails the dead weight of the beams, the cross-frames or diaphragms, the formwork, and the uncured non-composite deck. The Department also requires that a minimum live load of 20 psf should be considered. This minimum live load accounts for the weight of the finishing machine and other construction loads. The load factors for constructability are given in Article 3.4.2 of the AASHTO LRFD Bridge Design Specifications.

Note that the f_l term in Article 6.10.3.2 of the AASHTO LRFD Bridge Design Specifications is normally zero for routine bridges if Article 503.06 of the IDOT Standard Specifications is satisfied. See Section 3.3.5 and Design Guide 3.3.4 on the Bridges & Structures page for additional information.

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3.3.27 Minimum Steel Plate Sizes

The minimum width for flanges for plate girder bridges shall be 12 inches. Note that, for constructability loading, this may be too narrow. In general, wider, less thick flanges are better than narrower, thicker flanges. The wider flanges have higher moments of inertia in the lateral direction and help resist wind loads and deflections during construction.

The recommended minimum steel plate thicknesses for bridge construction are summarized in Table 3.3.27-1 below.

Minimum Steel Plate Thicknesses

(Inches)

Top Flange of Plate Girder	¾
Plate Girder Web	7/16
Intermediate Vertical Stiffeners	7/16
Flange Splice Plates	½

Table 3.3.27-1

3.3.28 Computation of Structural Steel Quantities

The weights of structural steel plates including an allowance for 50% of the permissible overweight given in ASTM-A6 in accordance with Article 505.12 of the Standard Specifications are given in the Section 4 Appendix. All quantity computations for structural steel shall be based on these weights. Note as well that the weight of bolts is included in the quantity of structural steel but the weight of field weld metal is not.

3.3.29 Structural Steel Coatings

Steel bridges in Illinois utilize one of four different coating systems: painted steel, weathering steel, galvanized steel, and metallized steel. Choice of coating system is determined as per the planning policy in Section 2.3.6. Design policies for the four coating systems are provided below.

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3.3.29.1 Painted Steel

Painted steel beams have an allowable fatigue stress range that corresponds with Fatigue Category A. See Section 6.6 of the AASHTO LRFD Bridge Design Specifications.

The additional weight of paint is marginal (around 0.25 psf) and need not be accounted for in design.

3.3.29.2 Weathering Steel

Weathering steel beams have an allowable fatigue stress range that corresponds with Fatigue Category B. This is less than the allowable fatigue stress range for painted steel beams. See Section 6.6 of the AASHTO LRFD Bridge Design Specifications.

As per Table 6.4.1-1 of the AASHTO LRFD Bridge Design Specifications, Grade 50W weathering steel has Minimum Tensile Capacity (Fu) of 70 ksi. This is important to note when designing connections.

Weathering steel with a yield strength of 36 ksi does not exist. Designers should note that any structure utilizing weathering steel will require a yield strength of 50 ksi or higher.

Drip plates may be utilized on structures where staining of substructures is a concern, or structures with high profile grades where water may run for long distances along the bottom flange. Drip plates consist of clips that are epoxy-glued to the bottom flange, are sized to fit around the bottom flange, and are angled in a direction that promotes water to run off the flange. They typically are located near joints within the limits of the painted steel. For example, if 10 feet of beam ends are painted, the drip plates are placed 10 feet from the end of the beam. An example of a drip plate detail is shown in Figure 3.3.29.2-1.

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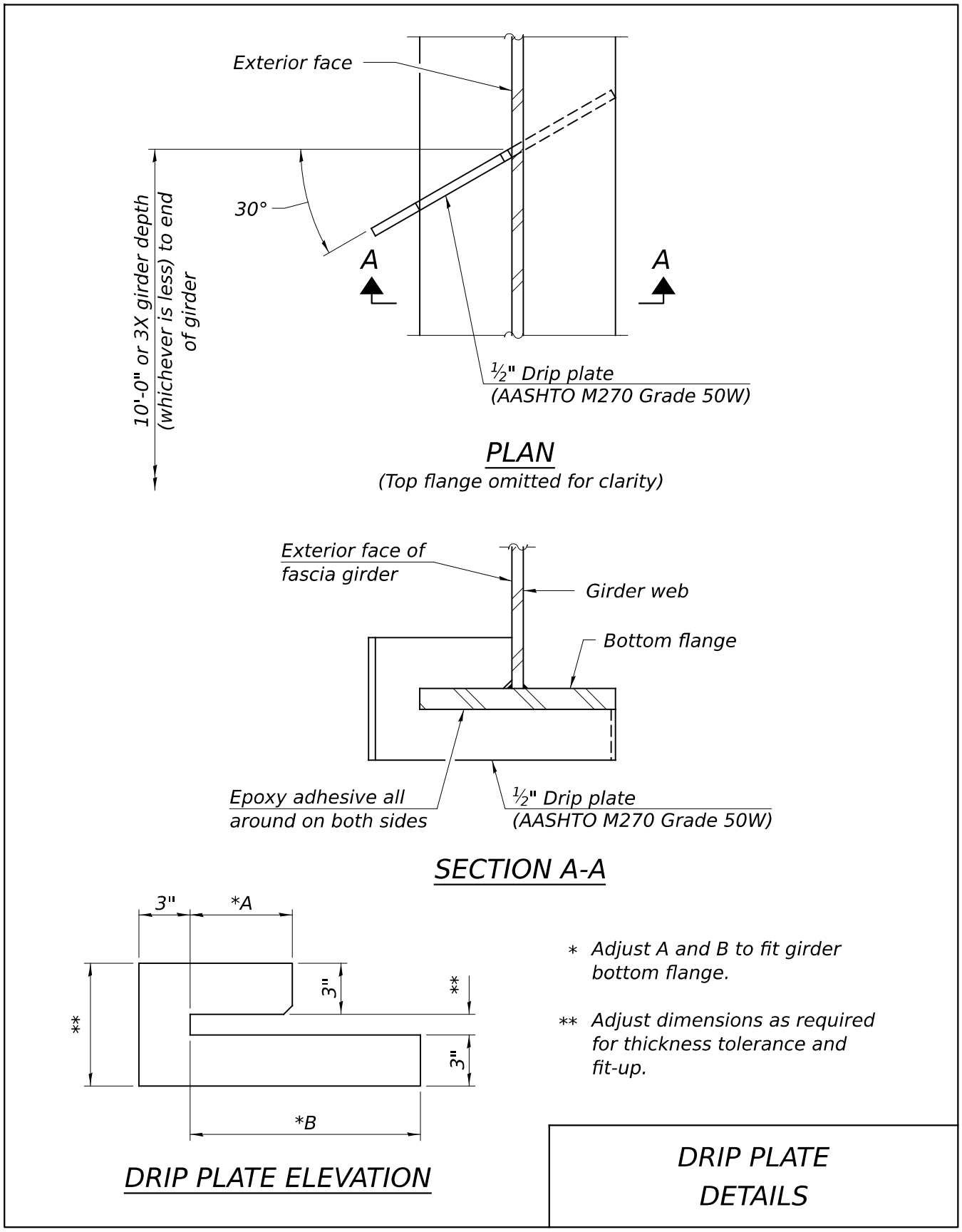


Figure 3.3.29.2-1

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3.3.29.3 Galvanized Steel

If galvanizing of structural steel is required, GBSP 83 shall be included in the contract. GBSP 83 refers to ASTM A 385, which states that overlapping or contacting surfaces that have not had all edges seal welded are undesirable. The Department has attempted to meet this requirement on various "all-around" weld notes in the figures of Section 3.3.22 and 3.3.23 when cross-frames are galvanized.

To facilitate hot-dip galvanizing, beam segments have specific requirements that must be met. See Section 2.3.6.1.1 for a table of these requirements. Because of these requirements, additional splices may be required. Designers should note that, due to the design requirements for holes in tension flanges found in Article 6.10.1.8 of the AASHTO LRFD Bridge Design Specifications, splices in high-moment areas may become problematic both in the steel-only erected state and in the final state and should be avoided. Additionally, splices that are required to be erected over rail traffic are a concern that should be avoided. There may be cases when on built-up members i.e. plate girders, galvanizing thickness will vary from member to member depending upon the thickness of the members. To avoid large discrepancies of galvanizing thickness on a plate girder, designers should attempt to proportion members such that no member is more than three times the thickness of any other connected member. For example, if a ½" thick web is used on a plate girder, designers should attempt to limit flange thicknesses to 1½".

Contact with dissimilar metals is not recommended. The designer should ensure this does not occur in the plan details, including lighting and conduit details. Future attachments such as galvanized conduits mounted on non-galvanized beams require properly insulated connection details.

Galvanizing adds weight to the beams and girders that shall be accounted for in dead load calculations. Zinc has a unit weight of 426 pcf, and the maximum thickness of galvanizing may be assumed to be 20 mils. This 20 mil thickness is seen as a worst-case scenario and a conservative value to use in design. 20 mils of galvanizing adds an additional 0.71 lb. / ft. per foot of perimeter of beam. While this seems like a marginal amount of weight, it can become substantial depending upon the perimeter of the beam. For example, for a typical plate girder with 48 inch deep webs and 18 inch wide flanges, this is around 10 lb. / ft. additional weight.

Galvanizing may result in bolt relaxation. Reduction in slip capacity shall follow Section 6.13.2.8 of the AASHTO LRFD Bridge Design Specifications.

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3.3.29.4 Metallized Steel

Reduction in slip capacity due to metallizing shall follow Section 6.13.2.8 of the AASHTO LRFD Bridge Design Specifications.

The additional weight of metallizing is considered to be marginal and need not be accounted for in design.

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3.4 PPC I-Beams, PPC Bulb T-Beams, and PPC IL-Beams

This section covers the Department's policies related to precast prestressed concrete (PPC) beams supporting loads and elements specified by the AASHTO LRFD Bridge Design Specifications. Standard shapes denoted as I-Beams, Bulb T-Beams, and IL-Beams are maintained by the Department for beam-and-slab configurations. These PPC beams and the selection charts herein comprise the standard basis for design when meeting the material requirements of the Department.

IL-beams are tailored for longer spans and are considerably heavier than the I-beams and bulb T-beams and therefore may not always be the best economic solution for shorter spans. Many of the IL-beam sections have depths similar to the I-beam and bulb T-beam sections, so it is important to differentiate between the sections by using the new and current beam designations accordingly.

I-Beams and Bulb T-Beams utilize different concrete strengths and strand sizes than IL-beams. Use of IL-beam concrete strengths, strand sizes, and fabrication/design techniques with PPC I-beams and bulb T-beams shall not be done. Only marginal span length improvements can be achieved in these beam sections since they are already optimized for the available strand locations. The Department intends to maintain different design standards for the different beam designations in order to avoid confusion among fabricators and designers when making and designing the beams.

3.4.1 Design of PPC I-Beams, PPC Bulb T-Beams, and PPC IL-Beams – General

Section 5 of the AASHTO LRFD Bridge Design Specifications contains unified provisions for both prestressed and non-prestressed reinforced concrete construction. Ultimate strength design of beams in simple span bridges and in positive moment regions of continuous span structures shall be according to the basic principles of prestressed concrete design. Over piers of continuous span bridges, the principles of non-prestressed concrete design shall be used with the reinforcement in the deck acting as the tensile portion of the resisting moment couple. The member shall be assumed to be fully continuous with a constant moment of inertia when determining moments due to composite loads. The design of beams for continuous span bridges shall be the same as for simple spans, except that for the composite dead load and live load plus impact, the beams shall be treated as continuous over the intermediate supports. Design Guide 3.4 under Design on the Bridges & Structures page presents detailed guidance and example calculations for PPC beam design using the AASHTO LRFD Bridge Design Specifications code.

For multi-span bridges where the number of strands required in each span does not vary by more than 15%, the designer should specify the strand pattern required for the longest span provided the minimum

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span length requirements (shown in Tables 3.4.3.1-1 through 3.4.3.1-25) are met. This allows for greater economy and simplicity during fabrication.

3.4.2 Stress Limits

Prestressing steel for all new PPC beam construction shall be uncoated 7-wire, low-relaxation (low-lax) strands with a nominal diameter of $^1/_2$ in or $^6/_{10}$ in. and an ultimate strength of 270 ksi. Stress relieved strands are not permitted for new construction of PPC beams.

The stress limits for low-lax strands immediately prior to transfer shall be according to AASHTO LRFD Bridge Design Specifications Table 5.9.2.2-1 as follows:

f_{pu} = 270 ksi
f_{pbt} = 0.75 f_{pu} (low-relaxation strands)

Where:
f_{pu} = ultimate tensile strength of prestressing steel (ksi)
f_{pbt} = initial stress limit in prestressing steel prior to transfer and before losses (ksi)

Table 3.4.2-1 tabulates force and stress limits for prestressing strands.

TABLE OF STANDARD PRESTRESSING LOAD (Low Relaxation)			
Nominal Diameter	Nominal Steel Area	Initial Prestressing Force (F_i)	Initial Steel Stress Limit (f_{pbt})
Inch	Sq Inch	kips	ksi
1/2	0.153	30.9	201.96
6/10	0.217	43.9	202.30

Table 3.4.2-1

The initial prestressing force was calculated and rounded down to the nearest tenth of a kip (based on the typical accuracy of the fabricator's prestressing equipment). The initial steel stress limit was then back calculated to produce that prestressing force.

Plain (non-prestressed) reinforcement shall be ASTM A706 Gr. 60 with the yield stress used for design taken as 60 ksi.

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3.4.3 Strand Pattern Charts for Moment

Standard strand patterns for use with the standard IDOT I-beam, bulb T-beam, and IL-beam shapes have been developed for the AASHTO LRFD Bridge Design Specifications. These patterns should be utilized by designers in most typical bridge applications. When special cases warrant a non-standard strand pattern, the BBS should be contacted.

Planning strand pattern selection charts located in Section 2.3 should be used in conjunction with the tables located in Section 3.4.3.1. The planning selection charts are tools which allow the designer to choose an appropriate trial strand pattern for a given beam cross section, span length, beam spacing and span type (simple or continuous). The designer shall verify by calculations that the final strand pattern is adequate.

3.4.3.1 Strand Patterns for Simple and Continuous Span Applications

Tables 3.4.3.1-1 through 3.4.3.1-25 detail the developed standard strand patterns for single and multi-span bridges for beams. The eccentricity “e” shown in the Tables 3.4.3.1-1 through 3.4.3.1-12 equals the distance from the centroid of the strand pattern to the neutral axis of the beam. Tables 3.4.3.1-13 through 3.4.3.1-25 provide the number of strands in each row, number and length of debonded strands in each row, number of draped strands and number of fully tensioned top strands. Figures 3.4.3.1-1 and 3.4.3.1-2 provide a key for understanding the naming conventions associated with the standard strand patterns.

For most cases the standard strand patterns should satisfy all the design requirements. However, there could be some situations where a modification will need to be made. These modifications apply to I-beams and bulb T-beams only, and shall be limited to draping additional strands up to a maximum of 16 and increasing the concrete strength up to a maximum of 7000 psi. In addition, these modifications should only be used to satisfy stress limits and not be used to increase the span lengths past the maximums shown in the charts. In the event that a designer wants to exceed these limits, the Bureau of Bridge & Structures shall be contacted for further disposition.

The standard AASHTO LRFD Bridge Design Specifications strand patterns for I-beams and bulb T-beams were developed based upon the following criteria:

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1. ½ in. diameter low-relaxation 7 wire strands with an ultimate tensile strength, f_{pu}, of 270 ksi.
2. Specified concrete compressive strengths for beams of 6 and 7 ksi with release strengths of 5 and 6 ksi, respectively. Specified concrete compressive strength for decks of 4.0 ksi.
3. HL-93 live loading using distribution methods outlined in Section 4.6.2.2 of the AASHTO LRFD Bridge Design Specifications.
4. 8 in. thick concrete deck.
5. 1 in. average fillet height for dead load only (not included in section properties).
6. 6 beam lines.
7. Standard 39" constant-slope concrete parapet with a weight of 0.525 k/ft.
8. 50 psf future wearing surface.
9. Multi-span tables and charts based upon 2 equal spans.

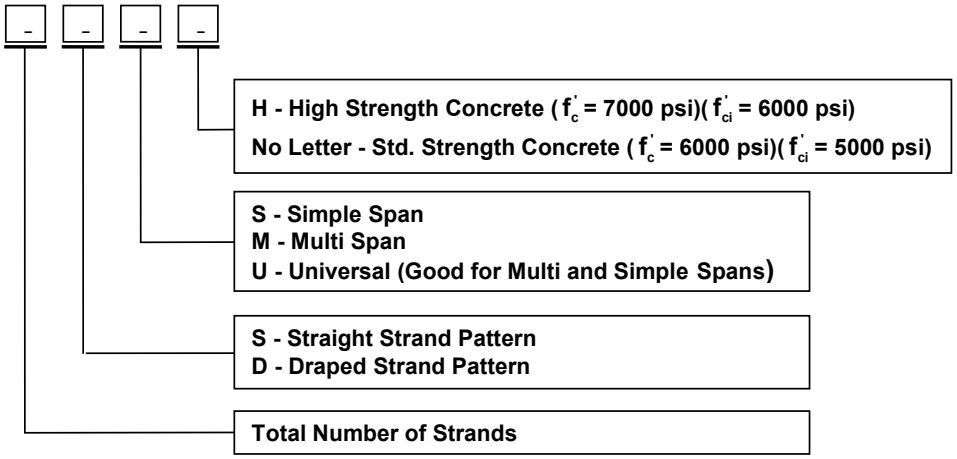


Figure 3.4.3.1-1

The standard strand patterns for the IL-Beams were developed based on the following criteria:

1. 0.6 in. diameter low-relaxation 7 wire strands with an ultimate strength, f_{pu}, of 270 ksi. The Initial Steel Stress Limit, f_{pbt}, is equal to 0.75 f_{pu}; however, the initial Prestressing Force, F_i, is rounded down to the nearest tenth of a kip based on the typical accuracy of a fabricator's prestressing equipment and the f_{pbt} is adjusted accordingly as shown.

TABLE OF STANDARD PRESTRESSING LOAD (Low Relaxation)			
Nominal Diameter	Nominal Steel Area	Initial Prestressing Force (F_i)	Initial Steel Stress Limit (f_{pbt})
Inch	Sq. Inch	kips	ksi
6/10	0.217	43.9	202.30

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2. A concrete compressive strength, f'c, for the beam of 8.5 ksi with a release strength, f'ci, of 6.5 ksi. A specified concrete compressive strength for the bridge deck of 4.0 ksi.
3. HL-93 live loading using the distribution specified in AASHTO LRFD Bridge Design Specifications Table 4.6.2.2.2b-1. The overhang of the exterior beam shall be limited to half the beam spacing or 3'-6", whichever is smaller. The lever rule need not be checked. Note the simplified live load equations for prestressed concrete beams shall not be used.
4. Simple Span
5. 6 beam lines.
6. 8 in. thick concrete deck.
7. 2.5 in. average fillet height for dead load only (not included in section properties).
8. Standard 39” constant-slope concrete parapet with a weight of 0.525 k/ft.
9. 50 psf future wearing surface.
10. Minimum beam spacing of 6 and maximum beam spacing of 10 feet.
11. A maximum of 10 draped strands, a maximum of 4 permanent, fully tensioned top strands, and debonded strands, according to AASHTO LRFD Article 5.9.4.3.3, were utilized to satisfy end stresses. Standard strand patterns will designate the number of each type of strand. See Item 16 below.
12. Initial tensile stress in top flange is limited such that the top reinforcement in the top flange satisfies AASHTO LRFD Bridge Design Specifications Figure C5.9.2.3.1b-1.
13. Limit beam camber to near zero or greater after deck placement, parapet placement, and ½ the weight of FWS.
14. Centerline of bearing located at 7 ½" from beam end utilizing a 15" x 36.5" x 1" fabric bearing pad. A bearing pad of 15" x 28.5" x 1" shall be used for the IL 27 beams. The pad size was increased to satisfy AASHTO LRFD Bridge Design Specifications Article 5.7.3.5.
15. A factor of safety 1.5 (cracking); 1.0 (compression); and 1.5 (failure) for beam stability during initial lifting.

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16. For the IL-beam sections, the strand detailing follows the AASHTO LRFD Bridge Design Specifications with these additional recommendations:

    a. Modified Debonding Locations: AASHTO Article 5.9.4.3.3 G specifies the maximum debond length should be the smaller of two criteria: (0.2 x span length) or (0.5 x span length minus the development length). Since the standard strand pattern tables are for simple span and multi-span structures, the AASHTO check has been modified to account for a range of maximum moment locations. Therefore, the Department recommended check for wide-top flanges is the smaller of: (0.2 x span length) or (0.4 x span length minus the development length). Also, long-term negative camber is not allowed by the Department due to clearance and serviceability concerns. To satisfy this requirement, the narrow top flange sections were governed by a maximum debond length of 0.15 x span length rather than 0.2 x span length.

    b. Strand Locations at Beam Ends: AASHTO Article 5.9.4.3.3 I specifies that all strands shall be bonded within the horizontal limits of the web when the bottom flange width to web ratio exceeds 4. This is the case for all IL-Beams and the standard strand patterns satisfy this requirement. Likewise, there should not be an area of the bottom flange below the web that is devoid of strands when the strands are draped. Many of the Department's previous strand patterns drape all the available rows of bottom strands leaving no strands in the web area at the beam ends in the bottom flange. The strand pattern charts herein have been updated to ensure 4 strands remain in the bottom flange area under draped strands.

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The following is the nomenclature for a typical IL-Beam strand pattern.

Example: Strand pattern 46B-2T-8db-4d for the IL45-3838 Beam

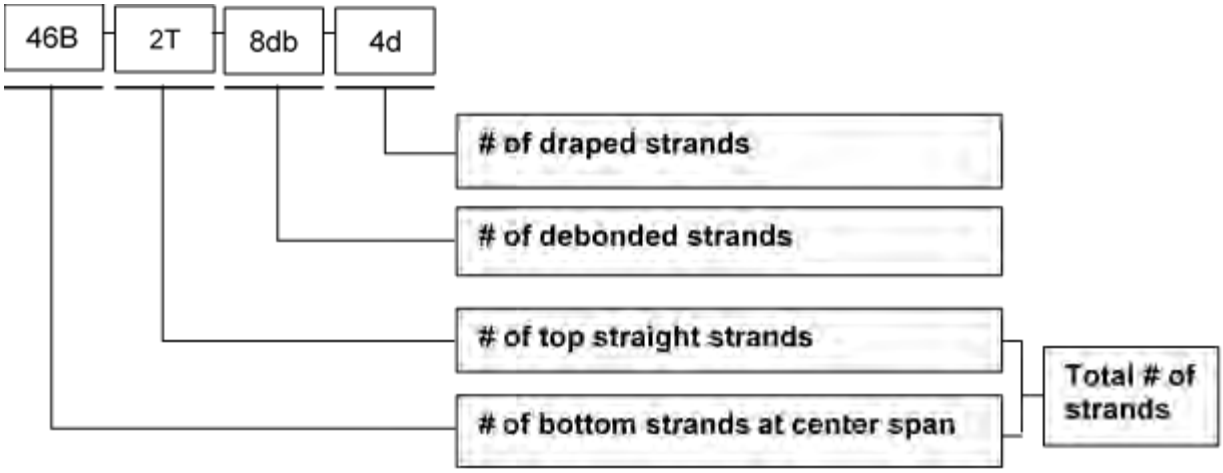


Figure 3.4.3.1-2

Detensioning sequences were not considered in the development of the standard strand patterns; however, the Department's standard reinforcement details have demonstrated good performance in satisfying the Precast/Prestressed Concrete Institute (PCI) crack tolerances. Designers and fabricators may check detensioning sequences and alter strand patterns accordingly provided the details are sealed by an Illinois Licensed Structural Engineer and documented on the Contract plans and/or shop drawings. Designers checking detensioning sequences using proprietary software should examine all boundary condition assumptions made in the software, such as the presence of holddown devices for harped strands or friction between formwork and the bottom of the bottom flange when detensioning. Proprietary software may not account for detensioning sequences and yield results with levels of conservatism, which is also acceptable to the Department.

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LRFD 36" I-Beam Simple Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
14SU	Center	2	2	4	2	2	2												8.513	-
	End	2	2	4	2	2	2												8.513	
14DS	Center	4	4	4	2														10.799	37
	End	4	4	4							2								7.227	
16DS	Center	4	4	4	4														10.370	37
	End	4	4	4	2						2								7.245	
18DS	Center	6	4	4	4														10.703	47
	End	6	4	2	2						2	2							5.148	
20DSH	Center	6	6	4	2	2													10.570	48
	End	6	6	4							2	2							5.970	

Table 3.4.3.1-1

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LRFD 36" I-Beam Multi-Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
14SU	Center	2	2	4	2	2	2												8.513	-
	End	2	2	4	2	2	2												8.513	
14DM	Center	4	4	4	2														10.799	37
	End	4	4	2							2	2							3.656	
16DM	Center	4	4	4	4														10.370	37
	End	4	2	2	2						2	2	2						0.995	
18DM	Center	6	4	4	2	2													10.481	45
	End	6	2	2							2	2	2	2					0.259	
20DMH	Center	6	4	4	4	2													10.170	42
	End	6	2	2	2						2	2	2	2					0.970	

Table 3.4.3.1-2

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LRFD 42" I-Beam Simple Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
16SU	Center	2	4	4	2	2	2												11.152	-
	End	2	4	4	2	2	2												11.152	
18SS	Center	2	4	4	4	2	2												10.985	-
	End	2	4	4	4	2	2												10.985	
18DS	Center	8	8	2															14.319	50
	End	8	8								2								10.652	
20DS	Center	8	8	4															14.052	53
	End	8	6	2							2	2							7.452	
22DS	Center	8	8	6															13.834	56
	End	8	6	4							2	2							7.834	
24DS	Center	8	8	8															13.652	59
	End	8	6	6							2	2							8.152	
26DSH	Center	10	10	4	2														13.806	62
	End	10	8	2							2	2	2						6.652	
28DSH	Center	10	10	6	2														13.652	64
	End	10	8	4							2	2	2						7.009	

Table 3.4.3.1-3

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LRFD 42" I-Beam Multi-Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
16SU	Center	2	4	4	2	2	2												11.152	-
	End	2	4	4	2	2	2												11.152	
16DM	Center	8	8																14.652	47
	End	8	6								2								10.277	
18DM	Center	8	8	2															14.319	50
	End	8	6								2	2							6.985	
20DM	Center	8	8	2	2														13.852	51
	End	8	6								2	2	2						4.552	
22DMH	Center	10	8	2	2														14.016	55
	End	8	6								2	2	2	2					2.743	
24DMH	Center	10	8	2	2	2													13.485	55
	End	8	6								2	2	2	2	2				1.402	
26DMH	Center	10	8	2	2	2	2												12.883	52
	End	8	6								2	2	2	2	2	2			0.421	
28DMH	Center	10	8	2	2	2	2	2											12.223	48
	End	8	6								2	2	2	2	2	2	2		-0.277	

Table 3.4.3.1-4

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LRFD 48" I-Beam Simple Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
18SU	Center	2	4	4	2	2	2	2											13.755	-
	End	2	4	4	2	2	2	2											13.755	
20SS	Center	2	2	4	4	4	2	2											13.088	-
	End	2	2	4	4	4	2	2											13.088	
20DU	Center	8	8	4															17.488	54
	End	8	6	2							2	2							9.688	
22DS	Center	10	10	2															17.815	61
	End	10	8								2	2							10.724	
24DS	Center	10	10	4															17.588	64
	End	10	8	2							2	2							11.088	
26DS	Center	10	10	6															17.396	67
	End	10	8	4							2	2							11.396	
28DS	Center	10	10	6	2														17.088	69
	End	10	8	4							2	2	2						9.159	
30DSH	Center	10	10	6	2	2													16.688	67
	End	10	8	4							2	2	2	2					7.355	
32DSH	Center	10	10	8	2	2													16.588	70
	End	10	8	6							2	2	2	2					7.838	

Table 3.4.3.1-5

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LRFD 48" I-Beam Multi-Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
18SU	Center	2	4	4	2	2	2	2											13.755	-
	End	2	4	4	2	2	2	2											13.755	
18DM	Center	8	8	2															17.755	50
	End	8	8								2								13.421	
20DU	Center	8	8	4															17.488	54
	End	8	6	2							2	2							9.688	
22DM	Center	8	8	6															17.270	57
	End	6	6	4							2	2	2						6.633	
24DMH	Center	10	10	2	2														17.421	61
	End	10	8								2	2	2						8.171	
26DMH	Center	10	10	4	2														17.242	64
	End	8	8	2							2	2	2	2					5.857	
28DMH	Center	10	10	4	2	2													16.802	64
	End	8	8	2							2	2	2	2	2				4.302	
30DMH	Center	10	10	4	2	2	2												16.288	64
	End	8	8	2							2	2	2	2	2	2			3.088	
32DMH	Center	10	10	4	2	2	2	2											15.713	62
	End	8	8	2							2	2	2	2	2	2	2		2.151	

Table 3.4.3.1-6

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LRFD 54" I-Beam Simple Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
18SU	Center	2	4	4	4	2	2												18.303	-
	End	2	4	4	4	2	2												18.303	
20SS	Center	2	4	4	4	2	2	2											17.570	-
	End	2	4	4	4	2	2	2											17.570	
20DS	Center	10	10																21.970	50
	End	10	8								2								17.270	
22DS	Center	10	10	2															21.697	55
	End	10	8								2	2							13.515	
24DS	Center	10	10	4															21.470	59
	End	10	8	2							2	2							13.970	
26DS	Center	10	10	6															21.278	62
	End	8	8	4							2	2	2						10.893	
28DS	Center	10	10	8															21.113	66
	End	8	8	6							2	2	2						11.470	
30DS	Center	10	10	8	2														20.837	75
	End	10	8	6							2	2	2						12.237	
32DSH	Center	10	10	8	4														20.595	67
	End	8	8	6	2						2	2	2	2					9.845	

Table 3.4.3.1-7

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LRFD 54" I-Beam Multi-Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
18SU	Center	2	4	4	4	2	2												18.303	-
	End	2	4	4	4	2	2												18.303	
18DM	Center	10	8																22.081	44
	End	10	6								2								16.859	
20DM	Center	10	10																21.970	50
	End	8	8								2	2							12.570	
22DM	Center	10	10	2															21.697	55
	End	8	8								2	2	2						9.425	
24DMH	Center	10	10	2	2														21.303	53
	End	10	8								2	2	2						10.553	
26DMH	Center	10	10	4	2														21.124	58
	End	8	8	2							2	2	2	2					7.893	
28DMH	Center	10	10	4	2	2													20.684	59
	End	8	8	2							2	2	2	2	2				6.041	
30DMH	Center	10	10	4	2	2	2												20.170	59
	End	8	8	2							2	2	2	2	2	2			4.570	
32DMH	Center	10	10	4	2	2	2	2											19.595	58
	End	8	8	2							2	2	2	2	2	2	2		3.408	

Table 3.4.3.1-8

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LRFD 63" Bulb T-Beam Simple Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
20SU	Center	4	6	6	4														27.120
	End	4	6	6	4														27.120
22SS	Center	4	6	6	4	2													26.665
	End	4	6	6	4	2													26.665
22DS	Center	12	10																29.211
	End	12	8								2								24.120
24DS	Center	12	12																29.120
	End	12	10								2								24.453
26DS	Center	12	12	2															28.889
	End	12	10								2	2							20.582
28DS	Center	12	12	4															28.691
	End	12	10	2							2	2							20.977
30DS	Center	12	12	6															28.520
	End	12	10	4							2	2							21.320
32DS	Center	12	12	8															28.370
	End	10	10	6							2	2	2						18.245
34DSH	Center	12	12	8	2														28.120
	End	10	10	6							2	2	2	2					15.885
36DSH	Center	12	12	8	2	2													27.787
	End	10	10	6							2	2	2	2	2				13.898

Table 3.4.3.1-9

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LRFD 63" Bulb T-Beam Multi-Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
20SU	Center	4	6	6	4														27.120
	End	4	6	6	4														27.120
20DM	Center	12	8																29.320
	End	12	6								2								23.720
22DM	Center	12	10																29.211
	End	10	8								2	2							19.029
24DM	Center	12	10	2															28.953
	End	10	8								2	2	2						15.453
26DM	Center	12	10	2	2														28.582
	End	10	8								2	2	2	2					12.582
28DMH	Center	12	10	2	2	2													28.120
	End	12	8								2	2	2	2					13.834
30DMH	Center	12	10	2	2	2	2												27.587
	End	12	8								2	2	2	2	2				11.587
32DMH	Center	12	10	2	2	2	2	2											26.995
	End	12	8								2	2	2	2	2	2			9.745
34DMH	Center	12	10	2	2	2	2	2	2										26.355
	End	12	8								2	2	2	2	2	2	2		8.238
36DMH	Center	12	12	2	2	2	2	2	2										26.453
	End	10	10								2	2	2	2	2	2	2	2	6.898

Table 3.4.3.1-10

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LRFD 72" Bulb T-Beam Simple Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
22SU	Center	6	4	4	2	2	2	2											30.236	-
	End	6	4	4	2	2	2	2											30.236	
24SS	Center	6	4	4	2	2	2	2	2										29.433	-
	End	6	4	4	2	2	2	2	2										29.433	
24DS	Center	12	12																33.600	48
	End	12	10								2								28.183	
26DS	Center	12	12	2															33.369	55
	End	12	10								2	2							23.677	
28DS	Center	12	12	4															33.171	60
	End	12	10	2							2	2							24.171	
30DS	Center	12	12	6															33.000	65
	End	12	10	4							2	2							24.600	
32DS	Center	12	12	8															32.850	70
	End	12	10	6							2	2							24.975	
34DS	Center	12	12	8	2														32.600	82
	End	12	10	6							2	2	2						21.835	
36DSH	Center	12	12	8	4														32.378	71
	End	10	10	6	2						2	2	2	2					18.822	
38DSH	Center	12	12	8	4	2													32.074	72
	End	10	10	6	2						2	2	2	2	2				16.547	

Table 3.4.3.1-11

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

LRFD 72" Bulb T-Beam Multi-Span Strand Patterns

Strand Pattern	Location	Row Numbers																e (Inches)	Min. Span (Feet)
		1	2	3	4	5	6	7	8		1T	2T	3T	4T	5T	6T	7T	8T	
22SU	Center	6	4	4	2	2	2	2											30.236	-
	End	6	4	4	2	2	2	2											30.236	
22DM	Center	12	10																33.691	38
	End	12	8								2								27.782	
24DM	Center	12	12																33.600	48
	End	10	10								2	2							22.767	
26DM	Center	12	12	2															33.369	55
	End	10	10								2	2	2						18.831	
28DM	Center	12	12	2	2														33.029	59
	End	10	10								2	2	2	2					15.600	
30DM	Center	12	12	2	2	2													32.600	62
	End	10	10								2	2	2	2	2				12.933	
32DMH	Center	12	12	2	2	2	2												32.100	58
	End	12	10								2	2	2	2	2				14.288	
34DMH	Center	12	12	2	2	2	2	2											31.541	59
	End	12	10								2	2	2	2	2	2			12.129	
36DMH	Center	12	12	2	2	2	2	2	2										30.933	59
	End	12	10								2	2	2	2	2	2	2		10.322	
38DMH	Center	12	12	4	2	2	2	2	2										30.916	64
	End	10	10	2							2	2	2	2	2	2	2	2	8.600	

Table 3.4.3.1-12

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL27-1830 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
12B-2T-0db-0d	Ctr.	8			2			2									2						30
	End	8			2			2									2						
14B-2T-0db-0d	Ctr.	10			2			2									2						38
	End	10			2			2									2						
16B-2T-2db-0d	Ctr.	10			4			2									2						42
	End	10	2	A	4			2									2						
18B-2T-0db-2d	Ctr.	10			6			2									2						45
	End	10			6												2	2					
20B-2T-0db-2d	Ctr.	10			6			4									2						46
	End	10			6			2									2	2					
22B-2T-0db-2d	Ctr.	10			8			4									2						49
	End	10			8			2									2	2					
24B-2T-0db-2d	Ctr.	12			8			2			2						2						52
	End	12			8			2									2	2					
26B-2T-2db-2d	Ctr.	12			8			4			2						2						53
	End	12	2	A	8			4									2	2					

A = lesser of [0.15 x span length] or [0.4 x span length - strand development length]

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Table 3.4.3.1-13

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL36-2438 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
26B-4T-0db-0d	Ctr.	12			8			4			2						2	2					40
	End	12			8			4			2						2	2					
26B-2T-0db-2d	Ctr.	12			8			6									2						60
	End	12			8			4									2	2					
28B-2T-2db-2d	Ctr.	12			10			6									2						63
	End	12	2	A	10			4									2	2					
30B-2T-2db-2d	Ctr.	14			8			8									2						65
	End	14	2	A	8			6									2	2					
32B-2T-4db-2d	Ctr.	14			10			8									2						68
	End	14	4	A	10			6									2	2					
34B-2T-2db-4d	Ctr.	18			10			4			2						2						72
	End	18	2	A	10			2									2	2	2				
38B-2T-2db-4d	Ctr.	16			14			6			2						2						75
	End	16	2	A	14			4									2	2	2				
42B-2T-4db-4d	Ctr.	16			16			8			2						2						78
	End	16	4	A	16			6									2	2	2				

A = lesser of [0.15 x span length] or [0.4 x span length - strand development length]

Table 3.4.3.1-14

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HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL36-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																				Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
26B-4T-0db-0d	Ctr.	12			6			6			2						2	2					40
	End	12			6			6			2						2	2					
26B-2T-0db-2d	Ctr.	14			6			6									2						62
	End	14			6			4									2	2					
28B-2T-2db-2d	Ctr.	12			10			6									2						63
	End	12	2	A	10			4									2	2					
30B-2T-2db-2d	Ctr.	14			10			6									2						67
	End	14	2	A	10			4									2	2					
34B-2T-4db-2d	Ctr.	14			12			8									2						70
	End	14	4	A	12			6									2	2					
38B-2T-2db-4d	Ctr.	16			12			6			4						2						73
	End	16	2	A	12			4			2						2	2	2				
42B-2T-2db-4d	Ctr.	16			12			10			4						2						75
	End	16	2	A	12			8			2						2	2	2				
44B-2T-4db-6d	Ctr.	18			14			6			4	2					2						82
	End	18	4	A	14			4			2						2	2	2	2			

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]

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Table 3.4.3.1-15

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL45-2438 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
32B-4T-0db-0d	Ctr.	12			10			6			4						2	2					50
	End	12			10			6			4						2	2					
32B-2T-2db-2d	Ctr.	12			12			4			4						2						70
	End	12	2	A	12			4			2						2	2					
34B-2T-4db-2d	Ctr.	14			12			4			4						2						74
	End	14	4	A	12			4			2						2	2					
36B-2T-4db-2d	Ctr.	14			14			4			4						2						76
	End	14	4	A	14			4			2						2	2					
38B-2T-0db-4d	Ctr.	16			14			6			2						2						85
	End	16			14			4									2	2	2				
40B-2T-2db-4d	Ctr.	18			12			6			4						2						87
	End	18	2	A	12			4			2						2	2	2				
42B-2T-4db-4d	Ctr.	18			16			4			4						2						90
	End	18	4	A	16			2			2						2	2	2				
46B-2T-2db-6d	Ctr.	18			18			6			2	2					2						92
	End	18	2	A	18			4									2	2	2	2			

A = lesser of [0.15 x span length] or [0.4 x span length - strand development length]

Table 3.4.3.1-16

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HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL45-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
26B-4T-0db-0d	Ctr.	14			10			2									2	2					50
	End	14			10			2									2	2					
28B-2T-4db-0d	Ctr.	14			6			6			2						2						66
	End	14	4	A	6			6			2						2						
30B-2T-2db-2d	Ctr.	14			10			6									2						76
	End	14	2	A	10			4									2	2					
32B-2T-4db-2d	Ctr.	16			14			2									2						81
	End	16	4	A	14												2	2					
36B-2T-0db-4d	Ctr.	18			10			4			4						2						82
	End	18			10			2			2						2	2	2				
40B-2T-2db-4d	Ctr.	18			14			4			4						2						87
	End	18	2	A	14			2			2						2	2	2				
44B-2T-4db-4d	Ctr.	18			18			4			4						2						91
	End	18	4	A	18			2			2						2	2	2				
48B-2T-4db-6d	Ctr.	18			18			8			2	2					2						93
	End	18	4	A	18			6									2	2	2	2			

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]

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Table 3.4.3.1-17

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL54-2438 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
34B-4T-0db-0d	Ctr.	12			12			6			4						2	2					60
	End	12			12			6			4						2	2					
34B-2T-4db-2d	Ctr.	14			12			8									2						86
	End	14	4	A	12			6									2	2					
36B-2T-4db-2d	Ctr.	14			14			8									2						89
	End	14	4	A	14			6									2	2					
38B-2T-0db-4d	Ctr.	16			16			4			2						2						92
	End	16			16			2									2	2	2				
40B-2T-0db-4d	Ctr.	16			16			4			4						2						93
	End	16			16			2			2						2	2	2				
42B-2T-0db-4d	Ctr.	16			16			6			4						2						95
	End	16			16			4			2						2	2	2				
44B-2T-4db-4d	Ctr.	18			18			6			2						2						99
	End	18	4	A	18			4									2	2	2				
48B-2T-8db-4d	Ctr.	18			18			10			2						2						102
	End	18	4	B	18	4	A	8									2	2	2				

A = lesser of [0.15 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter

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Table 3.4.3.1-18

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL54-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
32B-4T-0db-0d	Ctr.	10			10			8			4						2	2					60
	End	10			10			8			4						2	2					
32B-2T-2db-2d	Ctr.	18			8			6									2						85
	End	18	2	A	8			4									2	2					
34B-2T-4db-2d	Ctr.	18			14			2									2						89
	End	18	4	A	14												2	2					
36B-2T-6db-2d	Ctr.	18			16			2									2						91
	End	18	4	B	16	2	A										2	2					
40B-2T-8db-2d	Ctr.	16			16			8									2						93
	End	16	4	B	16	4	A	6									2	2					
44B-2T-6db-4d	Ctr.	18			18			6			2						2						98
	End	18	4	B	18	2	A	4									2	2	2				
48B-2T-8db-4d	Ctr.	18			18			6			6						2						100
	End	18	4	B	18	4	A	4			4						2	2	2				
52B-2T-8db-6d	Ctr.	18			18			12			2	2					2						103
	End	18	4	B	18	4	A	10									2	2	2	2			

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter

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Table 3.4.3.1-19

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL63-2438 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
36B-4T-0db-0d	Ctr.	14			8			6			4	4					2	2					70
	End	14			8			6			4	4					2	2					
36B-2T-4db-2d	Ctr.	16			10			10									2						94
	End	16	4	A	10			8									2	2					
38B-2T-6db-2d	Ctr.	16			14			8									2						98
	End	16	4	B	14	2	A	6									2	2					
40B-2T-8db-2d	Ctr.	16			14			10									2						100
	End	16	4	B	14	4	A	8									2	2					
42B-2T-8db-2d	Ctr.	14			14			14									2						100
	End	14	4	B	14	4	A	12									2	2					
44B-2T-2db-4d	Ctr.	14			14			14			2						2						102
	End	14	2	A	14			12									2	2	2				
46B-2T-6db-4d	Ctr.	18			18			8			2						2						107
	End	18	4	B	18	2	A	6									2	2	2				
50B-2T-12db-4d	Ctr.	18			18			12			2						2						111
	End	18	4	C	18	4	B	10	4	A							2	2	2				

A = lesser of [0.15 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter
C = B - 60 x strand diameter

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Table 3.4.3.1-20

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL63-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
34B-4T-0db-0d	Ctr.	12			12			6			2	2					2	2					70
	End	12			12			6			2	2					2	2					
34B-2T-4db-2d	Ctr.	18			14			2									2						93
	End	18	4	A	14												2	2					
36B-2T-4db-2d	Ctr.	16			16			4									2						95
	End	16	4	A	16			2									2	2					
38B-2T-8db-2d	Ctr.	18			18			2									2						99
	End	18	4	B	18	4	A										2	2					
42B-2T-10db-2d	Ctr.	16			16			10									2						101
	End	16	4	C	16	4	B	8	2	A							2	2					
46B-2T-8db-4d	Ctr.	18			18			8			2						2						105
	End	18	4	B	18	4	A	6									2	2	2				
50B-2T-10db-4d	Ctr.	18			18			12			2						2						109
	End	18	4	C	18	4	B	10	2	A							2	2	2				
54B-2T-6db-6d	Ctr.	18			18			14			2	2					2						111
	End	18	4	B	18	2	A	12									2	2	2	2			

Table 3.4.3.1-21

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter
C = B - 60 x strand diameter

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IL72-2438 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
40B-4T-4db-0d	Ctr.	14			10			6			4	2	2	2			2	2					80
	End	14	4	A	10			6			4	2	2	2			2	2					
40B-2T-8db-2d	Ctr.	14			14			12									2						104
	End	14	4	B	14	4	A	10									2	2					
42B-2T-4db-4d	Ctr.	18			18			4			2						2						109
	End	18	4	A	18			2									2	2	2				
44B-2T-4db-4d	Ctr.	18			18			6			2						2						111
	End	18	4	A	18			4									2	2	2				
46B-2T-6db-4d	Ctr.	18			18			8			2						2						113
	End	18	4	B	18	2	A	6									2	2	2				
48B-2T-8db-4d	Ctr.	18			18			10			2						2						115
	End	18	4	B	18	4	A	8									2	2	2				
50B-2T-2db-6d	Ctr.	18			18			10			2	2					2						116
	End	18	2	A	18			8									2	2	2	2			
54B-2T-4db-6d	Ctr.	18			18			14			2	2					2						120
	End	18	4	A	18			12									2	2	2	2			

A = lesser of [0.15 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter

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Table 3.4.3.1-22

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL72-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
36B-4T-4db-0d	Ctr.	14			10			8			4						2	2					80
	End	14	4	A	10			8			4						2	2					
36B-2T-4db-2d	Ctr.	16			14			6									2						99
	End	16	4	A	14			4									2	2					
38B-2T-8db-2d	Ctr.	18			18			2									2						104
	End	18	4	B	18	4	A										2	2					
42B-2T-4db-4d	Ctr.	18			18			4			2						2						107
	End	18	4	A	18			2									2	2	2				
46B-2T-8db-4d	Ctr.	18			18			8			2						2						111
	End	18	4	B	18	4	A	6									2	2	2				
50B-2T-12db-4d	Ctr.	18			18			12			2						2						125
	End	18	4	C	18	4	B	10	4	A							2	2	2				
54B-2T-8db-6d	Ctr.	18			18			14			2	2					2						127
	End	18	4	B	18	4	A	12									2	2	2	2			
58B-2T-8db-6d	Ctr.	18			18			16			4	2					2						130
	End	18	4	B	18	4	A	14			2						2	2	2	2			

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter
C = B - 60 x strand diameter

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Table 3.4.3.1-23

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

IL81-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																				Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
40B-4T-4db-0d	Ctr.	14			12			8			6						2	2					100
	End	14	4	A	12			8			6						2	2					
40B-2T-8db-2d	Ctr.	18			18			4									2						111
	End	18	4	B	18	4	A	2									2	2					
42B-2T-2db-4d	Ctr.	18			18			4			2						2						112
	End	18	2	A	18			2									2	2	2				
46B-2T-4db-4d	Ctr.	18			18			8			2						2						126
	End	18	4	A	18			6									2	2	2				
50B-2T-8db-4d	Ctr.	18			18			12			2						2						130
	End	18	4	B	18	4	A	10									2	2	2				
54B-2T-4db-6d	Ctr.	18			18			14			2	2					2						134
	End	18	4	A	18			12									2	2	2	2			
58B-2T-8db-6d	Ctr.	18			18			16			4	2					2						137
	End	18	4	B	18	4	A	14			2						2	2	2	2			
62B-2T-8db-8d	Ctr.	18			18			16			6	2	2				2						139
	End	18	4	B	18	4	A	14			4						2	2	2	2	2		

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter

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Table 3.4.3.1-24

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IL90-3838 Simple Span Strand Patterns

Strand Pattern	Loc.	Row Numbers																					Min. Span (ft.)
		1B			2B			3B			4B	5B	6B	7B	8B		1T	2T	3T	4T	5T	6T	
		T	db	L (in.)	T	db	L (in.)	T	db	L (in.)													
44B-4T-8db-0d	Ctr.	16			12			8			8						2	2					110
	End	16	4	B	12	4	A	8			8						2	2					
44B-2T-2db-4d	Ctr.	18			16			6			4						2						128
	End	18	2	A	16			4			2						2	2	2				
46B-2T-6db-4d	Ctr.	18			18			8			2						2						132
	End	18	4	B	18	2	A	6									2	2	2				
50B-2T-2db-6d	Ctr.	18			18			10			2	2					2						135
	End	18	2	A	18			8									2	2	2	2			
54B-2T-6db-6d	Ctr.	18			18			14			2	2					2						140
	End	18	4	B	18	2	A	12									2	2	2	2			
58B-2T-10db-6d	Ctr.	18			18			16			4	2					2						143
	End	18	4	C	18	4	B	14	2	A	2						2	2	2	2			
62B-2T-8db-8d	Ctr.	18			18			16			6	2	2				2						146
	End	18	4	B	18	4	A	14			4						2	2	2	2	2		
66B-2T-8db-10d	Ctr.	18			18			16			8	2	2	2			2						150
	End	18	4	B	18	4	A	14			6						2	2	2	2	2	2	

A = lesser of [0.2 x span length] or [0.4 x span length - strand development length]
B = A - 60 x strand diameter
C = B - 60 x strand diameter

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Table 3.4.3.1-25

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3.4.3.2 Standard Beam Cross Sections and Typical Strand Layout

The six standard IDOT beam cross sections for PPC I-beams and bulb T-beams are given in Figures 3.4.3.2-1 through 3.4.2.2-6. Table 3.4.3.2-1 shows pertinent beam section properties and the dead weight of the various PPC I-beams and bulb T-beams. Figure 3.4.3.2-7 provides the dimensions and section properties of the 13 PPC IL-beam sections. The beam designation ILXX-YYZZ denotes the following: XX = beam depth; YY = top flange width and ZZ = bottom flange width.

Figures 3.4.3.2-8 through 3.4.3.2-9 illustrate the typical layout of strands along prestressed beams. Included in Figure 3.4.3.2-8 are hold down points for draped strands on I and Bulb-T beams.

Figure 3.4.3.2-10 illustrates how to detail a strand pattern on base sheets. The designer should note the presentation for partially debonded strands and straight top strands.

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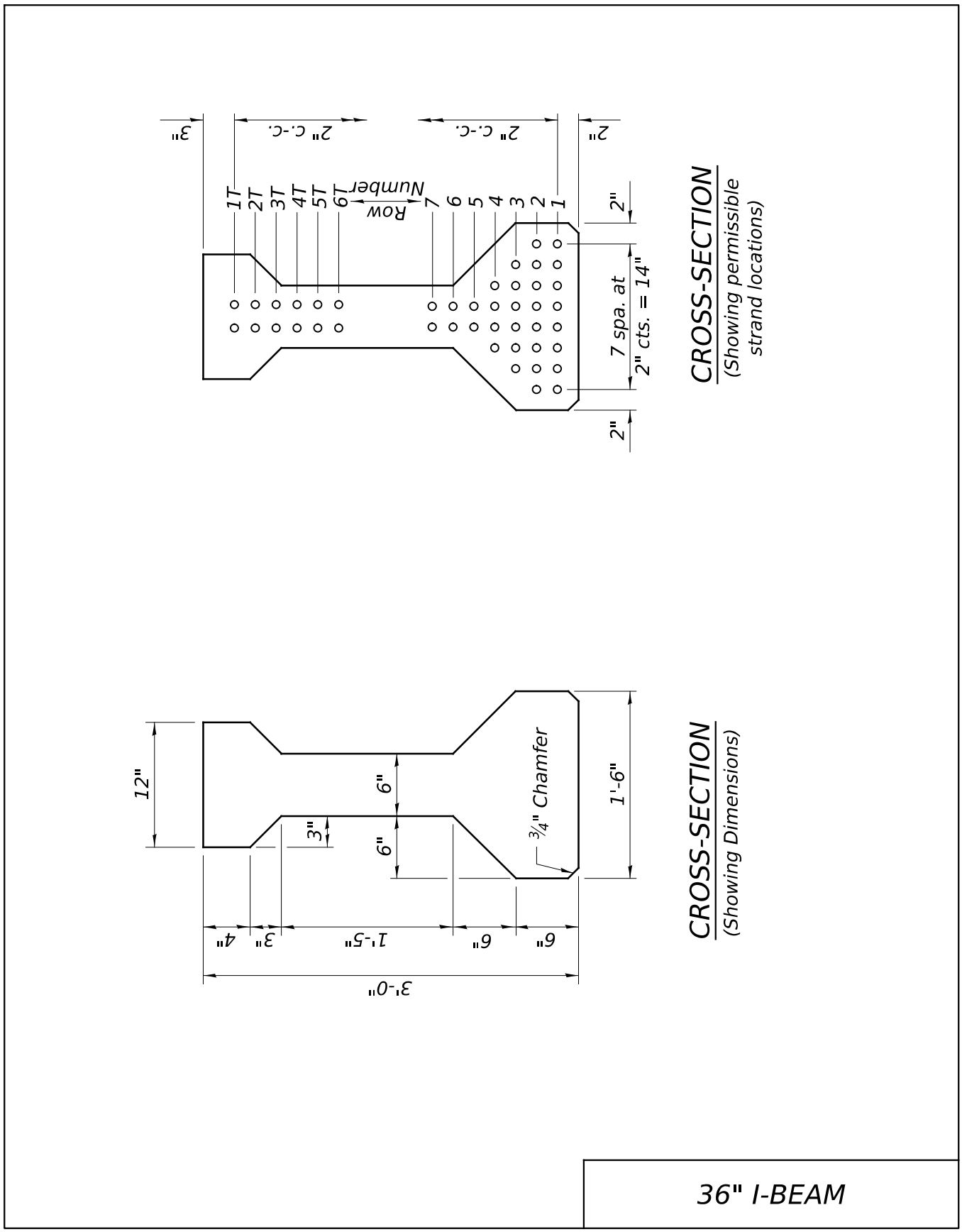
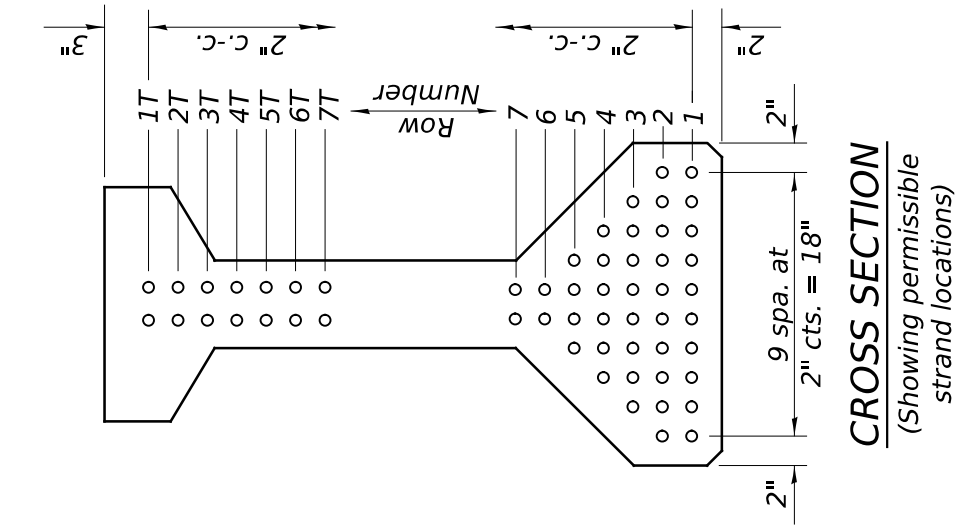
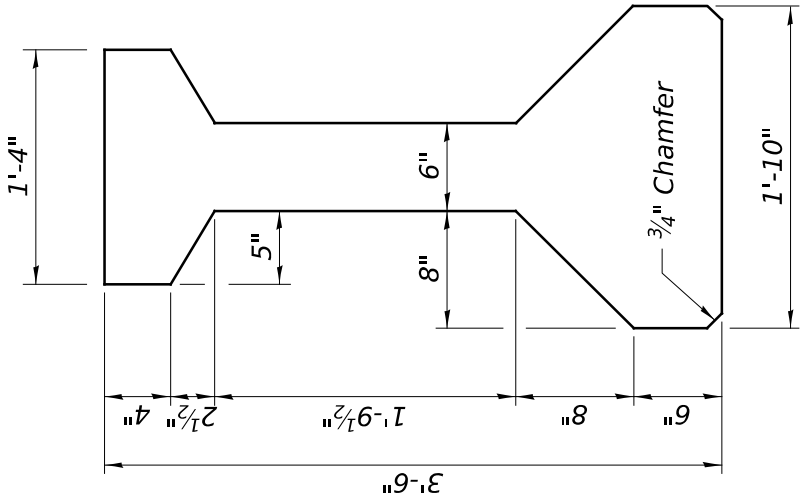


Figure 3.4.3.2-1

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CROSS SECTION
(Showing permissible strand locations)



CROSS-SECTION
(Showing Dimensions)

42" I-BEAM

Figure 3.4.3.2-2

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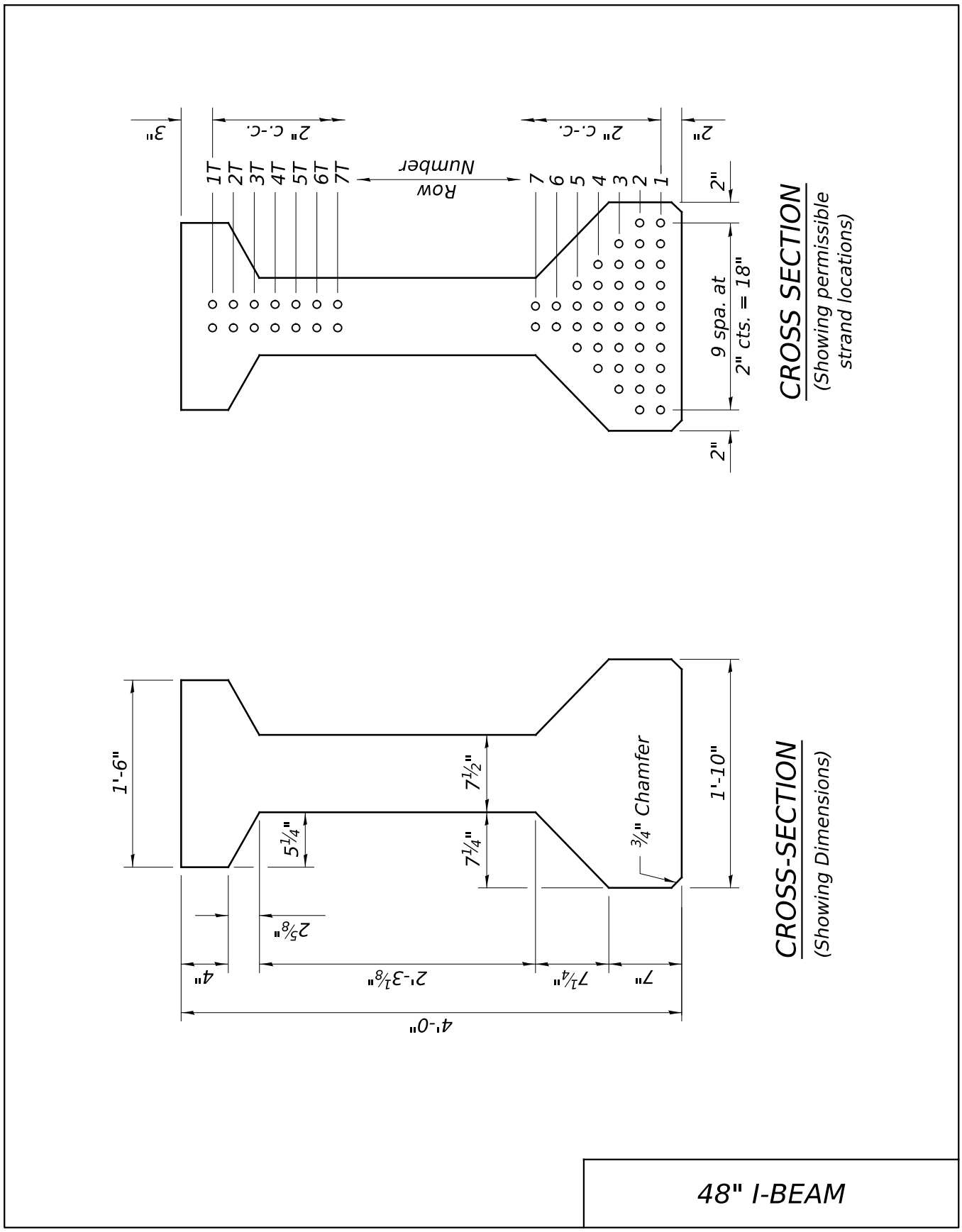


Figure 3.4.3.2-3

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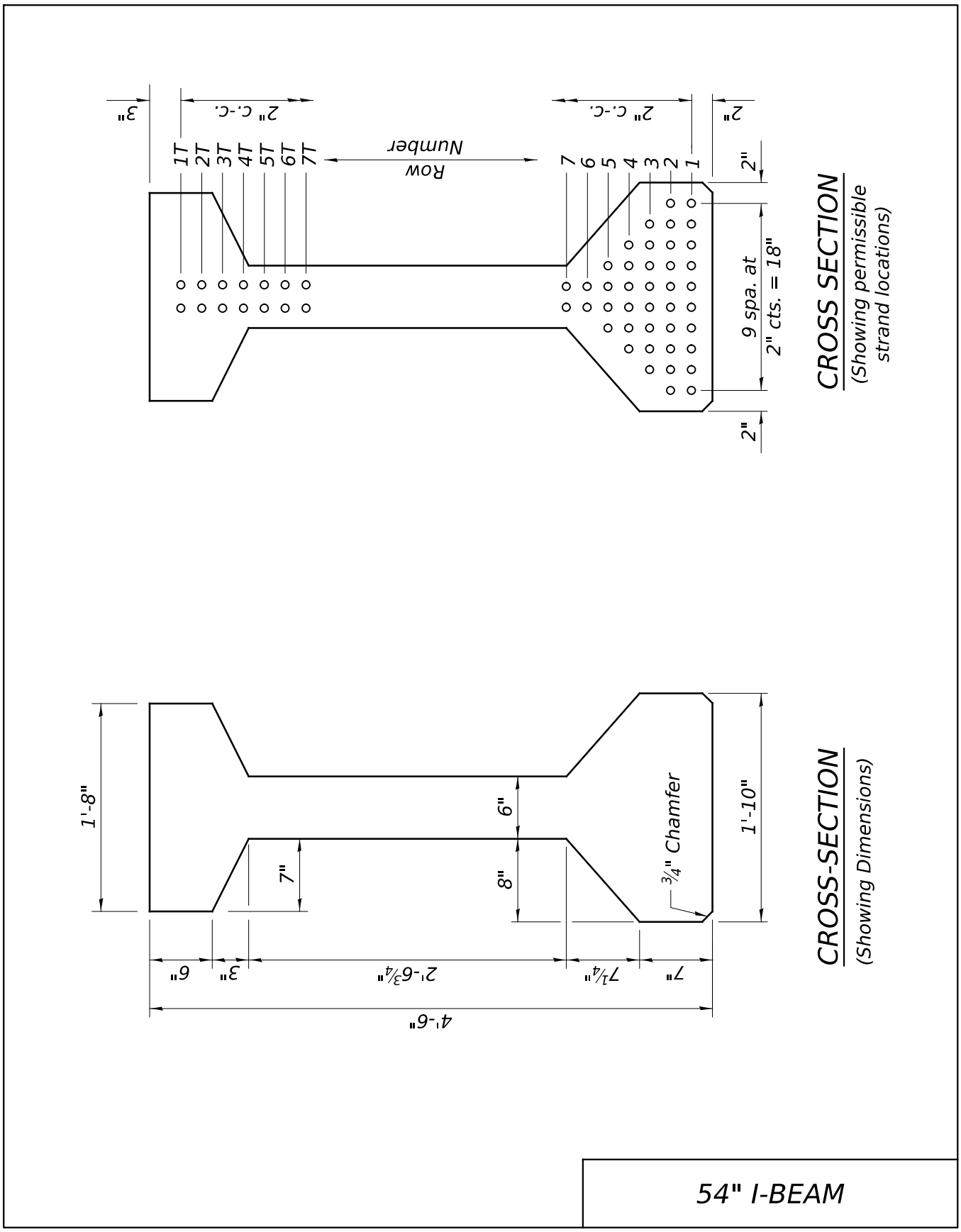


Figure 3.4.3.2-4

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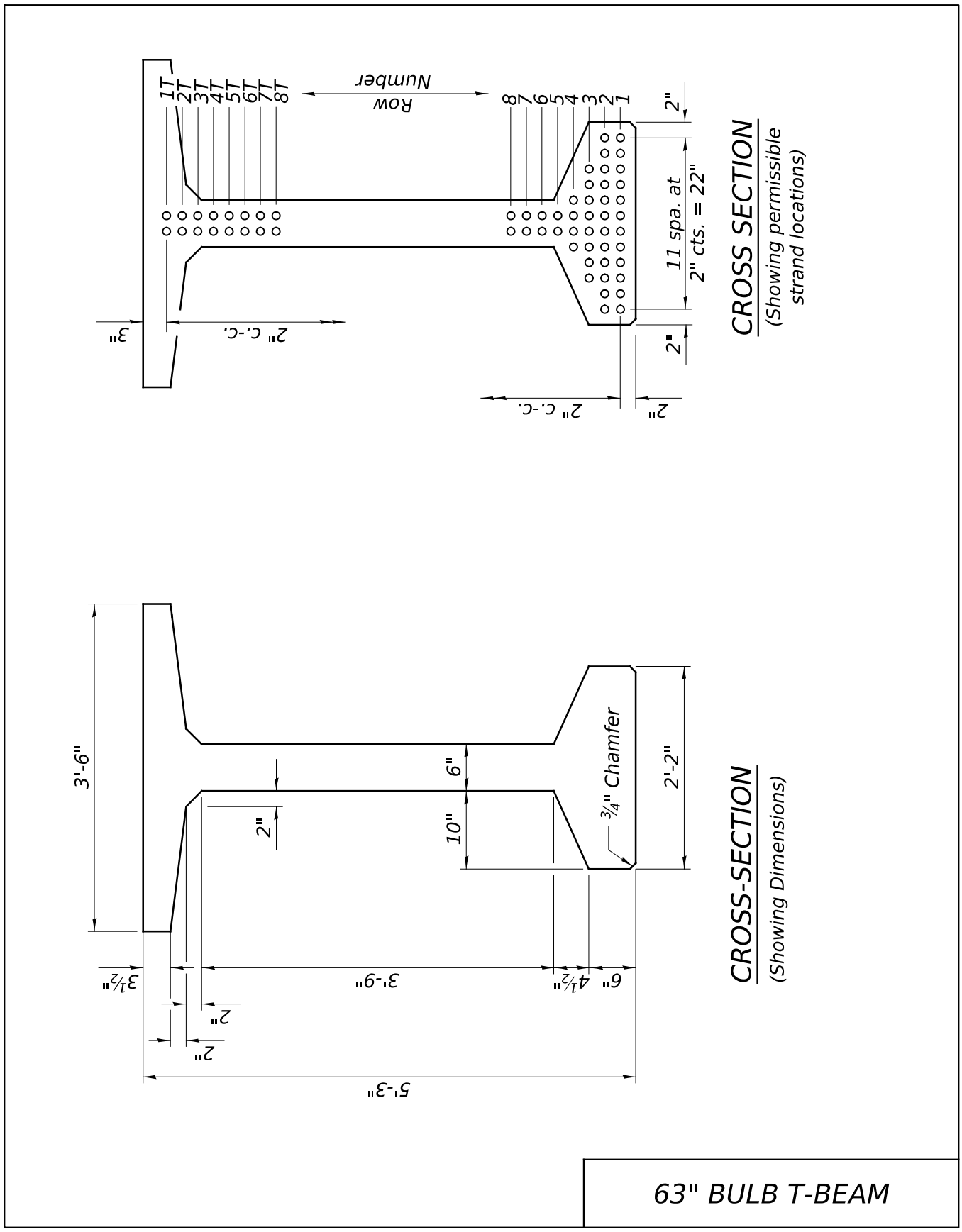


Figure 3.4.3.2-5

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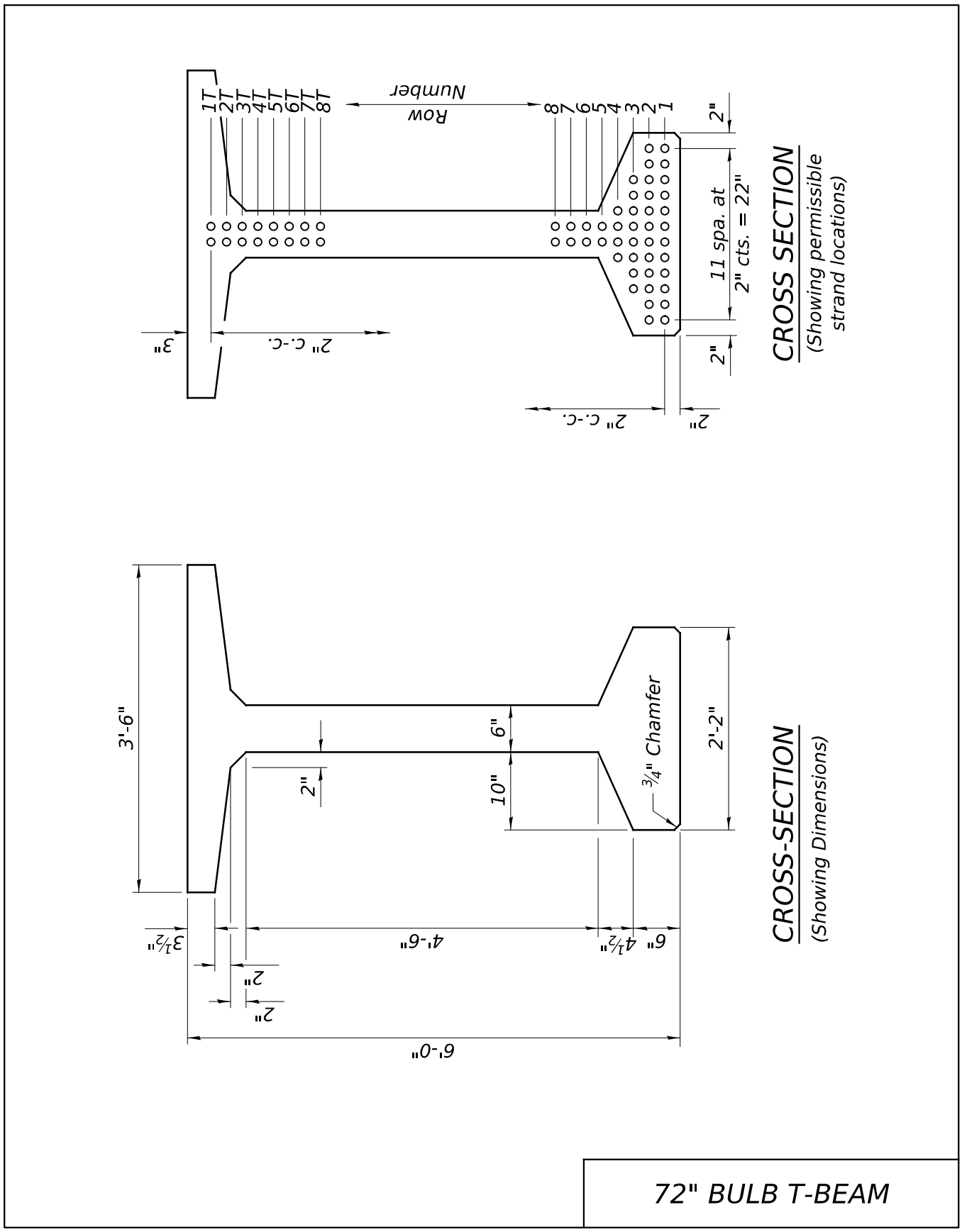


Figure 3.4.3.2-6

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Prestressed Beam Design Properties									
Beam		**Area**	**I_x**	**I_y**	**S_b**	**S_t**	**C_b**	**C_t**	**Weight**
Type	Size	($in.^2$)	($in.^4$)	($in.^4$)	($in.^3$)	($in.^3$)	(in.)	(in.)	(lbs/ft.)
I-Beam	36	357.0	48,648	-	3,165.1	2,358.1	15.37	20.63	375
	42	464.5	90,956	-	5,152.7	3,735.6	17.65	24.35	485
	48	569.8	144,117	-	6,834.1	5,355.1	21.09	26.91	595
	54	599.0	213,715	-	8,559.0	7,362.0	24.97	29.03	624
Bulb T-Beam	63	713.0	392,638	-	12,224.0	12,715.0	32.12	30.88	743
	72	767.0	545,894	-	14,915.0	15,421.0	36.60	35.40	799
IL-Beam	IL27-1830	457.9	33,879	20,442	3,060.4	2,126.7	11.07	15.93	477
	IL36-2438	728.0	100,433	50,889	6,832.2	4,715.2	14.70	21.30	759
	IL36-3838	805.4	124,639	69,530	7,563.0	6,385.2	16.48	19.52	840
	IL45-2438	791.0	182,623	51,146	10,045.3	6,809.2	18.18	26.82	825
	IL45-3838	868.4	223,604	69,787	11,004.1	9,060.1	20.32	24.68	905
	IL54-2438	854.0	295,427	51,403	13,551.7	9,174.8	21.80	32.20	890
	IL54-3838	931.4	357,078	70,044	14,730.9	11,998.6	24.24	29.76	971
	IL63-2438	917.0	441,689	51,661	17,294.0	11,791.0	25.54	37.46	956
	IL63-3838	994.4	527,741	70,302	18,687.7	15,182.4	28.24	34.76	1,036
	IL72-2438	980.0	624,180	51,918	21,237.8	14,648.7	29.39	42.61	1,021
	IL72-3838	1057.4	738,236	70,559	22,855.6	18,595.4	32.30	39.70	1,102
	IL81-3838	1120.4	991,193	70,816	27,223.1	22,229.0	36.41	44.59	1,168
	IL90-3838	1183.4	1,289,219	71,074	31,785.5	26,076.4	40.56	49.44	1,233

Table 3.4.3.2-1

Definitions for Table 3.4.3.2-1:

I_x = Moment of inertia of the prestressed beam about the x axis.

I_y = Moment of inertia of the prestressed beam about the y axis.

S_b = Non-composite section modulus for the bottom fiber of the prestressed beam.

S_t = Non-composite section modulus for the top fiber of the prestressed beam.

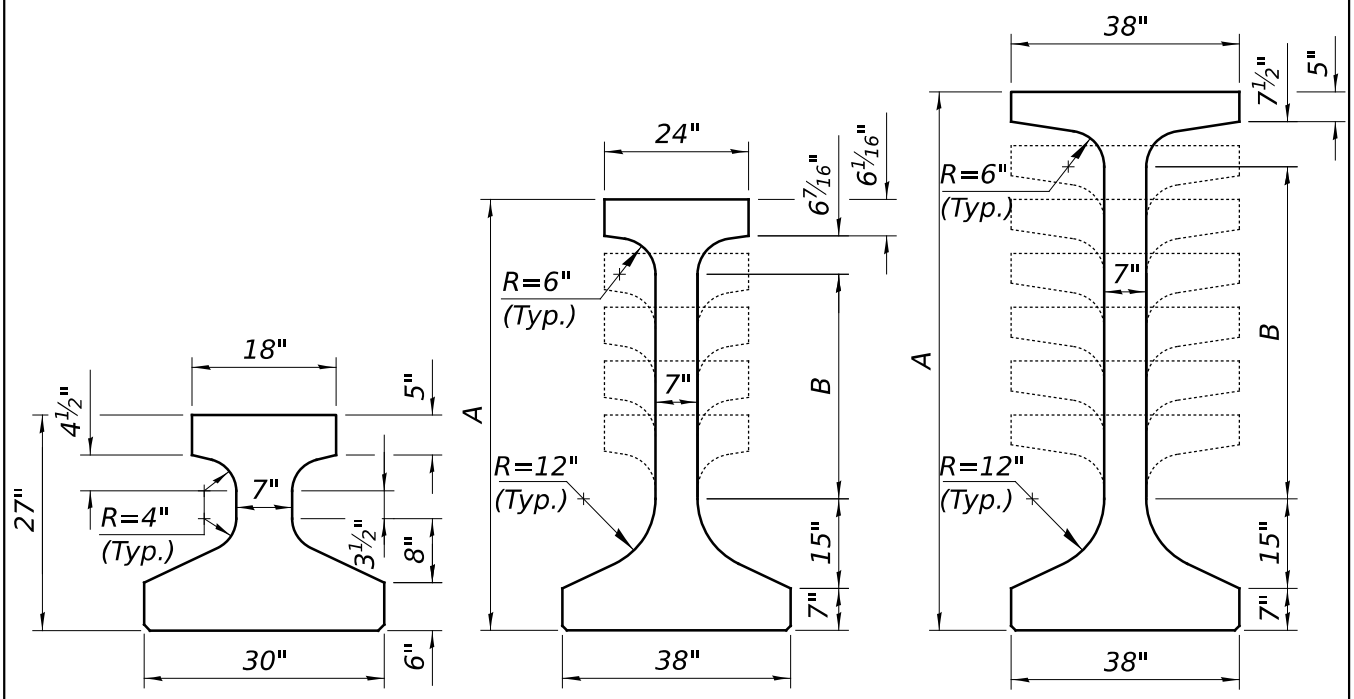
C_b = Distance from the centroid of the prestressed beam to the bottom of the beam.

C_t = Distance from the centroid of the prestressed beam to the top of the beam.

IL-Beam nomenclature: ILXX-YYZZ, where:

XX = Beam Depth YY = Top flange width ZZ = Bottom flange width

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Beam nomenclature: ILXX-YYZZ
XX=Beam depth
YY=Top flange width
ZZ=Bottom flange width

DIMENSIONS

Beam	A (in)	B (in)
IL36	36	1½
IL45	45	10½
IL54	54	19½
IL63	63	28½
IL72	72	37½
IL81	81	46½
IL90	90	55½

IL-BEAM SECTION PROPERTIES

Figure 3.4.3.2-7

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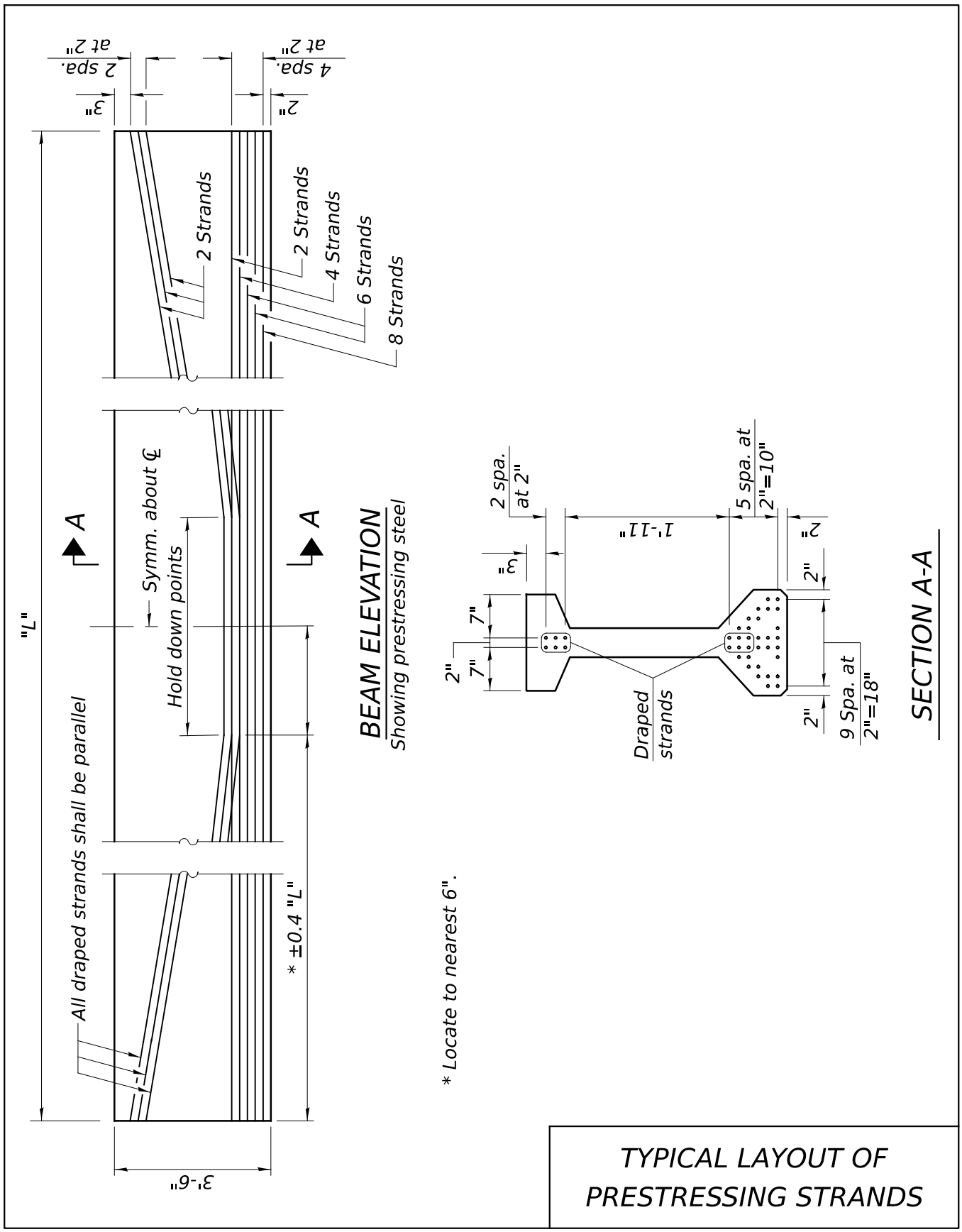


Figure 3.4.3.2-8

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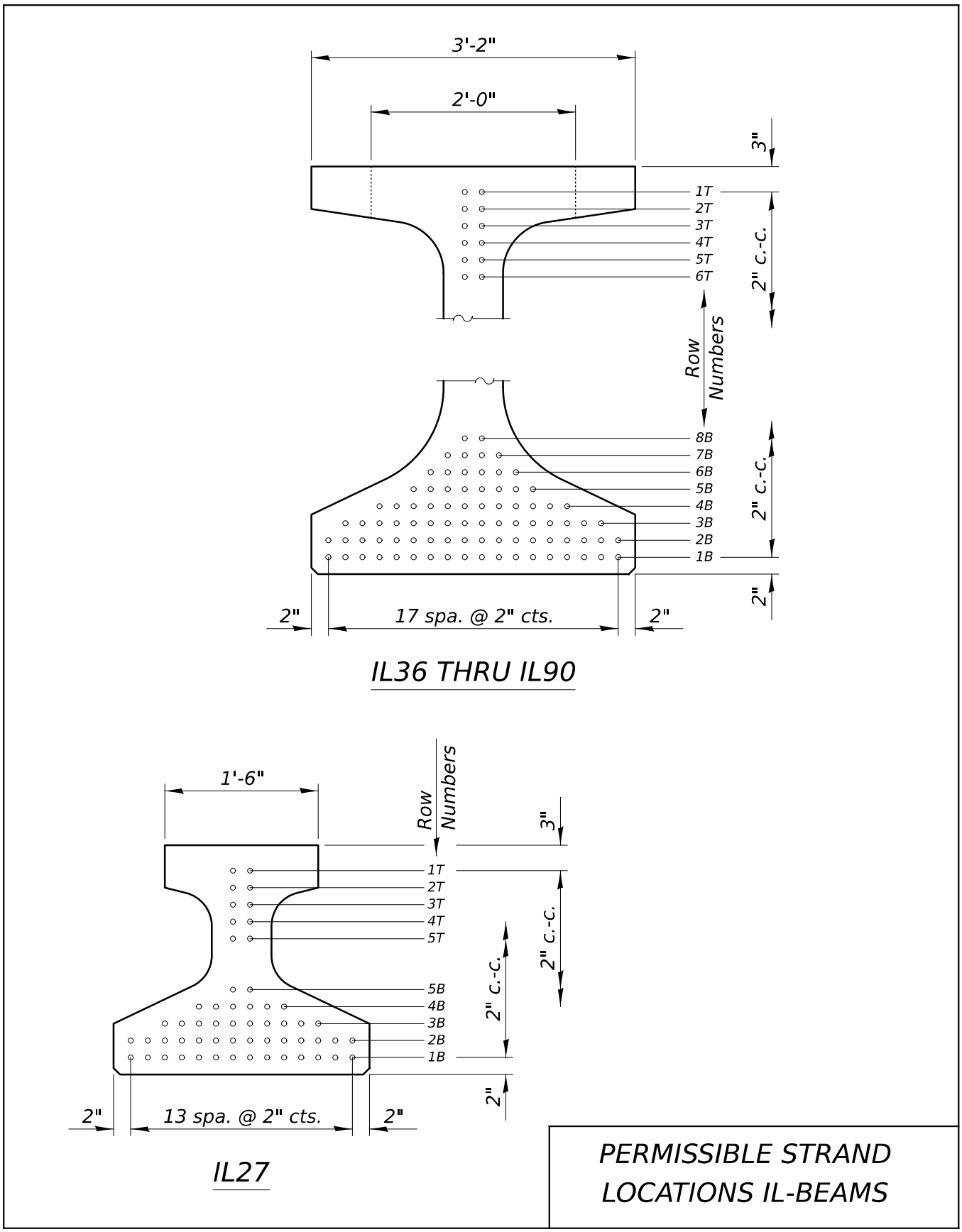


Figure 3.4.3.2-9

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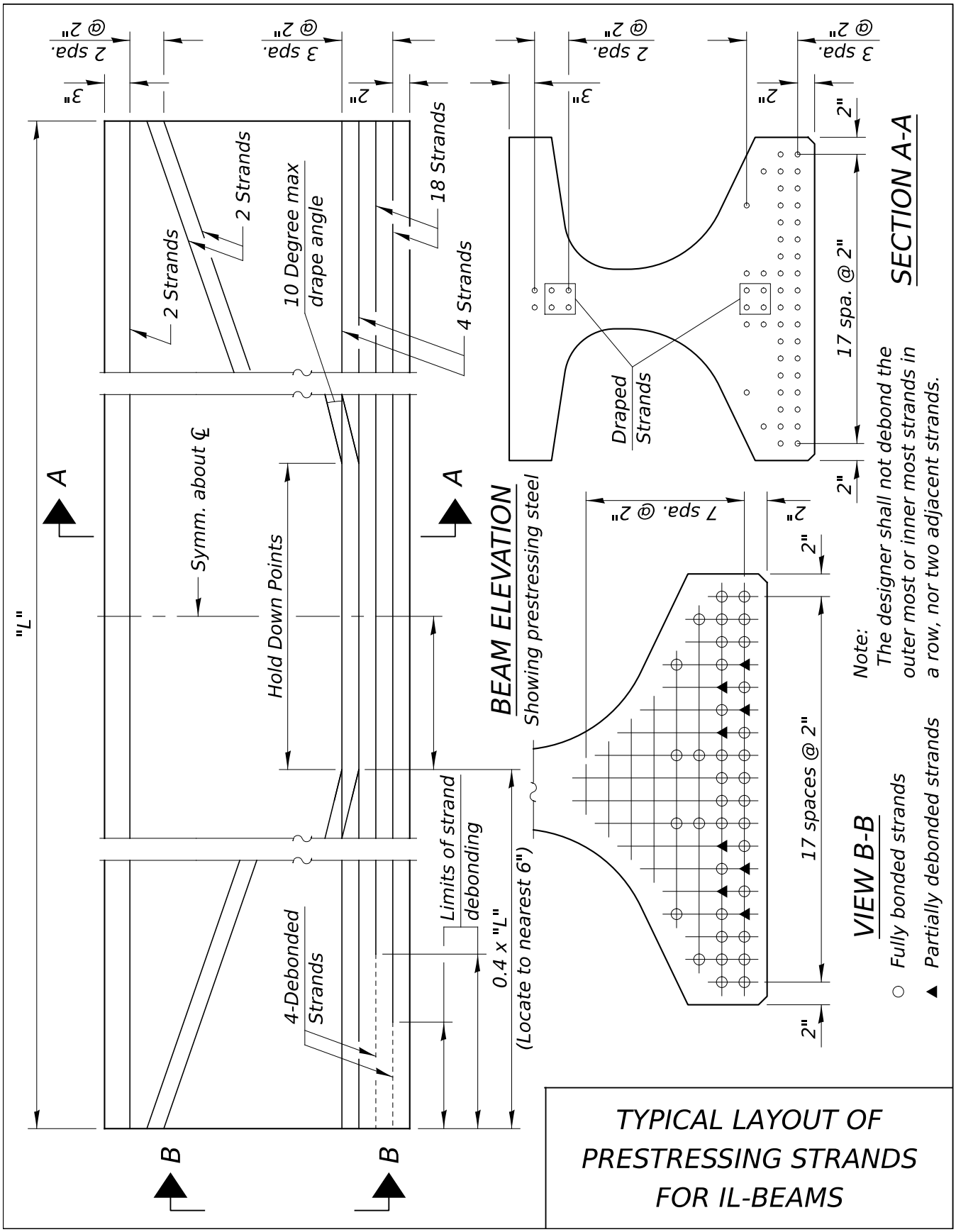


Figure 3.4.3.2-10

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3.4.3.3 Concrete Fillets over Prestressed Beams

The minimum fillet over any point on the prestressed beam shall be ½ inch. See also Figure 3.2.4-14. In locations where the fillet exceeds 3 ½ inches, additional reinforcement of #4 bars at 12 in. centers shall be detailed according to Figure 3.4.3.3-1.

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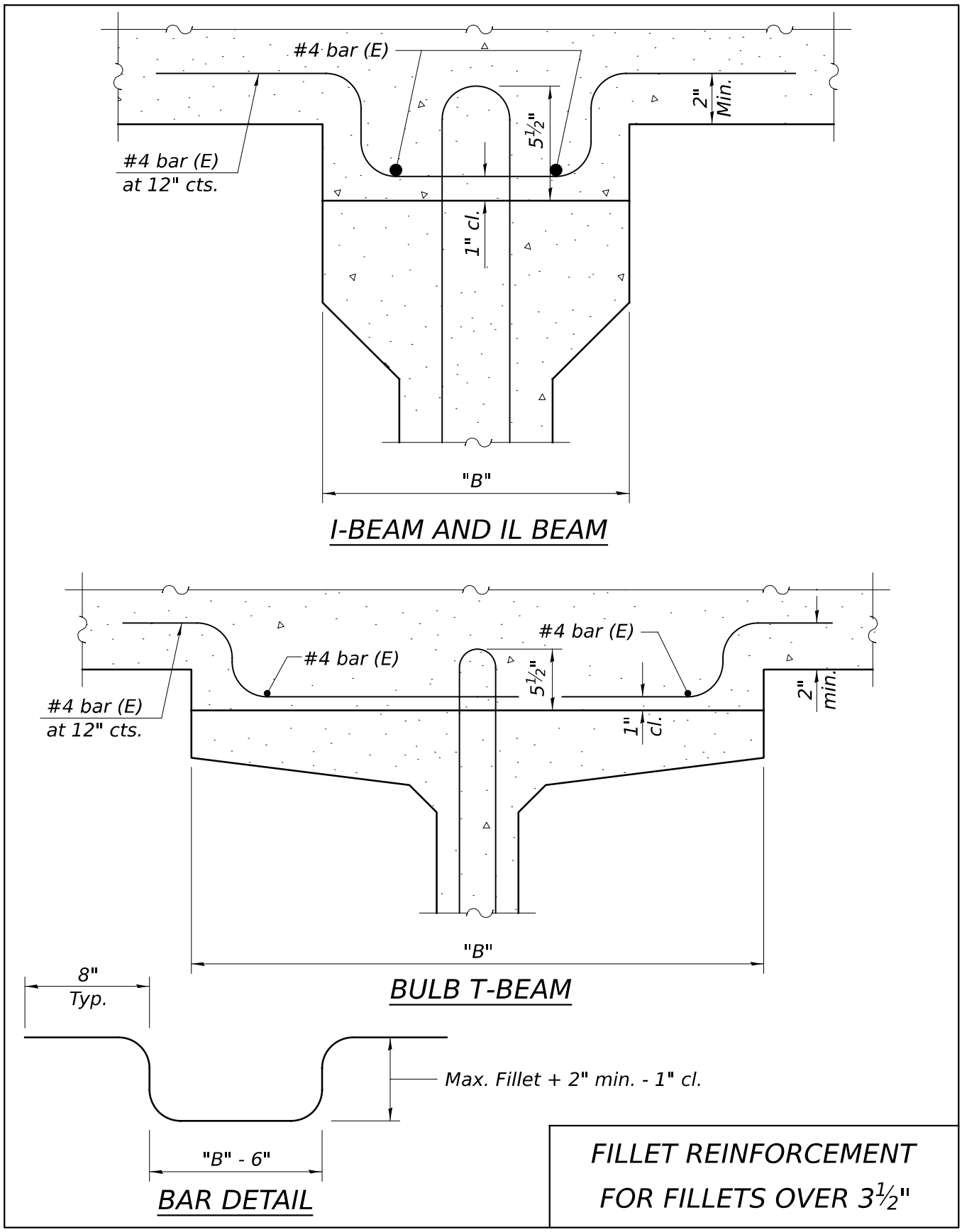


Figure 3.4.3.3-1

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3.4.4 Design for Shear

At end regions of prestressed beams, there are special requirements and details for splitting steel which are discussed in Section 3.4.7. Section 3.4.4.1 presents a brief discussion of transverse reinforcement design for ultimate strength using the AASHTO LRFD Bridge Design Specifications code. Standard transverse reinforcement patterns for PPC beams have also been developed by the Department and may be used for most typical bridges in lieu of design calculations. These patterns are presented in Section 3.4.4.2.

Transverse reinforcement spacing should be shown in three-inch increments (6 in., 9 in., 12 in. etc.).

3.4.4.1 Ultimate Strength Design of Primary Transverse Reinforcement

There are several options available in the AASHTO LRFD Bridge Design Specifications for shear design of prestressed beams. Either the strut-and-tie model or the sectional model (as discussed in the AASHTO LRFD Bridge Design Specifications) may be used in the design of transverse reinforcement for PPC beams. For either method, however, AASHTO LRFD Bridge Design Specifications Article 5.7.3 applies. This article deals with subjects such as minimum reinforcement, maximum spacing of reinforcement, permissible types of transverse reinforcement, etc.

While the standard transverse reinforcement patterns were generated using a conservative simplification of the procedures outlined in Appendix B5 of the AASHTO LRFD Bridge Design Specifications (see Design Guide 3.4 on the Bridges & Structures page), if detailed shear design calculations are required (i.e. the situation falls outside of the limits covered by the standard transverse reinforcement patterns given in Section 3.4.4.2), then the use of AASHTO LRFD Bridge Design Specifications Article 5.7.3.4.2 is preferred by the Department. The Department generally discourages use of the strut-and-tie model of AASHTO LRFD Bridge Design Specifications for typical designs.

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3.4.4.2 Standard Transverse Reinforcement Patterns for PPC I-beams and Bulb T-Beams

Standard AASHTO LRFD Bridge Design Specifications transverse reinforcement patterns are given in Table 3.4.4.2-1.

The standard transverse reinforcement patterns were developed based upon the criteria listed below. These patterns may be used by designers for all typical PPC I-beams and bulb T-beams within the span length and beam spacing limits shown in Table 3.4.4.2-1.

1. ½ in. diameter low-relaxation 7 wire strands with an ultimate strength, f_{pu}, of 270 ksi.
2. Specified concrete compressive strength for beams of 6 ksi.
3. HL-93 live loading using distribution methods outlined in Section 4.6.2.2 of the AASHTO LRFD Bridge Design Specifications.
4. 8 in. thick concrete deck.
5. 1 in. average fillet height for dead load only (not included in section properties).
6. 6 beam lines.
7. Standard 39” constant-slope concrete parapet with a weight of 0.525 k/ft.
8. 50 psf future wearing surface.
9. Grade 60 #4 reinforcement only.

When detailed transverse reinforcement design calculations are required, specified reinforcement spacings shall be limited to increments of 3 in. (e.g. 6, 9 and 12 in.) for constructability and economy. In addition, designers shall not specify transverse reinforcement spacings greater than those provided in Table 3.4.4.2-1. Small anomalies may occasionally occur between detailed transverse reinforcement design calculations using the simplifications outlined in Design Guide 3.4 and the spacings presented in Table 3.4.4.2-1. If a calculated spacing is less than 6 in., additional G_2 bars may be added to satisfy the transverse reinforcement requirements in such regions (see PPC I-beam and bulb T-beam Base Sheets in the “Beams-Prestressed Girders” IDOT CADD Bridge Cell Library).

3.4.4.3 Standard Transverse Reinforcement Patterns for PPC IL-Beams

IL-Beams use standardized epoxy coated welded wire reinforcement (WWR). This is intended to simplify fabrication and provide improved durability. Details of this reinforcement are given in the IL-beam base sheets.

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LRFD Standard Transverse Reinforcement Table

Limits for Standard Stirrup Patterns

Beam 36 in.	Span (ft.)	Beam Spacing from (ft.)	to (SS) (ft.)	to (CS) (ft.)
	30	4 1/2	9	---
	40	4 1/2	9	9
	50	4 1/2	9	9
	60	4 1/2	6	7
	70	4 1/2	---	5

Beam 42 in.	Span (ft.)	Beam Spacing from (ft.)	to (SS) (ft.)	to (CS) (ft.)
	50	4 1/2	9	---
	60	4 1/2	9	9
	70	4 1/2	7 1/2	8 1/2
	80	4 1/2	5 1/2	6 1/2
	90	4 1/2	---	5

Beam 48 in.	Span (ft.)	Beam Spacing from (ft.)	to (SS) (ft.)	to (CS) (ft.)
	60	4 1/2	9	9
	70	4 1/2	9	9
	80	4 1/2	7 1/2	9
	90	4 1/2	5 1/2	7

Beam 54 in.	Span (ft.)	Beam Spacing from (ft.)	to (SS) (ft.)	to (CS) (ft.)
	60	4 1/2	9	---
	70	4 1/2	9	9
	80	4 1/2	9	9
	90	4 1/2	7	8 1/2
	100	4 1/2	5	6 1/2
	110	4 1/2	--	5

Beam 63 in.	Span (ft.)	Beam Spacing from (ft.)	to (SS) (ft.)	to (CS) (ft.)
	70	4 1/2	9	---
	80	4 1/2	9	9
	90	4 1/2	9	9
	100	4 1/2	8	9
	110	4 1/2	6 1/2	8
	120	4 1/2	5	6 1/2
	130	4 1/2	--	5

Beam 72 in.	Span (ft.)	Beam Spacing from (ft.)	to (SS) (ft.)	to (CS) (ft.)
	80	4 1/2	9	---
	90	4 1/2	9	9
	100	4 1/2	9	9
	110	4 1/2	8	9
	120	4 1/2	7	8 1/2
	130	4 1/2	5 1/2	6 1/2
	140	4 1/2	--	5

Standard Stirrup Patterns

Beam	Location from (10th Pt.)	to (10th Pt.)	Bar Size (#)	Spacing (in.)
36 in.	0	0.3	4	6
	0.3	0.7	4	9
	0.7	1	4	6

Beam	Location from (10th Pt.)	to (10th Pt.)	Bar Size (#)	Spacing (in.)
42 in.	0	0.3	4	6
	0.3	0.7	4	9
	0.7	1	4	6

Beam	Location from (10th Pt.)	to (10th Pt.)	Bar Size (#)	Spacing (in.)
48 in.	0	0.3	4	6
	0.3	0.7	4	12
	0.7	1	4	6

Beam	Location from (10th Pt.)	to (10th Pt.)	Bar Size (#)	Spacing (in.)
54 in.	0	0.3	4	6
	0.3	0.7	4	12
	0.7	1	4	6

Beam	Location from (10th Pt.)	to (10th Pt.)	Bar Size (#)	Spacing (in.)
63 in.	0	0.3	4	6
	0.3	0.7	4	9
	0.7	1	4	6

Beam	Location from (10th Pt.)	to (10th Pt.)	Bar Size (#)	Spacing (in.)
72 in.	0	0.3	4	6
	0.3	0.7	4	9
	0.7	1	4	6

Notes:
1. SS = Simple Spans. CS = Continuous Spans.
2. Patterns outside upper or lower span or beam spacing ranges for simple and continuous beams shall be calculated.

Table 3.4.4.2-1

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3.4.5 Camber and Deflections

Camber is the result of the difference between the upward deflection caused by prestressing forces and the downward deflection due to the weight of the beam and the slab. Camber shall be considered when determining seat elevations. The top of the beam shall be set to provide a minimum positive fillet as defined in Section 3.4.3.3.

The Department updated the camber calculations and strand patterns for IL-Beams to account for beam age, debonded strands and certain superimposed dead loads. These adjustments are intended to ensure no negative camber in the final condition. Beams continue to camber until they are fully aged or until a bridge deck has been cast on them. Since the IL-Beams strand patterns were originally developed in 2015, the permissible beam shipping age was reduced. Additionally, the current camber calculations utilize the PCI camber factors of 1.8 for the upward camber from the prestressing force and 1.85 for the downward deflection of beam weight. These factors have been analyzed with proprietary software to predict the effects of debonded strands and the camber at the beam age just prior to placing the bridge deck and are reflected in the updated charts and strand patterns. The Department's policy for the minimum beam shipping age may be found in the Manual for Fabrication of Precast Prestressed Concrete Products.

The Department developed the following simplified camber method that envelops the results of the more exact proprietary computer runs used to develop the new charts. This procedure does not require proprietary software and may be used for the camber check. This procedure will eventually be added to the Design Guide.

a. Calculate the camber utilizing the PCI factors of 1.8 and 1.85.
b. Calculate the seat elevations using the weight of beam and deck only. (This has proven to be effective to prevent negative fillet.)
c. Calculate the Theoretical Grade Elevations Adjusted for Dead Load Deflection using the weight of beam, deck and superimposed dead loads (parapets, medians, railings, etc.)
d. Check for positive camber using the weight of beam, deck, superimposed deadloads, and ½ of the FWS.
e. Calculate the average fillet weight using b. and c. above. A 2.5 inch average fillet height was used to develop the strand patterns and serves as a good starting point. This assumption should be verified and revised only if the average fillet comes out greater than 2.5 inches.

More detailed procedures, guidance and example calculations for computing camber and deflection in PPC I-beams, bulb T-beams, and IL-beams will eventually be added to Design Guide 3.4.

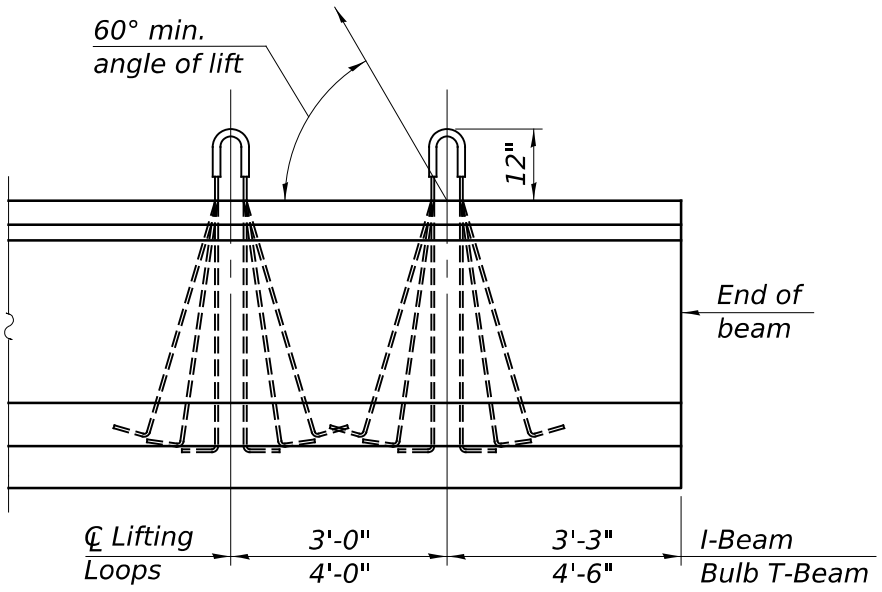
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3.4.6 Lifting Loops

The Department has developed standard details and designs for lifting loops which are fabrication friendly and satisfy guidelines in the PCI Design Handbook. Figure 3.4.6-1 presents the developed details and designs which are applicable to all PPC I-beams and bulb T-beams designed according to the LRFD Specifications. The selection of a particular design in Figure 3.4.6-1 is dependent upon the beam size (36 in. through 72 in.) and length.

Figure 3.4.6-2, Table 3.4.6-3, and Table 3.4.6-4 contain the lifting loop design and details for all IL-beams. These beam sections are much heavier per foot than the I-beam and bulb T-beam sections and can be much longer as well. Hence, the number of lifting loop locations per beam end may be 1, 2, or 4. Three lifting loops per beam end were not considered because it is not plausible to provide balanced lifting. The number, location, and specific directions for these lifting loops satisfy all beam stresses and stability for the standardized beams presented. Note there are different lifting loop locations for the larger, longer beams for lifting stability. Also note that lifting loops remain with the ½" diameter 270 ksi strands. The 0.6" diameter strand is not permitted for lifting loops.

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ELEVATION

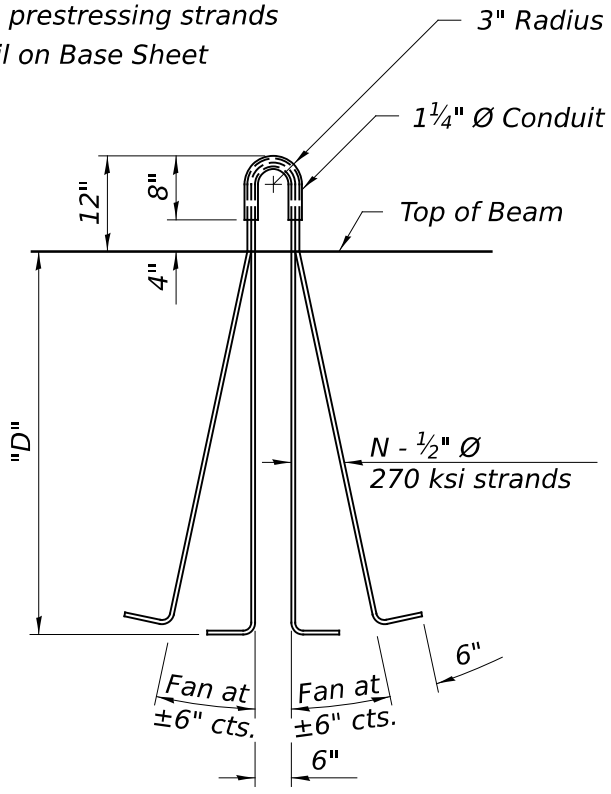
See table for the number of loops and strands per loop required. All lifting loop prestressing strands shall be ½" Ø-270 ksi. Detail on Base Sheet accordingly.

DESIGN CRITERIA

PPC Beam	Wt. (lbs./ft.)	D
36" I-Beam	375	2'-6"
42" I-Beam	485	3'-0"
48" I-Beam	595	3'-6"
54" I-Beam	624	4'-0"
63" Bulb T-Beam	743	4'-9"
72" Bulb T-Beam	799	5'-6"

LIFTING LOOP REQUIREMENTS

No. of loops each end of beam	No. of strands per loop (N)	Maximum Gross wt. of beam (lbs.)
1	3	41,500
1	4	55,400
2	3	83,100
2	4	110,800
2	5	138,500

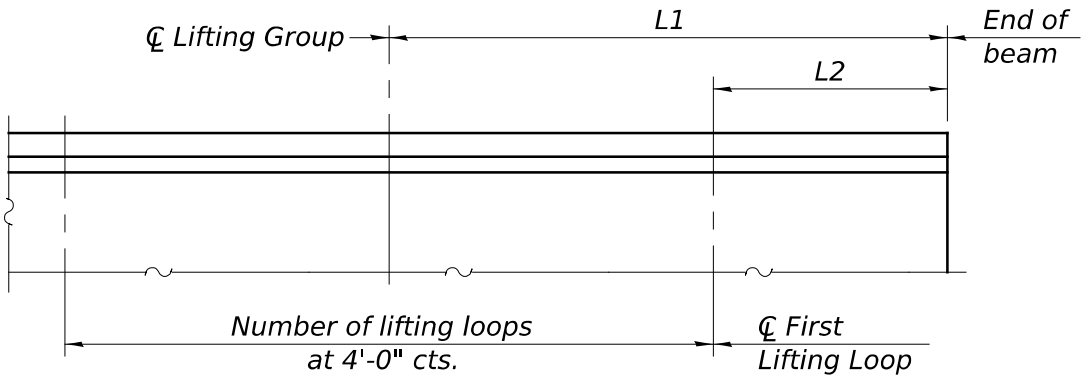


TYPICAL LIFTING LOOP

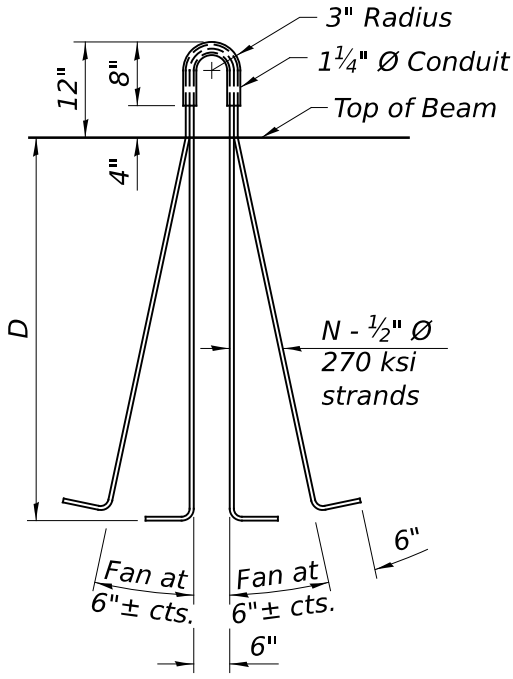
LIFTING LOOPS FOR I-BEAMS AND BULB T-BEAMS

Figure 3.4.6-1

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ELEVATION



TYPICAL LIFTING LOOP

LIFTING LOOP SPECIFICATIONS

No. of lifting loops (each end of beam)	No. of strands, N (per lifting loop)	Maximum Gross wt. of beam (lbs.)
1	3	41,500
1	5	69,200
2	3	83,100
2	5	138,500
4	5	277,100

Note: See Tables 3.4.6-3 and 3.4.6-4 for N, D, L1, L2 and the number of lifting loops for the beam size and length.

LIFTING LOOPS FOR IL-BEAMS

Figure 3.4.6-2

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Beam (Wt.)	D	Beam Length (ft)	L1	L2	No. of lifting loops (each end of beam)	No. of strands, N (per lifting loop)
IL27-1830 (477 lbs./ft.)	1'-9"	≤ 86'	4'-0"	4'-0"	1	3
IL36-2438 (759 lbs./ft.)	2'-6"	>91' to ≤ 128'	6'-6'	4'-6'	2	5
		≤ 91'	4'-6'	4'-6'	1	
IL36-3838 (840 lbs./ft.)		>82' to ≤ 130'	6'-6'	4'-6'	2	
		≤ 82'	4'-6'	4'-6'	1	
IL45-2438 (825 lbs./ft.)	3'-3"	>83' to ≤ 134'	6'-6'	4'-6'	2	
		≤ 83'	4'-6'	4'-6'	1	
IL45-3838 (905 lbs./ft.)		>76' to ≤ 136'	6'-6'	4'-6'	2	
		'≤ 76'	4'-6'	4'-6'	1	
IL54-2438 (890 lbs./ft.)	4'-0"	>77' to ≤ 139'	6'-6'	4'-6'	2	
		≤ 77'	4'-6'	4'-6'	1	
IL54-3838 (971 lbs./ft.)		>142' to ≤ 153'	10'-6'	4'-6'	4	
		≤ 142'	6'-6'	4'-6'	2	
IL63-2438 (956 lbs./ft.)	4'-9"	>142' to ≤ 154'	10'-6'	4'-6'	4	
		≤ 142'	6'-6'	4'-6'	2	
IL63-3838 (1,036 lbs./ft.)		>158' to ≤ 164'	12'-6'	6'-6'	4	
		>133' to ≤ 158'	10'-6'	4'-6'	4	
		≤ 133'	6'-6'	4'-6'	2	

Notes: See Figure 3.4.6-2 for accompanying details.

L1 was based on vertical stresses and beam stability during lifting. Larger beams require the lifting loops to be moved inward to maintain stability. If the beam length exceeds these limits, a special design will be required to determine L1 which satisfies both stability and vertical stresses.

Table 3.4.6-3

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Beam (Wt.)	D	Beam Length	L1	L2	No. of lifting loops (each end of beam)	No. of strands, N (per lifting loop)
IL72-2438 (1,021 lbs./ft.)	5'-6"	>163' to ≤ 169'	14'-6'	8'-6'	4	5
		>157' to ≤ 163'	12'-6'	6'-6'		
		>135' to ≤ 157'	10'-6'	4'-6'		
		≤ 135'	6'-6'	4'-6'	2	
IL72-3838 (1,102 lbs./ft.)		>168' to ≤ 176'	15'-6'	9'-6'	4	
		>161' to ≤ 168'	13'-0'	7'-0'		
		>125' to ≤ 161'	10'-6'	4'-6'		
		≤ 125'	6'-6'	4'-6'	2	
IL81-3838 (1,168 lbs./ft.)	6'-3"	>182' to ≤ 188'	18'-6'	12'-6'	4	
		>176' to ≤ 182'	16'-6'	10'-6'		
		>170' to ≤ 176'	14'-6'	8'-6'		
		>164' to ≤ 170'	12'-6'	6'-6'		
		>118' to ≤ 164'	10'-6'	4'-6'		
		≤ 118'	6'-6'	4'-6'	2	
IL90-3838 (1,233 lbs./ft.)	7'-0"	>194' to ≤ 200'	21'-6'	15'-6'	4	
		>189' to ≤ 194'	19'-6'	13'-6'		
		>185' to ≤ 189'	18'-0'	12'-0'		
		>179' to ≤ 185'	16'-6'	10'-6'		
		>173' to ≤ 179'	14'-6'	8'-6'		
		>167' to ≤ 173'	12'-6'	6'-6'		
		>112' to ≤ 167'	10'-6'	4'-6'		
		≤ 112'	6'-6'	4'-6'	2	

Notes: See Figure 3.4.6-2 for accompanying details.

L1 was based on vertical stresses and beam stability during lifting. Larger beams require the lifting loops to be moved inward to maintain stability. If the beam length exceeds these limits, a special design will be required to determine L1 which satisfies both stability and vertical stresses.

Table 3.4.6-4

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3.4.7 Splitting Steel

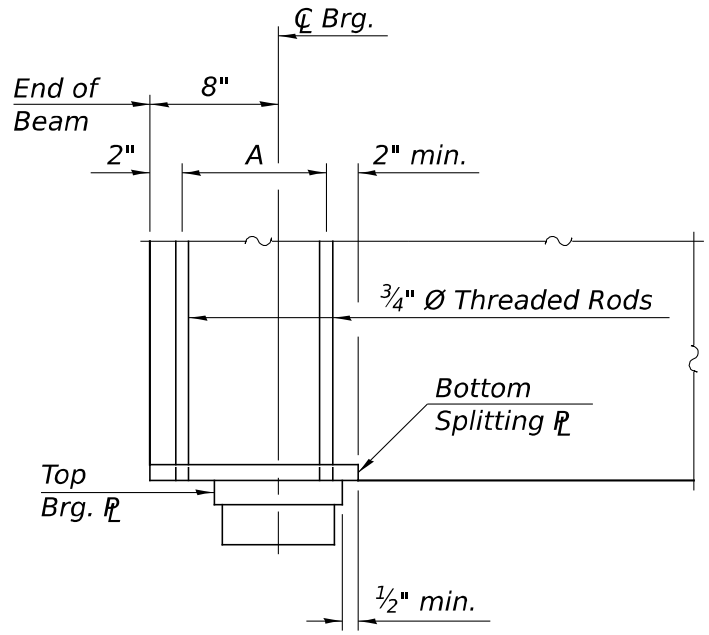
The Department has developed standard splitting steel details for the pretensioned anchorage zones at the ends of PPC beams. These details shall be specified for all beams. The required splitting steel details were developed and refined by the BBS through testing and evaluation at fabrication plants and observed performance of prototypes in the field. The strength of the developed details exceeds the requirements for pretensioned anchorage zones outlined in Article 5.9.4.4 of the AASHTO LRFD Bridge Design Specifications.

The splitting steel details for PPC I-beams and bulb T-beams consist of a system of threaded rods and plates at the beam ends which are immediately followed by a standard spacing of G_1 and G_2 bars. The top plate has been standardized and is detailed on the base sheets for the various beams used by the Department. The bottom plate is sized based on the presence of a bearing as illustrated in Figure 3.4.7-1. If there is not a bearing (i.e. the beams sits on a ½ in. grout bed or fabric bearing pads) then the minimum plate dimensions shown in the figure shall be used. These details have demonstrated better performance at reducing splitting force cracks in the anchorage zone caused by strand detensioning than those which had been traditionally specified by the Department.

The Departmental base sheets for PPC I-beams and bulb T-beams reflect the standard splitting steel details required by the Department. One primary base sheet for each beam span should be provided along with one companion details base sheet for each beam depth used.

Figure 3.4.7-2 and Figure 3.4.7-3 covers the splitting steel details for the IL-beams. The area of steel in the splitting zone was increased to handle the larger prestressing forces according to AASHTO LRFD Bridge Design Specifications Article 5.9.4.4.1. This steel consists of 1” diameter threaded rods and D31 bars based on the definition of the splitting zone (h/4) as defined by AASHTO. Other features include bottom flange confinement reinforcement welded to the bottom plate rather than using shear studs. The #3 bars will be separated into two groups: single #3 bars and double #3 bars. The double #3 bar allows for proper placement of the beam shear reinforcement. This provides better detailing, fabrication, confinement and reduces congestion. Also, jam nuts from the threaded rods to the bottom plate were added to reduce the slack in the threads and help minimize cracking. The standard plate lengths are detailed for integral abutment structures but longer bottom plates will be required when bearings are present.

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ELEVATION

(Studs and pintles not shown for clarity)

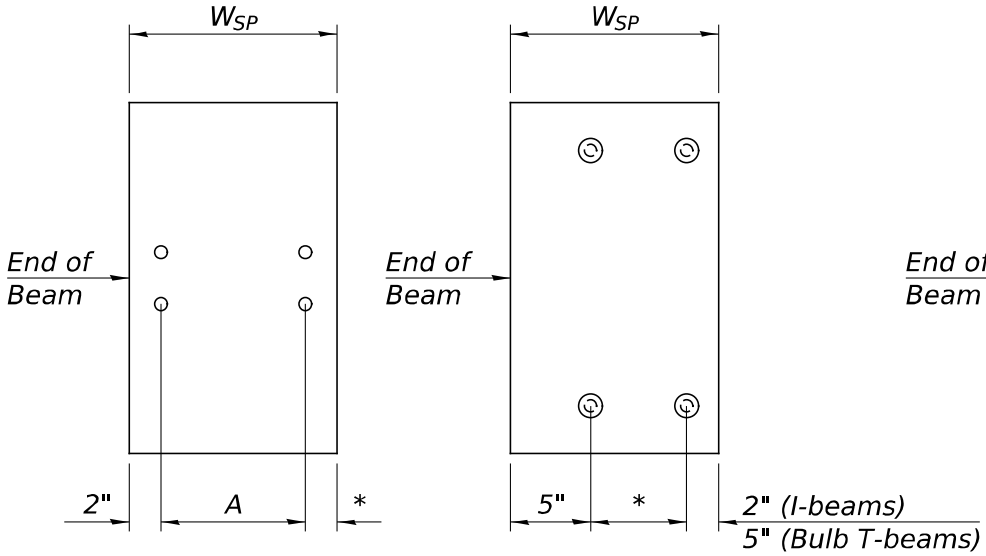
A = 3 spaces @ 3" for PPC I-beams

A = 4 spaces @ 3¼" for PPC Bulb T-beams

W_{SP} = Width of bottom splitting plate = 8" + ½ (top brg. plate width) + ½"

(W_{SP} = 13" min. I-beams)

(W_{SP} = 17" min. Bulb T-beams)

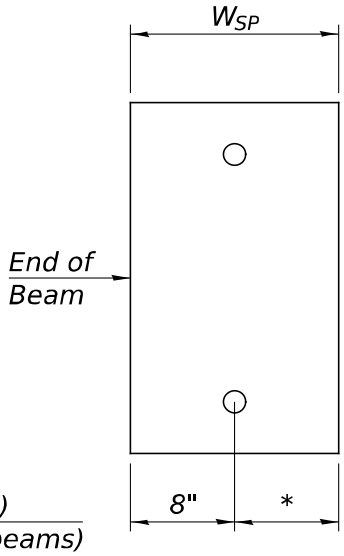


PLAN

(Showing threaded rods)

PLAN

(Showing studs)



PLAN

(Showing pintles used for bearings only)

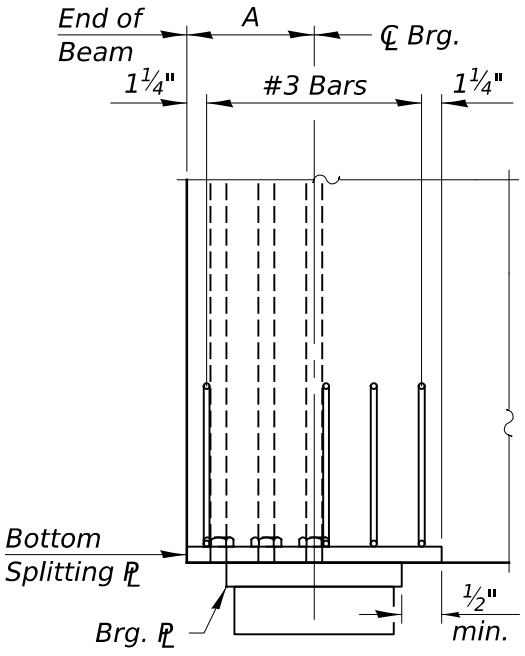
* Varies based on total width of bottom splitting plate (W_{SP}).

Note:
Calculate W_{SP} (width of bottom splitting plate) using the equation above.

BOTTOM SPLITTING PLATE FOR I-BEAMS AND BULB T-BEAMS WITH BEARINGS

Figure 3.4.7-1

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ELEVATION

Note:
The IL-Beam base sheets are detailed with the minimum bottom splitting plate width equal to 10". When elastomeric or fixed bearings are required, the plate width will need to be increased as shown. The formulas are based on adding width in 3" increments. One set of double #3 bars are also added for each 3" increment.

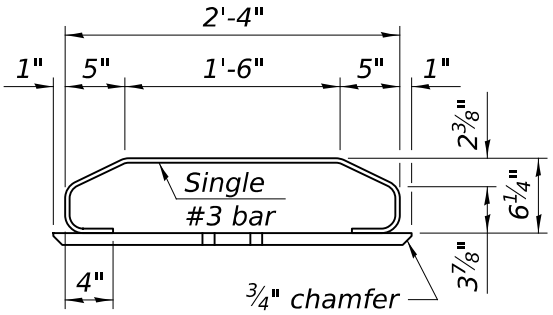
$$A = \frac{\text{Top Brg. PL width}}{2} \text{ or } 8"$$

whichever is greater.

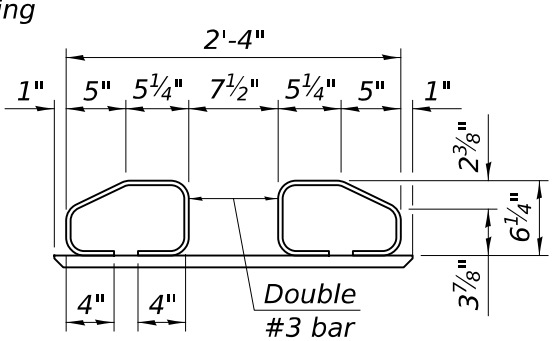
$$B = \left[\frac{\text{Top Brg. PL width}}{2} + A - 9\tfrac{1}{2}" \right] / 3$$

rounded up to nearest integer.

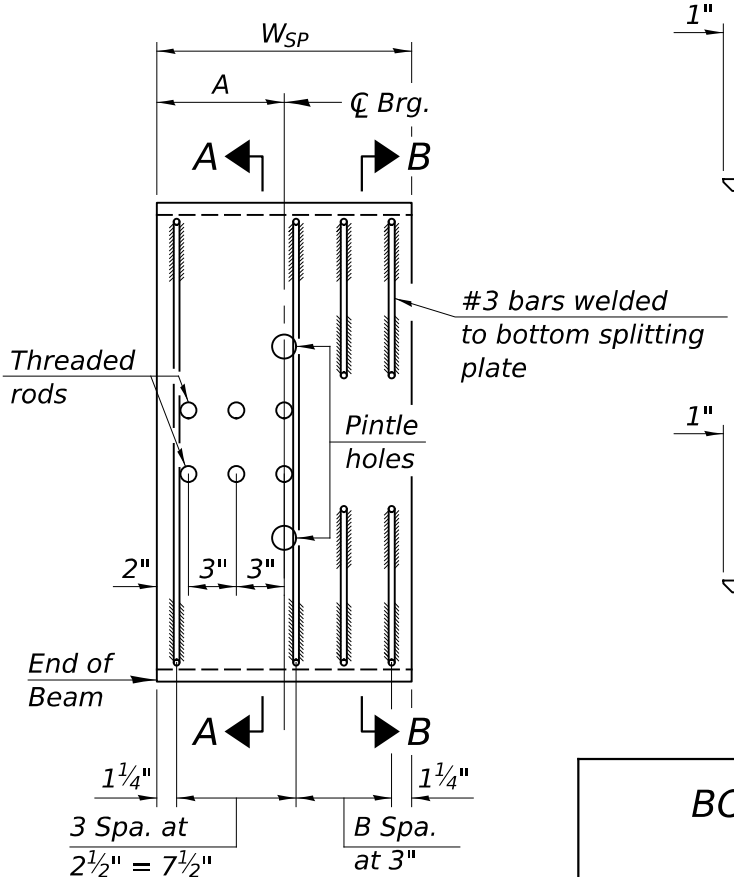
W_{SP} = Width of bottom splitting plate = B x 3" + 10"



SECTION A-A



SECTION B-B

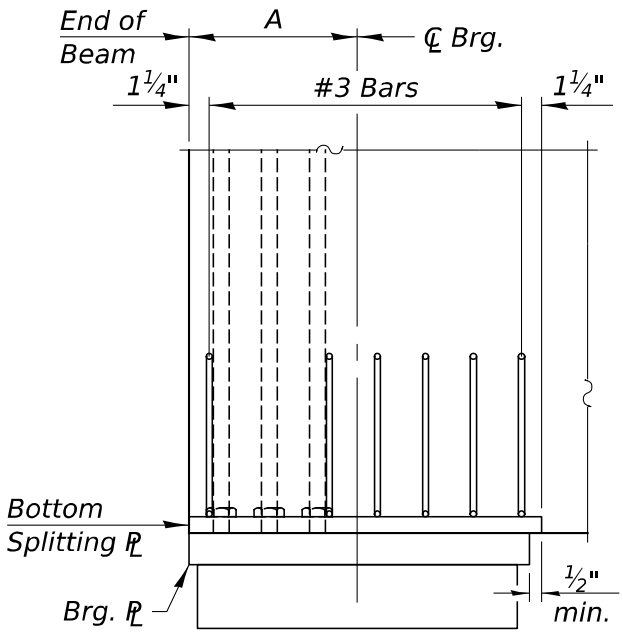


PLAN

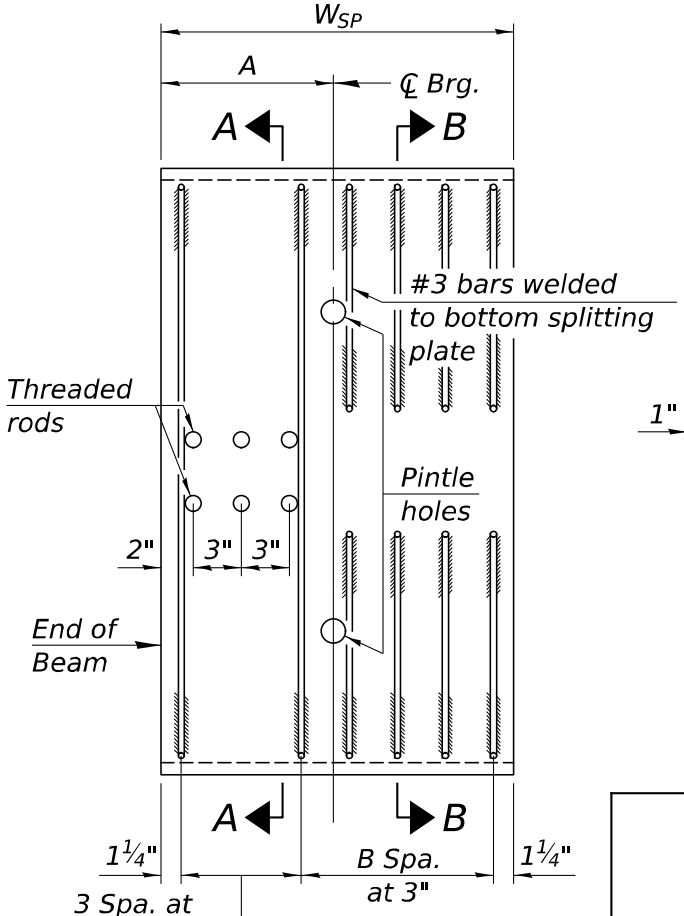
BOTTOM SPLITTING PLATE DETAILS FOR IL-27 BEAM SHAPES WITH BEARINGS

Figure 3.4.7-2

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ELEVATION



PLAN

Note:
The IL-Beam base sheets are detailed with the minimum bottom splitting plate width equal to 10". When bearings are required, the plate width will need to be increased as shown. The formulas are based on adding width in 3" increments. One set of double #3 bars are also added for each 3" increment.

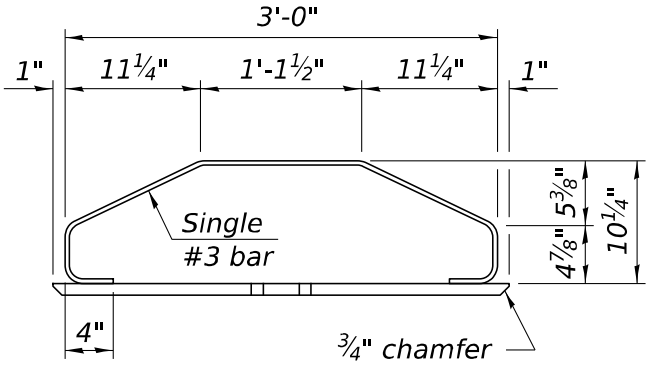
$$A = \frac{\text{Top Brg. ℄ width}}{2} \text{ or } 8"$$

whichever is greater.

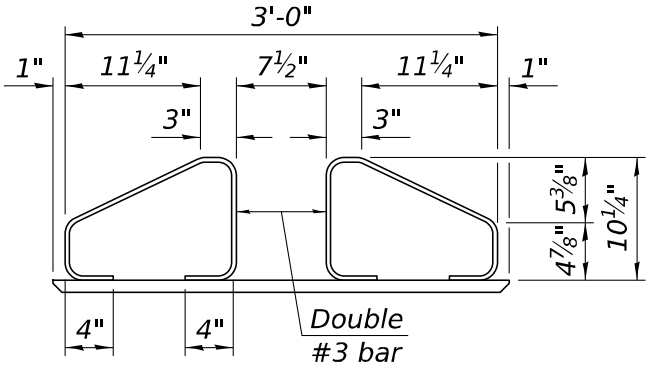
$$B = \left[\frac{\text{Top Brg. ℄ width}}{2} + A - 9\tfrac{1}{2}"\right] / 3$$

rounded up to nearest integer.

W_{SP} = Width of bottom splitting plate = B x 3" + 10"



SECTION A-A



SECTION B-B

BOTTOM SPLITTING PLATE DETAILS FOR IL-36 thru IL-90 BEAM SHAPES WITH BEARINGS

Figure 3.4.7-3

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3.4.8 Permanent Bracing

Permanent bracing shall be provided for all PPC beams and shall be detailed in the plans. This bracing is primarily provided to help ensure beam stability during erection and deck construction. Figure 3.4.8-1 illustrates permanent bracing details for 36 and 42 inch PPC I-Beams, Figure 3.4.8-2 illustrates permanent bracing details for 48 and 54 inch PPC I-Beams and Figure 3.4.8-3 illustrates permanent bracing details for Bulb T-Beams. Spans up to 90 feet shall be braced at 0.33L and 0.67L where L is the length of the beam. Spans over 90 feet in length shall be braced at 0.25L, 0.5L, and 0.75L.

Figures 3.4.8-4 through 3.4.8-7 illustrate permanent bracing details for the IL-beams. This bracing is provided primarily to ensure beam stability during erection and bridge deck construction. All IL-beams shall have a minimum of two brace points with a maximum spacing of 35 feet. The fabricator shall be responsible for adjusting the location of holes and inserts to miss strands within permissible tolerances. Structures with skews less than or equal to 20 degrees shall be braced along the skew by utilizing bent angles or plates. Structures with skews greater than 20 degrees shall be braced at right angles to the beam, with the formed holes in adjacent beams offset by an amount equal to the beam spacing multiplied by the tangent of the skew. All holes in webs shall be at right angles to the web. See Figure 3.4.8-8. Bracing is not required between beams across the stage construction line. Permanent bracing shall not be paid for separately but shall be included in the cost of furnishing and erecting the prestressed beams.

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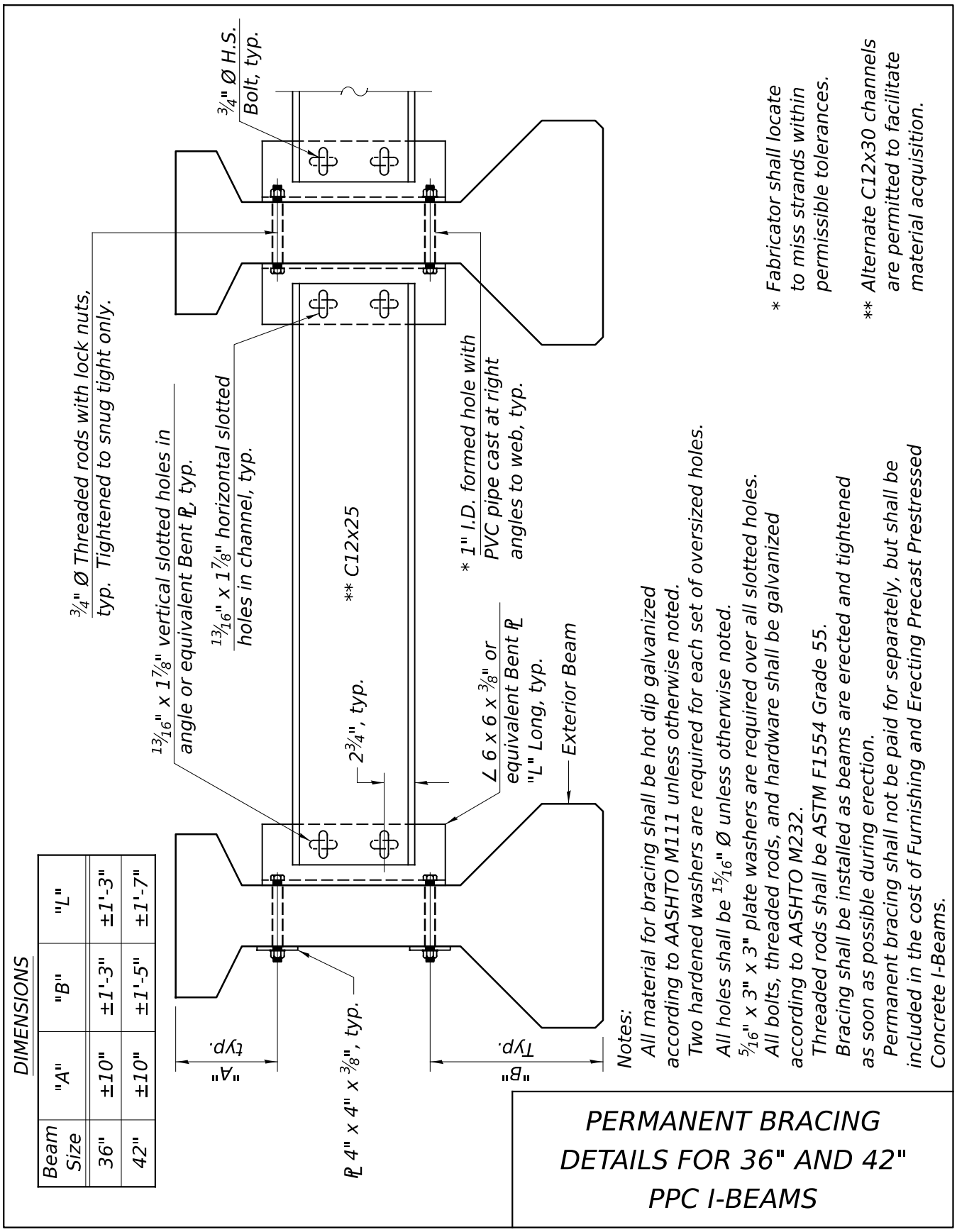


Figure 3.4.8-1

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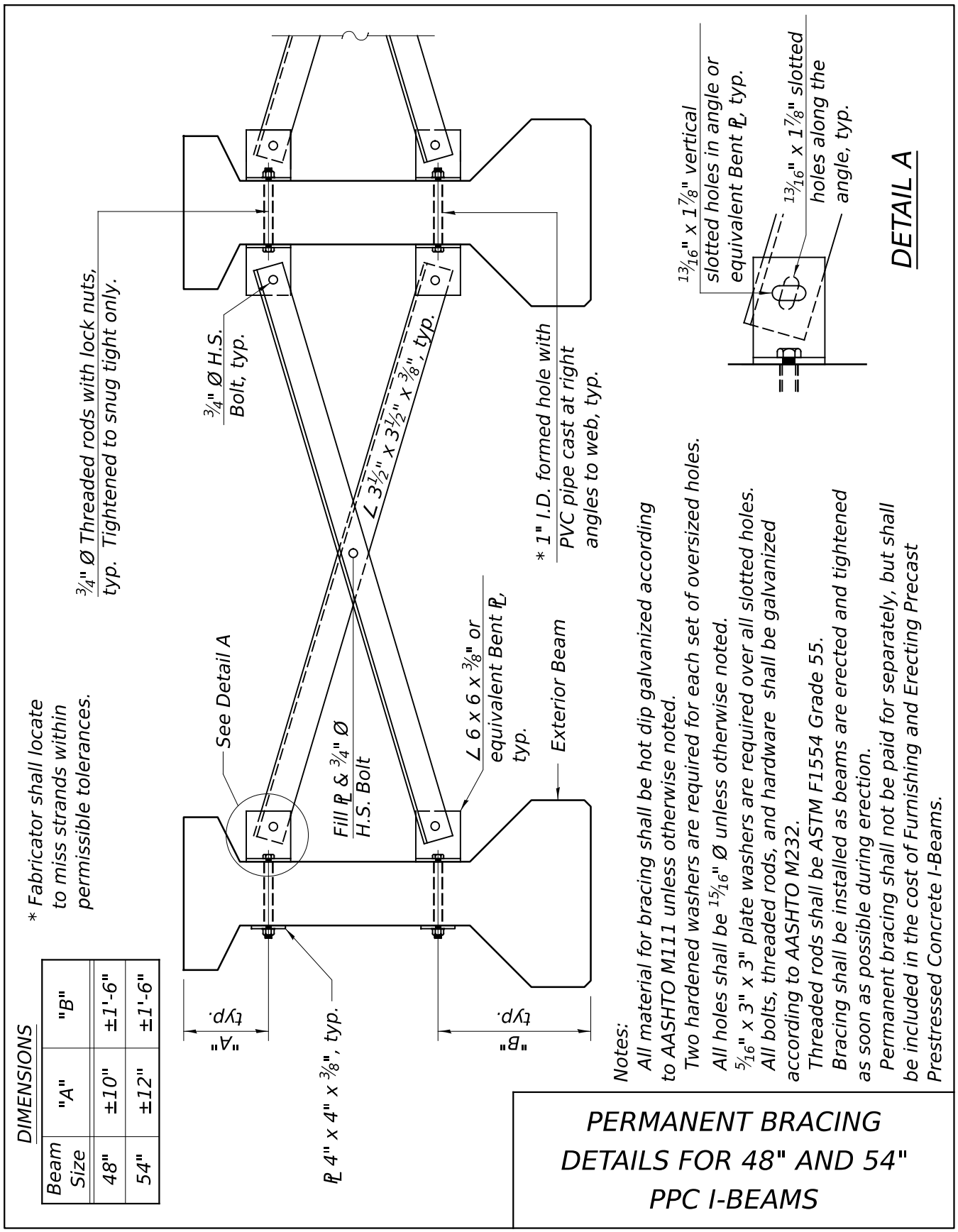


Figure 3.4.8-2

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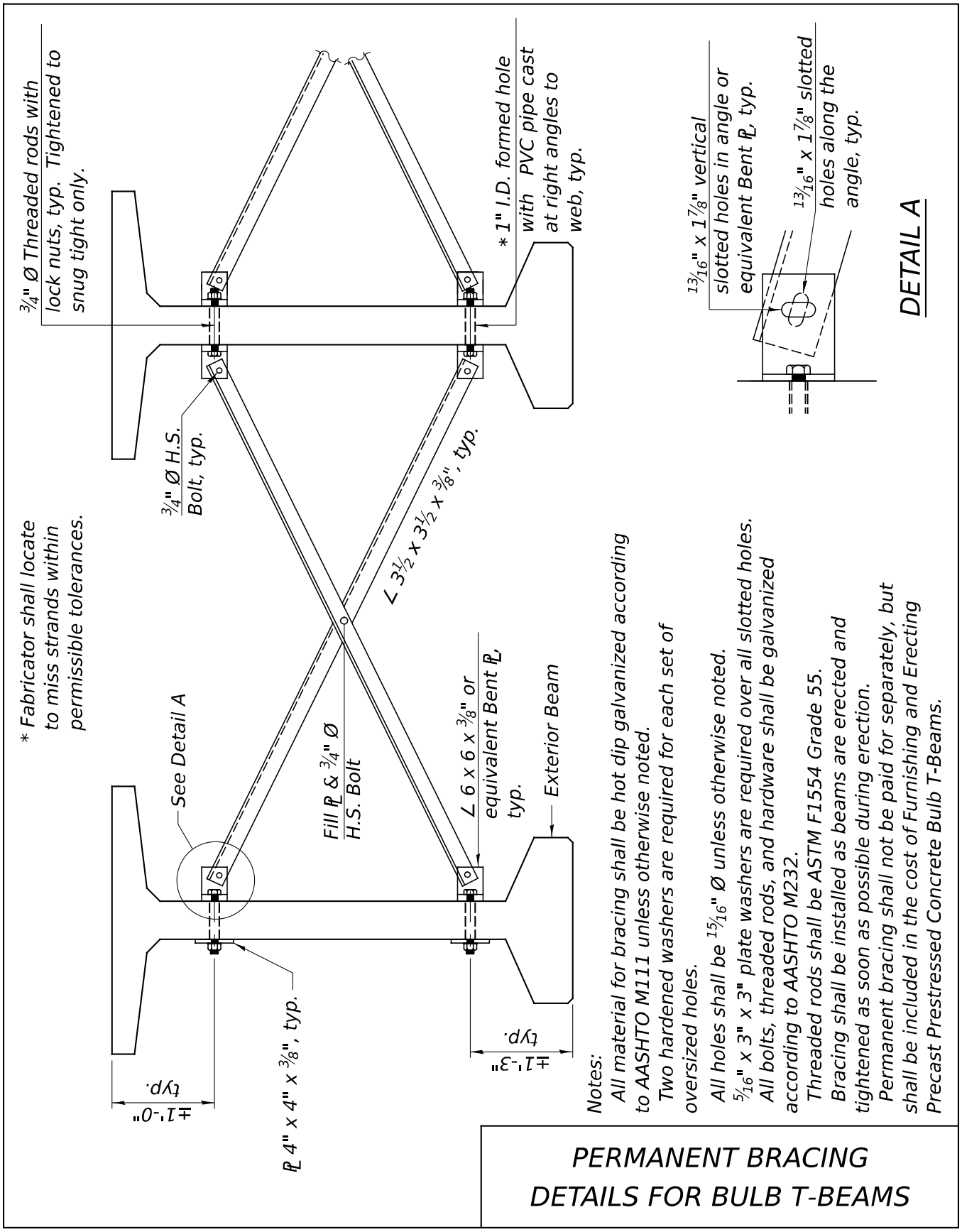
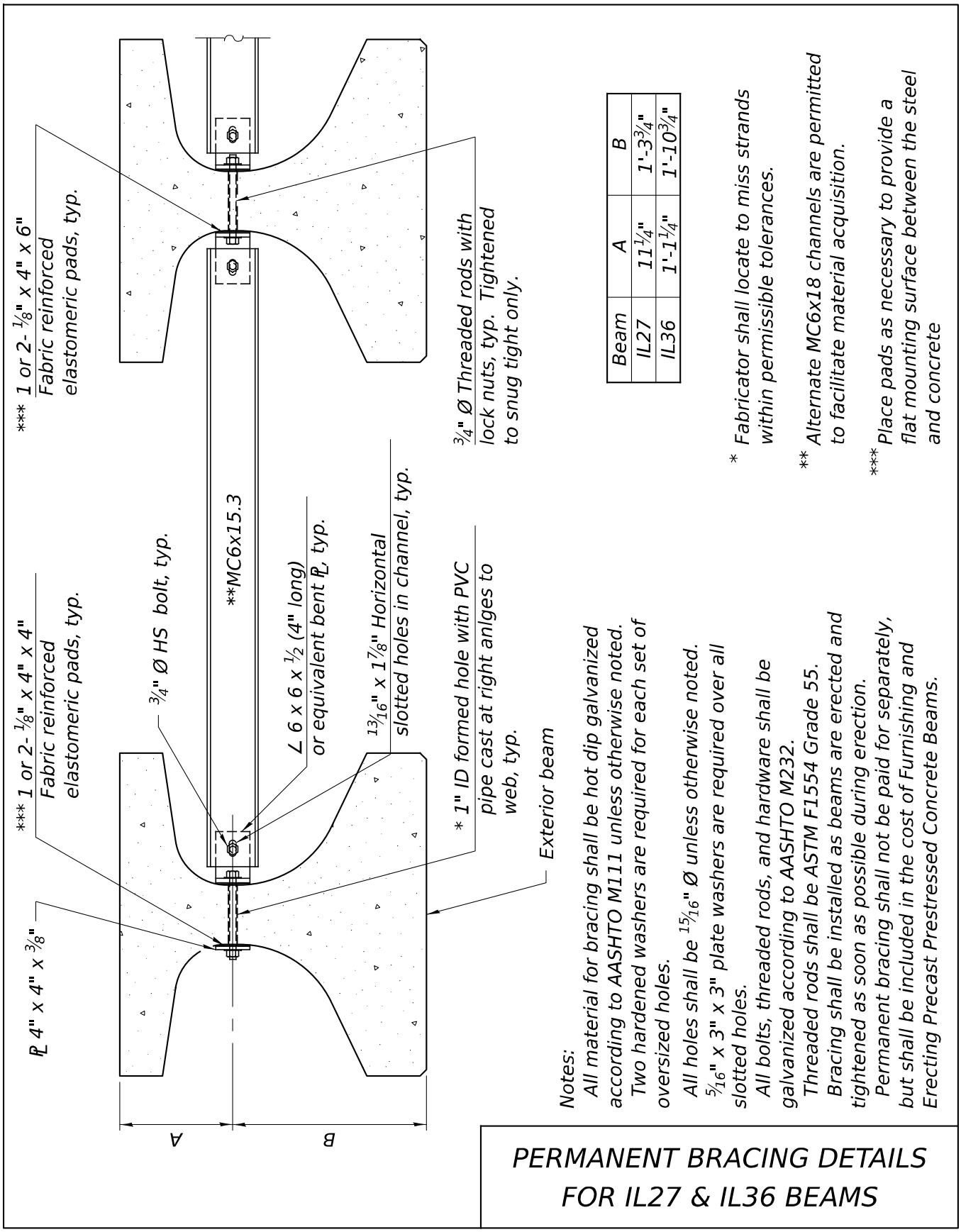


Figure 3.4.8-3

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Beam	A	B
IL27	11 1/4"	1'-3 3/4"
IL36	1'-1 1/4"	1'-10 3/4"

* Fabricator shall locate to miss strands within permissible tolerances.

** Alternate MC6x18 channels are permitted to facilitate material acquisition.

*** Place pads as necessary to provide a flat mounting surface between the steel and concrete

Notes:

All material for bracing shall be hot dip galvanized according to AASHTO M111 unless otherwise noted.

Two hardened washers are required for each set of oversized holes.

All holes shall be 15/16" Ø unless otherwise noted.

5/16" x 3" x 3" plate washers are required over all slotted holes.

All bolts, threaded rods, and hardware shall be galvanized according to AASHTO M232.

Threaded rods shall be ASTM F1554 Grade 55.

Bracing shall be installed as beams are erected and tightened as soon as possible during erection.

Permanent bracing shall not be paid for separately, but shall be included in the cost of Furnishing and Erecting Precast Prestressed Concrete Beams.

PERMANENT BRACING DETAILS FOR IL27 & IL36 BEAMS

Figure 3.4.8-4

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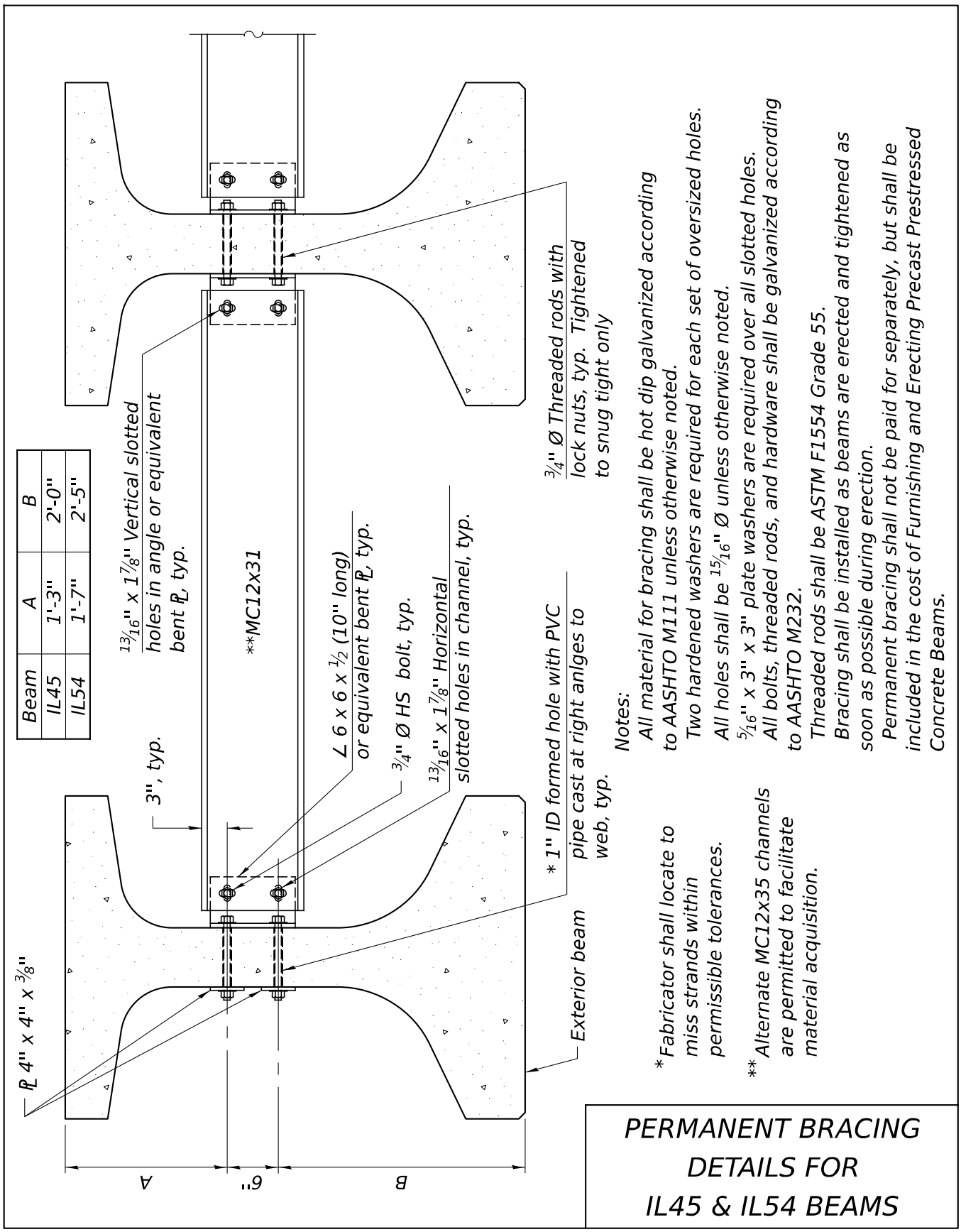


Figure 3.4.8-5

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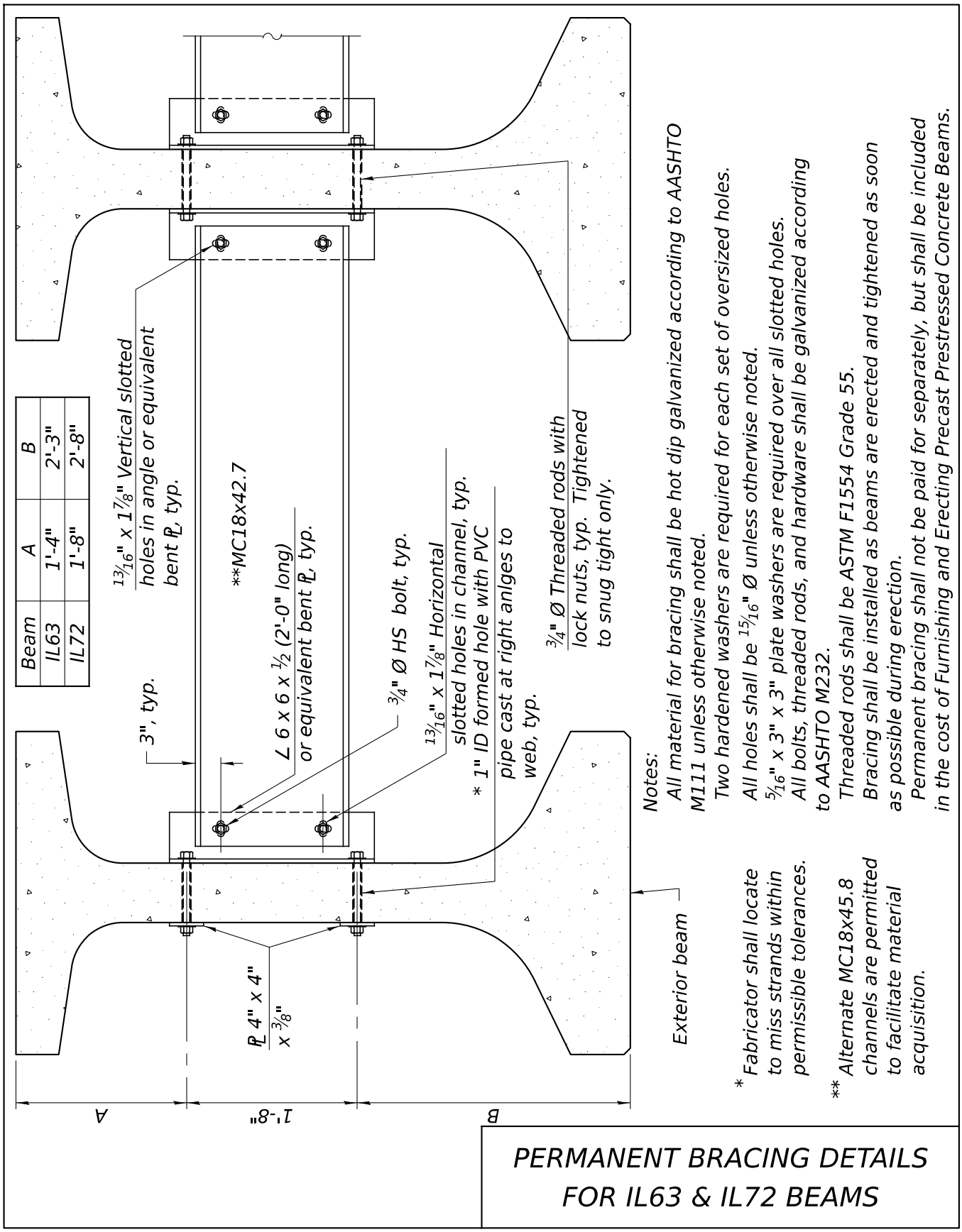


Figure 3.4.8-6

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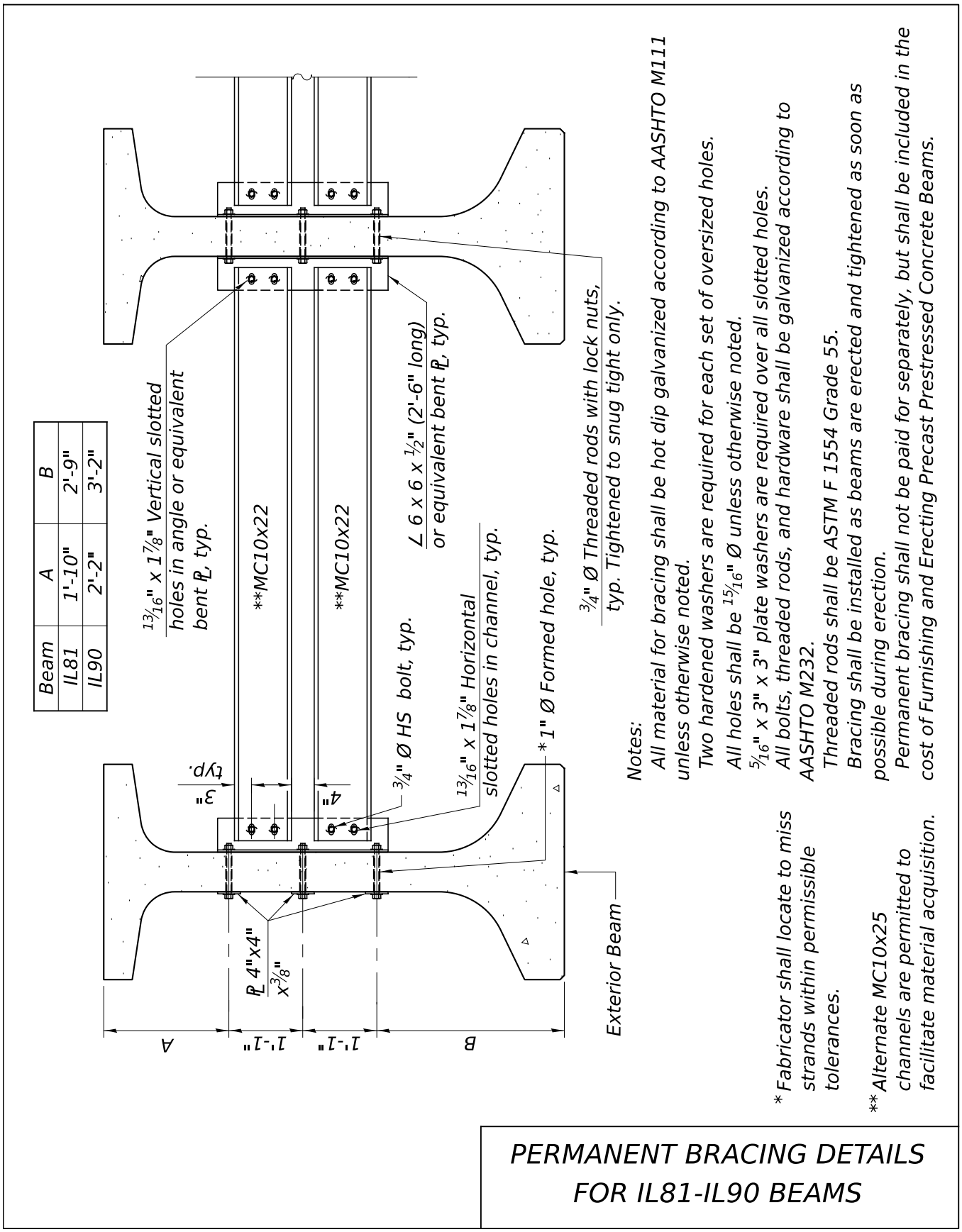


Figure 3.4.8-7

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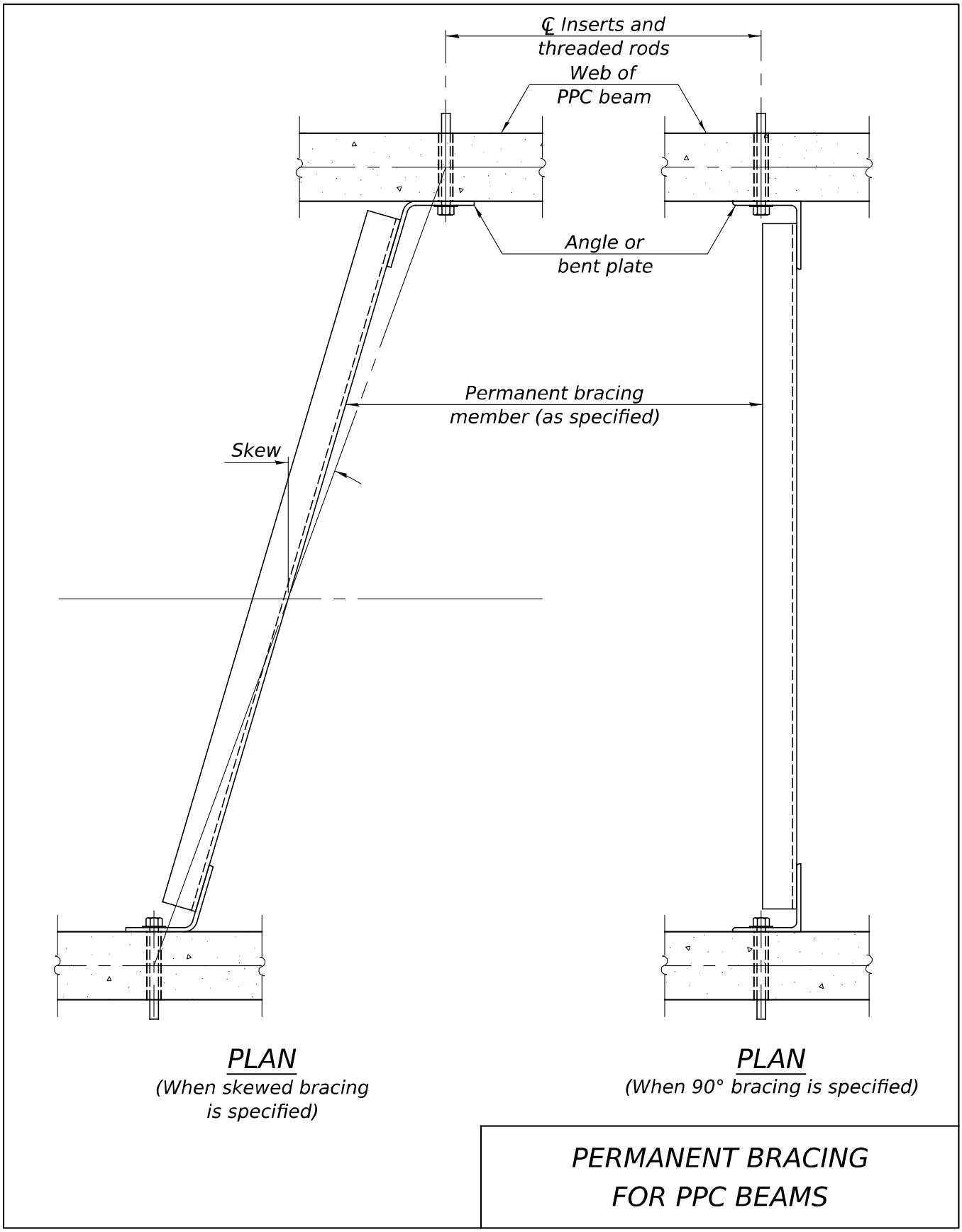


Figure 3.4.8-8

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3.4.9 Continuity Diaphragms over Piers

The details for continuity diaphragms are illustrated in Figures 3.4.9-1 thru 3.4.9-10. Most of these details are covered on the appropriate base sheets (see Beams-Prestressed Girders, Diaphragms-Expansion Pier Prestressed Beams, or Diaphragms-Fixed Pier Prestressed Beams base sheets at the IDOT CADD Bridge Cell Library) for the beam and deck configuration specified. The diaphragm width shall be as specified in Figure 3.4.9-6 or 3.4.9-9 and depends on the skew and type of prestressed beam used.

When specified on the respective beam base sheets for PPC I-beams and bulb T-beams, the positive moment restraint G_6 bar assemblies shall only be provided as detailed at piers.

When specified on the respective beam base sheets for PPC IL-beams, the positive moment restraint shall be provided with extended prestressing strands as detailed at piers.

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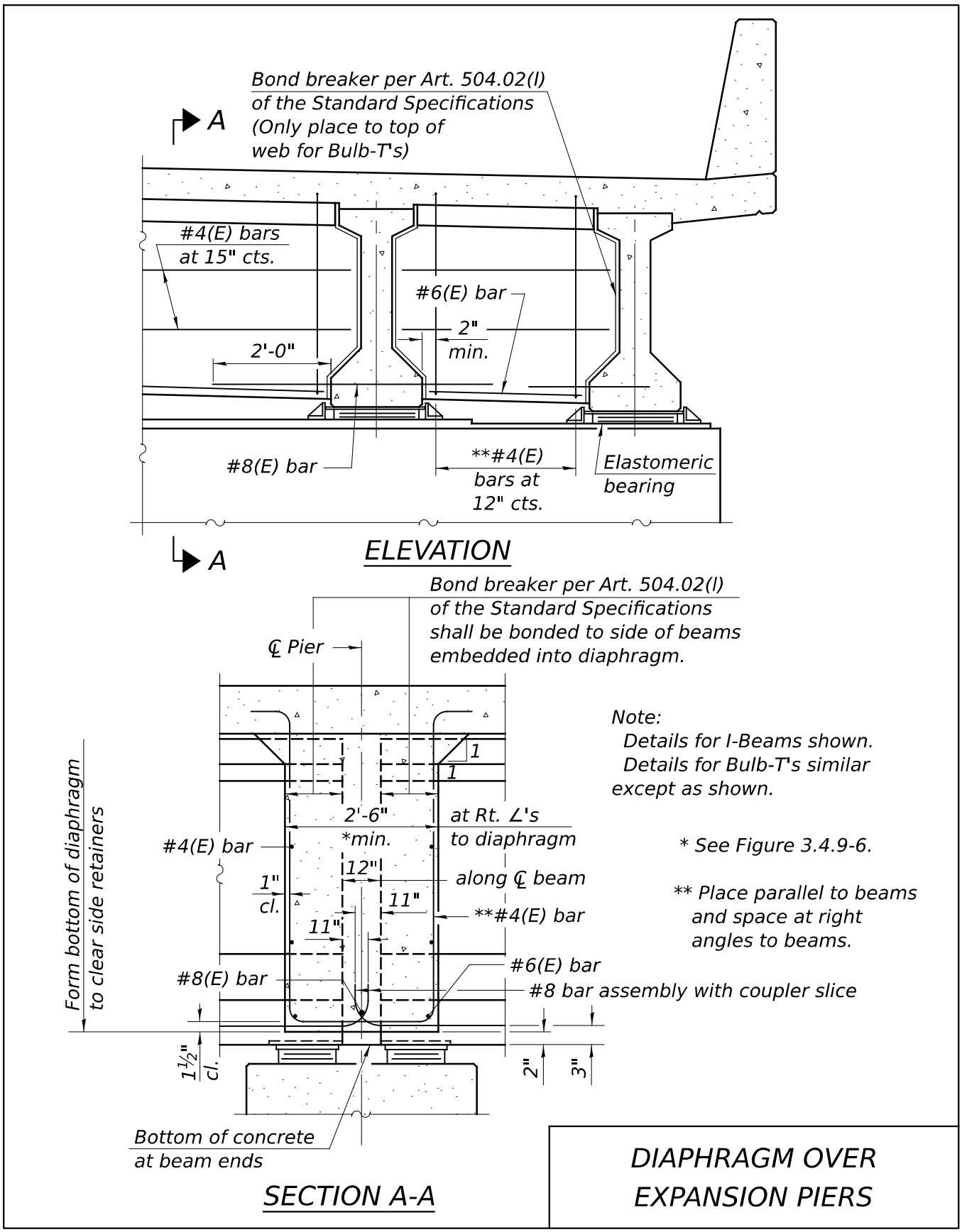
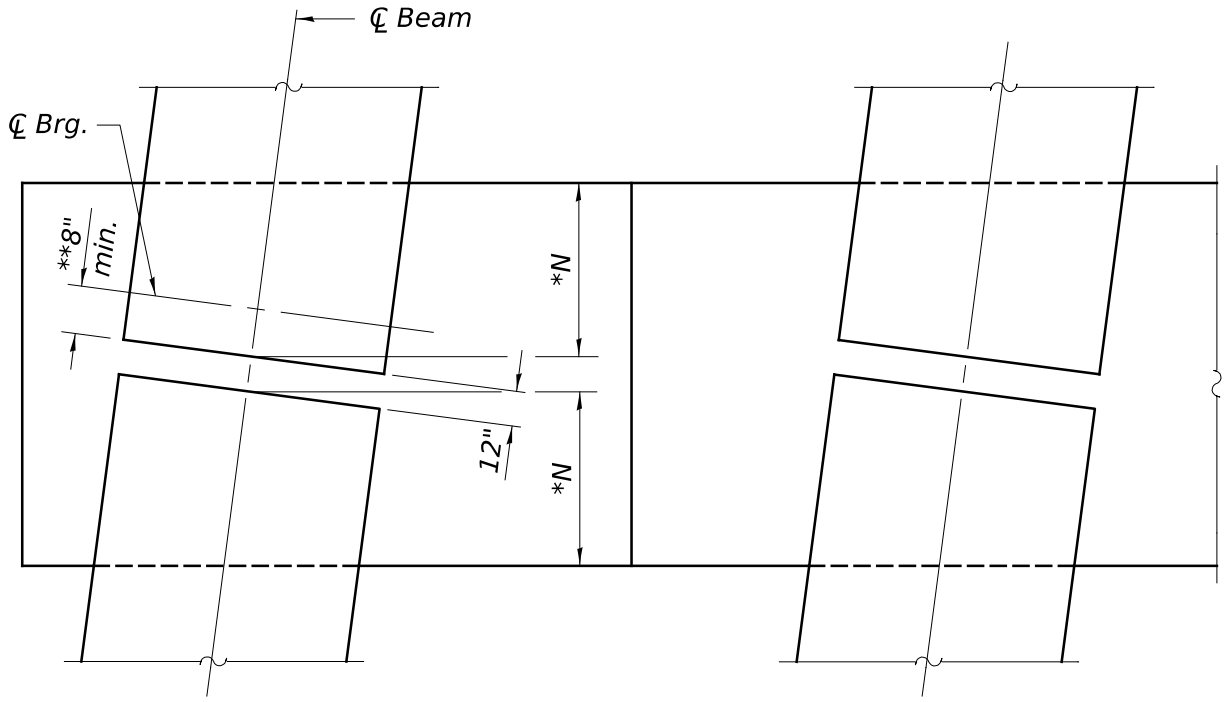


Figure 3.4.9-1

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PLAN

(Diaphragm not shown for clarity)

** Adequate seat length "N" shall be provided for each beam (for all seismic performance categories) and shall be determined according to Section 3.15. The shear reinforcement extending from the beams into the slab, the longitudinal slab reinforcement and the diaphragm details shown in this manual together shall be assumed to provide continuity over the supports; however, adequate seat length "N" shall be provided for each beam to assure neither beam will move beyond the support resulting in a shear failure.*

*** The 8" dimension is the minimum distance intended to use with bearings given in Section 3.7. Some of the larger bearings in Section 3.7, and bearings outside the limits of the Bridge Manual, will require this dimension to be increased.*

EXPANSION PIER DETAILS

Figure 3.4.9-2

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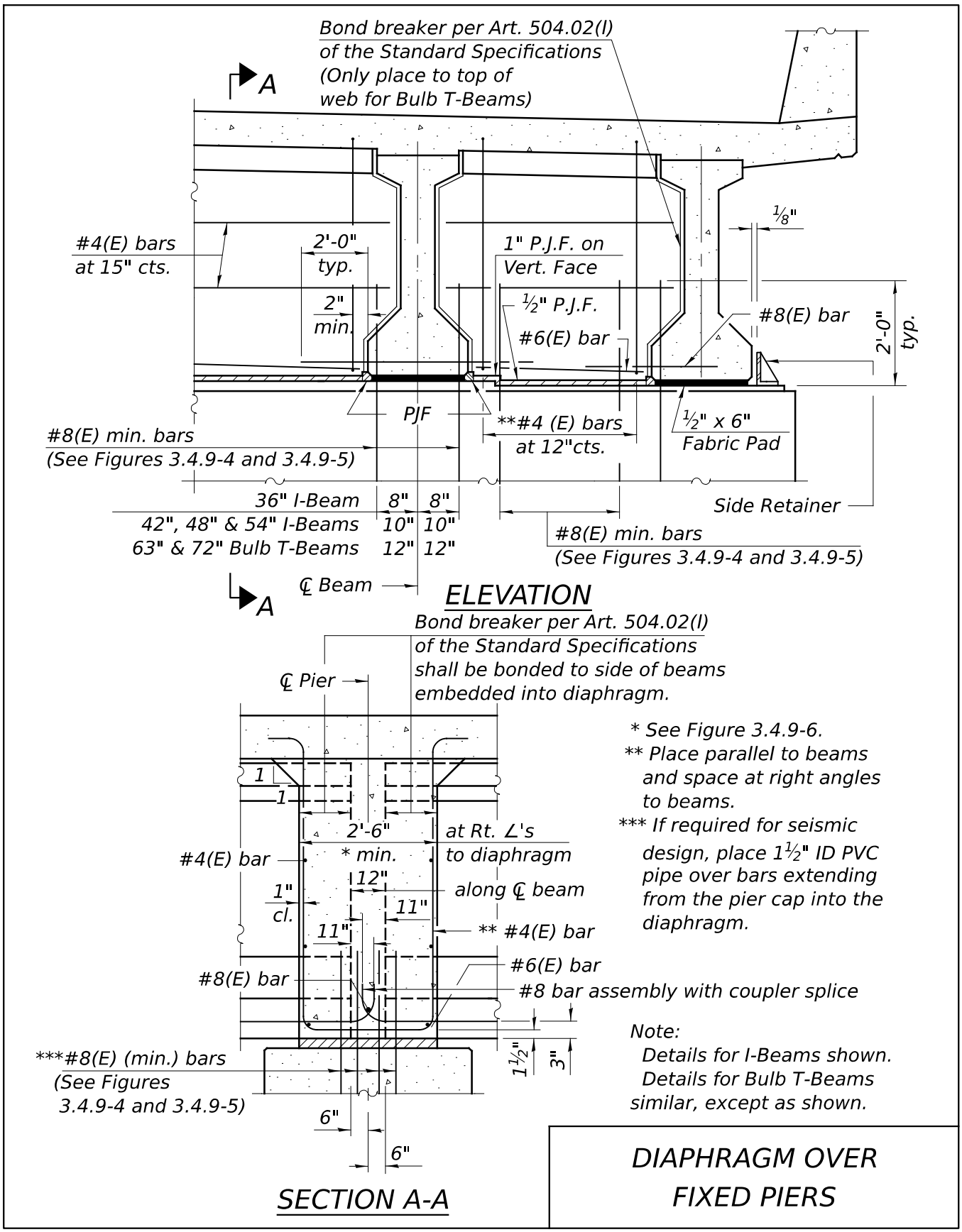
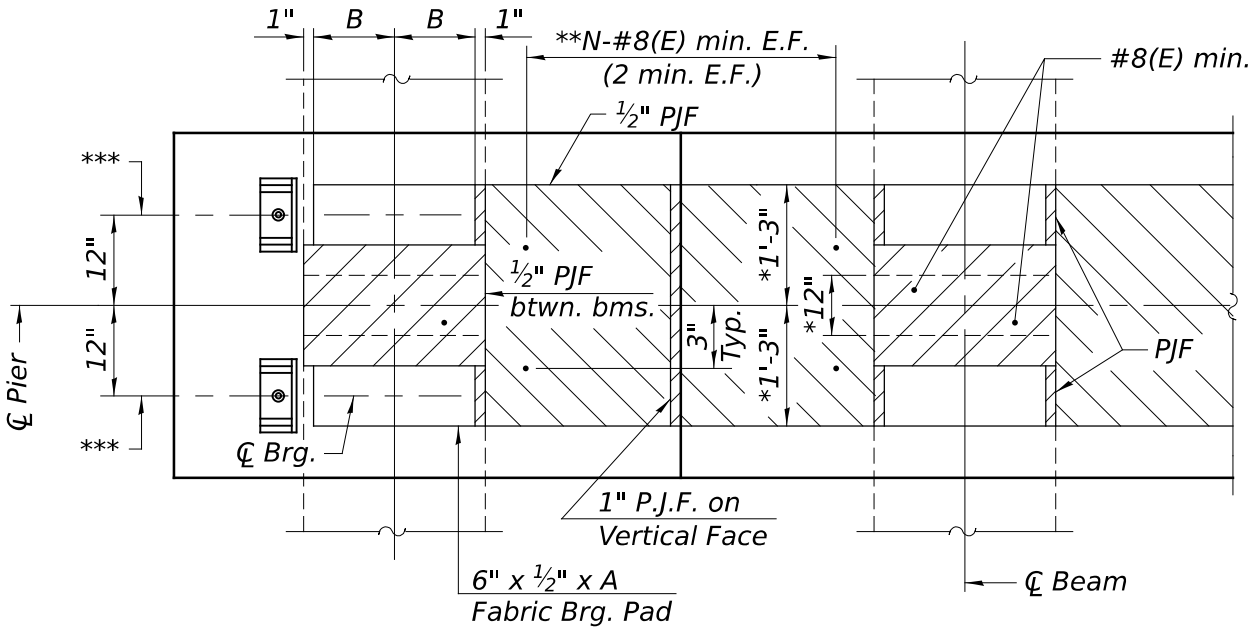


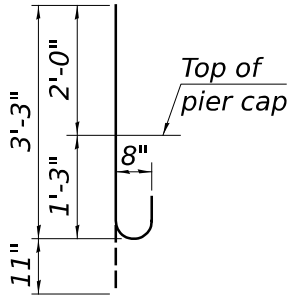
Figure 3.4.9-3

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STRAIGHT PIERS

Dimensions A and B to be determined by design.



#8(E) (MIN.) DOWEL BAR

* See Figure 3.4.9-6.

** $N = \frac{1}{2}\left[\frac{0.2\ DL}{28.3\ (S)} - 2\right]$

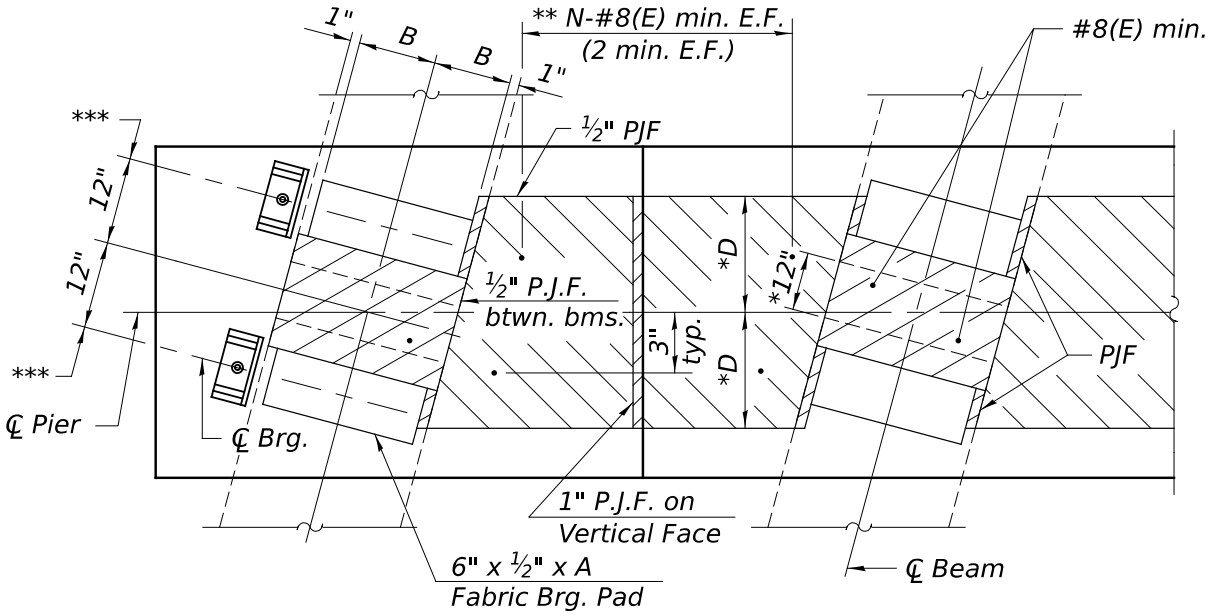
S = No. of beam spaces.
DL = Sum of all superstructure dead loads at the given pier under consideration (kips).

*** ℄ 1½" Ø x 18" Anchor bolts with 3" x 3" x 5 5/16" ℄ washer under nut. Holes in cap to be formed or drilled after beams are in place but prior to pouring concrete diaphragm.

FIXED PIER
DETAILS NO SKEW

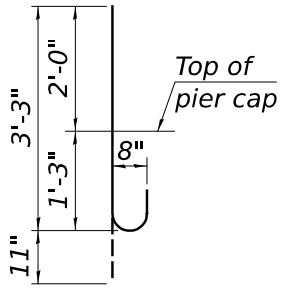
Figure 3.4.9-4

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SKEWED PIERS

Dimensions A and B to be determined by design.



#8(E) (MIN.) DOWEL BAR

* See Figure 3.4.9-6.

** $N = \frac{1}{2}\left[\frac{0.2\ DL}{28.3\ (S)} - 2\right]$

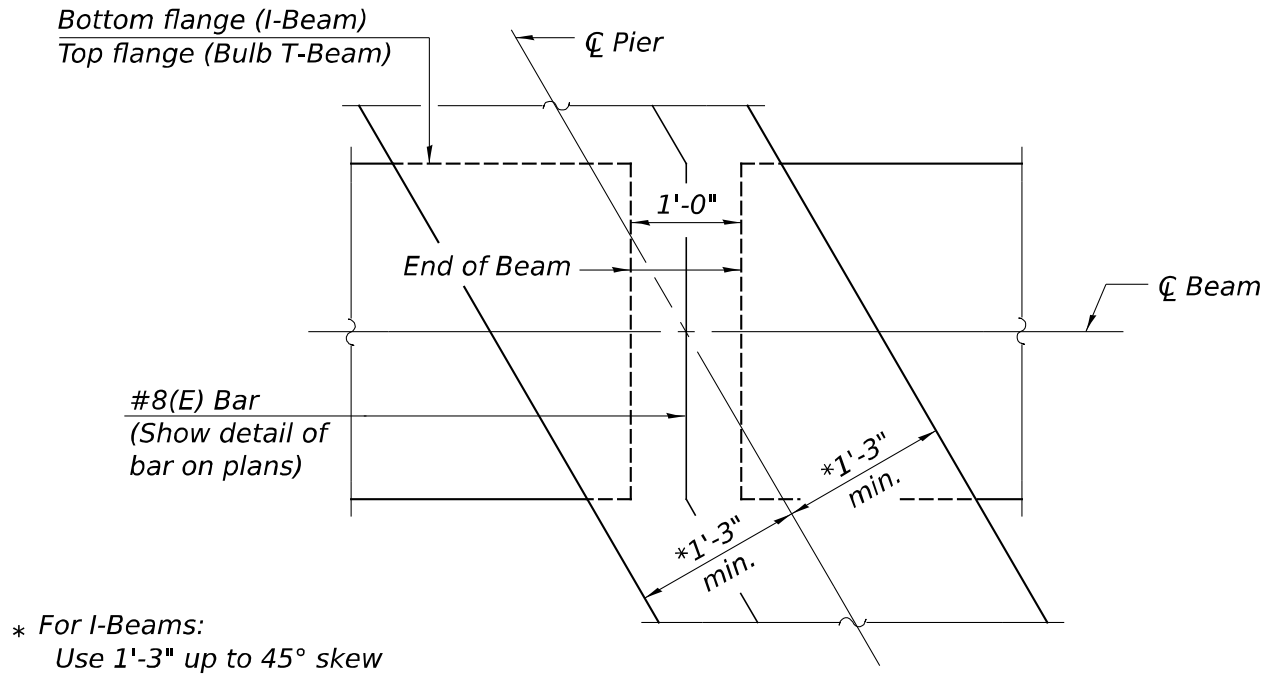
S = No. of beam spaces.
DL = Sum of all superstructure dead loads at the given pier under consideration (kips).

*** ℄ 1½" Ø x 18" Anchor bolts with 3" x 3" x 5 5/16" ℄ washer under nut. Holes in cap to be formed or drilled after beams are in place but prior to pouring concrete diaphragm.

FIXED PIER
DETAILS SKEWED

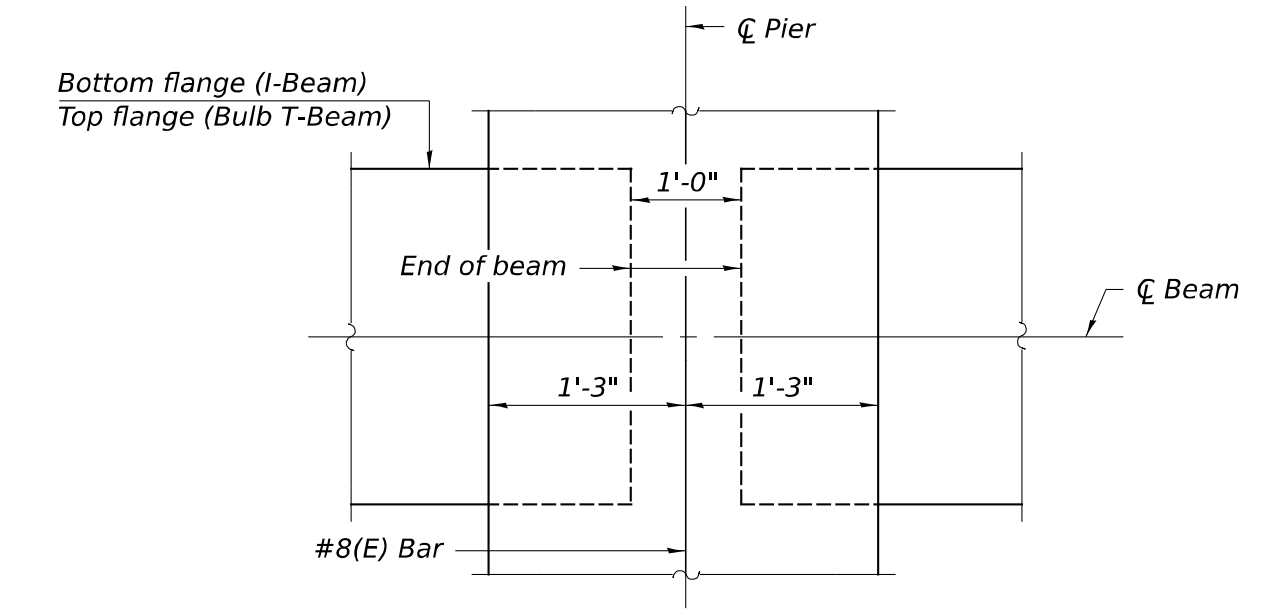
Figure 3.4.9-5

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* For I-Beams:
Use 1'-3" up to 45° skew
Use 1'-4½" over 45° skew
For Bulb T-Beams:
Use 1'-3" up to 20° skew
Use 1'-9" for skews between 20° and 40°
Use 2'-3" for skews over 40°

SKEWED DIAPHRAGM

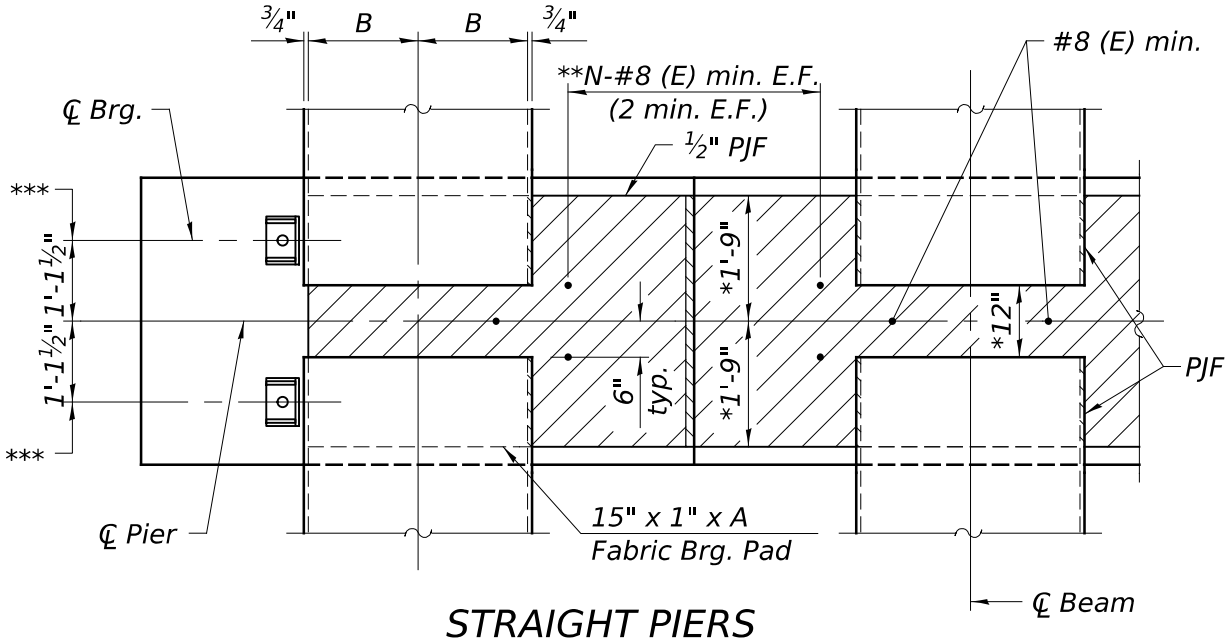


RT. ANGLE DIAPHRAGM

DIAPHRAGM DIMENSIONS AT PIERS

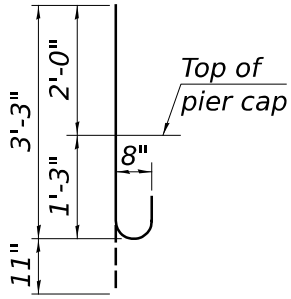
Figure 3.4.9-6

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STRAIGHT PIERS

A = Bott. flange width - 1½"
B = A/2



#8(E) (MIN.) DOWEL BAR

* See Figure 3.4.9-9.

** $N = \frac{1}{2}\left[\frac{0.2\ DL}{28.3\ (S)} - 2\right]$

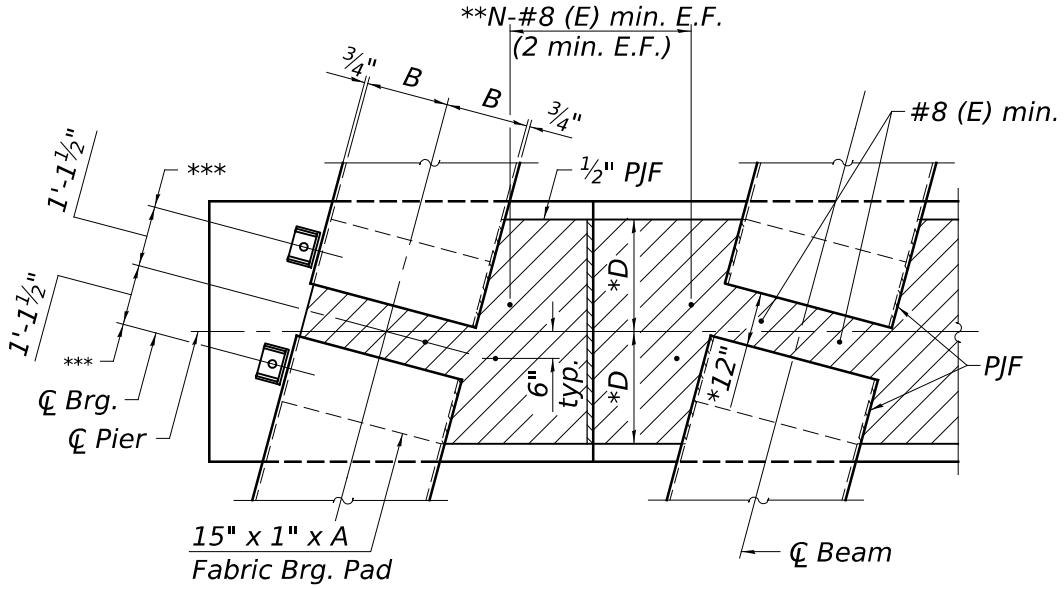
S = No. of beam spaces.
DL = Sum of all superstructure dead loads at the given pier under consideration (kips).

*** ℄ 1½" Ø x 18" Anchor bolts with 3" x 3" x 5 5/16" ℄ washer under nut. Holes in cap to be formed or drilled after beams are in place but prior to pouring concrete diaphragm.

FIXED PIER DETAILS
FOR IL-BEAMS NO SKEW

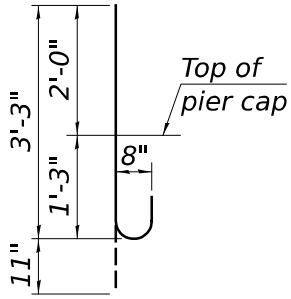
Figure 3.4.9-7

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SKEWED PIERS

A = Bott. flange width - 1½"
B = A/2



#8(E) (MIN.) DOWEL BAR

* See Figure 3.4.9-9.

** $N = \frac{1}{2}\left[\frac{0.2\ DL}{28.3\ (S)} - 2\right]$

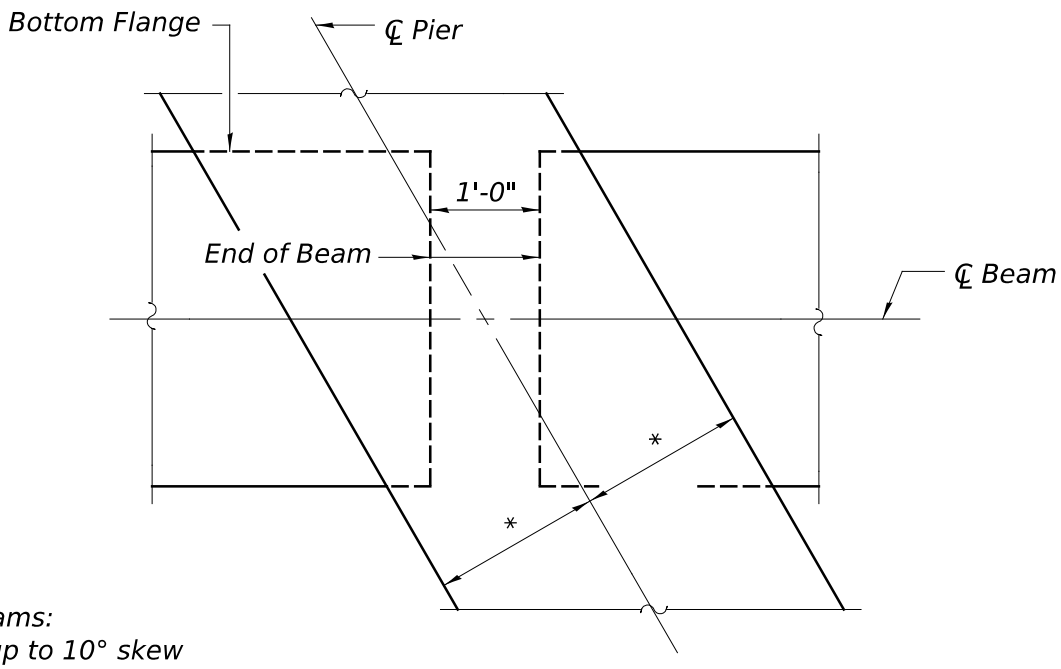
S = No. of beam spaces.
DL = Sum of all superstructure dead loads at the given pier under consideration (kips).

*** ℄ 1½" Ø x 18" Anchor bolts with 3" x 3" x 5 5/16" PL washer under nut. Holes in cap to be formed or drilled after beams are in place but prior to pouring concrete diaphragm.

FIXED PIER DETAILS
FOR IL-BEAMS SKEWED

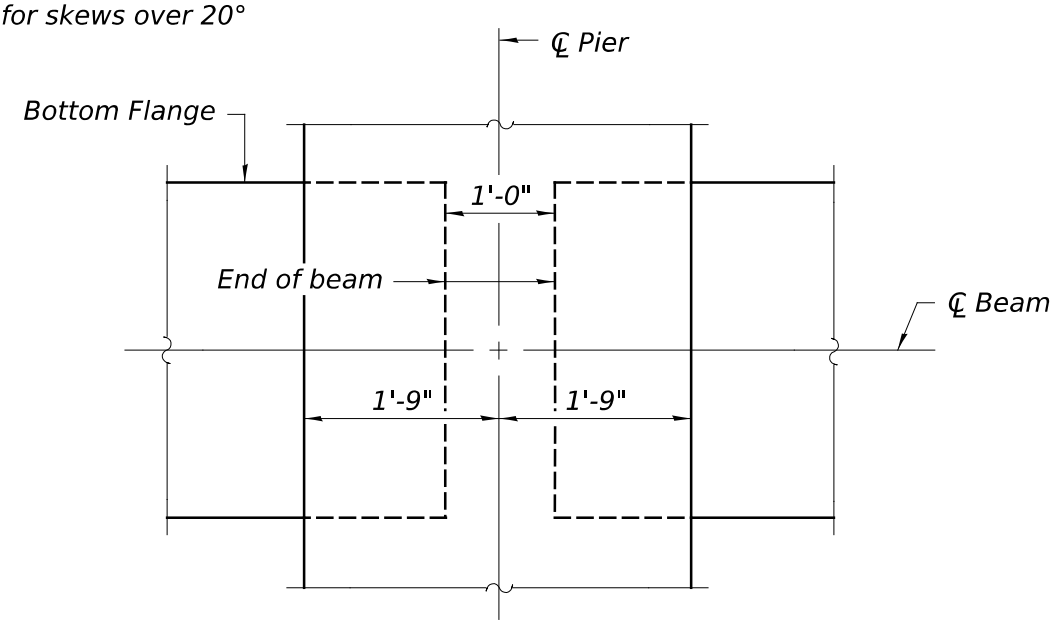
Figure 3.4.9-8

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∗ For IL27 Beams:
Use 2'-0" up to 10° skew
Use 2'-2" over 10° skew
For IL36 thru IL90 Beams:
Use 2'-0" up to 10° skew
Use 2'-2" for skews between 10° and 20°
Use 2'-4" for skews over 20°

SKEWED DIAPHRAGM

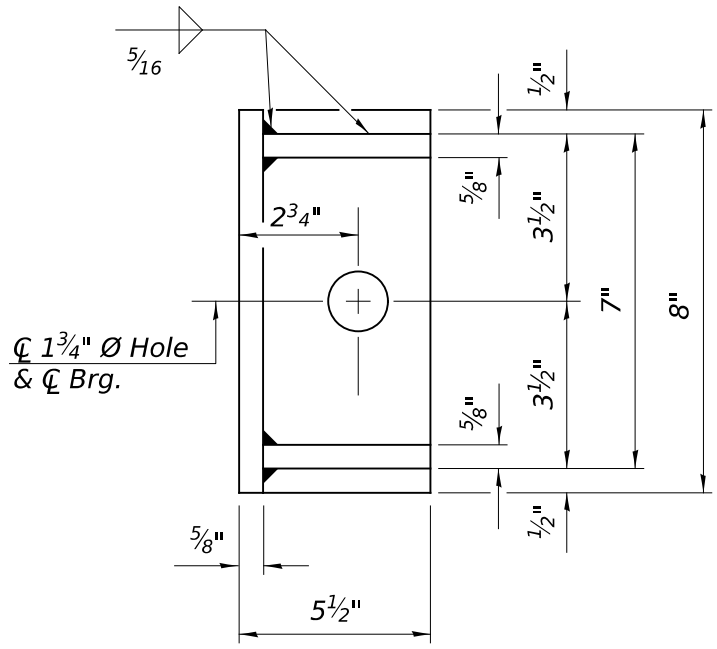


RT. ANGLE DIAPHRAGM

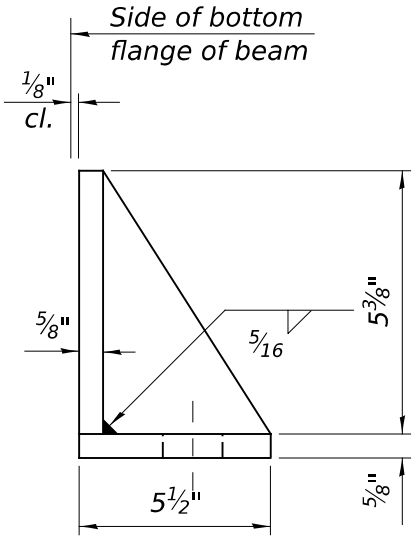
DIAPHRAGM DIMENSIONS
AT PIERS FOR IL-BEAMS

Figure 3.4.9-9

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PLAN



ELEVATION

Notes:

Equivalent rolled angle with stiffeners will be allowed in lieu of welded plates.

The side retainers shall be galvanized after shop fabrication according to AASHTO M 111 and ASTM 385.

SIDE RETAINER
AT FIXED PIERS

Figure 3.4.9-10

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3.5 PPC Deck Beams

This section covers the Department's policies and details related to precast prestressed concrete deck beam designs per the AASHTO LRFD Bridge Design Specifications. All deck beams on the State and Local system as well as all replacement beams shall be designed according to these specifications.

General selection charts based on standard loading cases and design criteria may be found in Section 2.3.6.1.2. These charts help choose an appropriate beam size and strand pattern for a given span length; however, they shall be verified through computations as described in this section and Design Guide 3.5 found on the Bridges & Structures page.

Deck beams shall be fixed at all supports for structures with lengths of 300 feet or less. Structures lengths greater than 300 feet shall typically have an expansion joint; however, all fixed supports may be utilized on structures with lengths greater than 300 feet provided all thermal forces are accounted for in the design.

All deck beam structures on State routes shall have an initial 5 inch minimum Concrete Wearing Surface (CWS). The 5 inch minimum CWS is not considered in the resisting moment section modulus or in the live load distribution scheme.

3.5.1 Design Guide

Design Guide 3.5 provides a detailed explanation of the Department's policies for various design components and is followed by a worked design example. The design components covered are:

- Dead Loads
- Live Loads and live load distribution factors
- Transverse tie locations
- Losses / Gains
- Section Properties
- Temporary stresses
- Service stresses
- Fatigue stresses
- Flexural Resistance
- Minimum Reinforcement
- Camber and Deflection

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Deck beams with a 5 in. concrete wearing surface (CWS) shall use the “Approximate” method (AASHTO LRFD Bridge Design Specifications Article 5.9.3.3) method for time dependent losses. The Department believes the concrete wearing surface behaves as a partially composite section with the beams even though it's not considered to be partially composite for strength calculations. This logic is based upon the required textured broom finish on the top surface of the beam, the D(E) bars that protrude from the tops of the exterior beams for F-shaped barriers and Type SM railings, and because there have been no signs of separation between the beams and the CWS on the Department’s inventory inspections.

Deck beams with no wearing surface or a bituminous wearing surface shall also use the “Approximate” for time dependent losses. However, in order to satisfy the spirit of the AASHTO LRFD Bridge Design Specifications, a correction factor shall be added to the results obtained from the “Approximate” method to more closely match the intent of the “Refined” method (AASHTO LRFD Bridge Design Specifications Article 5.9.3.4). The correction factors are as follows:

Deck beam depth (in.)	Correction factor (ksi)
11	4.5
17	4.5
21	4.5
27	4.5
33	3.5
42	2.5

For example: If the designer calculates an “Approximate” time dependent loss of 20 ksi for a 17 inch beam, a correction factor of 4.5 ksi should be added to make it 24.5 ksi.

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3.5.2 Stress Limits

The provisions for stress limits in Section 3.4.2 are also applicable for Precast Prestressed Deck Beams except that the concrete compressive strength is 6.0 ksi with a release strength of 5.0 ksi and only strands with a nominal diameter of ½ in. are utilized.

3.5.3 Strand Pattern Tables

Tables 3.5.3-1 through 3.5.3-6 detail standard strand patterns for use with standard IDOT deck beam shapes. The strand alignments are set up in a 2 inch by 2 inch grid similar to I-Beams and Bulb T-Beams rather than a staggered arrangement. This allows for better strand placement versatility and permits a common stressing block and template for all beam sections. The strand patterns are also intentionally configured with the number of strands in the bottom row equal to or less than the second row to provide for better redundancy against corrosion. Strand patterns with the least number of strands for a given section are governed by minimum reinforcement requirements of the AASHTO LRFD Bridge Design Specifications. All strand patterns were arranged such that the temporary stresses are satisfied. The upper most strand pattern for any given section is a function of these stresses which are controlled by the concrete strength.

The strand pattern designation format for deck beams is the same format used for I-Beams, Bulb T-Beams, and IL beams. Since deck beams are simple spans with straight strand patterns and standard strength concrete, the designations for deck beams is simply the number of strands followed by SS.

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LRFD 11x48 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.										
8SS	4	4										2.73
10SS	4	6										2.53
12SS	6	6										2.73
14SS	6	8										2.59
16SS	8	8										2.73

LRFD 11x52 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.										
8SS	4	4										2.73
10SS	4	6										2.53
12SS	6	6										2.73
14SS	6	8										2.59
16SS	8	8										2.73
18SS	8	10										2.62

Table 3.5.3-1

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LRFD 17x36 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	11 ¾ in.									
10SS	4	4	2									3.86
12SS	4	6	2									3.99
14SS	6	6	2									4.37
16SS	6	8	2									4.41
18SS	8	8	2									4.66

LRFD 17x48 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	11 ¾ in.									
12SS	4	6	2									4.01
14SS	6	6	2									4.40
16SS	6	8	2									4.43
18SS	8	8	2									4.68
20SS	8	10	2									4.68
22SS	10	10	2									4.86
24SS	10	12	2									4.85

Table 3.5.3-2

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LRFD 21x36 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	15 ¾ in.							
10SS	4	4	2									7.03
12SS	4	4	2	2								6.29
14SS	4	6	2	2								6.34
16SS	6	8			2							5.88
18SS	6	8	2		2							5.74
20SS	8	8	2		2							6.03
22SS	8	10		2	2							5.90

LRFD 21x48 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	15 ¾ in.							
14SS	6	6	2									7.23
16SS	6	6	2	2								6.65
18SS	8	8			2							6.21
20SS	8	10			2							6.25
22SS	10	10			2							6.47
24SS	10	10	2		2							6.32
26SS	10	10	2	2	2							6.04
28SS	10	12	2	2	2							6.08

Table 3.5.3-3

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LRFD 27x36 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	21 ¾ in.							
12SS	4	6	2									9.88
14SS	4	6	2	2								9.26
16SS	6	8			2							8.05
18SS	8	8			2							8.43
20SS	8	8	2		2							8.35
22SS	8	10	2		2							8.45
24SS	8	10	2	2	2							8.21

LRFD 27x48 Deck Beam Strand Patterns (½ in. ϕ strands)												
Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	21 ¾ in.							
16SS	6	8	2									10.08
18SS	6	8	2	2								9.58
20SS	8	10			2							8.58
22SS	10	10			2							8.86
24SS	10	12			2							8.92
26SS	12	12			2							9.12
28SS	12	12	2		2							9.01
30SS	12	12	2	2	2							8.78

Table 3.5.3-4

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LRFD 33x36 Deck Beam Strand Patterns (½ in. ϕ strands)

Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	9 ¾ in.	11 ¾ in.	13 ¾ in.	25 ¾ in.	27 ¾ in.			
14SS	4	6	2	2								12.18
16SS	4	6	2	2	2							11.47
18SS	6	6		2	2	2						11.13
20SS	8	10							2			10.87
22SS	8	10	2						2			10.83
24SS	10	10					2		2			10.47
26SS	10	10	2				2		2			10.47
28SS	10	10	2	2			2		2			10.32

LRFD 33x48 Deck Beam Strand Patterns (½ in. ϕ strands)

Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	9 ¾ in.	11 ¾ in.	13 ¾ in.	25 ¾ in.	27 ¾ in.			
18SS	6	8	2	2								12.52
20SS	6	8	2	2	2							11.92
22SS	8	8		2	2	2						11.61
24SS	8	8	2	2	2	2						11.52
26SS	12	12							2			11.59
28SS	12	12	2						2			11.52
30SS	12	12	2	2					2			11.32
32SS	12	12	2	2	2				2			11.02
34SS	12	14	2	2			2		2			10.87

Table 3.5.3-5

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LRFD 42x36 Deck Beam Strand Patterns (½ in. ϕ strands)

Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	9 ¾ in.	11 ¾ in.	13 ¾ in.	15 ¾ in.	17 ¾ in.	33 ¾ in.	35 ¾ in.	
16SS	4	4	2	2	2	2						14.88
18SS	8	8									2	14.21
20SS	8	8	2								2	14.28
22SS	8	8	2	2							2	14.15
24SS	8	8	2	2	2						2	13.88
26SS	8	8	2	2	2	2					2	13.49
28SS	8	10	2	2	2				2		2	13.31
30SS	8	10	2	2	2	2	2				2	13.28
32SS	8	10	2	2	2	2	2	2			2	12.75
34SS	10	10		2	2	2	2	2	2		2	12.41

LRFD 42x48 Deck Beam Strand Patterns (½ in. ϕ strands)

Strand Pattern	Location of Strands Up from Bottom of Beam											E (in.)
	1 ¾ in.	3 ¾ in.	5 ¾ in.	7 ¾ in.	9 ¾ in.	11 ¾ in.	13 ¾ in.	15 ¾ in.	17 ¾ in.	33 ¾ in.	35 ¾ in.	
20SS	8	8	2						2			16.13
22SS	6	8	2	2	2				2			15.12
24SS	10	12									2	15.10
26SS	10	10	2	2							2	14.78
28SS	10	10	2	2	2						2	14.51
30SS	10	12	2	2	2						2	14.67
32SS	10	12	2	2	2	2					2	14.31
34SS	12	12	2	2	2	2					2	14.58
36SS	12	12	2	2	2	2	2				2	14.16
38SS	14	14	2	2	2					2	2	13.78
40SS	14	14	2	2	2	2				2	2	13.53

Table 3.5.3-6

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3.5.4 Design Properties and Standard Beam Cross Sections

Table 3.5.4-1 shows pertinent beam section properties necessary for the design of each beam followed by Figures 3.5.4-1 through 3.5.4-12 which detail the twelve standard deck beam cross sections. These figures include the basic cross sectional dimensions and a layout of the standard grid system for location of strands.

Deck Beam Design Properties

Beam	Net Area ($in.^2$)	I ($in.^4$)	S_b ($in.^3$)	S_t ($in.^3$)	K	C_b (in.)	C_t (in.)	Net Weight (lbs/ft.)	Solid Weight (lbs/ft.)	Shear Key Wt. (lbs/ft.)
11x48	513.2	5191	947.0	940.8	0.59	5.48	5.52	-	535	13
11x52	557.2	5635	1027.7	1021.5	0.58	5.48	5.52	-	580	13
17x36	471.9	13924	1655.6	1621.0	0.72	8.41	8.59	492	621	14
17x48	615.9	18658	2213.0	2177.4	0.68	8.43	8.57	642	834	14
21x36	527.9	25255	2434.0	2377.2	0.75	10.38	10.62	550	771	14
21x48	679.9	33685	3237.8	3178.9	0.70	10.40	10.60	708	1033	14
27x36	569.9	49697	3738.1	3626.1	0.81	13.30	13.71	594	986	25
27x48	701.9	65288	4896.7	4777.2	0.73	13.33	13.67	731	1323	25
33x36	641.9	84463	5208.4	5032.6	0.87	16.22	16.78	669	1211	25
33x48	773.9	109761	6748.3	6558.3	0.78	16.27	16.74	806	1623	25
42x36	749.9	158672	7692.0	7424.3	0.96	20.63	21.37	781	1548	25
42x48	881.9	202984	9813.7	9522.5	0.84	20.68	21.32	919	2073	25

Table 3.5.4-1

I = Moment of inertia of the prestressed beam

S_b = Non-composite section modulus for the bottom fiber of the prestressed beam

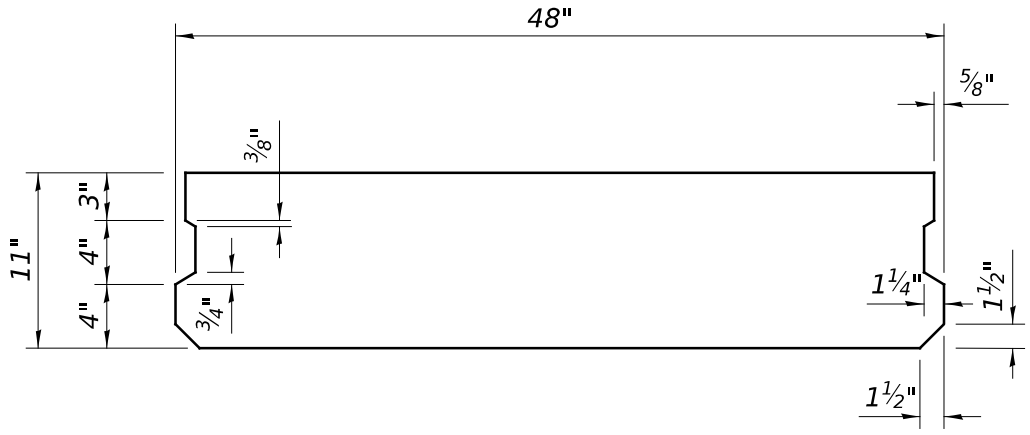
S_t = Non-composite section modulus for the top fiber of the prestressed beam

K = Constant for different types of construction

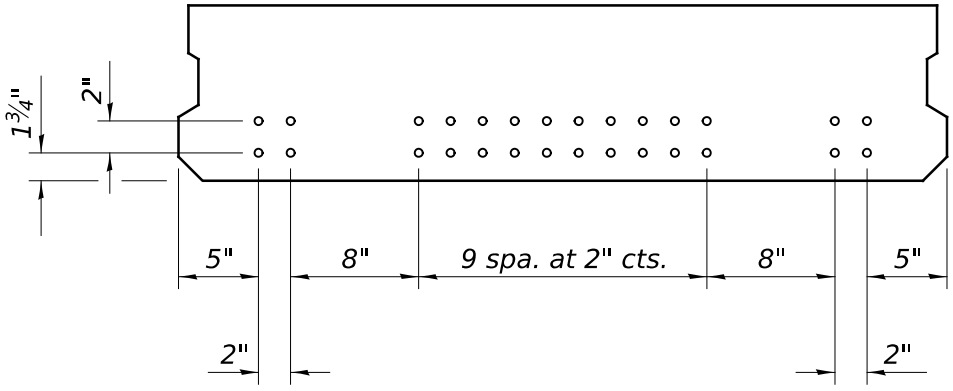
C_b = Distance from the centroid of the prestressed beam to the bottom of the beam

C_t = Distance from the centroid of the prestressed beam to the top of the beam

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CROSS SECTION
(Showing dimensions)

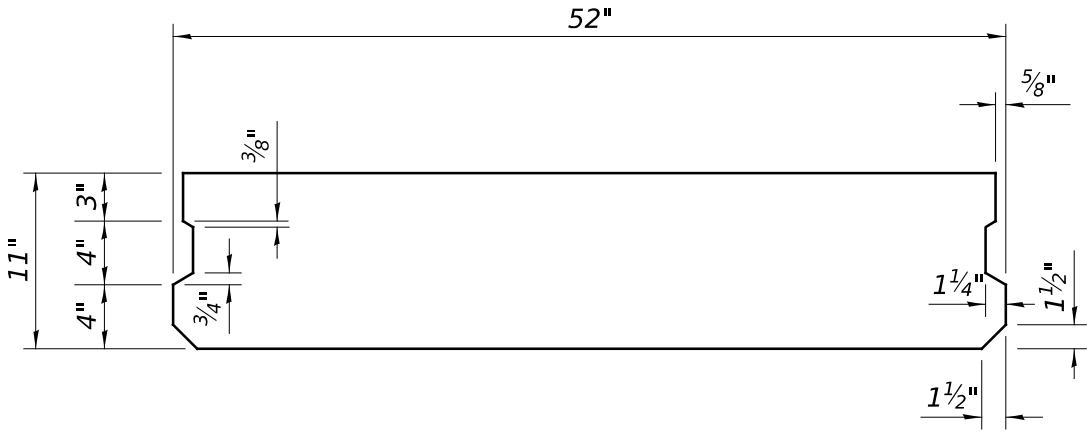


CROSS SECTION
(Showing permissible strand locations)

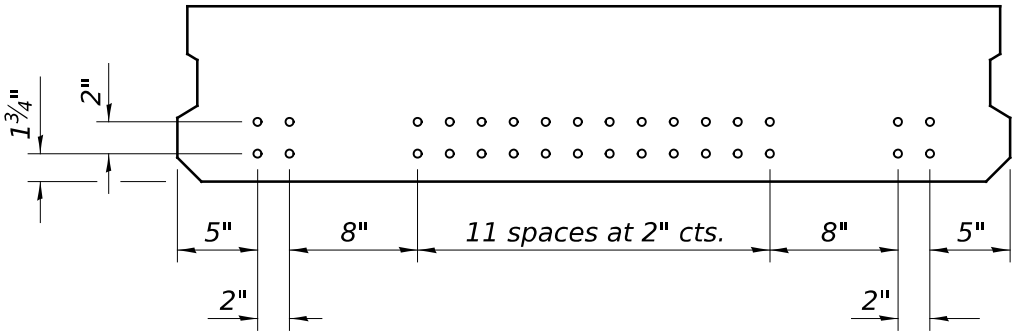
11" x 48" PPC
DECK BEAM

Figure 3.5.4-1

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CROSS SECTION
(Showing dimensions)

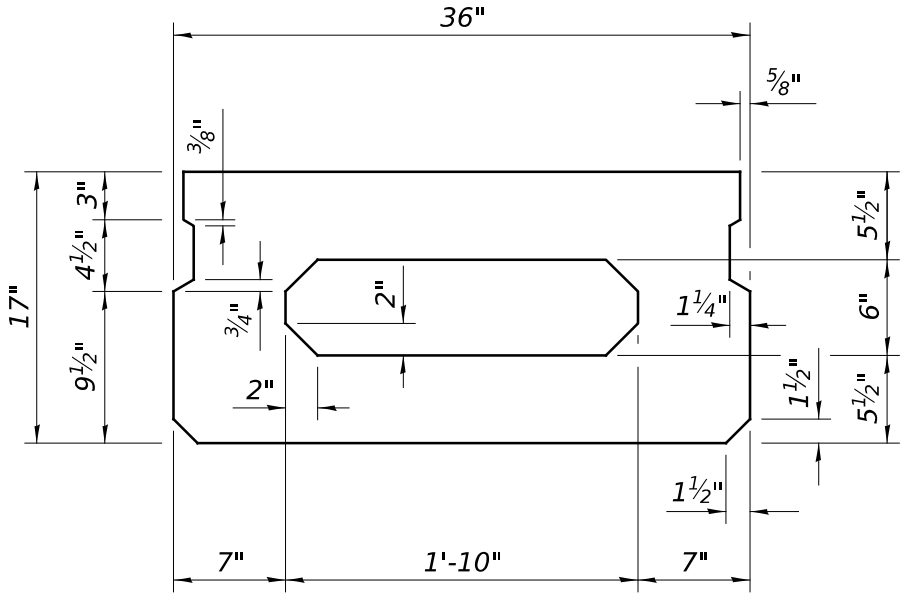


CROSS SECTION
(Showing permissible strand locations)

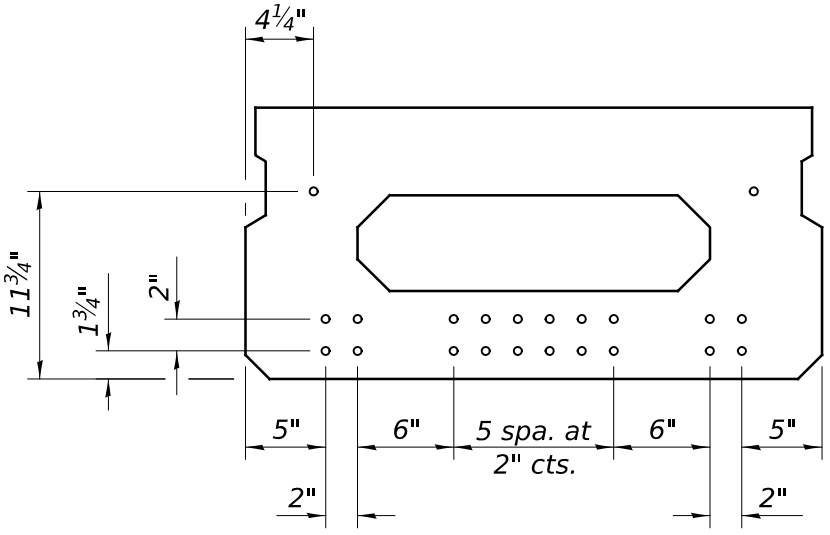
11" x 52" PPC
DECK BEAM

Figure 3.5.4-2

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CROSS SECTION
(Showing dimensions)

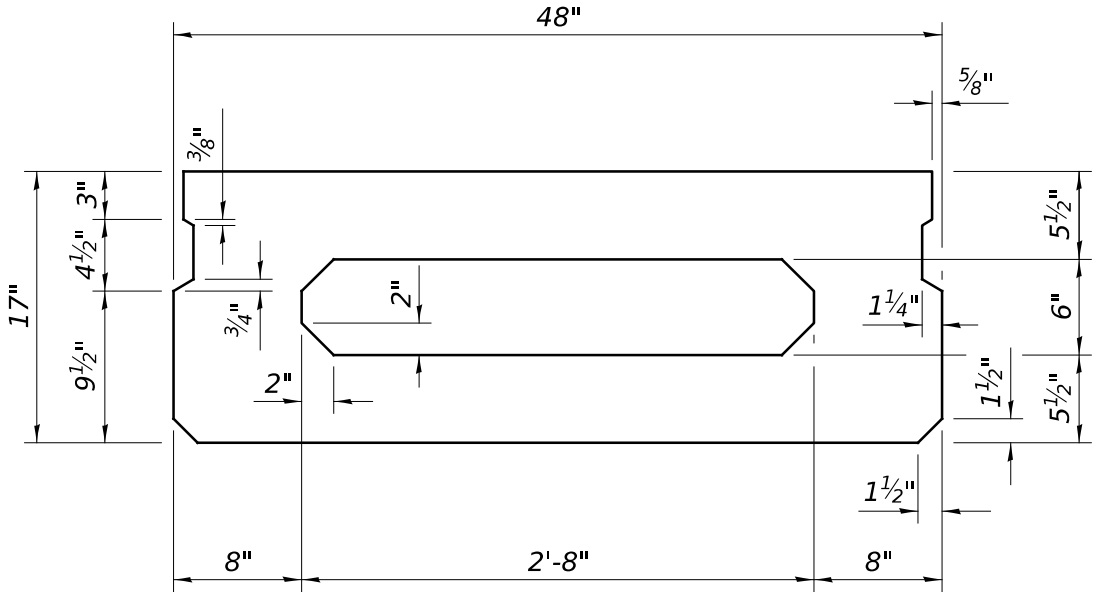


CROSS SECTION
(Showing permissible strand locations)

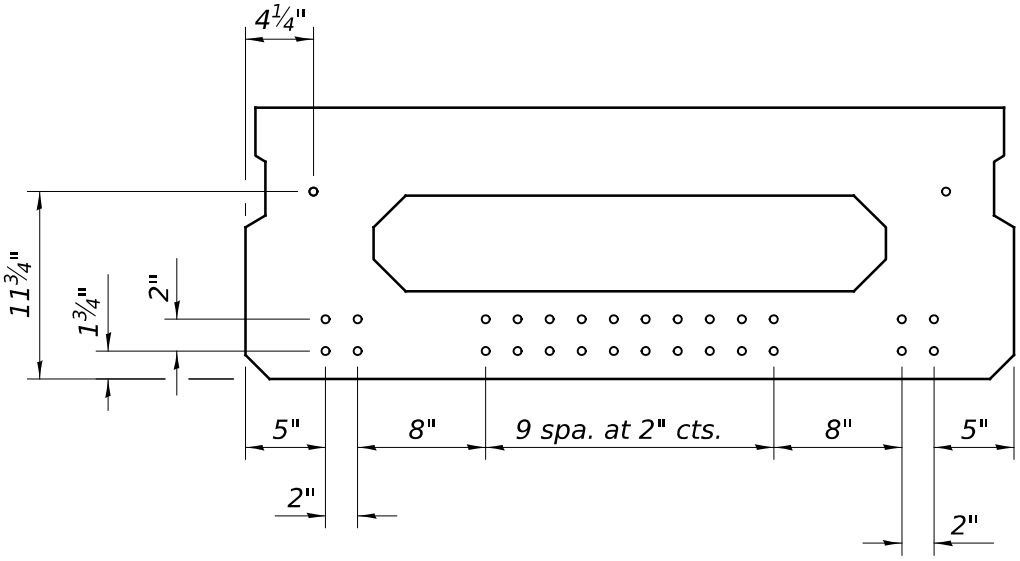
17" x 36" PPC
DECK BEAM

Figure 3.5.4-3

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree



CROSS SECTION
(Showing dimensions)

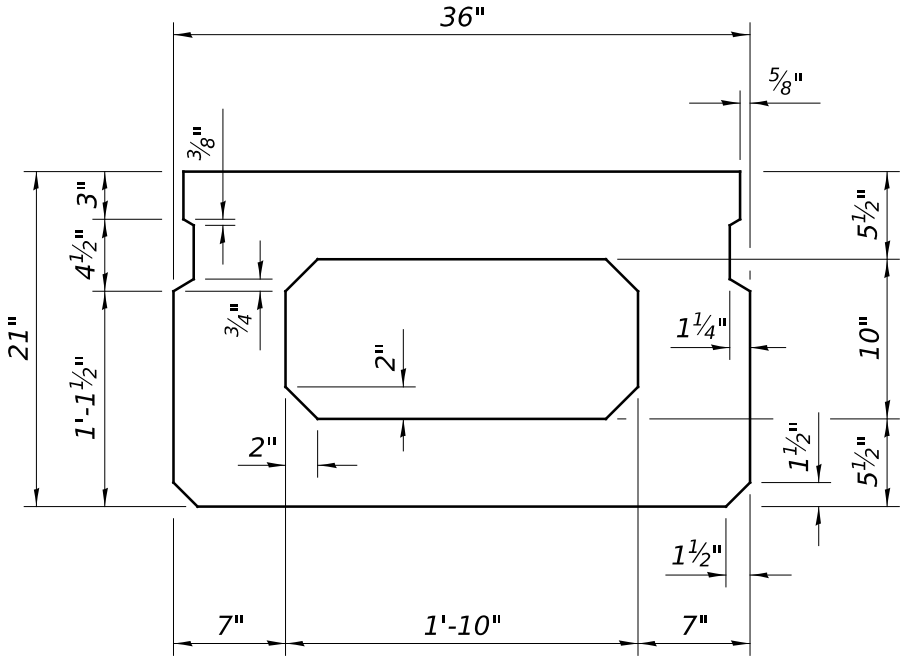


CROSS SECTION
(Showing permissible strand locations)

17" X 48" PPC DECK BEAM

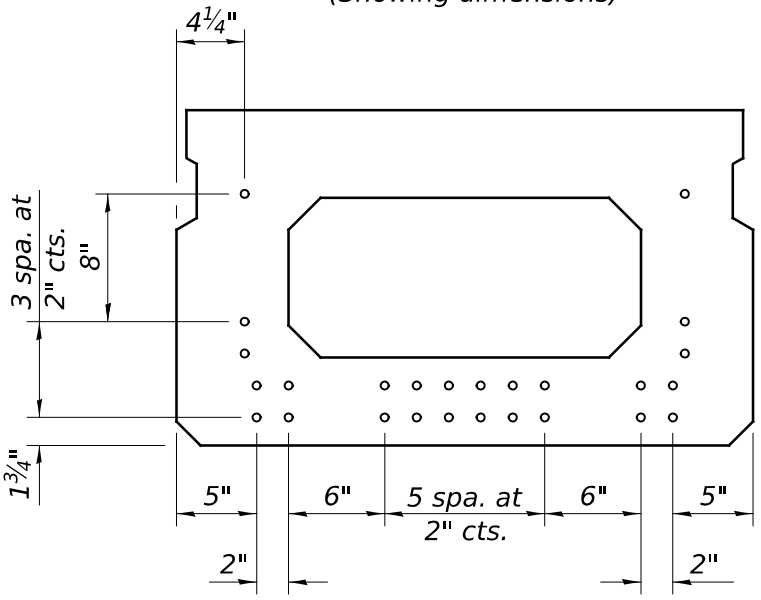
Figure 3.5.4-4

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree



CROSS SECTION

(Showing dimensions)



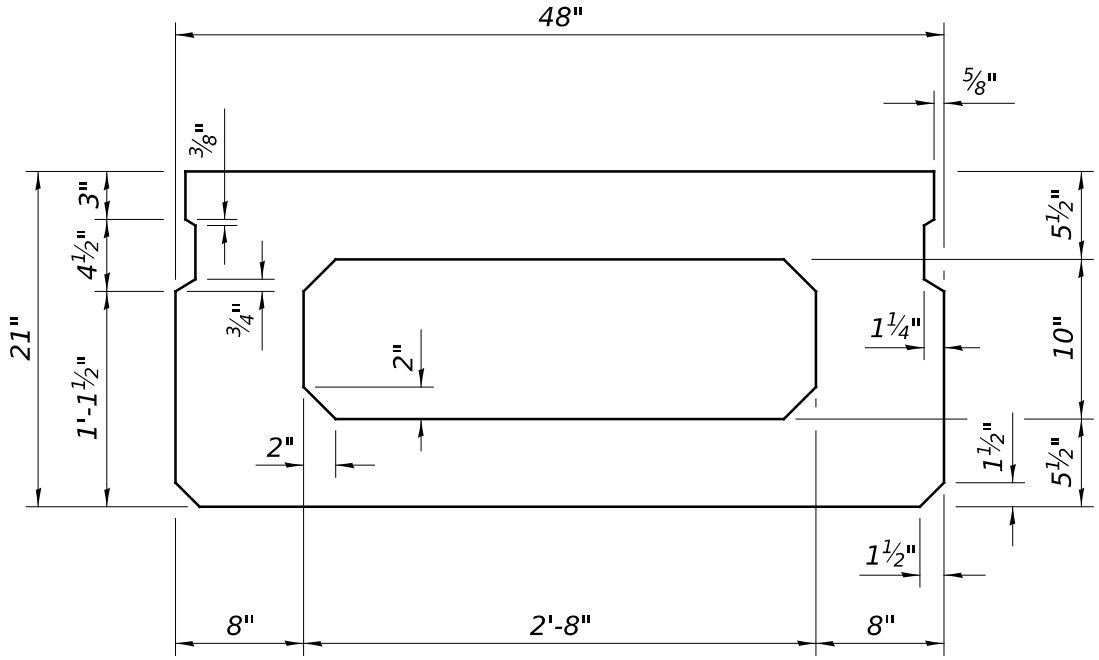
CROSS SECTION

(Showing permissible strand locations)

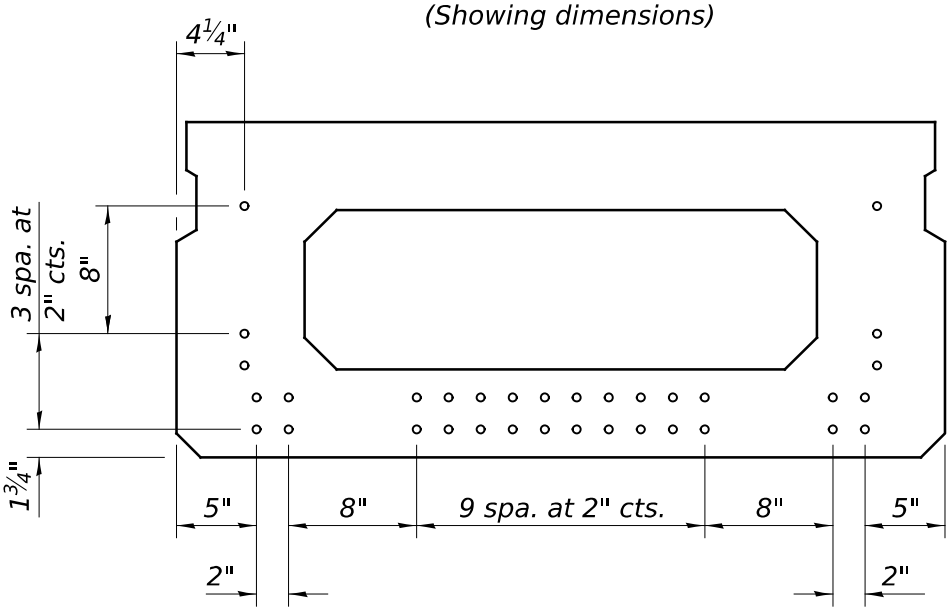
21" X 36" PPC DECK BEAM

Figure 3.5.4-5

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CROSS SECTION
(Showing dimensions)

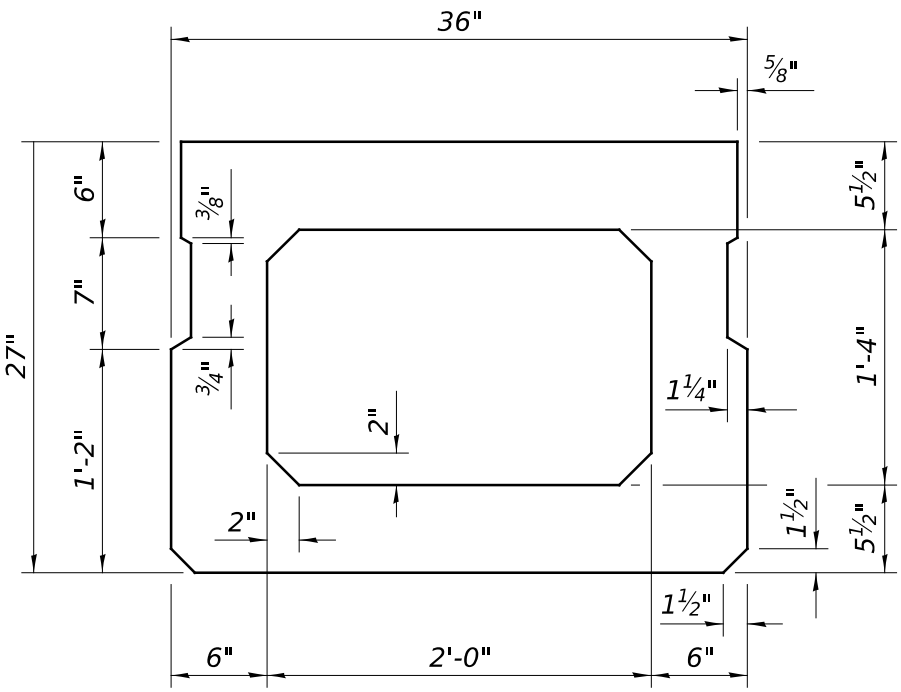


CROSS SECTION
(Showing permissible strand location)

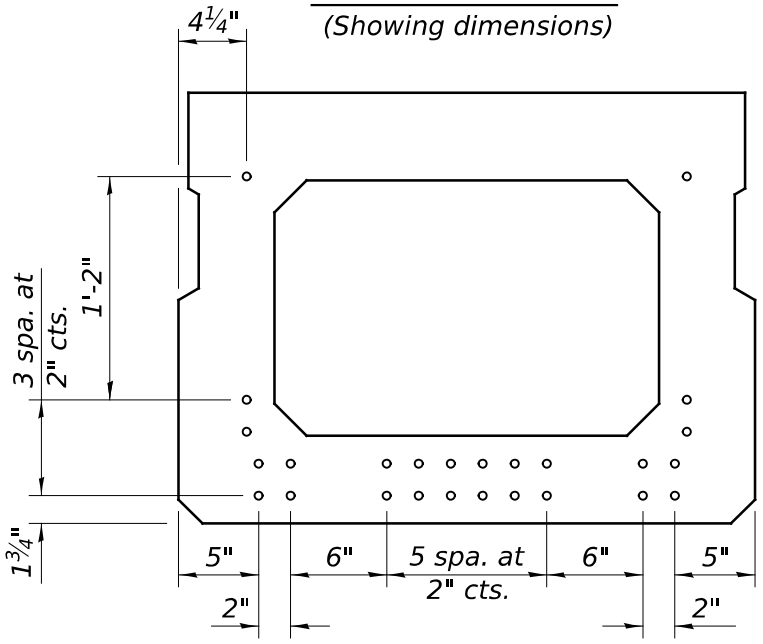
21" X 48" PPC
DECK BEAM

Figure 3.5.4-6

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CROSS SECTION
(Showing dimensions)



CROSS SECTION
(Showing permissible strand locations)

27" X 36" PPC
DECK BEAM

Figure 3.5.4-7

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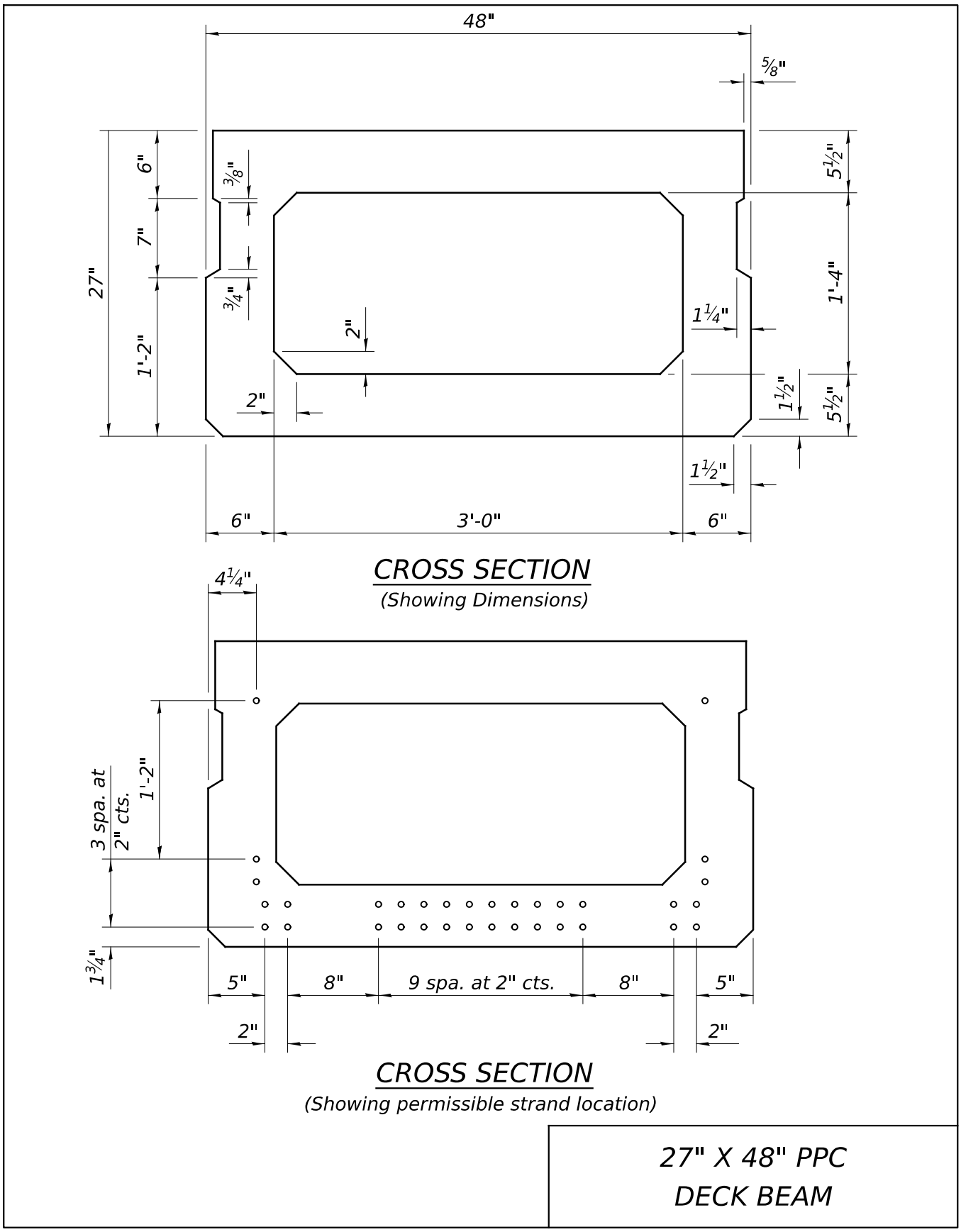
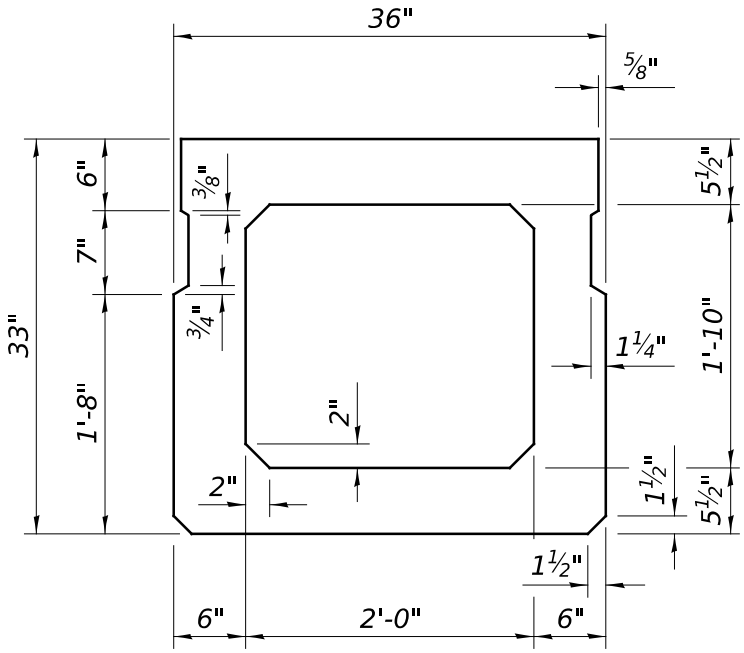
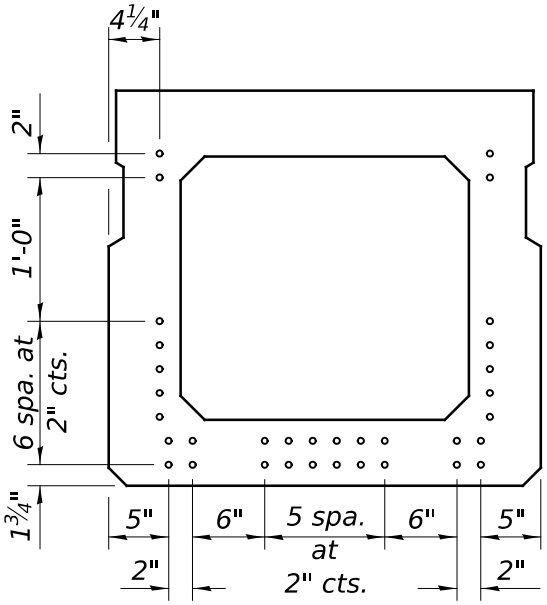


Figure 3.5.4-8

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CROSS SECTION
(Showing dimensions)

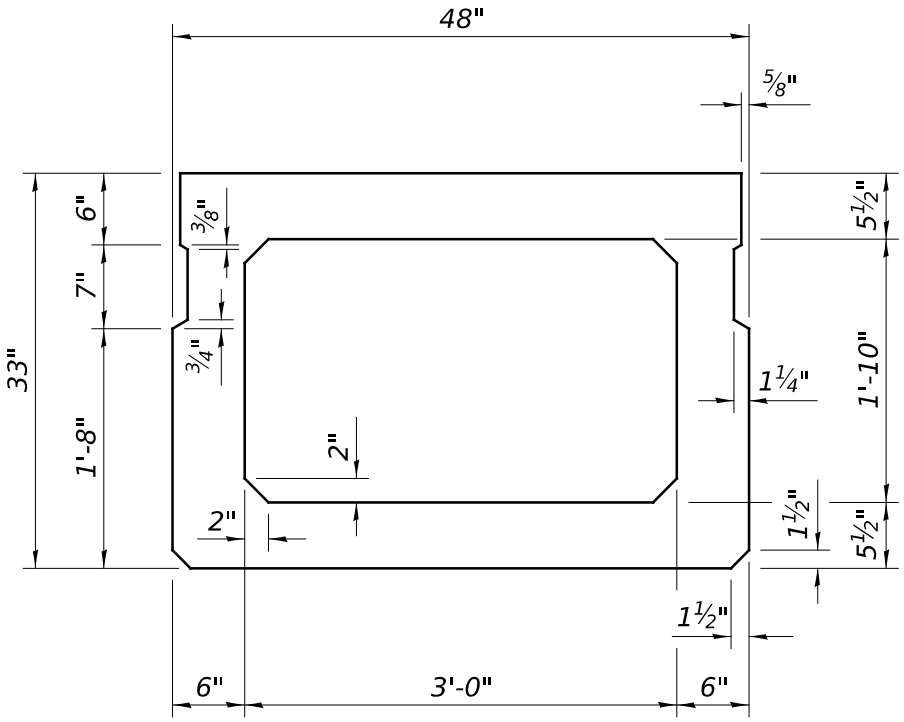


CROSS SECTION
(Showing permissible strand locations)

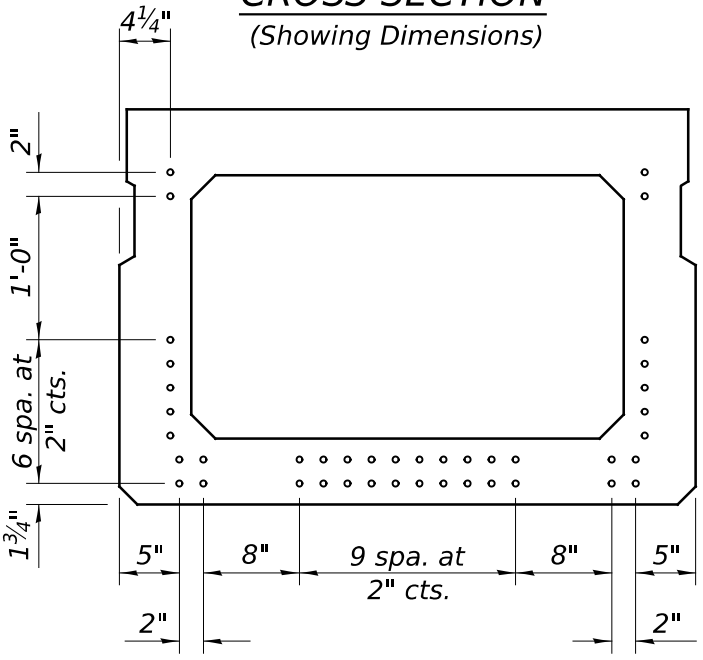
33" X 36" PPC DECK BEAM

Figure 3.5.4-9

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CROSS SECTION
(Showing Dimensions)



CROSS SECTION
(Showing permissible strand locations)

33" X 48" PPC
DECK BEAM

Figure 3.5.4-10

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

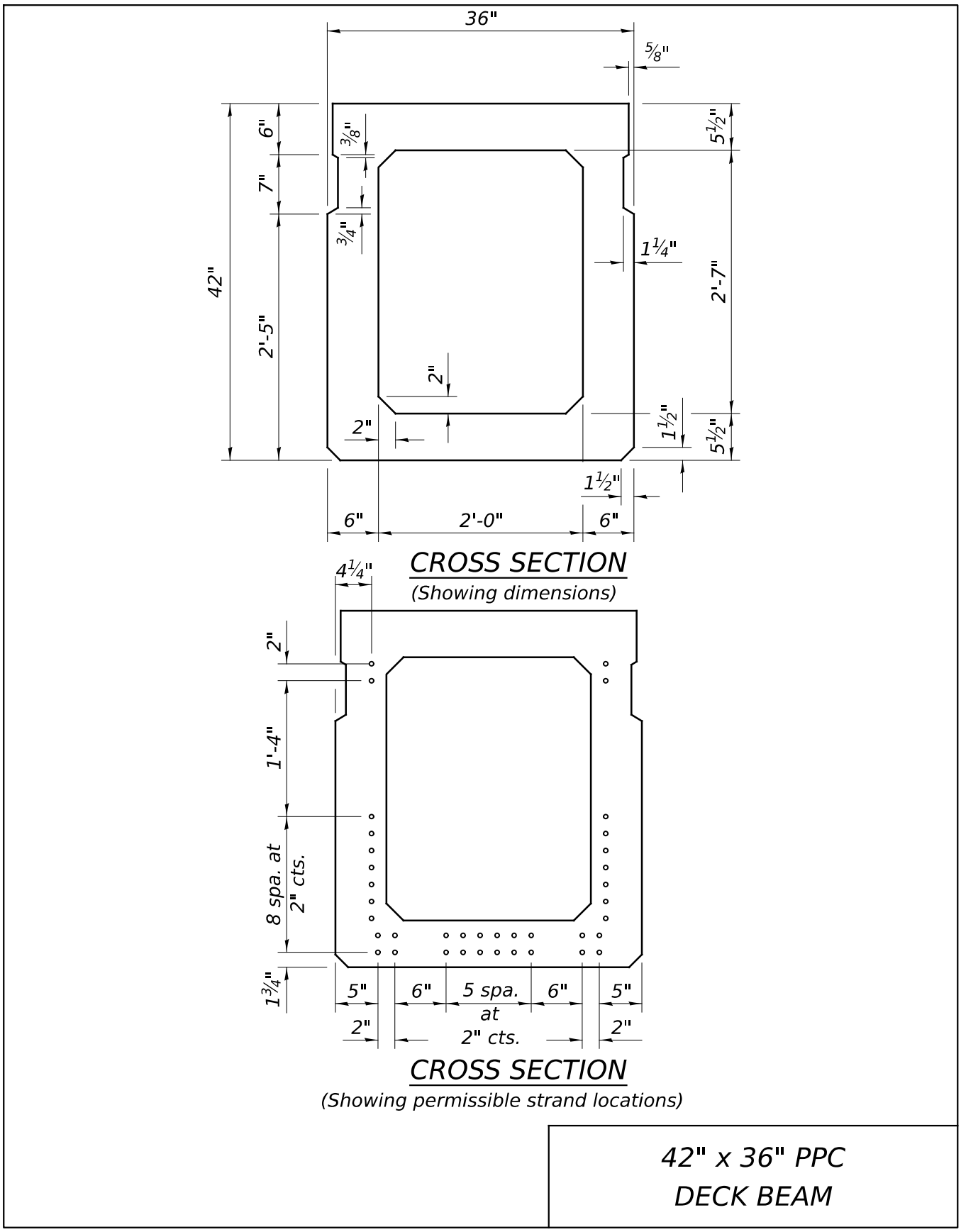


Figure 3.5.4-11

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

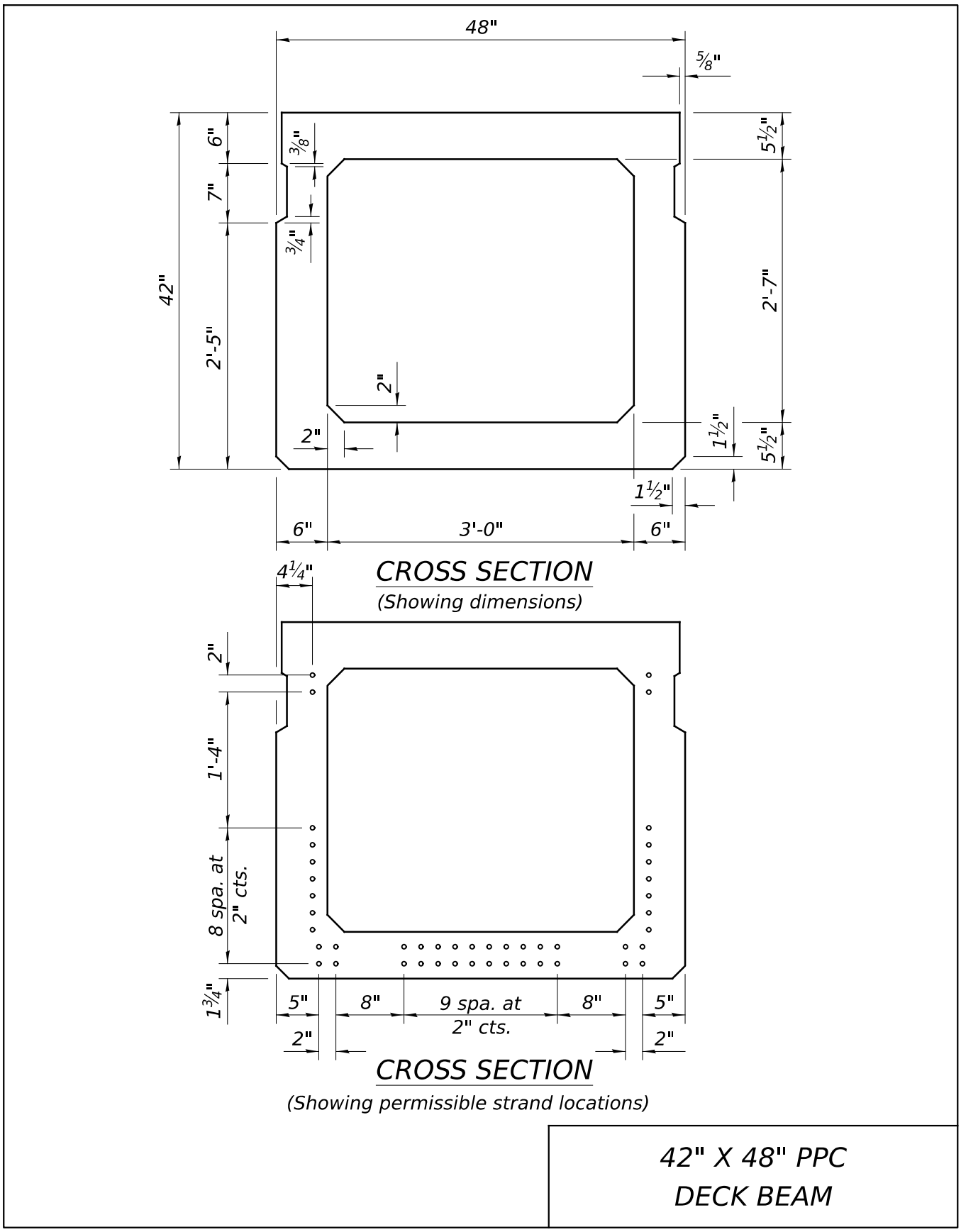


Figure 3.5.4-12

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3.5.5 Design for Shear

The shear reinforcement for the standard deck beam sections is shown on the base sheets which are available on the Department's web site. This shear reinforcement satisfies the requirements of the AASHTO LRFD Bridge Design Specifications for the standard load combinations specified in Section 2.3.6.1.2. Deck beams with skews, design properties and loads (i.e. sidewalks, medians, etc.) other than depicted in this manual and the base sheets shall be independently designed for shear according to the AASHTO LRFD Bridge Design Specifications . In no case shall an independently designed deck beam have less shear reinforcement than that specified in the base sheets.

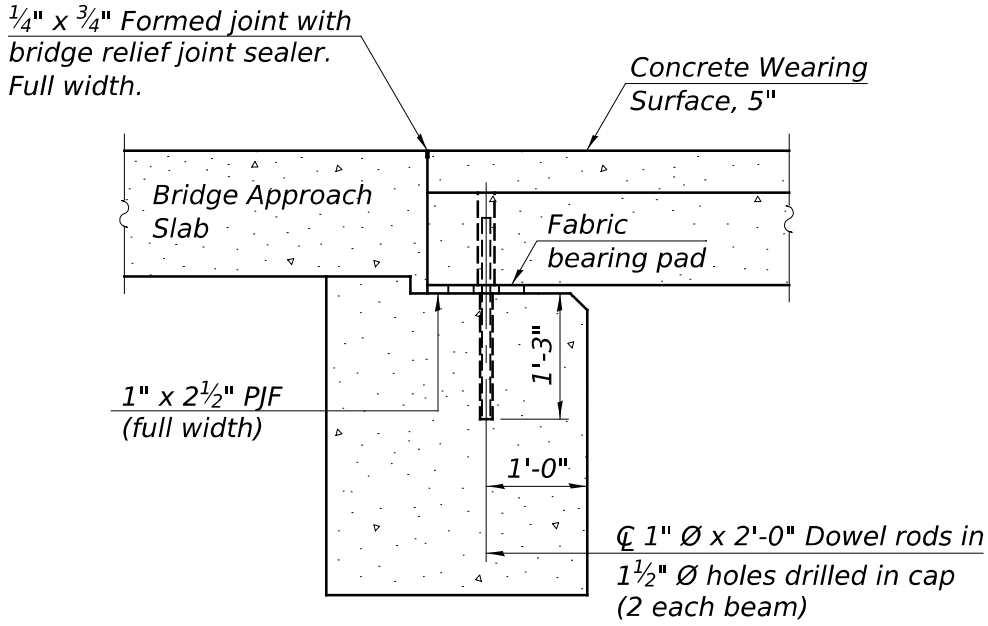
3.5.6 Fixed Substructures

Deck beam structures typically have fixed substructure connections as described previously in Section 3.5. The fixity is achieved by drilling and grouting dowel rods through the formed holes in the beam ends. The total sequence of construction is described in more detail in the standard specifications and in notes on the base sheets. Versions of the typical abutment and pier cross sections, suitable for inclusion on the base sheets, are available in the IDOT Detail Library.

3.5.6.1 Abutment Sections

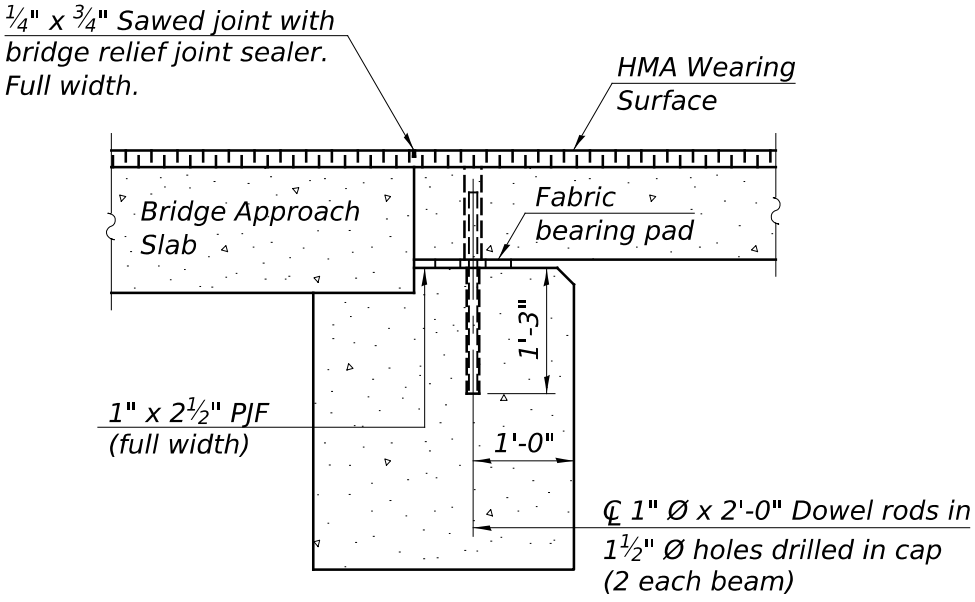
Figures 3.5.6.1-1 through 3.5.6.1-3 show the typical cross sections for the various deck beam depths having either a Concrete Wearing Surface (CWS) or a Hot Mix Asphalt (HMA) wearing surface. A joint is either formed for CWS or sawed for HMA in the wearing surface at the approach slab to deck beam interface where a crack is anticipated. The abutment is notched on the shallow 11 inch deck beams and a backwall is required on the deeper sections. Figure 3.5.6.1-4 shows the typical cross sections with no approach pavements which may be used on Local projects.

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SECTION THRU ABUTMENT

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



SECTION THRU ABUTMENT

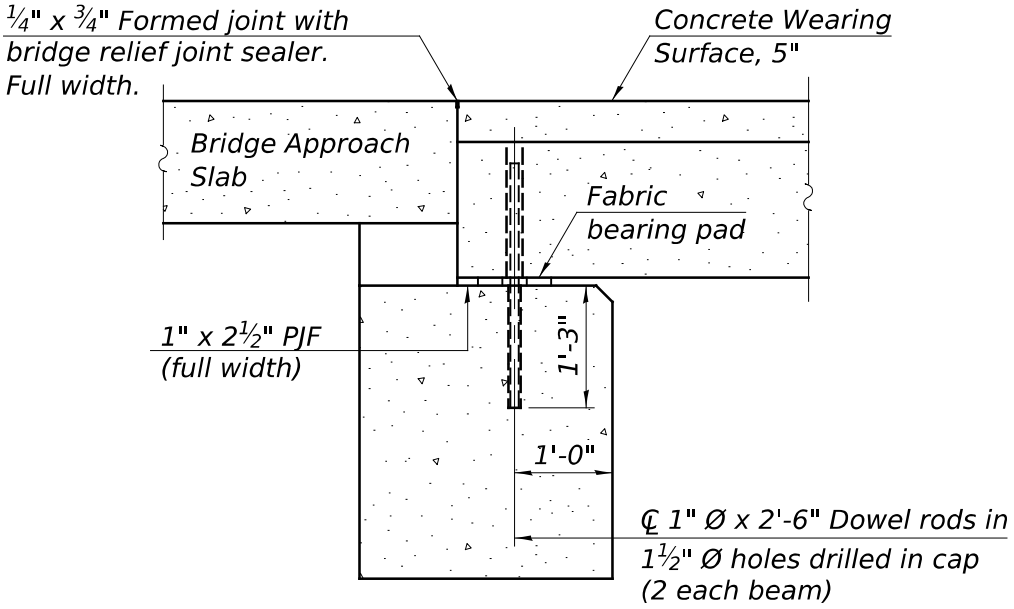
(With HMA wearing surface. Dimensions are at Rt. ∠'s)

Notes:
For abutment details and reinforcement, see Section 3.8.

FIXED ABUTMENT SECTIONS 11" BEAMS

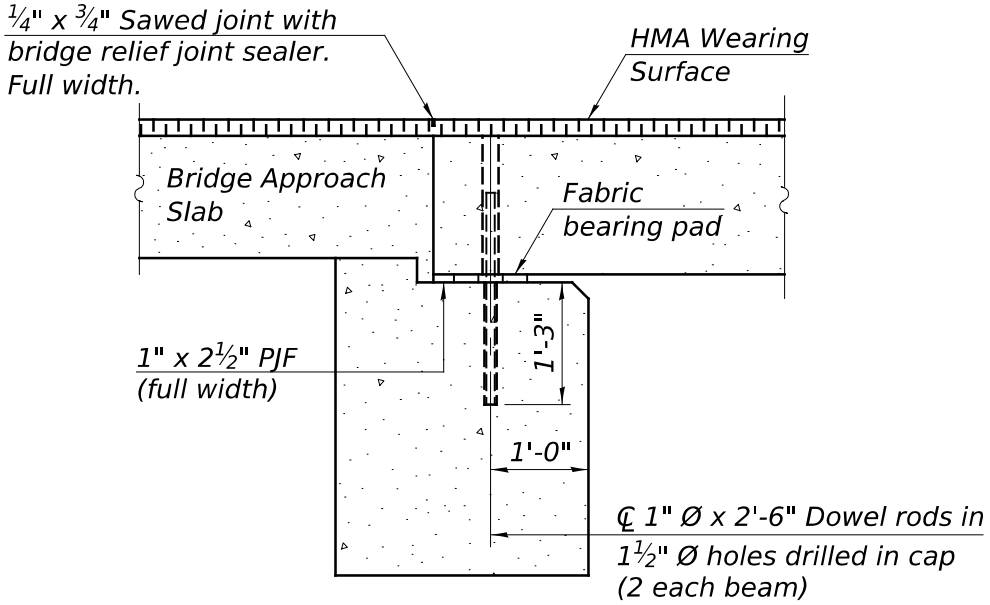
Figure 3.5.6.1-1

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SECTION THRU ABUTMENT

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



SECTION THRU ABUTMENT

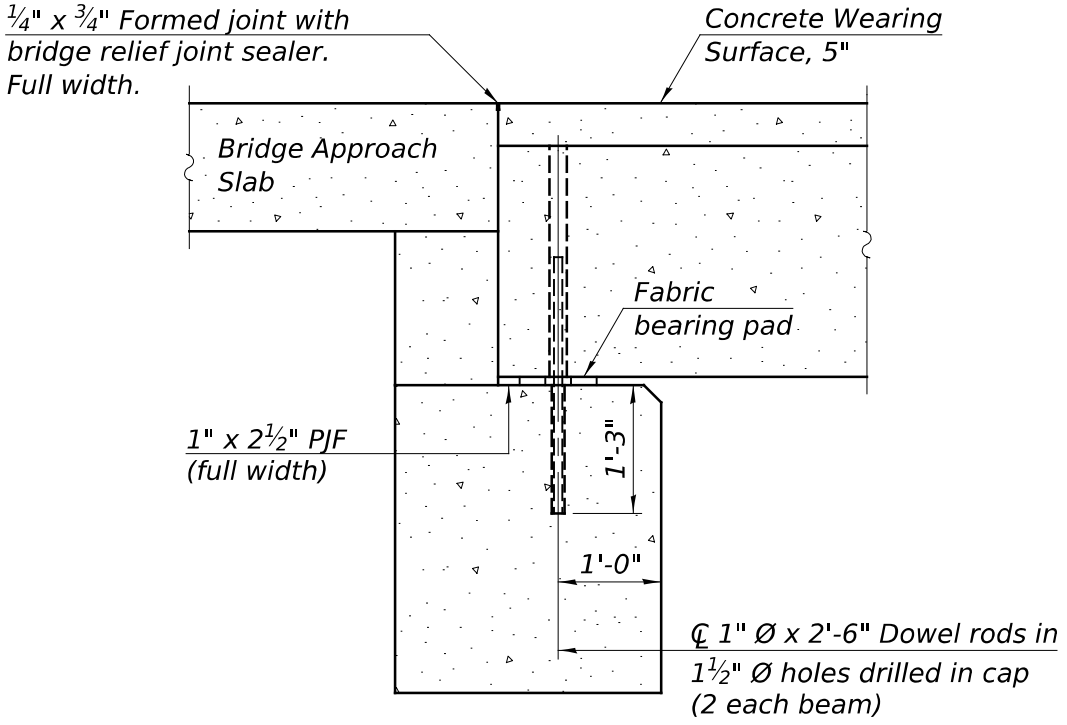
(With HMA wearing surface. Dimensions are at Rt. ∠'s)

Notes:
For abutment details and reinforcement, see Section 3.8.

FIXED ABUTMENT SECTIONS 17" BEAMS

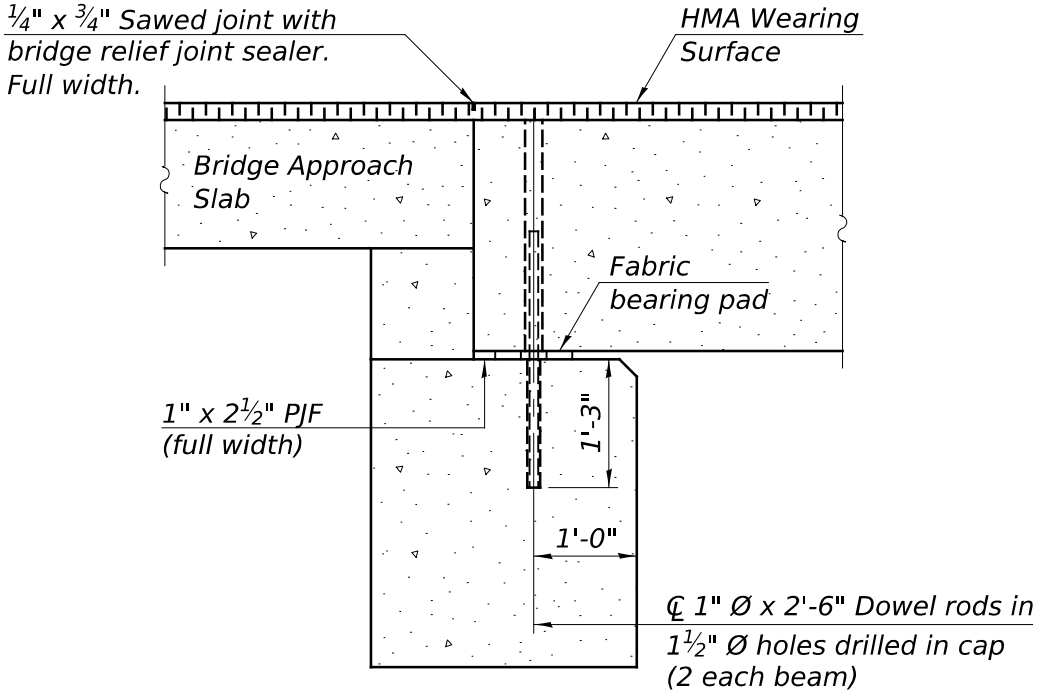
Figure 3.5.6.1-2

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SECTION THRU ABUTMENT

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



Notes:
For abutment details and reinforcement, see Section 3.8.

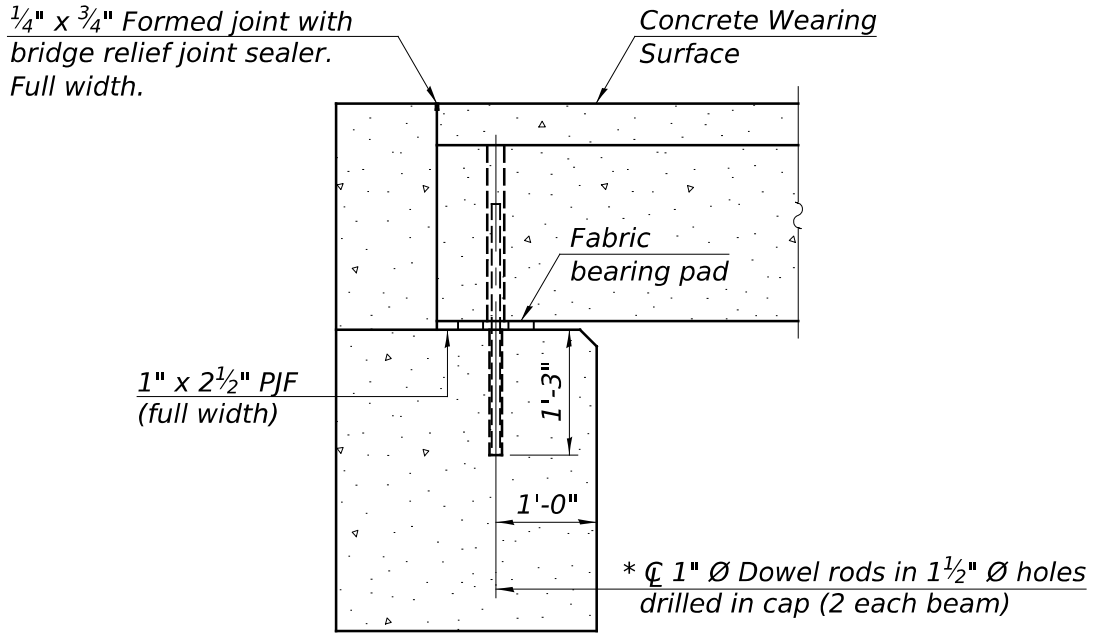
SECTION THRU ABUTMENT

(With HMA wearing surface. Dimensions are at Rt. ∠'s)

FIXED ABUTMENT SECTIONS 21"-42" BEAMS

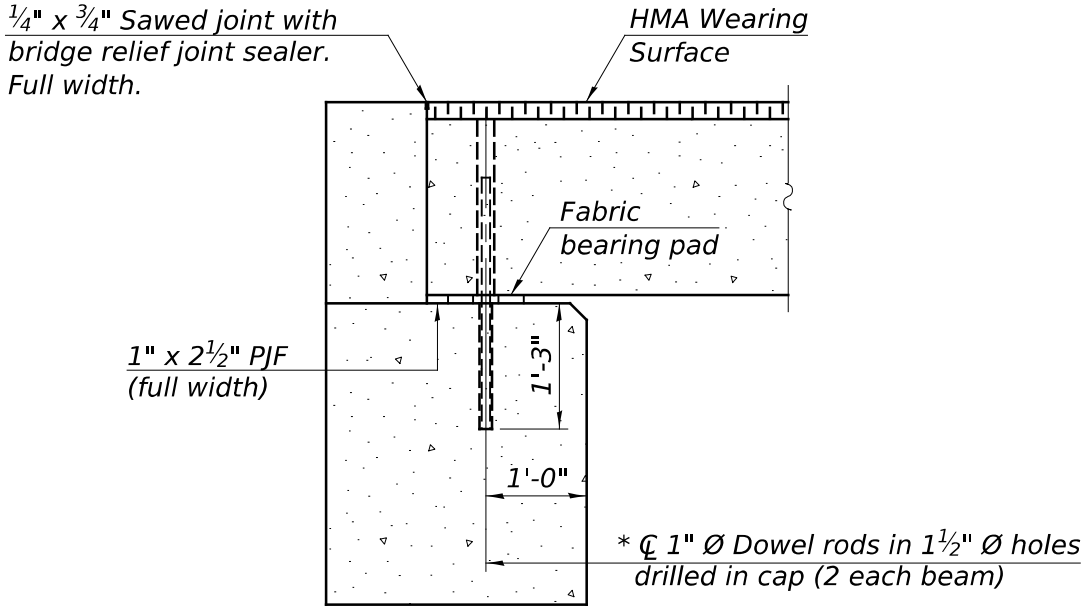
Figure 3.5.6.1-3

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SECTION THRU ABUTMENT

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



SECTION THRU ABUTMENT

(With HMA wearing surface. Dimensions are at Rt. ∠'s)

Notes:
For abutment details and reinforcement, see Section 3.8.
** 2'-0" long for 11" beams*
2'-6" long for 17" thru 42" beams

FIXED ABUTMENT SECTIONS
11"-42" BEAMS
NO APPROACH PAVEMENT

Figure 3.5.6.1-4

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3.5.6.2 Pier Sections

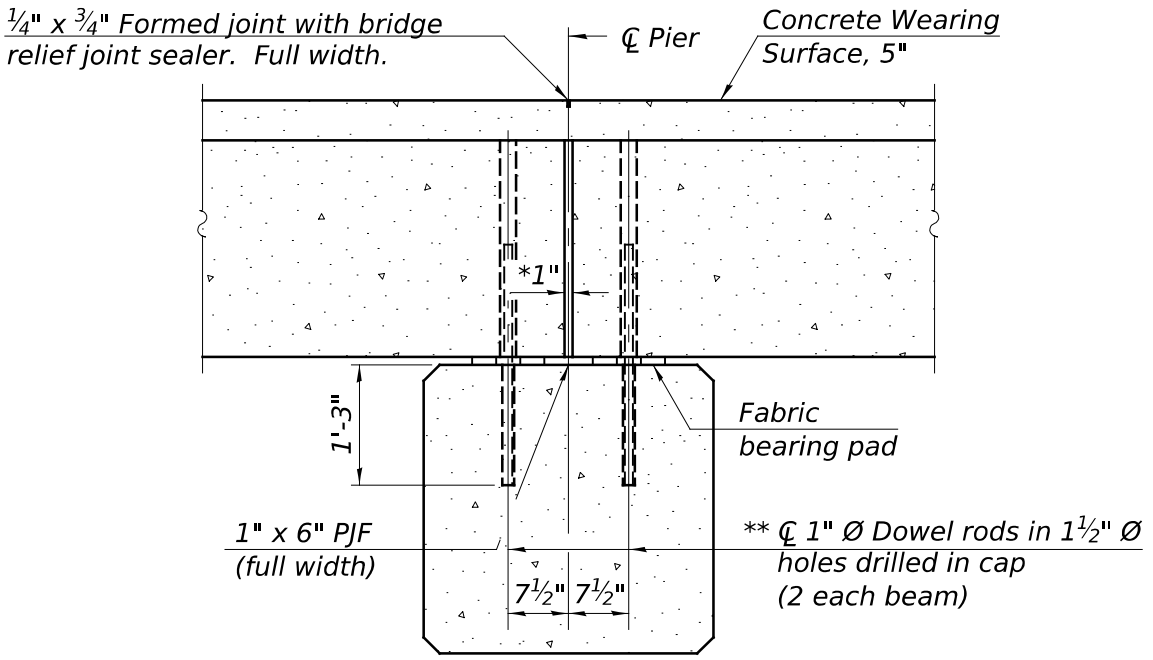
Figure 3.5.6.2-1 shows the typical cross section with either CWS or HMA. A joint is either formed for CWS or sawed for HMA in the wearing surface at the center line of the pier where a crack is anticipated. A 1 inch gap filled with non-shrink grout shall be provided between the deck beam ends. This gap may vary to account for tolerances in beam lengths.

3.5.6.3 Fixed Bearings

The typical fixed bearings for deck beams are detailed on the base sheets. A 1 inch fabric bearing pad is configured to straddle across two beam sections from dowel rod to dowel rod. Straddling the bearing pad with proper shimming helps to prevent differential beam movement and maintain the integrity of the shear key. If the profile grade of the structure is too steep such that the 1 inch bearing pad is not sufficient to prevent the deck beam from bearing directly on the abutment or pier cap, then the caps shall be sloped as necessary to prevent beam to substructure contact.

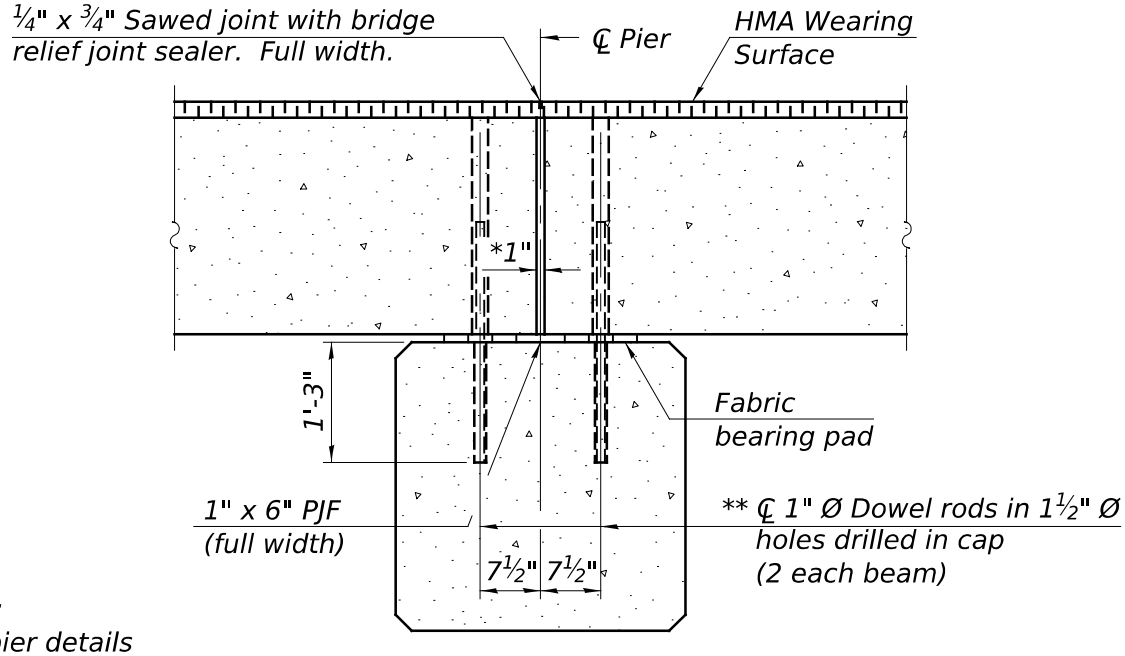
Skews larger than 35 degrees are not permitted on State projects because they create fabrication and constructability problems and make it more difficult to prevent differential beam movement. However, on Local projects the skew limit is sometimes permitted to be exceeded. In these cases, an alternate fixed bearing detail shall be used to help prevent differential beam movement. See Figures 3.5.6.3-1 through 3.5.6.3-3 for details.

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SECTION THRU PIER

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



SECTION THRU PIER

(With HMA wearing surface. Dimensions are at Rt. ∠'s)

Notes:
For pier details and reinforcement, see Section 3.9.

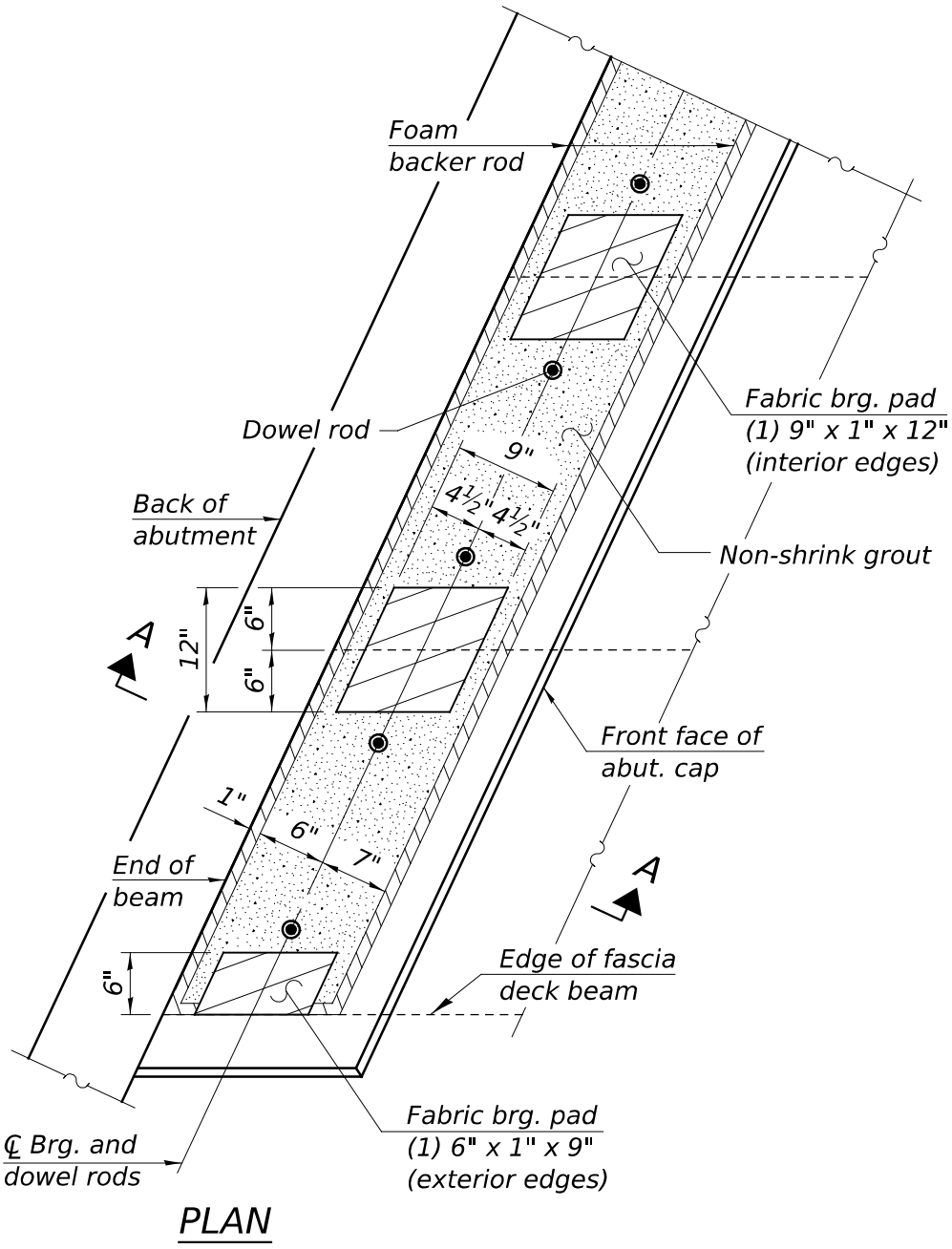
* 1" Joint shall be filled with non-shrink grout. 1" dimension may vary to accommodate tolerance in beam length.

** 2'-0" long for 11" beams
2'-6" long 17" thru 42" beams

FIXED PIER SECTIONS 11"-42" BEAMS

Figure 3.5.6.2-1

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PLAN

Notes:

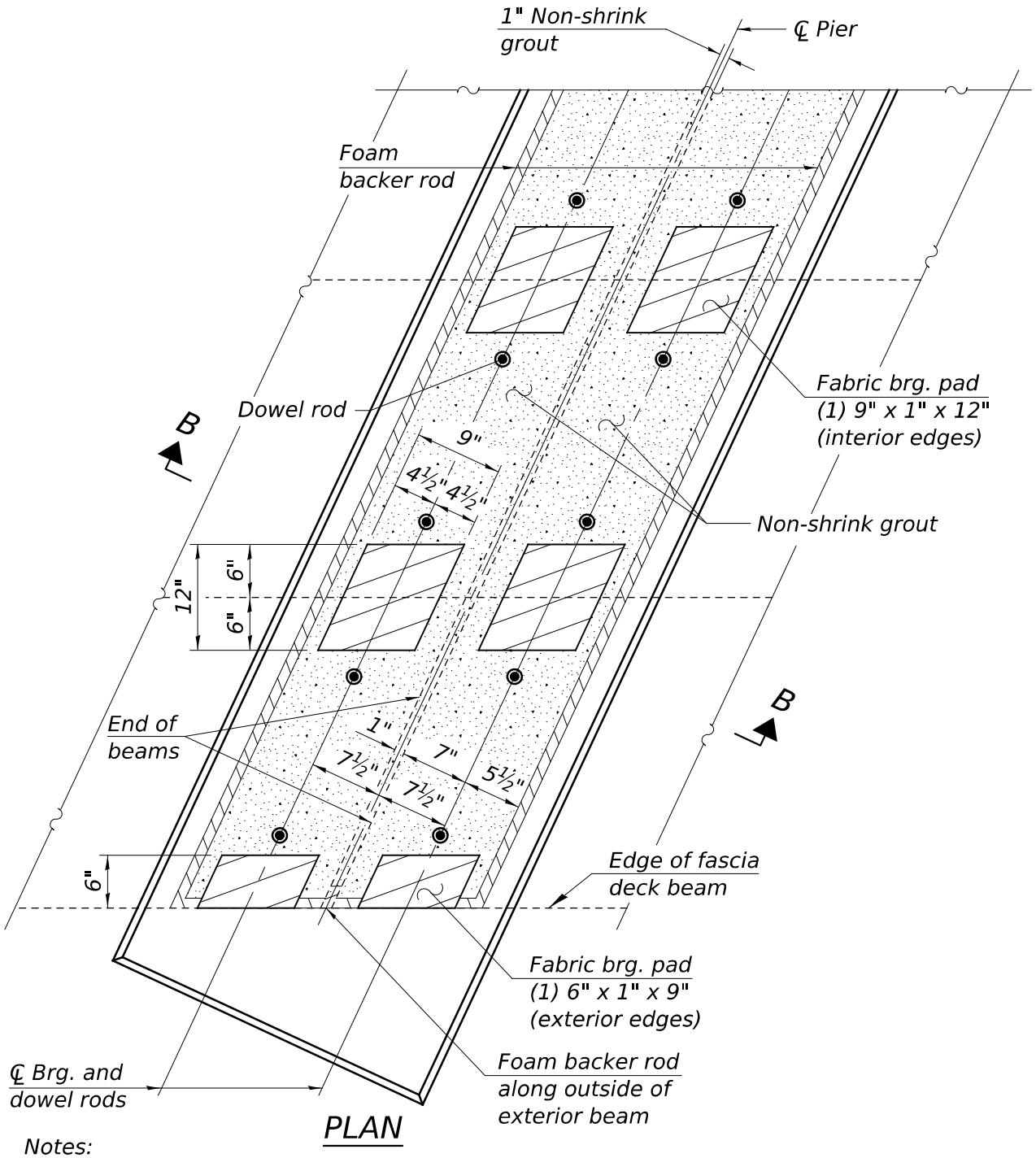
The alternate bearing detail shown may be used on a case by case basis to be approved by the Bureau of Bridges and Structures.

The bearing seat surfaces shall be adjusted by shimming the bearing to assure firm and even bearing prior to placement of grout. 2-⅛" fabric adjusting shims of the dimensions of the exterior bearing pad shown shall be provided for each bearing.

ALTERNATE FIXED BEARINGS AT ABUTMENT

Figure 3.5.6.3-1

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Notes:

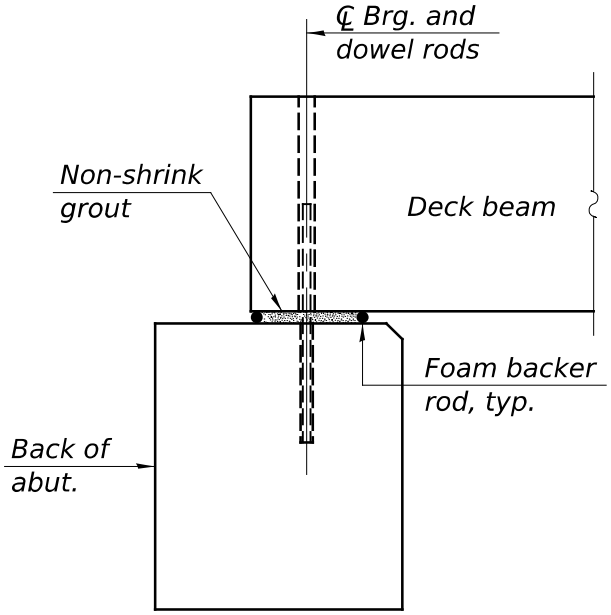
The alternate bearing detail shown may be used on a case by case basis to be approved by the Bureau of Bridges and Structures.

The bearing seat surfaces shall be adjusted by shimming the bearing to assure firm and even bearing prior to placement of grout. 2-⅛" fabric adjusting shims of the dimensions of the exterior bearing pad shown shall be provided for each bearing.

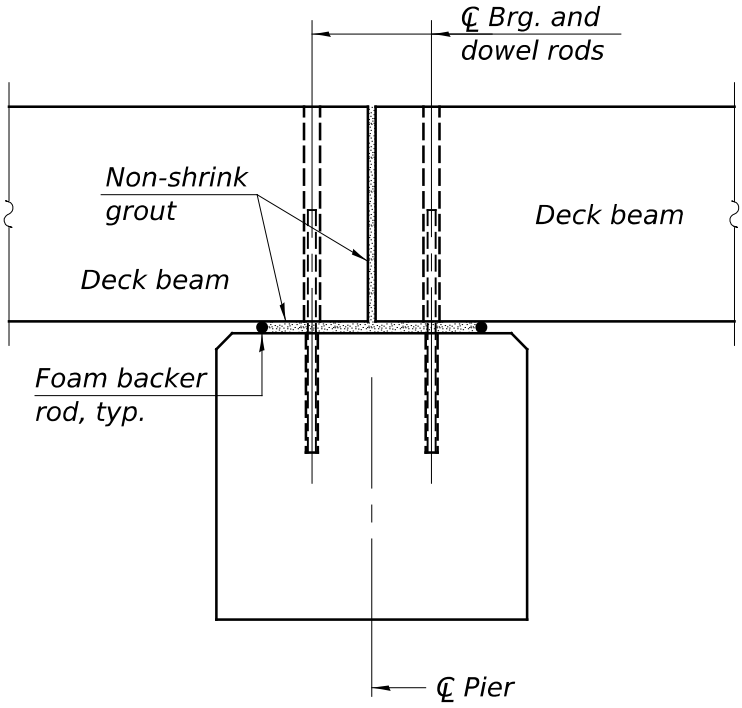
ALTERNATE FIXED BEARINGS AT PIER

Figure 3.5.6.3-2

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SECTION A-A



SECTION B-B

ALTERNATE FIXED
BEARING SECTIONS

Figure 3.5.6.3-3

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3.5.7 Expansion Substructures

Expansion substructures are necessary when the total deck beam structure length exceeds 300 feet. Versions of the typical abutment and pier cross sections, suitable for inclusion on the base sheets, are available in the "Details-Design" IDOT CADD Bridge Cell Library. Note that the standard base sheets are set up for fixed substructures since expansion deck beam structures are rare. Therefore, additional modifications to the standard base sheets are necessary when expansion substructures are required.

3.5.7.1 Abutment Sections

Figures 3.5.7.1-1 through 3.5.7.1-3 show the typical cross sections for the various deck beam depths having either a Concrete Wearing Surface (CWS) or a Hot Mix Asphalt (HMA) wearing surface. The abutment is notched on the shallow 11 inch deck beams and a backwall is required on the deeper sections. Figure 3.5.7.1-4 shows the typical cross sections with no approach pavements which may be used on Local projects.

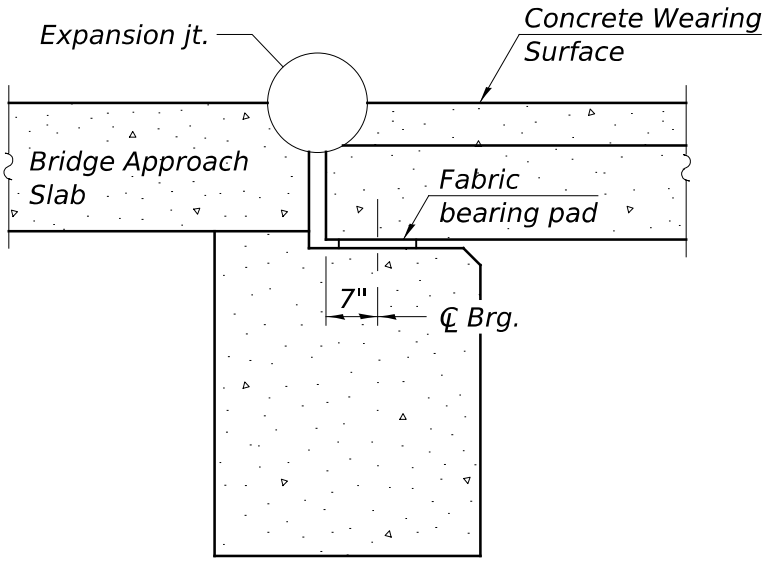
An expansion joint on an 11 inch deck beam superstructure with HMA wearing surface is not permitted because the 11 inch beam is too shallow to notch.

The approach slab is poured flush with the riding surface when an expansion joint is necessary on a deck beam structure with an HMA wearing surface. This is primarily for ease of constructability.

3.5.7.2 Pier Sections

Figure 3.5.7.2-1 shows the typical cross section with either CWS or HMA. Similar to the abutment cross sections, an expansion joint on an 11 inch deck beam superstructure with HMA wearing surface is not permitted.

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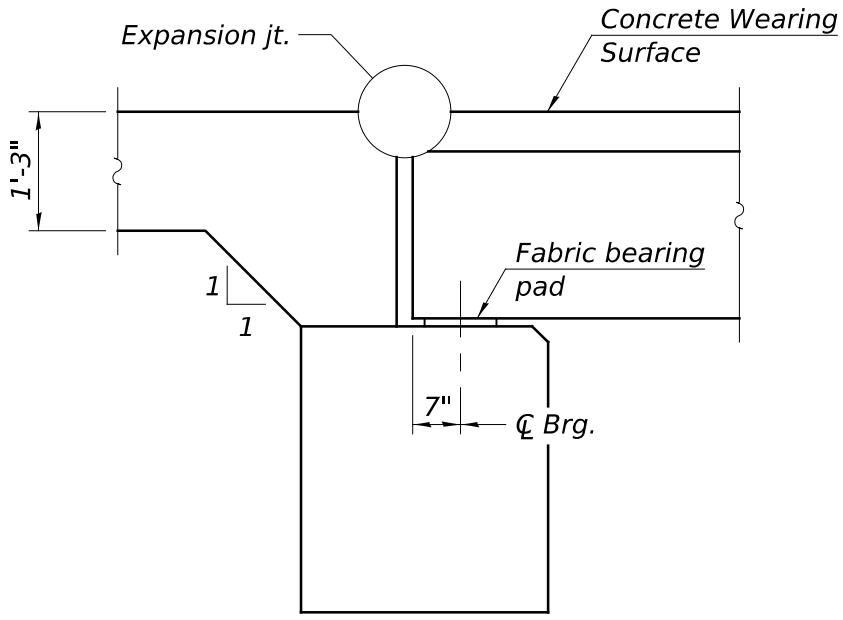
SECTION THRU ABUTMENT

(Dimensions are at Rt. ∠'s)

EXPANSION ABUTMENT SECTIONS 11" BEAMS

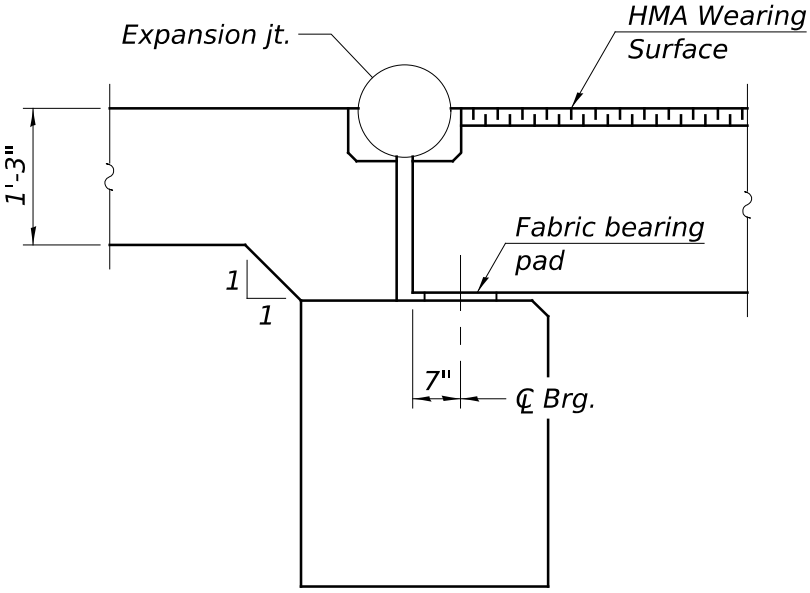
Figure 3.5.7.1-1

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SECTION THRU ABUTMENT

(With concrete wearing surface)
(Dimensions are at Rt. ∠'s)



SECTION THRU ABUTMENT

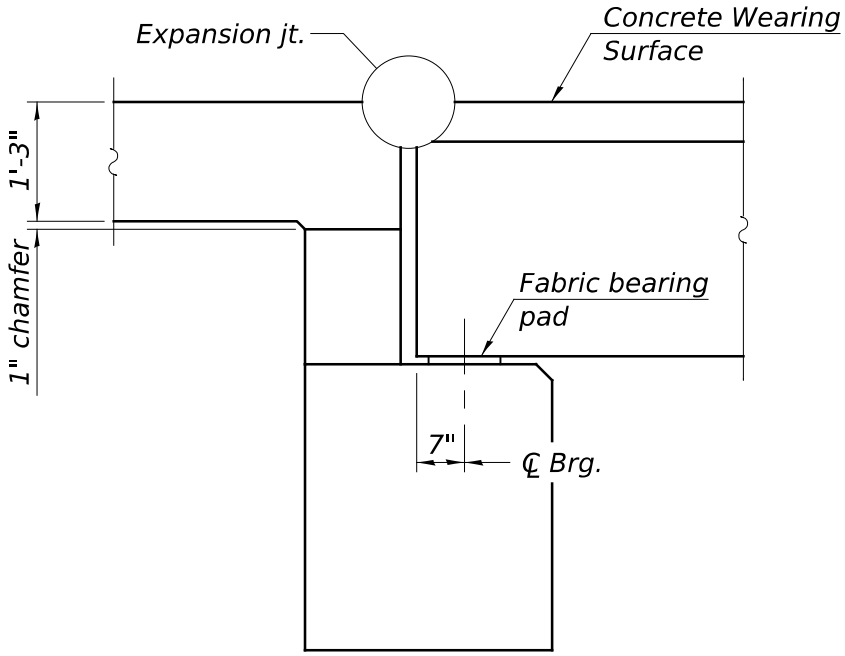
(With HMA wearing surface)
(Dimensions are at Rt. ∠'s)

Notes:
For abutment details and reinforcement, see Section 3.8.

EXPANSION ABUTMENT SECTIONS 17"-21" BEAMS

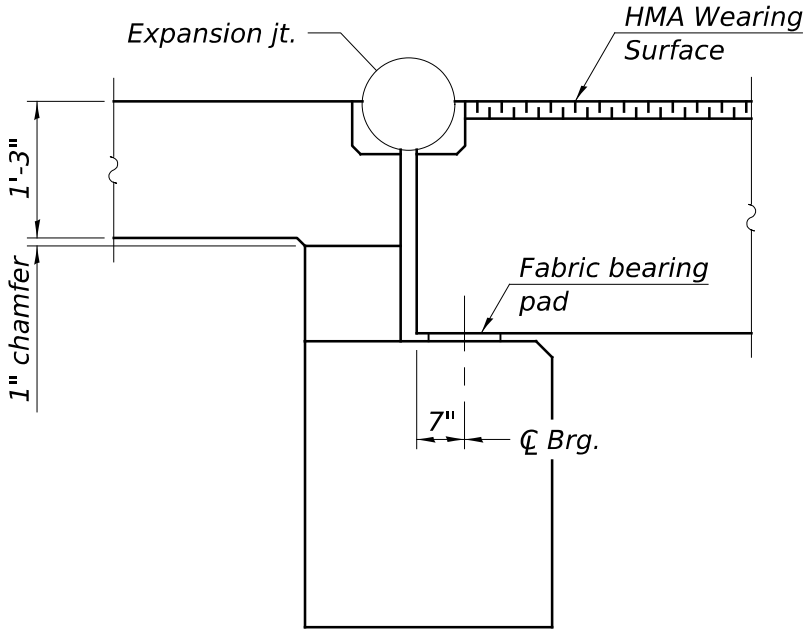
Figure 3.5.7.1-2

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SECTION THRU ABUTMENT

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



SECTION THRU ABUTMENT

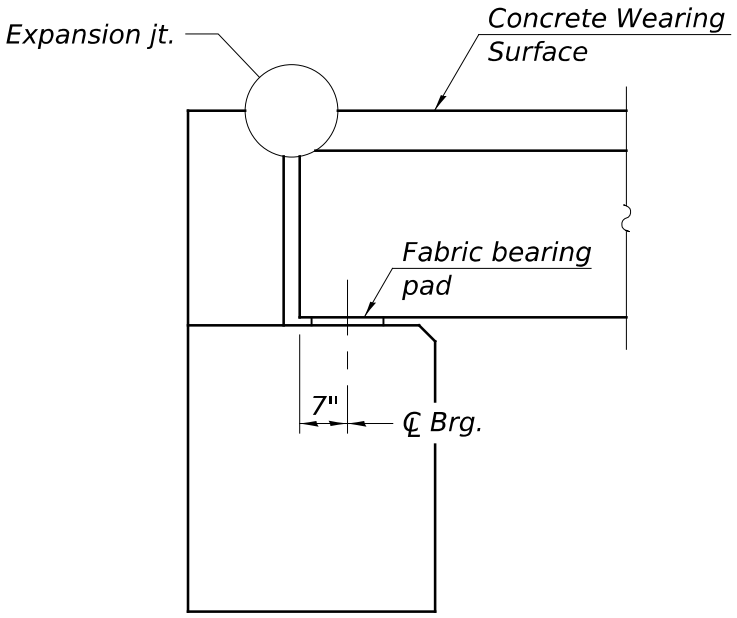
(With HMA wearing surface. Dimensions are at Rt. ∠'s)

Notes:
For abutment details and reinforcement, see Section 3.8.

EXPANSION ABUTMENT SECTIONS 27"-42" BEAMS

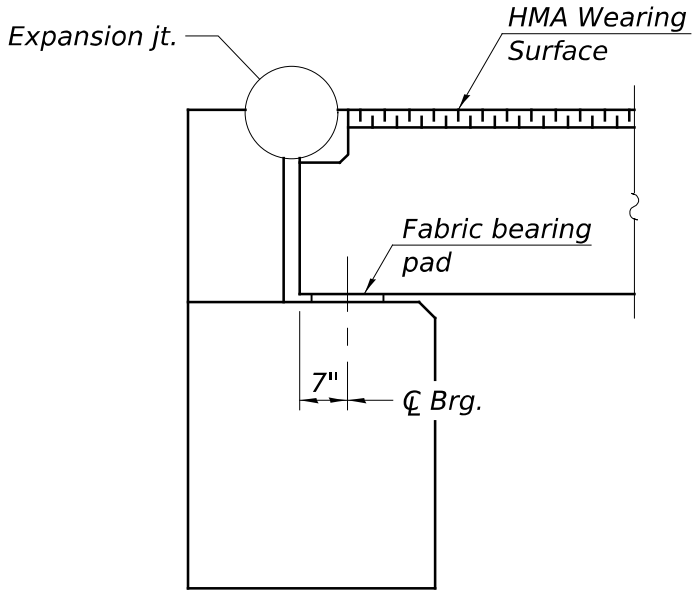
Figure 3.5.7.1-3

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SECTION THRU ABUTMENT

(With concrete wearing surface)
(Dimensions are at Rt. ∠'s)



∗ SECTION THRU ABUTMENT

(With HMA wearing surface)
(Dimensions are at Rt. ∠'s)

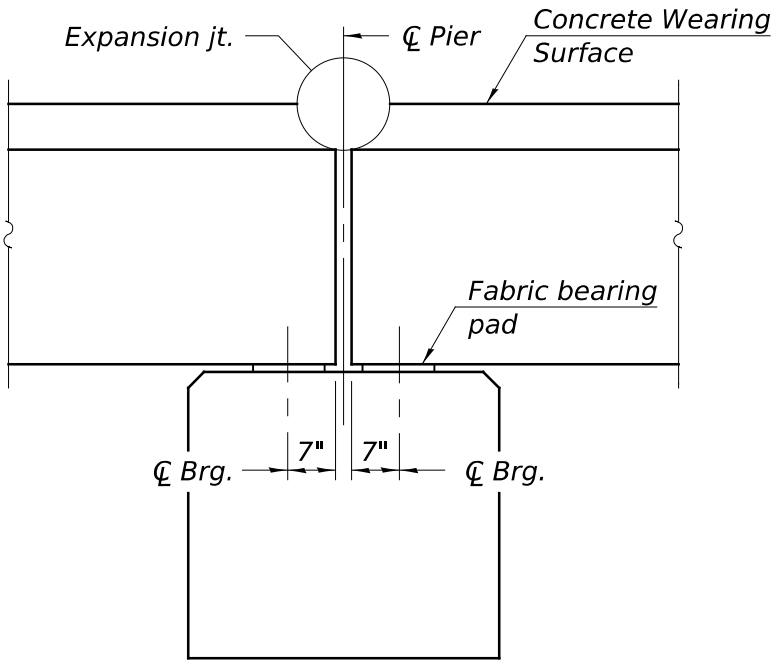
* *Section with HMA wearing surface not applicable for 11" beams since the notch cast into the end is not allowed for these beams.*

Notes:
For abutment details and reinforcement, see Section 3.8.

EXPANSION ABUTMENT SECTIONS
11"-42" BEAMS
NO APPROACH PAVEMENT

Figure 3.5.7.1-4

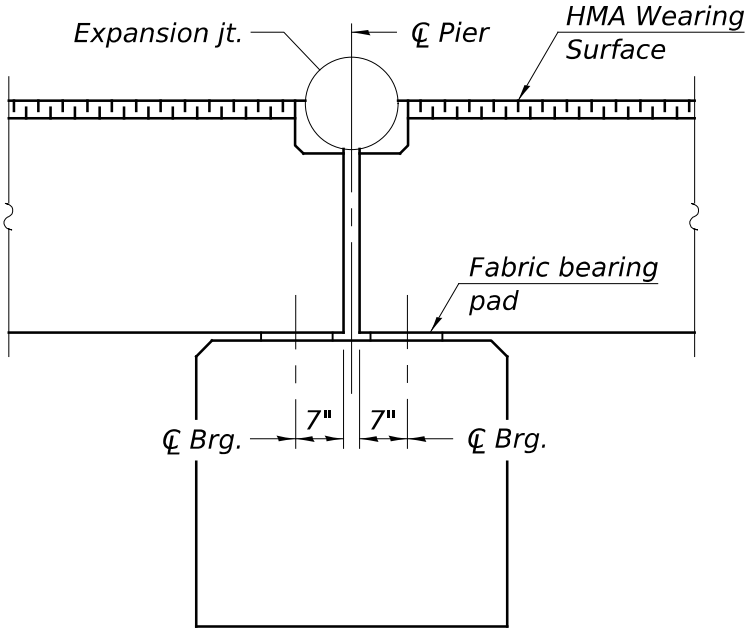
HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree



Notes:
For pier details and reinforcement, see Section 3.9.

SECTION THRU PIER

(With concrete wearing surface. Dimensions are at Rt. ∠'s)



* SECTION THRU PIER

(With HMA wearing surface. Dimensions are at Rt. ∠'s)

* Section with HMA wearing surface not applicable for 11" beams since the notch cast into the end is not allowed for these beams.

EXPANSION PIER SECTIONS 11"-42" BEAMS

Figure 3.5.7.2-1

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3.5.7.3 Expansion Bearings

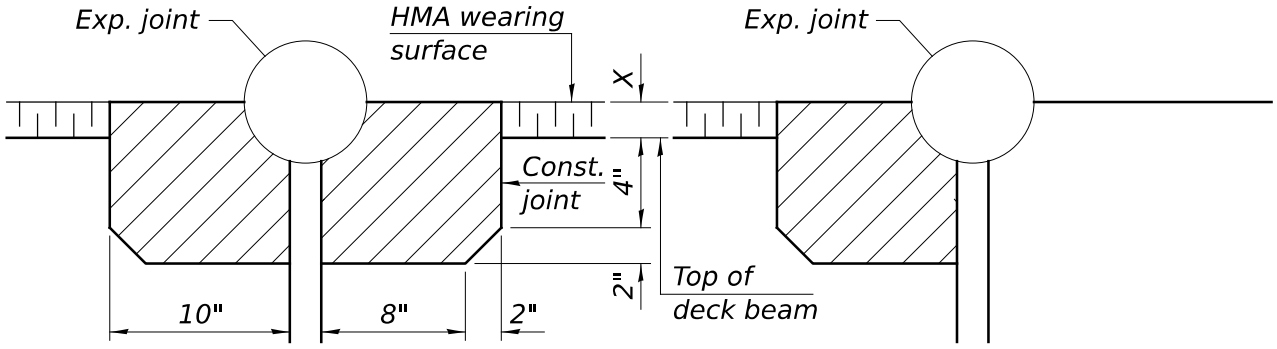
Expansion bearing details are similar to the fixed bearing details with two exceptions. The 1 inch fabric bearing pad shall have the same dimensions of the fixed bearing pad except the holes in the fabric bearing pads are not necessary since there are no dowel rods. The pad shall also be bonded to the substructure. These two exceptions are noted on the base sheets.

3.5.7.4 Expansion Joint Treatments

Figures 3.5.7.4-1 through 3.5.7.4-3 show typical expansion joint treatments for the various standard deck beam depths to be used with the strip seal expansion joint detailed in Figure 3.6.2.1-4. Deck beams with CWS do not need to be notched because the expansion joint is shallow enough to fit within the CWS. Deck beams with HMA require a notch for proper connection of the joint. The beam notch is filled with a field cast reinforced concrete block which enables the joint to be cast flush with the riding surface. The additional bars and inserts necessary to be cast into the beam for the notch/block are not on the standard base sheets and will need to be added when applicable.

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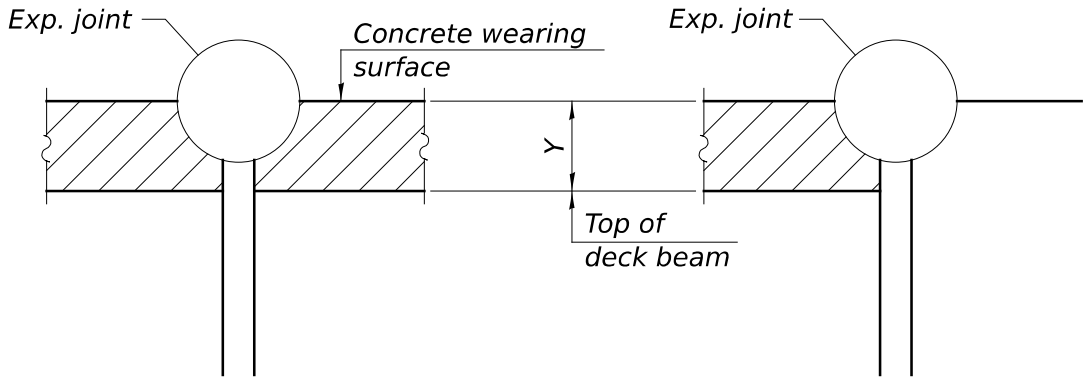
$X = 1\frac{1}{4}$" min. HMA wearing surface plus beam camber.
$Y = 5$" min. concrete wearing surface plus beam camber.



AT PIER AT ABUTMENT

EXPANSION JOINT TREATMENT

(17"-42" beams, with HMA wearing surface)



AT PIER AT ABUTMENT

EXPANSION JOINT TREATMENT

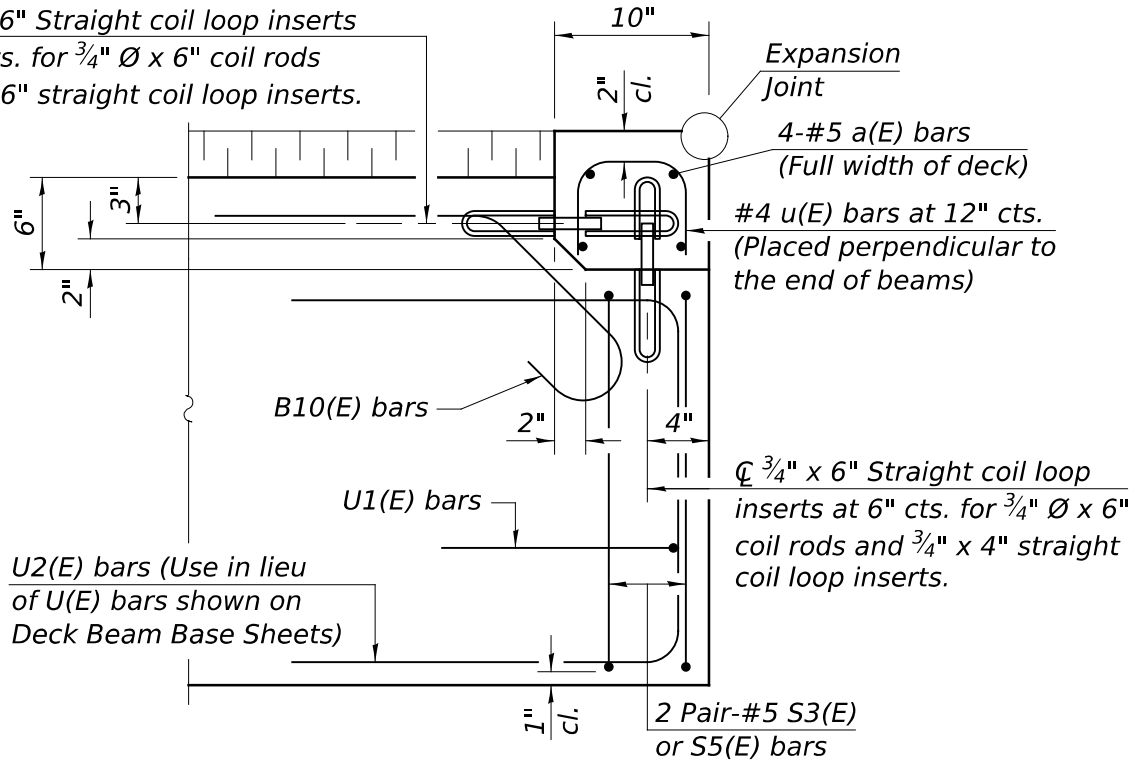
(11"-42" beams, with concrete wearing surface)

Notes:
For expansion joint details, see Section 3.6.
Dimensions are at right angles.
Hatched areas to be poured after beams have been erected and joints grouted. Ends of beams shall be aligned at the expansion joints. Any lineal variation in the beam lengths shall be placed at the fixed joint.

EXPANSION JOINT TREATMENT

Figure 3.5.7.4-1

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END OF BEAM (EXP. END)

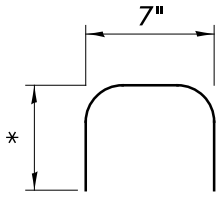
(Dimensions are at Rt. ∠'s)

Notes:

1½" cl. for reinforcement bars unless otherwise noted.

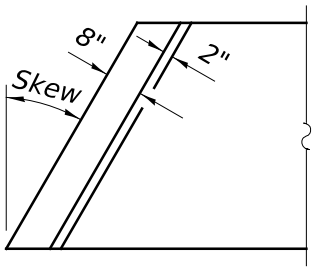
Typical reinforcement not shown for clarity. See Deck Beam Base Sheets for additional reinforcement details.

Cost of Inserts & Coil Rods included with Precast Prestressed Concrete Deck Beams.



BAR u(E)

** Determined by Designer based on thickness of HMA and the clearances shown.*

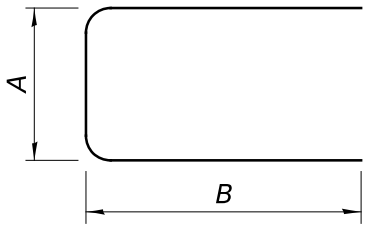


PLAN AT EXPANSION END OF BEAM

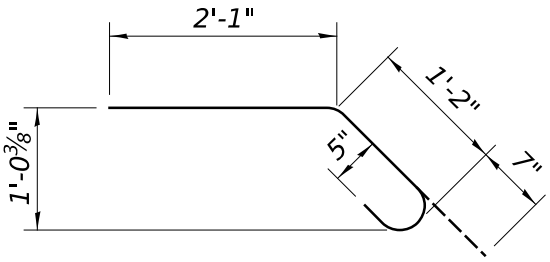
END OF DECK BEAM TREATMENT AT EXPANSION JOINT WITH HMA OVERLAY 17"-42" BEAMS

Figure 3.5.7.4-2

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BARS S3(E), S5(E) & U2(E)



BAR B10(E)

Beam Size	B10(E) Bars		**U1(E) Bars		U2(E) Bar				S3(E) or S5(E) Bars			
	No.	Size	No.	Size	No.	Size	A	B	No.	Size	A	B
17" x 36"	5	#5	0	#5	4	#5	7½"	2'-1"	4	#4	8½"	*
17" x 48"	6	#5	0	#5	6	#5	7½"	2'-1"	4	#4	8½"	*
21" x 36"	5	#5	1	#5	4	#5	11½"	2'-1"	4	#4	12½"	*
21" x 48"	6	#5	1	#5	6	#5	11½"	2'-1"	4	#4	12½"	*
27" x 36"	5	#5	1	#5	4	#5	1'-5½"	2'-1"	4	#4	1'-6½"	*
27" x 48"	6	#5	1	#5	6	#5	1'-5½"	2'-1"	4	#4	1'-6½"	*
33" x 36"	5	#5	2	#5	4	#6	1'-11½"	2'-1"	4	#4	2'-0½"	*
33" x 48"	6	#5	2	#5	6	#6	1'-11½"	2'-1"	4	#4	2'-0½"	*
42" x 36"	5	#5	3	#5	4	#6	2'-8½"	2'-1"	4	#4	2'-9½"	*
42" x 48"	6	#5	3	#5	6	#6	2'-8½"	2'-1"	4	#4	2'-9½"	*

* Determined by Designer based on skew and required lap length.

** See Deck Beam Base Sheets for U1(E) bar details.

END OF DECK BEAM TREATMENT AT EXPANSION JOINT WITH HMA OVERLAY 17"-42" BEAMS

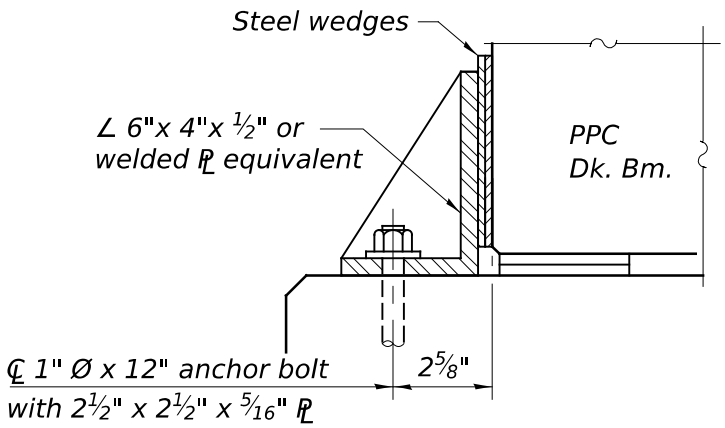
Figure 3.5.7.4-3

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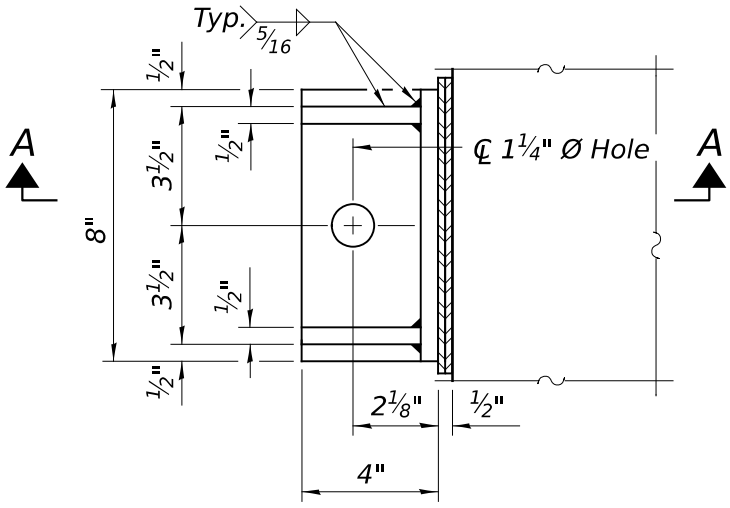
3.5.7.5 Retainer Angles

Retainer angles at expansion joints are required and shall remain in place. See Figure 3.5.7.5-1 for details.

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SECTION A-A



PLAN

Notes:

Cost of retainer and accessories are included with Precast Prestressed Concrete Deck Beams.

Equivalent rolled angle with stiffeners will be allowed in lieu of welded plates.

The side retainers shall be galvanized after shop fabrication according to AASHTO M 111 and ASTM 385.

Anchor bolts and plate washers shall be galvanized according to AASHTO M 232.

After the notch or concrete overlay are poured and cured, the steel wedges shall be removed.

RETAINER ANGLE
AT EXPANSION JOINT

Figure 3.5.7.5-1

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3.5.8 Railings

Section 2.3.6.1.7 states that the preferred bridge railing is the 39" Constant Slope parapet and the preferred steel railing is the Type IL-OH side mounted steel bridge rail (IDOT's R-40 label series). Both of these railings are crashworthy for a MASH Test Level 4 (TL-4) and are suitable for structures on the State system. The Type SM side mounted steel bridge railing (IDOT's R-34 label series), although only crashworthy for a MASH TL-2, continues to be a deck beam option for local jurisdictions as detailed in the "Railings" base sheet IDOT CADD Bridge Cell Library. Note that the 11 inch deck beam with CWS or HMA is too shallow for the Type IL-OH railing and therefore a 39" Constant Slope parapet is required for 11 inch beams on the State system.

The deck beam base sheets with the HMA wearing surface are denoted with "HMAWS" in the cell/model name. The exception again is with the 11 inch deck beam which cannot use the 39" Constant Slope parapet or the Type IL-OH railing and therefore it is detailed with a Type CO-10 railing with curb. This railing is crashworthy for a TL-4.

3.5.9 Miscellaneous Details

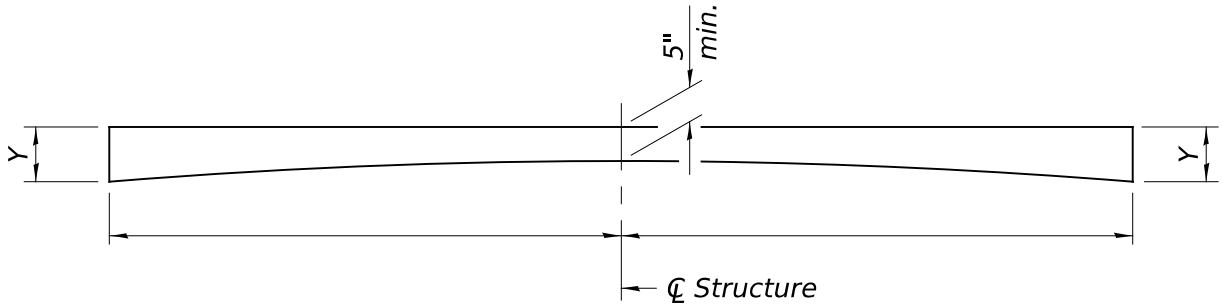
Figure 3.5.9-1 shows the general wearing surface profiles for CWS and HMA. The deck beam Design Guide 3.5 (found on the Bridges & Structures page) demonstrates how to calculate the camber so that the total anticipated wearing surface thickness on the beam ends may be determined and provided on the base sheets.

Figure 3.5.9-2 shows the standard shear key clamping detail which is used along stage construction lines to align and hold the two stages together so that the final stage-line shear key may be properly placed. Similar details of this Figure are available in the Detail Library and shall be placed in the contract plans of applicable projects.

Figure 3.5.9-3 shows how to determine the length of the deck beam end block for skews and for expansion joints.

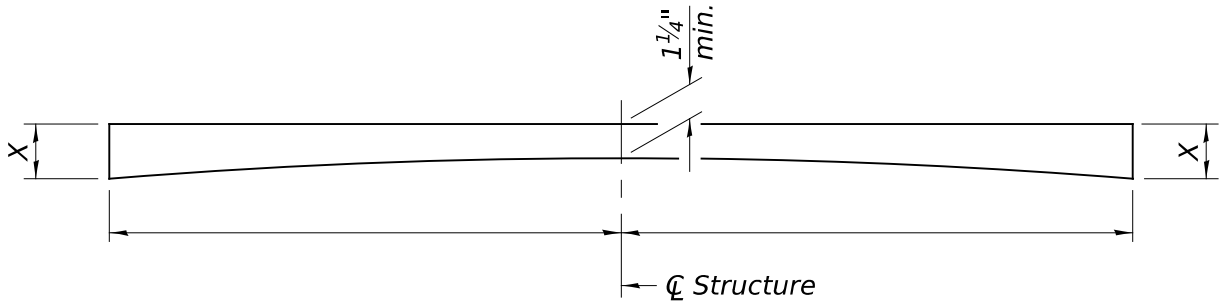
Figure 3.5.9-4 provides guidance on the proper location, number and embedment depth of lifting loops required for a specific deck beam. Design Guide 3.5 demonstrates how to calculate the beam weight and determine the proper number of lifting loops. The Department funded a research product which produced the report entitled "Development of Standard for Lifting Loops in Precast Deck Beams". This report verified that the IDOT lifting loop data and details satisfies the requirements of PCI.

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CONCRETE WEARING SURFACE (CWS) PROFILE

Y = 5" + Camber



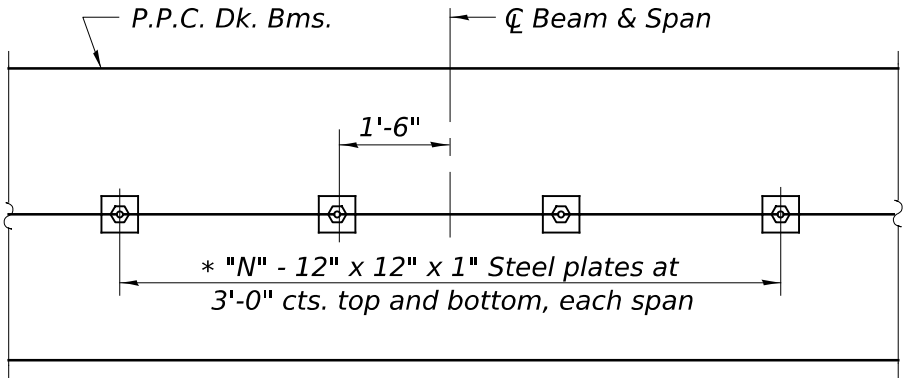
HOT-MIX ASPHALT WEARING SURFACE (HMA) PROFILE

X = 1¼" + Camber

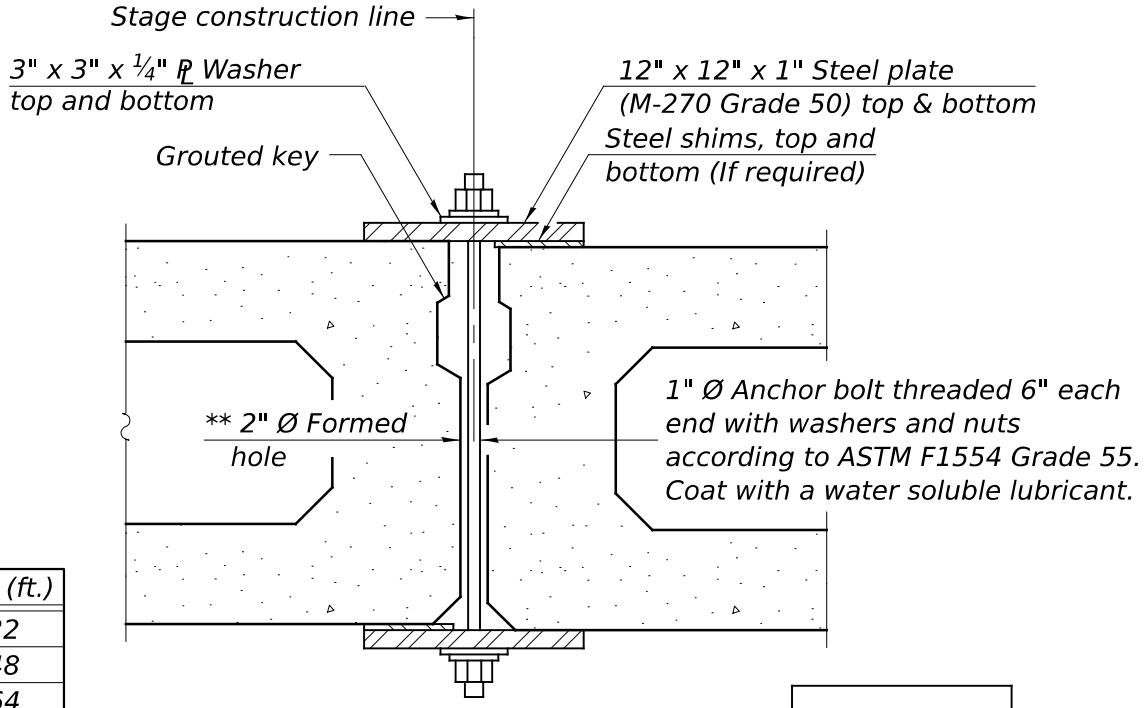
WEARING SURFACE PROFILES

Figure 3.5.9-1

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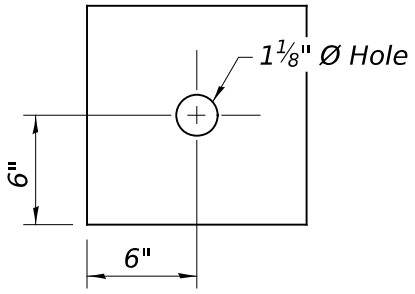


PLAN



SECTION

N	Span (ft.)
4	≤ 32
6	≤ 48
8	≤ 64
10	≤ 80
12	> 80



CLAMPING PLATE

* Space plates to miss temporary bridge rail posts. (Add where applicable)

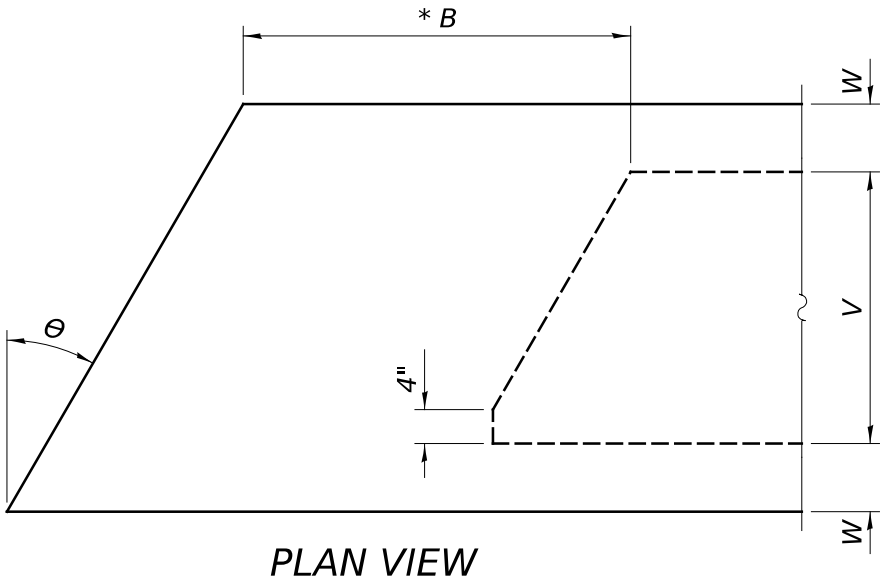
** Cast semi-circular recesses in the sides of each beam adjacent to the stage construction line. These recesses should align to form a hole at the appropriate locations for the clamping device bolts.

Notes:
See Stage Construction Detail for traffic lanes.

SHEAR KEY CLAMPING DETAIL
AT STAGE CONST. JOINT

Figure 3.5.9-2

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PLAN VIEW

* Increase B by $\frac{10"}{Cos\ \Theta}$ for beam ends that require an expansion joint notch.

DIMENSIONS TABLE

Beam	A (in.)	W (in.)	V (in.)
11x48 & 11x52	-	-	-
17x36	30	7	22
17x48	30	8	32
21x36	30	7	22
21x48	30	8	32
27x36	30	6	24
27x48	30	6	36
33x36	36	6	24
33x48	36	6	36
42x36	42	6	24
42x48	42	6	36

Note:
Dimension B, in inches, is the larger of the following equations:

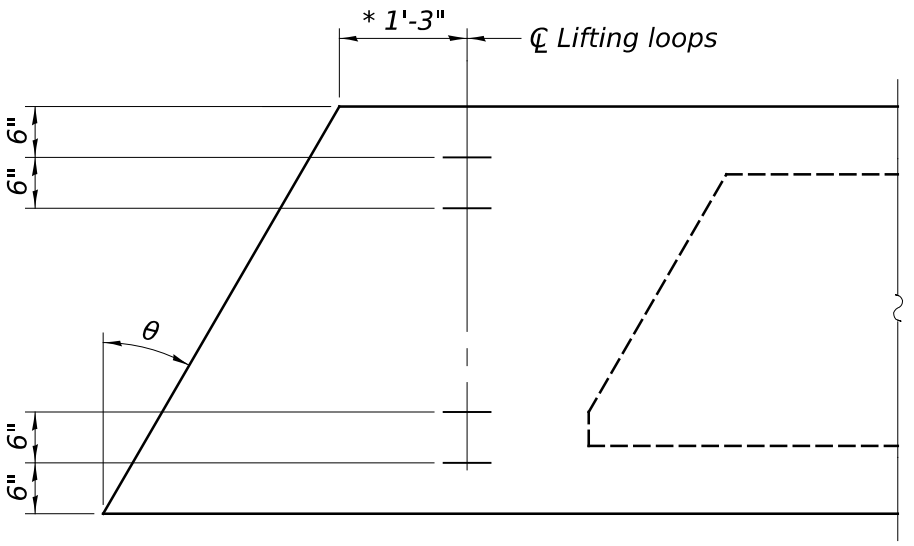
$$B = \frac{A}{Cos\ \Theta} - W(Tan\ \Theta)$$

$$B = 27 + V(Tan\ \Theta)$$

END BLOCK DETAILS

Figure 3.5.9-3

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PLAN VIEW
(Showing lifting loop locations)

TABLE A

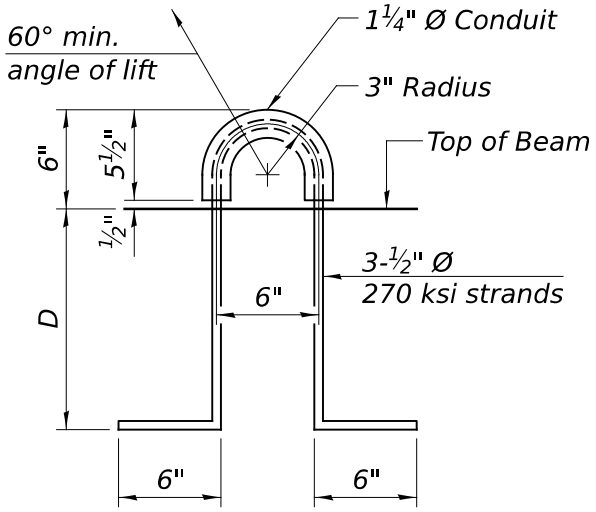
Beam	Net Weight (lbs/ft)	Solid Weight (lbs/ft)	D
11x48	-	535	7"
11x52	-	580	7"
17x36	492	621	1'-1"
17x48	642	834	1'-1"
21x36	550	771	1'-5"
21x48	708	1033	1'-5"
27x36	594	986	1'-11"
27x48	731	1323	1'-11"
33x36	669	1211	2'-5"
33x48	806	1623	2'-5"
42x36	781	1548	3'-2"
42x48	919	2073	3'-2"

TABLE B

No. of loops each end of beam	Gross wt. of beam (lbs)
** 2	60000
4	120000

* For beam ends that require a notch for the expansion joint increase this length by $\frac{10"}{Cos\Theta}$

** Use outermost lifting loop locations.



LIFTING LOOP DETAIL

Note:

Table A provides the lifting loop embedment depth and the weights of the net and solid sections of the beam used to calculate the gross weight of the beam.

Table B shows the number of lifting loops required at each end of the beam.

LIFTING LOOP FOR DECK BEAMS

Figure 3.5.9-4

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3.6 Expansion Devices

This section covers the Department's policies and details related to joints for bridge decks and overlays. The Department maintains details for three types of expansion joints: preformed joint strip seals, finger plate joints, and modular joints. Performed joint strip seals also may be used in fixed joint configurations. Devices that meet the material and pre-qualification requirements of the Department comprise the standard basis for design.

3.6.1 Expansion Joint Limitations

The type of expansion device to be used on a bridge deck depends upon the contributing expansion length for which the device is provided and the skew angle of the opening. The design temperature range shall be -20 ˚F to 120 ˚F with a normal installation temperature of 50 ˚F. For design purpose, the coefficient of linear expansion shall be 0.0000065/˚F for steel and concrete structures.

3.6.2 Expansion Joint Types

All expansion joints shall be sealed or include a trough system to control deck drainage at the bridge deck joint openings.

3.6.2.1 Strip Seal Joints

Strip seals for expansion joints are detailed in Figures 3.6.2.1-1 to 3.6.2.1-2. A point block detail with a sliding plate at parapets shown in Figures 3.6.2.1-5 and 3.6.2.1-6 shall be included with this joint type for structures with skews over 30 degrees. See Figure 2.3.6.1.6-1. Figure 3.6.2.1-1 shows strip seal details for sections thru raised sidewalks, medians, and deck level sidewalks. The sidewalk details feature an ADA compliant sidewalk sliding plate.

The strip seal bridge joint system shall be used for structures with contributing expansion lengths less than or equal to 305 ft. with skews equal to 0 degrees, reduced to 150 ft. at a skew of 60 degrees. These limits ensure that the deck surface gap opening at the minimum temperature will not exceed 4 in. in the direction of travel according to AASHTO LRFD Bridge Design Specifications Article 14.5.3.2. In addition, the deck surface gap opening shall not be less than 1 ½ inches at right angles to the edge rails at 50 °F to ensure the seal can be installed. This restriction should only affect smaller expansion lengths, larger skews, or a combination thereof.

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The distance “A” (See Figure 3.6.2.1-3) shall be calculated according to the following formula.

$$\text{“A”} = [\gamma\ \alpha\ L\ (\Delta t) / 2] \cos(S) + ½\text{”}$$

Where:

“A” = clear distance opening at right angles at 50 ºF (in)

γ = 1.2 (AASHTO LRFD Bridge Design Specifications Table 3.4.1-1 and Table 3.4.1-6; max load factor for TU due to Uniform Temperature)

α = 0.0000065/ºF (i.e. 6.5×10^{-6} /ºF)

L = contributing expansion length (in)

Δt = 140 ºF (-20 ºF to 120 ºF)

S = skew angle (degrees), measured from the baseline to a line perpendicular to the centerline of joint

The additional ½” is provided to ensure there is no hard contact between adjacent surfaces. See Base Sheet EJ-SS in the “Base Sheet-General” IDOT CADD Bridge Cell Library for details of the expansion strip seal joints.

Joint openings and geometry details for the Rolled Rail Strip Seal Joint shall be dimensioned on the superstructure sheets. When Strip Seal Joints are used at fixed bearing locations per Section 3.6.3, the deck surface gap opening dimension shall be 1 ½”. The following note shall be placed with the superstructure sheets to alert the contractor of potential dimensional revisions:

Dimensions are based on a Rolled Rail Strip Seal Joint. If the Contractor elects to use the Welded Rail Strip Seal Joint, deck dimensions may require adjustments to satisfy the details on Base Sheet EJ-SS.

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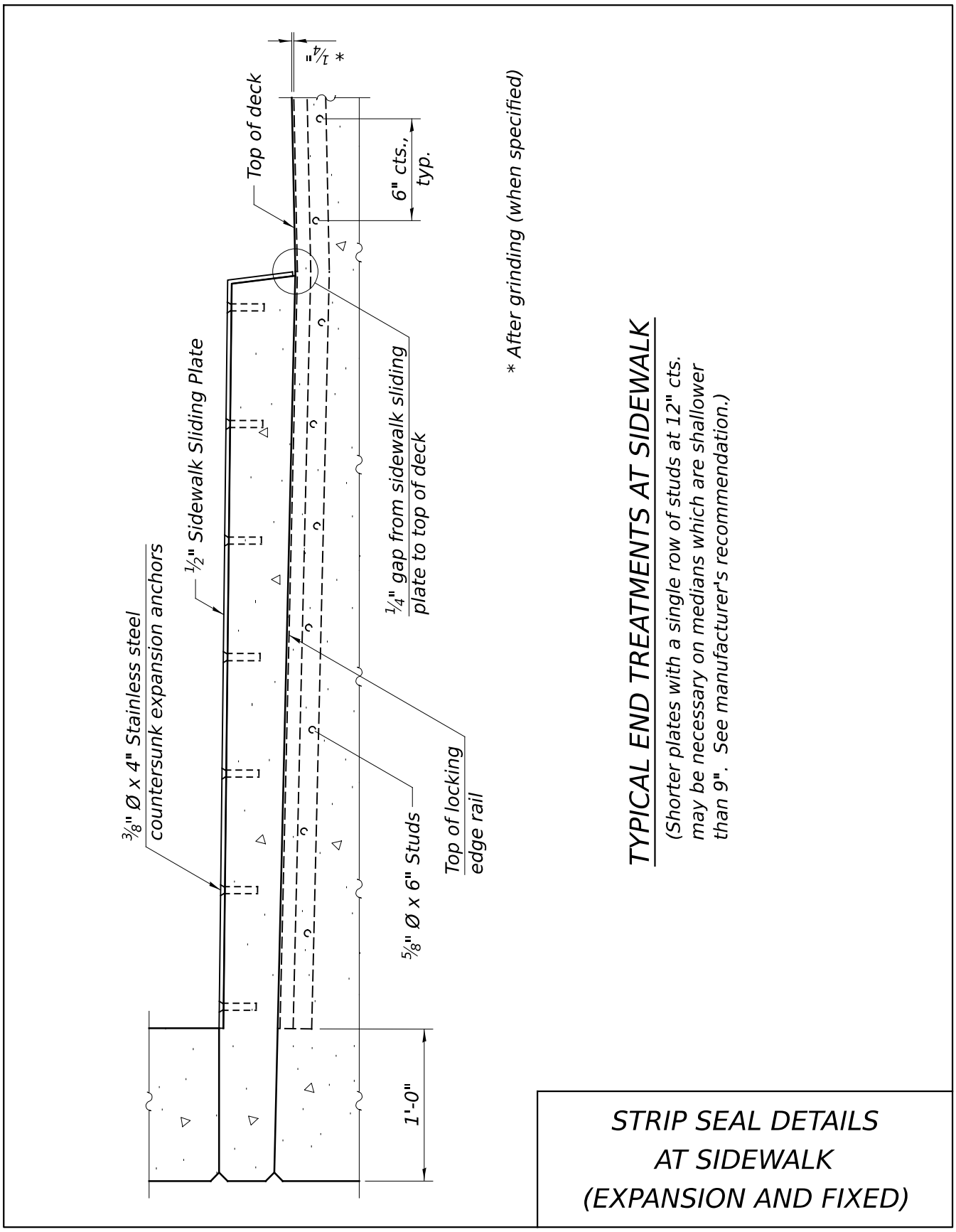
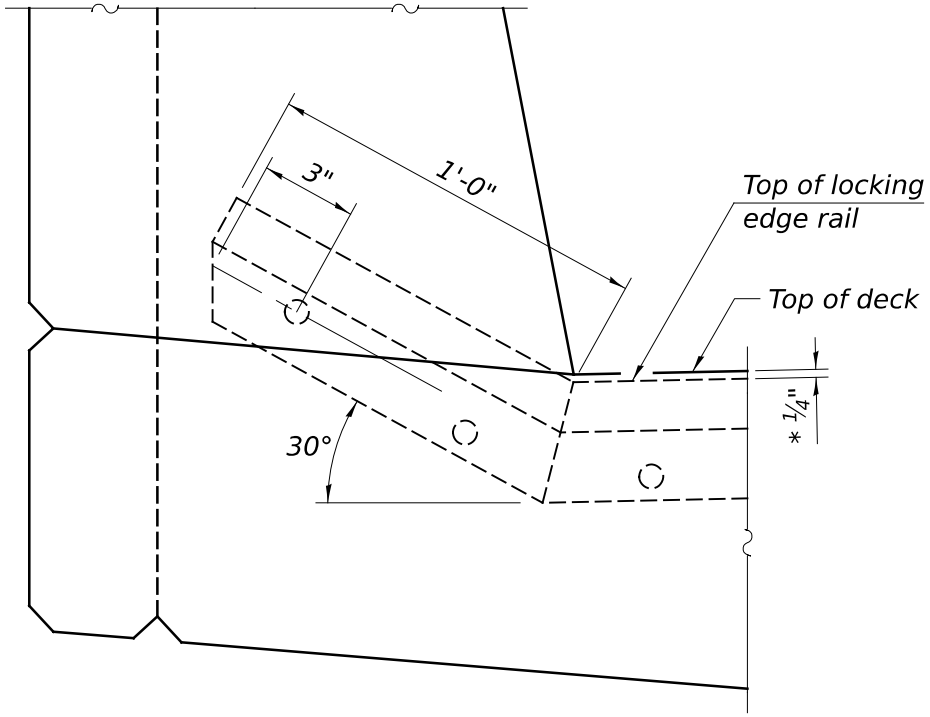


Figure 3.6.2.1-1

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* After grinding (when specified)

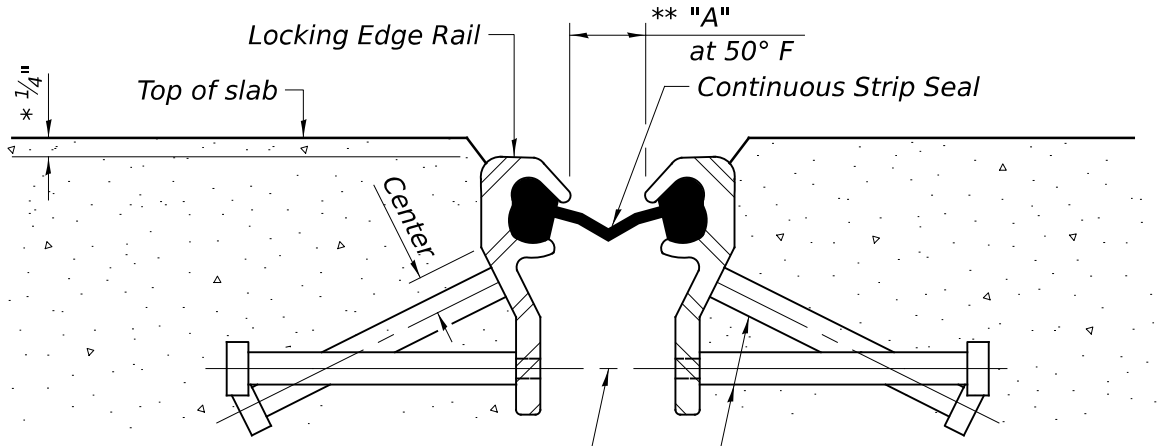
TYPICAL END TREATMENTS AT PARAPET

(See Section A-A of Fig. 3.6.2.1-5 for end treatment of skews >30°)

STRIP SEAL DETAILS
AT PARAPET
(EXPANSION AND FIXED)

Figure 3.6.2.1-2

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7/16" Ø holes at 4'-0" cts. for 3/8" Ø bolts. All bolts shall be burned, sawed, or chipped off flush with the plates after forms are removed, typ.

Place 3/8" Ø x 6" granular or solid flux filled headed studs conforming to Article 1006.32 of the Std. Specs., automatically end welded at 1'-0" alt. cts.

SECTION THRU EXPANSION STRIP SEAL JOINT

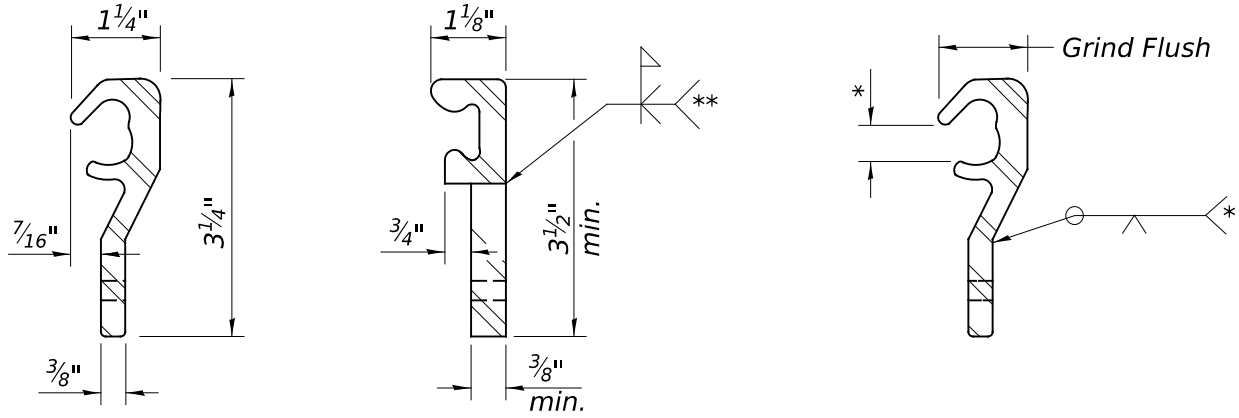
** After grinding (when specified)*

*** The minimum dimension shall be 1½" for installation purposes.*

STRIP SEAL EXPANSION JOINT

Figure 3.6.2.1-3

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ROLLED (EXTRUDED) RAIL

WELDED RAIL

LOCKING EDGE RAIL

LOCKING EDGE RAIL SPLICE

Rolled rail shown, welded rail similar.

Notes:

The strip seal shall be made continuous and shall have a minimum thickness of 1/4". The configuration of the strip seal shall match the configuration of the Locking Edge Rails.

The height and thickness of the Locking Edge Rails shown are minimum dimensions. The actual configuration of the Locking Edge Rails and matching strip seal may vary from manufacturer to manufacturer. Flanged edge rails will not be allowed.

The inside of the Locking Edge Rail groove shall be free of weld residue.

Locking Edge Rails may be spliced at slope discontinuities and stage construction joints.

The manufacturer's recommended installation methods shall be followed.

All steel components shall be galvanized after fabrication according to Article 520.03 of the Standard Specifications.

Maximum space between rail segments at stage lines shall be 3/16", sealed with a suitable sealant.

* *Omit weld at seal opening.*

** *Back gouge not required if complete joint penetration is verified by mock-up.*

STRIP SEAL EXPANSION JOINT DETAILS

Figure 3.6.2.1-4

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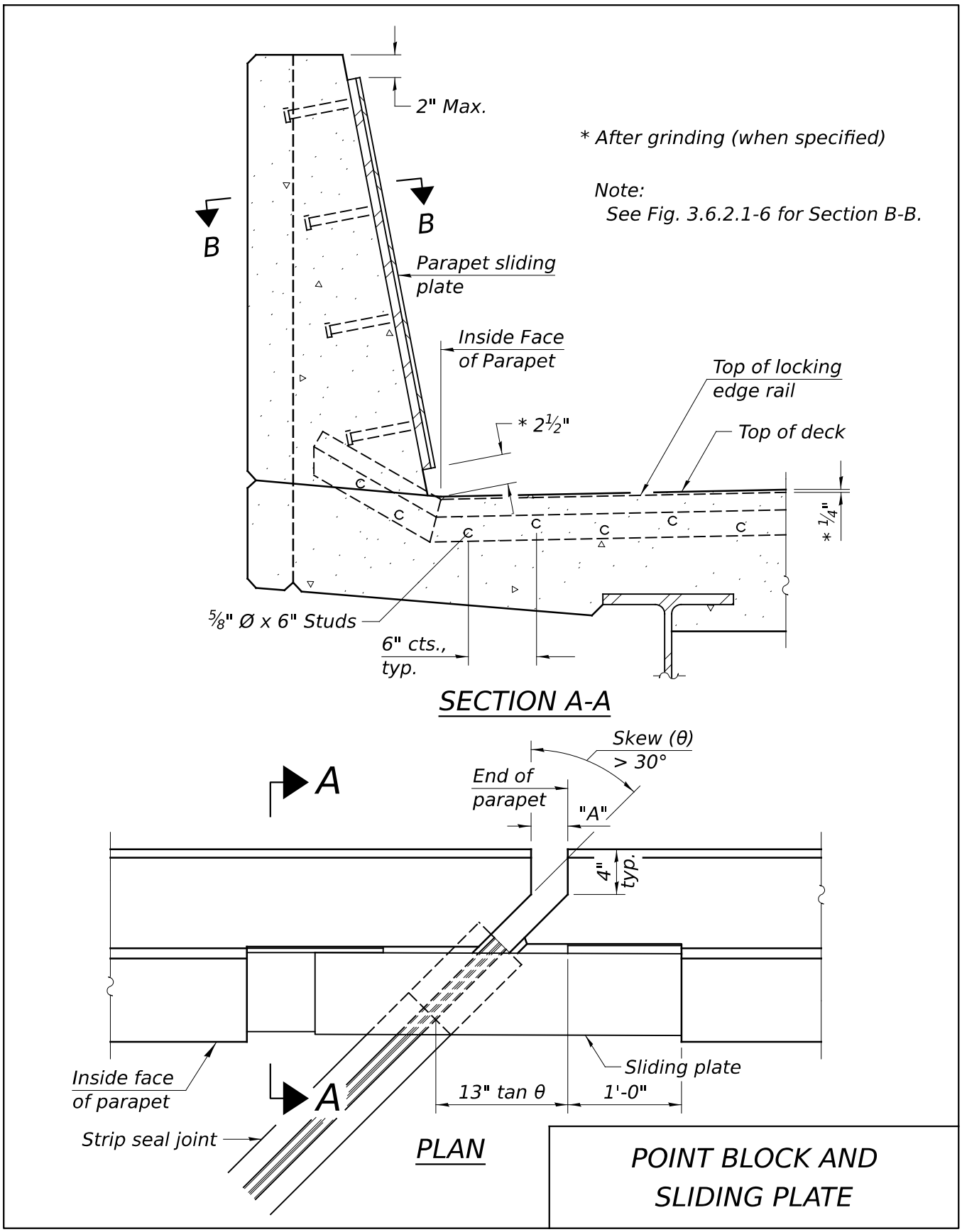


Figure 3.6.2.1-5

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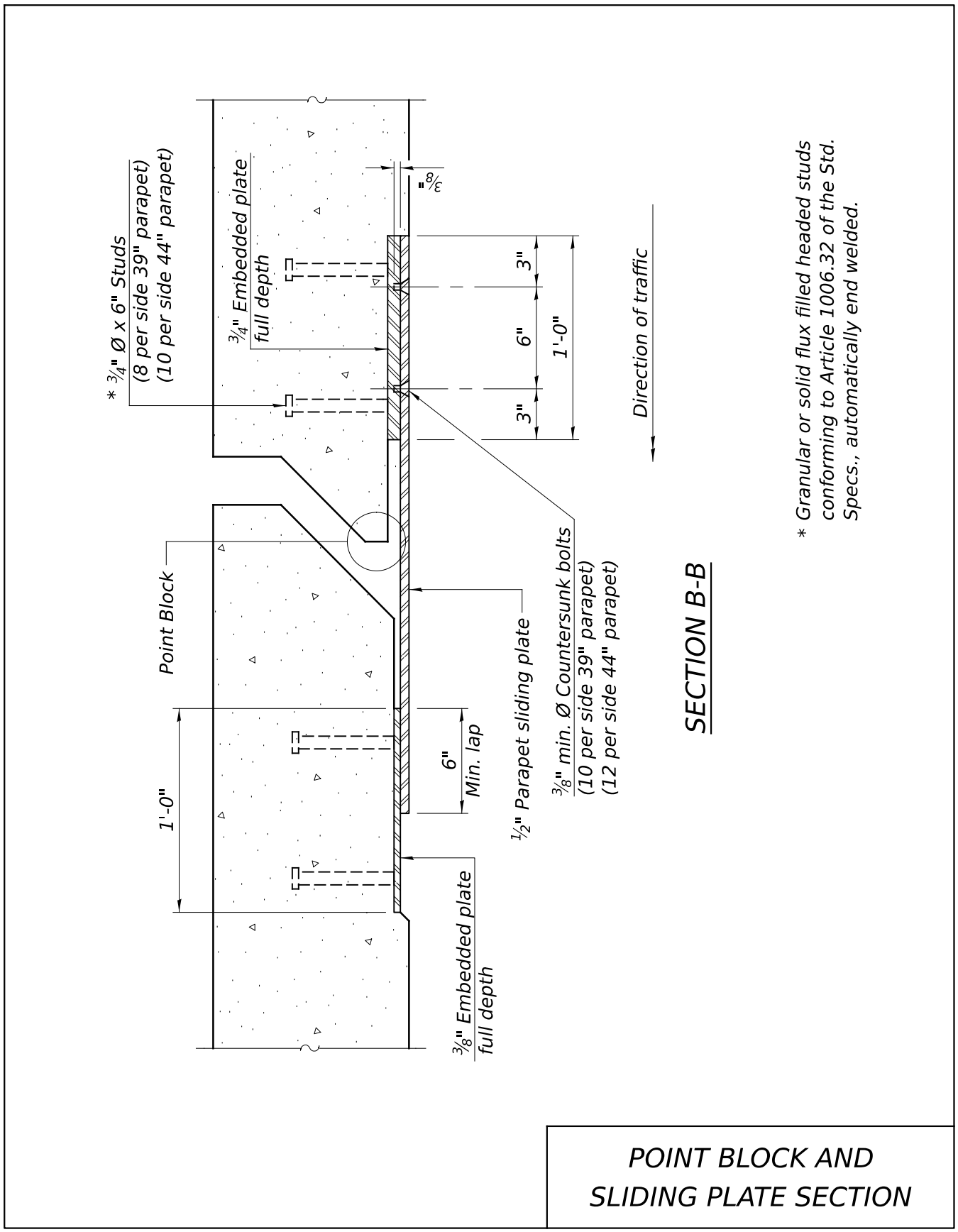


Figure 3.6.2.1-6

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3.6.2.2 Finger Plate Expansion Joint with Trough

Finger plate expansion joints may be used when the limits for strip seal expansion joints are exceeded. The design of finger plates is governed by two basic parameters:

1. Expansion length
2. Live load plus impact

A 16 kip wheel load with an impact factor of 1.75 shall be uniformly distributed and applied equally to the fingers for only one side of the joint within a 20 in. design width. The maximum distribution length shall be 10 in. For fingers over 10 in. in length, the distribution shall start at the free end of the finger.

Figure 3.6.2.2-1 assists the designer in determining some of the basic geometry of a finger plate expansion joint, including the finger length. Figure 3.6.2.2-2 provides the required finger thickness and weld length for various design finger lengths. The design parameters included in Figure 3.6.2.2-2 are:

1. Minimum finger length = 5 in.
2. Maximum finger length = 14 in.
3. Skew range of 0 to 60 degrees.
4. Finger radius = ⅞ in.
5. Finger to stool fillet weld = $\frac{5}{16}$ in.
6. ASTM A709 Grade 50 steel (CVN) or Grade 50W steel (CVN) shall be used throughout the finger plate joint.
7. Fatigue. Assume category A for the fingers, category C for the stool welds = AASHTO LRFD Bridge Design Specifications Section 6.6.1.2.5; assuming a 35 year design life, 5 cycles per truck.
8. $\frac{3}{8}$ in. ≤ stool web thickness ≤ ½ in.
9. Three ¾ in. ϕ studs shall be evenly spaced between stools and connected 2 in. from the back end of the finger plate.
10. Maximum stool spacing between stringers = 2 ft. – 0 in.

Figures 3.6.2.2-3 through 3.6.2.2-11 show additional details for finger plate expansion joints with troughs. Adequate elevations and dimensions shall be furnished on the plans to guide fabricators in detailing the stools, apron plates, and trough support plates.

When a finger plate expansion joint intersects a concrete median or a sidewalk, the finger plate joint shall continue through at the top of bridge deck elevation. The median or sidewalk shall be gapped as necessary for the joint. The gap shall be covered with sliding plates similar to the details shown in Figure

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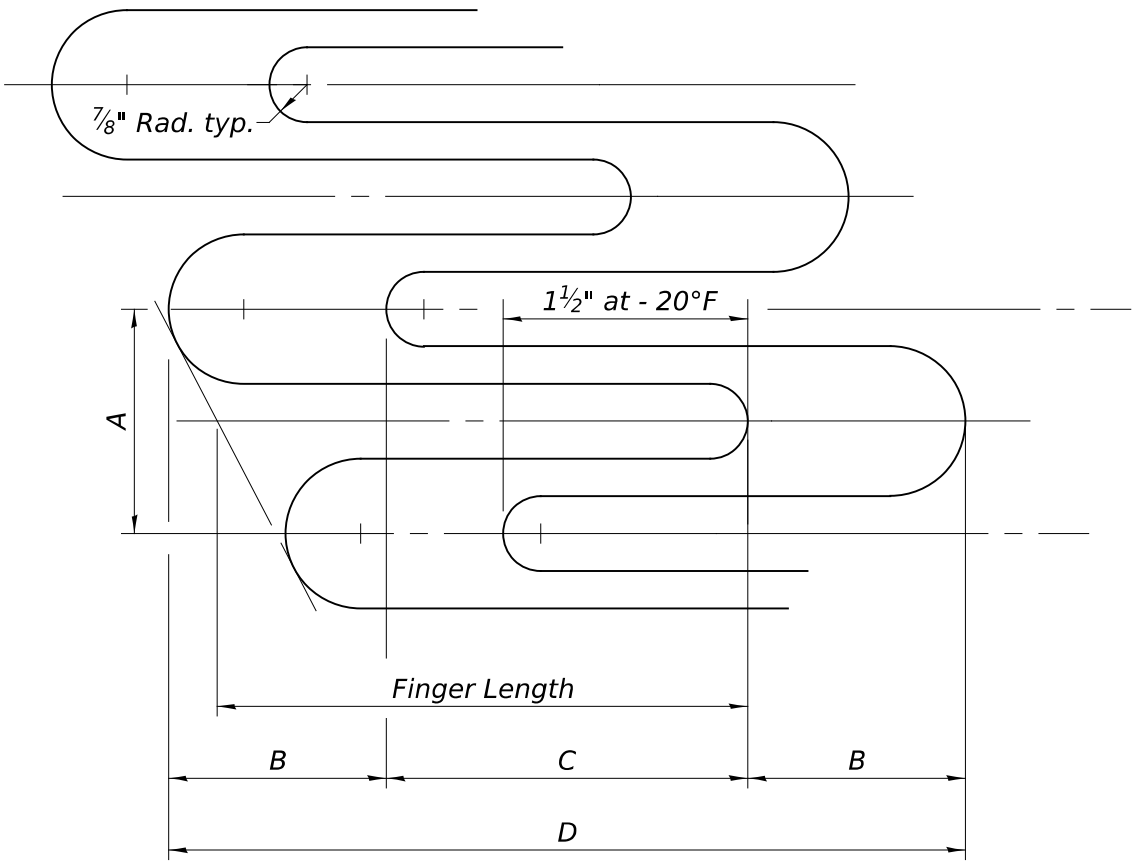
3.6.2.2-10 for spanning the gap in the parapet. The exposed surface of the top sliding plate on a sidewalk shall be textured. The trough shall be continuous under sidewalks and medians and terminate as illustrated in Figures 3.6.2.2-5 and 3.6.2.2-9.

Finger plates should not be used on structures with significantly curved girders (radius < 2000 ft.). When finger plate expansion joints are appropriate for curved structures, the finger joint should be oriented such that the finger expansion is placed along the chord extending from the expansion joint to the fixed substructure.

Joints with finger lengths greater than 14 in. require modular or special expansion joint designs beyond those detailed in this manual. The Bureau of Bridges & Structures should be consulted to verify the viability and acceptability of any proposed special joint design.

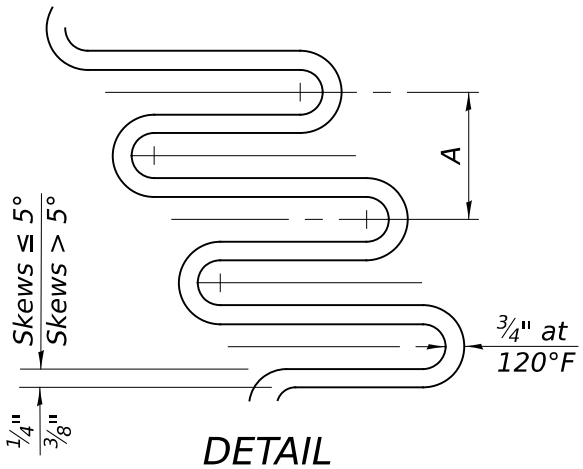
Finger plate expansion joints are paid for per ft. furnished according to Article 520.13 of the Standard Specifications.

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JOINT OPENING AND GEOMETRY DETAIL

Notes: A = 4" for skew ≤ 5°
A = 4¼" for skew > 5°
B = Exp. + ¾" at Normal Temp.
C = Exp. at Normal Temp. + 1½" + A (Tan skew)
D = 2B + C
Finger Length = B + C - A/2 (Tan skew)



DETAIL

FINGER PLATE DIMENSIONS

Figure 3.6.2.2-1

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Finger Length (in.)	Finger Thickness (in.)	Weld Length (in.) each side of stool
5.0	1.0	5.75
5.25	1.125	6.25
5.5	1.125	6.75
5.75	1.25	7.0
6.0	1.25	7.5
6.25	1.375	7.75
6.5	1.375	8.25
6.75	1.5	8.75
7.0	1.5	9.25
7.25	1.625	9.5
7.5	1.625	9.75
7.75	1.625	10.25
8.0	1.75	10.5
8.25	1.75	10.75
8.5	1.875	11.25
8.75	1.875	11.5
9.0	2.0	11.75
9.25	2.0	12.0
9.5	2.125	12.25
9.75	2.125	12.5
10.0	2.25	12.75
10.25	2.25	13.0
10.5	2.375	13.25
10.75	2.375	13.5
11.0	2.5	13.75
11.25	2.5	14.0
11.5	2.75	14.25
11.75	2.75	14.5
12.0	2.75	14.75
12.25	2.75	15.0
12.5	3.0	15.25
12.75	3.0	15.25
13.0	3.0	15.5
13.25	3.0	15.75
13.5	3.0	15.75
13.75	3.25	16.0
14.0	3.25	16.0

FINGER PLATE DESIGN TABLE

Figure 3.6.2.2-2

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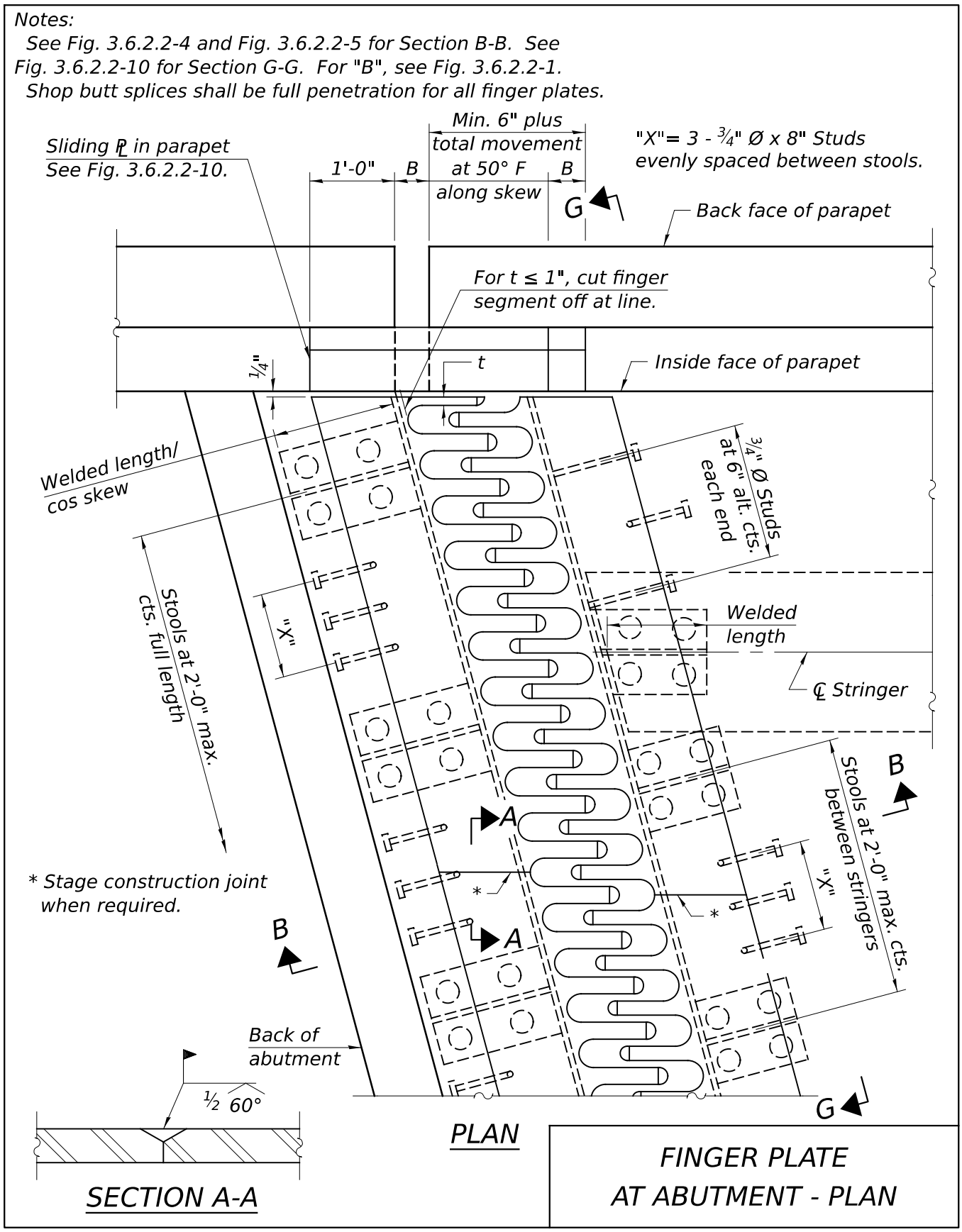
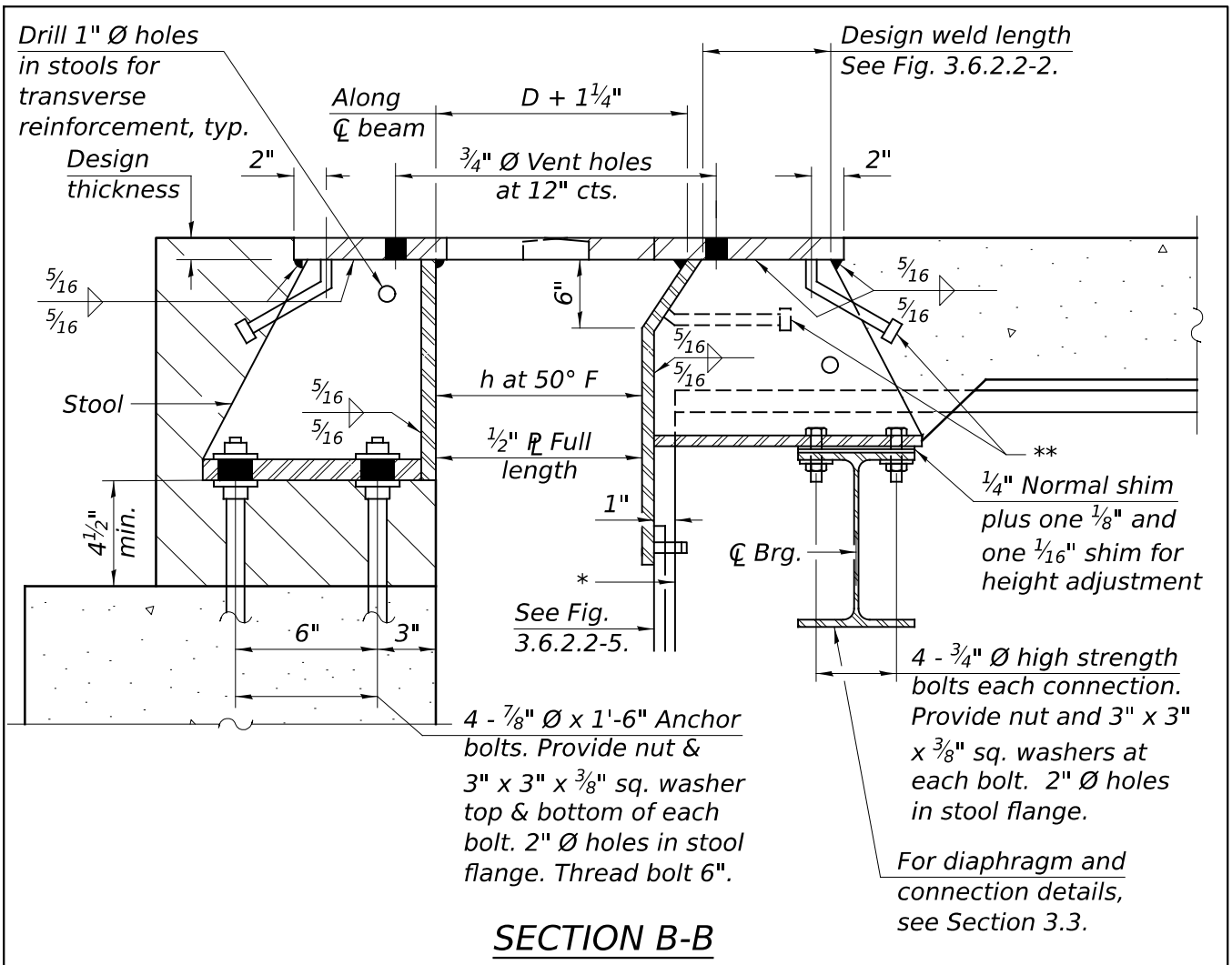
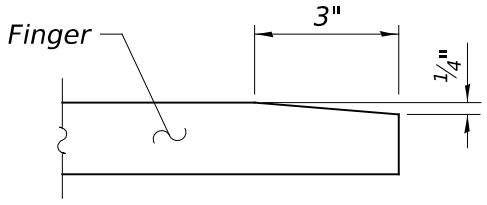


Figure 3.6.2.2-3

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SECTION B-B



BEVEL DETAIL

* Cut end of beam parallel to skew.

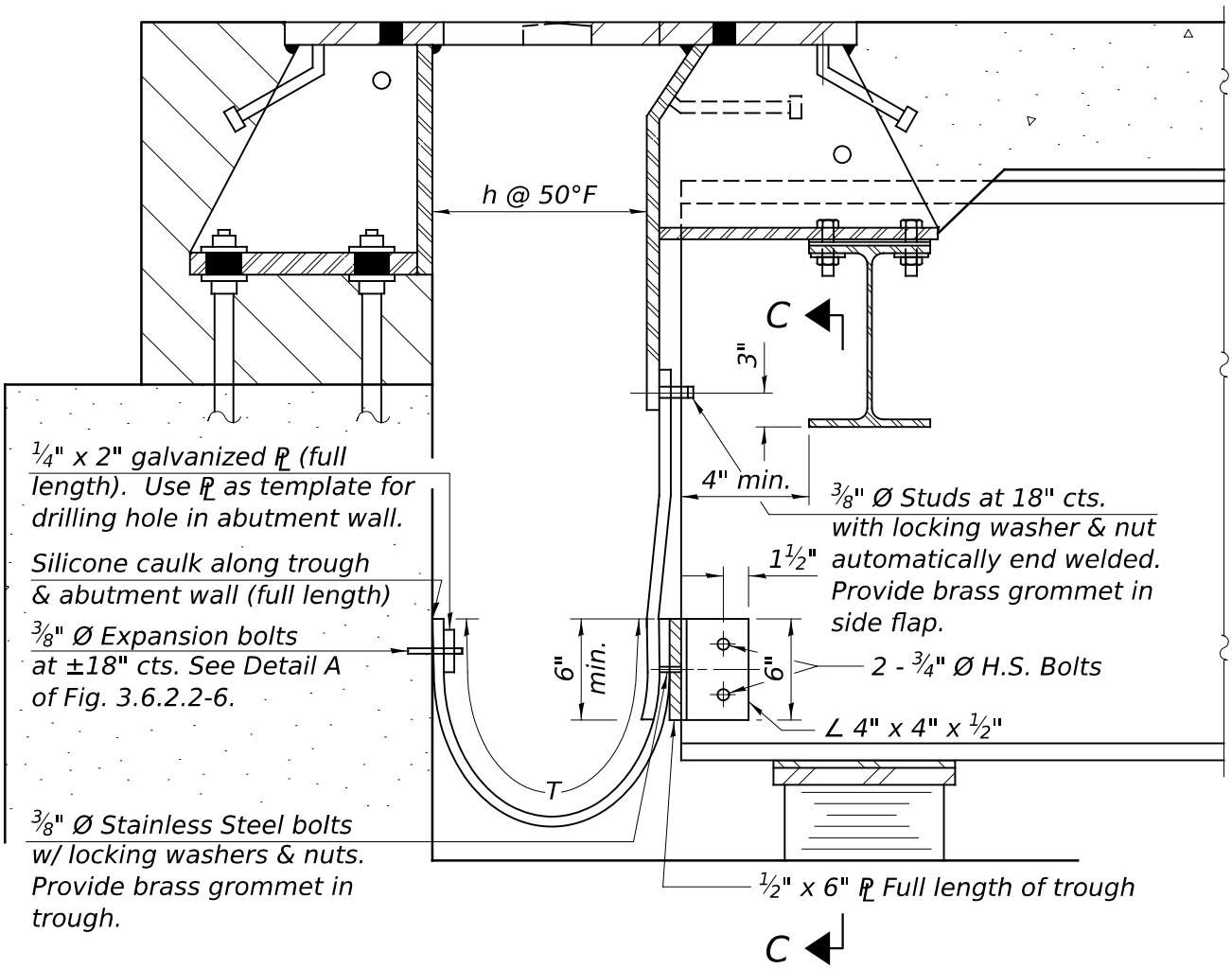
** ¾" Ø x 8" Granular or solid flux filled headed studs conforming to Art. 1006.32 of the Std. Specifications.

Notes
For "D", see Fig. 3.6.2.2-1.
Hatched area to be poured after expansion assembly has been adjusted. Reinforce as shown in Section 3.8.
h = 3" plus (Exp x cos skew)

FINGER PLATE AT ABUTMENT - SECTION

Figure 3.6.2.2-4

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SECTION B-B

(Refer to Fig. 3.6.2.2-4)

Notes:
T = 1.5h + 15"
For "h," see notes of Fig. 3.6.2.2-4.
For Section C-C, see Fig. 3.6.2.2-6.

TROUGH AT ABUTMENT - SECTION

Figure 3.6.2.2-5

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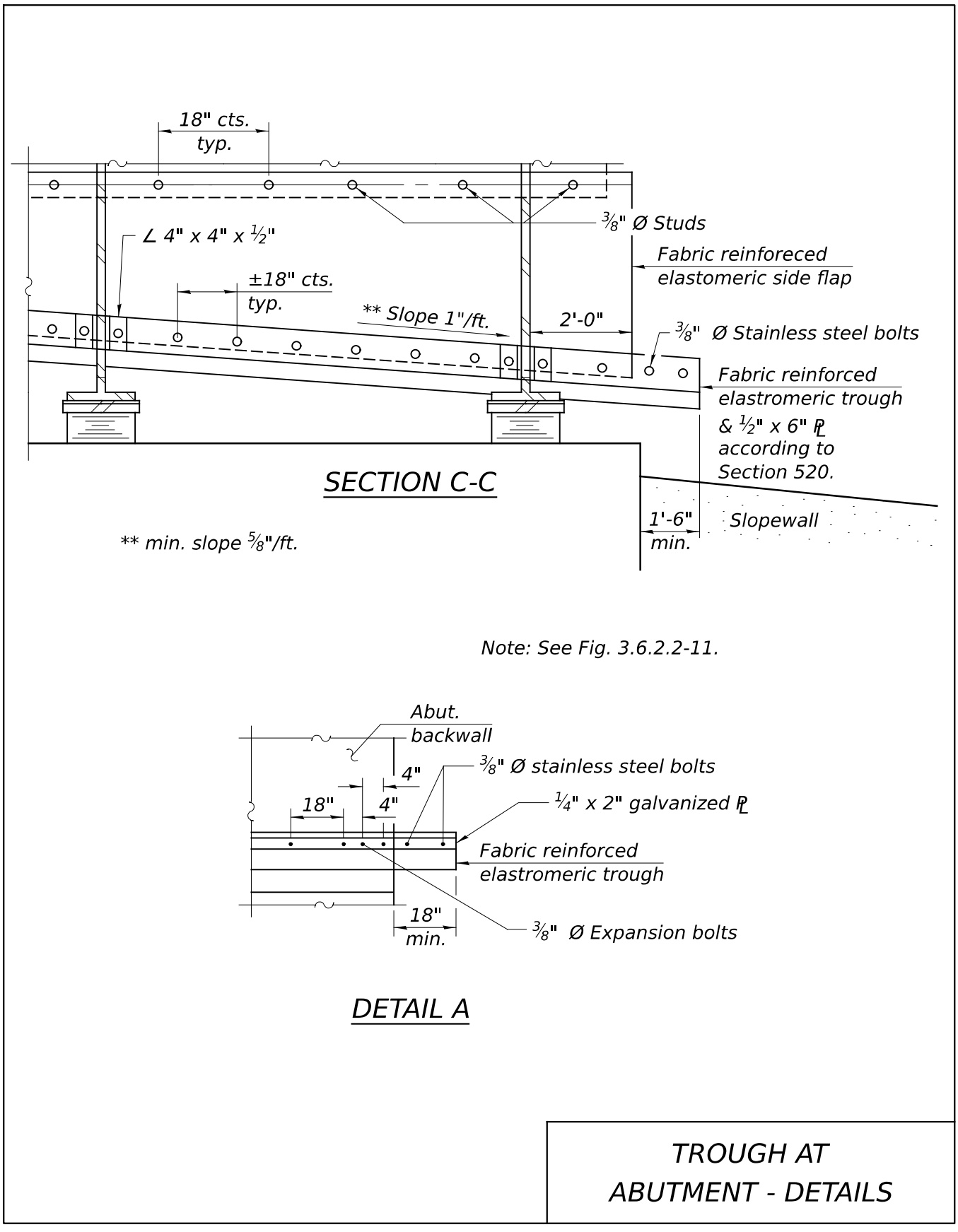


Figure 3.6.2.2-6

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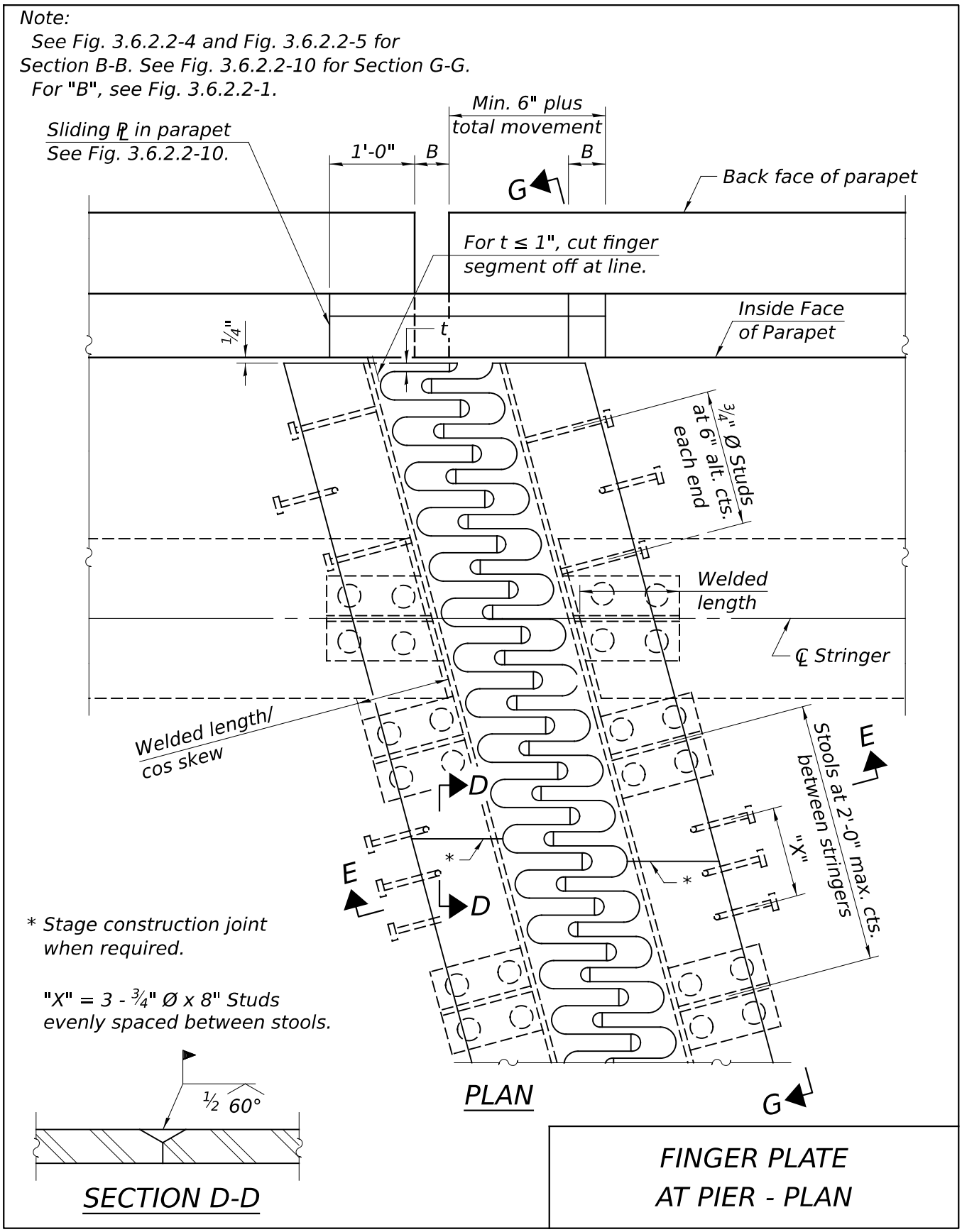
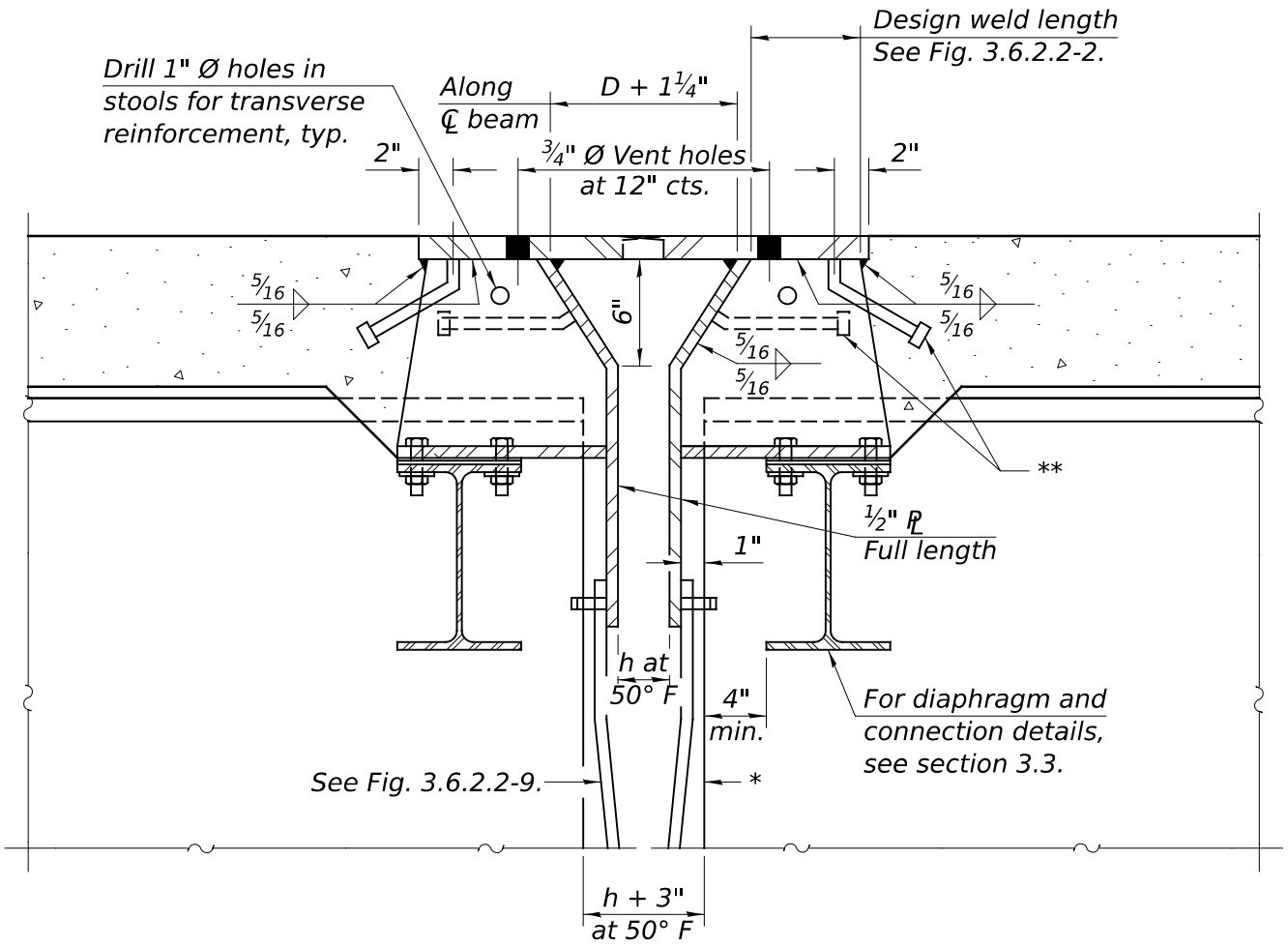
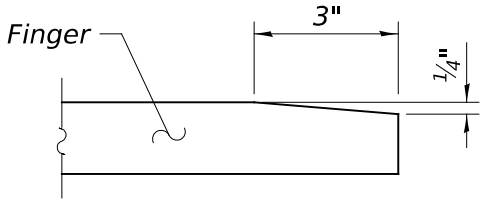


Figure 3.6.2.2-7

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SECTION E-E



BEVEL DETAIL

* Cut end of beam parallel to skew.

** ¾" Ø x 8" Granular or solid flux filled headed studs conforming to Art. 1006.32 of the Std. Specifications.

Notes:
For "D", see Fig. 3.6.2.2-1.
h = 3" plus (Exp x cos skew)

FINGER PLATE AT PIER - DETAILS

Figure 3.6.2.2-8

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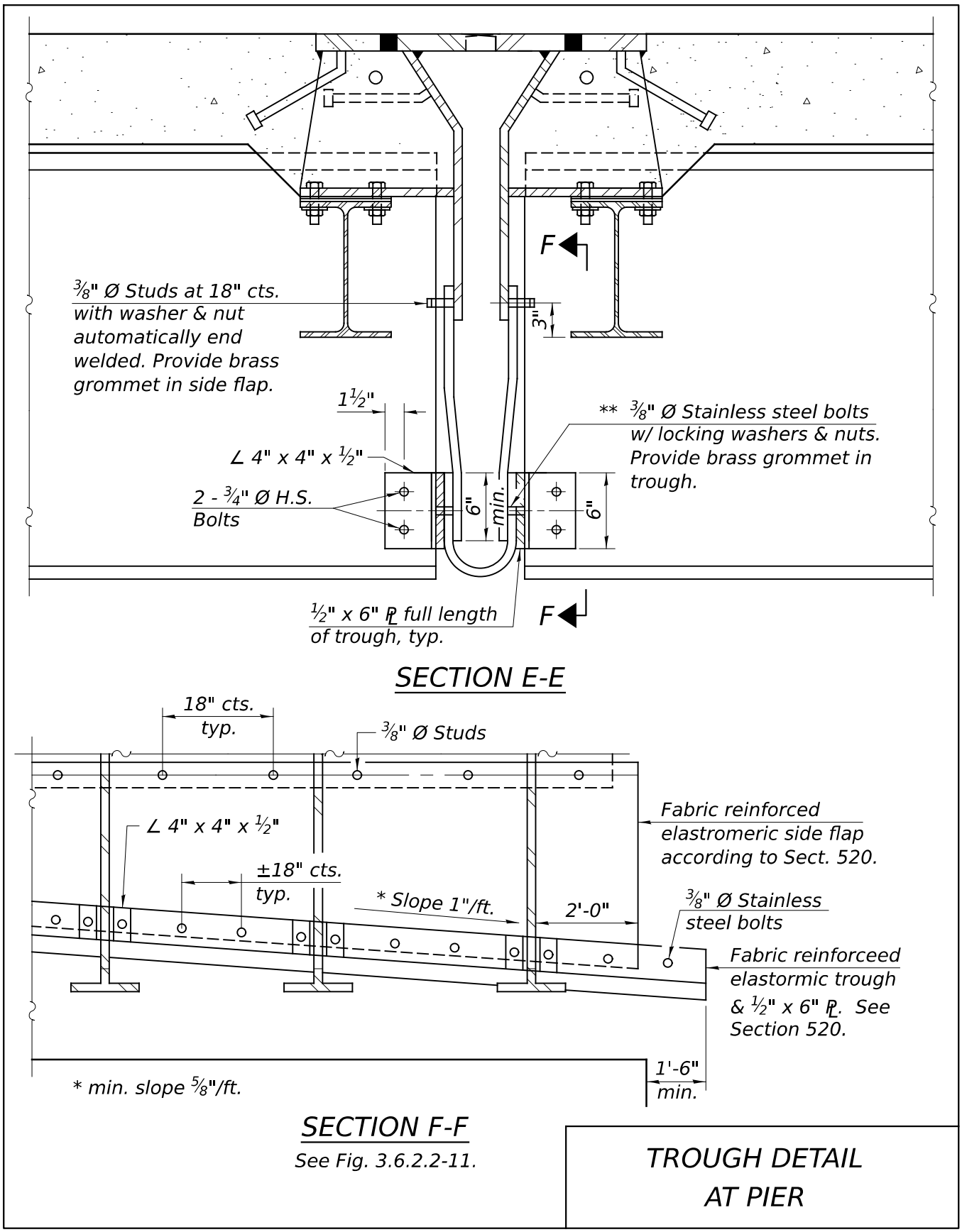
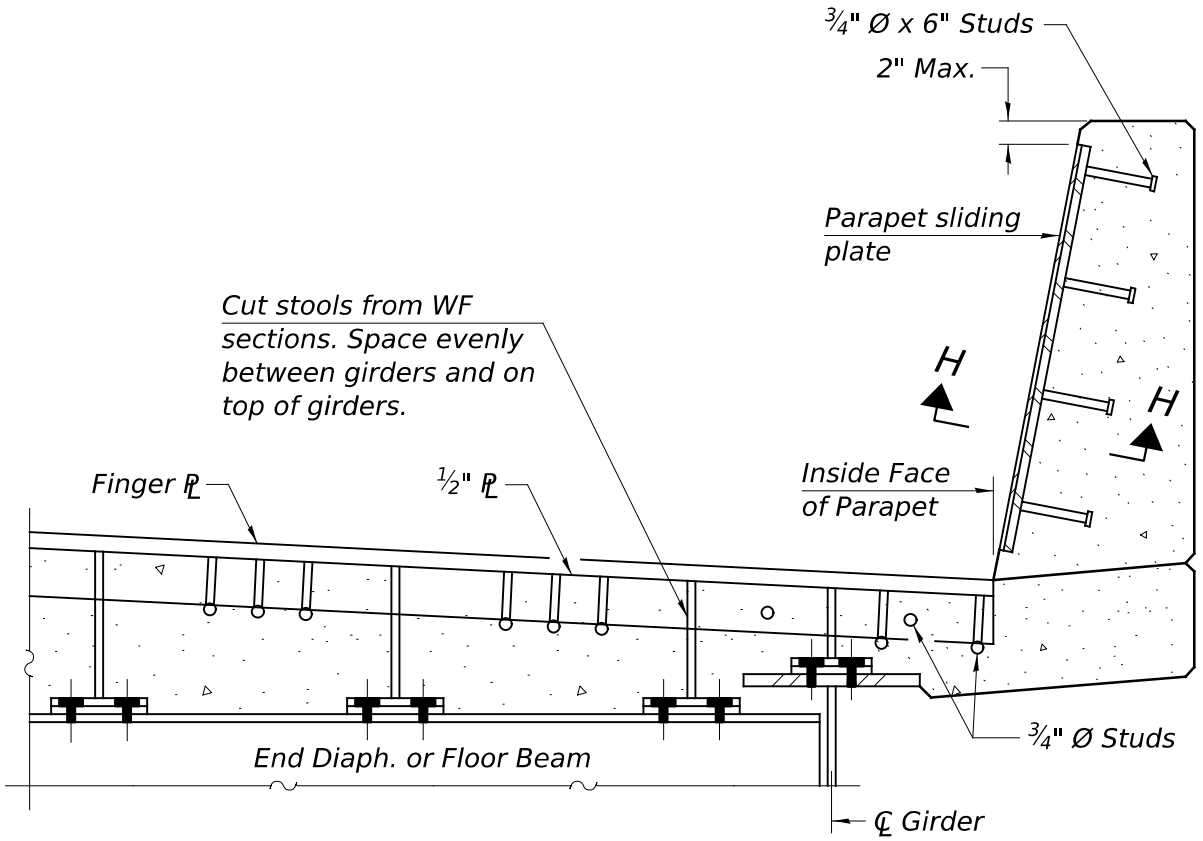


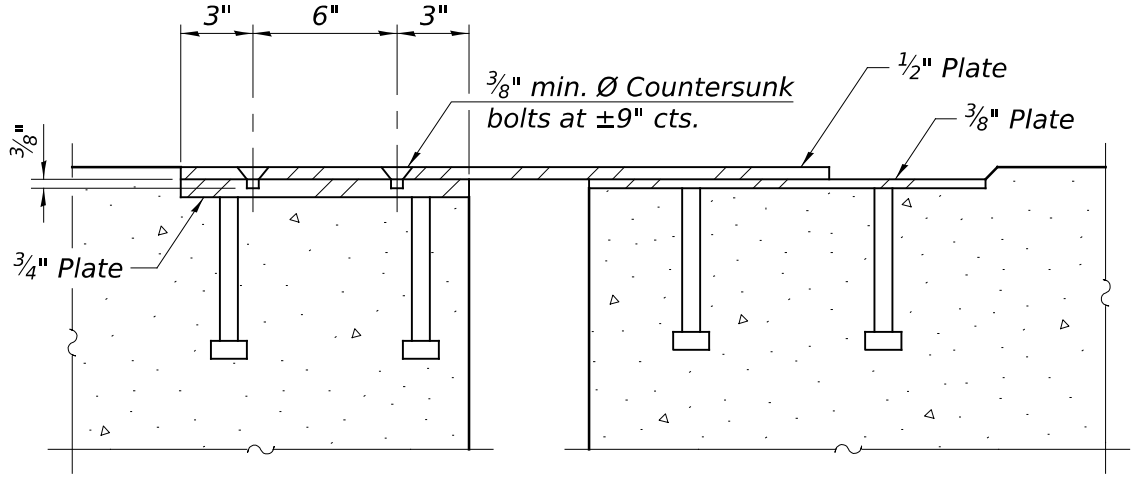
Figure 3.6.2.2-9

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SECTION G-G

(See Fig. 3.6.2.2-3 or 3.6.2.2-7)

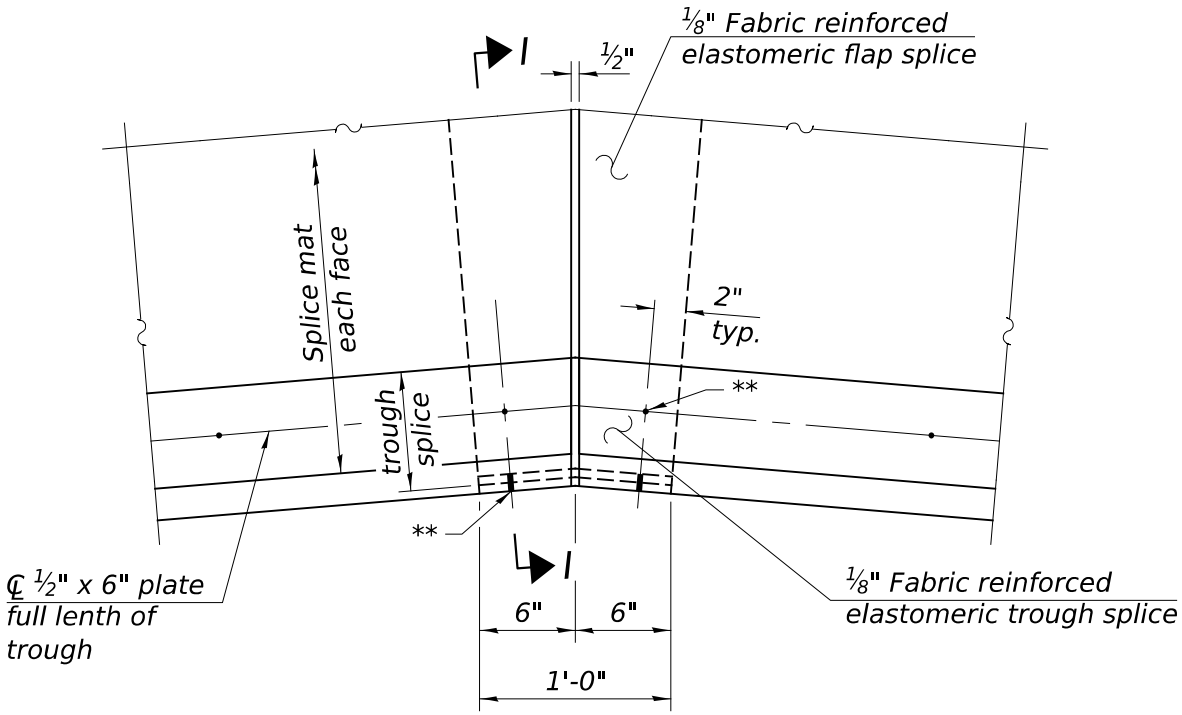


SECTION H-H

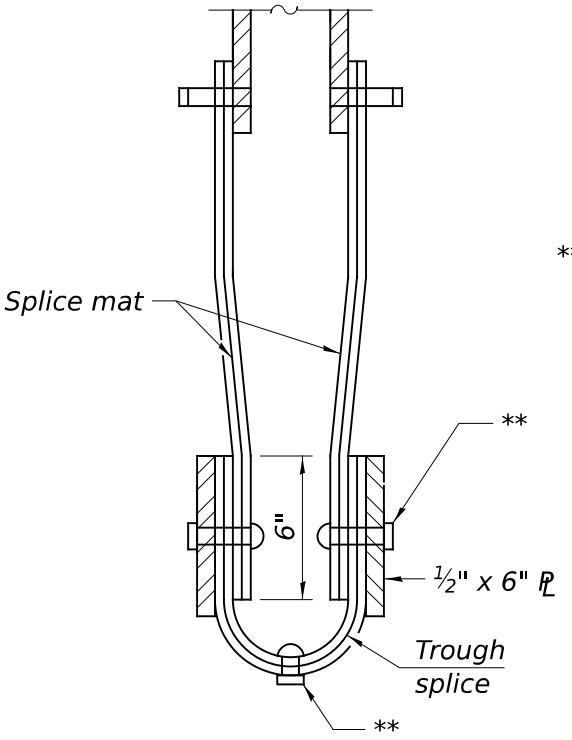
FINGER PLATE AND SLIDING PLATE DETAILS

Figure 3.6.2.2-10

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TROUGH SPLICE DETAIL



SECTION I-I

** ⅜" Ø Stainless Steel bolts w/washers & nuts. Provide brass grommet in trough.

TROUGH SPLICE DETAIL

Figure 3.6.2.2-11

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3.6.2.3 Modular Joints

A modular joint is basically a series of strip seals supported by rails and support bars to form a joint system capable of accommodating movements from 3 ⅛ to 15 in. This system may be used in lieu of a fingerplate and trough system. A hybrid (swivel) modular joint system is a type of modular joint that is designed to accommodate differential non-parallel longitudinal movements, and it shall be used for skews greater than 60 degrees, for structures where lateral displacements are anticipated to exceed ¾", for critical or essential structures that must remain open after a seismic event, for flared superstructures, or for curved structures to account for the effects of curvature. A Guide Bridge Special Provision was developed for and shall be used with this type of expansion joint.

Modular and swivel modular expansion joints are theoretically a very useful expansion device for longer structures and currently the only joints readily available to accommodate significant lateral movement. Improvements have been made to the concrete details encasing these joints, intended to prolong the service life and reduce lifecycle costs. This section herein introduces bridge details for accommodating modular joint systems and updated policies and GBSP 18 applicable for modular joints.

3.6.2.3.1 Joint Application

Modular expansion joints are an expansion device option, along with steel finger plate joints, for structures with contributing expansion lengths as shown in Figure 2.3.6.1.6-1. The joints consist of a series of strip seal glands, held in place by center beams and edge beams studded into deck concrete. Wheel loads are transferred from the center beams to support bars and their bearings. A concrete support ledge, or the bridge girder end bracing, under the support bar bearings is utilized to direct the load into the girder and bearings at the joint.

In addition to allowing for longitudinal expansion, standard modular joints also allow for minimal lateral movement (typically around 3/8" per gland), while swivel modular joints allow for much larger lateral movements. In comparison, steel finger joints allow for 3/8" lateral movement total.

The more complex, multi-directional swivel modular joints have a higher cost and more complex installation procedure. For structures that must remain open to all traffic after seismic events (critical or essential structures), swivel modular joints shall be the primary joint option considered. For joints requiring higher lateral movements, consideration shall be given to selecting a larger standard modular joint size in lieu of using a swivel joint. The benefit of reducing any possible normal racking due to temperature movements typically outweighs the small additional cost to increase the joint size.

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The joint type (modular, finger plate, or other) shall be specified on the Type, Size, and Location (TSL) plan, but the size of the joint does not need to be shown. The specific joint size and the need for a swivel modular joint will be determined during the design phase.

Additional consideration shall be given to the bearings on structures with lateral movement. In some cases, the expected lateral movement at the expansion bearings is greater than the typical distance between the beam and the side retainer. If this occurs, the retainers should be placed at least the expected lateral movement distance away from the beam flanges.

3.6.2.3.2 Contributing Expansion Length and Total Longitudinal Movement

For structures with only one fixed pier, the contributing expansion length for each abutment shall be the length from the joint at the abutment to the centerline of the fixed pier. For structures with multiple fixed piers (typically two) the contributing expansion length shall be the length from the joint at the abutment to the centroid of fixity, typically the middle of the span between the fixed piers. For structures with joints at the piers, the expansion length shall be the distance from one joint to the center of fixity.

The total longitudinal movement in inches shall be shown on the contract plans and calculated using the following equation:

$$T = \gamma \alpha L(\Delta t)$$

Where:

T = total longitudinal movement along centerline of roadway (in.)

γ = 1.2 (AASHTO LRFD Bridge Design Specifications Table 3.4.1-1 and Table 3.4.1-6; max load factor for TU due to Uniform Temperature)

α = 0.0000065/ºF

L = contributing expansion length (in.)

Δt = 140 ºF (-20 ºF to 120 ºF)

The designer shall then choose the joint size based on this calculated movement. The dimension shown for the actual modular joint width is determined by the manufacturer and shall be left as a starred dimension with a note as shown on the figures and base sheets.

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The distance between the concrete at right angles, “A” at 50 ºF, shown in Figure 3.6.2.3-1, Figure 3.6.2.3-2, Figure 3.6.2.3-7, and Figure 3.6.2.3-8 shall be calculated as:

$$"A" = (T/2)\cos(S) + 3/4"$$

Where:

S = skew angle (degrees), measured from the baseline to a line perpendicular to the centerline of joint

The additional 3/4" is provided to ensure there is no hard contact between the concrete surfaces.

For the purposes of calculating the quantity for the contract plans, the length shall be calculated along the centerline of the joint from out-to-out of the deck.

Structures with larger lateral movement demands requiring swivel modular joints shall also specify the maximum vertical displacements, transverse displacements, and horizontal rotations on the contract plans.

3.6.2.3.3 Modular Joint Deck Details

Two types of deck end details have been developed or expanded upon to improve the performance of the modular joints and the deck surrounding them.

Full Depth Deck Blockout: A deck end detail utilizing a full depth deck blockout is shown in Figure 3.6.2.3-1 thru Figure 3.6.2.3-6. Full depth deck blockouts allow for easier forming and reinforcement bar placement, greater flexibility to adjust rebar in the field to avoid interferences and provides a larger monolithic cast of concrete around the joint assembly. The joint assembly shall be temporarily supported off of the girder ends until the concrete is placed. If intermittent support is required due to large skews, large beam spacing, or heavy joints then the joint shall be additionally supported off of the cross-frame/diaphragm top members. This detail type is applicable for all girder sizes. Longitudinal bar splicers are not required for the full depth blockout detail type.

A blockout and construction joint is preferred according to AASHTO 14.5.5.1. It may be optional if the designer evaluates the pouring sequence and determines that there will be no adverse effects, such as excessive rotations, that will inhibit concrete cover or cause joint misalignment. The blockout and construction joint shall be labeled on the plans as optional, as applicable.

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Partial Depth (Ledge) Deck Blockout: A deck end detail utilizing a partial depth blockout and concrete ledge is shown in Figure 3.6.2.3-7 thru Figure 3.6.2.3-13. A concrete ledge provides a formed concrete platform to easily place and adjust the joint assembly. While the increased concrete thickness below the support box reduces reinforcement congestion, the ledge requires complicated formwork and reinforcement bar placement in order to prevent modular joint assembly interference. Due to the increased concrete thickness this detail may be more suitable for deeper girder sections but is not recommended for structures with web depths less than 54". The bar splicer/reinforcement detail shown in Figure 3.6.2.3-13 shall be included on the bar splicer base sheet for plans utilizing this concrete ledge type detail.

Both types of deck end details shall have a minimum clearance of 4¼" above the support box and the full depth blockout shall have a minimum clearance of 6" below the support box.

3.6.2.3.4 Modular Joint Beam End Details

Beam ends shall be coped whenever feasible to avoid support box interference and allow the support boxes to have a uniform spacing. Plate girders shall utilize the standard coping details shown in Figure 3.6.2.3-14 and Figure 3.6.2.3-15. If the beams are not coped then the modular joint assembly support boxes shall be spaced to miss the beam flanges.

3.6.2.3.5 Modular Joint End Cross-Frames and Diaphragms

Cross-frames are the preferred bracing method for plate girder structures with modular joints. Cross-frame details for various skews are shown in Figure 3.3.23.1-1 thru Figure 3.3.23.1-6.

The centerline of the diagonal members of the cross-frame are not required to intersect the centerline of the top chord, but it should be as close as practical. When the diagonals of the cross-frame cannot practically have a minimum angle of 30 degrees (as measured from the bottom chord) due to beam spacing, skew, and/or web/cope depth, an alternate bent plate channel is required as shown in Figure 3.3.23.1-6. The bent plate channel is not allowed for skews greater than or equal to 45 degrees. Additional analysis is required if the minimum diagonal angle cannot be satisfied for skews 45 degrees or greater.

For end cross-frames and end bent plate diaphragms at stage construction joints, see Figure 3.3.23.1-7 and Figure 3.3.23.1-8.

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3.6.2.3.6 Modular Joint Resources

Twelve base sheets have been developed that show the minimum amount of information required on the plans for the deck end, beam end, and sliding plate details for the structure. Modular joint details, deck end reinforcement, and beam end/end diaphragm details may be found in the Base Sheets – General, Superstructure – Expansion Steel Beams, and Diaphragms – Expansion Abutment Steel Beams base sheet libraries, respectively.

GBSP 18 (Modular Expansion Joint) includes additional guidance on construction and fabrication requirements, such as:

- Additional weld requirements have been added and tolerances have been developed for various joint dimensions.
- Updated fatigue requirements are clearly specified for different elements of the joint.
- The entire width of the joint shall be designed for the vehicular live load shown on the General Plan & Elevation sheet of the contract plans.
- All non-stainless steel structural elements of the joint assembly shall be hot-dip galvanized.

Modular joint suppliers are required to certify their joint assemblies according to the Department's Prequalified Structural Systems (PQSS) process and then recertify at least every five years to remain on the List of Prequalified Modular Expansion Joints. Recertification will be required immediately if any changes are made to the materials, or joint details. Refer to the latest procedures found at the primary PQSS page.

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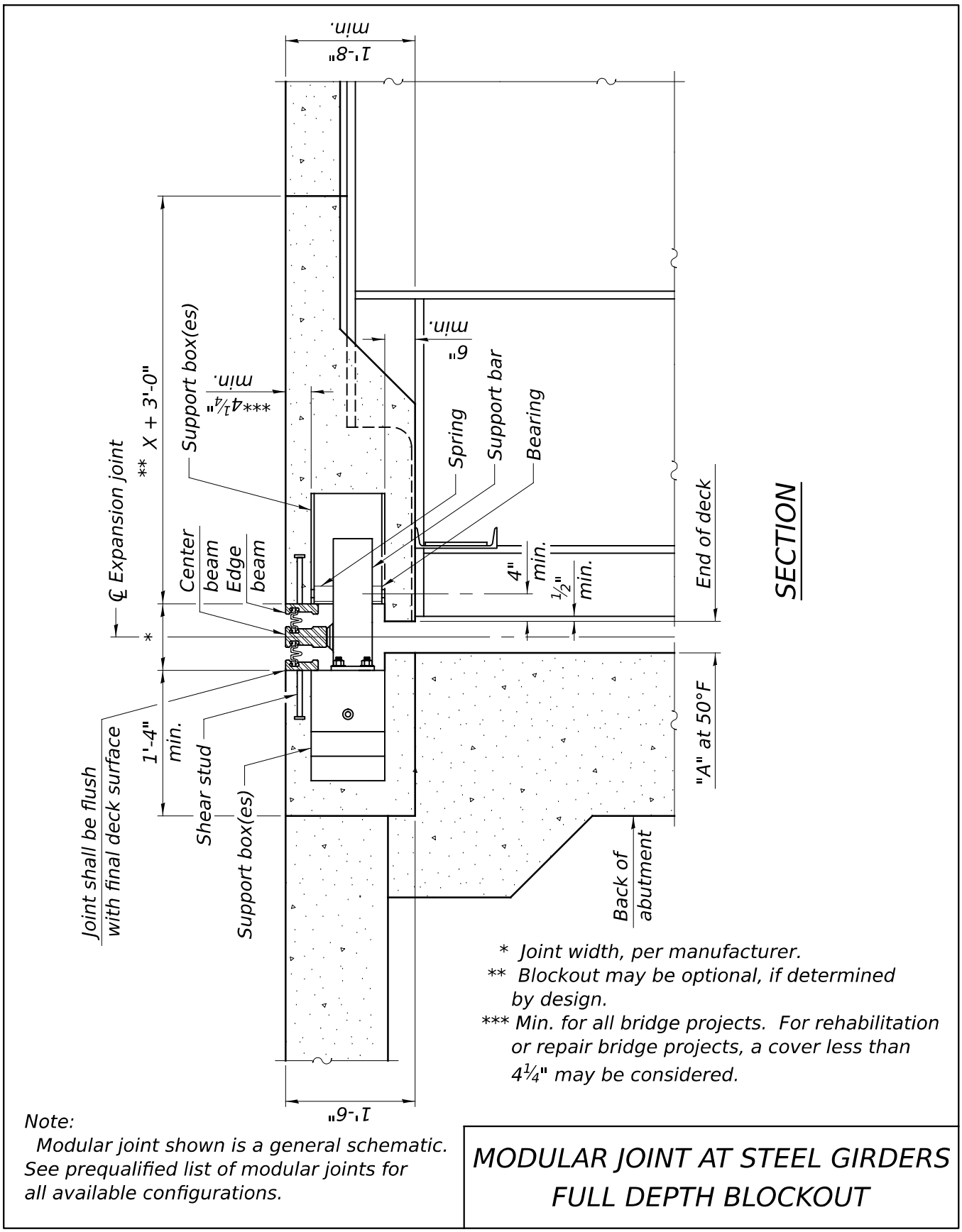


Figure 3.6.2.3-1

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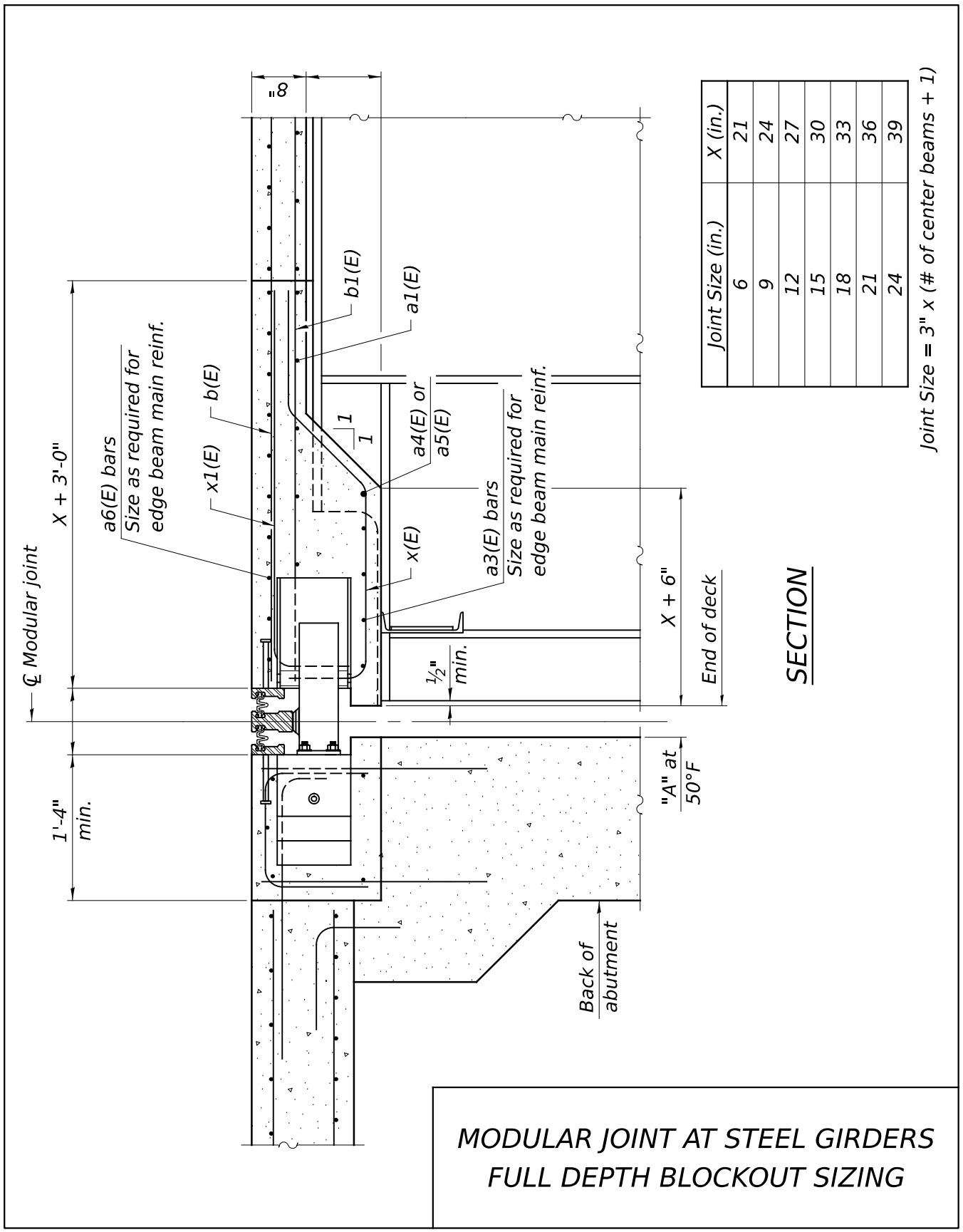
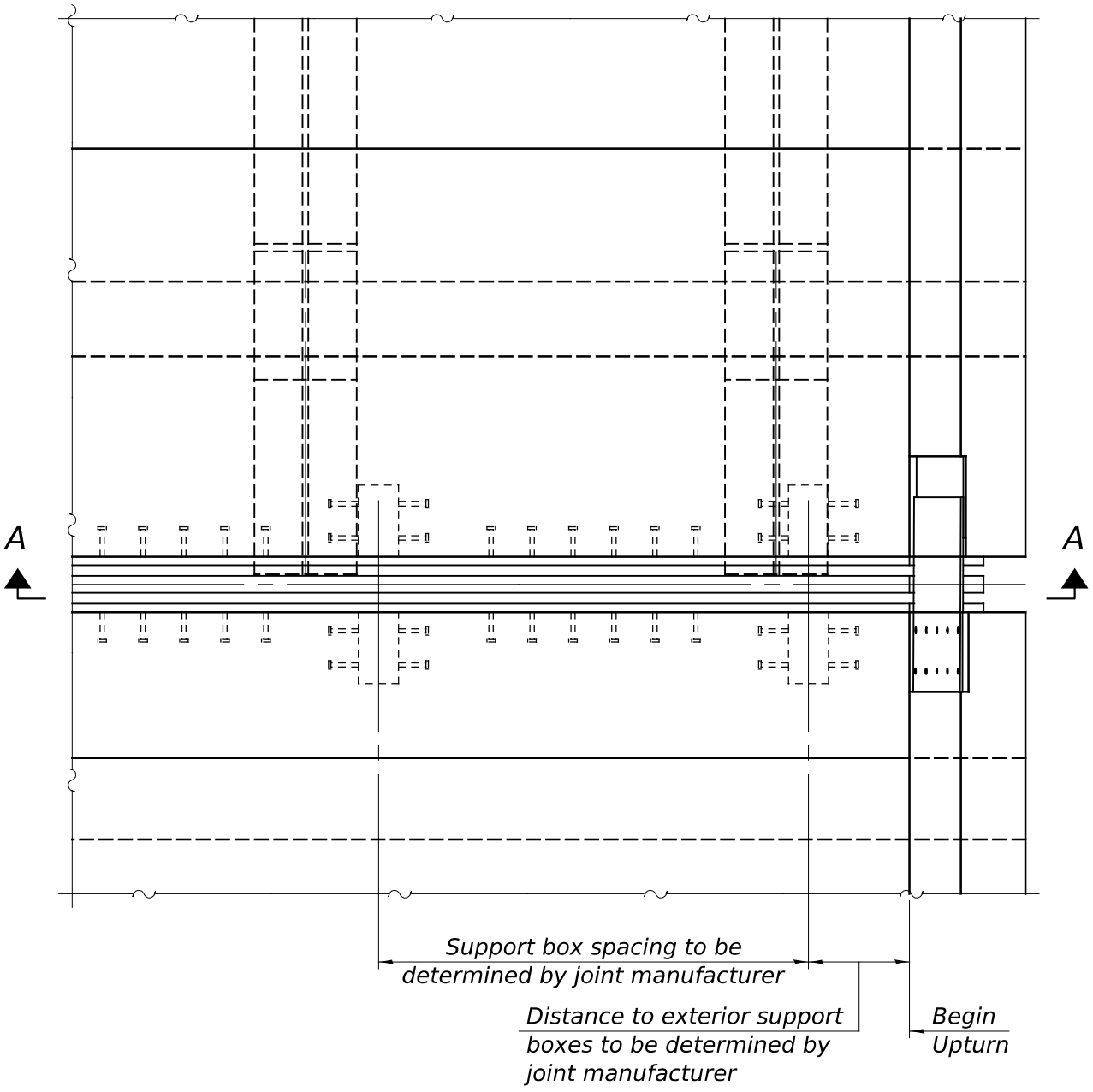


Figure 3.6.2.3-2

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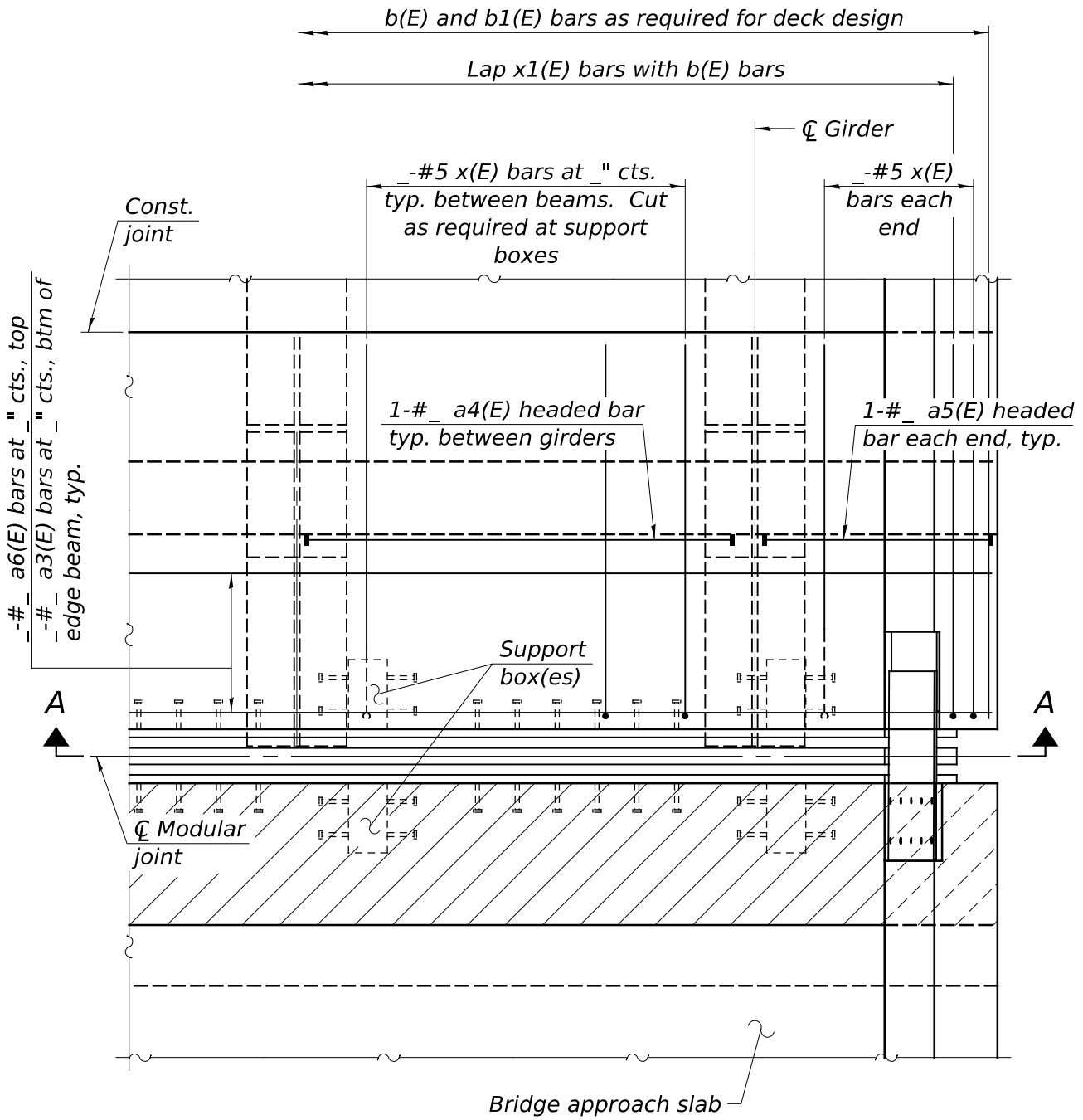


Note:
For Section A-A, see Fig. 3.6.2.3-5 and Fig. 3.6.2.3-6.

SUPERSTRUCTURE PLAN
MODULAR JOINT AT STEEL GIRDERS
FULL DEPTH BLOCKOUT

Figure 3.6.2.3-3

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PARTIAL PLAN

Notes:

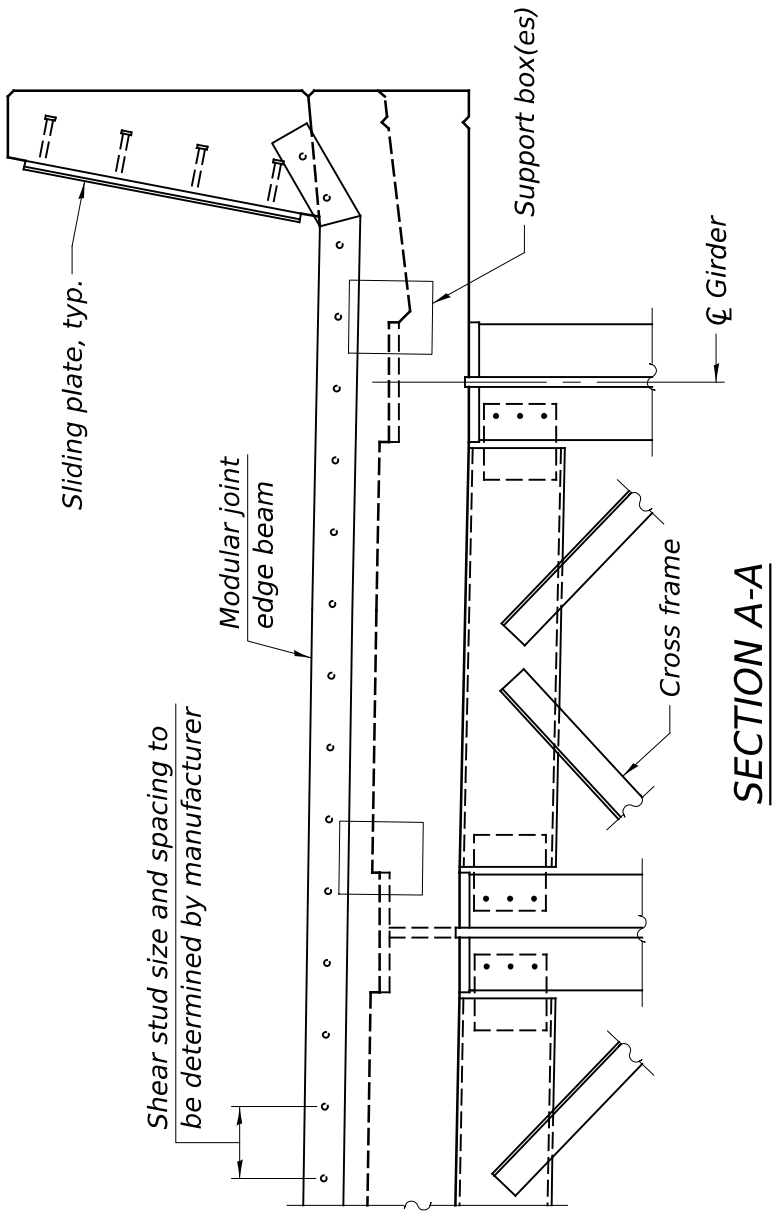
Size and count as required by design for edge beam main reinforcing and bar splicers.

For Section A-A, see Fig. 3.6.2.3-5 and Fig. 3.6.2.3-6.

SUPERSTRUCTURE DETAILS
MODULAR JOINT AT STEEL GIRDERS
FULL DEPTH BLOCKOUT

Figure 3.6.2.3-4

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MODULAR JOINT AT STEEL GIRDERS
FULL DEPTH BLOCKOUT SECTION

Figure 3.6.2.3-5

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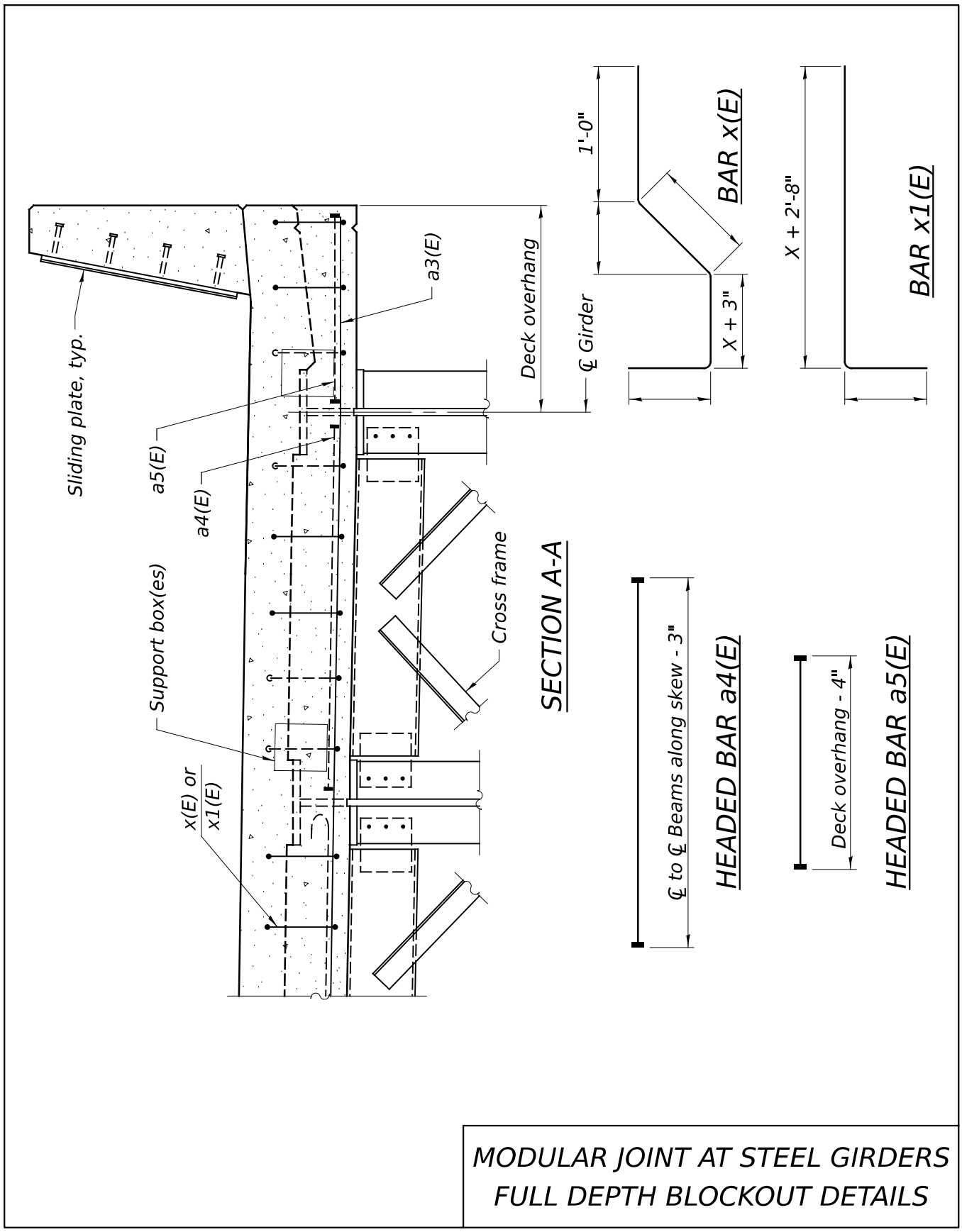
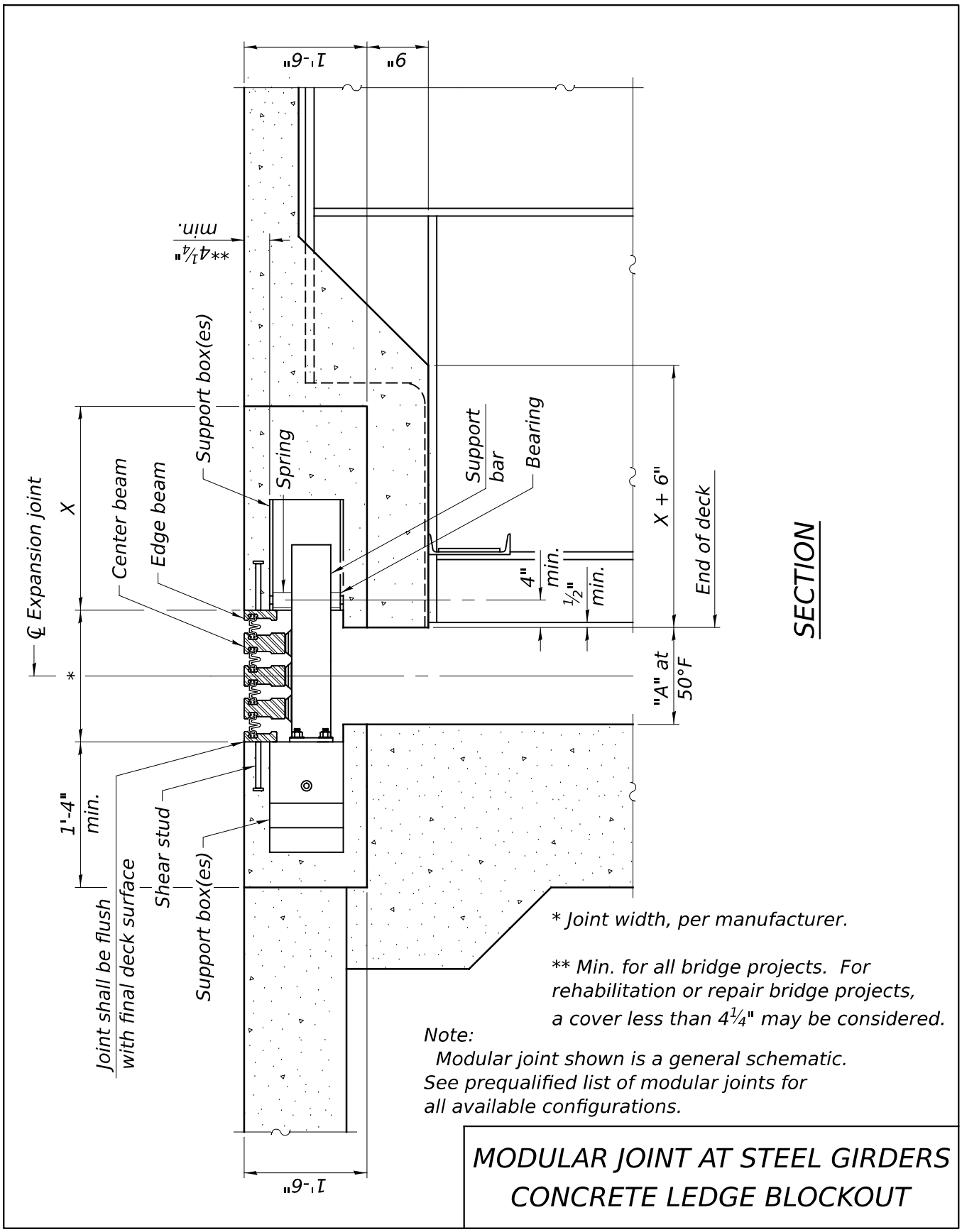


Figure 3.6.2.3-6

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* Joint width, per manufacturer.

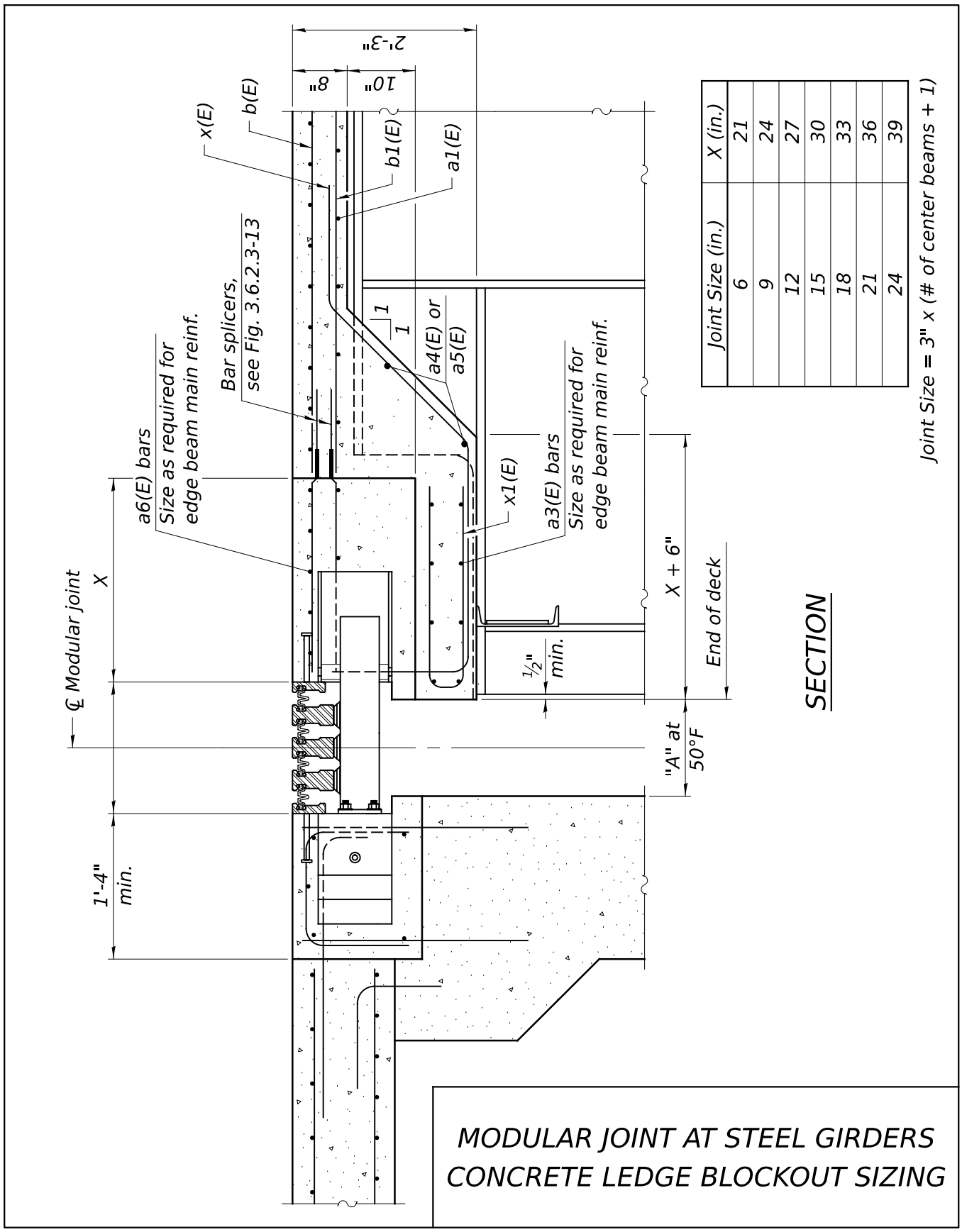
** Min. for all bridge projects. For rehabilitation or repair bridge projects, a cover less than 4¼" may be considered.

Note:
Modular joint shown is a general schematic. See prequalified list of modular joints for all available configurations.

MODULAR JOINT AT STEEL GIRDERS CONCRETE LEDGE BLOCKOUT

Figure 3.6.2.3-7

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Joint Size (in.)	X (in.)
6	21
9	24
12	27
15	30
18	33
21	36
24	39

Figure 3.6.2.3-8

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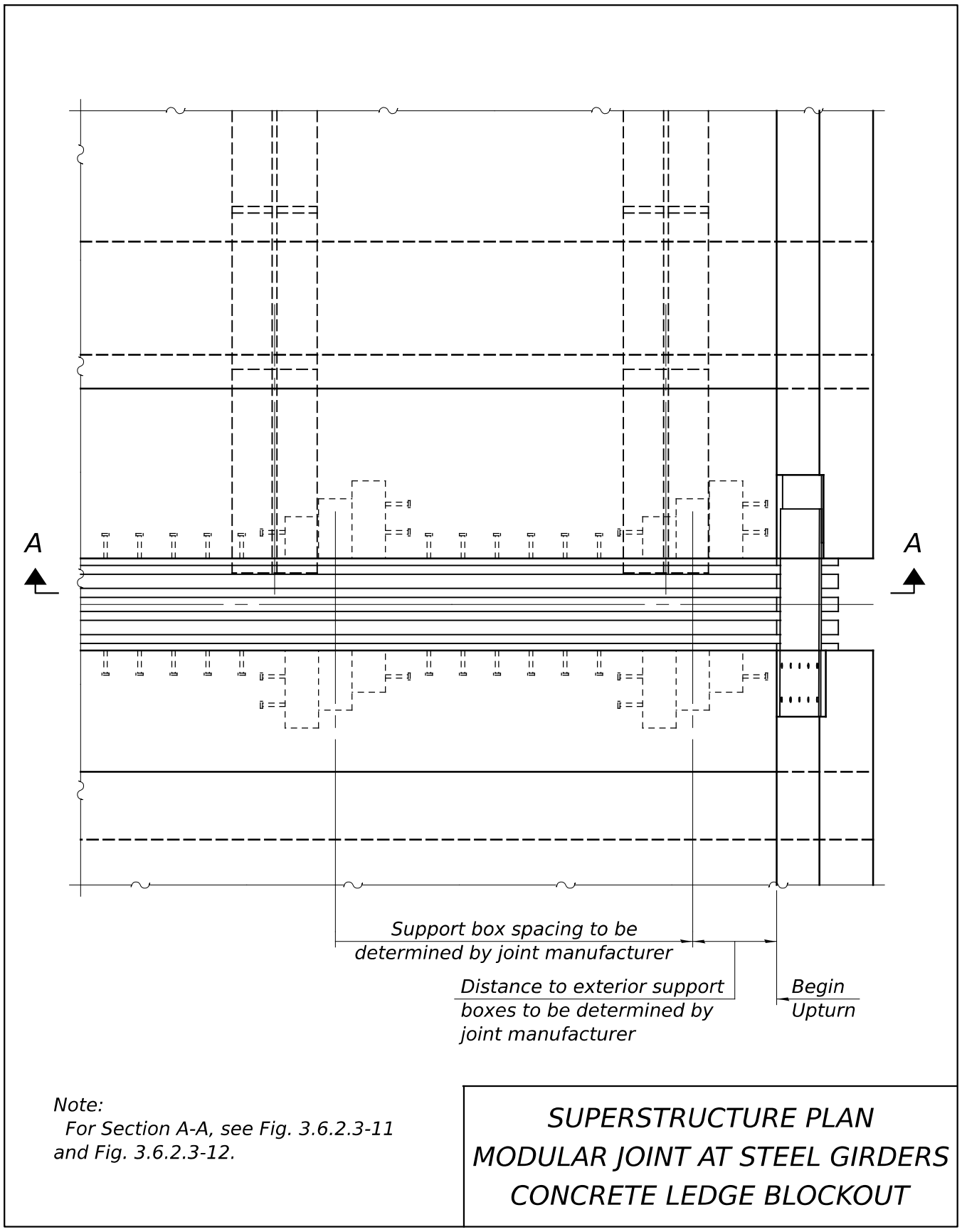
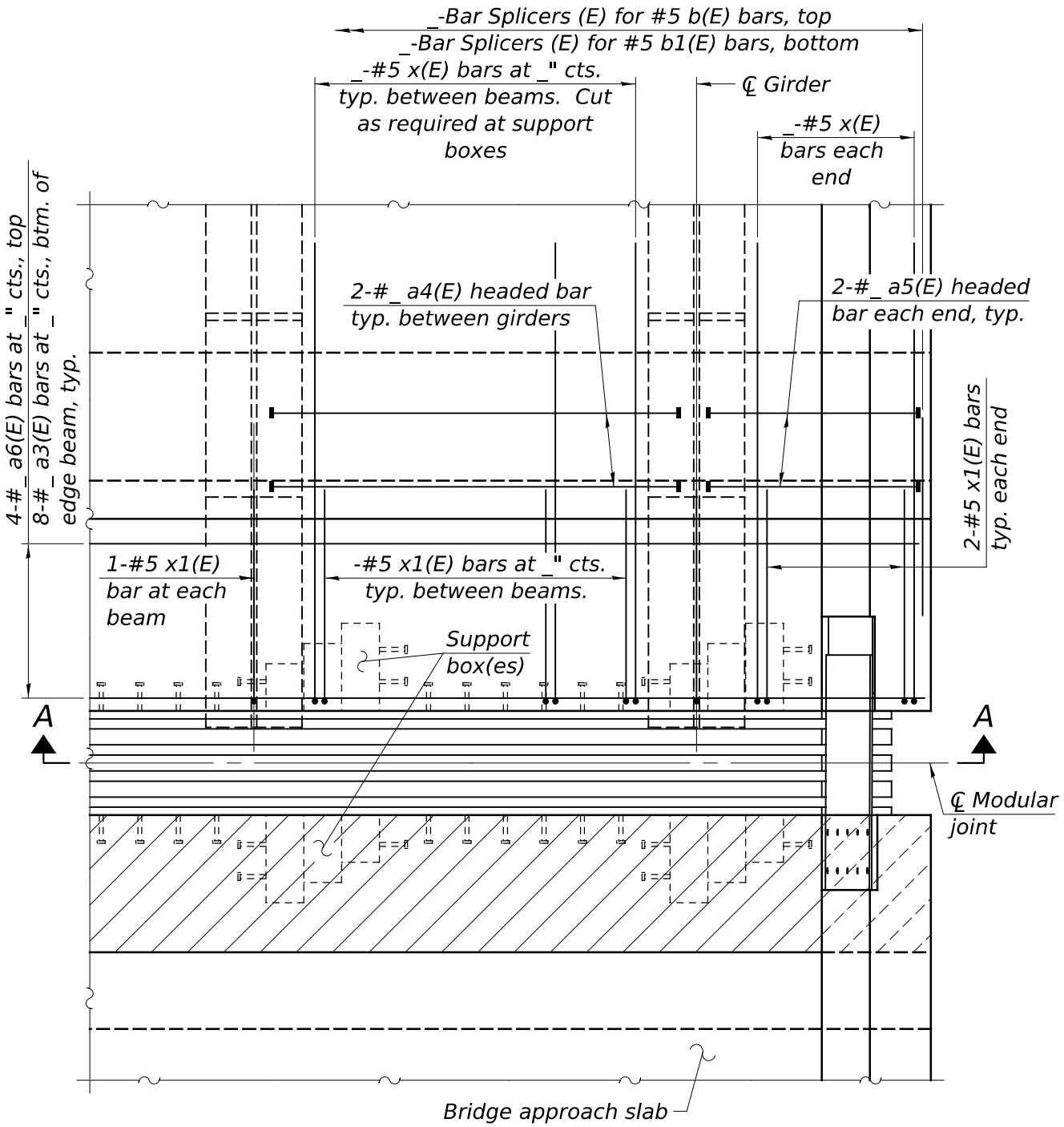


Figure 3.6.2.3-9

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PARTIAL PLAN

Notes:

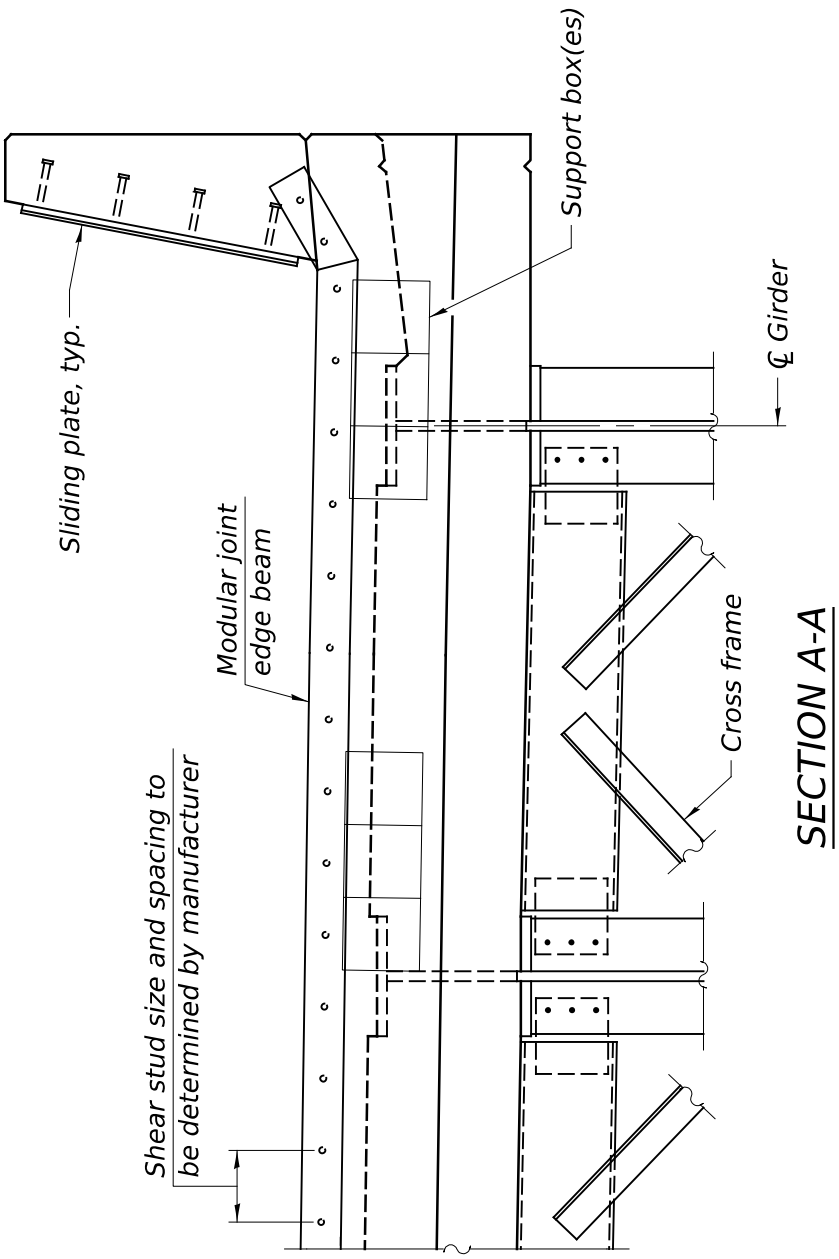
Size and count as required by design for edge beam main reinforcing and bar splicers.

For Section A-A, see Fig. 3.6.2.3-11 and Fig. 3.6.2.3-12.

SUPERSTRUCTURE DETAILS
MODULAR JOINT AT STEEL GIRDERS
CONCRETE LEDGE BLOCKOUT

Figure 3.6.2.3-10

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MODULAR JOINT AT STEEL GIRDERS
CONCRETE LEDGE BLOCKOUT SECTION

Figure 3.6.2.3-11

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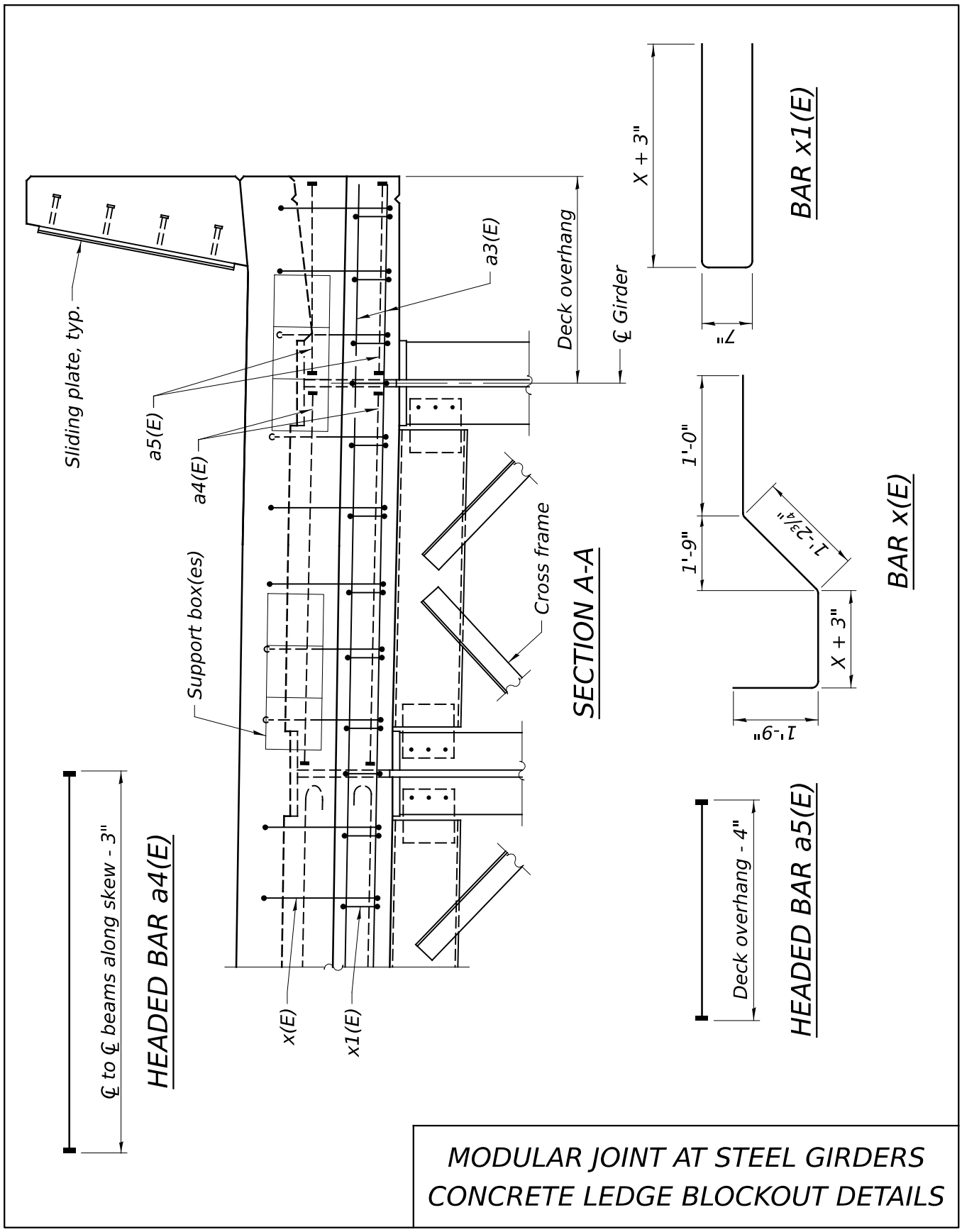
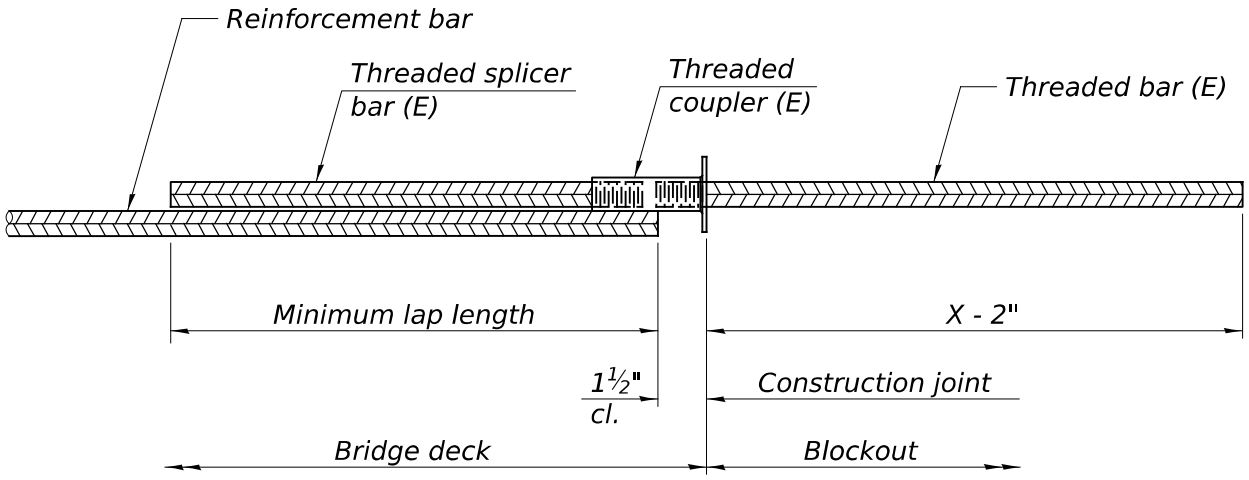


Figure 3.6.2.3-12

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CONCRETE LEDGE BAR SPLICER ASSEMBLY PLAN

(All components shall be provided from one supplier)

Threaded splicer bar length = min. lap length + 1½" + thread length
Threaded bar length = X - 2" + 1 thread length

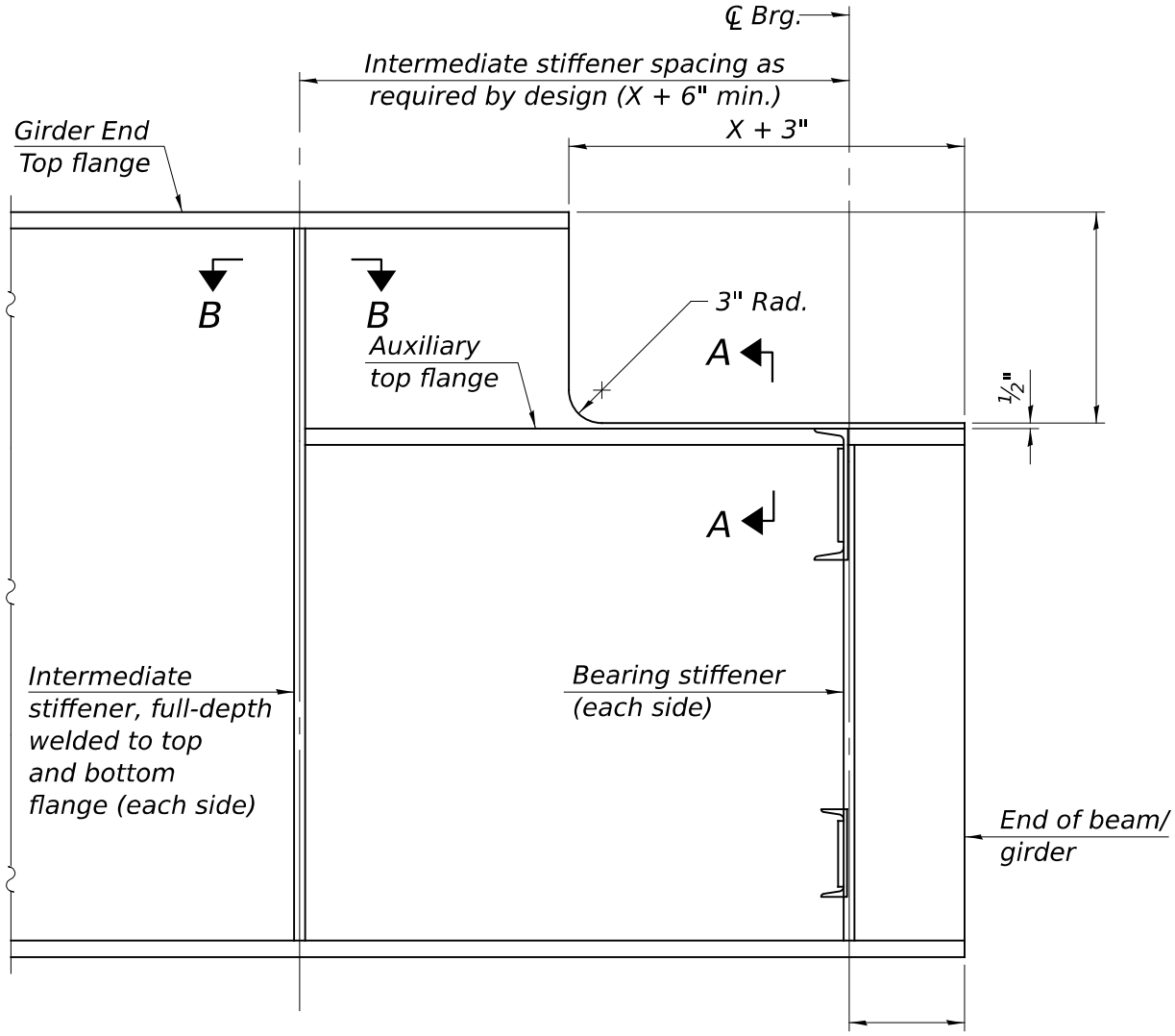
Location	Bar size	No. threaded bars req'd	Length
Bridge deck	#5		
Blockout	#5		

Note:
One bar splicer assembly includes a threaded splicer bar, threaded bar, and threaded coupler.

CONCRETE LEDGE BAR SPLICER ASSEMBLY

Figure 3.6.2.3-13

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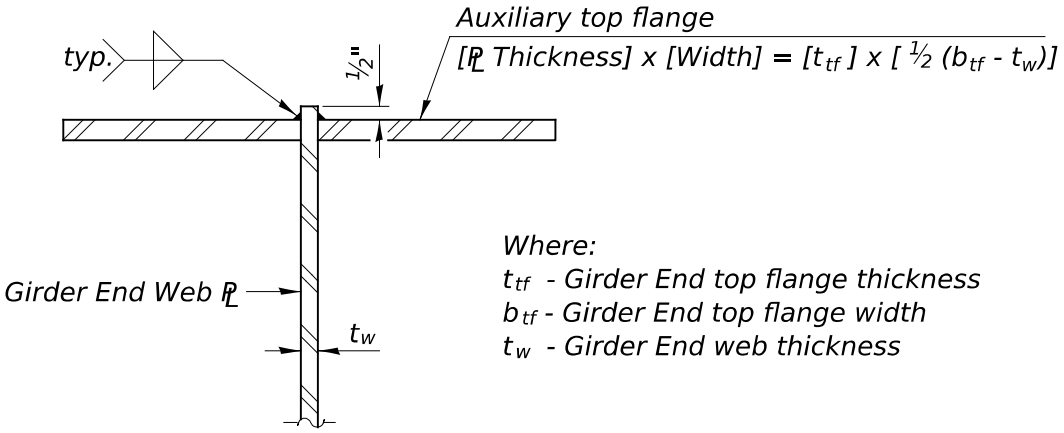
TYPICAL GIRDER END

Note:
For Section A-A and Section B-B, see Fig. 3.6.2.3-15.

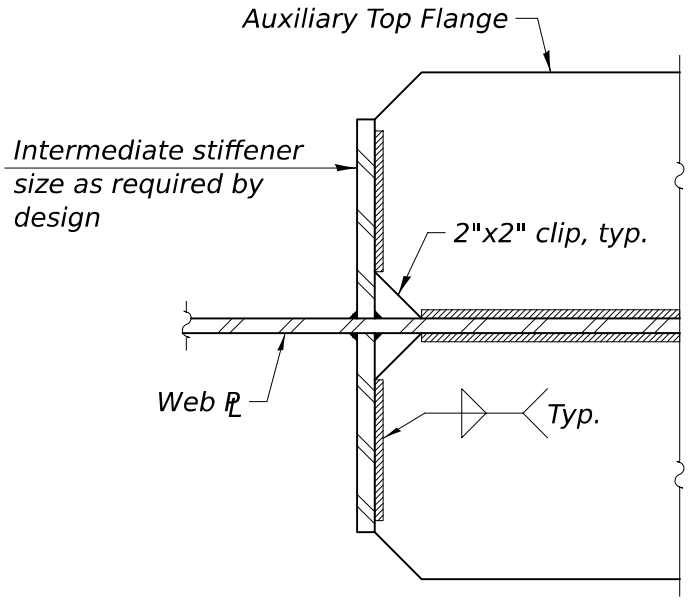
MODULAR JOINT AT STEEL GIRDERS - END ELEVATION

Figure 3.6.2.3-14

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SECTION A-A



SECTION B-B

MODULAR JOINT AT STEEL GIRDER - END SECTIONS

Figure 3.6.2.3-15

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3.6.3 Fixed Joints

A strip seal for a fixed joint is detailed the same as presented in Figures 3.6.2.1-1 to 3.6.2.1-6 with the exception that the minimum joint opening shall be set at 1 ½ in. at all temperatures for installation purposes as noted on the appropriate figures. There is no upper skew limitation for fixed joints, however it shall be placed along the skew out-to-out of the deck.

Base Sheet EJ-SS also depicts the fixed strip seal joint.

3.6.4 Substructure Treatments

The substructure elements under expansion joints shall be protected by an application of Concrete Sealer as an additional protective treatment for the epoxy coated reinforcement. See Sections 3.8 and 3.9 for more information.

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3.7 Bearings

This section covers the Department's policies related to structural bearings to accommodate loads and demands specified by the AASHTO LRFD Bridge Design Specifications. Assemblies and detailing that meet the material and pre-qualification requirements of the Department comprise the standard basis for design.

3.7.1 Bearing Assemblies

3.7.1.1 Elastomeric Bearings

Standard Elastomeric Expansion Bearing Assemblies for conventional structures are detailed in Section 3.7.4. These bearings shall be utilized for all new designs that are within the parameters outlined in Section 3.7.4. The maximum elastomeric bearing size detailed in Section 3.7.4 is 20 x 24, which is based on the Department's capacity for testing. Larger elastomeric bearings may be permitted on certain projects provided the cost of testing the bearing by outside agencies is considered, larger substructures are not required to accommodate the larger bearing footprint, and approval is obtained from the Bureau of Bridges & Structures. Typically, it is more economical to switch to an HLMR bearing if capacities larger than the 20x24 elastomeric bearing are required.

3.7.1.2 Fixed Bearings

The standard fixed bearing used in conjunction with standard elastomeric bearings is the low profile fixed bearing detailed in Figure 3.7.1.2-1. Additional requirements for steel beams on integral abutments and the standard fixed bearing for PPC beams are detailed in Figures 3.7.1.2-2 and 3.7.1.2-3, respectively.

Thickness of Plates: Bearing plate thicknesses and connecting weld sizes for low profile standard fixed (non-integral abutment) bearings shall be determined by use of the formulas given in Figure 3.7.1.2-4. The thickness of sole plates on girders with bearing stiffeners shall be 80% of the thickness required if no stiffeners were present. The design loadings shall be as detailed in Section 3.7.4.

On steel beams and girders, sole plates shall be beveled if the beam grade is ¼ in. in 12 in. or greater.

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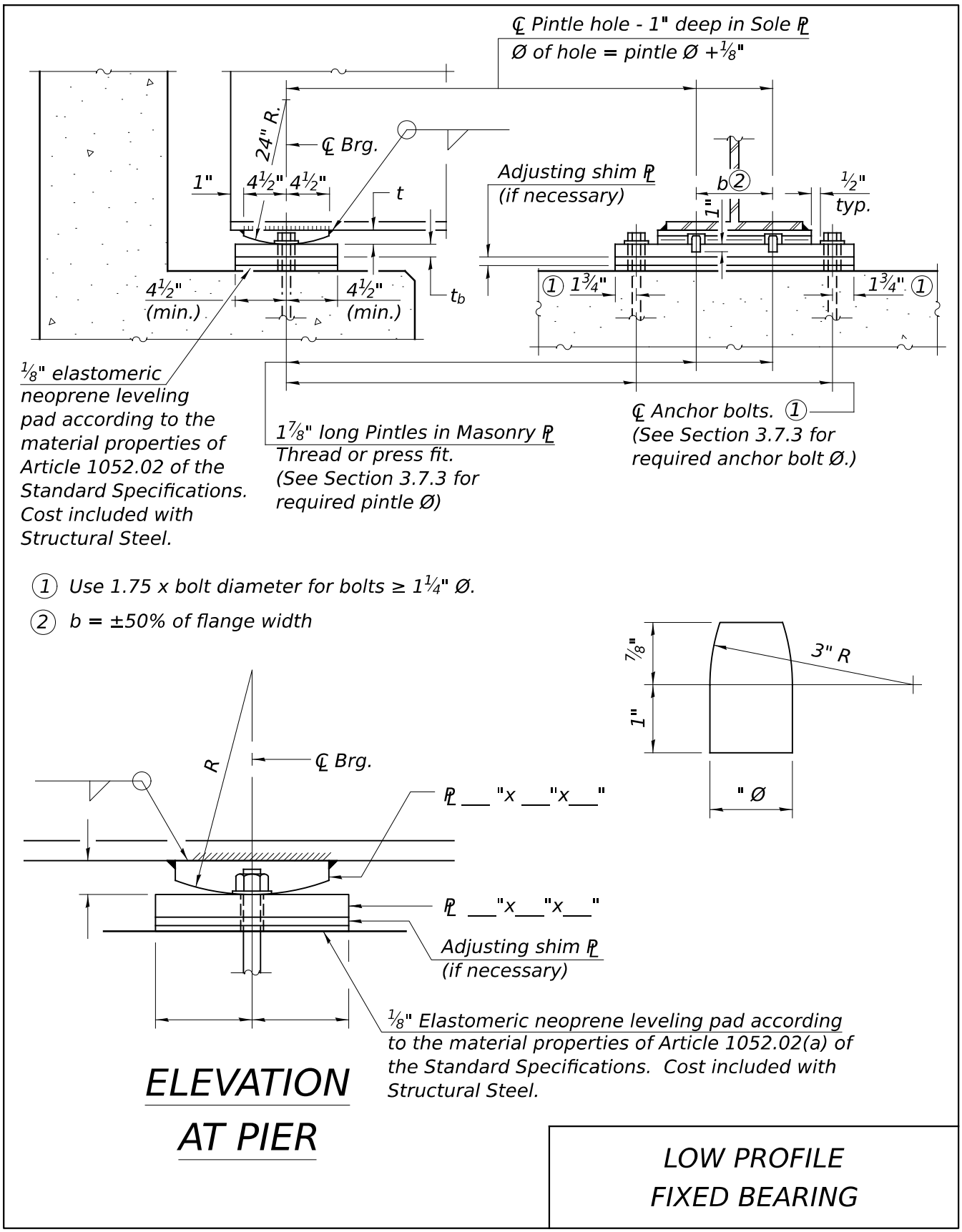


Figure 3.7.1.2-1

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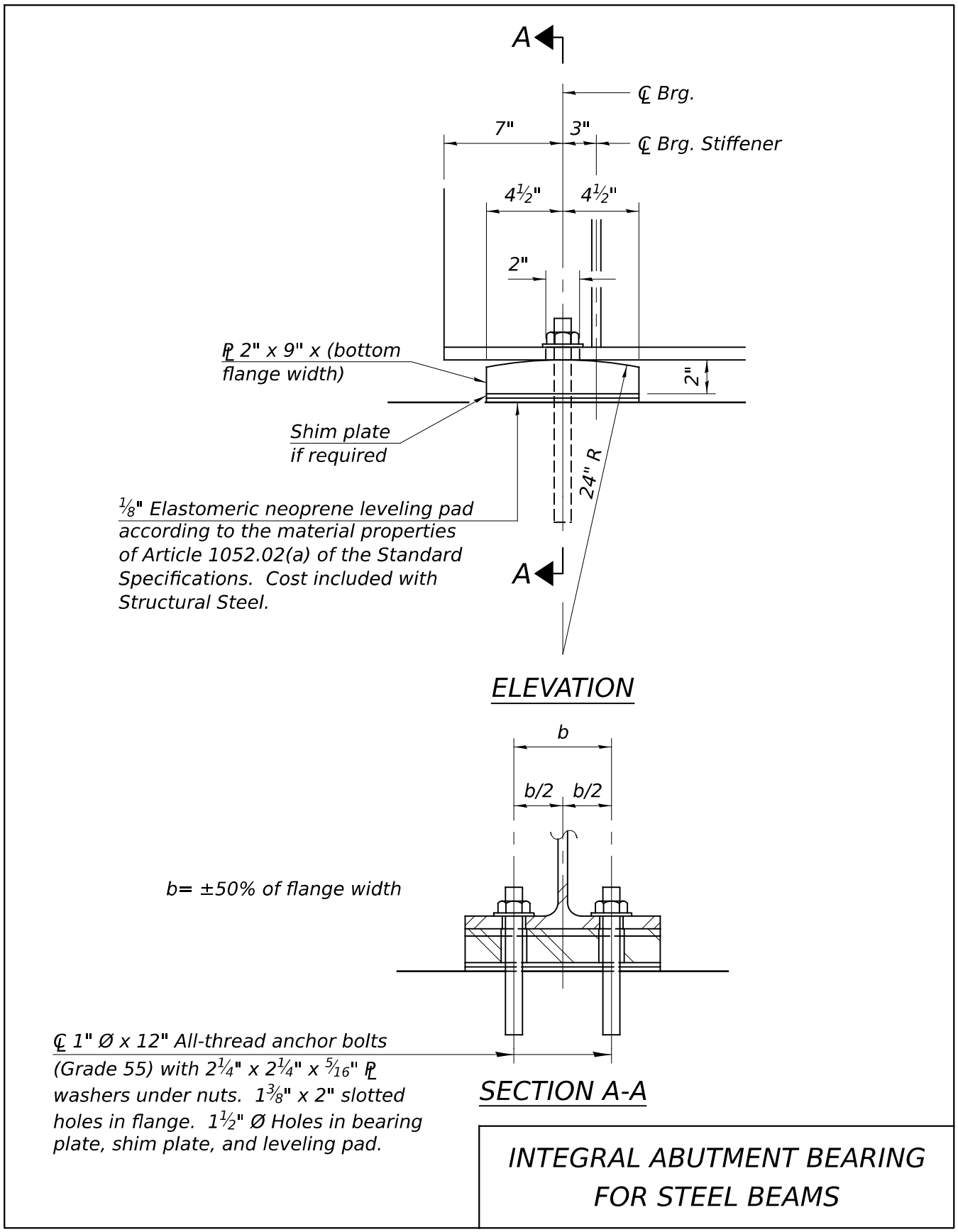
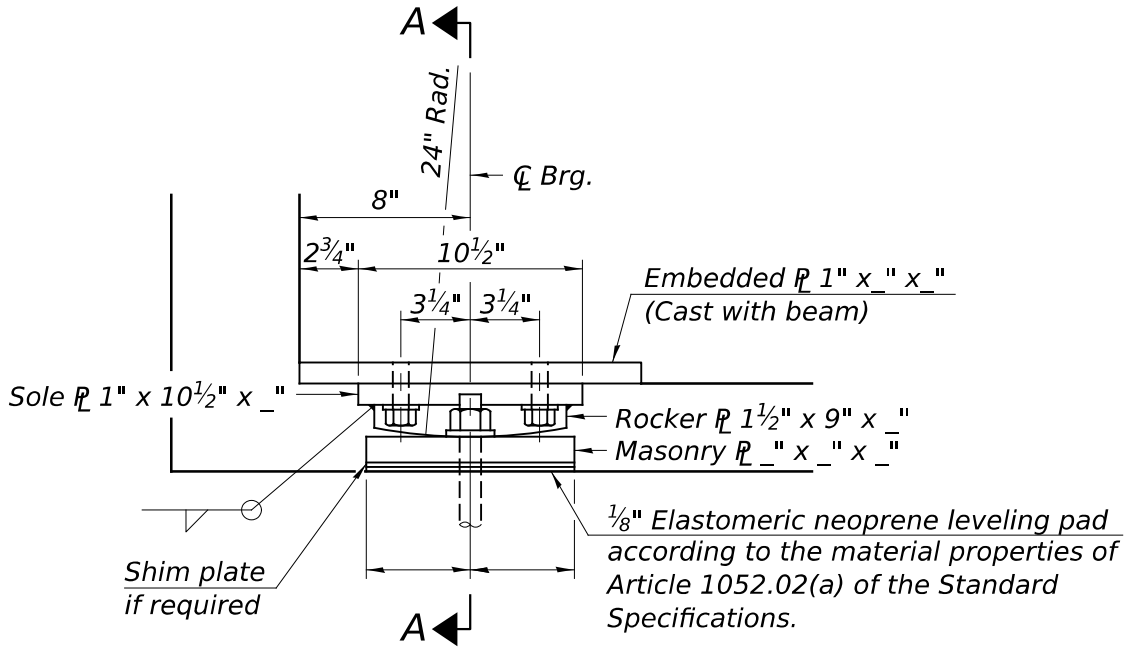
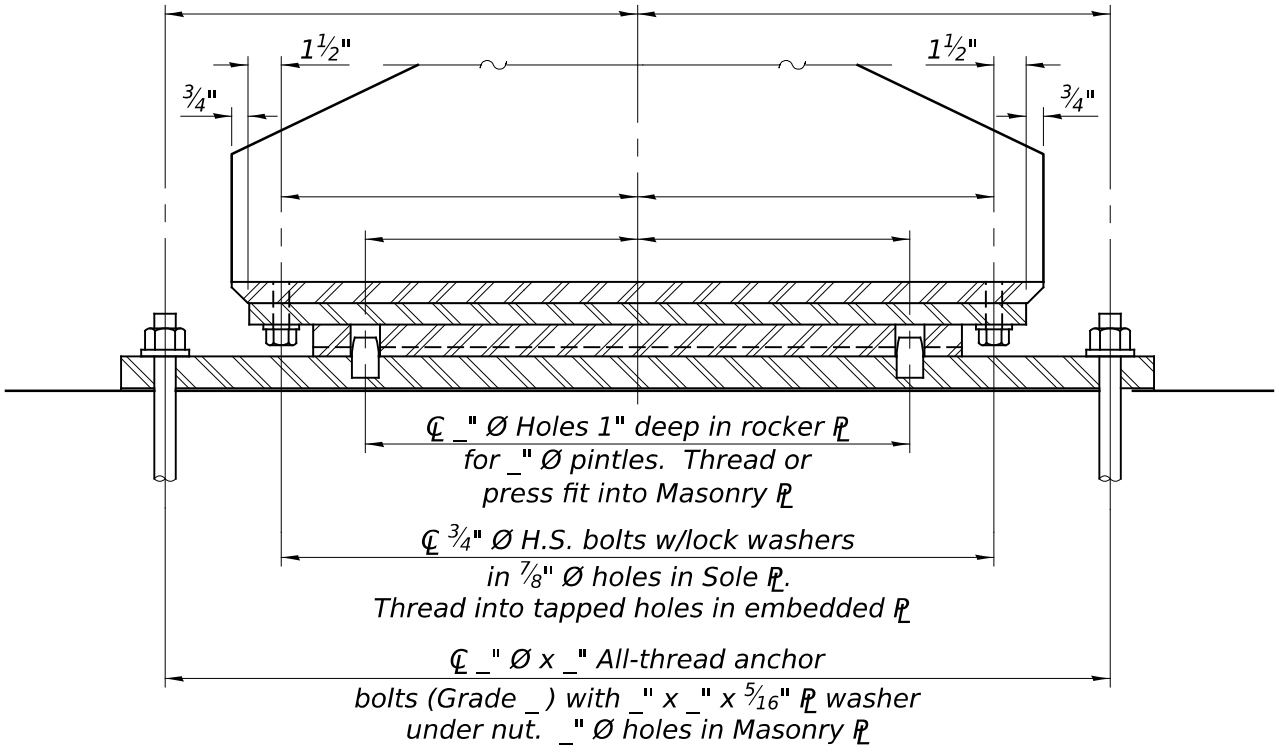


Figure 3.7.1.2-2

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ELEVATION AT ABUTMENT

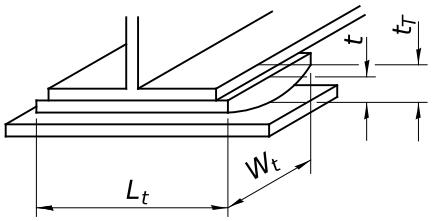


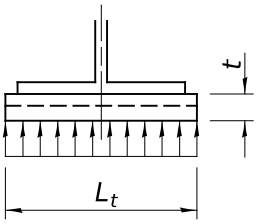
SECTION A-A

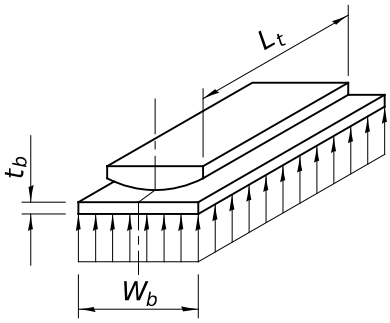
FIXED BEARING DETAILS
FOR PPC BEAMS

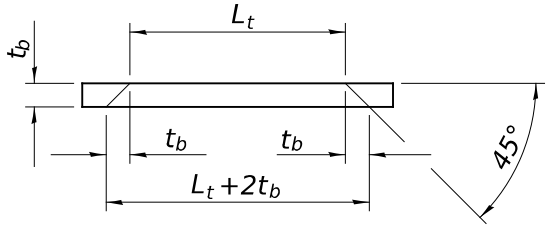
Figure 3.7.1.2-3

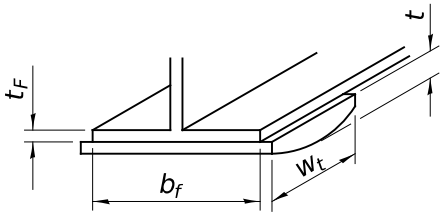
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Final weld size shall be 5/16" or greater.

*** With bearing stiffeners use ½" weld.*

R = Reaction in kips (Strength I)
Include impact for top plate only.
C = 0.122 for f_y = 50 ksi
$L_t \geq$ Bottom flange width + 2(weld size) + ½"

SOLE PLATES

$$t_T = C\sqrt{\frac{R\ L_t}{W_t}}$$

$t = t_T$ - flange thickness
Min. t = 1"

** With bearing stiffeners, use 80% t.*

MASONRY PLATES

$$W_b = \frac{R}{f_m(L_t+2t_b)} \geq 9''$$

$$t_b = C\sqrt{\frac{RW_b}{(L_t+2t_b)}}$$

$f_m = \emptyset_b 0.85\ f'c$ *(ksi)*

f_m may be increased by a modification factor: $\sqrt{\frac{A_2}{A_1}} \leq 2$

See 5.6.5 of the AASHTO LRFD Bridge Design Specifications

SOLE WELD DESIGN

** (No Bearing Stiffeners)*
S_w = Shear per inch of weld (K/in)

$$S_w = \frac{3\ R\ t_F t\ b_f}{4(t_F+t)^3\ (b_f+W_t)}$$

or

$$S_w = \frac{max\ (R_{DLE}\ /\ 5,\ R_T)}{(2b_f+2W_t)}$$

Whichever is larger

R_{DLE} = Unfactored superstructure dead load reaction in kips of the given bearing under consideration (extreme event I).

R_T = Unfactored wind, thermal, and braking reaction in kips (Strength I).

NON-ELASTOMERIC BEARING PLATE DESIGN

Figure 3.7.1.2-4

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3.7.1.3 High Load Multi-Rotational (HLMR) Bearings – Pot and Disc

High Load Multi-Rotational (HLMR) bearings such as pot and disc are better suited for bearing designs beyond the limitations depicted in the charts of Section 3.7.4. Structures requiring bearing sizes larger than standard IDOT elastomeric bearings shall utilize HLMR bearings. They are typically necessary for long span or unique structures. There are a variety of HLMR bearing designs that may or may not conform to the Department's specifications and design requirements. See the Department's website for the Prequalified Structural System List for High Load Multi-Rotational (HLMR) bearings. The Bureau of Bridges & Structures should be consulted to verify the acceptability of any HLMR bearing recommended for use. Section 3.7.5 provides designers with guidance on proper contract plan details and specific design criteria for HLMR bearings. A Special Provision (GBSP 13) provides information for both pot and disc bearings.

If HLMR bearings are used in conjunction with PPC I-beams, Bulb T-Beams, or IL-Beams, the designer shall evaluate the beam splitting plate dimensions to determine that they are large enough such that the HLMR bearing can be adequately connected to the splitting plate. These bearings shall be connected to an embedded plate similar to the fixed bearing shown on base sheet PI-2FB.

3.7.1.4 Bearings for Prestressed Beam Bridges

Most elastomeric and steel bearings for prestressed concrete I-beams are detailed on the base sheets. The design aids and procedures presented in Section 3.7.4, may be used to size elastomeric bearings for prestressed concrete I-beams.

3.7.1.5 Seismic Isolation Bearings

IDOT has selectively employed seismic isolation bearings in the southern part of Illinois where earthquake loadings are a concern. For some bridges, either new or retrofitted, seismic isolation bearings may be a viable alternative depending on where they are located in Southern Illinois, the soil type at the bridge site, and other factors. The Bureau of Bridges & Structures shall be consulted to verify if seismic isolation bearings are acceptable and/or warranted for a particular bridge. If isolation bearings are approved by the BBS, the "AASHTO Guide Specifications for LRFD Seismic Bridge Design" should be utilized.

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3.7.1.6 Uncommon or Historical Bearing Types

Details of bearings no longer commonly used (previously given in the 2003 Edition of the Bridge Manual in Section 3.10) may be obtained by request from the Bureau of Bridges & Structures. These bearings may be used in special situations and for rehabilitation projects where in-kind replacement or matching of existing bearings is necessitated.

3.7.2 Anchor Bolt Details

Anchor bolts shall be minimum ¾" diameter. Typical details are shown below in Table 3.7.2-1.

ANCHOR BOLT DETAILS		
Bolt Dia. X Minimum Length (in.)	**Plate Washer (in.)**	**Bearing Plate Hole Dia. (in.)**
$^{3}/_{4}$ x 12	2 x 2 x $^{5}/_{16}$	1 $^{1}/_{4}$
1 x 12	2 $^{1}/_{4}$ x 2 $^{1}/_{4}$ x $^{5}/_{16}$	1 $^{1}/_{2}$
1 $^{1}/_{4}$ x 15	2 $^{3}/_{4}$ x 2 $^{3}/_{4}$ x $^{5}/_{16}$	1 $^{3}/_{4}$
1 $^{1}/_{2}$ x 18	3 x 3 x $^{5}/_{16}$	2
2 x 24	3 $^{1}/_{2}$ x 3 $^{1}/_{2}$ x $^{5}/_{16}$	2 $^{1}/_{2}$
2 $^{1}/_{2}$ x 30	4 x 4 x $^{5}/_{16}$	3

Table 3.7.2-1

3.7.3 Seismic Requirements

3.7.3.1 Seismic Requirements for Non-Deck Beam Bridges

The connection of the superstructure to substructure for bridges, not including deck beam bridges, shall be designed to withstand horizontal forces equal to 20% of the dead load reaction of the superstructure, regardless of the Seismic Performance Zone, earthquake return period, or seismic design accelerations. This connection design is consistent with IDOT's Earthquake Resisting Strategy. See Article 3.15 for more information.

Given below are equations and tables which provide guidelines for meeting the nominal connection requirements described above.

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A simple method for nominally designing the anchor bolts is to consider shear as the only failure mode. For this approach, the number of anchor bolts required along each beam line is given by the following equation.

$$N = \frac{C_{il} \times (DL)}{F}$$

Where:

N = number of anchor bolts required for the given bearing under consideration

DL = superstructure dead load at the given bearing under consideration (service or factored extreme event elastic seismic force)

C_{il} = 0.2 (20%) for SPZ 1 to 4 (LRFD)

F = $\phi 0.5 A_b F_u N_s$ (AASHTO 6.13.2.12-1) where ϕ = 0.75 (according to AASHTO 6.5.4.2) and N_s = 1. These values are provided in Table 3.7.3-1.

To ensure that the shear failure occurs at the anchor bolt level in a seismic event, the bolted connection between the top bearing plate and the bottom flange shall be designed such that the capacity exceeds the capacity of one anchor bolt in the connection. This typically is not a controlling item in design.

Allowable Anchor Bolt Shear			
Bolt Diameter (in.)	Bolt Area, A_b ($in.^2$)	F(kips)	F(kips)
		F1554 Gr. 55 (F_u = 75 ksi)	F1554 Gr. 105 (F_u = 125 ksi)
3/4	0.44	12.4	20.7
1	0.79	22.1	36.8
1 1/4	1.23	34.5	57.5
1 1/2	1.77	49.7	82.8
2	3.14	88.4	147.3
2 1/2	4.91	138.1	230.1

Table 3.7.3-1

When feasible, the maximum size of anchor bolts should be limited to not greater than 1 ½ in. diameter.

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Side retainers shall be provided on both sides of all elastomeric bearings as shown in Figures 3.7.4-13 through 3.7.4-19.

The number and size of pintles required at fixed bearings shall be designed for the same horizontal forces as the anchor bolts provided for at that bearing. The allowable pintle loads given in Table 3.7.3-2 are based upon nominal strengths (ϕ = 1.0). To ensure that the shear failure occurs at the anchor bolt level in a seismic event, the pintle group design capacity should be verified to be larger than the sum of the anchor bolt capacities. If resistance factors of ϕ = 0.75 and ϕ = 1.0 are used for anchor bolts and pintles, respectively, this should not be a concern. The number of pintles which are required along each beam line is given by the following equation.

$$N_p = \frac{C_{il} \times (DL)}{F_p}$$

Where:

N_p = number of pintles required for the given bearing under consideration

DL = unfactored superstructure dead load reaction at the given bearing under consideration

C_{il} = 0.2 (20%) for SPZ 1 to 4 (LRFD)

F_p = $\phi 0.56 A_b F_u N_s$ (AASHTO 6.13.2.7-1)where ϕ = 0.75 (according to AASHTO 6.5.4.2) and N_s = 1. These values are provided in Table 3.7.3-2.

Allowable Pintle Shear			
Pintle Diameter (in.)	Pintle Area, A_b ($in.^2$)	F(kips) M270 Gr. 36 (F_u = 58 ksi)	F(kips) M270 Gr. 50 (F_u = 65 ksi)
1 1/4	1.23	29.9	33.5
1 3/8	1.48	36.2	40.5
1 1/2	1.77	43.0	48.2
1 5/8	2.07	50.5	56.6
1 3/4	2.41	58.6	65.7
1 7/8	2.76	67.3	75.4
2	3.14	76.5	85.8
2 1/8	3.55	86.4	96.8

The hole diameter in the receiving plate for a pintle is typically the pintle diameter + 1/8".

Table 3.7.3-2

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3.7.3.2 Seismic Requirements for Deck Beam Bridges

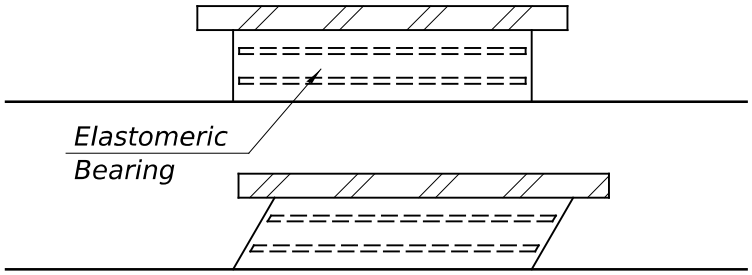
Due to typically small dead load reactions per bearing, designing deck beam bearing connections for 0.2DL often necessitates using dowel rods that are unrealistically small. For this reason, deck beam bearing connections should always utilize a one inch dowel rod, and the connection should not be considered a fuse in seismic design. See Section 3.15 for seat width requirements for deck beam bridges.

3.7.4 Elastomeric Expansion Bearing Assemblies

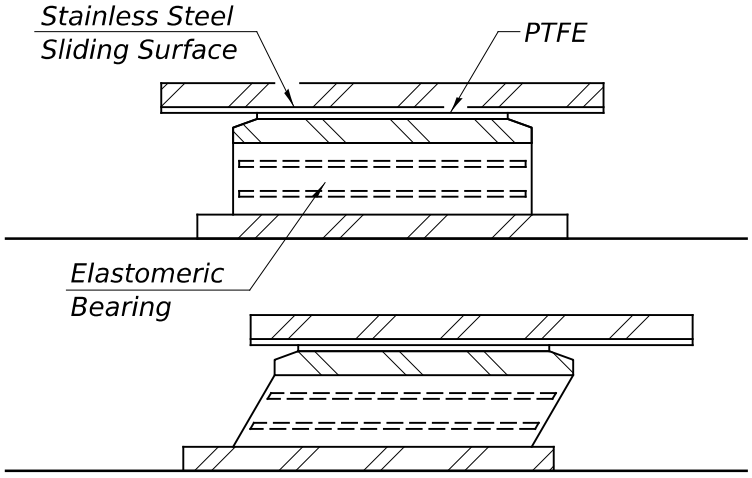
Elastomeric bearing assemblies are divided into three types according to the expansion lengths that they will accommodate. The details of the types are shown in Figure 3.7.4-1.

The Type I assembly is a conventional laminated pad where all of the movement is taken by distortion of the rubber. This type shall be limited to expansion lengths of 109 ft. or less for the 6 in. wide bearing and 379 ft. or less for the 20 in. wide bearing.

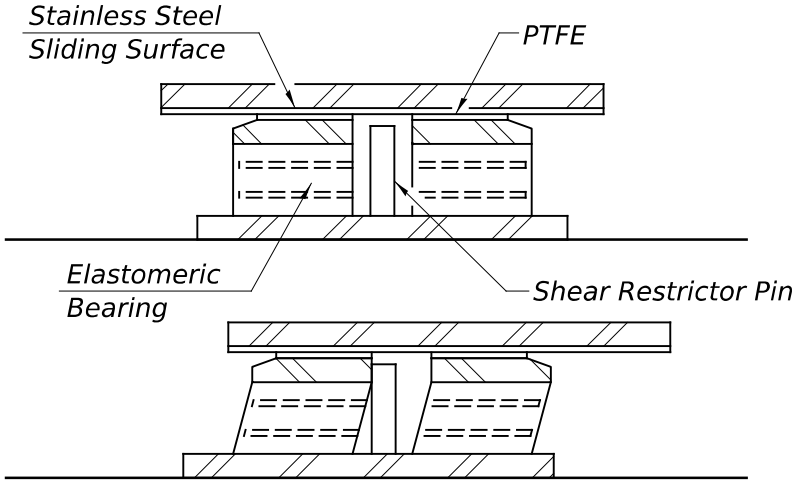
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TYPE I BEARING



TYPE II BEARING



TYPE III BEARING

ELASTOMERIC &
PTFE ELASTOMERIC
EXPANSION BEARINGS

Figure 3.7.4-1

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The Type II assembly has a teflon sliding surface incorporated to provide additional movement capacity. The movement is accomplished by both deformation of the elastomer and slippage on the teflon surface. The elastomer deforms until the internal shear force equals the friction force required to slip the teflon surface. The expansion length limitations of the Type II assembly shall be 218 ft. for the 6 in. wide bearing and 758 ft. for the 20 in. wide bearing.

The Type III bearing was developed to accommodate expansion lengths which exceed the limitations of the Type II bearing. The Type III is essentially a Type II with a shear restrictor pin added to prevent the rubber from overstressing in shear as movement occurs. The movement in excess of that allowed by the shear restrictor pin is accommodated by slippage on the teflon surface.

There is no limitation on the expansion length for which the Type III may be used as long as sufficient travel capability is provided by the shear restrictor pin and the size of the stainless steel plate. The rubber thickness is based on the deformation allowed by the shear restrictor pin and the rotational requirements for nonparallel load surfaces.

Figures 3.7.4-15 through 3.7.4-19 depict standard details for Type I, Type II, and Type III elastomeric expansion bearings.

The design of elastomeric bearings is governed by four basic parameters. These parameters are as follows:

1. Dead Load Reaction.
2. Dead Load plus Live Load Reaction.
    a. Impact not included.
3. Expansion Length.
    a. Distance from point where bridge superstructure is assumed fixed for thermal expansion to expansion bearing.
4. Percent Slope (in the longitudinal bridge direction) due to Nonparallel Surfaces.
    a. Dead load rotation.
    b. Camber of prestressed beams.
    c. Profile grade of beam.

The design criteria are subject to certain limitations. These limitations are in terms of both allowable stresses and minimum dimensions. They are as follows:

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1. The total effective rubber thickness (ERT) of the elastomer shall be a least 2 times the maximum shear deformation at the service limit state, Δ_s, for the Type I bearing. See Section 3.6 for temperature range and linear expansion coefficient.
    a. The total effective rubber thickness is defined as the summation of the individual layers of rubber including the top and bottom layers.
    b. For the Type II bearing, the ERT need only be equal to Δ_s, due to the use of the Teflon and stainless steel sliding surfaces.
    c. For the Type III bearing, the ERT is not directly related to Δ_s provided that the sliding surfaces remain in full contact as shown in Figure 3.7.4-1.
2. The width of the bearing parallel to the direction of movement shall be at least 3 times the total effective rubber thickness with exception to bearings 10-d, 13-e, 14-e, and 15-e in Figures 3.7.4-21 through 3.7.4-23, where width of the bearing as small as 2.85 times the total ERT is allowed.
3. The stress due to dead load shall be between 200 and 500 psi.*
4. The stress due to dead load plus live load without impact shall be between 200 and 800 psi.*
5. Sufficient rubber thickness or a tapered plate shall be provided to avoid a lift-off condition on the leading edges of the pad.

    **The 200 psi minimum requirement is intended for preventing the horizontal crawling of the bearing when it is not attached to the top surface. This requirement has been applied to the bearing designs detailed in this manual even though these bearings are detailed with positive attachment to the flange of the girder. Compliance with the requirement is desirable but is not mandatory if it results in a special bearing design or special superstructure treatments.*

Design aids have been produced which incorporate the design parameters and limitations and are applicable for non-LRFD designed bridges under rehabilitation. LRFD factored service reactions from load case Service I (Article 3.4.1) shall govern the design. The end shear/reaction amplification for skewed bridges (Article 4.6.2.2.3c) is not applicable for interior and exterior beam bearing designs at continuous piers. The factored resistance of structural steel in bearing assemblies shall be calculated as currently stipulated in this Manual.

The following is a step-by-step procedure for using the design tables:

1. The size of bearing required is determined by entering dead load reaction and the dead load plus live load reaction in the tables in Figure 3.7.4-2 or 3.7.4-3. The limits of the Type III bearings are reduced because of the reduction in area due to the holes for the shear restrictor pins.

2. The type of bearing required is determined from the table in Figure 3.7.4-4. If a Type III is required, the table in Figure 3.7.4-3 shall be checked to see that the limits of the Type III are not exceeded.

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3. The thickness of the bearings is determined from Figures 3.7.4-5 through 3.7.4-13. Each bearing type is divided according to plan dimensions and thicknesses. The thicknesses (series) are designated by letters such as a, b, c, etc. which correspond to dimensions given in Figures 3.7.4-21 through 3.7.4-23. For Type I and II bearings, the thickness required for expansion requirements is found in Figures 3.7.4-5 and 3.7.4-6. The thickness required to satisfy slope requirements for Type II and III bearings is given in Figures 3.7.4-7 through 3.7.4-13. For Type I bearings the slope requirements are shown in Figure 3.7.4-23. Both of these thicknesses shall be determined and the maximum used. If a tapered plate is used, the slope becomes zero and series “a” will satisfy slope requirements only. For Type III bearings, the only requirement is to meet the slope requirements. Therefore, only the tables in Figures 3.7.4-7 through 3.7.4-13 are used to determine which series is required. If a tapered plate is used, again the slope becomes zero and a series “a” is required. After the correct series is determined, the detailing dimensions are found in Figures 3.7.4-22 through 3.7.4-25. Additional figures are included to provide detailing aids.

Type I bearings shall be used whenever possible in Seismic Performance Zones 2, 3, and 4, even if doing so requires using a bearing with a footprint larger than the smallest footprint required. If a larger footprint bearing allows Type I bearing to be used versus Type II bearing, the smallest footprint which allows Type I bearing to be utilized shall be chosen.

A force acting in a direction parallel to the movement shall be applied to the substructure at the base of the bearing. This force is due to the uniform temperature force effects due to superimposed deformations, this force transfers from the superstructure to the bearing. The resulting force acting on the substructure is a frictional force (load force FR per AASHTO 3.3.2). This force is due to either the force required to deform the elastomer of the Type I bearing or the friction force required for slippage of the teflon surface on the Types II and III bearings. The magnitude of the force shall be 44 pounds per square inch of bearing area for Type I bearings and 0.07 times the dead load reaction for Types II and III bearings.

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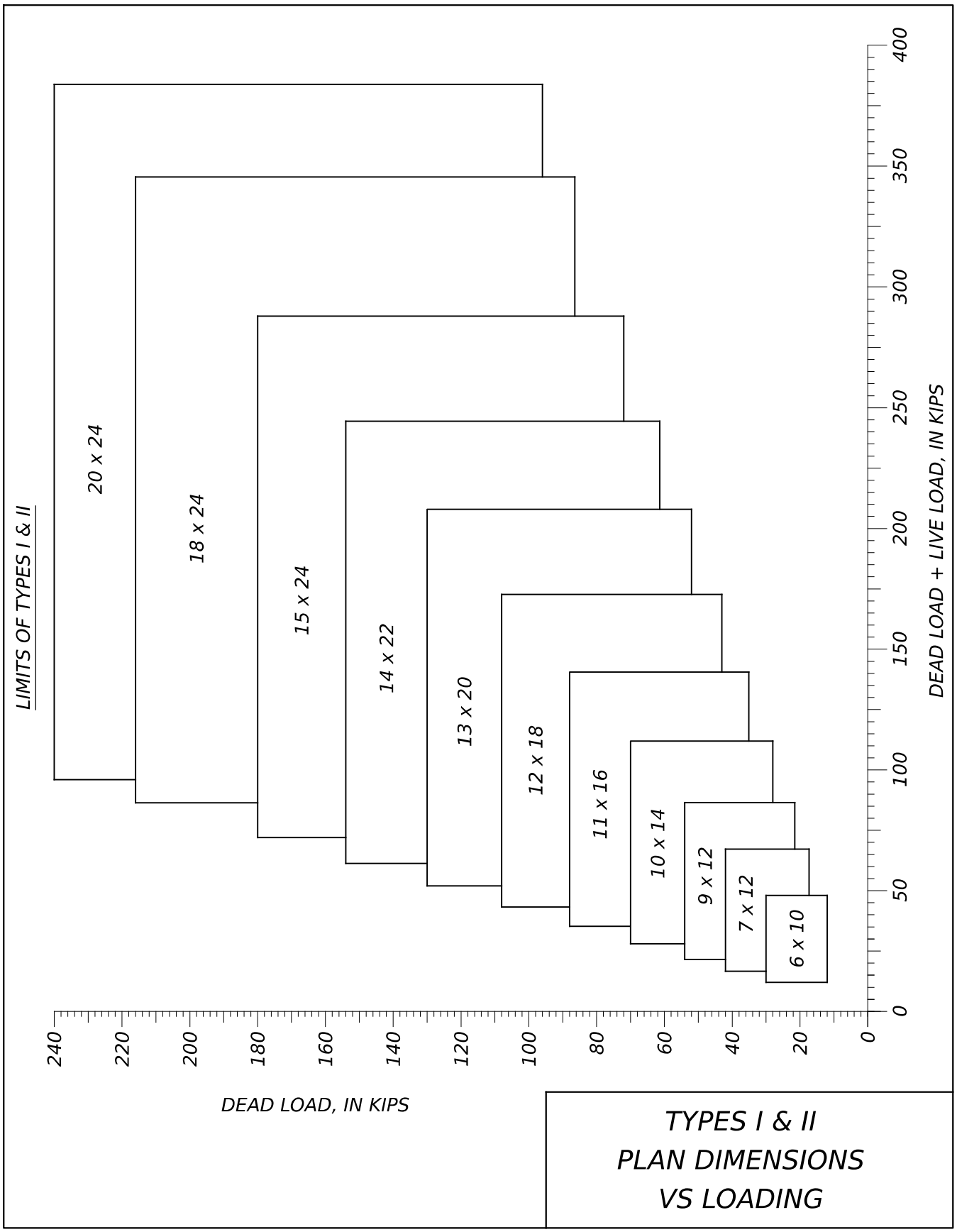


Figure 3.7.4-2

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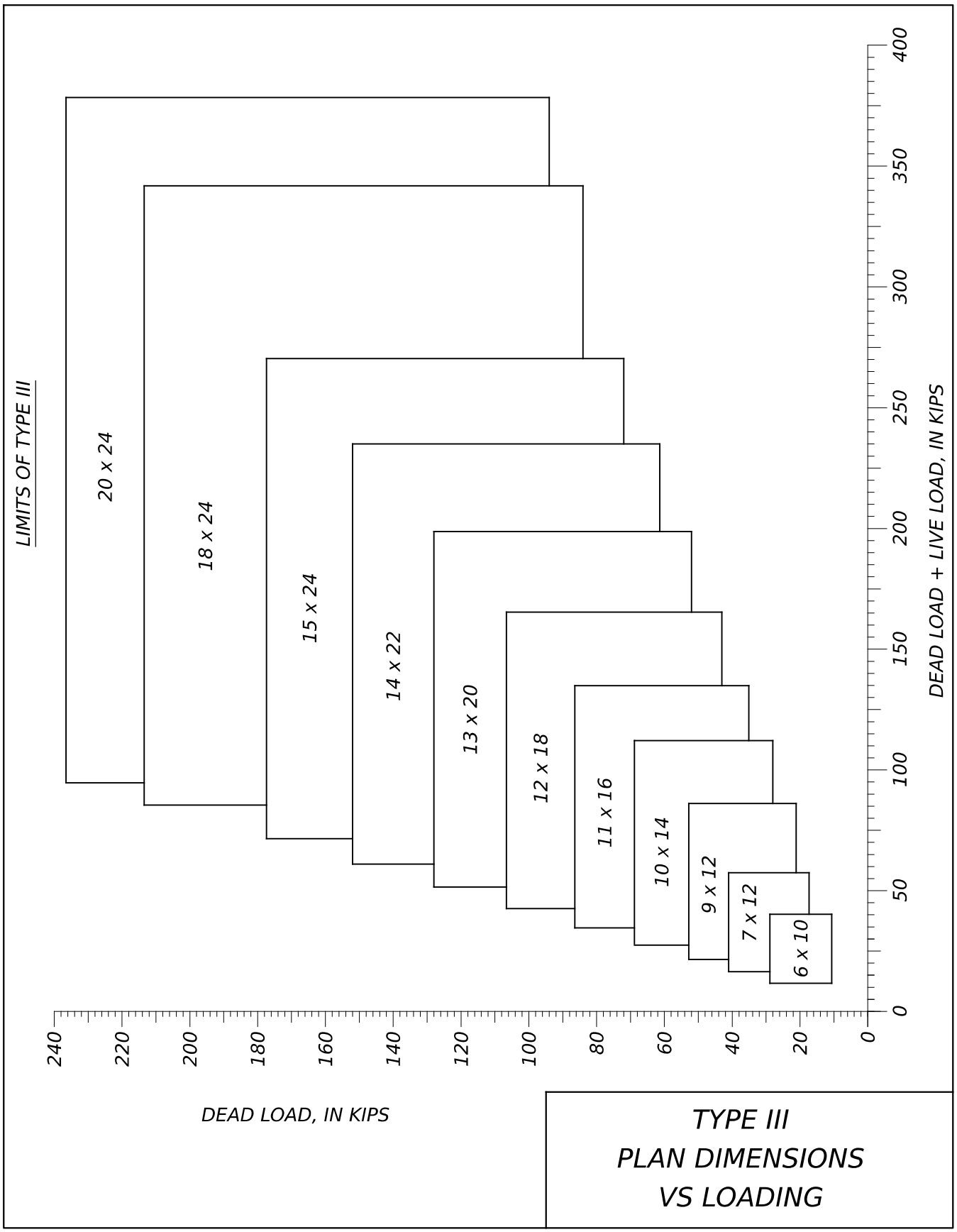


Figure 3.7.4-3

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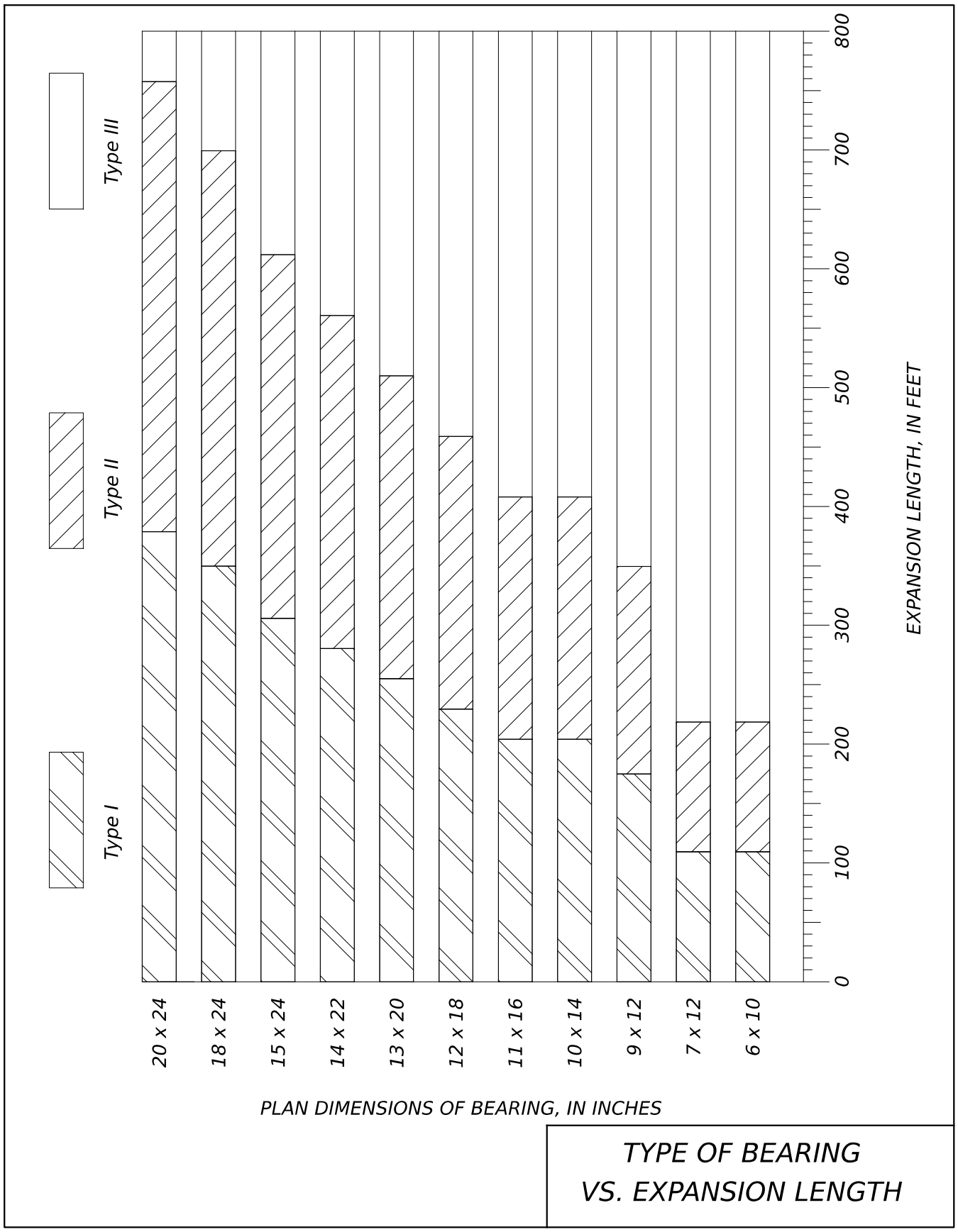


Figure 3.7.4-4

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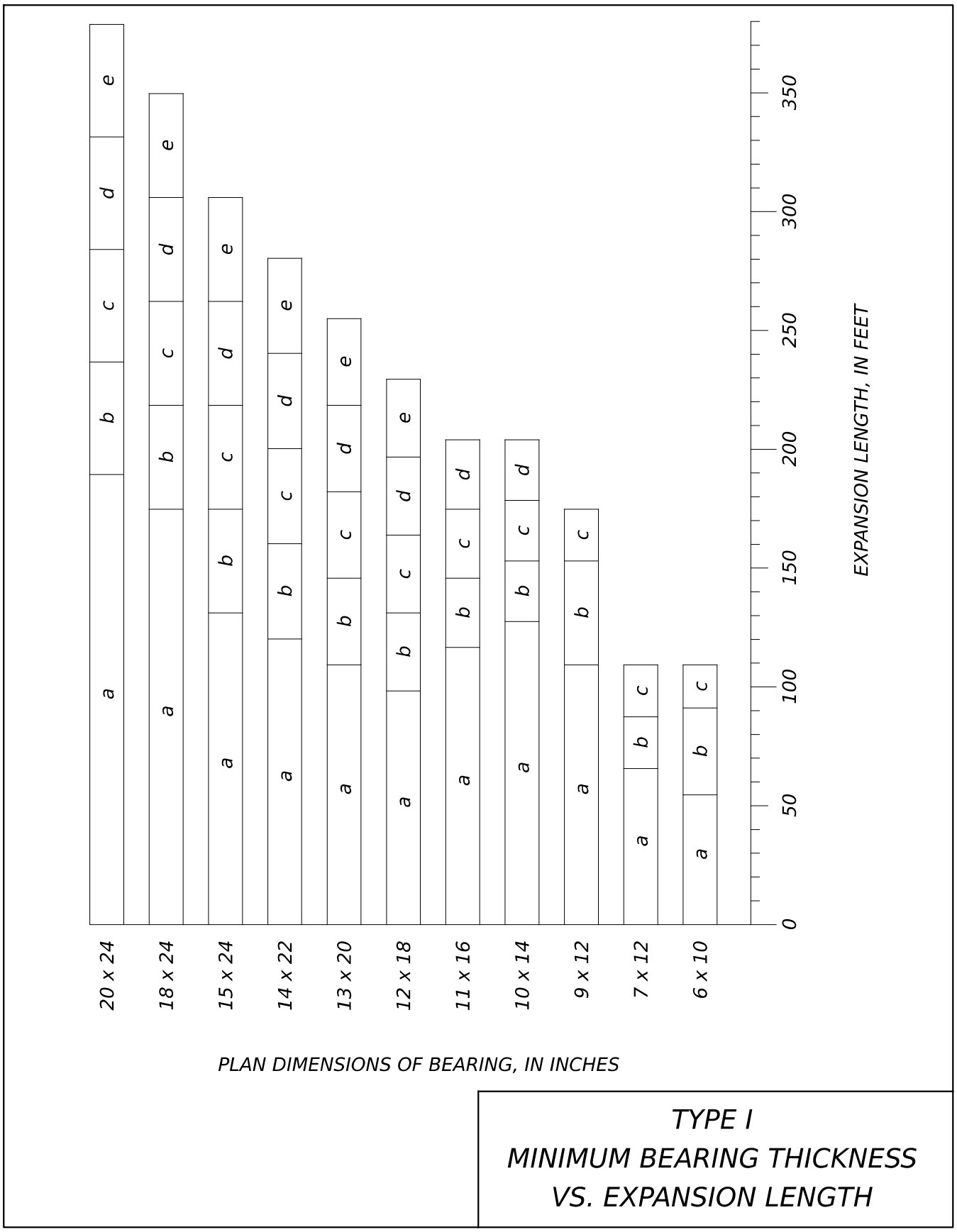


Figure 3.7.4-5

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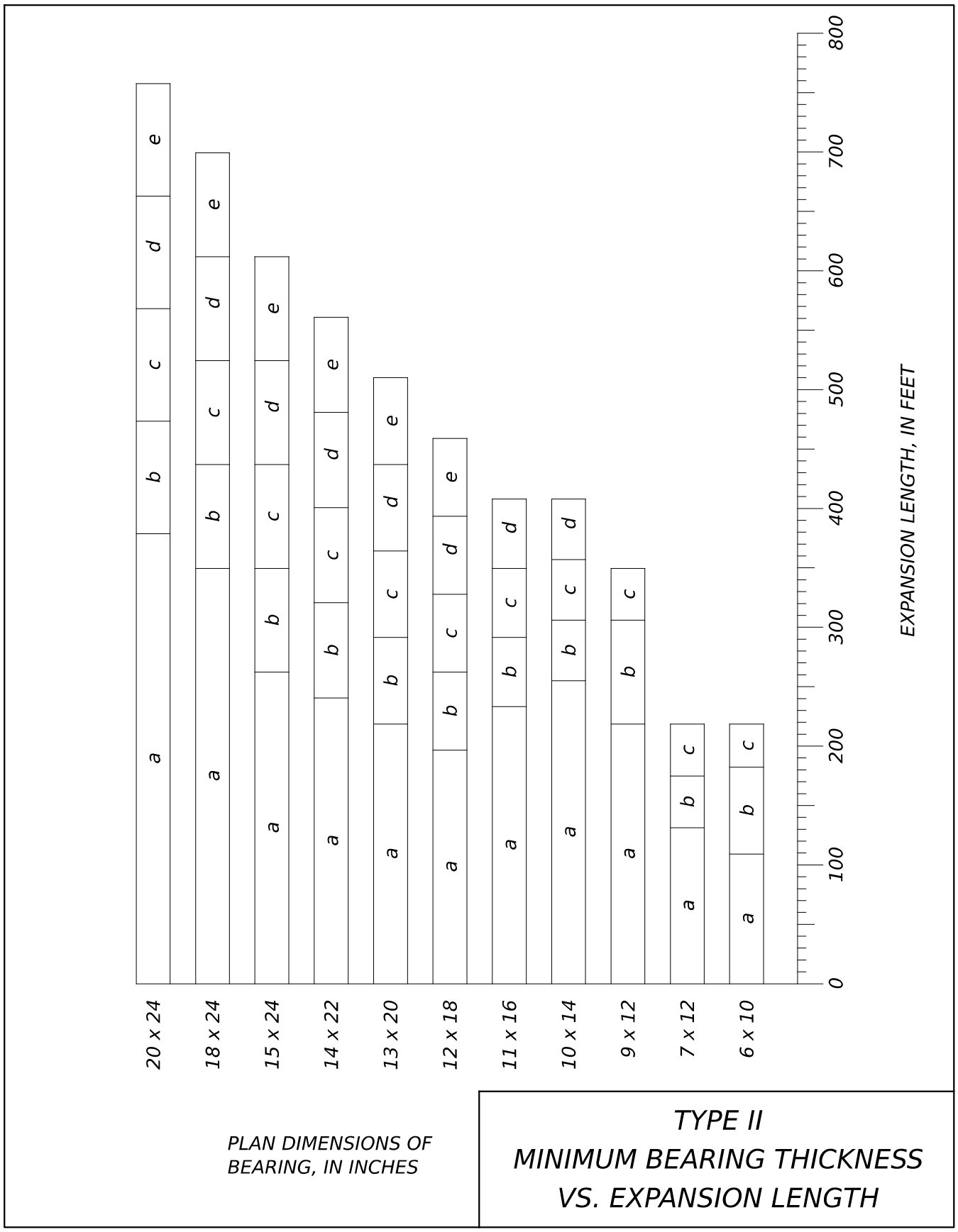


Figure 3.7.4-6

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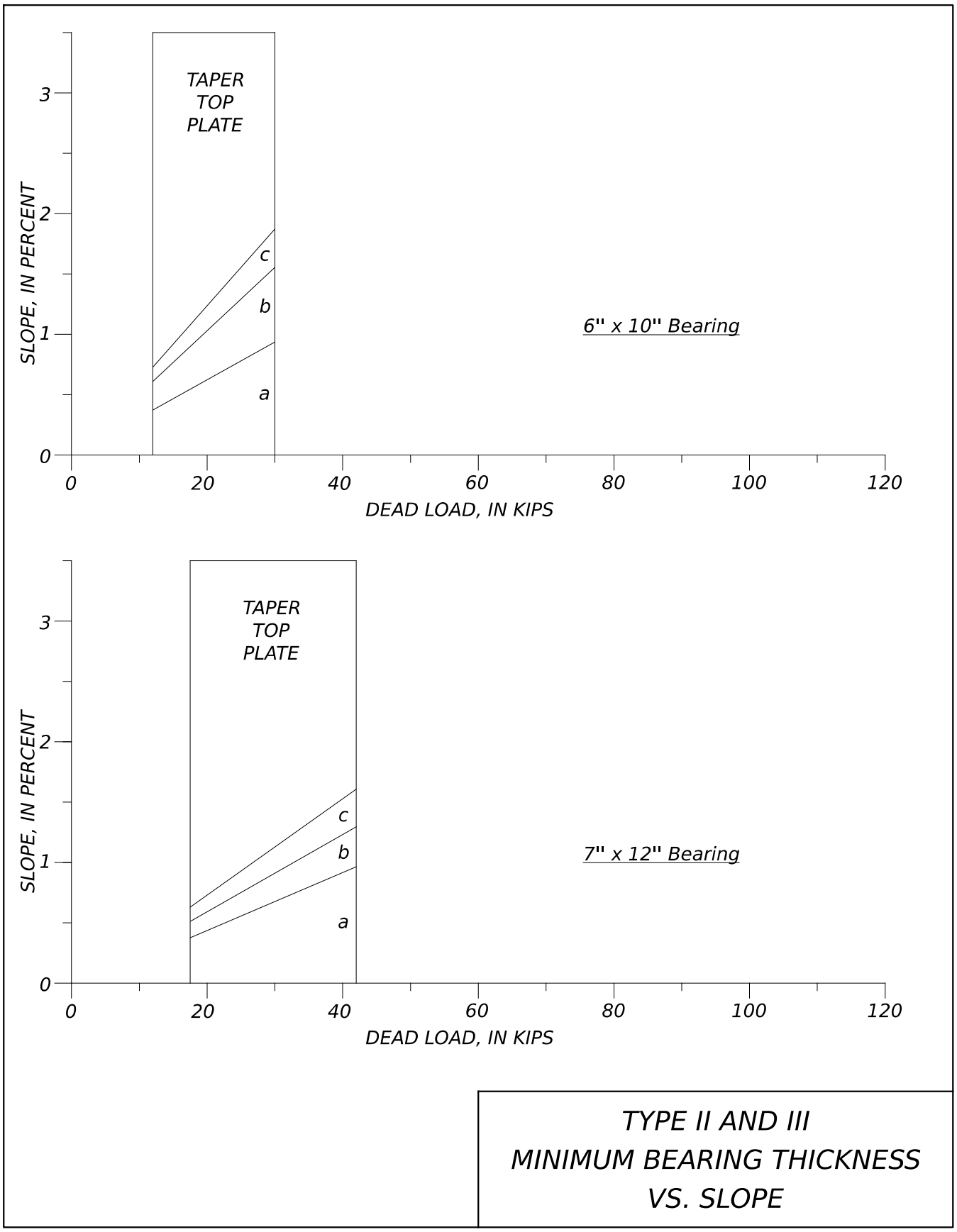


Figure 3.7.4-7

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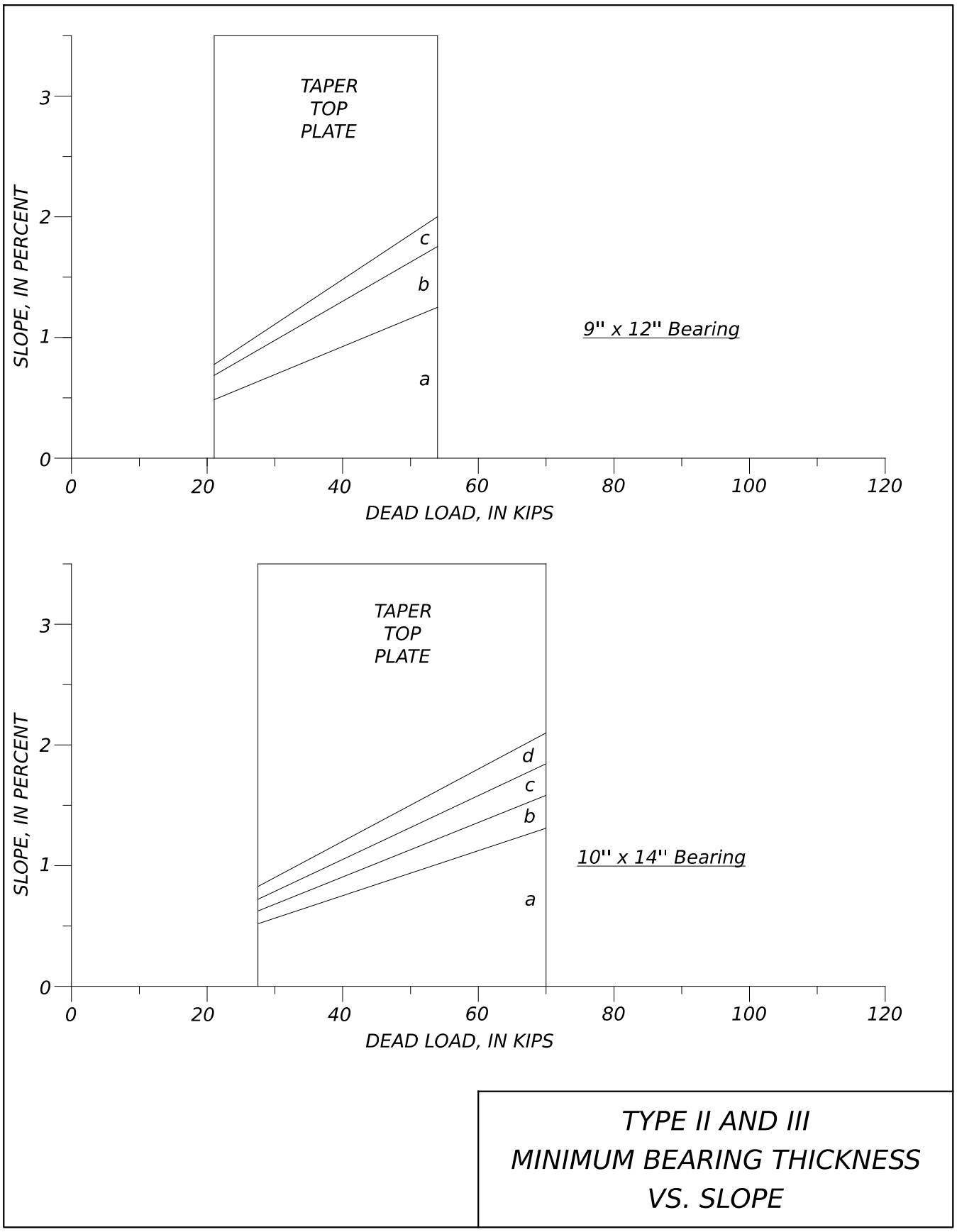


Figure 3.7.4-8

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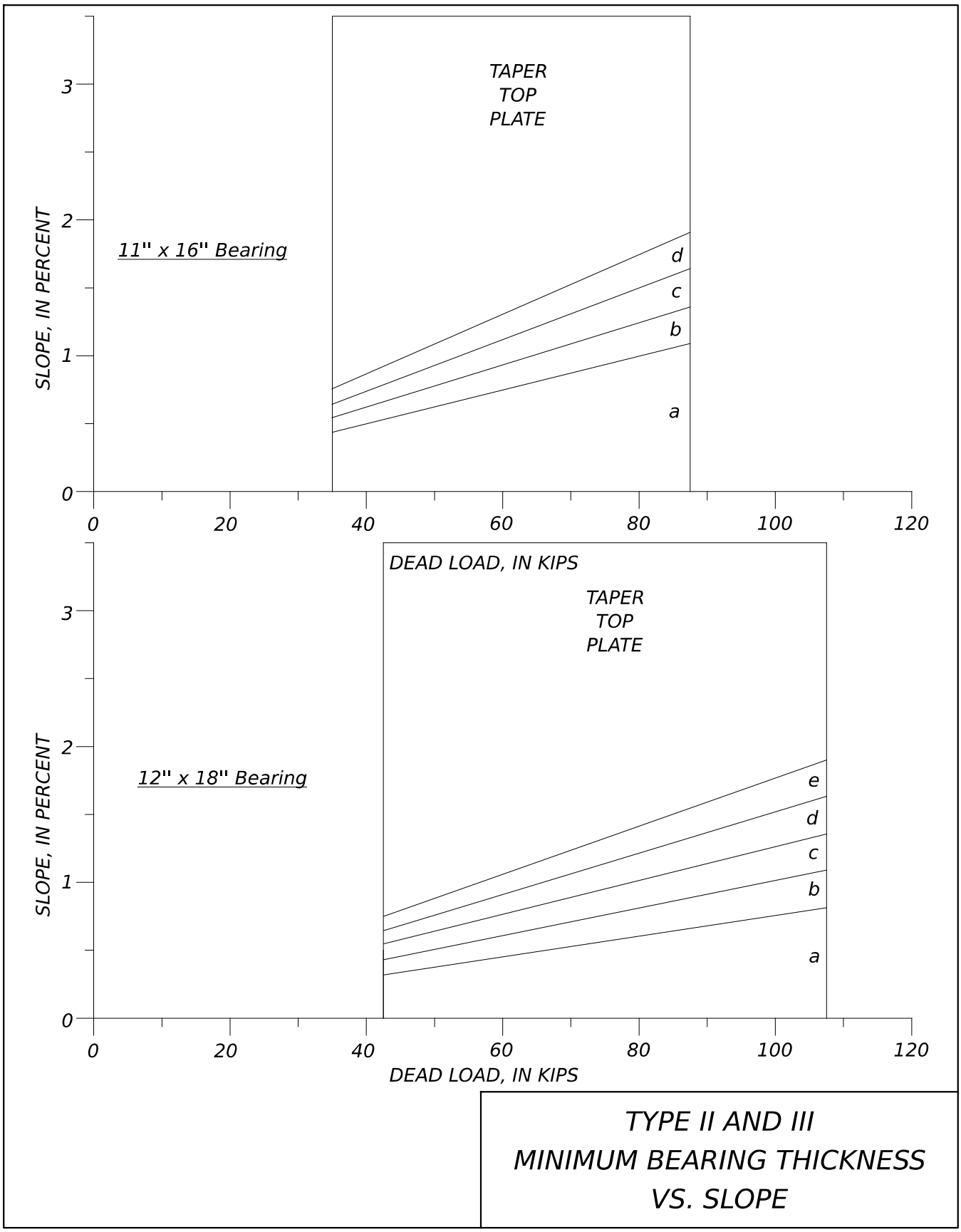


Figure 3.7.4-9

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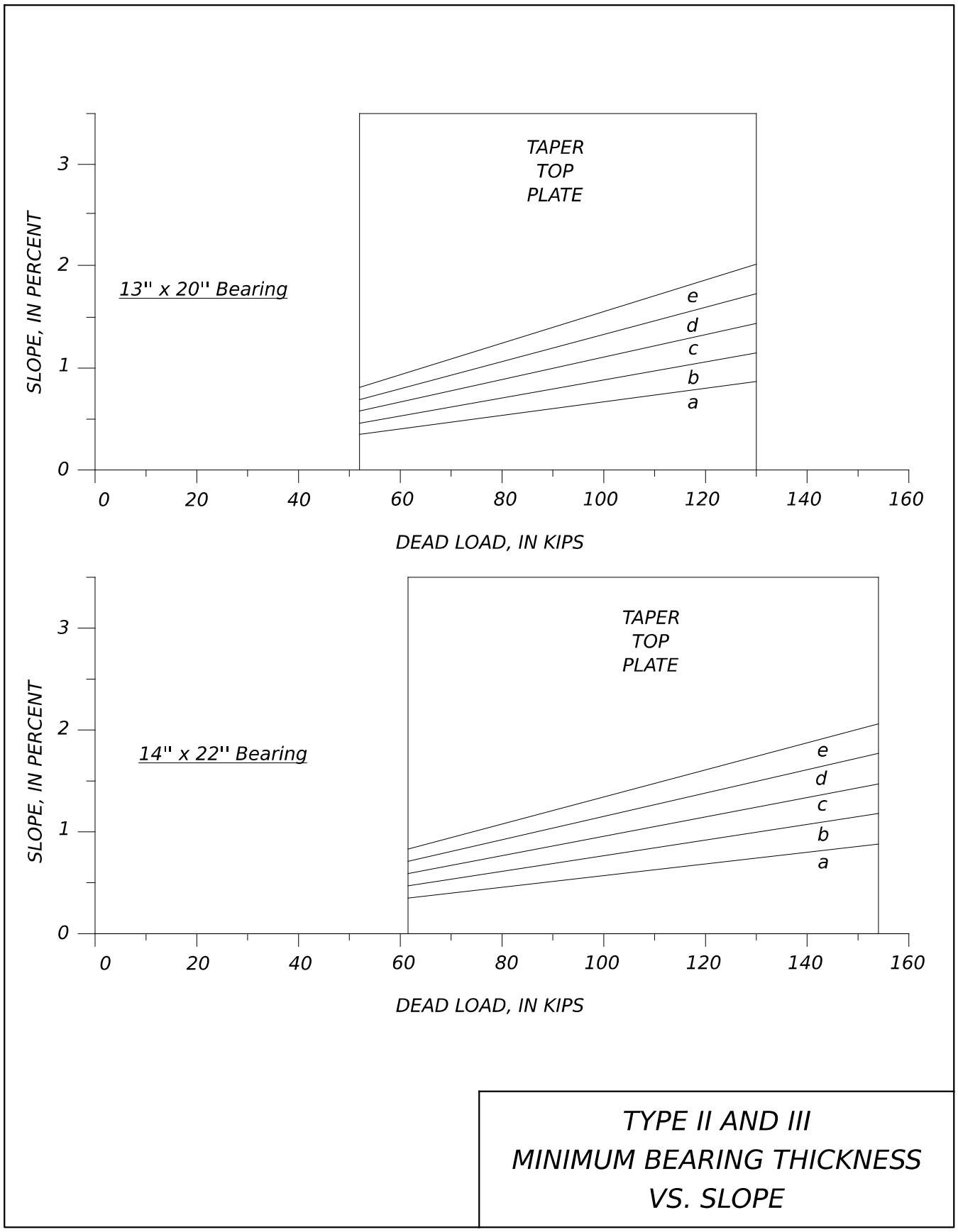
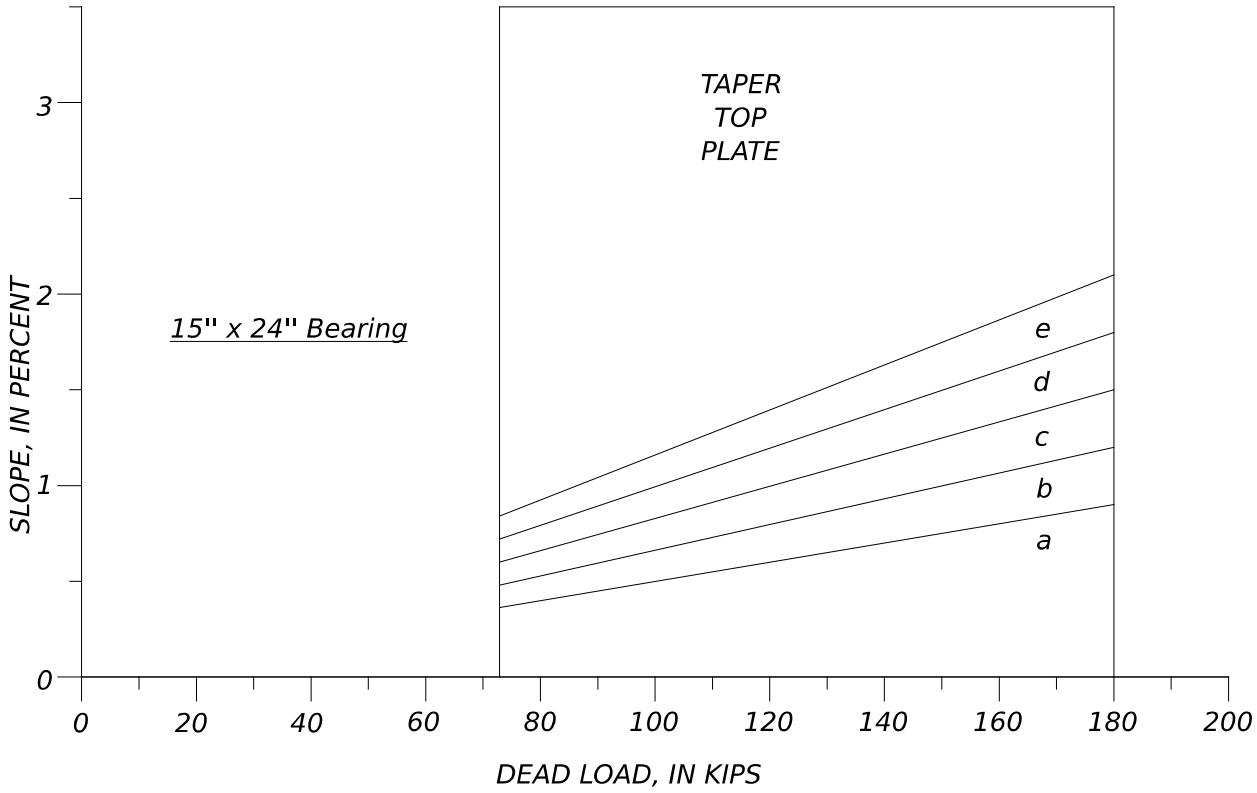


Figure 3.7.4-10

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TYPE II AND III
MINIMUM BEARING THICKNESS
VS. SLOPE

Figure 3.7.4-11

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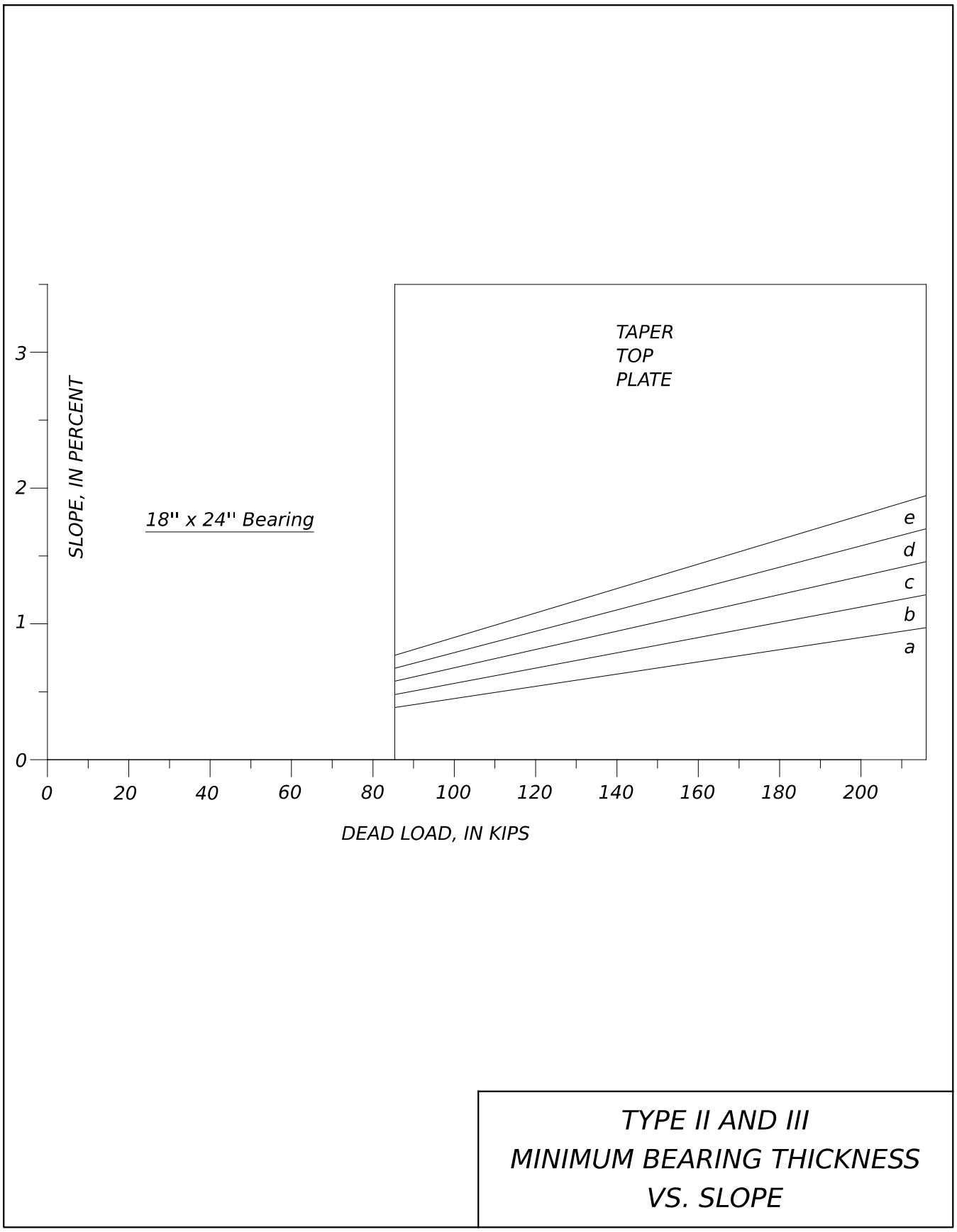
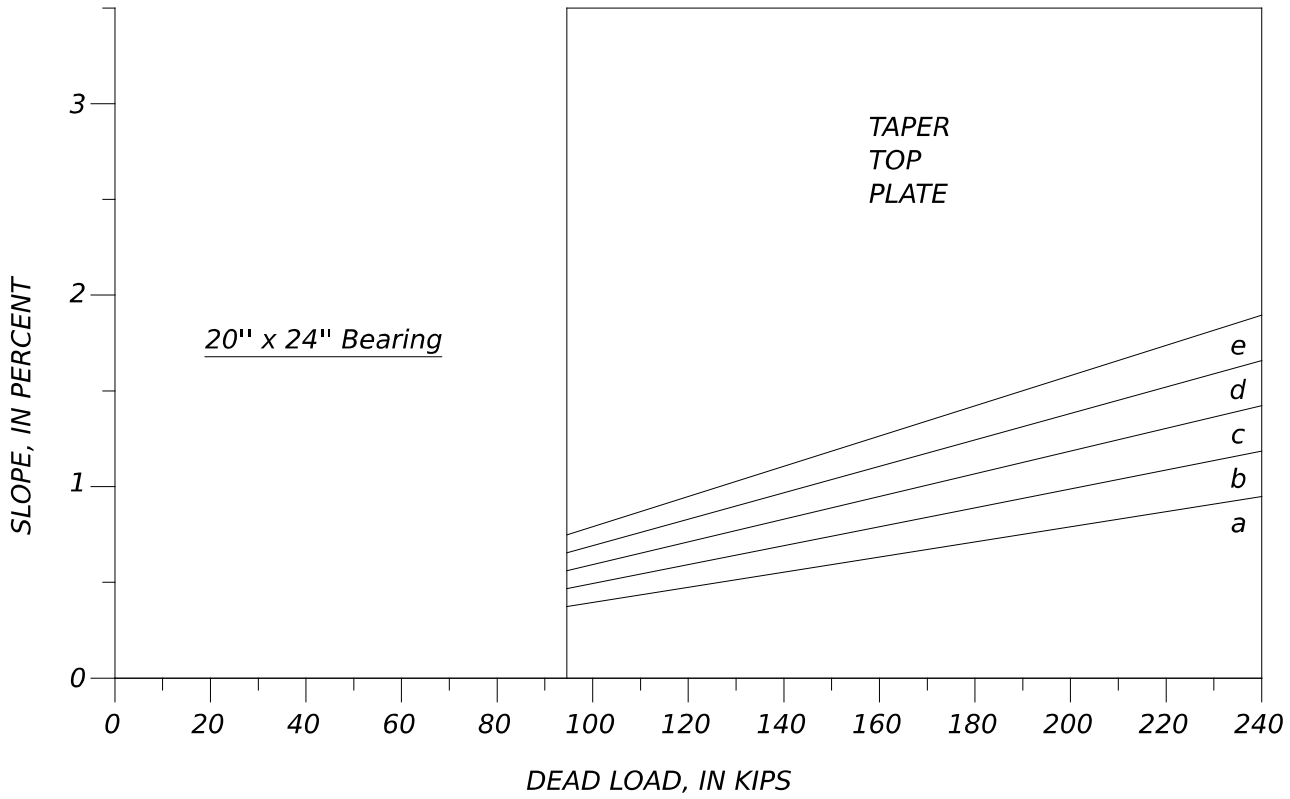


Figure 3.7.4-12

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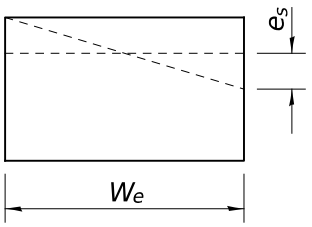
TYPE II AND III
MINIMUM BEARING THICKNESS
VS. SLOPE

Figure 3.7.4-13

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TYPE I

(Maximum edge strain due to nonparallelism ≤ .06 ERT)
(as given in Table of Dimensions Type I Bearing)



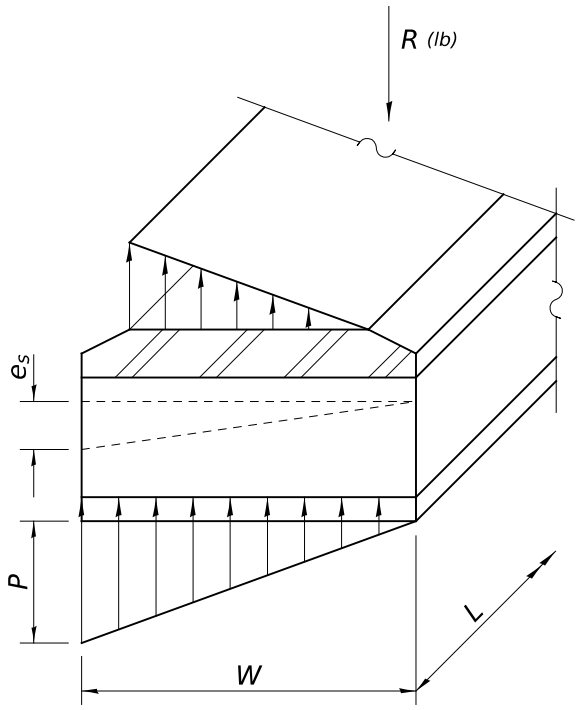
$$e_s = .06\ ERT$$

$$\text{Maximum \% Slope} = \frac{100\ e_s}{W_e/2}$$

$$\text{Maximum \% Slope} = \frac{12\ (ERT)}{W_e}$$

TYPE II & III

(No lift-off permitted under dead load)
(Recommended design as given in applicable Minimum Bearing Thickness vs. Slope figure.)



$$P = 1.4\ \frac{R}{WL}$$

$$e_s = \frac{P}{Er}\ (ERT)$$

$$\text{Maximum \% Slope} = \frac{100\ e_s}{W}$$

$$\text{Maximum \% Slope} = 0.012\ \frac{R\ (ERT)}{W^2 L}$$

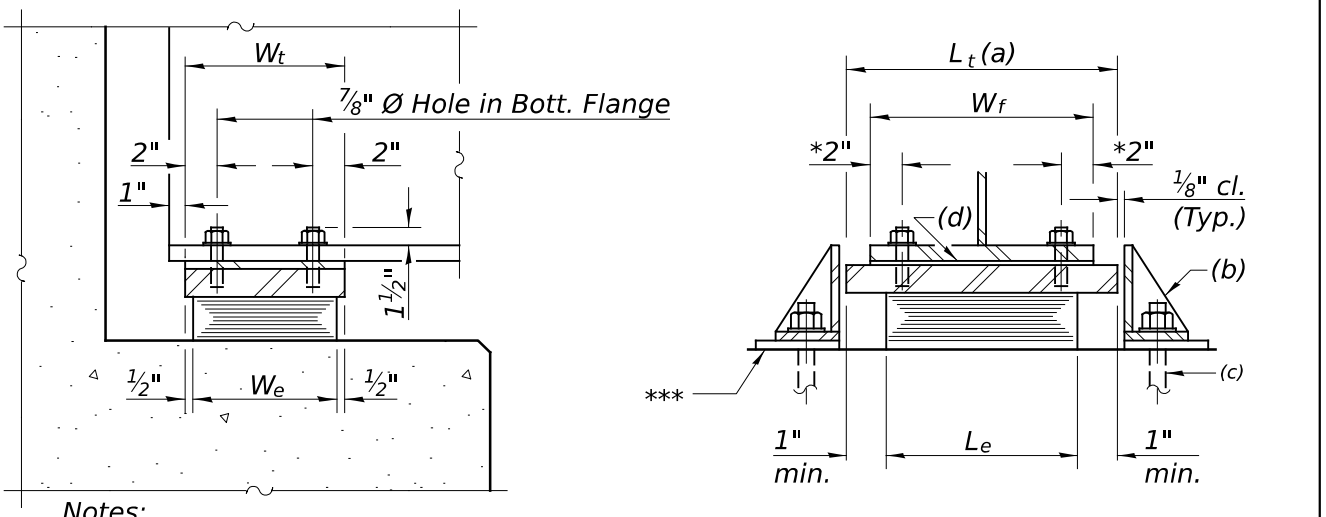
$$Er = 1.2 \times 10^4\ \frac{psi}{in./(ERT)}$$

For 5 ≤ SF ≤ 7 &
55±5 Durometer Rubber

SLOPE LIMITATIONS FOR ELASTOMERIC EXPANSION BEARINGS

Figure 3.7.4-14

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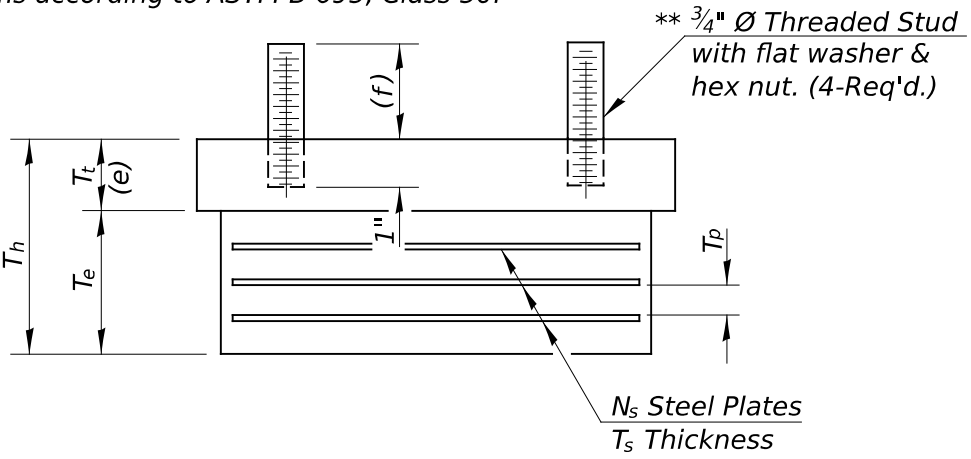
Notes:

(a) $L_t = W_f$ or $L_e + 2$" whichever is greater.
(b) Side retainer. See Figure 3.7.4-20.
(c) Anchor bolt. See Section 3.7.3 for required sizes.
(d) Adjusting shim plates. Fill plate, if required.
(e) 1½" Minimum thickness.
(f) 1½" + flange thickness + ¼" (for adjusting shim plates) + thickness of any required fill plate.

Side retainers, pintles, anchor bolts, nuts, washers, and bearing plates may be galvanized for steel beams and shall be galvanized for PPC I-beams, PPC Bulb T-beams, and PPC IL-beams according to AASHTO M111 or M232 (as applicable).

H.S. bolts may be galvanized for steel beams, when specified, according to ASTM B 695, Class 50.

H.S. bolts shall be galvanized for PPC I-beams, PPC Bulb T-beams, and PPC IL-beams according to ASTM B 695, Class 50.

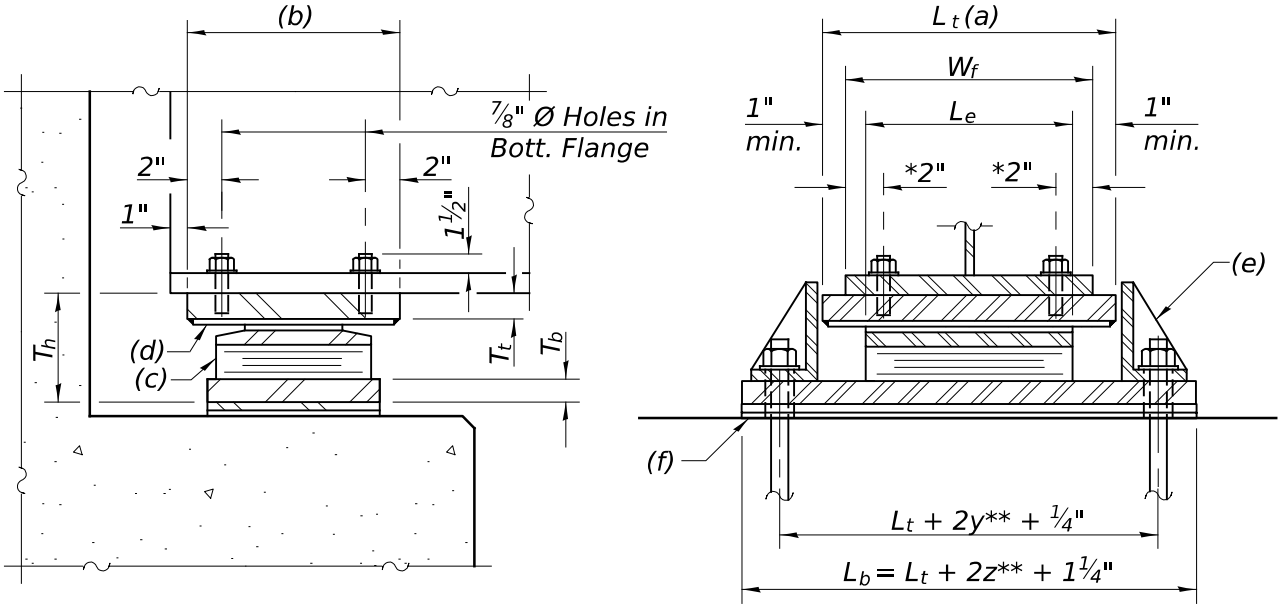


* *May be 1½" for smaller W shapes.*
** *A449 (Type 3 when unpainted steel is used).*
*** *¼" stainless steel plate under side retainer, typ.*

TYPE I ELASTOMERIC EXPANSION BEARING

Figure 3.7.4-15

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Notes:

(a) $L_t = W_f$ or $L_e + 2$" whichever is greater.

(b) See following figure.

(c) PTFE - Elastomeric bearing.

(d) See Figure 3.7.4-17.

(e) Side retainer. See Figure 3.7.4-20.

(f) Any required fill plate or shim plates. ⅛" Elastomeric neoprene leveling pad according to the material properties of Article 1052.02 of the Standard Specifications. Cost of pad included with bearing.

Side retainers, pintles, anchor bolts, nuts, washers, and bearing plates may be galvanized for steel beams and shall be galvanized for PPC I-beams, PPC Bulb T-beams, and PPC IL-beams according to AASHTO M111 or M232 (as applicable).

H.S. bolts may be galvanized for steel beams, when specified, according to ASTM B 695, Class 50.

H.S. bolts shall be galvanized for PPC I-beams, PPC Bulb T-beams, and PPC IL-beams according to ASTM B 695, Class 50.

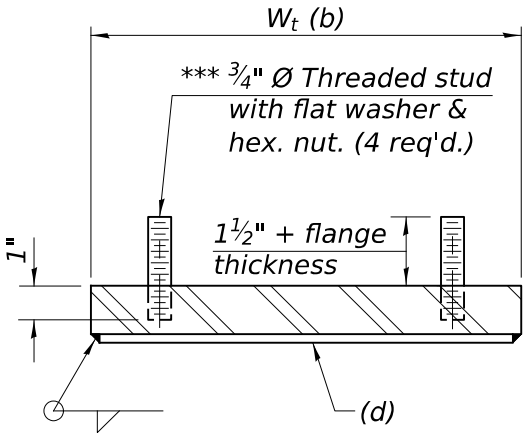
* *May be 1½" for smaller W shapes.*

** *See Figure 3.7.4-20 for value of y and z.*

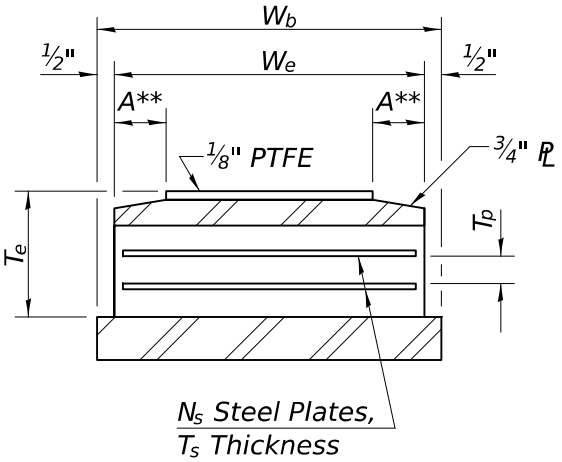
TYPE II ELASTOMERIC EXPANSION BEARING

Figure 3.7.4-16

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TOP BEARING ASSEMBLY



BOTTOM BEARING ASSEMBLY

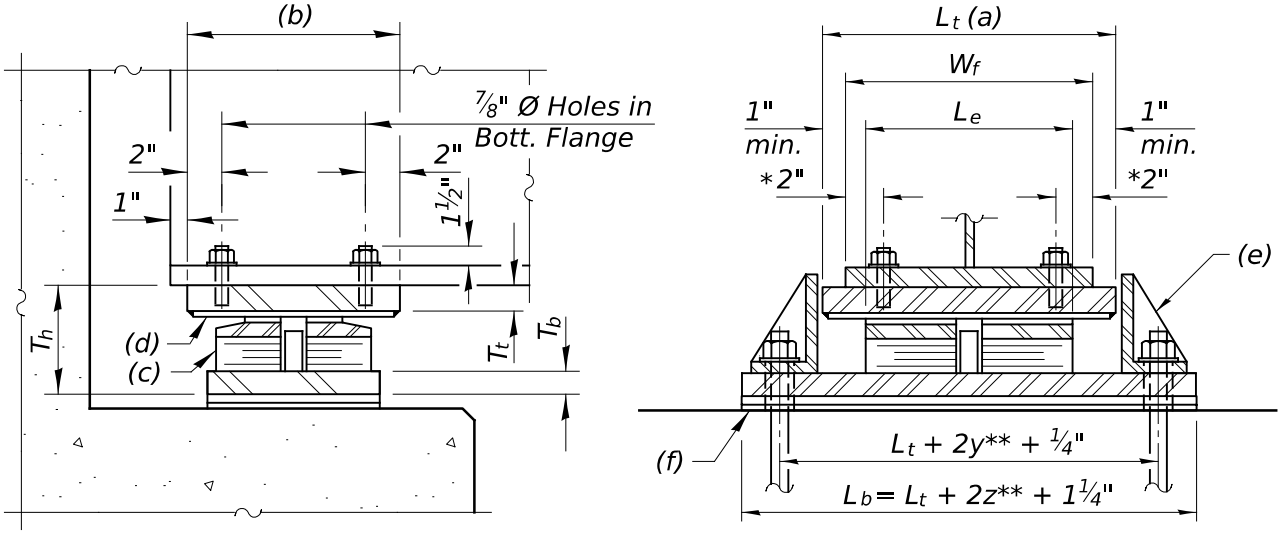
(b) $W_t = W_e - 2A + 2E + 2"$, E = Expansion from 50° normal temperature.
(d) 1/16" Stainless steel sliding plate, A240, Type 304, 2B finish.

** See Figure 3.7.4-20 for value of A.
*** M164, (Type 3 when weathering steel is used).

TYPE II
ELASTOMERIC EXPANSION
BEARING DETAILS

Figure 3.7.4-17

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree



Notes:

(a) $L_t = W_f$ or $L_e + 2$", whichever is greater.

(b) See Figure 3.7.4-19.

(c) PTFE - Elastomeric bearing.

(d) See Figure 3.7.4-19.

(e) Side retainer. See Figure 3.7.4-20.

(f) Any required fill plate or shim plates. ⅛" Elastomeric neoprene leveling pad according to the material properties of Article 1052.02 of the Standard Specifications. Cost of pad included with bearing.

(g) See Figure 3.7.4-19.

Side retainers, pintles, anchor bolts, nuts, washers, and bearing plates may be galvanized for steel beams and shall be galvanized for PPC I-beams, PPC Bulb T-beams, and PPC IL-beams according to AASHTO M111 or M232 (as applicable).

H.S. bolts may be galvanized for steel beams, when specified, according to ASTM B 695, Class 50.

H.S. bolts shall be galvanized for PPC I-beams, PPC Bulb T-beams, and PPC IL-beams according to ASTM B 695, Class 50.

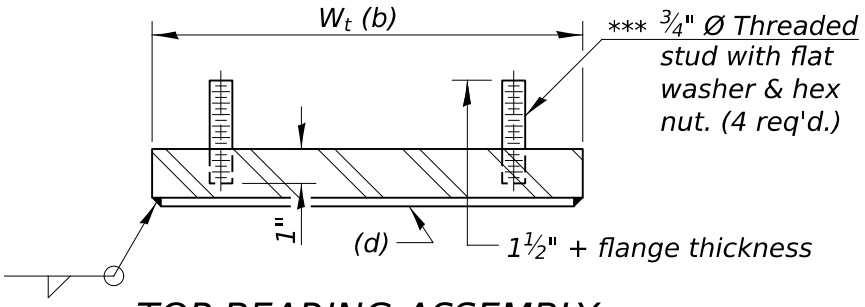
** May be 1½" for smaller W shapes.*

*** See Figure 3.7.4-20 for value of y and z.*

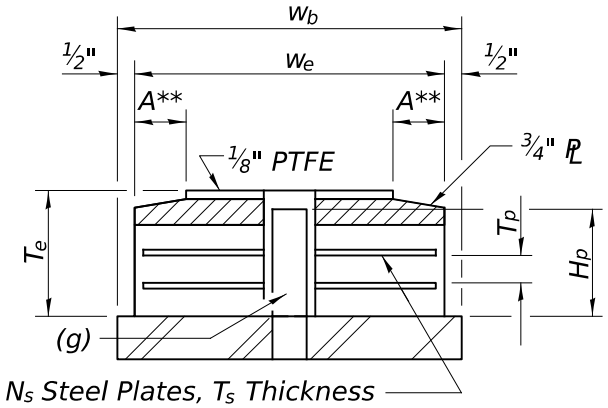
TYPE III ELASTOMERIC EXPANSION BEARING

Figure 3.7.4-18

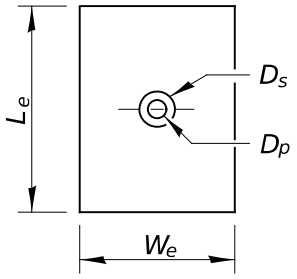
HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree



TOP BEARING ASSEMBLY



BOTTOM BEARING ASSEMBLY



SLOT DETAIL

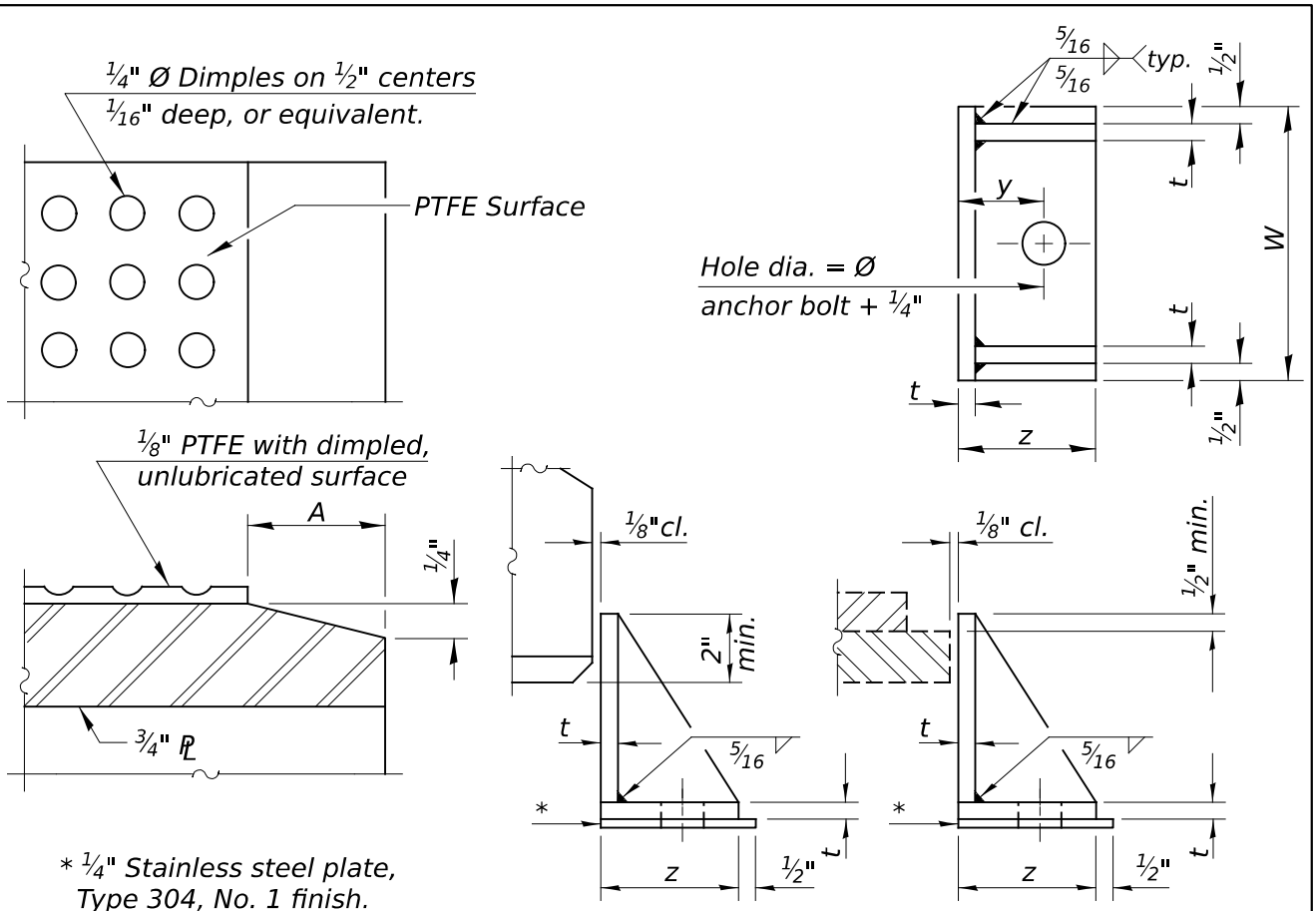
(b) $W_t = W_e - 2A + 2E + 2"$, E = Expansion from 50° normal temperature.
(d) 1/16" Stainless steel sliding plate, A240, Type 304, 2B finish.
(g) Shear restrictor pin according to the material properties of Article 1083.02(d) of the Standard Specifications.

** See Figure 3.7.4-20 for value of A.
*** A449 (Type 3 when weathering steel is used).

TYPE III
ELASTOMERIC EXPANSION
BEARING DETAILS

Figure 3.7.4-19

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree



* ¼" Stainless steel plate, Type 304, No. 1 finish. Only used with Type I.

SIDE RETAINER

W_e	6"	7"	9"	10"	11"	12"	13"	14"	15"	18"	20"
A	1"	1"	1½"	1½"	1½"	1½"	1½"	1½"	1½"	1½"	1½"
W	8"	8"	8"	8"	8"	10"	12"	14"	16"	19"	21"
z	3⅞"	4¼"	5½"	5½"	6¼"	7¼"	7⅞"	8¾"	9⅜"	10⅜"	10⅜"

Bolt Ø	y	t
¾"	1⅞"	½"
1"	2⅛"	½"
1¼"	2⅜"	½"
1½"	2¾"	⅝"
2"	3¼"	⅝"
2½"	3¾"	⅝"

Note:

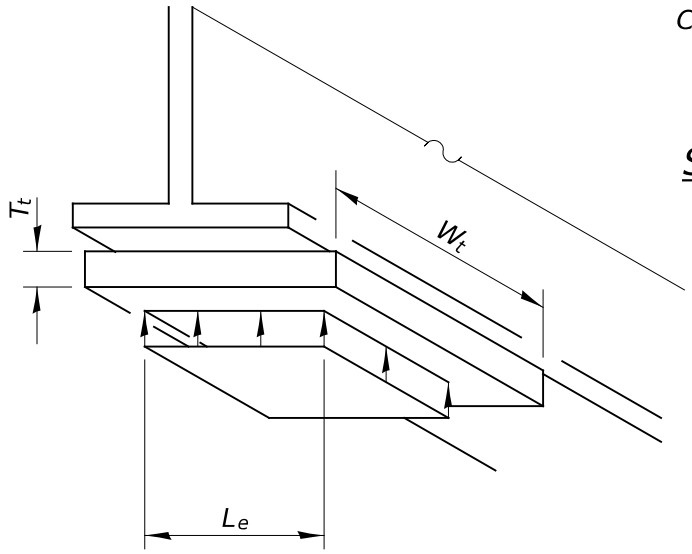
The ⅛" PTFE sheet shall be bonded directly to the top steel plate with a two-component, medium viscosity epoxy resin, conforming to the requirements of the Federal Specification MMM-A-134, Type I. The bond agent shall be applied on the full area of the contact surfaces.

MISCELLANEOUS DETAILS FOR ELASTOMERIC EXPANSION BEARINGS

Figure 3.7.4-20

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

R = Reaction in kips (Strength I)
Include impact for top plate only.
C = 0.122 for f_y = 50 ksi.

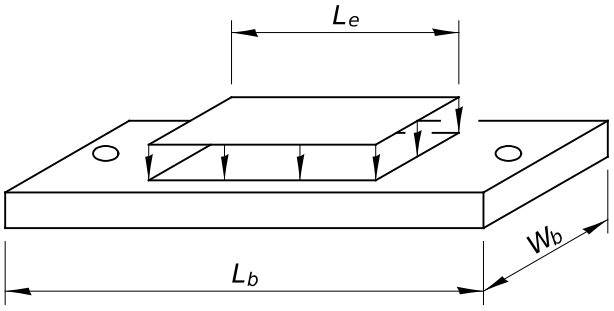


SOLE PLATE THICKNESS

$$*T_t = C\sqrt{\frac{R\,L_e}{W_t}}$$

*With bearing stiffeners use 80% T_t

Minimum T_t = 1½"



MASONRY PLATE THICKNESS

$$T_b = C\,(L_b - L_e)\sqrt{\frac{R}{L_b W_b}}$$

Minimum T_b = 1"

For Type III bearings, see Figure 3.7.4-25 for mininum T_b values for bearing.

ELASTOMERIC
BEARING PLATE DESIGN

Figure 3.7.4-21

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

Bearing Size (in.)	Type I and II (in kips) PSI 200	500	800	S.F.	Type III (in kips) PSI 200	500	*σs	S.F.
6 x 10	12.0	30.0	48.0	6.00	11.6	29.1	40.2	5.08
7 x 12	16.8	42.0	67.2	5.89	16.4	41.1	57.4	5.13
9 x 12	21.6	54.0	86.4	6.86	21.1	52.8	84.5	5.93
10 x 14	28.0	70.0	112.0	6.67	27.5	68.8	110.0	5.88
11 x 16	35.2	88.0	140.8	6.52	34.6	86.4	134.8	5.73
12 x 18	43.2	108.0	172.8	6.40	42.6	106.4	165.3	5.71
13 x 20	52.0	130.0	208.0	6.30	51.4	128.4	198.6	5.69
14 x 22	61.6	154.0	246.4	6.22	61.0	152.4	234.9	5.66
15 x 24	72.0	180.0	288.0	6.15	71.2	178.0	270.2	5.58
18 x 24	86.4	216.0	345.6	6.86	85.4	213.5	341.7	6.20
20 x 24	96.0	240.0	384.0	6.71	94.6	236.5	378.3	5.97

*σ_s = 136 x S.F.

Maximum σ_s = 800 psi

LOAD CAPACITY AND SHAPE FACTOR
TYPE I, II, AND III BEARINGS

Figure 3.7.4-22

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

Bearing	We	Le	Tp	Np	Ts	Ns	ERT	Te	Slope Max. %	W	z
6-a	6"	10"	5/16"	3	14 ga.	2	0.94"	1-1/16"	1.88	8-0"	3-7/8"
6-b	6"	10"	5/16"	5	14 ga.	4	1.56"	1-7/8"	3.13	8-0"	3-7/8"
6-c	6"	10"	5/16"	6	14 ga.	5	1.88"	2-1/4"	3.75	8-0"	3-7/8"
7-a	7"	12"	3/8"	3	3/32"	2	1.13"	1-5/16"	1.93	8-0"	4-1/4"
7-b	7"	12"	3/8"	4	3/32"	3	1.50"	1-3/4"	2.57	8-0"	4-1/4"
7-c	7"	12"	3/8"	5	3/32"	4	1.88"	2-1/4"	3.21	8-0"	4-1/4"
9-a	9"	12"	3/8"	5	3/32"	4	1.88"	2-1/4"	2.50	8-0"	5-1/2"
9-b	9"	12"	3/8"	7	3/32"	6	2.63"	3-3/16"	3.50	8-0"	5-1/2"
9-c	9"	12"	3/8"	8	3/32"	7	3.00"	3-5/8"	4.00	8-0"	5-1/2"
10-a	10"	14"	7/16"	5	1/8"	4	2.19"	2-11/16"	2.63	8-0"	5-1/2"
10-b	10"	14"	7/16"	6	1/8"	5	2.63"	3-1/4"	3.15	8-0"	5-1/2"
10-c	10"	14"	7/16"	7	1/8"	6	3.06"	3-13/16"	3.68	8-0"	5-1/2"
10-d	10"	14"	7/16"	8	1/8"	7	3.50"	4-3/8"	4.20	8-0"	5-1/2"
11-a	11"	16"	1/2"	4	1/8"	3	2.00"	2-3/8"	2.18	8-0"	6-1/4"
11-b	11"	16"	1/2"	5	1/8"	4	2.50"	3-0"	2.73	8-0"	6-1/4"
11-c	11"	16"	1/2"	6	1/8"	5	3.00"	3-5/8"	3.27	8-0"	6-1/4"
11-d	11"	16"	1/2"	7	1/8"	6	3.50"	4-1/4"	3.82	8-0"	6-1/4"
12-a	12"	18"	9/16"	3	3/16"	2	1.69"	2-1/16"	1.69	10-0"	7-1/4"
12-b	12"	18"	9/16"	4	3/16"	3	2.25"	2-13/16"	2.25	10-0"	7-1/4"
12-c	12"	18"	9/16"	5	3/16"	4	2.81"	3-9/16"	2.81	10-0"	7-1/4"
12-d	12"	18"	9/16"	6	3/16"	5	3.38"	4-5/16"	3.38	10-0"	7-1/4"
12-e	12"	18"	9/16"	7	3/16"	6	3.94"	5-1/16"	3.94	10-0"	7-1/4"
13-a	13"	20"	5/8"	3	3/16"	2	1.88"	2-1/4"	1.73	12-0"	7-7/8"
13-b	13"	20"	5/8"	4	3/16"	3	2.50"	3-1/16"	2.31	12-0"	7-7/8"
13-c	13"	20"	5/8"	5	3/16"	4	3.13"	3-7/8"	2.88	12-0"	7-7/8"
13-d	13"	20"	5/8"	6	3/16"	5	3.75"	4-11/16"	3.46	12-0"	7-7/8"
13-e	13"	20"	5/8"	7	3/16"	6	4.38"	5-1/2"	4.04	12-0"	7-7/8"
14-a	14"	22"	11/16"	3	3/16"	2	2.06"	2-7/16"	1.77	14-0"	8-3/4"
14-b	14"	22"	11/16"	4	3/16"	3	2.75"	3-5/16"	2.36	14-0"	8-3/4"
14-c	14"	22"	11/16"	5	3/16"	4	3.44"	4-3/16"	2.95	14-0"	8-3/4"
14-d	14"	22"	11/16"	6	3/16"	5	4.13"	5-1/16"	3.54	14-0"	8-3/4"
14-e	14"	22"	11/16"	7	3/16"	6	4.81"	5-15/16"	4.13	14-0"	8-3/4"
15-a	15"	24"	3/4"	3	3/16"	2	2.25"	2-5/8"	1.80	16-0"	9-3/8"
15-b	15"	24"	3/4"	4	3/16"	3	3.00"	3-9/16"	2.40	16-0"	9-3/8"
15-c	15"	24"	3/4"	5	3/16"	4	3.75"	4-1/2"	3.00	16-0"	9-3/8"
15-d	15"	24"	3/4"	6	3/16"	5	4.50"	5-7/16"	3.60	16-0"	9-3/8"
15-e	15"	24"	3/4"	7	3/16"	6	5.25"	6-3/8"	4.20	16-0"	9-3/8"
18-a	18"	24"	3/4"	4	3/16"	3	3.00"	3-9/16"	2.00	19-0"	10-3/8"
18-b	18"	24"	3/4"	5	3/16"	4	3.75"	4-1/2"	2.50	19-0"	10-3/8"
18-c	18"	24"	3/4"	6	3/16"	5	4.50"	5-7/16"	3.00	19-0"	10-3/8"
18-d	18"	24"	3/4"	7	3/16"	6	5.25"	6-3/8"	3.50	19-0"	10-3/8"
18-e	18"	24"	3/4"	8	3/16"	7	6.00"	7-5/16"	4.00	19-0"	10-3/8"
20-a	20"	24"	13/16"	4	3/16"	3	3.25"	3-13/16"	1.95	21-0"	10-3/8"
20-b	20"	24"	13/16"	5	3/16"	4	4.06"	4-13/16"	2.44	21-0"	10-3/8"
20-c	20"	24"	13/16"	6	3/16"	5	4.88"	5-13/16"	2.93	21-0"	10-3/8"
20-d	20"	24"	13/16"	7	3/16"	6	5.69"	6-13/16"	3.41	21-0"	10-3/8"
20-e	20"	24"	13/16"	8	3/16"	7	6.50"	7-13/16"	3.90	21-0"	10-3/8"

TABLE OF DIMENSIONS
TYPE I BEARING

Figure 3.7.4-23

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

Bearing	We	Le	Tp	Np	Ts	Ns	ERT	Te
6-a	6"	10"	5/16"	3	14 ga.	2	0.94"	1-15/16"
6-b	6"	10"	5/16"	5	14 ga.	4	1.56"	2-3/4"
6-c	6"	10"	5/16"	6	14 ga.	5	1.88"	3-1/8"
7-a	7"	12"	3/8"	3	3/32"	2	1.13"	2-3/16"
7-b	7"	12"	3/8"	4	3/32"	3	1.50"	2-5/8"
7-c	7"	12"	3/8"	5	3/32"	4	1.88"	3-1/8"
9-a	9"	12"	3/8"	5	3/32"	4	1.88"	3-1/8"
9-b	9"	12"	3/8"	7	3/32"	6	2.63"	4-1/16"
9-c	9"	12"	3/8"	8	3/32"	7	3.00"	4-1/2"
10-a	10"	14"	7/16"	5	1/8"	4	2.19"	3-9/16"
10-b	10"	14"	7/16"	6	1/8"	5	2.63"	4-1/8"
10-c	10"	14"	7/16"	7	1/8"	6	3.06"	4-11/16"
10-d	10"	14"	7/16"	8	1/8"	7	3.50"	5-1/4"
11-a	11"	16"	1/2"	4	1/8"	3	2.00"	3-1/4"
11-b	11"	16"	1/2"	5	1/8"	4	2.50"	3-7/8"
11-c	11"	16"	1/2"	6	1/8"	5	3.00"	4-1/2"
11-d	11"	16"	1/2"	7	1/8"	6	3.50"	5-1/8"
12-a	12"	18"	9/16"	3	3/16"	2	1.69"	2-15/16"
12-b	12"	18"	9/16"	4	3/16"	3	2.25"	3-11/16"
12-c	12"	18"	9/16"	5	3/16"	4	2.81"	4-7/16"
12-d	12"	18"	9/16"	6	3/16"	5	3.38"	5-3/16"
12-e	12"	18"	9/16"	7	3/16"	6	3.94"	5-15/16"
13-a	13"	20"	5/8"	3	3/16"	2	1.88"	3-1/8"
13-b	13"	20"	5/8"	4	3/16"	3	2.50"	3-15/16"
13-c	13"	20"	5/8"	5	3/16"	4	3.13"	4-3/4"
13-d	13"	20"	5/8"	6	3/16"	5	3.75"	5-9/16"
13-e	13"	20"	5/8"	7	3/16"	6	4.38"	6-3/8"
14-a	14"	22"	11/16"	3	3/16"	2	2.06"	3-5/16"
14-b	14"	22"	11/16"	4	3/16"	3	2.75"	4-3/16"
14-c	14"	22"	11/16"	5	3/16"	4	3.44"	5-1/16"
14-d	14"	22"	11/16"	6	3/16"	5	4.13"	5-15/16"
14-e	14"	22"	11/16"	7	3/16"	6	4.81"	6-13/16"
15-a	15"	24"	3/4"	3	3/16"	2	2.25"	3-1/2"
15-b	15"	24"	3/4"	4	3/16"	3	3.00"	4-7/16"
15-c	15"	24"	3/4"	5	3/16"	4	3.75"	5-3/8"
15-d	15"	24"	3/4"	6	3/16"	5	4.50"	6-5/16"
15-e	15"	24"	3/4"	7	3/16"	6	5.25"	7-1/4"
18-a	18"	24"	3/4"	4	3/16"	3	3.00"	4-7/16"
18-b	18"	24"	3/4"	5	3/16"	4	3.75"	5-3/8"
18-c	18"	24"	3/4"	6	3/16"	5	4.50"	6-5/16"
18-d	18"	24"	3/4"	7	3/16"	6	5.25"	7-1/4"
18-e	18"	24"	3/4"	8	3/16"	7	6.00"	8-3/16"
20-a	20"	24"	13/16"	4	3/16"	3	3.25"	4-11/16"
20-b	20"	24"	13/16"	5	3/16"	4	4.06"	5-11/16"
20-c	20"	24"	13/16"	6	3/16"	5	4.88"	6-11/16"
20-d	20"	24"	13/16"	7	3/16"	6	5.69"	7-11/16"
20-e	20"	24"	13/16"	8	3/16"	7	6.50"	8-11/16"

TABLE OF DIMENSIONS
TYPE II AND III BEARINGS

Figure 3.7.4-24

HARD COPIES UNCONTROLLED - SEE public.powerdms.com/IDOT/tree

Bearing	Tb*(Gr 36)	Tb*(Gr 50)	Dp	Ds	Hp
6-a	1"	1"	1"	1-1/2"	1-1/2"
6-b	1"	1"	1"	1-1/2"	2-1/4"
6-c	1"	1"	1"	1-1/2"	2-3/4"
7-a	1"	1"	1"	1-1/2"	1-3/4"
7-b	1-1/4"	1"	1"	1-1/2"	2-1/4"
7-c	1-1/4"	1"	1"	1-1/2"	2-3/4"
9-a	1-1/4"	1-1/4"	1-1/4"	1-3/4"	2-3/4"
9-b	1-1/2"	1-1/4"	1-1/4"	1-3/4"	3-3/4"
9-c	1-1/2"	1-1/4"	1-1/4"	1-3/4"	4"
10-a	1-1/2"	1-1/4"	1-1/4"	1-3/4"	3-1/4"
10-b	1-3/4"	1-1/2"	1-1/4"	1-3/4"	3-3/4"
10-c	1-3/4"	1-1/2"	1-1/4"	1-3/4"	4-1/4"
10-d	2"	1-3/4"	1-1/4"	1-3/4"	4-3/4"
11-a	1-1/2"	1-1/2"	1-1/2"	2"	2-3/4"
11-b	1-3/4"	1-1/2"	1-1/2"	2"	3-1/2"
11-c	1-3/4"	1-1/2"	1-1/2"	2"	4"
11-d	2"	1-3/4"	1-1/2"	2"	4-3/4"
12-a	1-3/4"	1-1/2"	1-1/2"	2"	2-1/2"
12-b	1-3/4"	1-1/2"	1-1/2"	2"	3-1/4"
12-c	2"	1-3/4"	1-1/2"	2"	4"
12-d	2-1/4"	1-3/4"	1-1/2"	2"	4-3/4"
12-e	2-1/4"	2"	1-1/2"	2"	5-1/2"
13-a	1-3/4"	1-1/2"	1-1/2"	2"	2-3/4"
13-b	2"	1-3/4"	1-1/2"	2"	3-1/2"
13-c	2-1/4"	2"	1-1/2"	2"	4-1/4"
13-d	2-1/2"	2-1/4"	1-1/2"	2"	5-1/4"
13-e	2-3/4"	2-1/4"	1-1/2"	2"	6"
14-a	2"	1-3/4"	1-1/2"	2"	3"
14-b	2-1/4"	2"	1-1/2"	2"	3-3/4"
14-c	2-1/2"	2-1/4"	1-1/2"	2"	4-3/4"
14-d	2-3/4"	2-1/2"	1-1/2"	2"	5-1/2"
14-e	3"	2-1/2"	1-1/2"	2"	6-1/2"
15-a	2"	1-3/4"	1-3/4"	2-1/4"	3"
15-b	2-1/2"	2"	1-3/4"	2-1/4"	4"
15-c	2-3/4"	2-1/4"	1-3/4"	2-1/4"	5"
15-d	3"	2-1/2"	1-3/4"	2-1/4"	6"
15-e	3"	2-1/2"	1-3/4"	2-1/4"	6-3/4"
18-a	2-1/2"	2"	2"	2-1/2"	4"
18-b	2-3/4"	2-1/4"	2"	2-1/2"	5"
18-c	3"	2-1/2"	2"	2-1/2"	6"
18-d	3-1/4"	2-3/4"	2"	2-1/2"	6-3/4"
18-e	3-1/4"	2-3/4"	2"	2-1/2"	7-3/4"
20-a	2-1/2"	2-1/2"	2-1/2"	3"	4-1/4"
20-b	2-3/4"	2-1/2"	2-1/2"	3"	5-1/4"
20-c	2-3/4"	2-1/2"	2-1/2"	3"	6-1/4"
20-d	3"	2-1/2"	2-1/2"	3"	7-1/4"
20-e	3-1/4"	2-3/4"	2-1/2"	3"	8-1/4"

*Tb thickness is governed by the bearing stress of the pin to plate; however, Tb shall not be less than Dp.
Hp = Te – ((3/4 ÷ 2) + 1/8") then round up to nearest ¼".

TABLE OF DIMENSIONS
TYPE III BEARING

Figure 3.7.4-25

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3.7.5 High Load Multi-Rotational (HLMR) Bearings

HLMR bearings support high loads and are able to rotate in any direction. They typically are necessary for long span or unique structures. They are suited for bearing designs beyond the limitations of the standard elastomeric bearings shown in Section 3.7.4. If a curved bridge is equivalently straight or bearing locations on curved bridges will result in lateral movements not exceeding the lateral movement capacity of standard elastomeric bearings, then standard elastomeric bearings may be used. For curved bridges that are not equivalently straight or when the elastomeric bearing lateral movement capacity is exceeded, HLMR bearings shall be required for structures designed for curvature.

The HLMR Bearing types (Pot or Disc) can be divided into one of three classes based on its expansion requirements. The three classes of HLMR bearings are: Fixed, Guided expansion, and Non-Guided expansion. Fixed and guided expansion HLMR bearings are analogous to fixed bearings and elastomeric bearings with side retainers and are used in similar scenarios. A non-guided expansion HLMR bearing is similar to an elastomeric bearing without a side retainer and is applicable in cases where both longitudinal and lateral displacements are required to be accommodated, such as very wide structures, some curved structures, or structures with large lateral seismic displacements.

The base sheets are in the “Bearings-Steel Beams” IDOT CADD Bridge Cell Library. The base sheet details have been thoroughly vetted and should result in better serviceability than alternate details. Because of unknown availability and potential serviceability issues, alternate bearing types such as inverted bearings or center-guided bearings are not allowed.

3.7.5.1 HLMR Bearing Selection

Most companies that supply HLMR bearings use standardized bearing dimensions which are designed to provide a lateral resistance of 10% of the total service axial load capacity. If the service lateral design load is greater than (>) 10% but less than or equal to (≤) 20% of the design vertical load, a larger HLMR bearing may be selected based upon the lateral load. When service lateral design load exceeds a threshold value of 20% of the vertical design load, the designer should not select a significantly larger HLMR bearing to satisfy the lateral load requirement. Rather, designers should select a HLMR bearing size based on the vertical design load and the fabricator shall be responsible for modifying any necessary components of the bearing to meet the lateral load and expansion length demands for LRFD as applicable.

The HLMR bearing and its components will be designed by the bearing manufacturer to meet the project specific requirements provided in the Design Data table on the plans and the AASHTO LRFD Bridge

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Design Specifications. The bridge designer shall contact the bearing manufacturer when selecting the bearing size to meet the design requirements. Pot bearings shall be shown on the plans for vertical load capacities less than 300 kips and disc bearings shall be shown on the plans for vertical load capacities equal to and greater than 300 kips. However, if pot and disc bearings are required on the same structure, it is recommended to detail disc bearings at the pot bearing locations for economy. Elastomeric bearings may be used in addition to HLMR bearings on the same structure provided the same bearing type is used across the same substructure unit.

3.7.5.2 HLMR Bearing Design

Manufacturer literature for the Department's Prequalified List for HLMR Bearing manufacturers may be found on the IDOT web site at Prequalified Structural Systems under Find Your List. This list also states which manufacturers are pre-approved for pot bearings, disc bearings, or both.

The bridge designer shall calculate the Service I Factored Vertical Reaction to determine the type of HLMR bearing – pot or disc – to use. The bridge designer shall calculate the Maximum Strength or Extreme Event Lateral Reaction. The bridge designer shall determine rotations and movement using AASHTO 14.4.2.2 and the current AASHTO / NSBA Steel Bridge Collaboration "Steel Bridge Bearing Design and Detailing Guidelines". The bridge designer will show this information in the Design Data table on the plan HLMR bearing sheet.

For each bearing size, type, and class on a bridge, the bridge designer shall evaluate bearing manufacturer literature from the Prequalified Structural Systems (PQSS) List to determine the appropriate bearing depth, and pot and piston outside diameter (pot bearings) or disc diameter and disc plate diameters (disc bearings). These dimensions shall be chosen based upon the Service I Factored Vertical Reaction shown in the Design Data table shown on the plans.

Article C14.6.1 of the AASHTO LRFD Bridge Design Specifications states that bearings loaded to less than 20 percent of their vertical capacity require special design. This occurs when the ratio of dead loads to live loads is very small. Therefore, when choosing a bearing size, bridge designers shall verify that the Unfactored Dead Load Vertical Reaction is not less than 20 percent of the Service I Factored Vertical Reaction:

$$\frac{\text{Unfactored Dead Load Vertical Reaction}}{\text{Service I Factored Vertical Reaction}} > 0.2$$

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The bridge designer shall also verify that there is not any uplift at the bearing location due to Service I Factored Live Loads:

Service I Factored Vertical Reaction > 0

If either of these conditions are encountered and cannot be avoided by choosing a different bearing size, the bridge designer shall provide a vertical restraint at the bearing location to prevent uplift. These vertical restraints are typically in the form of horizontal tabs or clamps extending from the Masonry Plate over the top of the bottom flange. Bearing suppliers are aware of this issue and have their own supplier-specific and bearing-specific details. Therefore, details for vertical restraints are not included on the base sheets. Vertical restraints shall be shown on the plans when required.

The bridge designer shall determine required dimensions of the Sole Plate and Masonry Plate for the plate dimensions and loads assumed in design. The bridge designer shall detail the bevel slope of the Sole Plate, if required. See Section 3.7.5.4 “Sole and Masonry Plate Dimensions” for more information.

The bridge designer should evaluate the bearing literature from as many PQSS manufacturers as feasible and choose the bearing with the largest bearing height for the type and vertical capacity required. The overall bearing height and plate thicknesses stated on the contract plans shall be chosen such that more than one manufacturer is capable of bidding on the project.

The design of HLMR bearing components will be the responsibility of the bearing manufacturer in accordance with AASHTO based on the parameters outlined below.

The bearing will be designed by the supplier for the exact parameters specified in the plan Design Data table. The load value specified in the pay item name is an approximate vertical load capacity that is used for letting and bidding purposes only. See Section 3.7.5.7 “Plan Presentation” and Section 3.7.5.8 “Completing the Bill of Material” for more information.

The Maximum Strength or Extreme Event Lateral Reaction will be used by the bearing manufacturer to design required bearing components such as guide bars, pot thicknesses, etc. Due to this, details provided by the bearing manufacturer such as Sole and Masonry Plate dimensions may be slightly larger to account for the additional lateral loads. These slightly larger details should not be expected to be so large that the Sole and Masonry Plate thicknesses determined by the bridge designer will be inadequate. The bearing manufacturer may refine the dimensions of the Sole Plate and Masonry Plate when designing the bearing.

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Upon receipt of the shop drawings, the bridge designer shall verify the Sole and Masonry Plate thicknesses shown on the shop drawings. The supplier has the option of recessing various elements, such as pistons and pots, into the Sole and Masonry Plates. When this occurs, the minimum plate thicknesses shall be verified against the thickness of the plate in the location of the recess.

The bearing plate lengths, widths, and thicknesses prescribed in Section 3.7.5.4 are intended to be used as guidance for achieving a similar bearing in cost and overall depth by all suppliers. It also assists the designer in determining the seat elevation and construction gap necessary for the bearing. However, the lengths and widths of the bearing plates actually provided by the supplier may vary substantially from those shown on the contract plans. These variances, used in conjunction with the plate thickness formulas prescribed in Section 3.7.5.4, may result in substantial variances in plate thicknesses. When checking shop drawings for HLMR bearings, plate lengths and widths provided on shop drawings may vary, but plate thicknesses thinner by more than 25% of those shown on the contract plans should be returned with a comment requesting verification.

The bridge designer shall be responsible for the design of the threaded studs of the Sole Plate to flange connection and anchor bolts.

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3.7.5.3 HLMR Bearing Dimensions

The dimensions used for pot bearings are:

D	=	Base Cylinder Diameter, inch
L_b	=	Masonry Plate Length, inch
L_p	=	Transverse Piston Diameter, inch
L_t	=	Sole Plate Length, inch
T_b	=	Masonry Plate Thickness, inch
T_h	=	Bearing Assembly Total Height, inch
T_t	=	Sole Plate Thickness, inch
W_b	=	Masonry Plate Width, inch
W_t	=	Sole Plate Width, inch

The dimensions used for disc bearings are:

D_d	=	Disc Diameter, inch
D_{db}	=	Bottom Disc Plate Diameter, inch
D_{dt}	=	Top Disc Plate Diameter, inch
L_d	=	Transverse length of the top disc plate for fixed bearings or disc diameter for expansion bearings, inch
T_b	=	Masonry Plate Thickness, inch
T_h	=	Bearing Assembly Total Height, inch
T_t	=	Sole Plate Thickness, inch
W_b	=	Masonry Plate Width, inch
W_t	=	Sole Plate Width, inch

Dimensions L_p and L_d will equal the diameter value for either the piston or top disc plate, respectively. The “L” nomenclature dimensions are meant to be analogous to the L_e dimension shown in Figure 3.7.4-21.

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3.7.5.4 Sole and Masonry Plate Dimensions

Masonry Plate dimensions assume a 2 inch minimum edge distance for anchor bolts. This is based upon the minimum edge distance for a 1½ inch diameter anchor bolt according to Table 6.13.2.6.6-1 of the AASHTO LRFD Bridge Design Specifications. If a different anchor bolt size / edge distance is used, the equations below for L_b shall be modified for the given edge distance. For Masonry Plates with anchor bolts with diameters greater than 1½ inch, the edge distance shall be increased according to the minimum edge distance requirements found in the AASHTO specification.

In the plate dimension equations below, 2 inches is allowed between the edge of Sole Plate and the line of anchor bolts in the Masonry Plate. If a different spacing is used, the equations below for L_b shall be modified for the given spacing between the Sole Plate and the line of anchor bolts.

The calculations for L_t, W_t, L_b and W_b are minimum dimensions. The dimensions may be adjusted if other factors dictate.

The IDOT policy for treatment of vertical bearing loads for HLMR and elastomeric bearings using the LRFD Code is analogous. When determining the plate thicknesses T_t and T_b, Load Case Service I using the HL-93 loading shall be used for determining the vertical reaction. See Section 3.7.4. Plate thicknesses T_t and T_b shall be determined according to top and bottom bearing plate equations found on Figure 3.7.4-21. The dimensions of the Sole Plate and Masonry plate shall be as outlined below:

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Fixed Pot Bearings (Figure 3.7.5.4-1):

Sole Plate Length and Width

L_t = D + 2(½ in.) = D + 1 in.

W_t = D + 2(½ in.)+ 1 in. = D + 2 in.

Masonry Plate Length and Width

L_b = L_t + 2(2 in.) + 2(2 in.) = L_t +8 in.

W_b = D + 2(1¼ in.) = D + 2½ in.

Guided Expansion Pot Bearings (Figure 3.7.5.4-2 and 3.7.5.4-3):

Sole Plate Length and Width

L_t = D + 2(½ in.) + 2(1/16 in.) + 2(1¾ in.) + 2(1 in.) = D + 6⅝ in.

W_t = L_t + 1 in.

Masonry Plate Length and Width

L_b = L_t + 2(2 in.) + 2(2 in.) = L_t + 8 in.

W_b = D + 2(1¼ in.) = D + 2½ in.

Non-Guided Expansion Pot Bearings (Figure 3.7.5.4-4):

Sole Plate Length and Width

L_t = D + 2(½ in.) + 2(1½ in.) = D + 4 in.

W_t = L_t + 1 in.

Masonry Plate Length and Width

L_b = L_t + 2(2 in.) + 2(2 in.) = L_t + 8 in.

W_b = D + 2(1¼ in.) = D + 2½ in.

For pot bearings, to determine Sole Plate thickness, T_t, the transverse dimension of the pot, dimension L_p, shall be substituted for the variable L_e.

To determine Masonry Plate thickness, T_b, the outside diameter of the base cylinder, dimension D, shall be substituted for the variable L_e.

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Fixed Disc Bearings (Figure 3.7.5.4-5):

Sole Plate Length and Width

L_t = D_{dt} + 2(1½ in.) = D_{dt} + 3 in.

W_t = D_{dt} + 2(1½ in.) + (1 in.) = D_{dt} + 4 in.

Masonry Plate Length and Width

L_b = L_t + 2(2 in.) + 2(2 in.) = L_t + 8 in.

W_b = D_{db} + 2(1in.) = D_{db} + 2 in.

To determine Sole Plate thickness, T_t, the diameter of the top disc plate, dimension D_{dt}, shall be substituted for the variable L_e.

To determine Masonry Plate thickness, T_b, the diameter of the bottom disc plate, dimension D_{db}, shall be substituted for the variable L_e.

Guided Expansion Disc Bearings (Figure 3.7.5.4-6):

Sole Plate Length and Width

L_t = D_{dt} + 2(1/16 in.) + 2(1¾ in.) + 2(1 in.) = D_{dt} + 5⅝ in.

W_t = L_t + 1 in.

Masonry Plate Length and Width

L_b = L_t + 2(2 in.) + 2 (2 in.) = L_t + 8 in.

W_b = D_{db} + 2(1 in.) = D_{db} + 2 in.

To determine Sole Plate thickness, T_t, the diameter of the disc, dimension D_d, shall be substituted for the variable L_e.

To determine Masonry Plate thickness, T_b, the diameter of the bottom disc plate, dimension D_{db}, shall be substituted for the variable L_e.

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Non-Guided Expansion Disc Bearings (Figure 3.7.5.4-7):

Sole Plate Length and Width

$L_t = D_{dt} + 2(1½ \text{ in.}) = D_{dt} + 3 \text{ in.}$

$W_t = L_t + 1 \text{ in.}$

Masonry Plate Length and Width

$L_b = L_t + 2(2 \text{ in.}) + 2(2 \text{ in.}) = L_t + 8 \text{ in.}$

$W_b = D_{db} + 2(1 \text{ in.}) = D_{db} + 2 \text{ in.}$

To determine Sole Plate thickness, T_t, the diameter of the disc, dimension D_d, shall be substituted for the variable L_e.

To determine Masonry Plate thickness, T_b, the diameter of the bottom disc plate, dimension D_{db}, shall be substituted for the variable L_e. If a bottom disc plate is not used, the diameter of the disc, dimension D_d shall be substituted for the variable L_e.

The bridge designer shall combine the design thicknesses for Sole and Masonry Plates with the maximum height of bearing components from manufacturers' literature to calculate the total bearing height, T_h. Shim plates and ⅛ inch leveling pad are not included in total bearing height.

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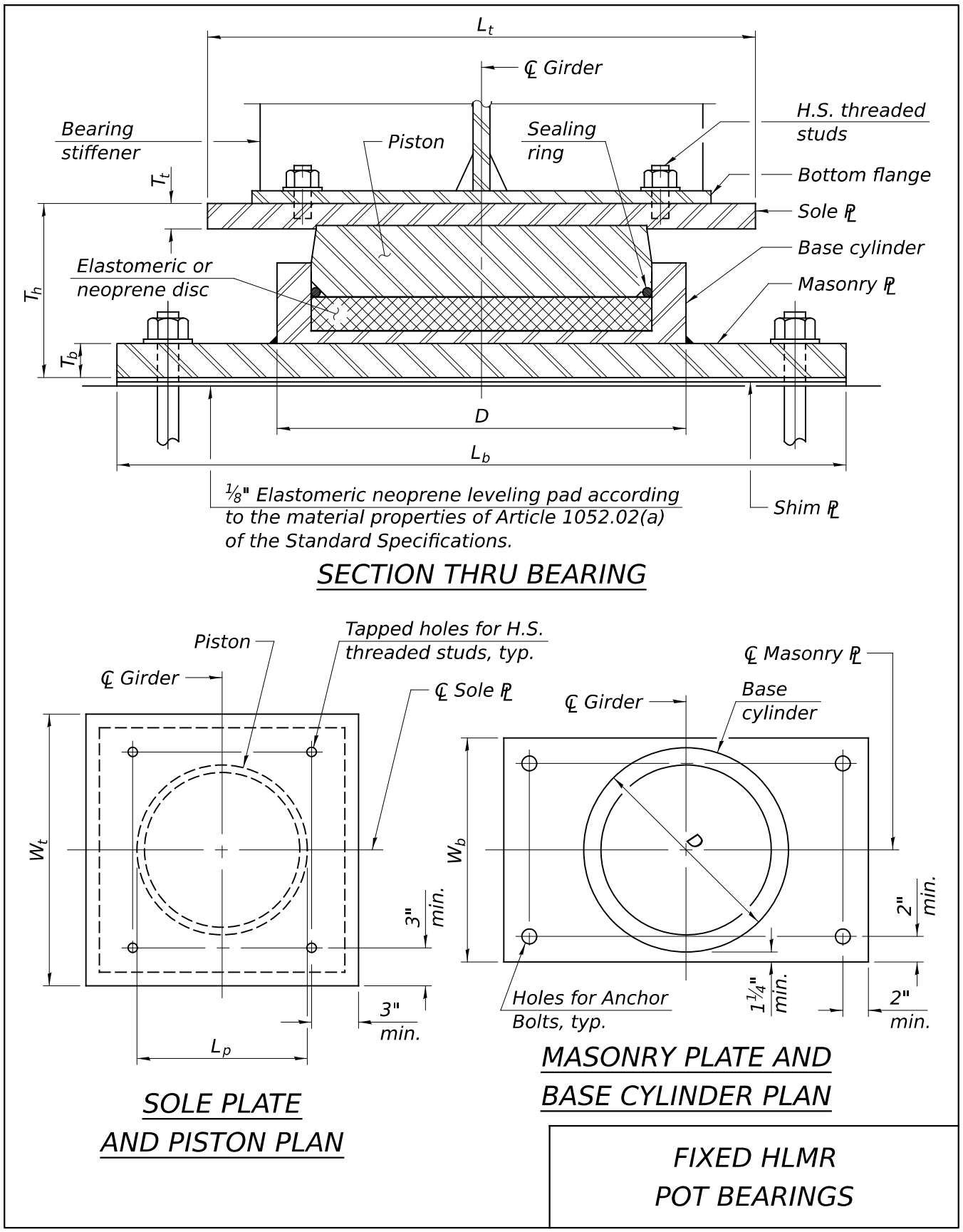
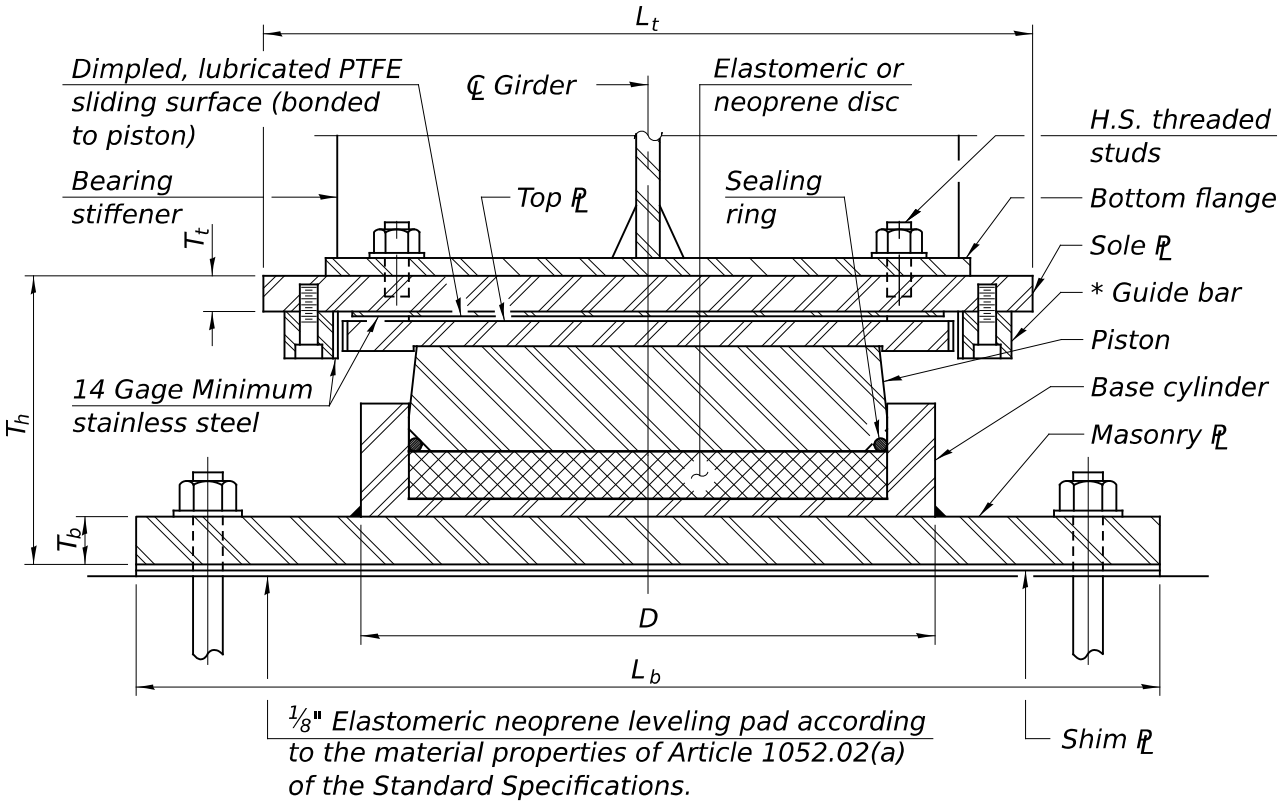


Figure 3.7.5.4-1

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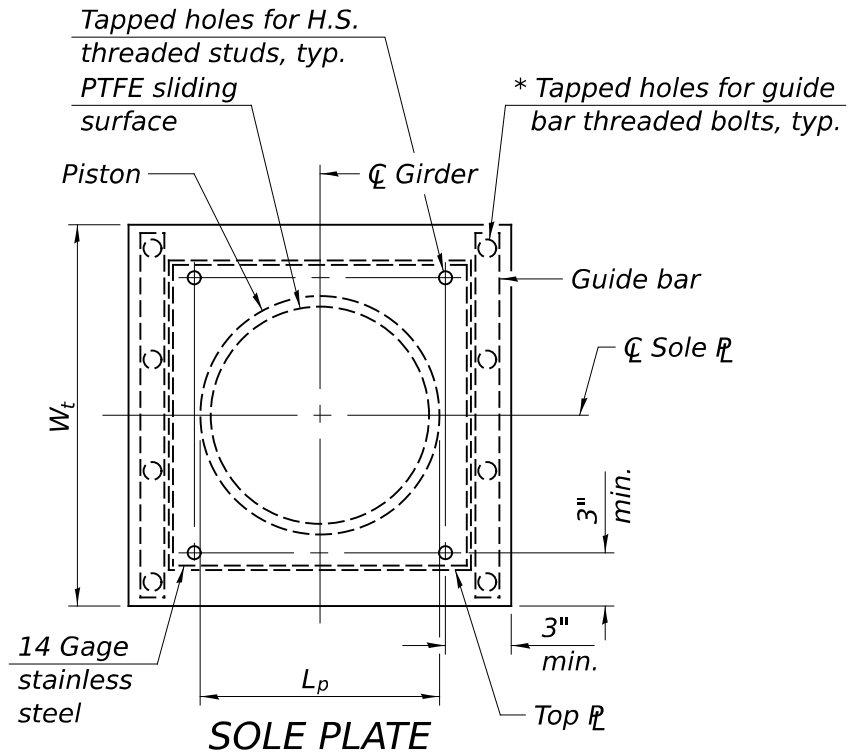
SECTION THRU BEARING

* As alternates to the bolted connection shown, the guide bars may be connected to the top bearing plate by groove welds or the guide bars and top bearing plate may be fabricated as a single piece.

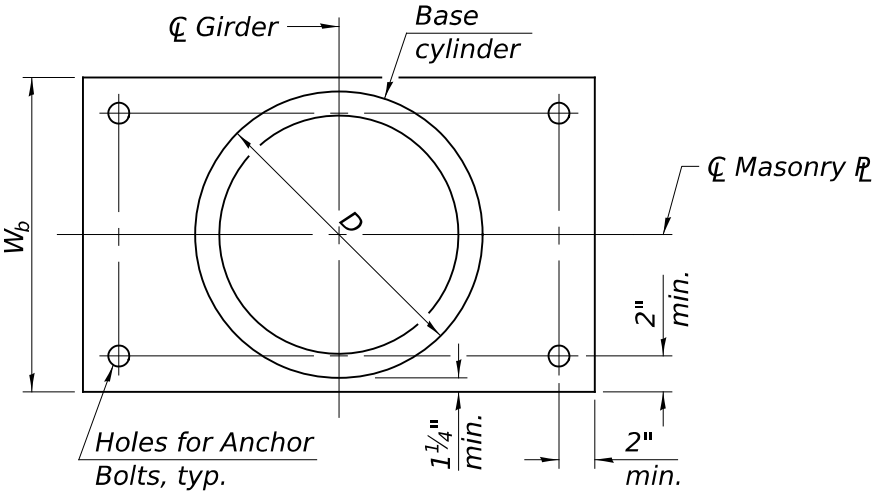
GUIDED EXPANSION
HLMR POT BEARINGS

Figure 3.7.5.4-2

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SOLE PLATE AND PISTON PLAN



MASONRY PLATE AND BASE CYLINDER PLAN

* As alternates to the bolted connection shown, the guide bars may be connected to the top bearing plate by groove welds or the guide bars and top bearing plate may be fabricated as a single piece.

GUIDED EXPANSION HLMR POT BEARINGS

Figure 3.7.5.4-3

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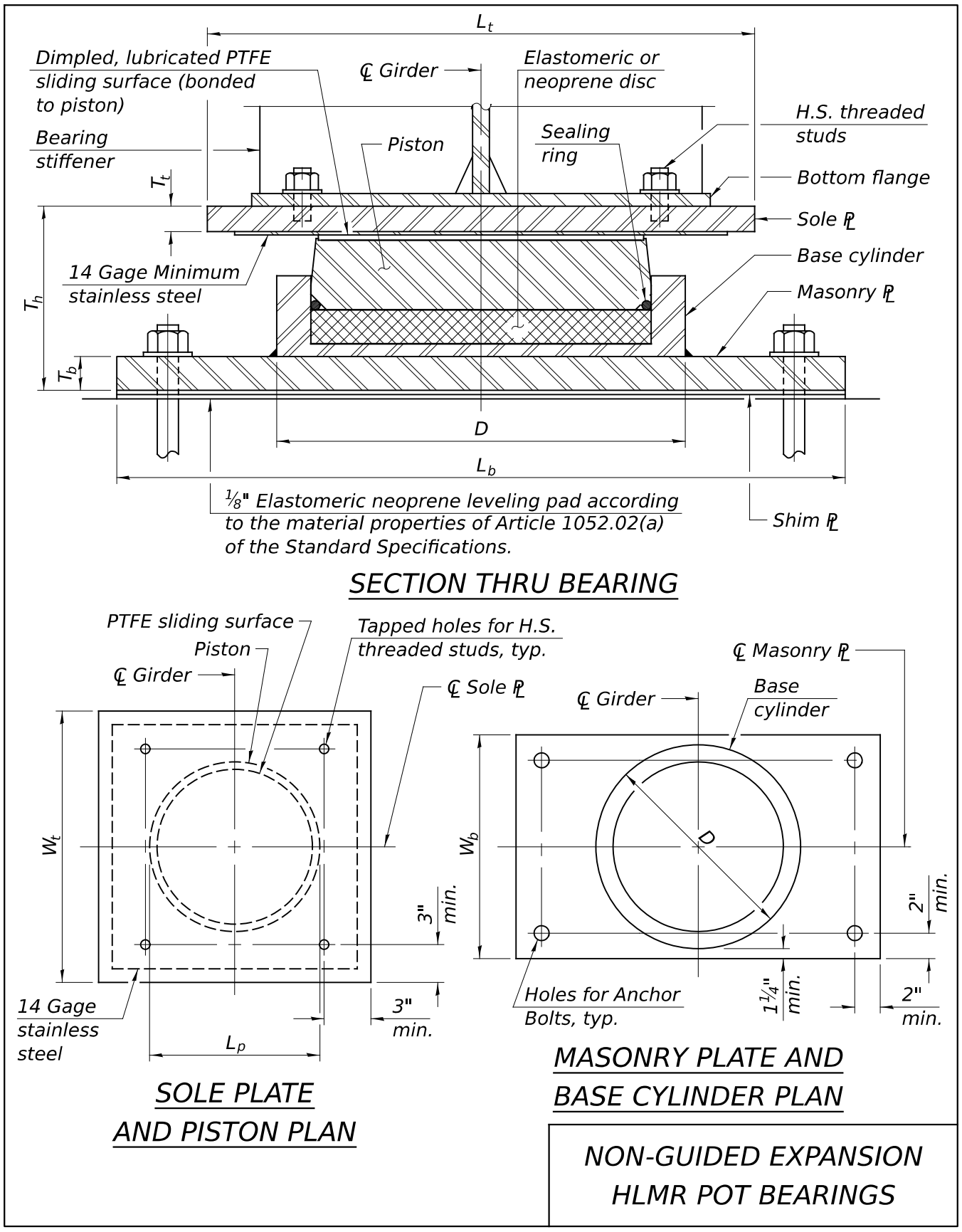


Figure 3.7.5.4-4

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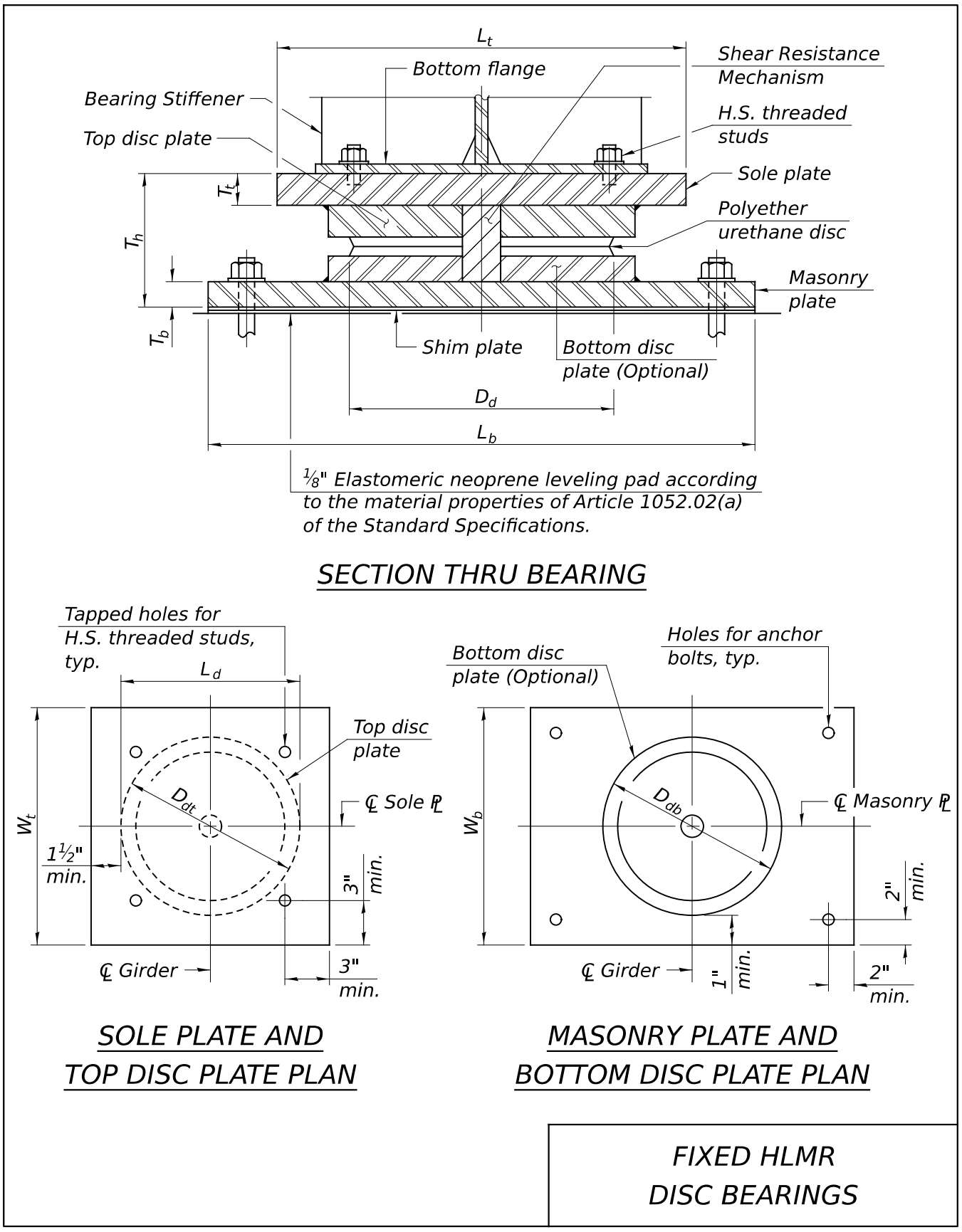
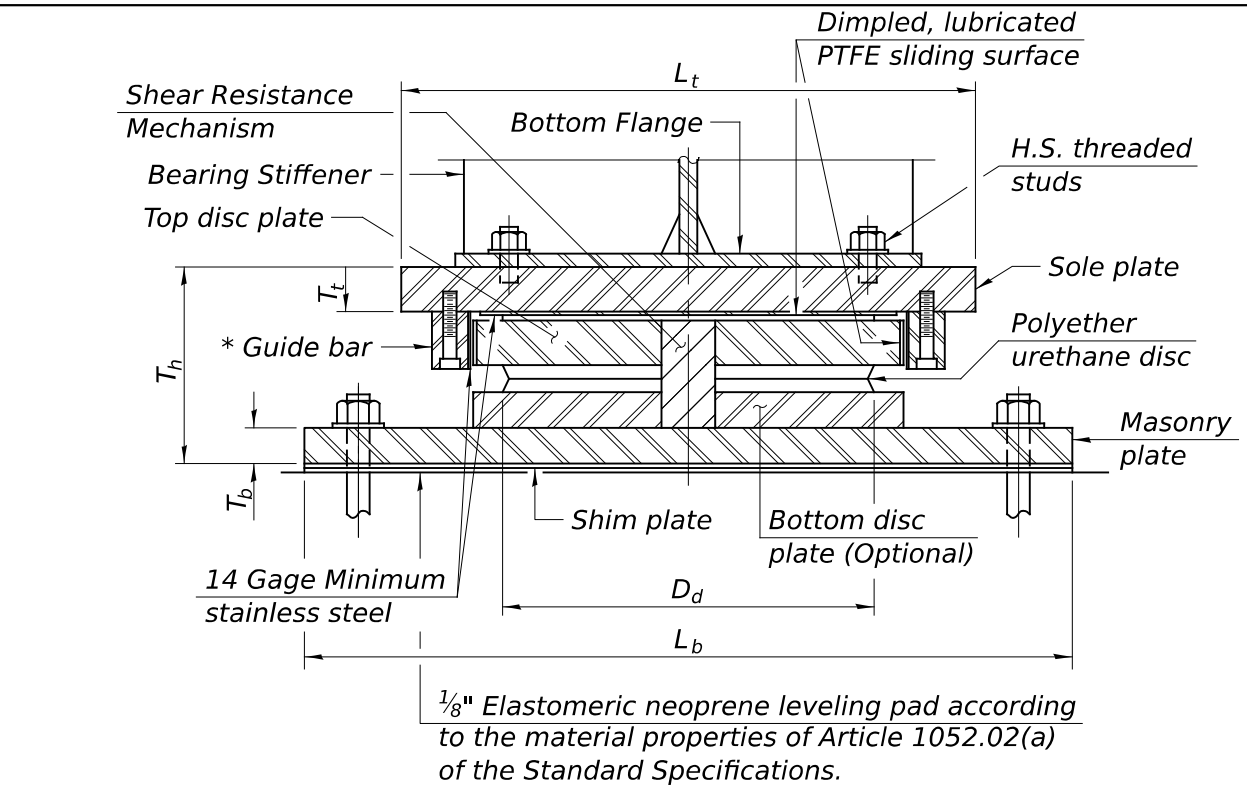
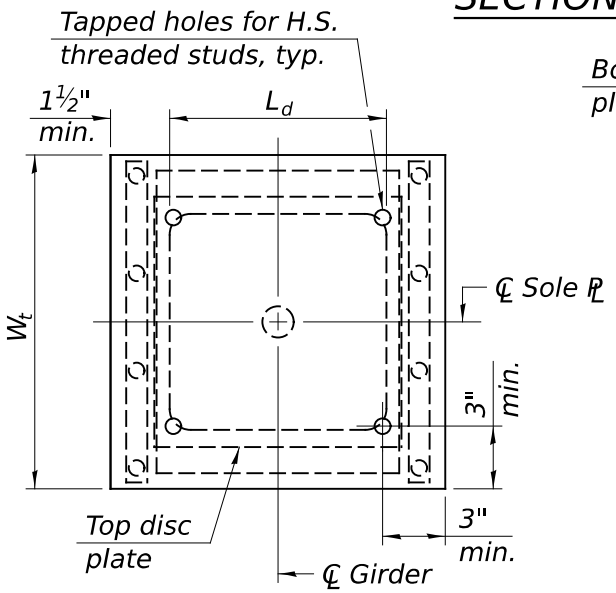


Figure 3.7.5.4-5

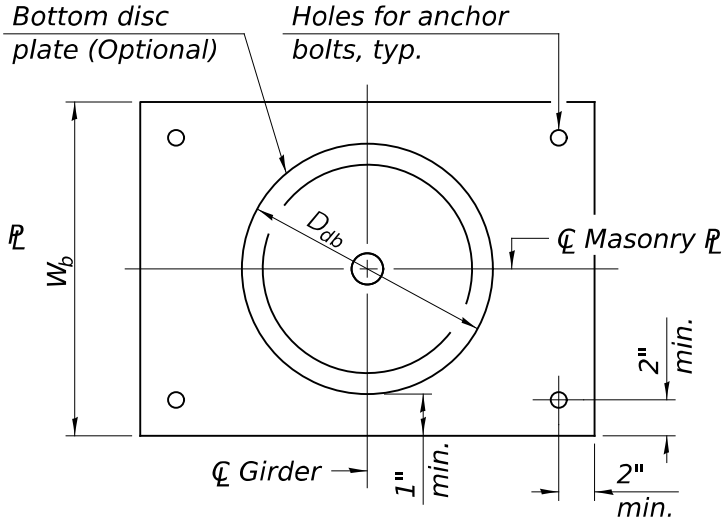
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SECTION THRU BEARING



SOLE PLATE AND TOP DISC PLATE PLAN



MASONRY PLATE AND BOTTOM DISC PLATE PLAN

* As alternates to the bolted connection shown, the guide bars may be connected to the top bearing plate by groove welds or the guide bars and top bearing plate may be fabricated as a single piece.

GUIDED EXPANSION HLMR DISC BEARINGS

Figure 3.7.5.4-6

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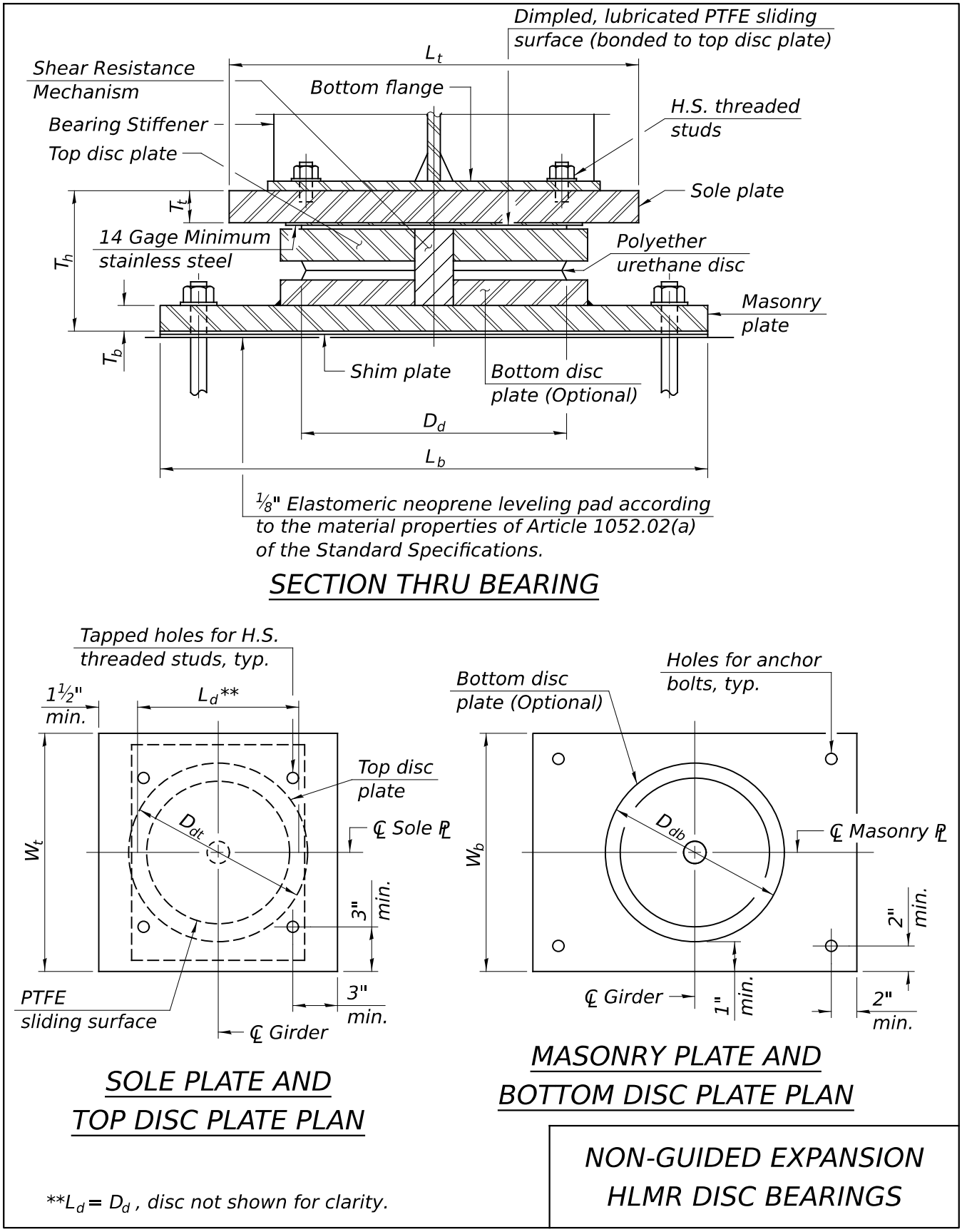


Figure 3.7.5.4-7

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3.7.5.5 Flange Connection Design

The bridge designer shall design the Sole Plate to flange connection for the Maximum Strength or Extreme Event Lateral Reaction. The bridge designer shall determine the number of stud bolts for the connection between the Sole Plate and bottom flange. A minimum of four stud bolts shall be used. Bridge designers may increase the size or number of stud bolts as required by design.

3.7.5.6 Anchor Bolt Design

Anchor bolts for HLMR bearings shall be designed according to Section 3.7.3 of the Bridge Manual. A minimum of four anchor bolts shall be used. Additional anchor bolts may be added as required by design.

Note that HLMR bearings and their connections to the substructure (anchor bolts) are intended to act as fuses during an earthquake just as elastomeric and fixed bearings are according to the Department's ERS philosophy. The lateral earthquake design force for SPZ 1 to 4 (LRFD) is 20% of the dead load reaction as described previously in Section 3.7.3 according to Departmental policy. Usually, this load is less than 10% of the total service vertical design load. HLMR bearing connections to the substructure should be designed for the greater of the earthquake (20% of dead load reaction) or service design lateral force for the HLMR bearing according to Section 3.7.3.

3.7.5.7 Plan Presentation

The bridge designer shall use the applicable base sheet for the bearing type and class selected. A separate base sheet shall be used for each bearing type and class. The bridge designer shall provide the bearing height, T_h and the Sole and Masonry Plate dimensions L_t, L_b, W_b, T_t, and T_b. When bearings of the same type and class, but with different loads, are required on the same bridge, the base sheet may be modified to account for the different bearing dimensions and loads on the same sheet.

The bridge designer shall specify the diameter, grade, and number of anchor bolts on the plans.

The bridge designer shall specify the diameter and number of stud bolts for the Sole Plate to flange connection on the plans.

The bridge designer shall provide the required reactions, vertical and lateral loads, thermal movements, and rotations to complete the Design Data table on the base sheet.

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3.7.5.8 Completing the Bill of Material

The bridge designer shall complete the Bill of Material for each bearing, type and loads shown on the base sheet. The pay items will be for the type and class of bearing, bearing load capacity and anchor bolt required. Pay items will be available for the type and class of bearing and load capacity specified. The load capacities will be available in 100 kip increments up to 1000 kips. The load increment will increase in 250 kip increments thereafter. Please contact the Department if a bearing's load capacity pay item is not available, and a pay item will be created.

When determining pay items, the bridge designer shall round up the Service I Factored Vertical Reaction to the next largest 100 kips increment available (for bearings with Service I Factored Vertical Reaction up to and including 1000 kips) or next largest 250 kips increment available (for bearings with Service I Factored Vertical Reaction exceeding 1000 kips) to choose an approximate bearing capacity for the pay item. The pay item will be High Load Multi-rotational Bearing of the type, class and designated vertical capacity.

The Anchor Bolt pay item shall be for the diameter required.

Available HLMR bearing and anchor bolt pay items may be found on the IDOT web site at Letting Specific Items under Coded Pay Items.

3.7.5.9 Guide Bridge Special Provision (GBSP)

The Department's GBSP 13 (High Load Multi-Rotational Bearings) describes material, fabrication, installation and testing requirements for all types of HLMR bearings.

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3.8 Abutments

This section covers the Department's policies related to substructure components and elements supporting the ends of a superstructure that transfer loads to the foundation piles, shafts, or footings. Detailing that meets the base sheet requirements and policies of the Department comprise the standard basis for design.

3.8.1 General

Common abutment types built in Illinois fall into three main categories. These are open, closed, and mechanically stabilized earth (MSE). Historically, open abutments have also been referred to as pile bent or spill through. Open includes integral, semi-integral and stub abutments. Closed abutments are restrained top and bottom or cantilever. Abutments with MSE components can be detailed with integral, semi-integral, stub, or spread footing layouts.

Vaulted abutments remain present in Illinois' current inventory but have been uncommon for new construction projects. Vaulted abutments are categorized as filled or unfilled.

The dynamic load allowance (impact) shall not be included in the design of elements below the bearing. Impact, however, shall be included in both the substructures and foundations when:

- the superstructure is poured monolithically with the substructure(s) on which it bears.
- there is a concrete diaphragm in the superstructure that sits on the substructure cap (see the base sheets of the "Diaphragms-Integral Abutment" IDOT CADD Bridge Cell Library).

If the length of an abutment is greater than 90 ft., a 1 in. expansion relief joint should be located between bearings near mid-length. The joint shall be full depth of the abutment unless a proposed footing is also constructed, for which the joint shall terminate at the top of the footing. On stage constructed abutments, a total width of 120 ft. or less may be permitted without requiring a 1 in. expansion joint. The base of an abutment under a superelevated roadway shall be constructed level if the difference between the low and high elevation of the bridge seat is 1 ft. – 6 in. or less. For a difference in elevation greater than 1 ft. – 6 in., the base of the abutment cap shall be stepped with the reinforcement continuous through the transitions. The base of the abutment cap shall always remain level in areas other than sloped transitions.

Abutments under deck joints shall have all exposed surfaces of backwalls, bridge seats, and front faces of pile caps treated with Concrete Sealer. For cases involving both existing and new concrete, such as structure widening, Concrete Sealer shall only be applied to new concrete. A film-forming Concrete Sealer shall be used for horizontal surfaces.

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3.8.2 Reinforcement

All reinforcement bars in abutment elements of IDOT maintained structures, shall be epoxy coated unless an alternate protective system (ASTM 1035, TEC, Galvanized, etc.) is approved by the Department.

For abutment caps continuously supported by (i) conventional piling and (ii) solid wall encasement or prepared subgrade, distribution reinforcement in Article 5.6.7 of the AASHTO LRFD Bridge Design Specifications need not be checked because flexural tension is low, and the typical #7 p(E) bars have been sufficient from the Department's years of experience. Additional longitudinal cap reinforcement should be provided to address temperature and shrinkage requirements if the designer's calculated spacing exceeds 18".

Placement of reinforcement in abutment caps shall be detailed so as not to interfere with anchor bolt locations. Multiple reinforcement layers should be considered to alleviate congestion.

3.8.3 Open Abutments: Integral

The preferred open abutment type for bridges built in Illinois is integral if the limitations detailed in Section 2.3.6.2.1 are satisfied.

Bridges designed with expansion joints and other structural releases allow the superstructure to expand and contract freely with changing temperatures. Integral abutment bridges eliminate expansion joints at the ends of bridge decks. Analysis of the thermal forces introduced into bridge elements when expansion joints and other structural releases are omitted is not required on structures which meet the limitations of Section 2.3.6.2.1.

Interactive 3D examples of various bridge or component models will be placed at the primary IDOT CADD Page as they become available.

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When utilizing integral abutments, the design considerations should be made as follows:

General

1. Abutment and Backfill Treatments herein are for all structures being designed using an LRFD force-based design approach when approved by the Department.
2. Abutment and Backfill Treatments using current Displacement-based seismic design approaches in SDC B-D shall be designed using Section 8 of the IDOT Seismic Manual and the latest seismic memorandums found at the primary Bridges & Structures Page.
3. Integral abutments are unique for using a single row of single piles beneath each bridge beam. See the "Bridge Stiffness and Bearing/Pile Design" section below.

Bridge Geometry and Orientation

4. Total structure length is 610 ft. maximum. The maximum simple span length for an integral abutment structure is 170 feet. The maximum end span for in a multi-span integral abutment structure is 200 feet.
5. Abutments shall be parallel to each other. Flared superstructures are permitted. Integral structures with curved girders are not permitted.
6. Abutments shall typically be provided with "dog-ear" type wingwalls. The length of these wingwalls shall be limited to 10 ft. If wingwall lengths greater than 10 ft. are required, the dog-ear wingwall lengths should extend to 10 ft. with the remaining "wing extension" lengths retained by independent walls, gabions, or rip rap that allows soil to wrap around wingwalls as shown in Figure 3.8.3-1. The joint between the dog ear wingwall and the wingwall extension shall be sealed with a fabric reinforced elastomeric mat. Minimum wingwall extension length shall be 4 feet. Wingwalls on skewed structures shall be placed parallel to the centerline of the abutment; however, they may be placed at acute angles to the centerline of the roadway as required with slopes not to exceed 2:1. For design, k_0 (earth pressure at rest) = 0.5 should be used and the equivalent fluid pressure should be 60 pcf.

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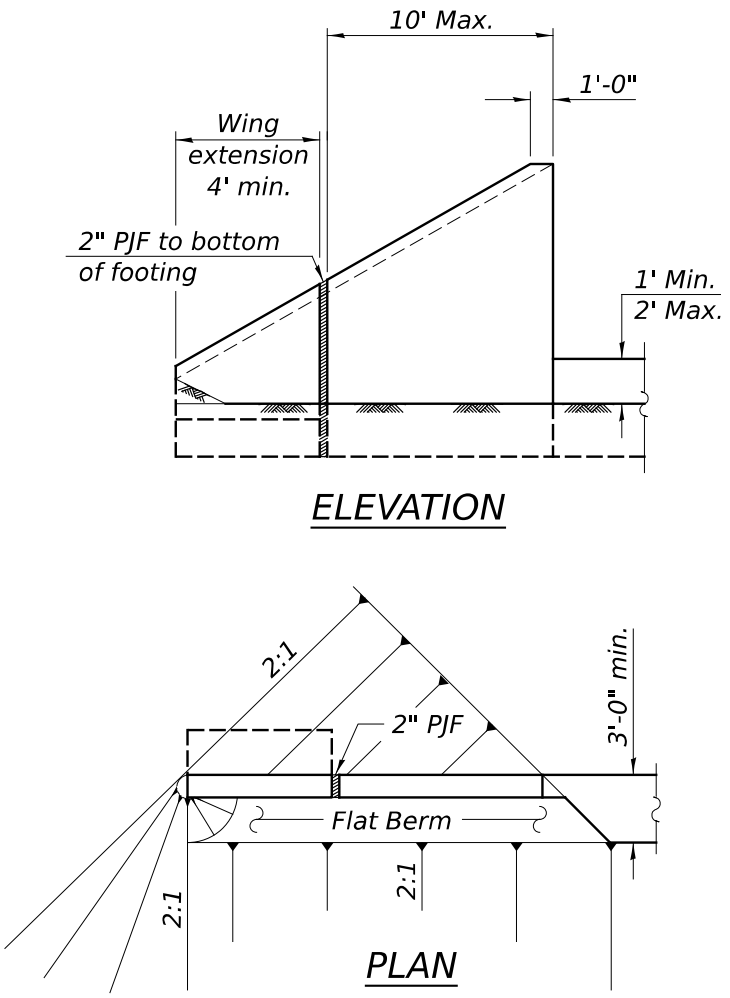


Figure 3.8.3-1

Drainage and Foundations

7. Backfill shall be Granular Backfill for Structures according to Section 586. The backfill is not required to be compacted, but can be compacted as per 205.06 of the Standard Specifications for design of seismic dense-material “springs.” The backfill shall be compacted for structures with steel railings extending beyond the bridge when a bridge approach slab is not provided. A note shall be provided on the plans to require compaction to override Section 586 of the Standard Specifications. The compaction provides for secure driven posts and was accounted for in design assumptions. See Figure 3.8.3-2 for details of abutment and backfill.

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Detailing (General)

8. Pile reinforcement shall be provided in all metal shell piles at abutments as shown in the "Piles" IDOT CADD Bridge Cell Library. Additional reinforcement is provided in the top portion of the pile to provide corrosion protection as detailed on the base sheet (to account for section loss already assumed in the Departments "Integral Abutment Feasibility Analysis" spreadsheet in Section 2.3.6.2.1). For pile sizes HP 12x74 piles and larger, and for pile sizes MS14x0.25 and larger, additional spiral reinforcement is required to help resist the moment at the pile/cap interface as shown in Figure 3.8.3-3 and Figure 3.8.3-4.

9. Headed bars shall be placed according to Figure 3.8.3-6 for steel and PPC beams.

Bridge Deck and Diaphragm

10. The bridge deck is required to be cast prior to backfilling behind the abutment and casting the bridge approach slab. The entire diaphragm shall be cast with the bridge deck.

Detailing - Steel Superstructures

11. Steel beams shall be detailed as shown in Figure 3.8.3-3 and Figure 3.8.3-11 (similar detailing for semi-integral abutments is shown in Figure 3.8.4-3). All steel beams are required to have bearing stiffener placed at right angles to the beam web and 10" from the beam end. This stiffener will provide anchorage into the abutment diaphragm. The stiffener plate shall be 1" thick for wide flanges and a 1" minimum or by design for plate girders. The bearing plates shall be detailed on the plans similar to Figure 3.7.1.2-2.
12. To ensure stability during construction, interior cross-frames/diaphragms shall be provided near the abutments for all steel beams and plate girders. The cross-frames/diaphragms shall be placed as shown in Figure 3.3.22-1.

Detailing – PPC Beam Superstructures

13. PPC beams shall be detailed as shown in Figure 3.8.3-4 and Figure 3.8.3-12.
14. Abutment width and sloping of the abutment caps for PPC beams on large grades shall be detailed according to Figure 3.8.3-8.

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Detailing – Slab Superstructures

15. Slab bridges shall be detailed as shown in Figure 3.8.3-5. Single unit slab bridges are considered integral structures and are limited to a total structure length of 130 feet and a maximum individual span of 40 feet. Precast approach slabs are not required for slab bridges. The slab shall be connected to the abutment cap with a minimum of #5 bars at 12 in. cts. However, no reinforcement shall connect the slab to the approach. The s2(E) bar shown in Figure 3.8.3-5 is only required when the bridge slab thickness is greater than the approach slab thickness. A single row of piles is preferred for fixed conditions.

Skew Accommodation

16. Skewed integral abutments shall be detailed according to Figure 3.8.3-7.
17. Layout and top flange clipping for steel or PPC beams shall be detailed according to Figure 3.8.3-11 through Figure 3.8.3-14 to eliminate interference with approach slabs for high skews.

Bridge Approach Elements

18. The approach slab ledge for integral abutments shall be detailed as shown in Figure 3.8.3-9.
19. Design considerations in Section 3.2.12 shall be followed.
20. A formed joint with bridge relief joint sealer shall be placed at the end of the bridge deck to provide a controlled crack for potential rotation.

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Bridge Stiffness and Bearing/Pile Design

21. Foundation shall consist of a single row of vertical H-piles or Metal Shell (MS) piles. Design reactions for the piles and abutment shall include impact. A pile is required under each beam. If more piles are required, additional piles shall be placed at the midpoint between beams. See Section 3.10 for information on the design of piles. If the unconfined compressive strength of the soil, Q_u, exceeds 3.0 tsf, the piles may be precored and filled with bentonite. Refer to the "Integral Abutment Pile Selection" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical. Applicable General Notes in Section 3.1.3 for "precoring of pile locations" shall be placed on the appropriate abutment sheets.
22. H-piles shall have their webs perpendicular to the centerline of structure as shown in Figure 3.8.3-10 and embedded 2 ft. minimum into the cap. The Integral Abutment Feasibility Analysis spreadsheet (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Foundations and Geotechnical) already analyzes the pile for strength and service loads and already accounts for corrosion.
23. The Department's integral abutment analysis considers the stiffness of the superstructure in addition to the abutment. The stiffness of the piers can also play a role. To minimize the stiffness effects at the piers, IDOT's preference is to utilize fixed bearings for flexible pier types such as pile bents and encased piles bent piers and expansion bearings for stiffer piers with footings, crash walls, drilled shafts, and rigid frame piers. The integral analysis also considers the variable stiffness of a potential different number of piles at each abutment, but does not consider the variable stiffness for different pile types at each abutment. It is recommended to design the same type of pile at each abutment for economy with the pile driving hammer. On rare cases when a different pile type is designed at each abutment, the designer shall evaluate the pile properties given in the "Integral Abutment Feasibility Analysis" spreadsheet, determine if a hammer is suitable for both pile types, and use engineering judgment to determine the effect on the integral analysis.

Integral abutments are permitted on MSE walls as discussed in Section 3.8.9.1.

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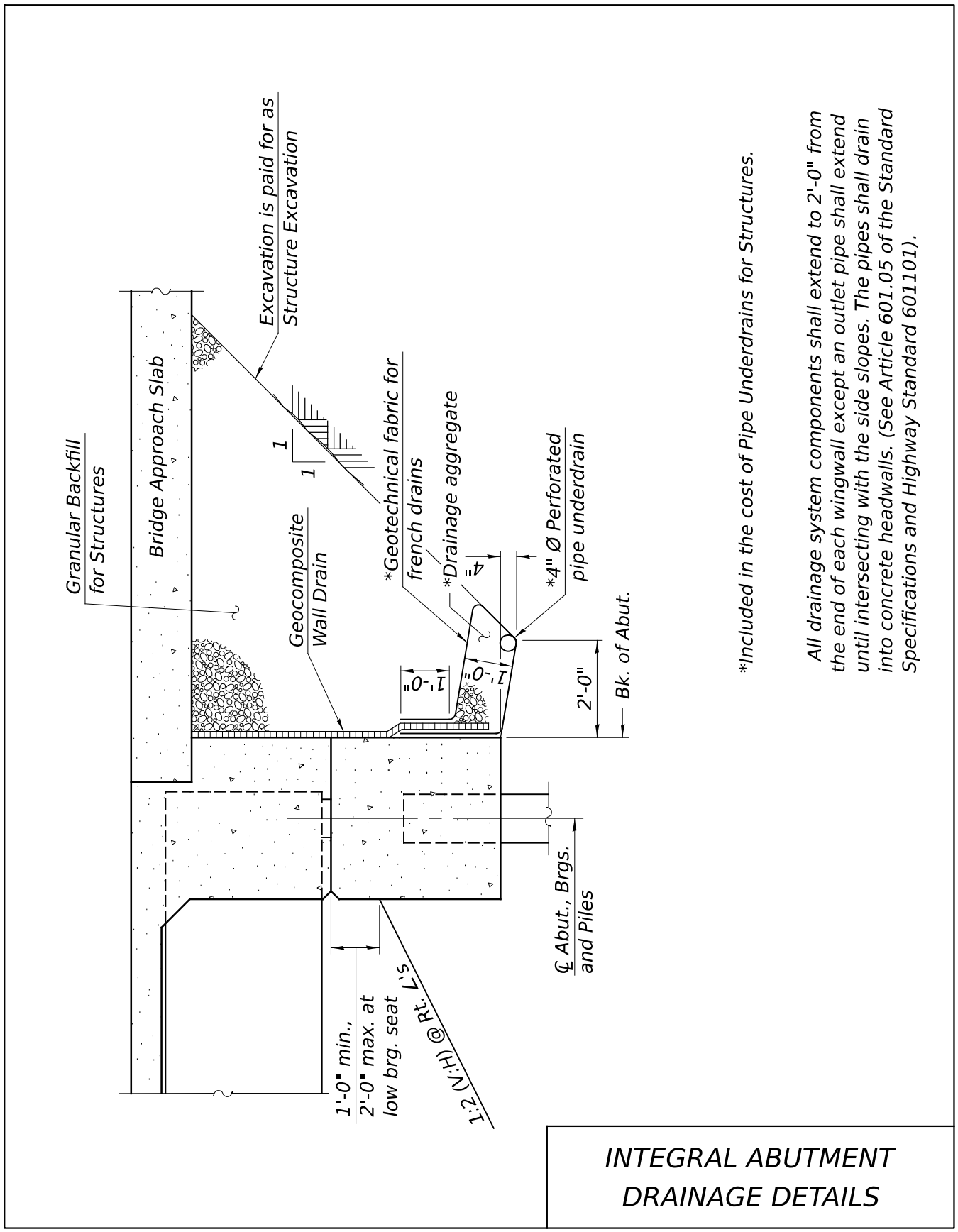


Figure 3.8.3-2

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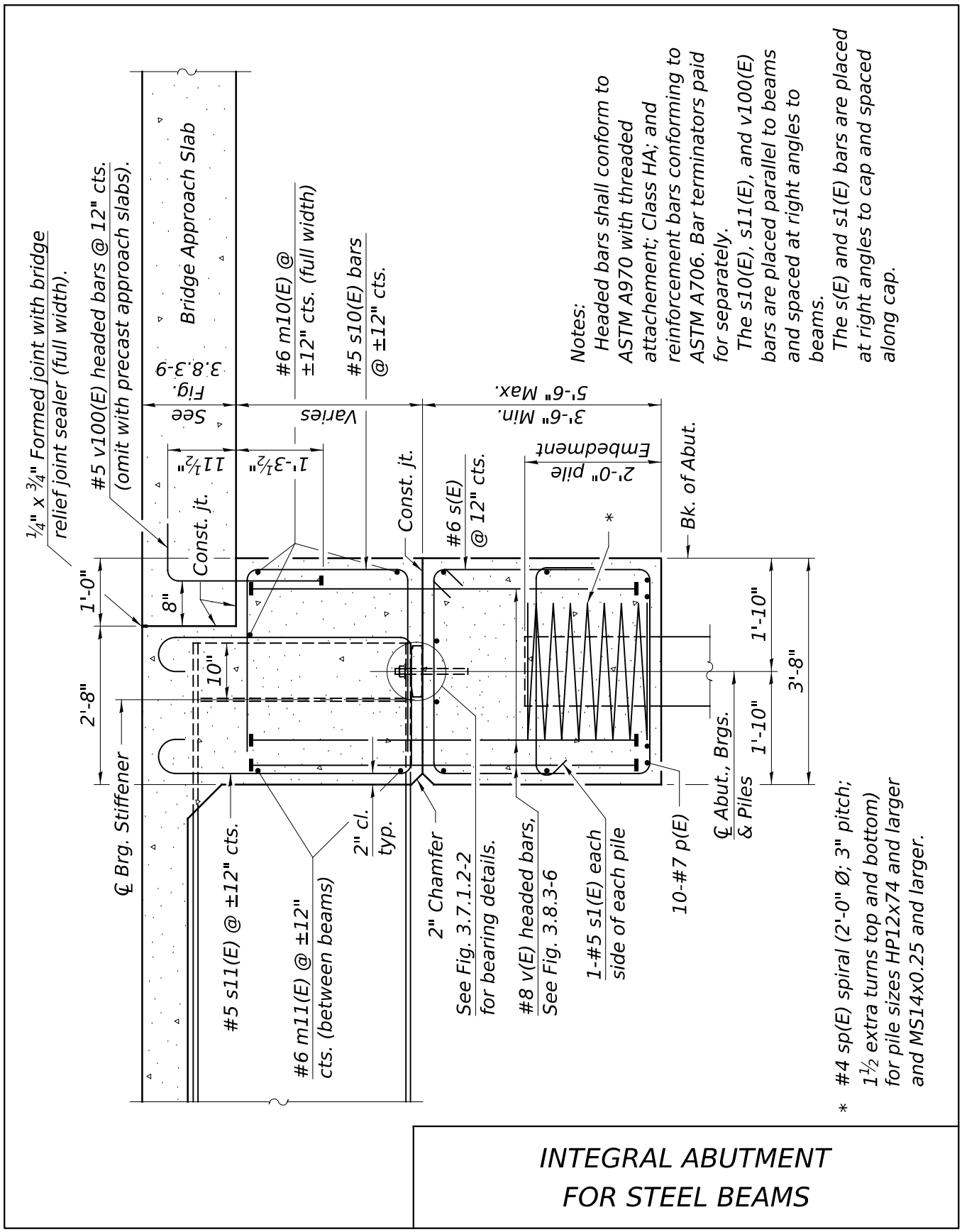


Figure 3.8.3-3

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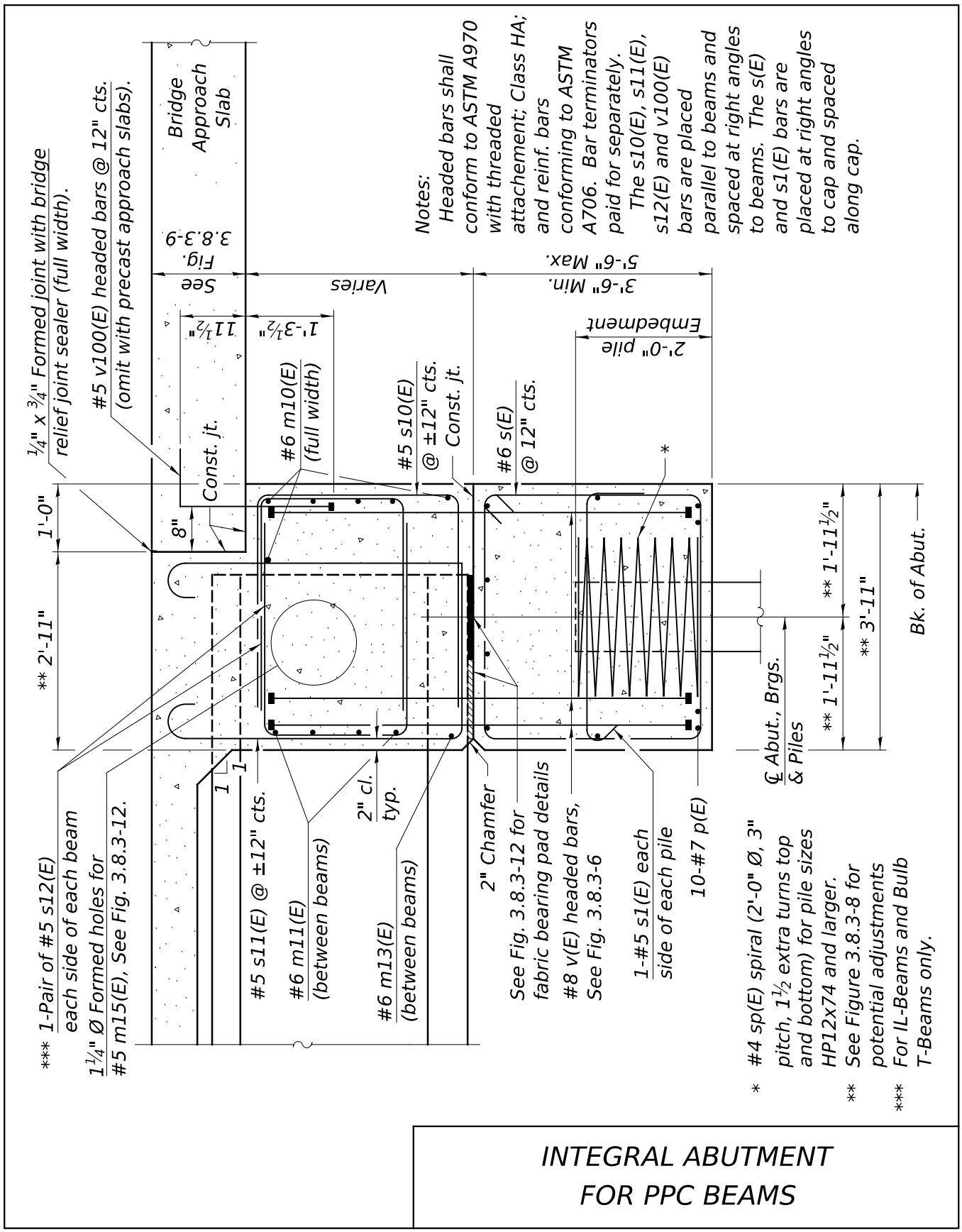


Figure 3.8.3-4

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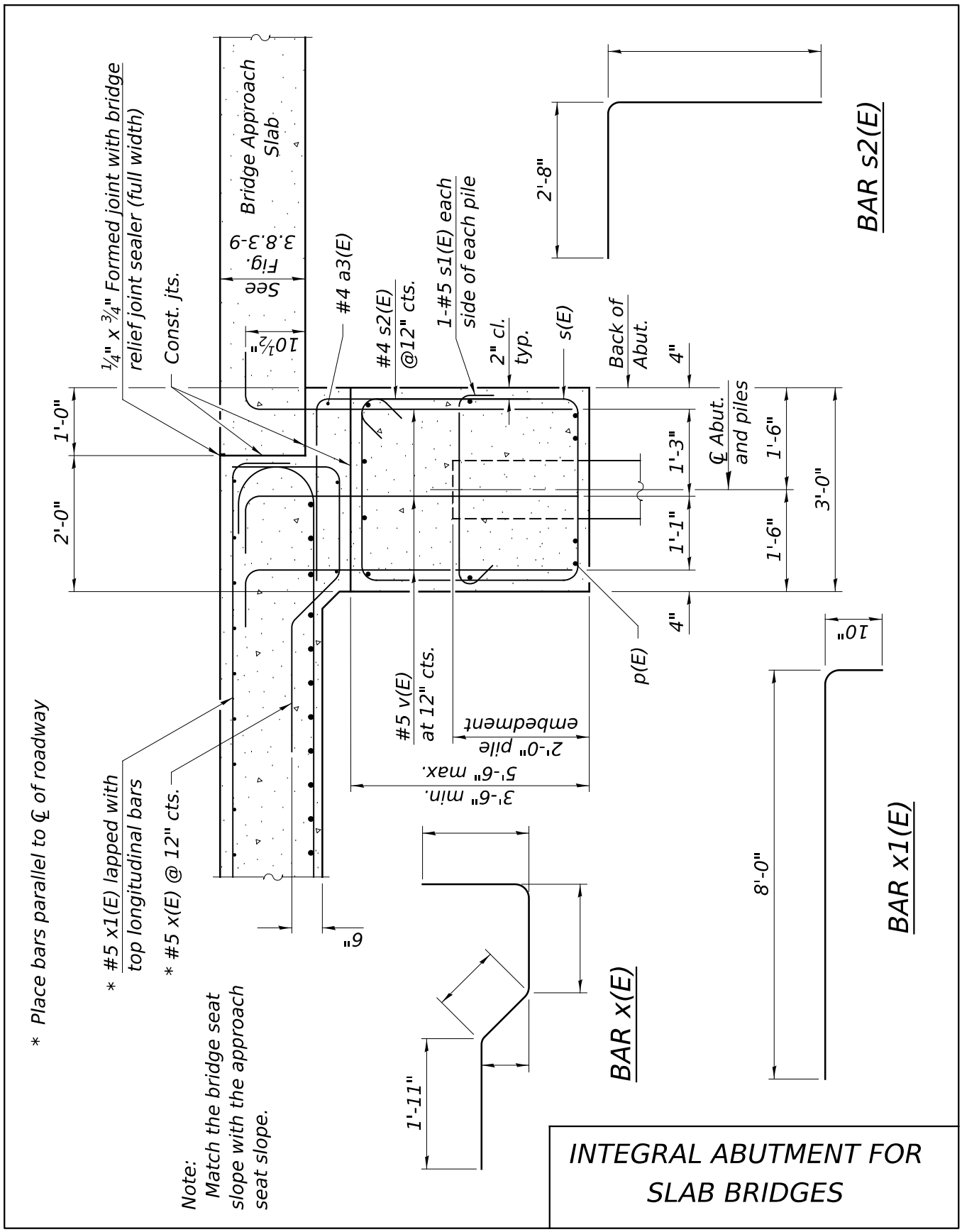


Figure 3.8.3-5

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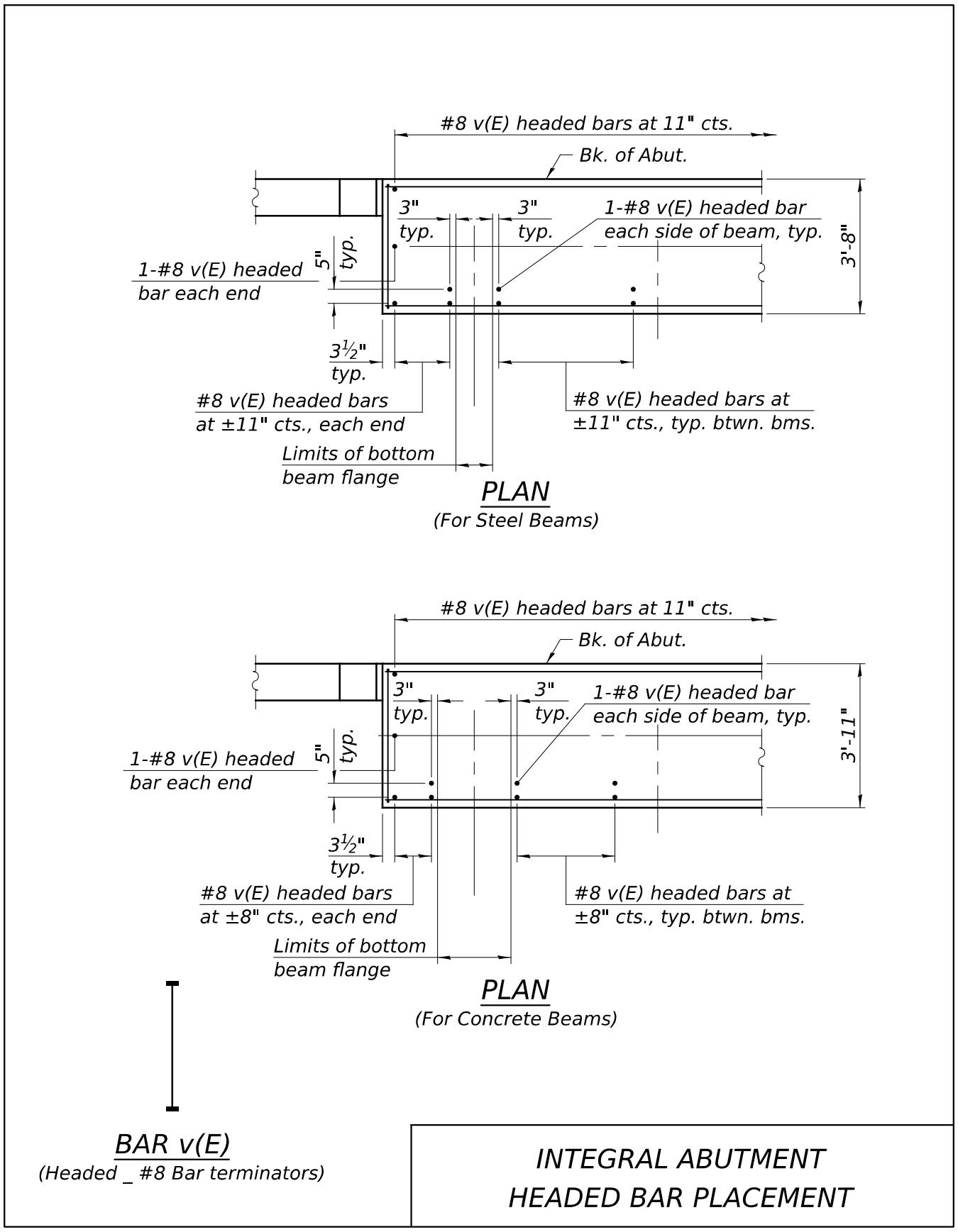
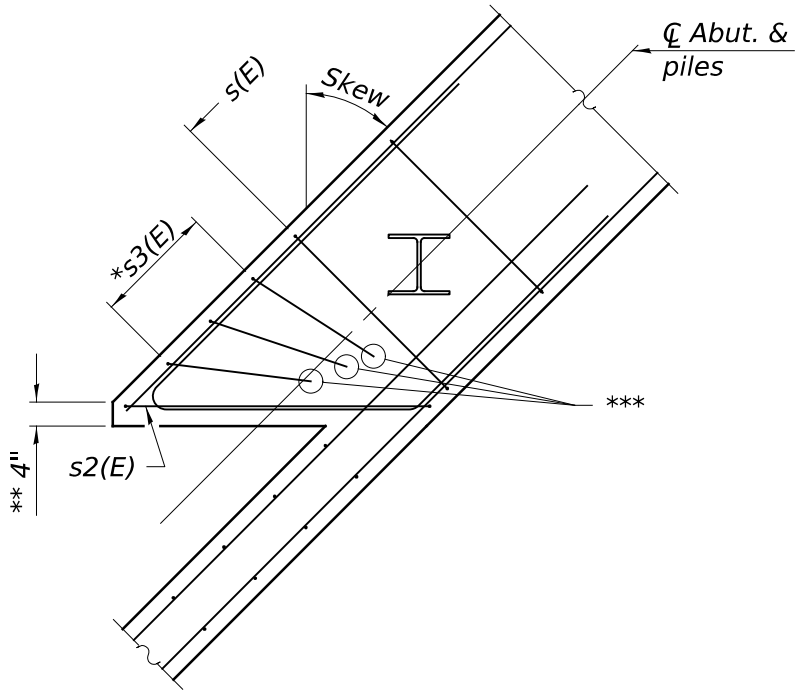


Figure 3.8.3-6

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ABUTMENT CAP PLAN

* *Add s3(E) bars when the spacing between s(E) and s2(E) bars exceeds what is required by design or 12".*

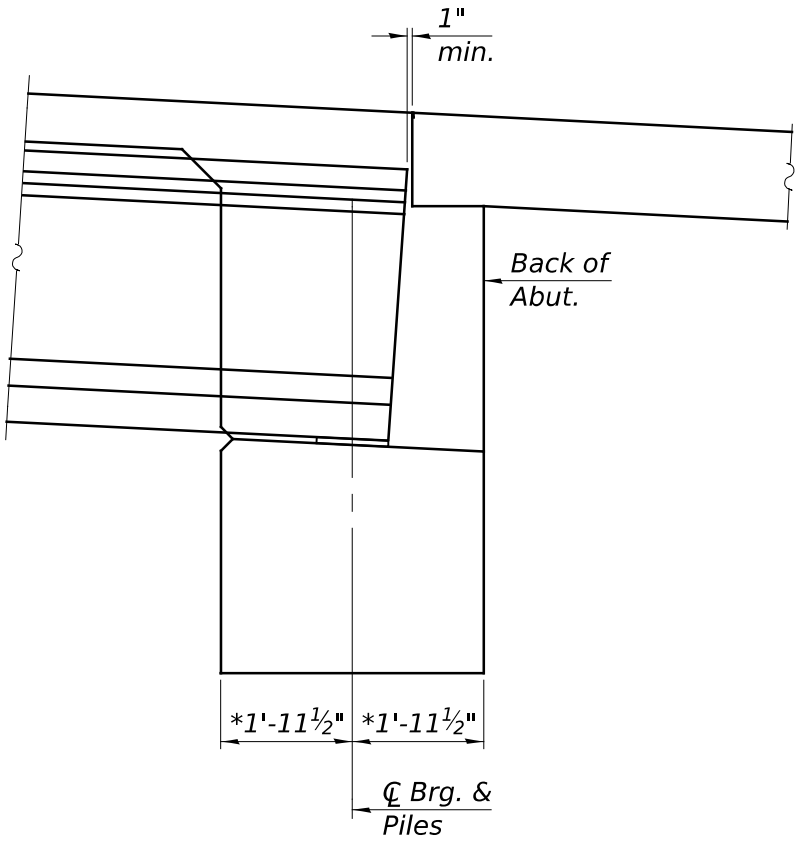
** *Block out sharp corner of abutment cap and diaphragm as shown for skews 25 degrees and larger.*

*** *Detail legs of s3(E) bars such that they do not all come to a point causing congestion.*

CORNER TREATMENT OF SKEWED INTEGRAL ABUTMENTS

Figure 3.8.3-7

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SECTION THRU ABUTMENT

* Increase as needed to maintain 1" minimum clearance between beam and approach pavement. The skew angle will also need to be considered (See top flange clip details) as well as the camber.

Notes:

The grade of the bridge shall be considered when detailing integral abutments with PPC beams. Since the beams are cast such that the beam ends are 90 degrees to the top and bottom flanges the beam ends will not be vertical if the beams are placed on a grade. In addition beam camber will cause further rotation of the beam end.

Abutment caps shall be sloped to match the grade when the change in elevation exceeds ¼" from the front face of abutment to the end of the beam. This is done to ensure the fabric bearing pad and polystyrene are under a more uniform bearing pressure. Reinforcement as shown on the base sheets may need to be adjusted to accommodate the slope.

Integral abutments with steel beams do not need these adjustments since the beam ends can be clipped vertically and the steel rocker plate can accommodate grades.

INTEGRAL ABUTMENT DETAILS FOR PPC BEAMS ON LARGE GRADES

Figure 3.8.3-8

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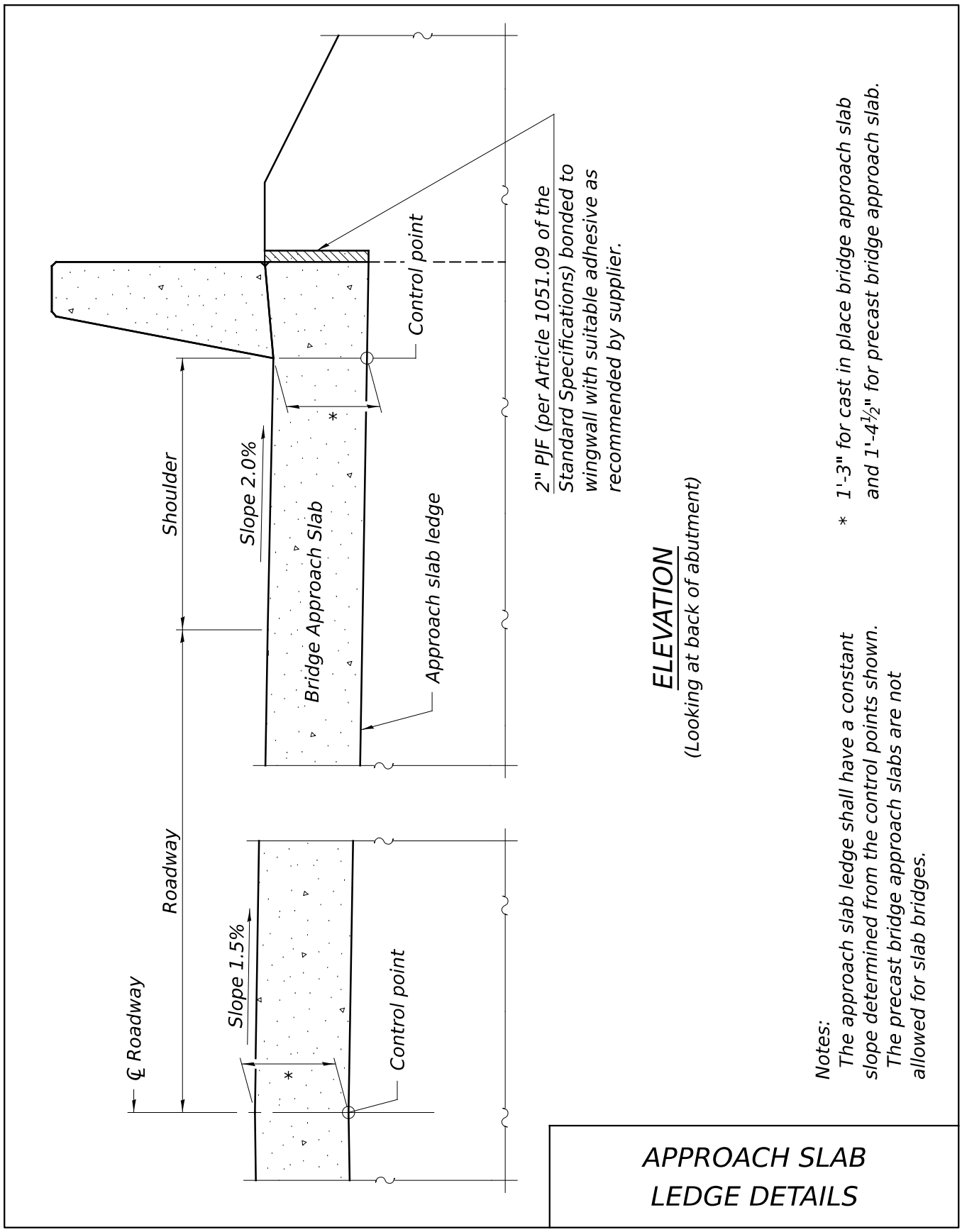


Figure 3.8.3-9

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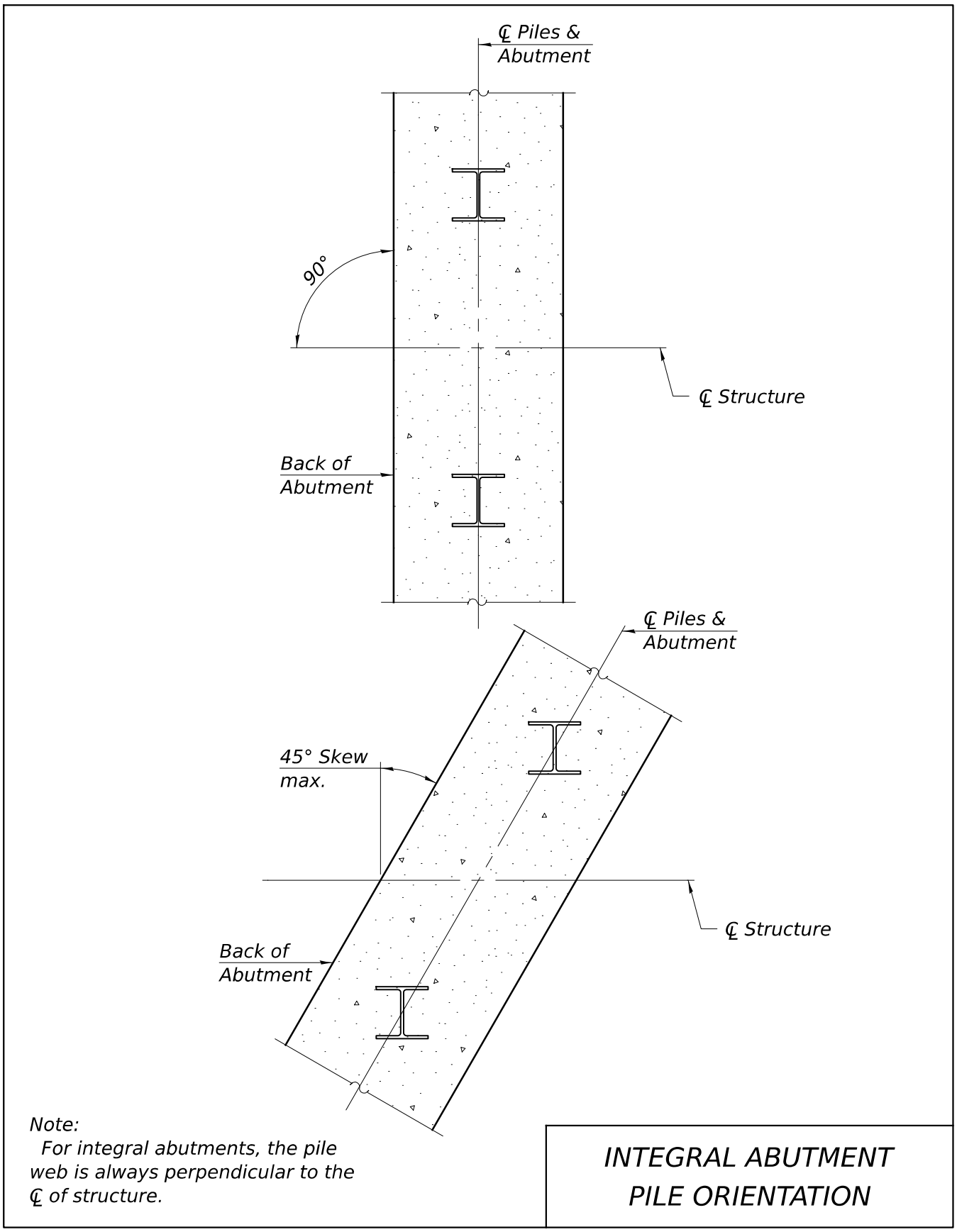
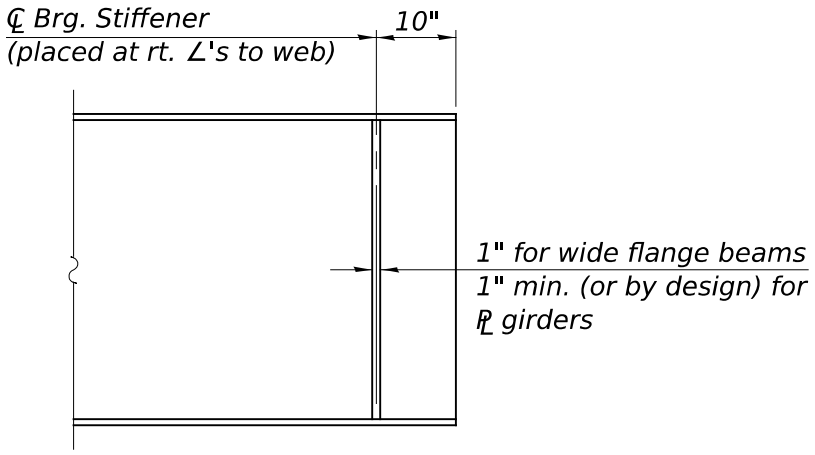
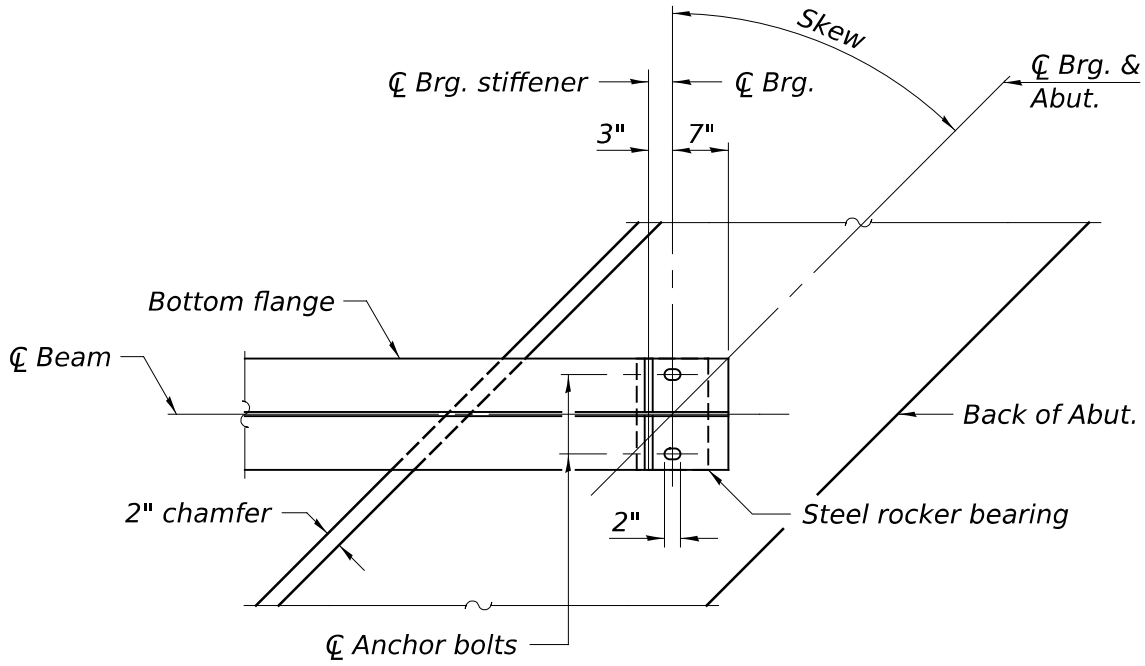


Figure 3.8.3-10

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END ELEVATION



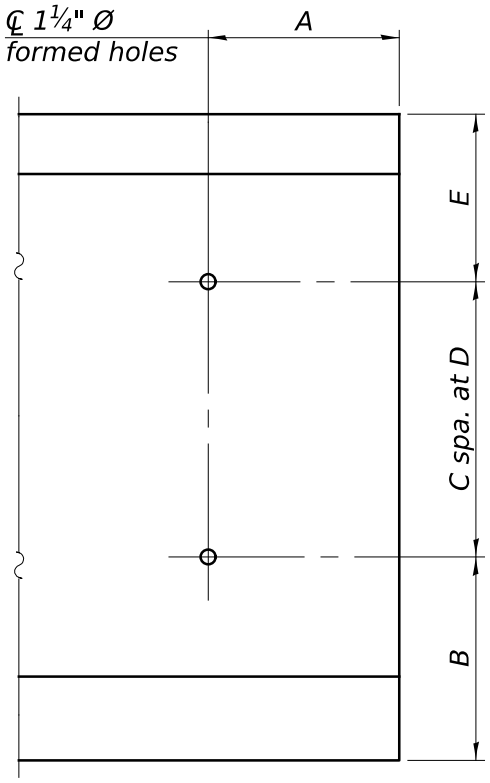
PLAN

Note:
See Figure 3.7.1.2-2 for bearing and anchor bolt details.

STEEL BEAM TO DIAPHRAGM CONNECTION DETAIL FOR INTEGRAL ABUTMENTS

Figure 3.8.3-11

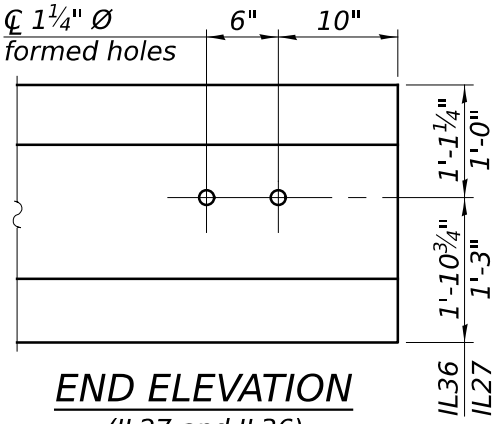
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END ELEVATION
(I36-I54, BT63-BT72 and IL45-IL90)

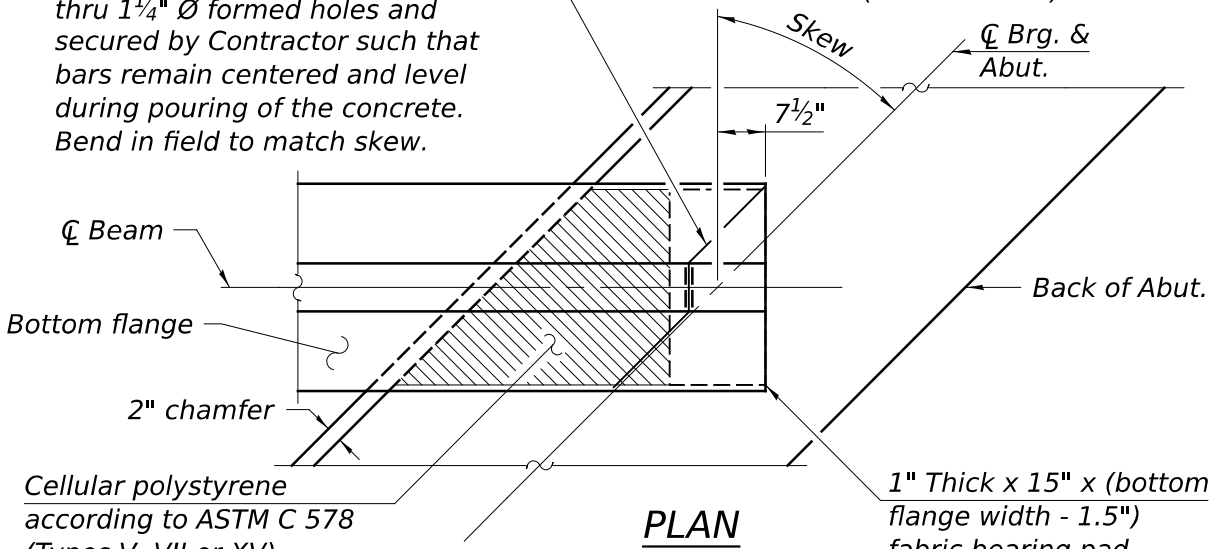
TABLE OF DIMENSIONS

Beam	A	B	C	D	E
I36	1'-0"	1'-3"	1	11"	10"
I42	1'-0"	1'-5"	1	1'-3"	10"
I48	1'-0"	1'-7"	1	1'-5"	1'-0"
I54	1'-0"	1'-7"	1	1'-11"	1'-0"
BT63	1'-4"	1'-5"	2	1'-4"	1'-2"
BT72	1'-4"	1'-5"	2	1'-8½"	1'-2"
IL45	1'-0"	1'-11"	1	8"	1'-2"
IL54	1'-0"	1'-11"	1	1'-5"	1'-2"
IL63	1'-0"	1'-11"	2	1'-1"	1'-2"
IL72	1'-0"	1'-11"	2	1'-5½"	1'-2"
IL81	1'-0"	1'-11"	4	11"	1'-2"
IL90	1'-0"	1'-11"	4	1'-1¼"	1'-2"



END ELEVATION
(IL27 and IL36)

#5 m15(E) bars (4'-0" long) placed thru 1¼" Ø formed holes and secured by Contractor such that bars remain centered and level during pouring of the concrete. Bend in field to match skew.



Cellular polystyrene according to ASTM C 578 (Types V, VII or XV). Provide slightly thicker piece than measured gap height to tightly fill the hatched area shown between abutment cap and bottom of beam.

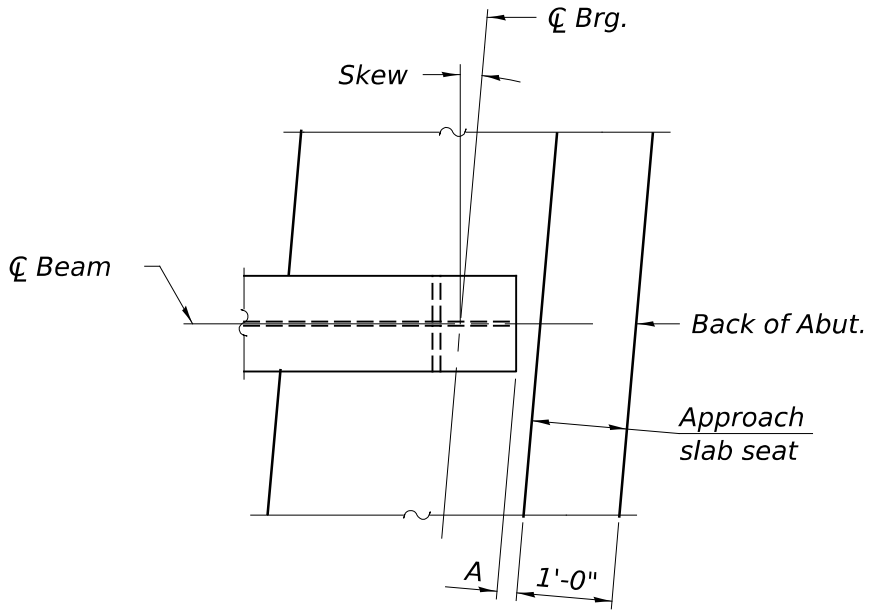
PLAN

1" Thick x 15" x (bottom flange width - 1.5") fabric bearing pad

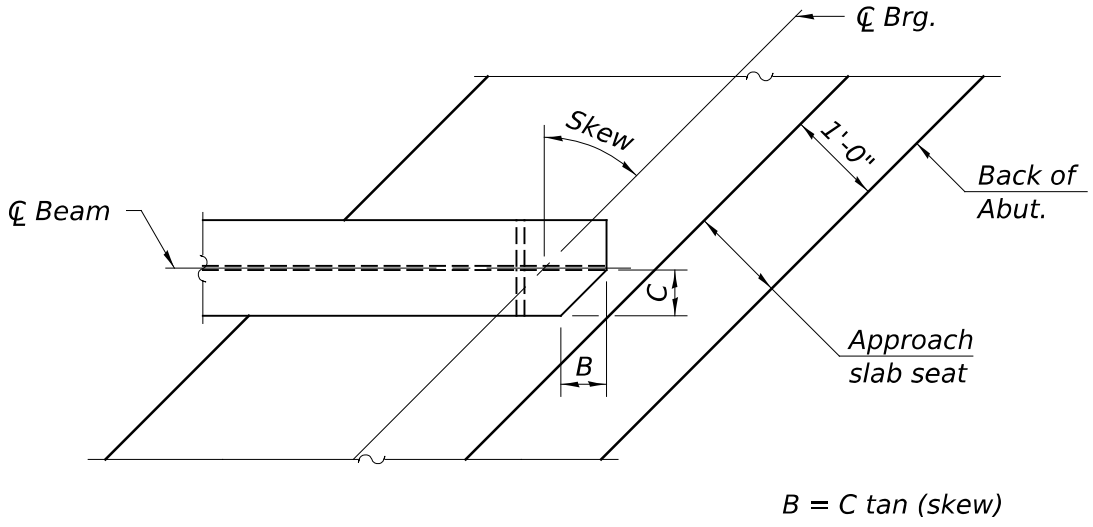
PPC BEAM TO DIAPHRAGM CONNECTION DETAIL FOR INTEGRAL ABUTMENT

Figure 3.8.3-12

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TOP FLANGE PLAN - NO CLIP



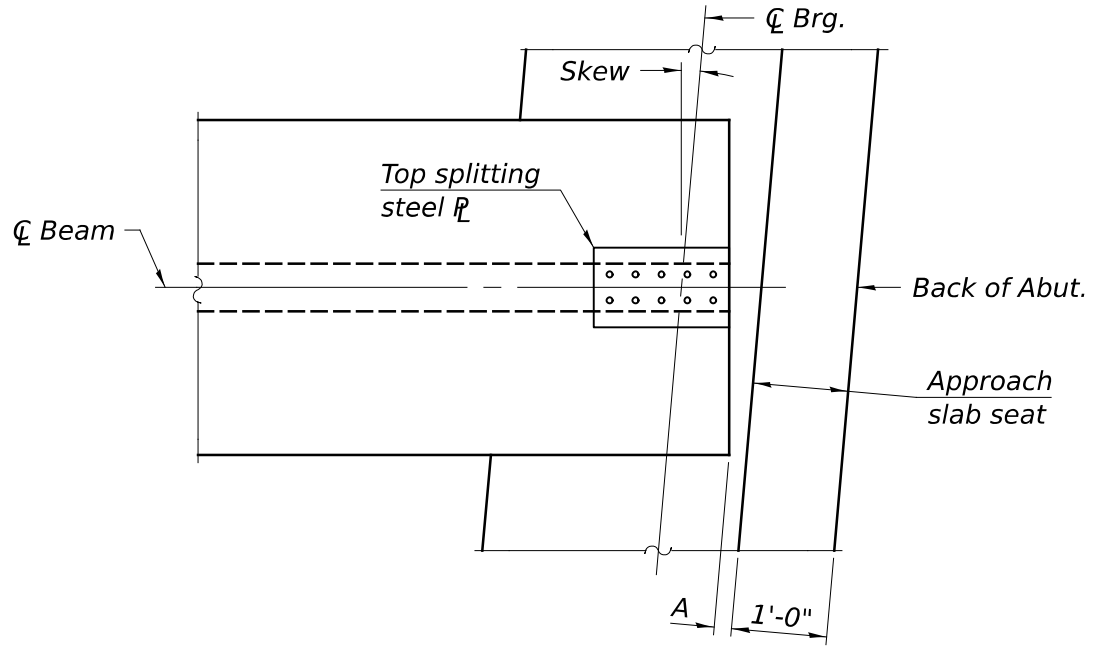
TOP FLANGE PLAN - CLIPPED

Notes:
Clip top flange when dimension "A" is less than 1". Calculate dimension "B" based on skew angle. Dimension "C" is half the top flange width minus half the web thickness.

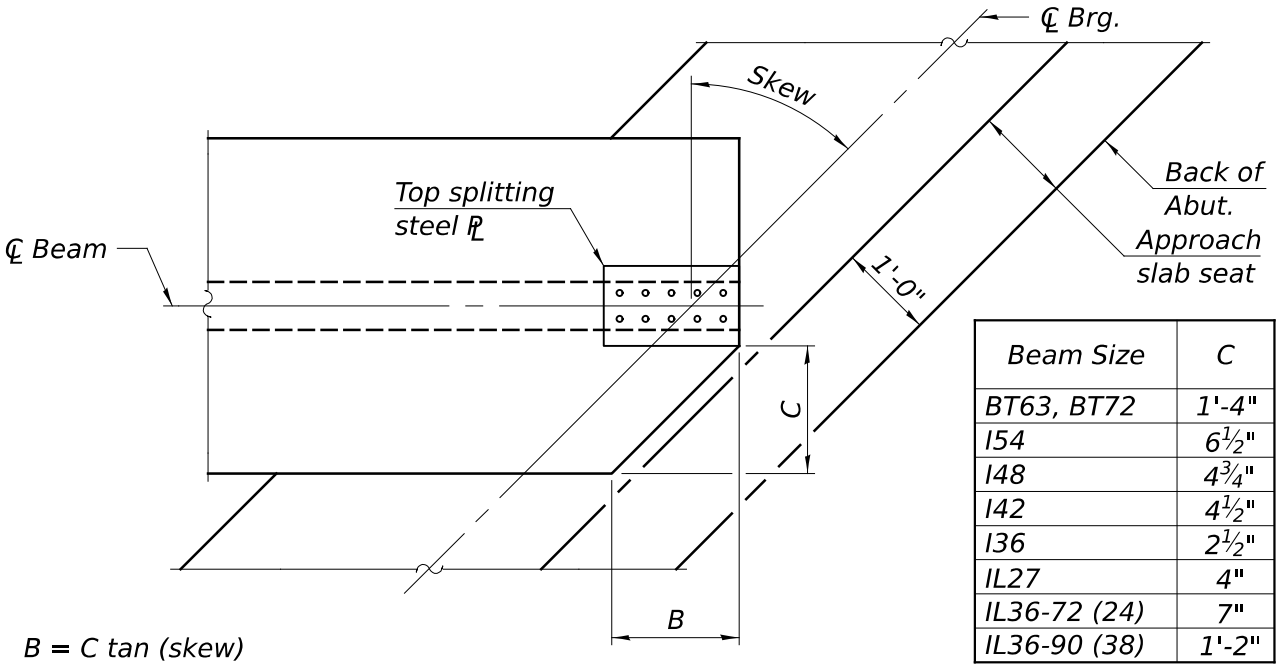
TOP FLANGE CLIP DETAIL
FOR STEEL BEAMS
ON INTEGRAL ABUTMENTS

Figure 3.8.3-13

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TOP FLANGE PLAN - NO CLIP



B = C tan (skew)

Beam Size	C
BT63, BT72	1'-4"
I54	6½"
I48	4¾"
I42	4½"
I36	2½"
IL27	4"
IL36-72 (24)	7"
IL36-90 (38)	1'-2"

TOP FLANGE PLAN - CLIPPED

Note:
Clip top flange when dimension "A" is less than 1". Calculate dimension "B" based on skew angle. Dimension "C" is provided in table.

TOP FLANGE CLIP DETAIL FOR PPC BEAMS ON INTEGRAL ABUTMENTS

Figure 3.8.3-14

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3.8.4 Open Abutments: Semi-Integral for New Construction

Semi-integral jointless abutment details are shown in Figures 3.8.4-1 through 3.8.4-7. Applications for these abutments are described in Sections 2.3.6.2.1 and 2.4.2.3. The general limitations for length/alignment/parallel-layout of integral abutment structures outlined in Section 2.3.6.2.1 are also applicable for semi-integral abutment structures. Wingwalls shall not be connected to the superstructure when semi-integral jointless abutments are used. Loads transferred from the bridge approach slab to the substructure shall be considered to act through the bearing, not the backwall.

To ensure stability during construction, semi-integral structures with new steel beams/girders shall be additionally braced as specified in "Detailing – Steel Superstructures" of Section 3.8.3. The reuse of existing cross-frames and diaphragms (due to interference with reinforcement bars for the concrete diaphragm, or where standard reinforcement bars need to be adjusted) on a structure which will be made semi-integral will be handled on a case-by-case basis with the concurrence of the BBS.

Interactive 3D examples of various bridge or component models will be placed at the primary IDOT CADD Page as they become available.

MSE walls for semi-integral abutments shall be detailed similar to integral abutments as described in Section 3.8.3. See Figure 3.8.4-8 for section details.

Semi-integral abutments are permitted on MSE walls as discussed in Section 3.8.9.2.

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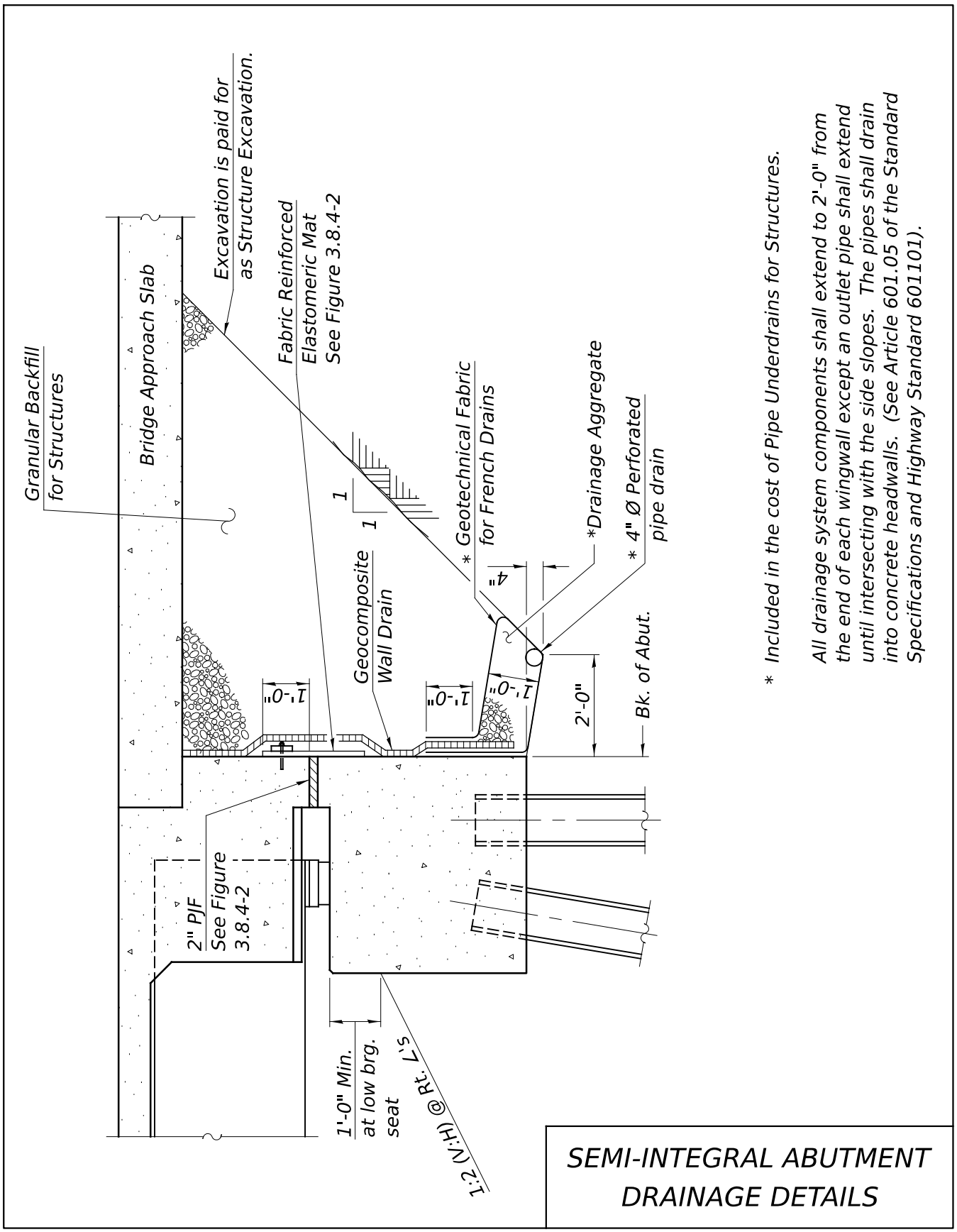


Figure 3.8.4-1

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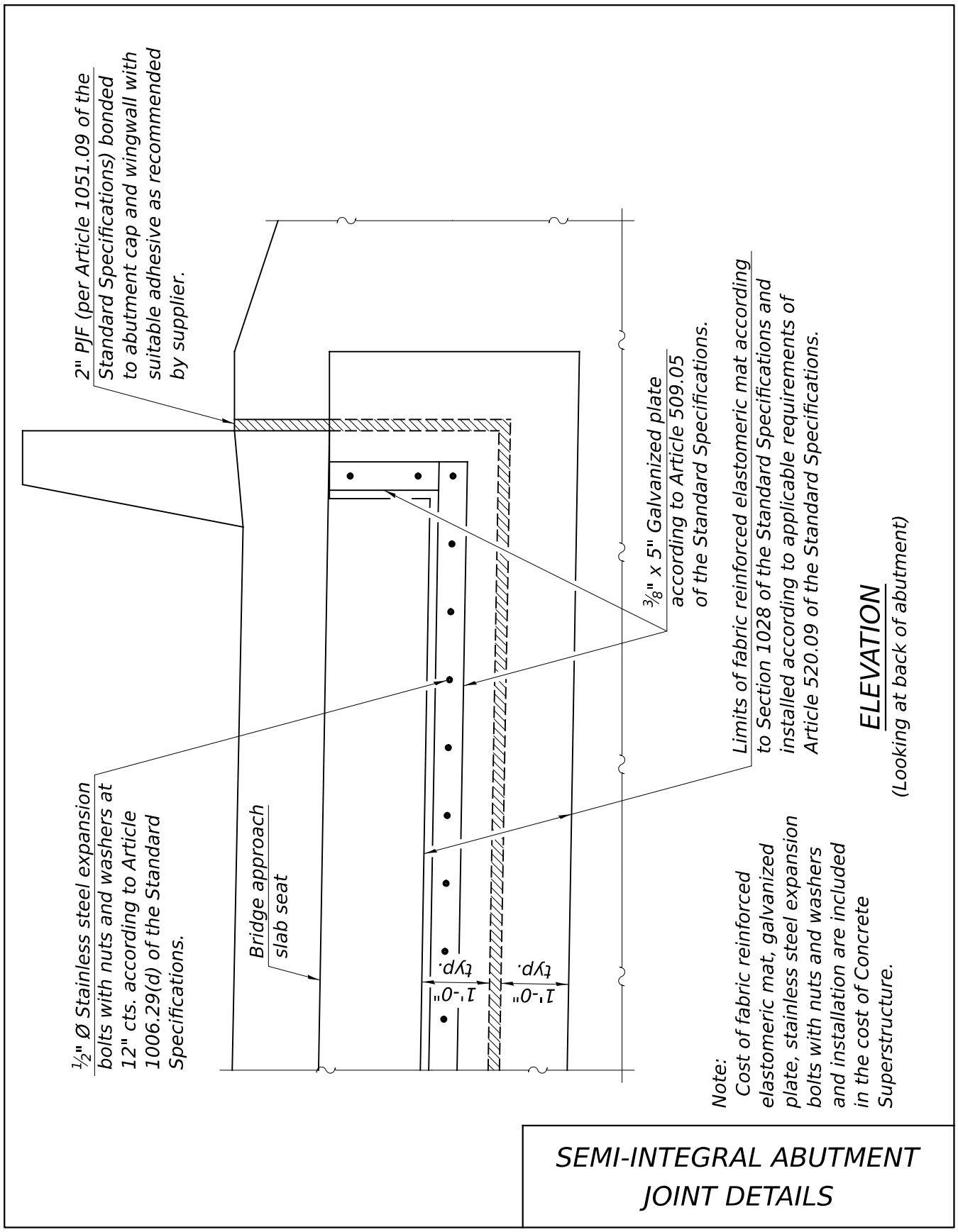


Figure 3.8.4-2

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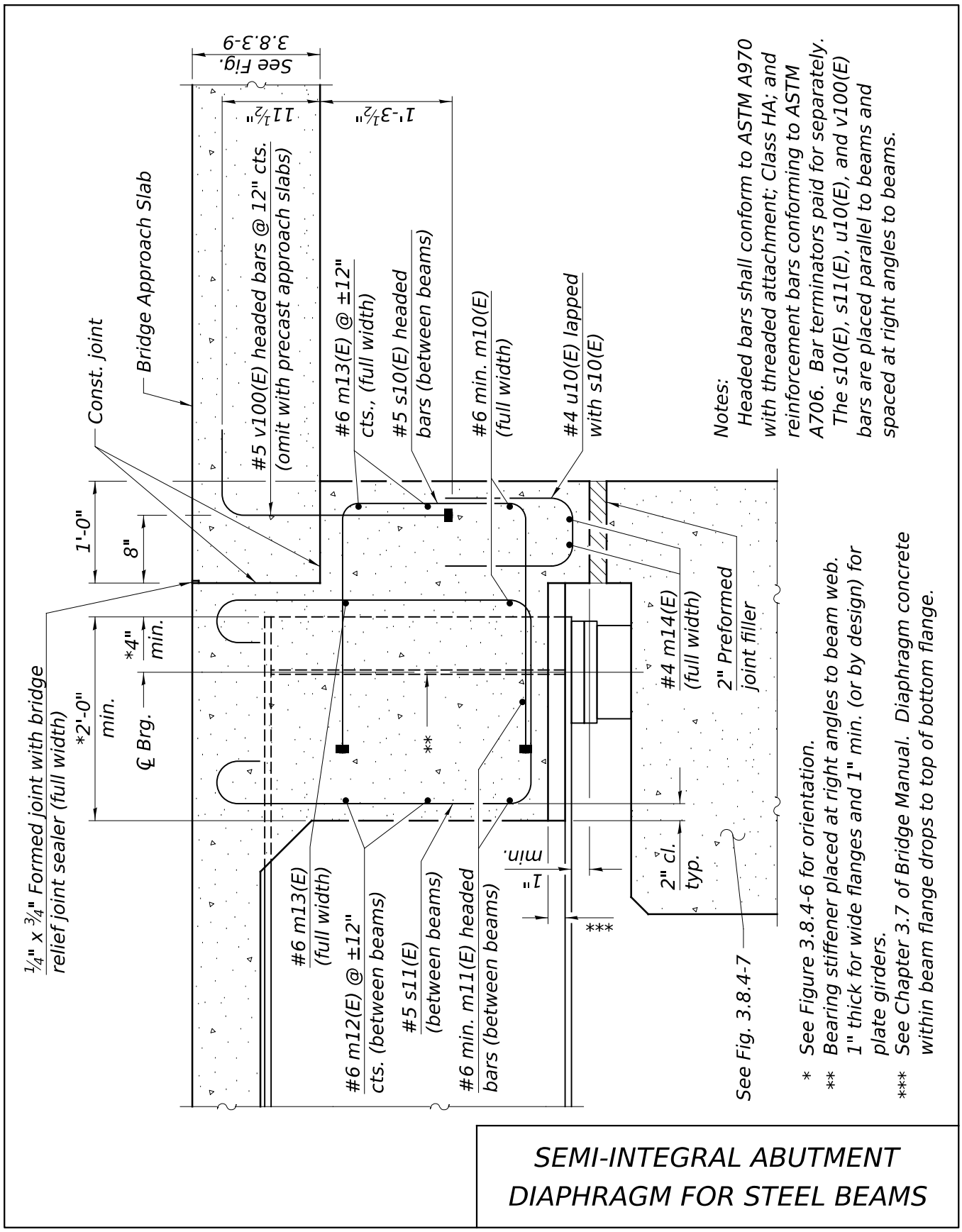


Figure 3.8.4-3

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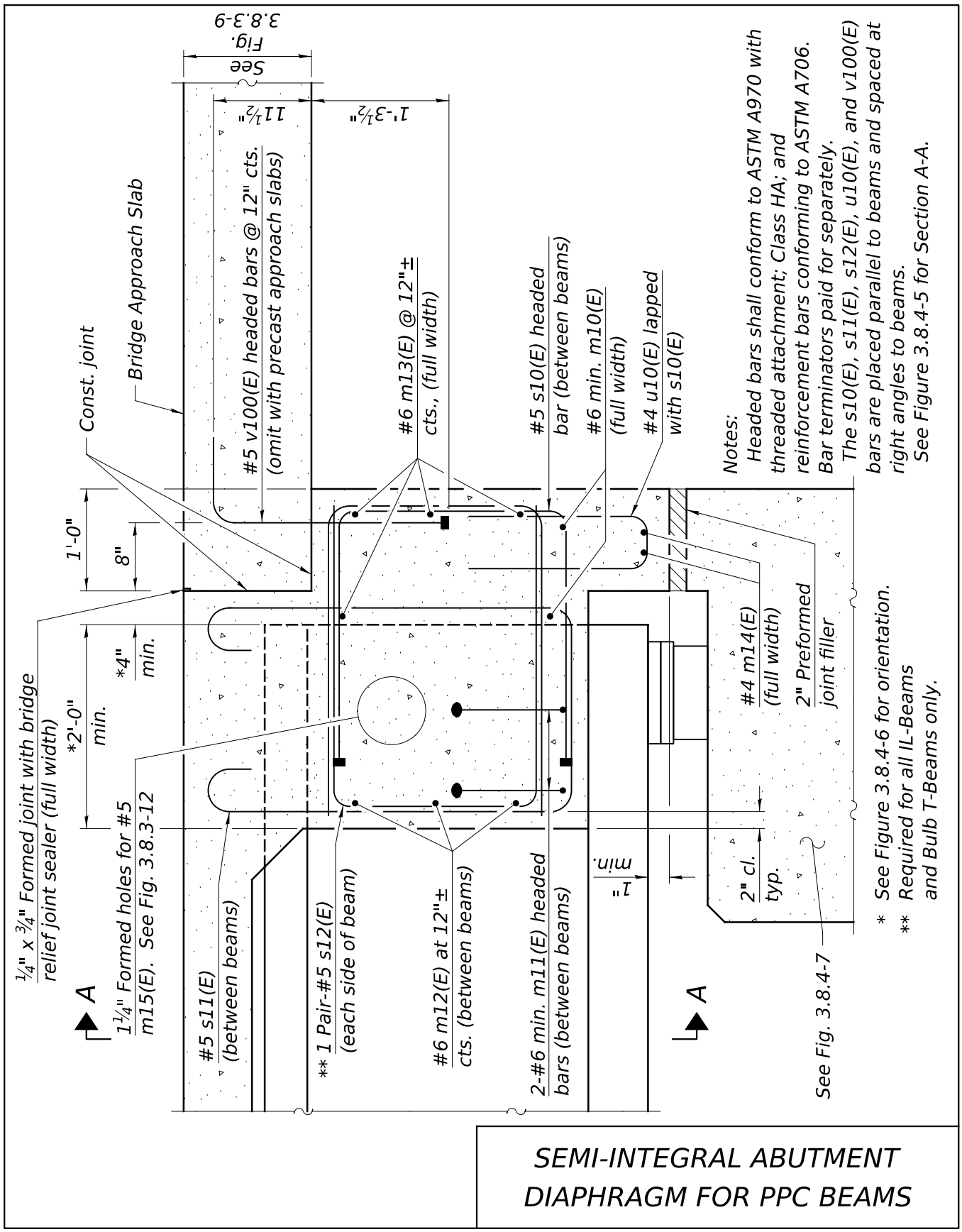


Figure 3.8.4-4

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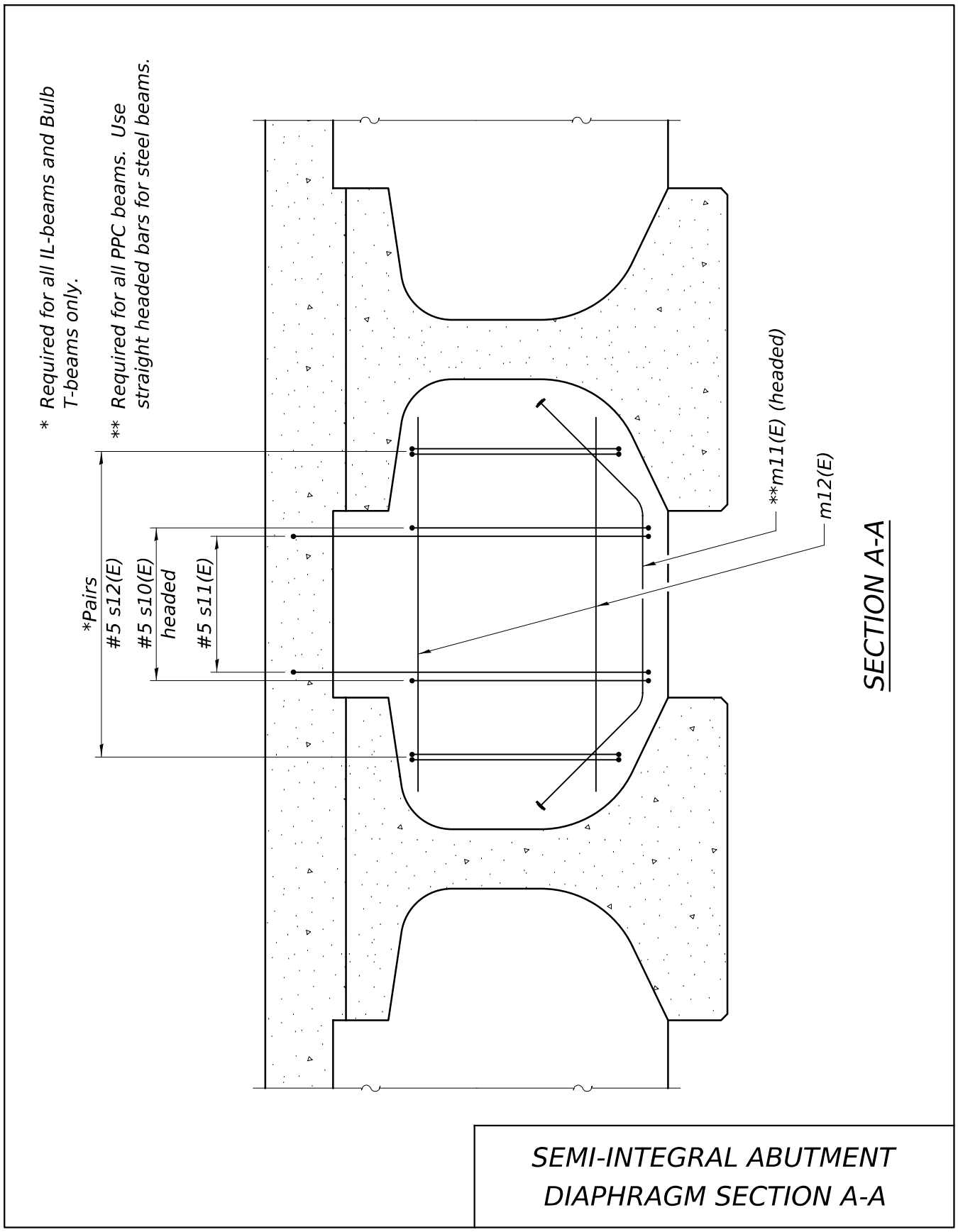


Figure 3.8.4-5

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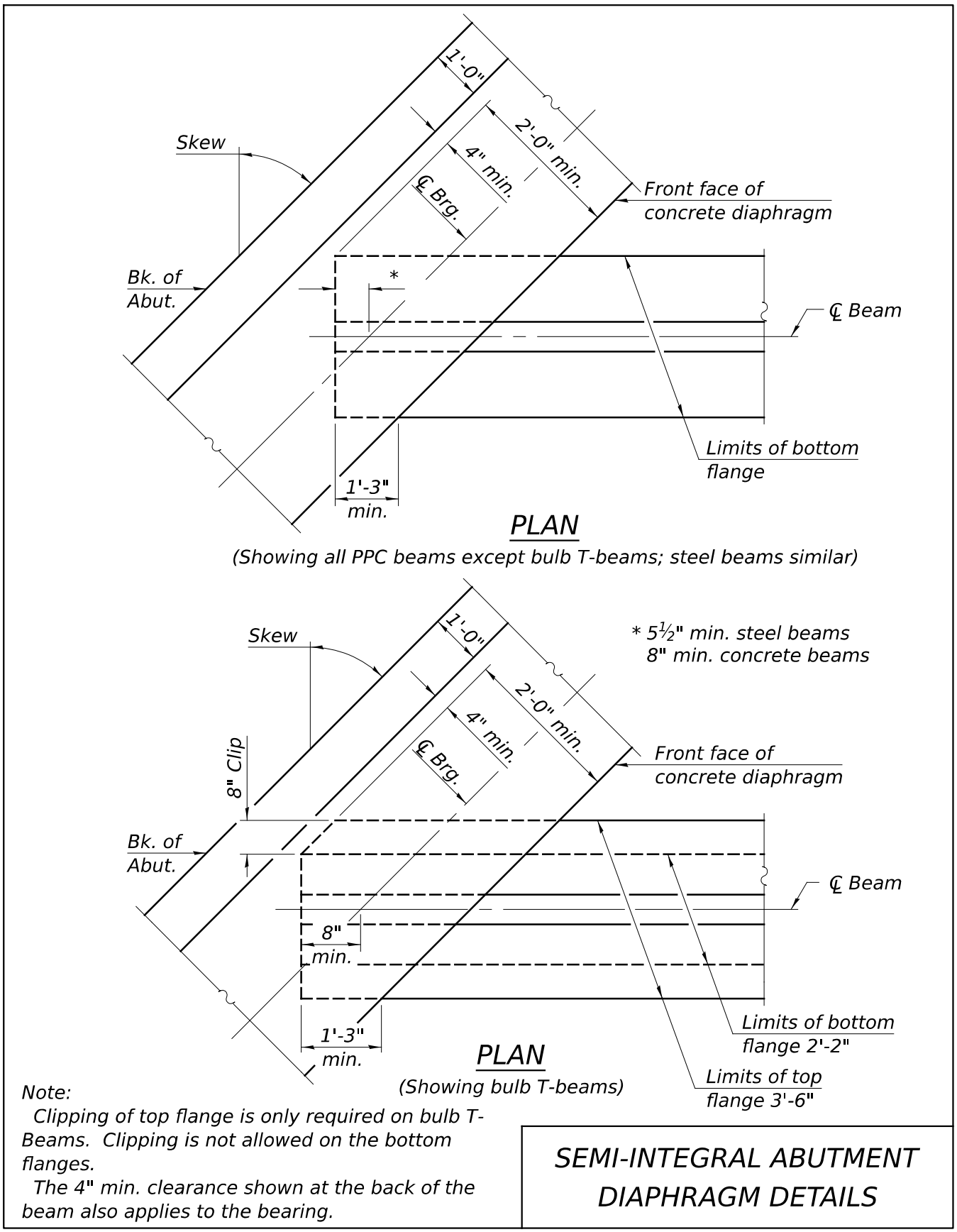


Figure 3.8.4-6

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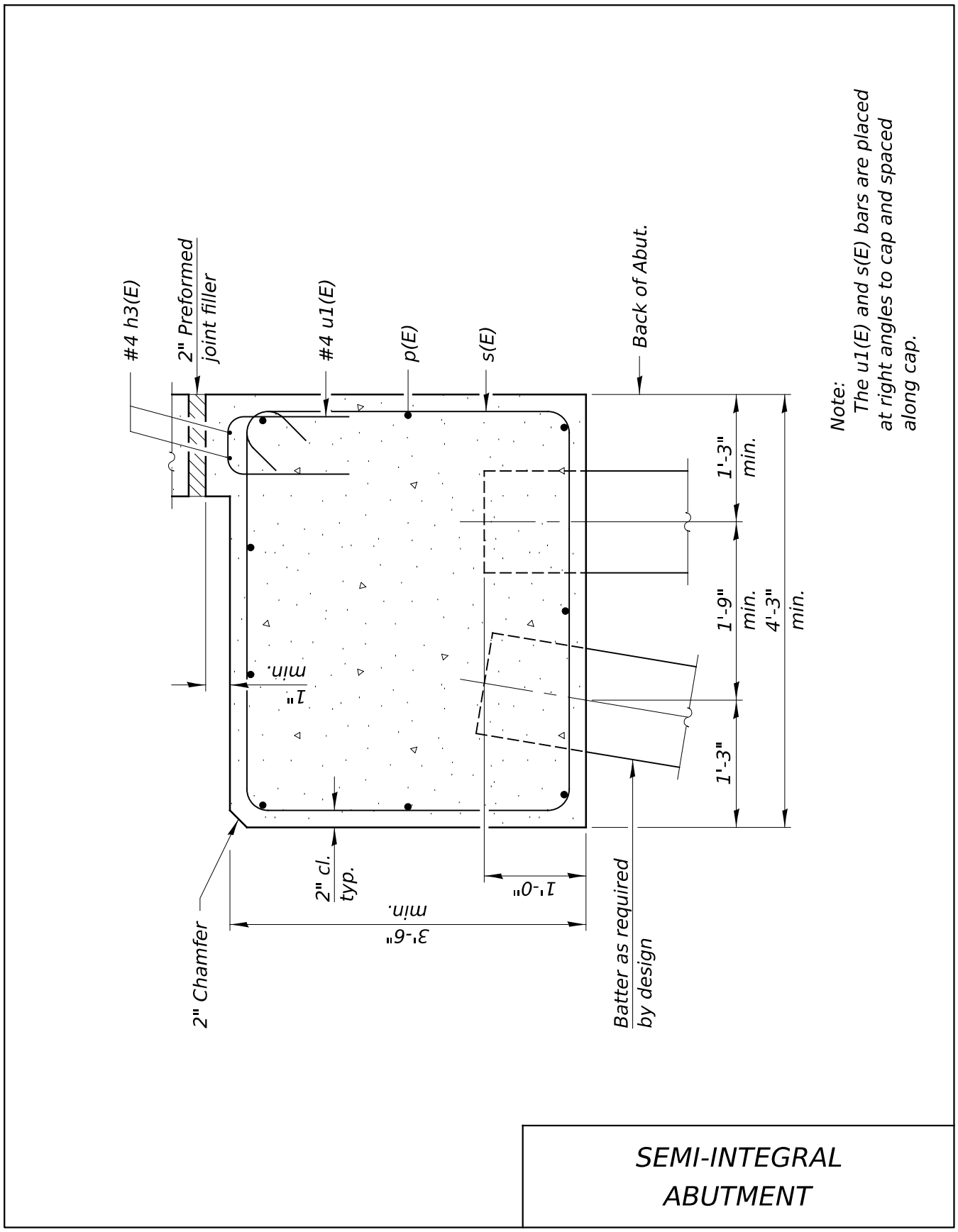


Figure 3.8.4-7

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3.8.4.1 Semi-Integral for Rehabilitation and Reconstruction

When the Scope of Work of an existing structure identifies the abutment expansion joint should be eliminated and the repairs on the bridge deck are so extensive that bridge deck replacement is recommended, the structure shall follow the recommended rehabilitation and reconstruction guidelines as illustrated in interactive 3D examples at the primary IDOT CADD Page as they become available. These interactive 3D examples illustrate the Department's preferred details for proper structure movement which help prevent diaphragm snag, restraint, and punch thru of reconstructed elements.

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3.8.5 Open Abutments: Stub

Drainage details for stub abutments are presented in Figure 3.8.5-1.

The general details and design criteria for “standard” stub abutments supported by piles are shown in Figure 3.8.5-2 through Figure 3.8.5-3. The “standard” option is intended for smaller expansion joints such as strip seals, where the joint anchorage is sufficiently small enough to fit in a standard hatch block.

The general details and design criteria for “corbel” stub abutments supported by piles are shown in Figure 3.8.5-4 through Figure 3.8.5-5. The “corbel” option is intended for larger expansion joints such finger plate or modular joints, where the joint anchorage is too large to fit in a standard hatch block.

See Section 2.3 for dimension A_{bw} and A_{ba} layout equations.

When calculating pile reactions, the live load may be placed on either the approach span or the bridge span to maximize the live load reaction. A live load surcharge equal to two feet of soil shall be applied when no approach slab is provided.

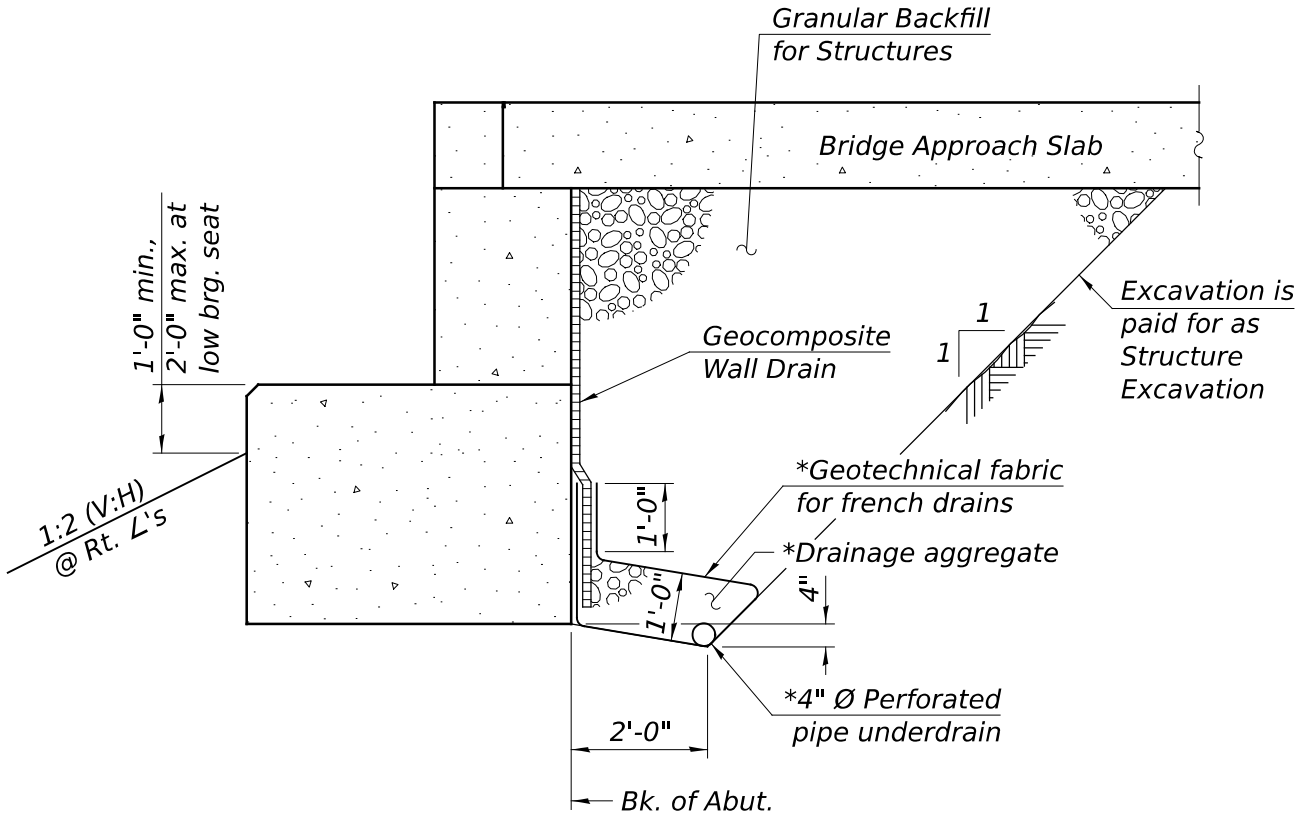
Guidance for wingwall lengths and detailing is shown in Figure 3.8.5-6 through Figure 3.8.5-7 which can accommodate deep beams including 90” precast prestressed concrete IL-beams. For longer wingwalls, guidance is provided for when additional piles are needed at the midpoints of the wingwalls.

Layout of the top of backwall for stub abutments is detailed for the “standard” and “corbel” versions in relation to the bridge approach slab. See Figure 3.8.5-8.

Figure 3.8.5-9 provides expansion bearing locations for PPC beams on stub abutments.

Stub abutments are permitted on MSE walls as discussed in Section 3.8.9.4.

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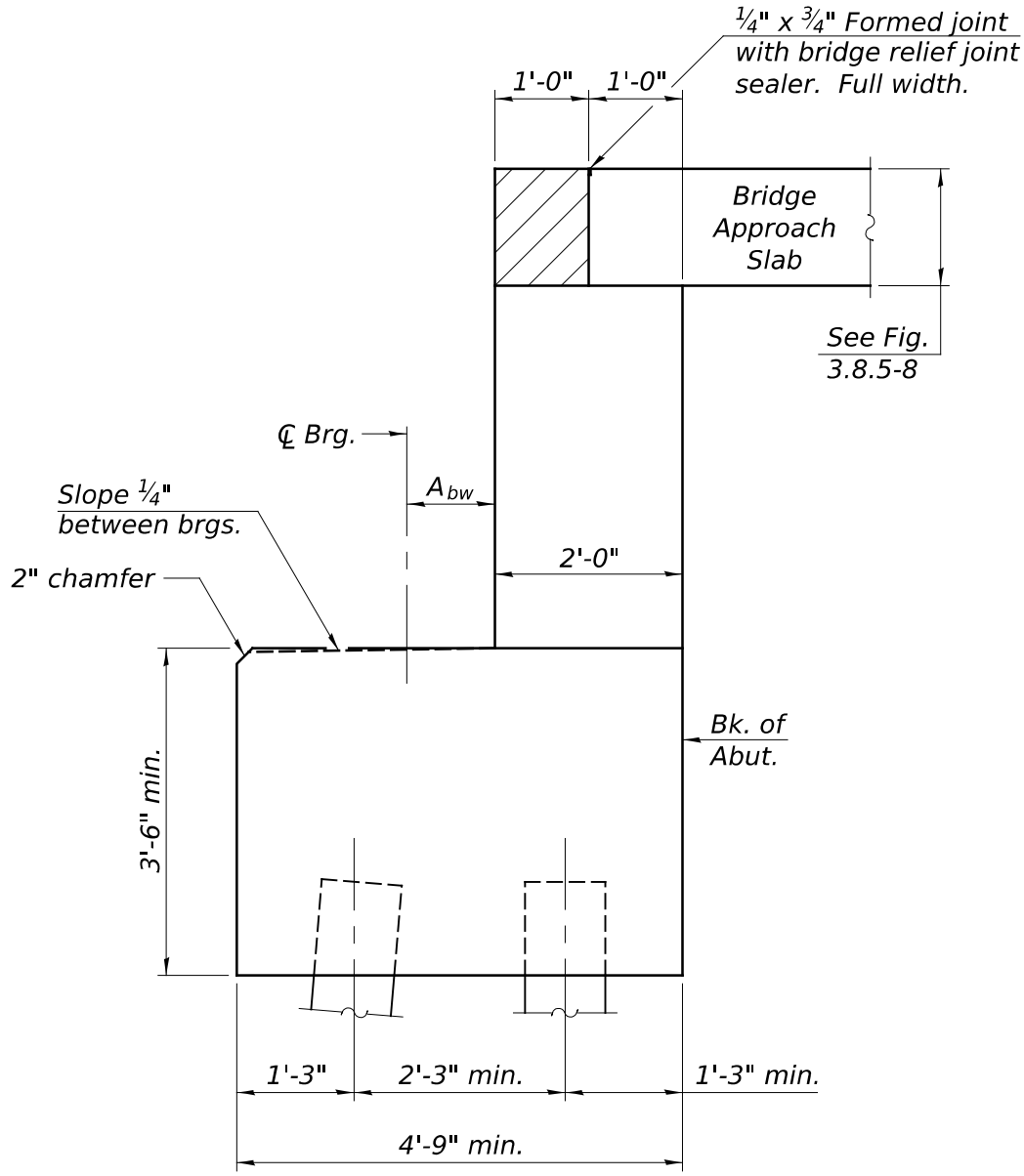
* Included in the cost of Pipe Underdrains for Structures.

All drainage system components shall extend parallel to the abutment back wall until they intersect the wingwalls. The pipe shall extend under the wingwall until intersecting the side slopes. The pipes shall drain into concrete headwalls. (See Article 601.05 of the Standard Specifications and Highway Standard 601101).

STUB ABUTMENT
DRAINAGE DETAILS

Figure 3.8.5-1

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SECTION THRU ABUTMENT

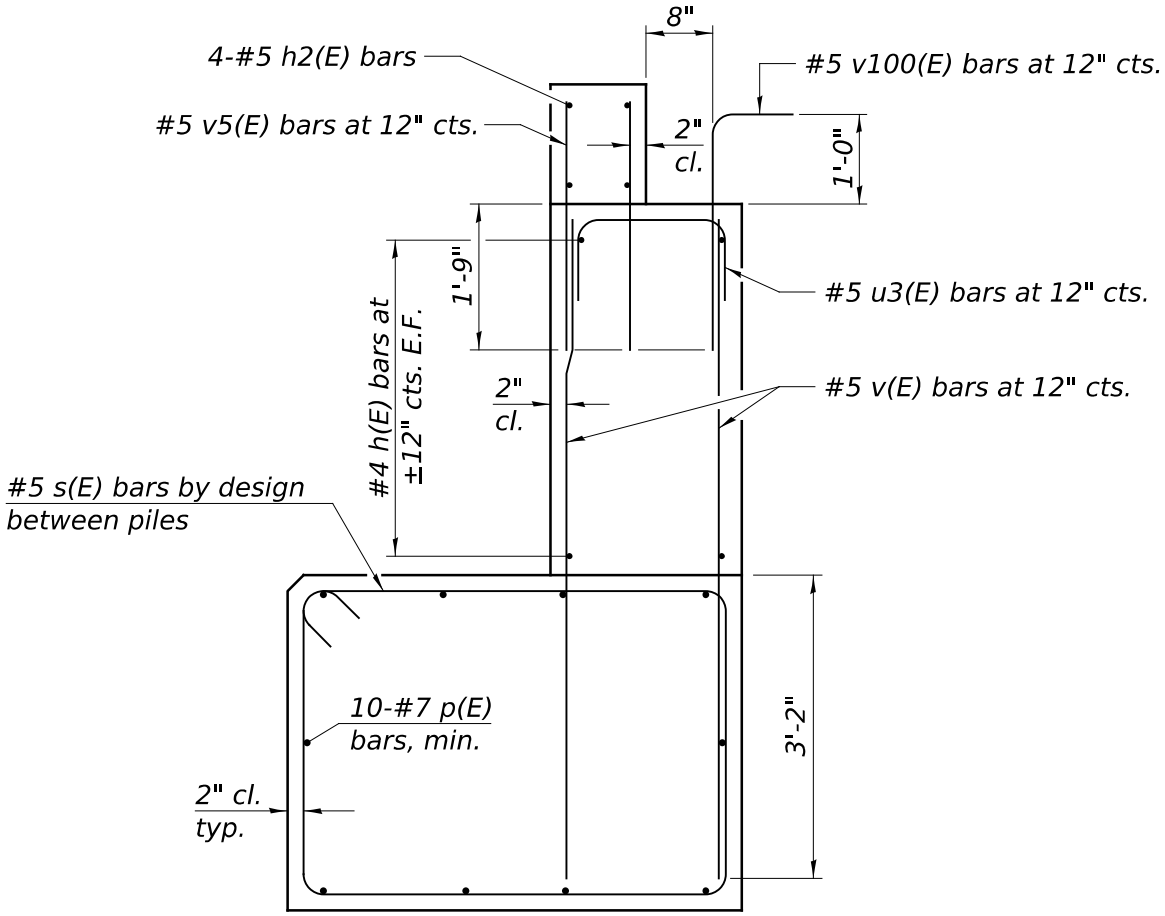
(Strip Seal expansion joints)

Note:
Hatched area to be poured separately after superstructure falsework has been removed and after approach slab side formwork has been removed.
See Fig. 2.3.4.4.1-1 for dimension A_{bw}.

STANDARD STUB ABUTMENT DIMENSIONAL DETAILS

Figure 3.8.5-2

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SECTION THRU ABUTMENT

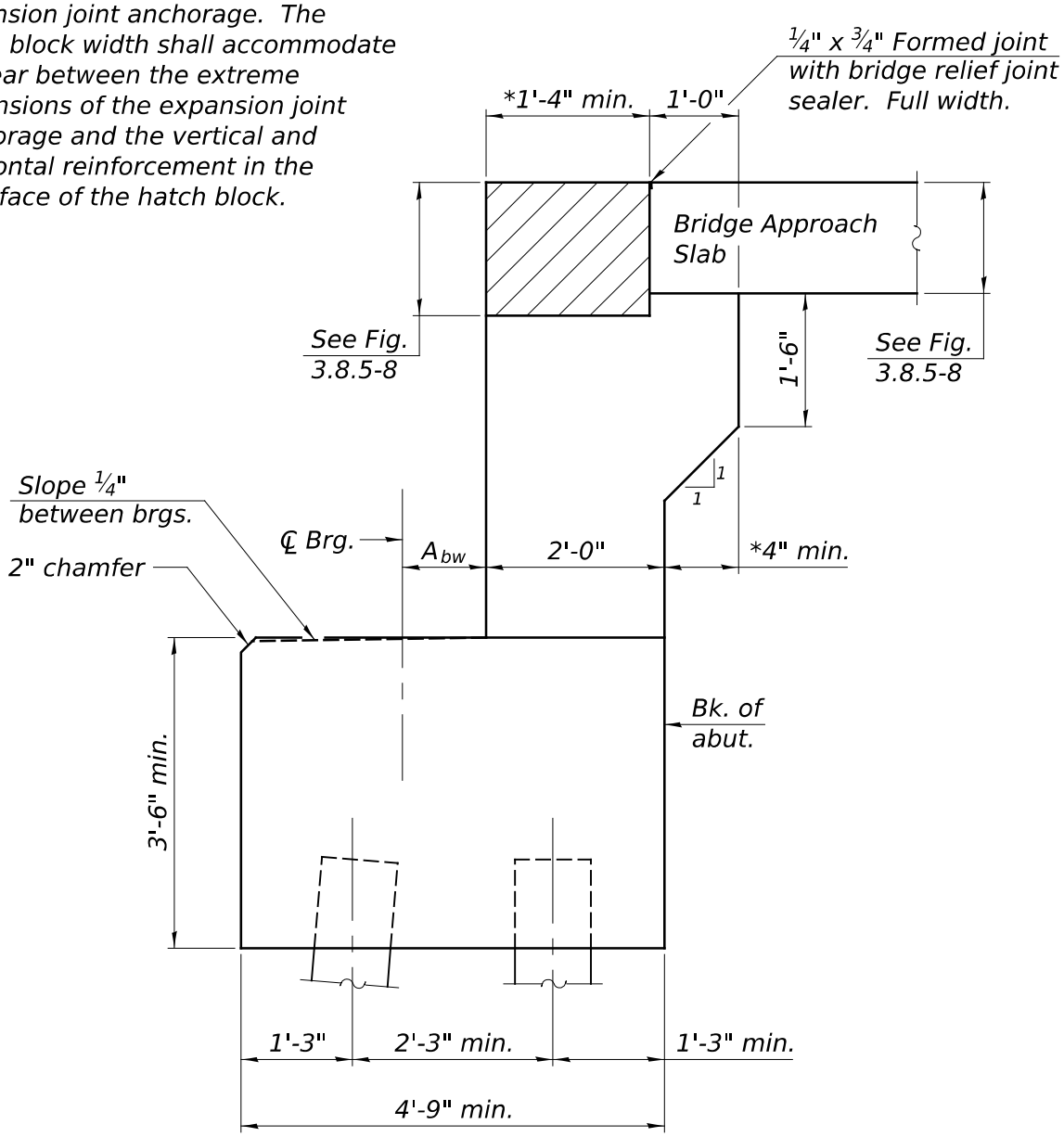
Note:
The u3(E) and v100(E) bars shall be placed parallel to centerline of roadway for skewed bridges.

STANDARD STUB ABUTMENT REINFORCEMENT DETAILS

Figure 3.8.5-3

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* Increase as needed to accommodate expansion joint anchorage. The hatch block width shall accommodate 2" clear between the extreme dimensions of the expansion joint anchorage and the vertical and horizontal reinforcement in the back face of the hatch block.



SECTION THRU ABUTMENT

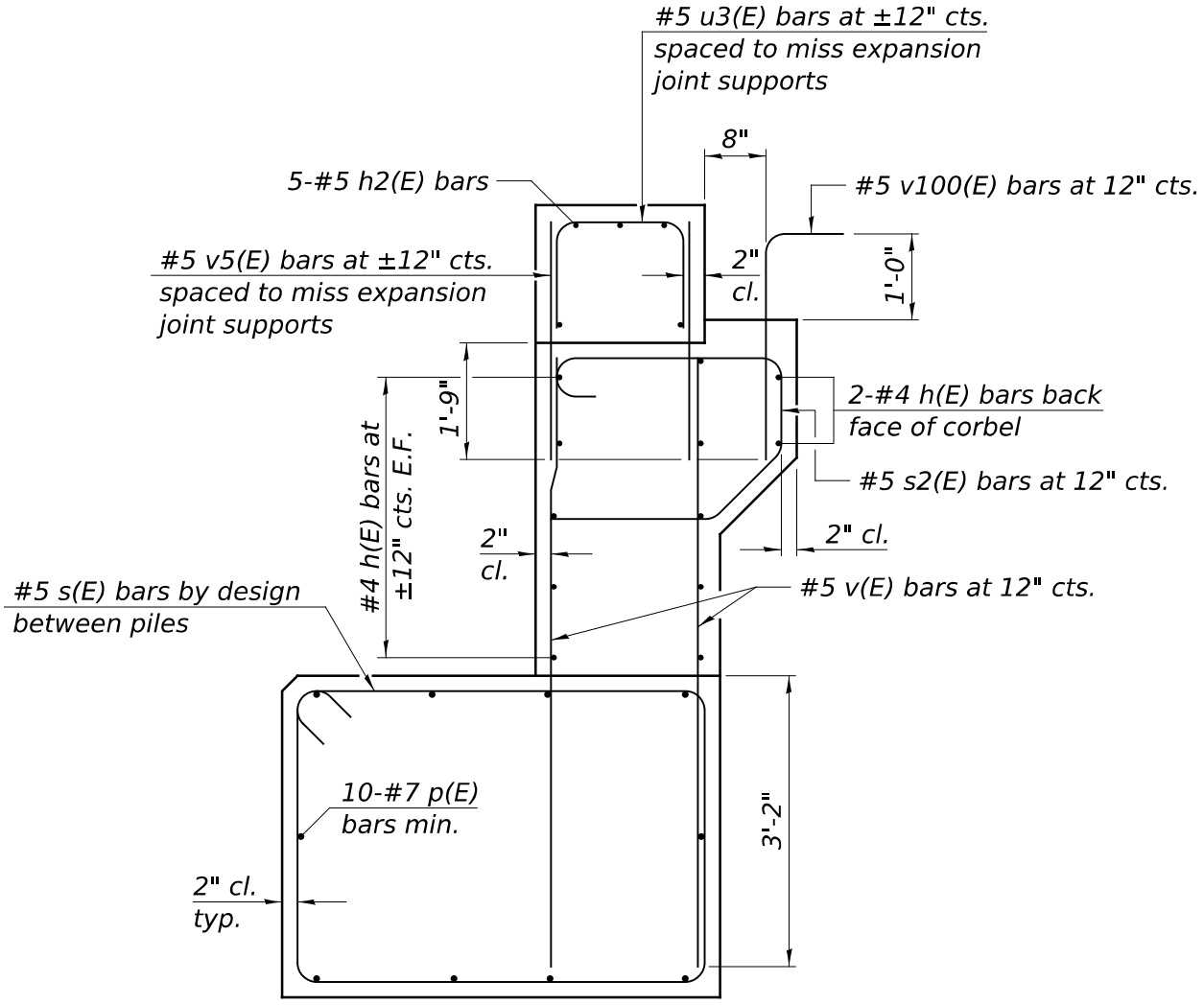
(Modular or finger plate joints, when required anchorage length requires additional hatch block width)

Note:
Hatched area to be poured separately after superstructure falsework has been removed and after approach slab side formwork has been removed. See Fig. 2.3.4.4.1-1 for dimension A_{bw}.

CORBEL STUB ABUTMENT DIMENSIONAL DETAILS

Figure 3.8.5-4

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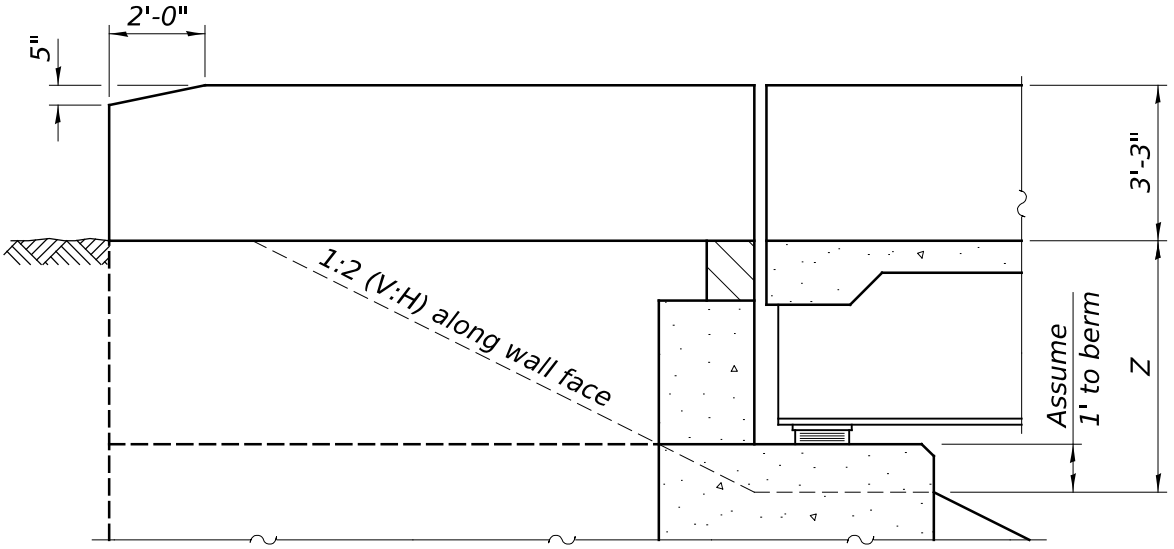
SECTION THRU ABUTMENT

Note:
The s2(E), u3(E) and v100(E) bars shall be placed parallel to centerline of roadway for skewed bridges.

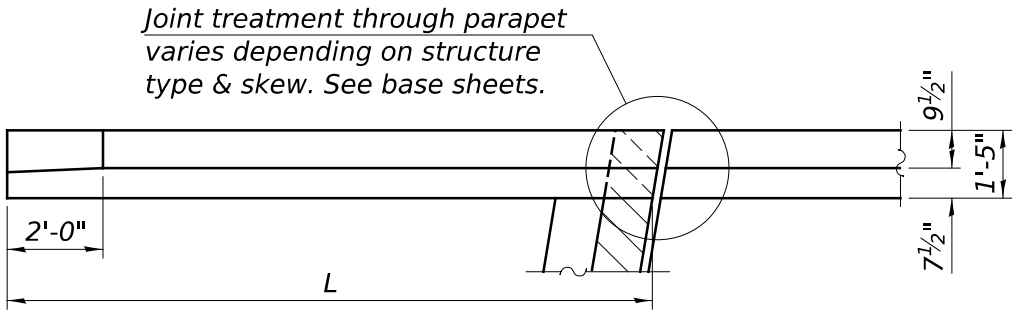
CORBEL STUB ABUTMENT REINFORCEMENT DETAILS

Figure 3.8.5-5

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INSIDE VIEW



PLAN

WINGWALL LENGTH	
BEAM DEPTH	*L
≤ 27"	12'-0"
> 27" to ≤ 36"	13'-6"
> 36" to ≤ 48"	15'-6"
> 48" to ≤ 60"	17'-6"
> 60" to ≤ 72"	19'-6"
> 72" to ≤ 90"	22'-6"

** For normal crown only. Wingwall length shall be checked when structure is superelevated. See Bridge Manual Planning Section, Fig. 2.3.6.3.3-7. Wingwall lengths ≤ 15'-6" require one pile at end of wingwall, and Wingwall lengths > 15'-6" require one pile at end of wingwall, and one pile at mid-length of wingwall.*

$$Z = \frac{\text{beam depth} + 8\text{" deck} + 1\text{" fillet} + 6\text{" bearing} + 12\text{" to berm}}{12\text{"}}$$

$$L = 2Z + 3'$$

WINGWALL LENGTHS
FOR STUB ABUTMENTS

Figure 3.8.5-6

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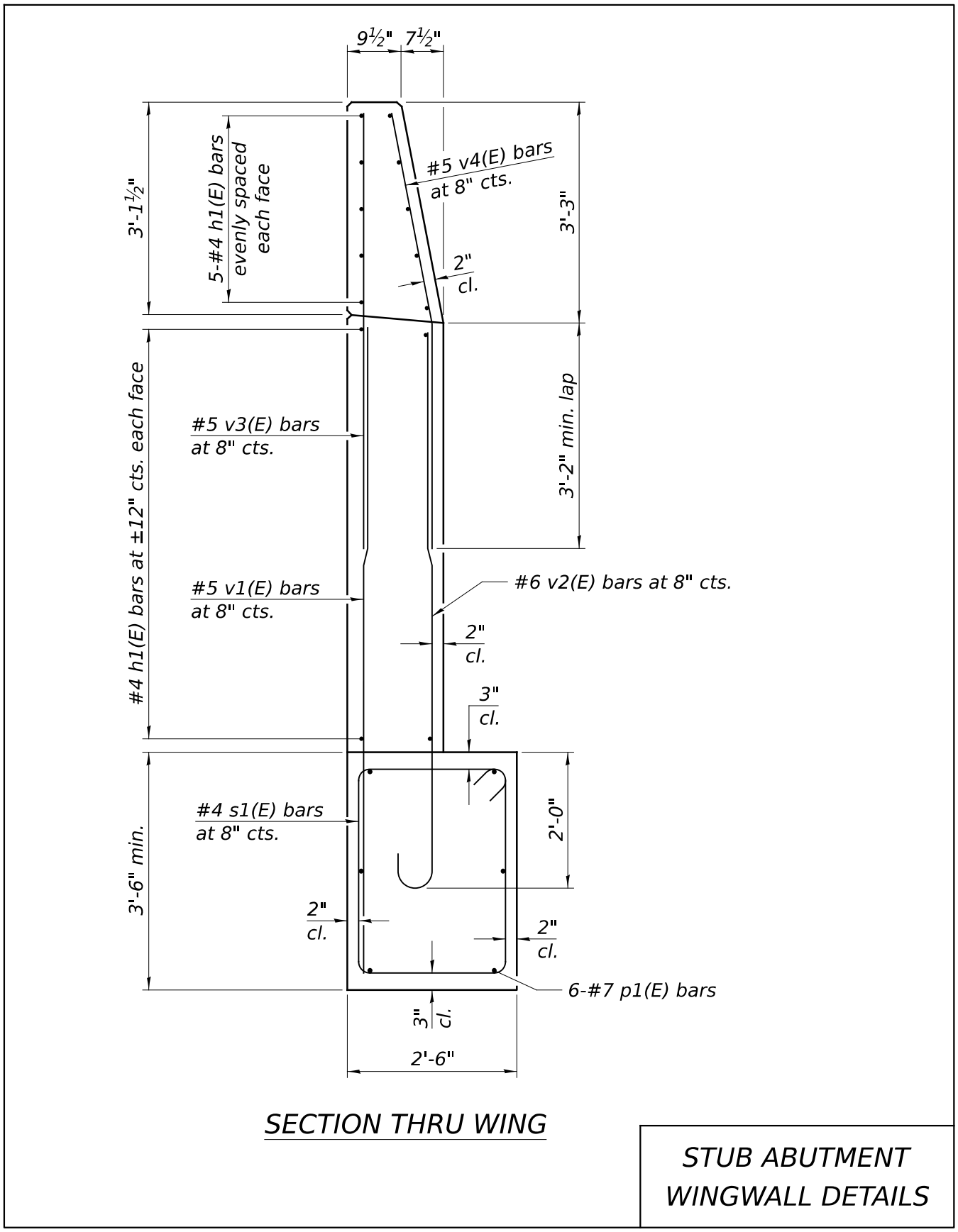
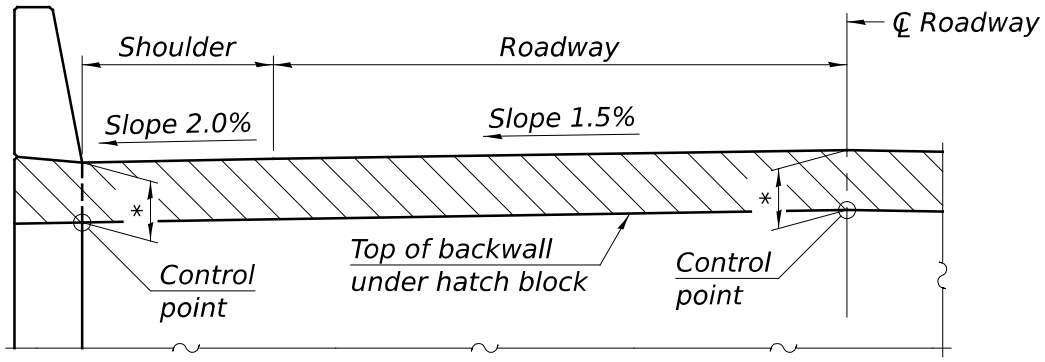
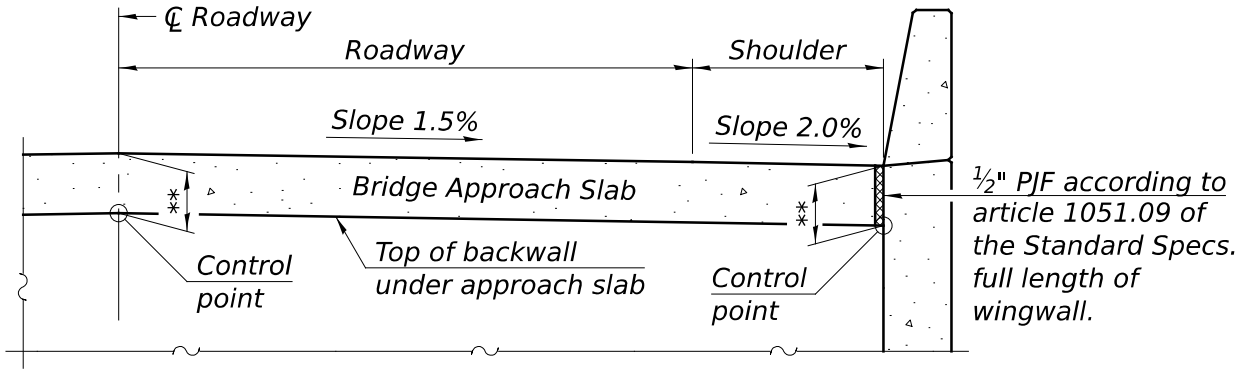


Figure 3.8.5-7

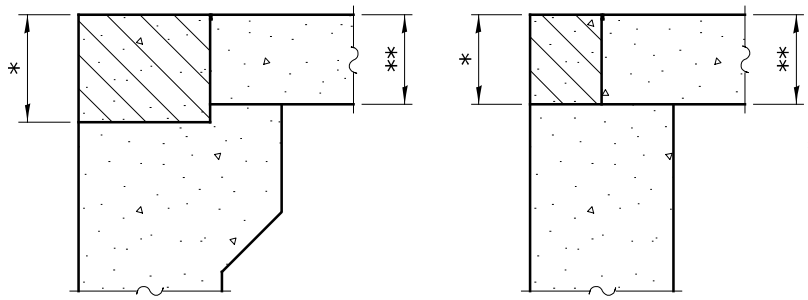
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FRONT FACE OF BACKWALL



BACK FACE OF BACKWALL



SECTION THRU BACKWALL

* 1'-3" for standard version. 1'-6" min. for corbel version to be detailed by designer based on expansion joint size.

** 1'-3" for cast in place bridge approach slab. Precast bridge approach slab not allowed for stub abutments.

Notes:

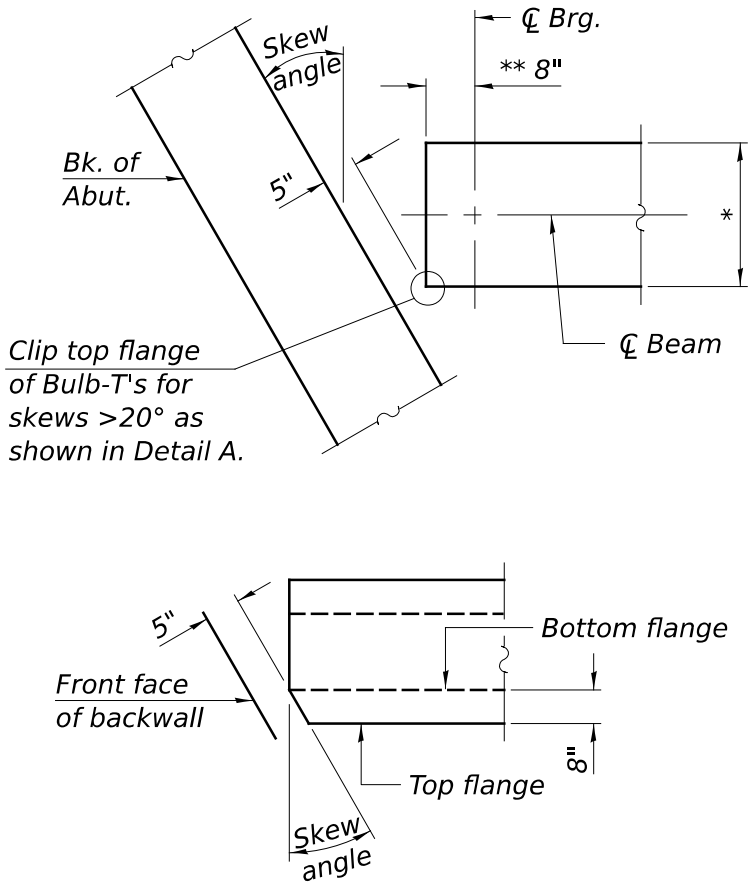
The top of backwall under the hatch block and approach slab shall have a constant slope determined from the control points shown.

The standard version is used for strip seal joints and the corbel version is used for finger plate and modular joints.

TOP OF BACKWALL DETAILS FOR STUB ABUTMENTS

Figure 3.8.5-8

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DETAIL A

* 18" (36" I-Beam)
22" (42",48" & 54" I-Beam)
42" (63" & 72" Bulb-T (top flange))

** The 8" dimension is intended for use with bearings given in Section 3.7. When bearings outside the limits of the Bridge Manual are used, this dimension may need to be increased.

Note:
For seismic design of seat lengths, see Section 3.15.

EXPANSION BEARING LOCATION AT STUB ABUTMENTS FOR PPC BEAMS

Figure 3.8.5-9

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3.8.6 Closed Abutments – General

Joints in closed concrete abutments shall be similar to those illustrated in Figure 3.11.2-1. Reinforcement shall be continuous through construction joints, but no reinforcement shall pass through expansion joints. Only construction joints are permitted in the footings.

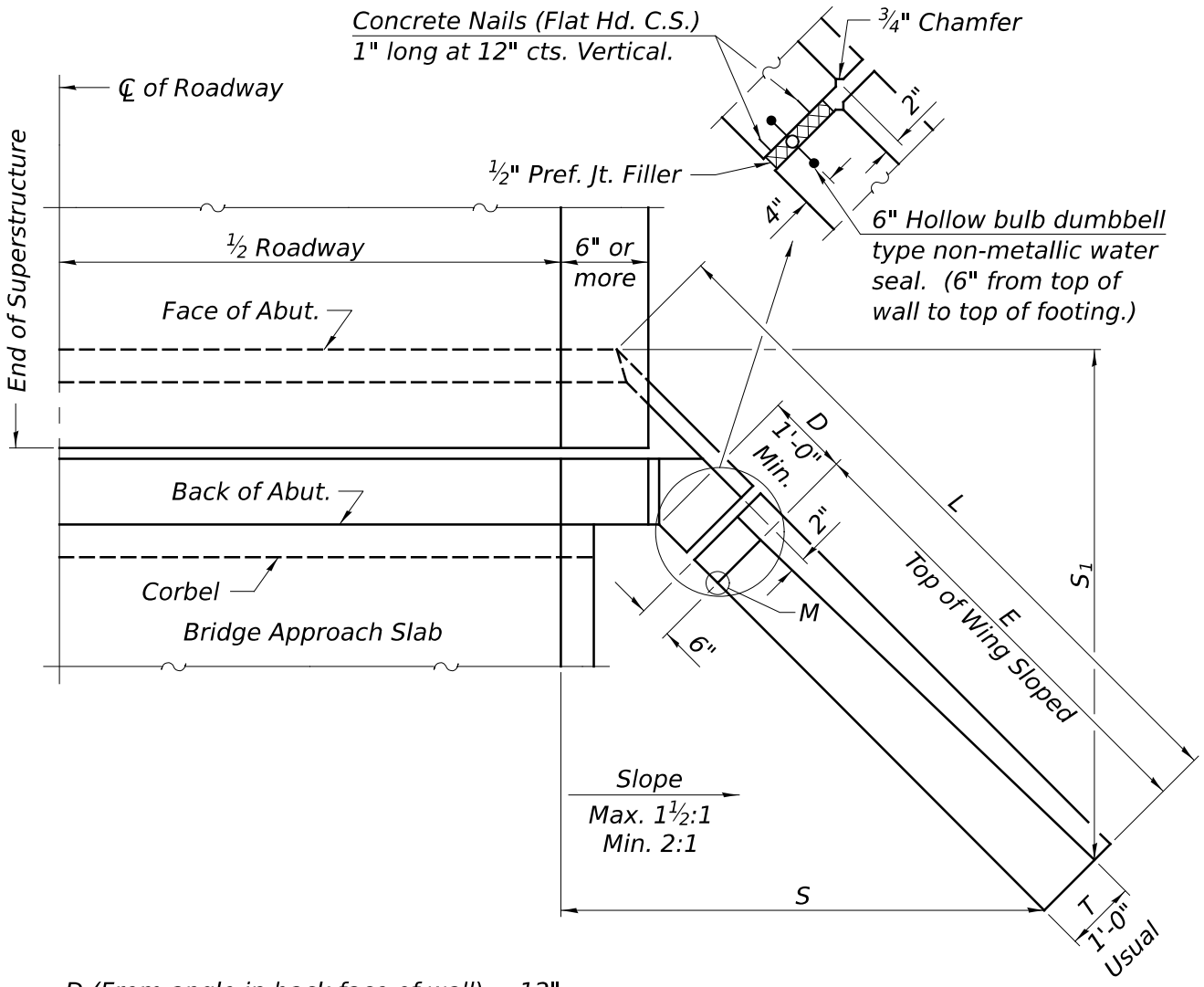
Wingwalls of a closed abutment shall not be poured integrally with the abutment wall, but shall be separated from the abutment wall with a ½ in. joint filled with preformed joint filler. The front face of the wingwall shall be set back 2 in. from the face of the abutment wall at the top. See Figure 3.8.6-1 for wingwall dimensions and heights. The footing under the wingwall shall be poured continuously with and have the same thickness as the abutment footing. The wingwall footing thickness is usually constant. However, for long wings, the footing width may be reduced, reflecting the reduced overturning moment.

The longitudinal reinforcement (w bars) in a pile supported footing shall be #5 bars at 12 in. cts. minimum placed between piles. The transverse reinforcement (t bars) shall be designed. See Section 3.10.1 for pile spacing requirements of pile supported footings. For the design of a pile supported footing in flexure and in shear, the pile load shall be distributed over a width equal to $0.8X + 3.75$ ft. but not wider than the longitudinal pile spacing, where X is the distance from the edge of the vertical wall to the center line of the pile under consideration.

Wingwalls shall be designed according to the criteria given in the Culvert Manual (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design) with batter on the front face up to a maximum of ½ in. per ft. of height. If a greater wall thickness is required, place the additional batter on the back face. The batter shall be constant for the full length of the wing. The minimum thickness of any wingwall with a closed abutment shall be 12 in.

Weep holes are to be provided at 8 ft. centers in all closed abutment walls and wingwalls unless the sidewalks or roadways near the face would be affected by drainage or ice. Figure 3.8.6-2 illustrates a typical section through a closed abutment with a weep hole. If weep holes are not provided, an alternate system of pipe underdrains should be employed. Figure 3.8.6-3 includes a pipe underdrain detail.

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D (From angle in back face of wall) = 12".

Wing Length - L:

(1) If roadway slopes are 1½ : 1, figure L to nearest 6" to allow 1½ : 1 slope along S + T + S_1 from actual shoulder elevation to streambed elevation. If roadway slopes are 2 : 1 or flatter, design wings for 2 : 1 slope along S + T + S_1 instead of 1½ : 1. Otherwise as above.

Wing Heights & Drop:

(1) Top of wing to be at top of curb or sidewalk along distance D to M.

(2) Figure drop to nearest 3" so that inside face at end is same distance above earth slope as M is exposed above ground.

DIMENSIONS
CLOSED ABUTMENTS

Figure 3.8.6-1

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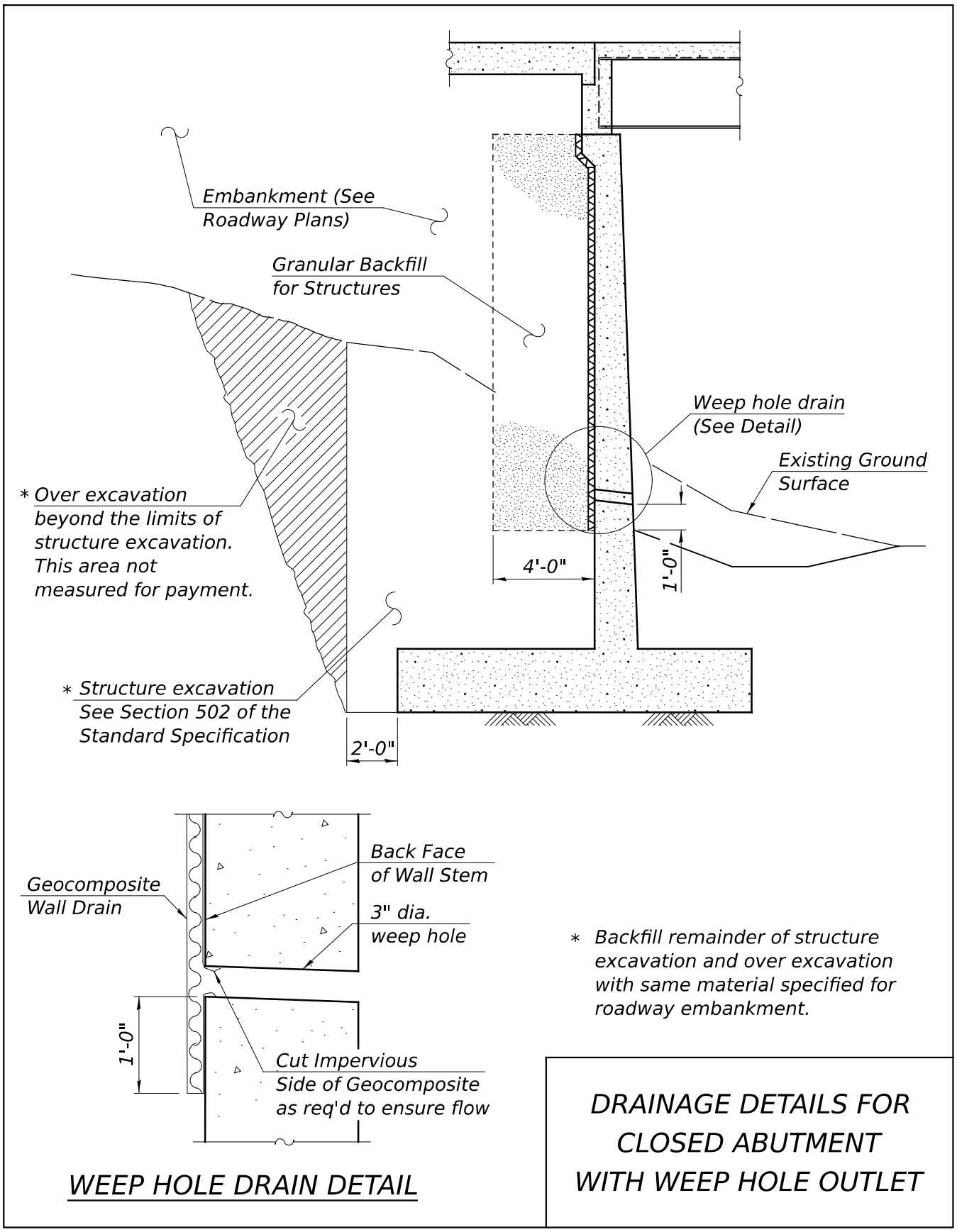


Figure 3.8.6-2

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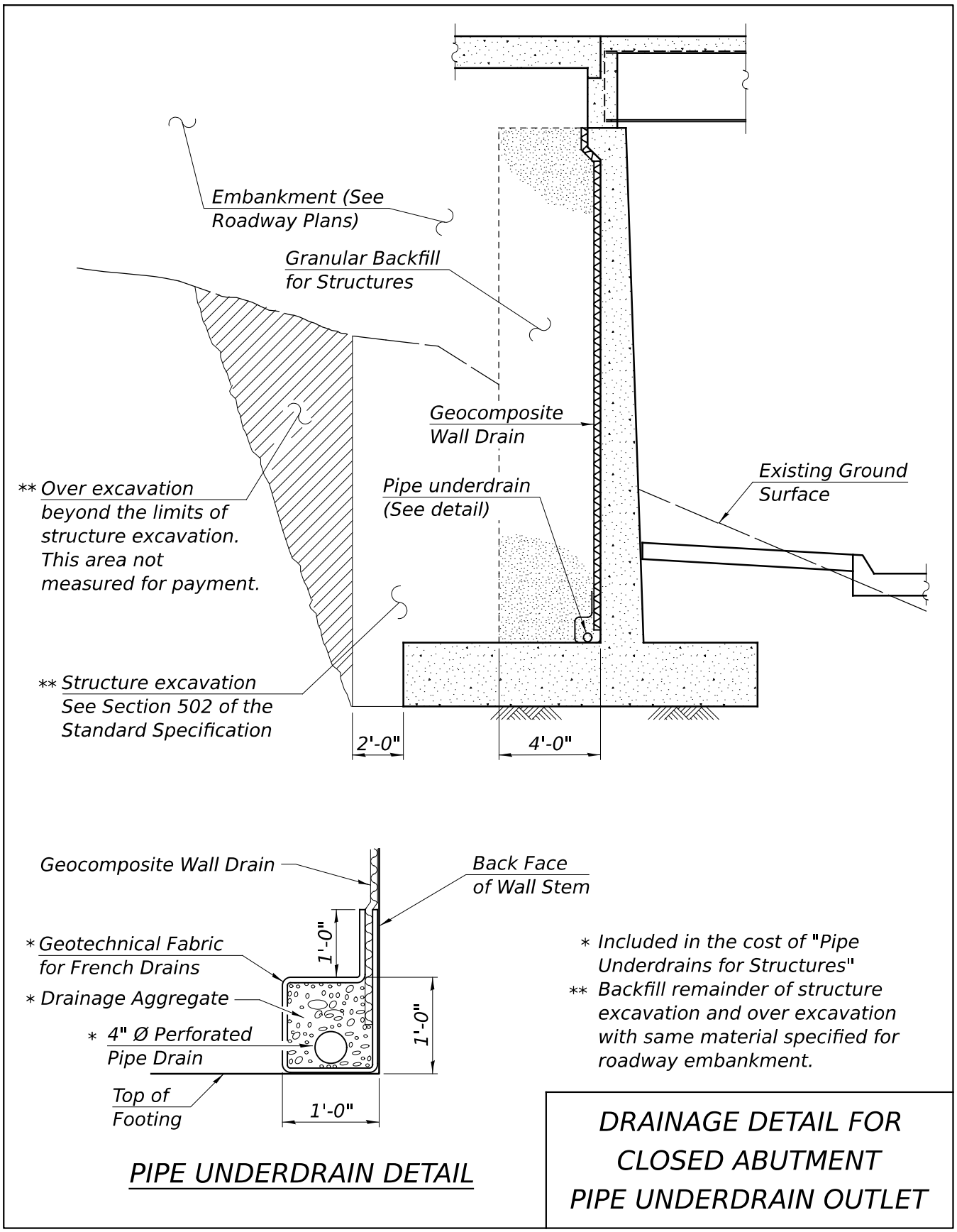


Figure 3.8.6-3

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3.8.7 Closed Abutments – Restrained Top and Bottom

Simple spans supported on closed abutments may be fixed at both supports when the back-to-back of abutment dimension measured along the centerline of roadway does not exceed 45 ft. When both supports are fixed, the abutments shall be designed as restrained top and bottom. The design earth pressure should be calculated assuming at-rest conditions. If the approach roadway is a non-rigid type, a live load surcharge of 2 ft. of soil shall be added to the earth pressure.

Figure 3.8.7-1 presents general design details for closed abutments restrained top and bottom.

Figures 3.8.7-2, 3.8.7-3 and 3.8.7-4 detail closed abutments with fixed supports for slab bridges, R.C. girder bridges and steel stringer bridges, respectively. These details are generally associated with closed abutments restrained top and bottom.

The longitudinal reinforcement (w bars) in a pile supported footing shall be #5 bars at 12 in. cts. minimum placed between piles. The transverse reinforcement (t bars) shall be designed. See Section 3.10.1 for pile spacing requirements of pile supported footings. For the design of a pile supported footing in flexure and in shear, see the requirements of Article 5.12.8 of the AASHTO LRFD Bridge Design Specifications.

Drainage details similar to those in Figures 3.8.6-2 and 3.8.6-3 shall apply.

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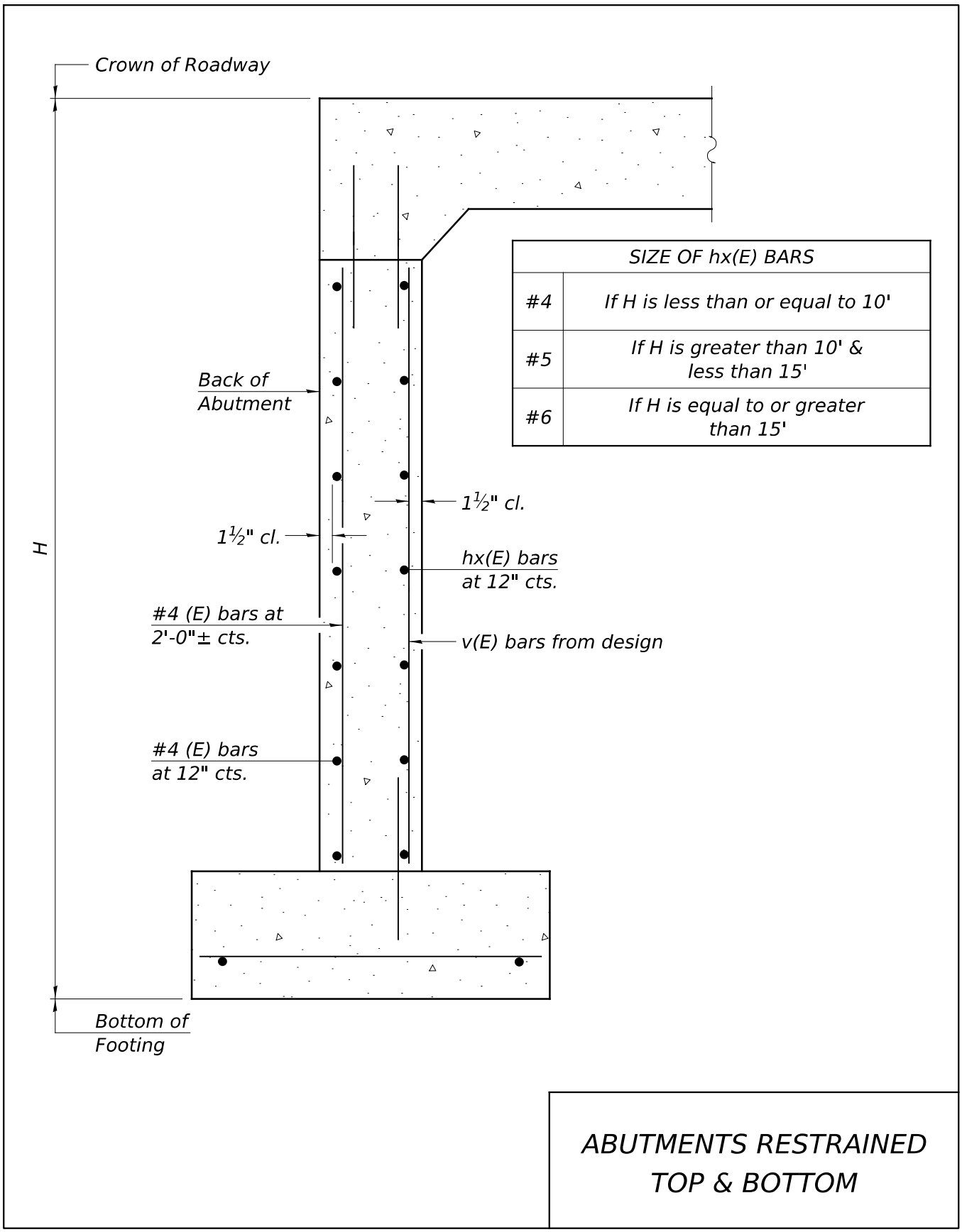
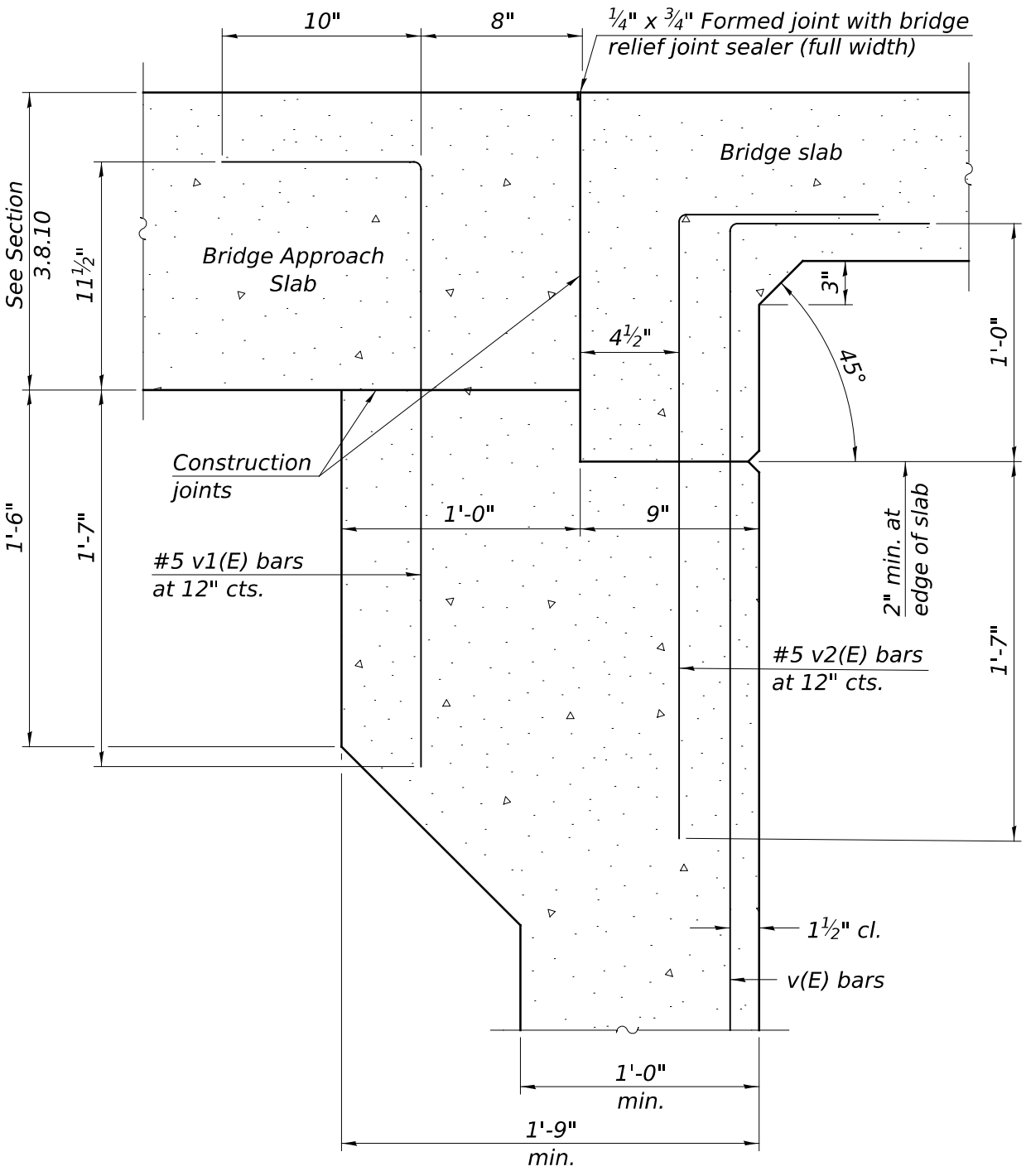


Figure 3.8.7-1

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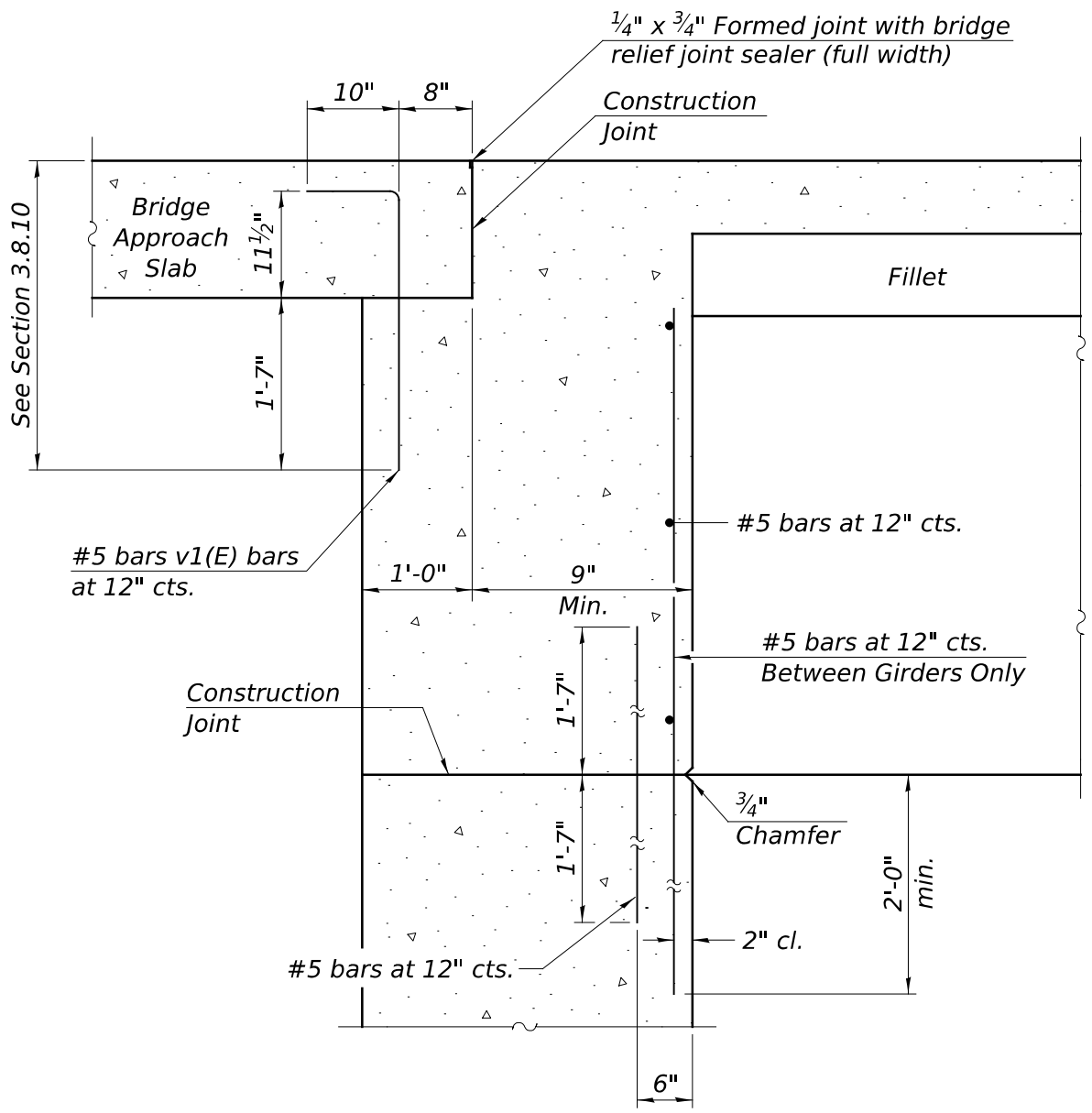


* *When total differential crown elevation is 8" to 1'-0", revise dimensions accordingly to maintain required clearances. See Section 3.8.10.*

CLOSED ABUTMENTS SLAB BRIDGES

Figure 3.8.7-2

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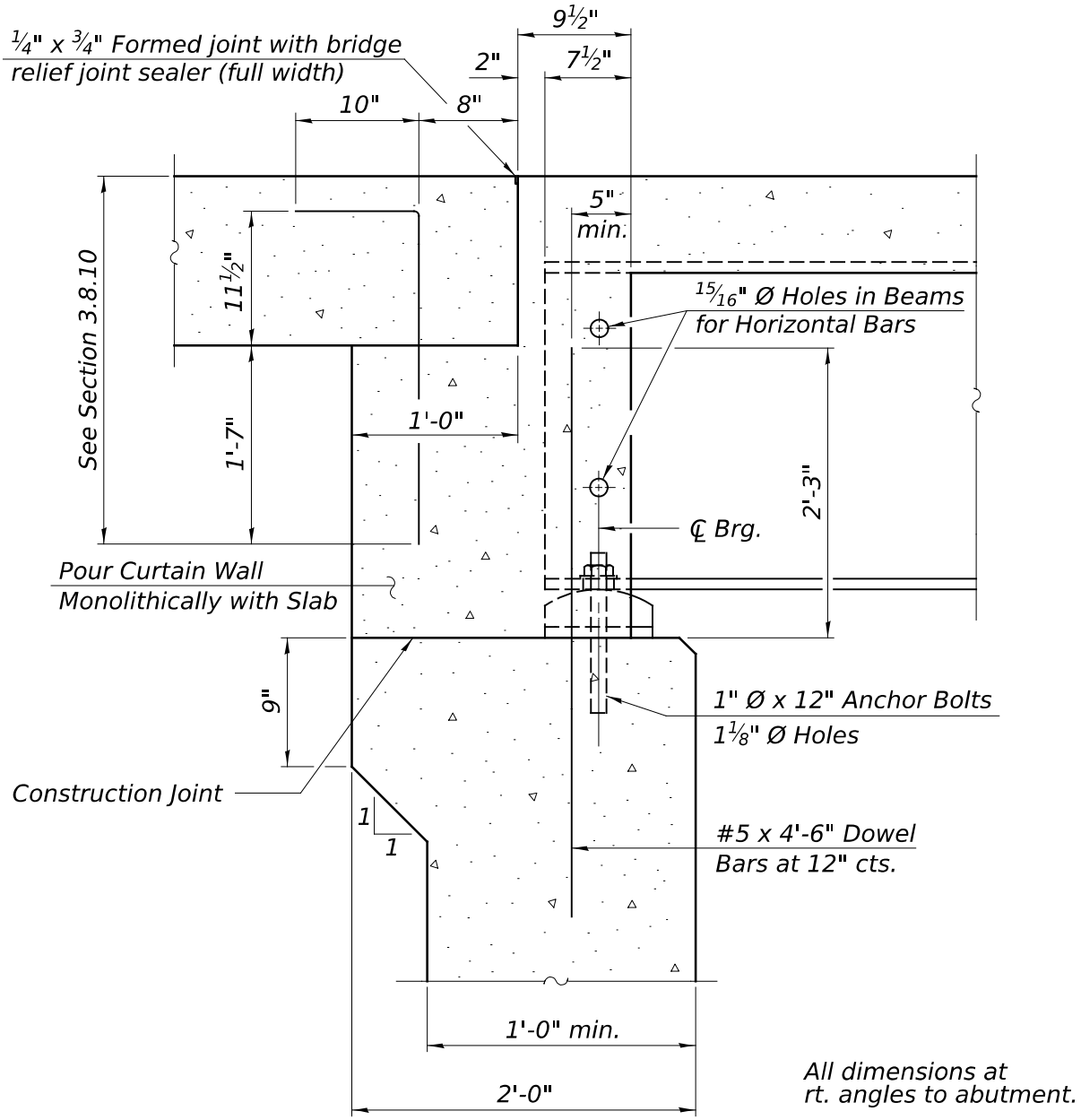
FOR SINGLE SPAN UNDER 45 FT.

Note:
Design breast wall supported at top & bottom.

CLOSED ABUTMENTS
R.C. GIRDER BRIDGES

Figure 3.8.7-3

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FOR SINGLE SPAN UNDER 45 FT.

CLOSED ABUTMENTS
STEEL STRINGER BRIDGES

Figure 3.8.7-4

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3.8.8 Closed Abutments – Cantilever Type

When the back-to-back dimension between closed abutments exceeds 45 ft., the supports for a simple span shall be fixed at one end and expansion at the other end. For this situation, both closed abutments shall be designed as free cantilevers. The wall stem should be designed to minimize wall deflections and maintain the required joint opening. This may entail use of at-rest earth pressures instead of active earth pressures. If the approach roadway is a non-rigid type, there shall be added to the earth pressure a live load surcharge of 2 ft. of soil.

The vertical reinforcement in the back face shall be designed neglecting vertical loads. Vertical reinforcement in the front face shall be #4 bars at $\pm$ 4 ft. – 0 in. cts. Horizontal temperature reinforcement in the front face shall be based on the maximum wall thickness. For walls up to 15 in. thick, use #4 bars at 12 in. cts., and for walls thicker than 15 in., use #5 bars at 12 in. cts. Horizontal reinforcement in the back face shall be #4 bars at $\pm$ 3 ft. – 0 in. centers. Batter requirements shall be the same as that specified for wingwalls.

The longitudinal reinforcement (w bars) in a pile supported footing shall be #5 bars at 12 in. cts. minimum placed between piles. The transverse reinforcement (t bars) shall be designed. See Section 3.10.1 for pile spacing requirements of pile supported footings. For the design of a pile supported footing in flexure and in shear, the pile load shall be distributed over a width equal to $0.8X + 3.75$ ft. but not wider than the longitudinal pile spacing, where X is the distance from the edge of the vertical wall to the center line of the pile under consideration.

Drainage details similar to those in Figures 3.8.6-2 and 3.8.6-3 shall apply.

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3.8.9 Mechanically Stabilized Earth (MSE) Abutments

When abutments are placed within an MSE wall system, they are usually supported by piles, but spread footings and drilled shafts can also be employed. See Figure 3.8.9-1. A suggested sequence of construction should be provided on the plans to address the various steps, such as when piles should be driven relative to wall construction. When utilizing MSE walls with or adjacent to bridge abutments, the design considerations should be made as follows:

General

1. All future MSE wall projects will require the new Bureau of Design & Environment Special Provision (BDE SP). The Special Provision shall consist of the following changes:
    a. Clarification of construction tolerance by revising the Standard Specification Article 522.09(b)(5)(a)(2) that states “The maximum allowable offset in any panel joint shall be ¾ in.” to “The maximum allowable offset between adjacent panels shall be ¾ in.”
    b. Insertion of new panel width tolerances in specifications as 522.09(b)(5)(a)(5) that states, “The thickness of each panel joint shall be within ¼ in. (6 mm) of that shown on the plans.”

Design

2. See Section 3.11.1 for required soil reinforcement design loads.
3. See Section 3.11.1 for loading requirements where traffic barriers and anchorage slabs are present at the top of the wall.
4. See Section 3.11.1 for layout and design loads where flexible post and beam barriers are present.

Geometry and Orientation

5. Vertical clearance between the top of the abutment and top of coping/ CIP slope shall be a minimum of 3’-0” for girder bridges, and 5’-0” for non-girder, slab, deck beams, and box girder bridges.
6. A defined clear distance between front face of abutment and front face of wall (a.k.a. “walkway”) shall be a minimum of 4’-0”.

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Drainage and Foundations

7. MSE panels have bearing pads between panels, but the pads are not full width. Suppliers have a geofabric behind the joints and the pore pressure is relieved through these joints. Therefore, the designer need not specify separate pay items for drainage on these walls.
8. For walls where reinforcements will be splayed (e.g., steel strip reinforcements), require a minimum offset from the back face of the wall panels and the front of deep foundation elements as the greater of 4 ft or 1 times the diameter of the deep foundation. Thus, for example, if a drilled shaft of 5 ft diameter is used then the minimum clearance is 5 ft. These criteria provide the necessary clear space to achieve proper compaction of the soil in this area and adequate distance for splaying of reinforcements within the acceptable limits.

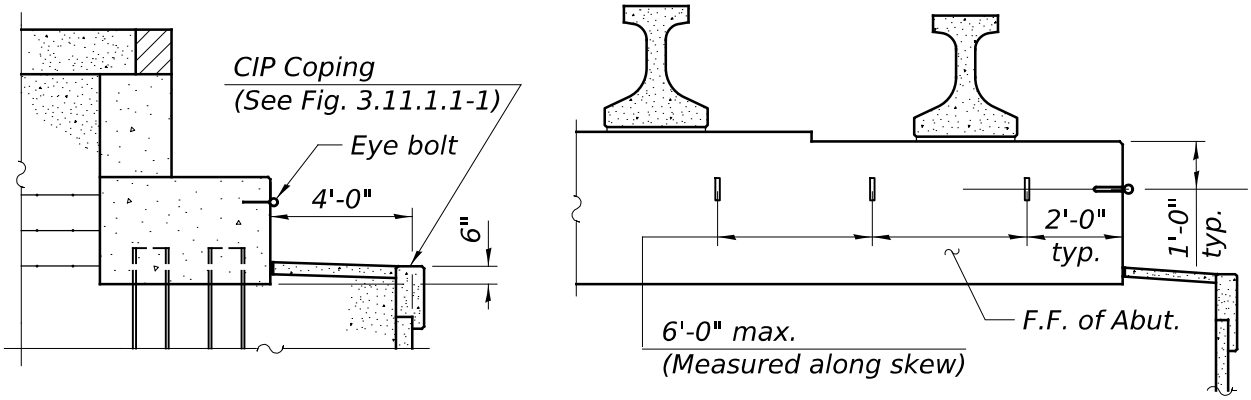
Detailing (General)

9. Safety for inspectors shall be provided via eye bolts embedded in the abutment. See Figure 3.8.9-1 for details.
10. Bond breaker is required between MSE panels and the abutment. See Figure 3.8.9.5-1 and Figure 3.8.9.5-4. See Section 3.11.1 for other locations requiring bond breaker detailing.

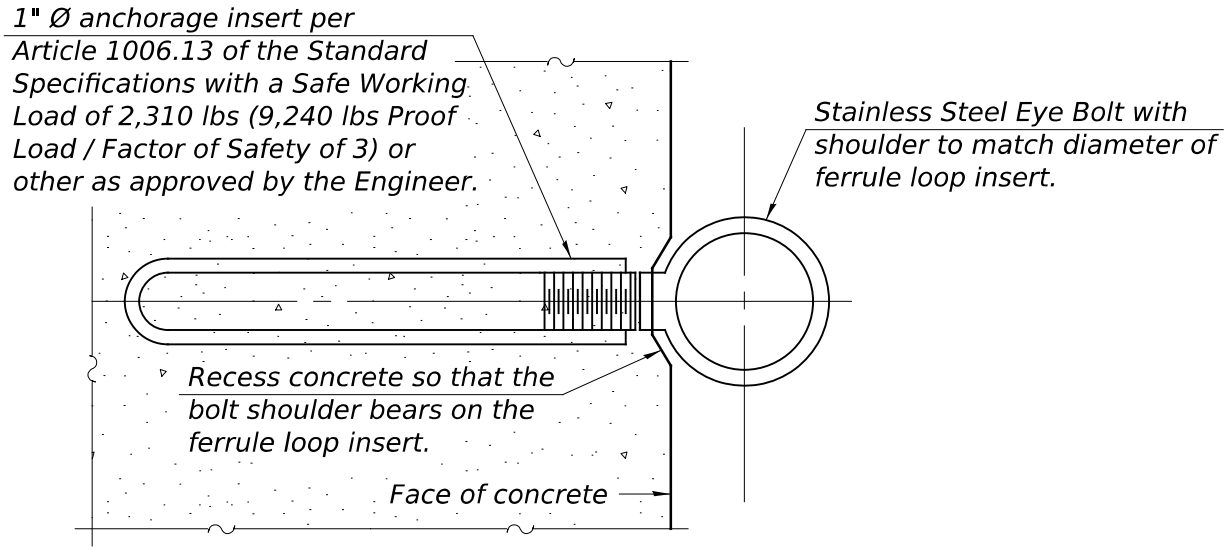
Skew Accommodation for Wrap-Around Walls

11. For MSE walls supporting abutments with wrap-around details per 3.8.9.5, the maximum skew angle of the wall is 20 degrees (e.g. forms an interior acute angle of 70 degrees). See Figure 3.8.9.5-2 and Figure 3.8.9.5-4. Larger abutment skews can be accomadated by increasing the plan area of the wall.

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PARTIAL ELEVATION OF F.F. ABUTMENT SHOWING EYE BOLT FALL PROTECTION OPTION



EYE BOLT DETAIL

(Cost included with Concrete Structures)

FALL PROTECTION DETAILS

Figure 3.8.9-1

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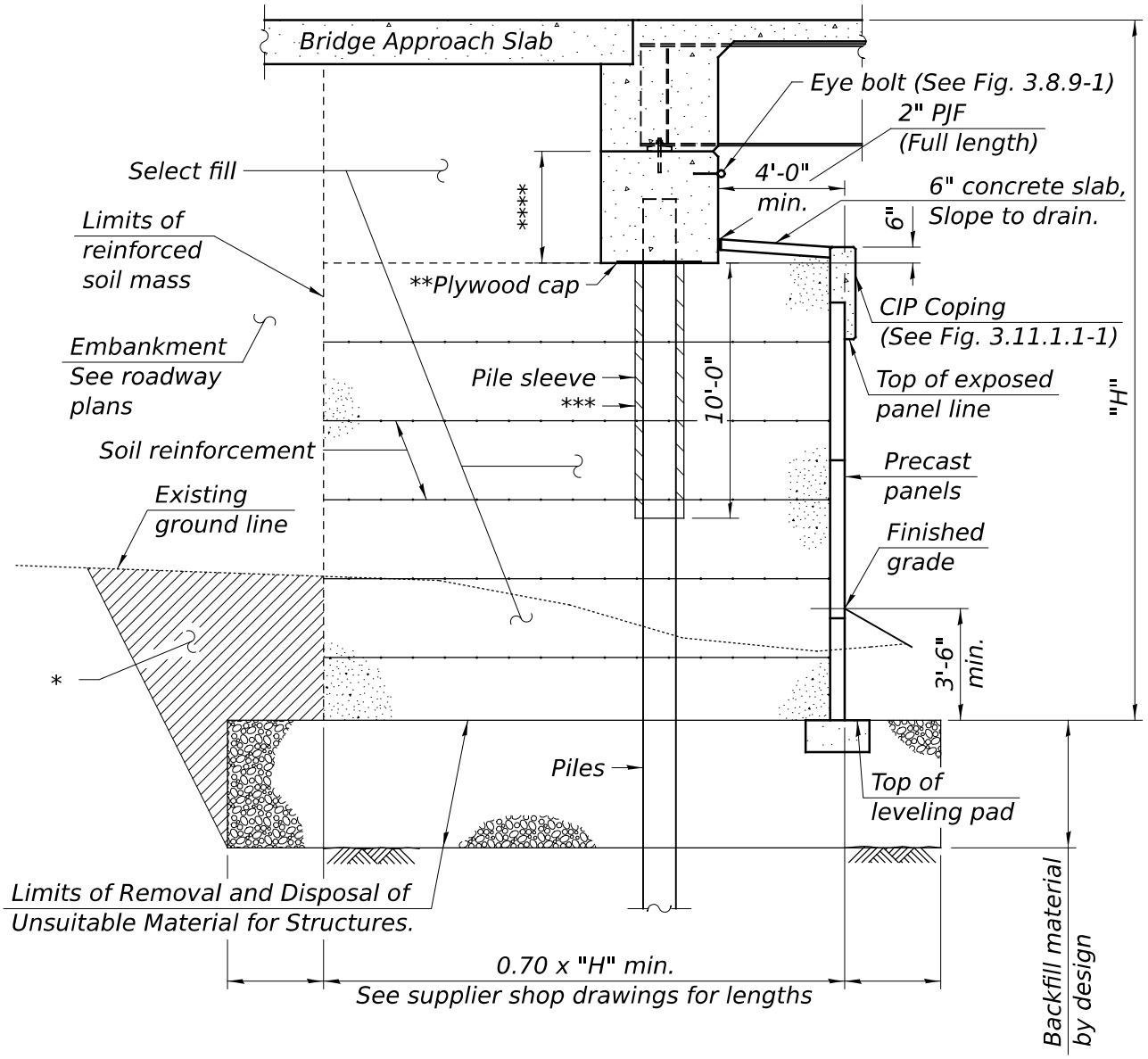
3.8.9.1 MSE Abutments: Integral

Integral abutments are permitted on MSE walls when it is determined to be the most feasible option and are required to meet the following additional requirements:

1. A pile sleeve of either polymer, plastic, or steel pipe shall be placed around each pile for the top 10 feet of the MSE select backfill, when the structure length is greater than 100 feet. The void between the pile and the pile sleeve shall remain empty. The minimum space between the pile and the pile sleeve shall be 3" or as required by design to accommodate the total pile movement.
2. The preferred integral abutment MSE wall orientation is straight and parallel to the feature crossed beneath. Depending on the vicinity of adjacent bridge or roadway elements, the MSE wall may need "kink" points in plan view. When necessary, the kink point(s) of these walls:
    a. shall be beyond the wingwalls
    b. shall be beyond where top strap clears bottom of abutment elevation.
3. A wrap-around MSE wall configuration can be specified per Section 3.8.9.5.
    a. "Wide" Wrap-around MSE integral abutment walls are preferred for layout and skews according to Section 3.8.9.5.
    b. "Compact" and "Access" Wrap-around MSE integral abutment walls shall not be used as these configurations were not modeled in the Department's Integral Abutment Feasibility Analysis.
4. The top of coping elevation shall be 6" above the bottom of abutment cap elevation. Wingwalls parallel with the abutment cap shall be used to control the remainder of the soil slope.

See Figure 3.8.9.1-1 for additional details for integral abutments on MSE walls.

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* *Overexcavation beyond structure excavation and removal of unsuitable material. This area not measured for payment. Backfill overexcavation with same material used for select fill used in MSE wall.*

** *Bottom of cap poured against top of plywood. Cut opening to match pile perimeter within 18". Support with bars tack welded to webs rated for 500 lbs. Seal gaps to keep concrete out. Cost included with Furnishing Piles of the type/size specified.*

*** *Sleeve to remain empty in hatched region.*

**** *3'-6" min. for girder bridges and 4'-6" min. for all other cases.*

MSE WALL SECTION AT INTEGRAL ABUTMENT

Figure 3.8.9.1-1

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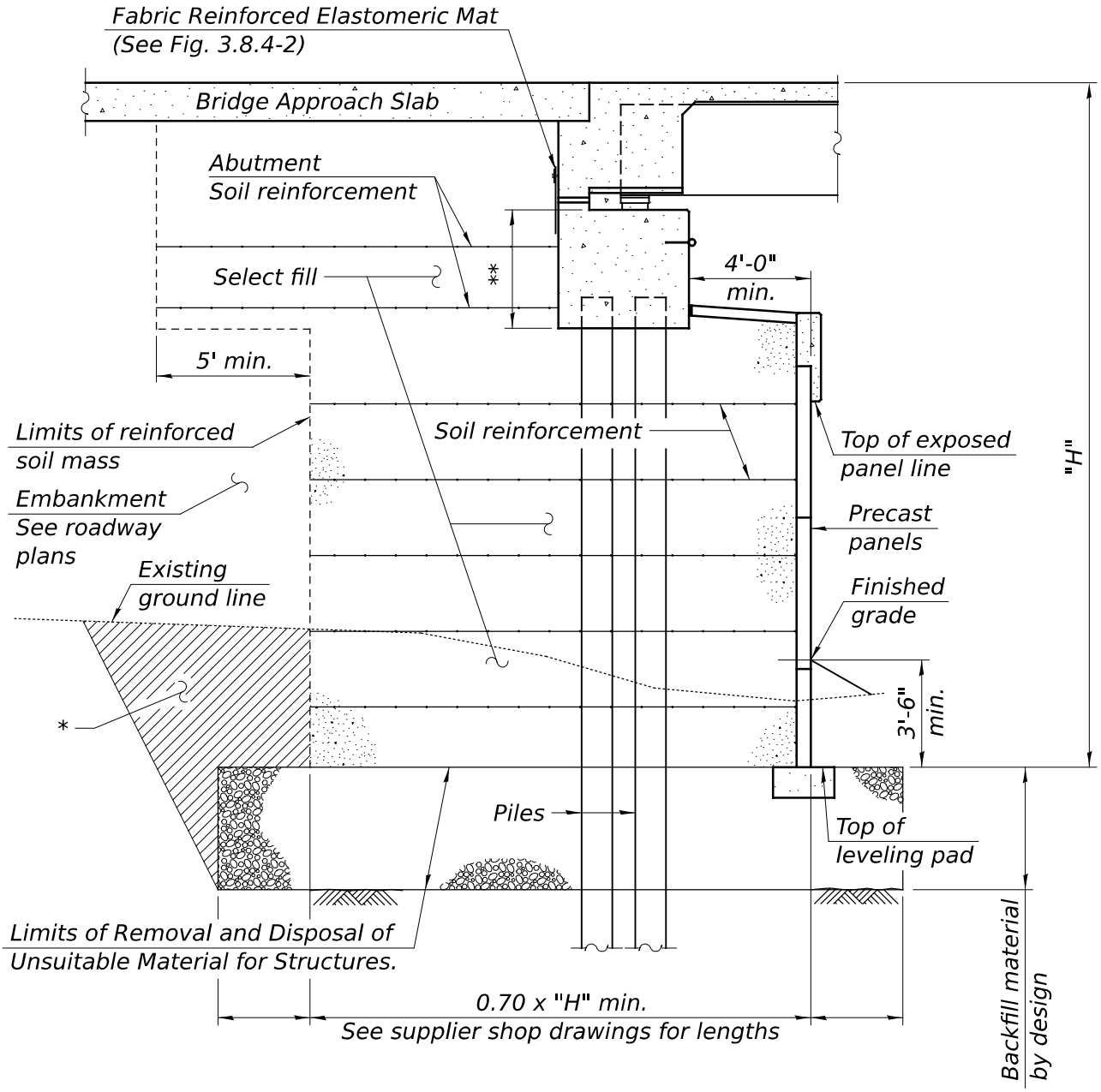
3.8.9.2 MSE Abutments: Semi-Integral

MSE walls for semi-integral abutments shall be detailed similar to integral abutments as described in requirements 1-5 of Section 3.8.9.1, except that for requirement 3

3. A wrap-around MSE wall configuration can be specified per Section 3.8.9.5.
    a. "Wide" Wrap-around MSE semi-integral abutment walls are permitted for layout and skews according to Section 3.8.9.5.
    b. "Compact" Wrap-around MSE semi-integral abutment walls shall not be used as this configuration contains limited room to accommodate the necessary seat, bearing, and pile elements.
    c. Use of "Access" Wrap-around MSE semi-integral abutment walls should be discussed with the BBS to ensure that the adjacent wing wall types can function with the longitudinal substructure movements.

See Figure 3.8.9.2-1 and Figure 3.8.9.2-2 for section details. Semi-integral abutments are permitted to have soil reinforcement attached to the back of the abutment below the Fabric Reinforced Elastomeric Mat.

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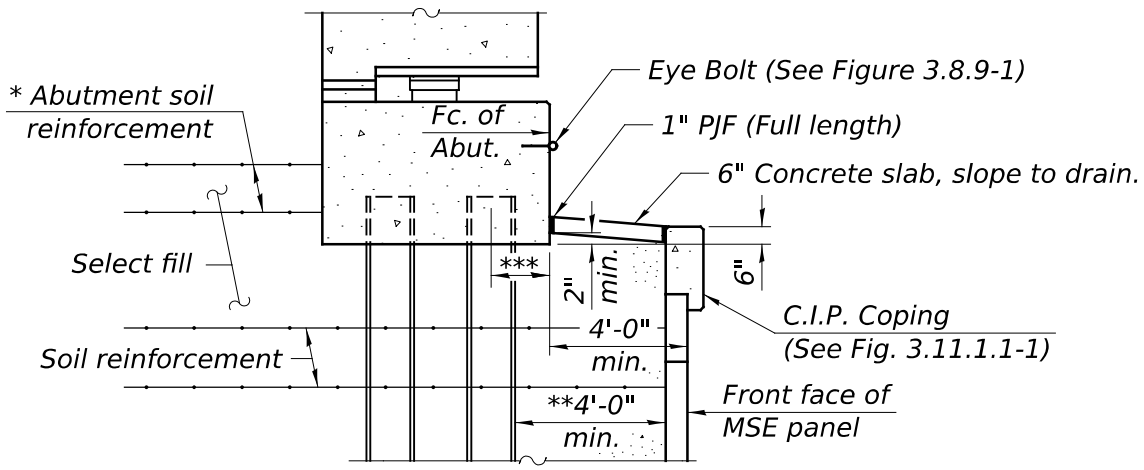
* *Overexcavation beyond structure excavation and removal of unsuitable material. This area not measured for payment. Backfill overexcavation with same material used for select fill used in MSE wall.*

** *3'-6" min. for girder bridges and 5'-6" min. for all other cases.*

MSE WALL SECTION AT SEMI-INTEGRAL ABUTMENT

Figure 3.8.9.2-1

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Notes:
Semi-integral abutment shown, stub-type abutment similar. Piles shown, drilled shaft similar. Battered piles not permitted for new abutments on MSE walls

** The MSE wall supplier shall design the abutment soil reinforcement to resist a horizontal force of _ kips/ft. of abutment.*

*** When using drilled shafts larger than 4 ft. in diameter, increase clear space to equal the diameter of the drilled shaft.*

**** See Figure 3.8.4-7 for semi-integral abutments. See Figure 3.8.5-2 for stub abutments.*

PILE SUPPORTED ABUTMENT ON MSE WALL DETAIL

Figure 3.8.9.2-2

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3.8.9.3 MSE Abutments: Spread Footing

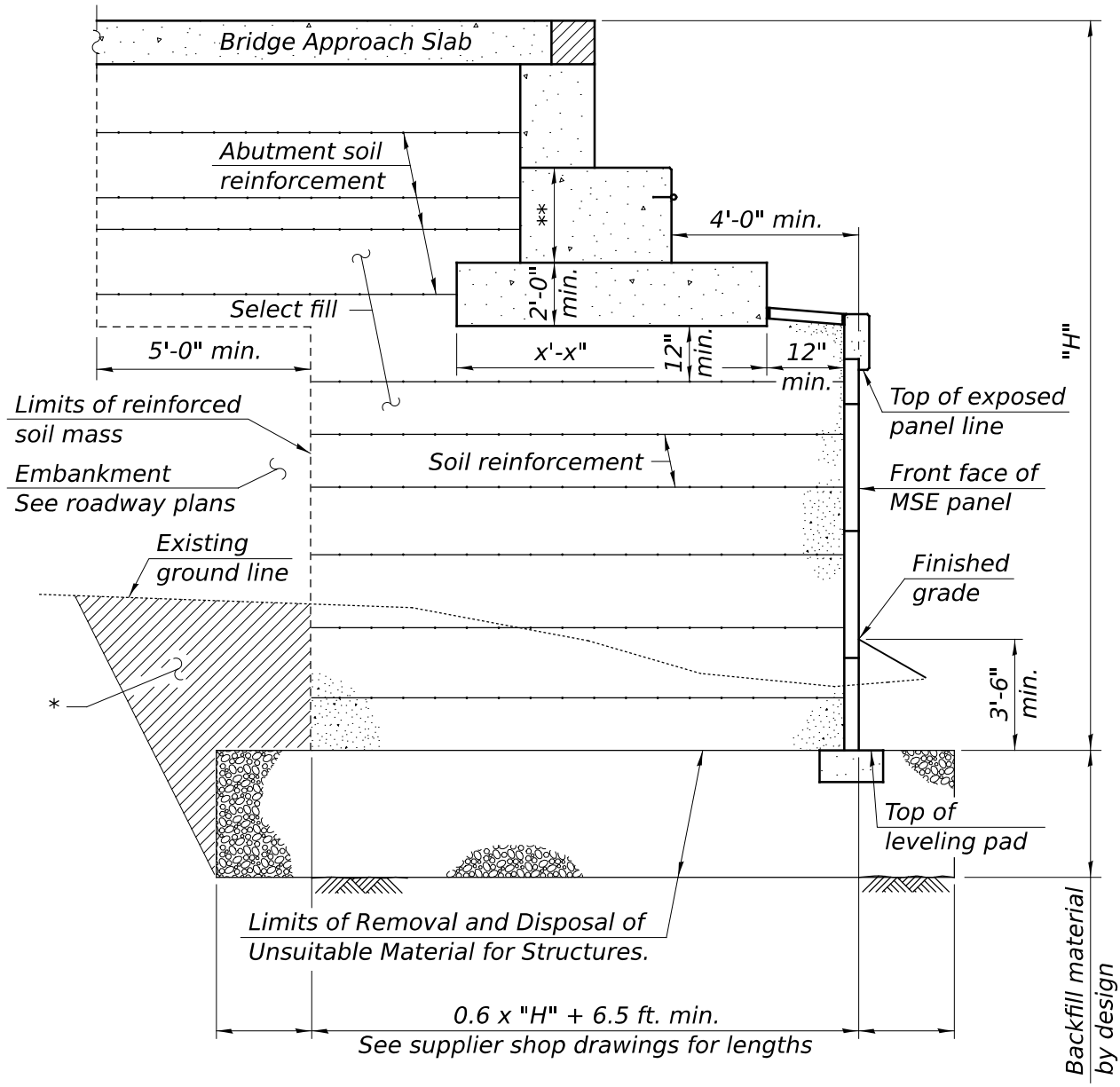
Details for abutments on spread footings retained with MSE walls are given in Figure 3.8.9.3-1 and Figure 3.8.9.3-2. This configuration may be considered when the integral or semi-integral configurations cannot be used. If the MSE system is situated on stiff soils or high rock, and if the surcharge demands from the superstructure are manageable, this configuration omits the need to drive piles.

A minimum of 12” between the bottom of spread footings and the soil reinforcement below it is required. This reduces the stress in the soil reinforcement and its connection. The top of the coping shall be a minimum of 6” higher than the bottom of the spread footing. The coping shall not be higher than the toe of the spread footing.

For abutments on spread footings, the designer shall size the footing such that the service surcharge load from the abutment is no more than 4 ksf. The Contract plans shall also include a note which indicates the maximum horizontal service sliding force along with the vertical surcharge load so the MSE wall supplier can account for it during design.

A wrap-around MSE wall configuration can be specified per Section 3.8.9.5.

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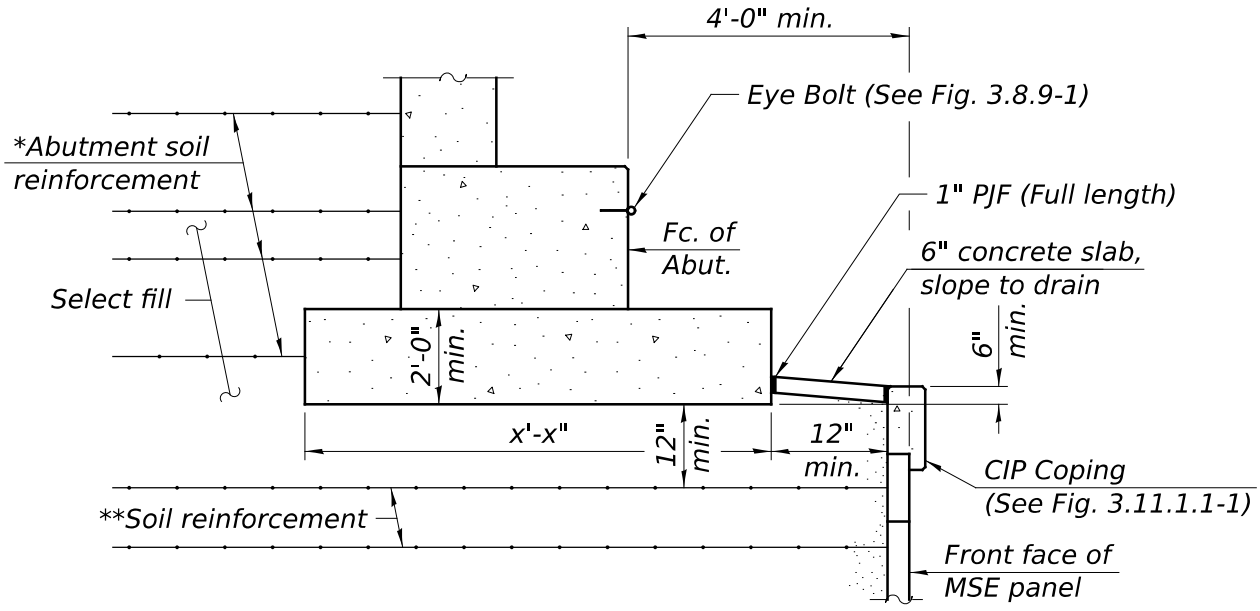
* Overexcavation beyond structure excavation and removal of unsuitable material. This area not measured for payment. Backfill overexcavation with same material used for select fill used in MSE wall.

** 3'-6" min. for girder bridges and 5'-6" min. for all other cases.

MSE WALL SECTION AT SPREAD FOOTING ABUTMENT

Figure 3.8.9.3-1

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* The MSE wall supplier shall design the abutment soil reinforcement to resist a horizontal force of _ kips/ft. of abutment.

** The MSE wall supplier's internal stability design shall account for the footing's bearing pressure surcharge of 4 ksf and horizontal sliding force of _ kips/ft. of abutment.

SPREAD FOOTING ABUTMENT ON MSE WALL DETAIL

Figure 3.8.9.3-2

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3.8.9.4 MSE Abutments: Stub-Type

The Semi-Integral Abutment figure in Figure 3.8.9.2-1 and Figure 3.8.9.2-2 may also be used as standard detailing for a pile supported stub abutment, except stub abutments require more soil reinforcement.

Stub-type pile supported abutments require soil reinforcement to be attached to the full height of the backwall and abutment cap because the MSE wall prevents the use of battered piles to resist lateral loadings. See Figures 3.8.9.2-1 and 3.8.9.2-2. As such, the Contract plans shall include a note which indicates the maximum horizontal service force applied to the abutment so the MSE wall supplier can account for it during design (see Figure 3.8.9.2-2).

A wrap-around MSE wall configuration can be specified per Section 3.8.9.5.

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3.8.9.5 MSE Abutments: Wrap-Around

An MSE Wall can be oriented in a wrap-around configuration about an abutment. Variations of this wrap-around MSE abutment depend on the structural planner/designer needs for minimal plan footprint or consistent access for bridge inspection, and roadway elements between the MSE walls/footings at grade separations shall be the responsibility of the roadway planner/designer. The Department identifies wrap-around MSE abutments into three categories shown in Table 3.8.9.5-1. Guidance on the preferred abutment type and their permitted wingwall types can be found in Table 3.8.9.5-2.

Wrap-Around MSE Abutment "Type"	**Plan Footprint**	**Applications**
Compact	Smallest	Limited urban or geometric clearances
Access	Medium	Structures requiring inspection access (e.g. adjacent to railroads and high-AADT highways)
Wide	Largest	Accommodates elements adjacent to bridge (e.g. wingwalls, walks & pathways)

Table 3.8.9.5-1

M.S.E. Abutment Type		**IA**	**SIA**	**Stub**	**Spread**
Wrap-Around MSE Abutment Type	Compact	X	X	●	●
	Access	X	-	√	√
	Wide	√	●	●	●
Wing Wall Types Permitted on M.S.E. Abutment		CIP	CIP	CIP & M.S.E.	CIP & M.S.E.

Legend:

Check (√)= Preferred, soils permitting.

Dot (●)= Permitted

Dash (-) = Contact BBS

X = Not permitted

IA = Integral abutment

SIA = Semi-integral abutment

Table 3.8.9.5-2

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“Compact” wrap-around MSE abutments offer the smallest plan footprint. This "high and tight" geometry accommodates grade separations with minimal clearance or situations not suitable for traditional closed abutments.



Photo: “Compact” Wrap-Around Example

“Wide” wrap-around MSE abutments can situate around integral abutments with wingwalls parallel to the feature crossed, and the MSE walls can wrap around those wingwalls, thus resulting in the largest plan footprint while accommodating the ROW needs of the project.



Photo: “Wide” Wrap-Around Example

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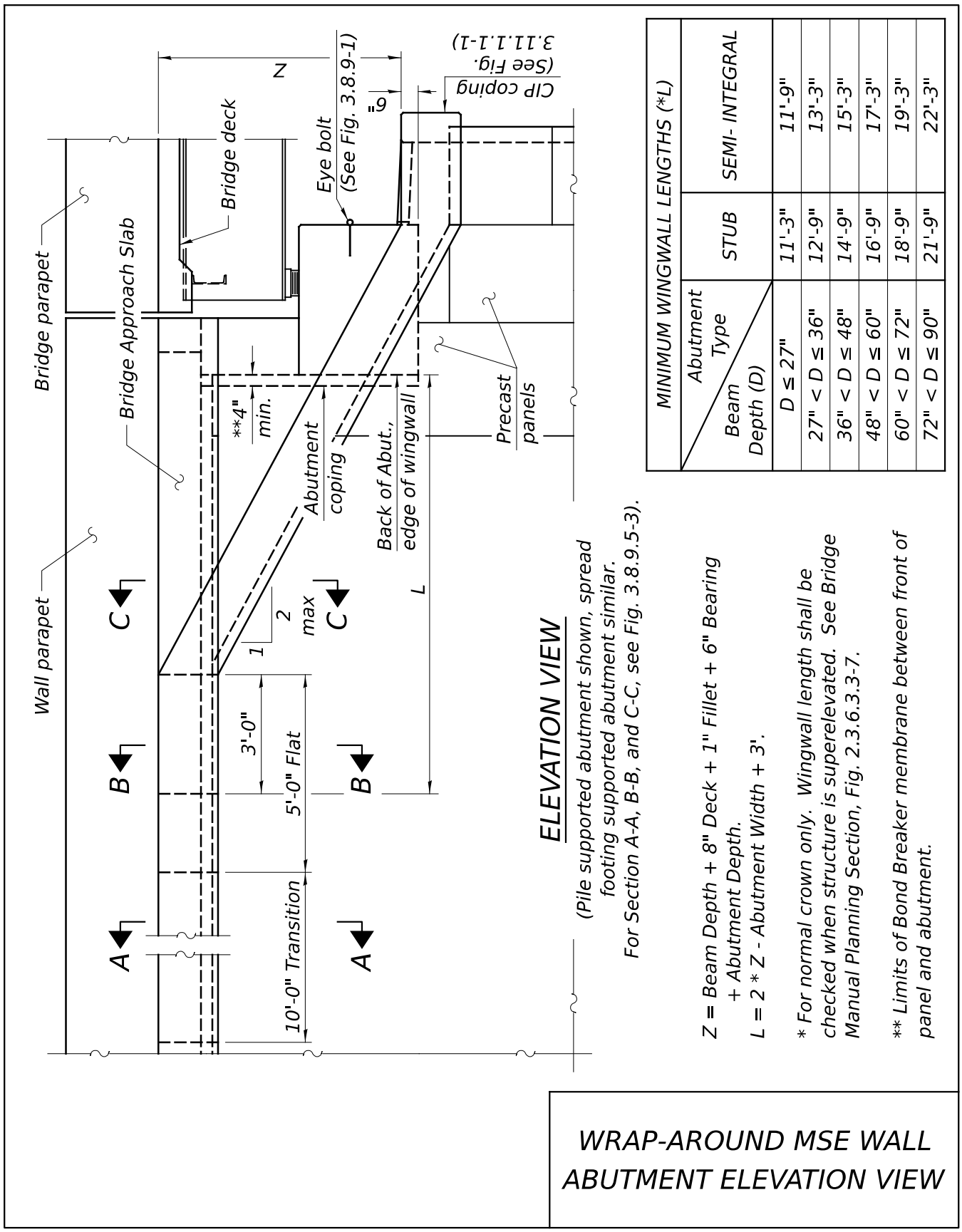
"Access" wrap-around MSE abutments provide a path/walkway for easier inspection of elements at bridge ends. Because projects adjacent to railroads or high-ADT roads have restrictions for equipment (e.g. ladders and lifts), the integrated access and components offer multiple safety measures for the Owner to consider. Figure 3.8.9.2-1 through Figure 3.8.9.2-3 presents standard details for a pile supported abutment with wrap-around MSE wingwalls. Corner details are shown on Figure 3.8.9.2-4.



Photo: "Access" Wrap-Around Example

A 4' wide walkway shall be required between the wingwalls and the front face of the MSE retaining wall in front and wrapping around it. This allows the walkway to extend around the sides of the abutment and wingwalls so that inspectors can walk down the walkway to inspect underneath the bridge. This may require a superimposed (tiered) analysis depending on the wall heights. Guidance for superimposed (tiered) walls can be found in the FHWA "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes" NHI manuals (NHI-10-024 and NHI-10-025).

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MINIMUM WINGWALL LENGTHS (*L)		
Beam Depth (D) / Abutment Type	STUB	SEMI- INTEGRAL
D ≤ 27"	11'-3"	11'-9"
27" < D ≤ 36"	12'-9"	13'-3"
36" < D ≤ 48"	14'-9"	15'-3"
48" < D ≤ 60"	16'-9"	17'-3"
60" < D ≤ 72"	18'-9"	19'-3"
72" < D ≤ 90"	21'-9"	22'-3"

WRAP-AROUND MSE WALL ABUTMENT ELEVATION VIEW

Figure 3.8.9.5-1

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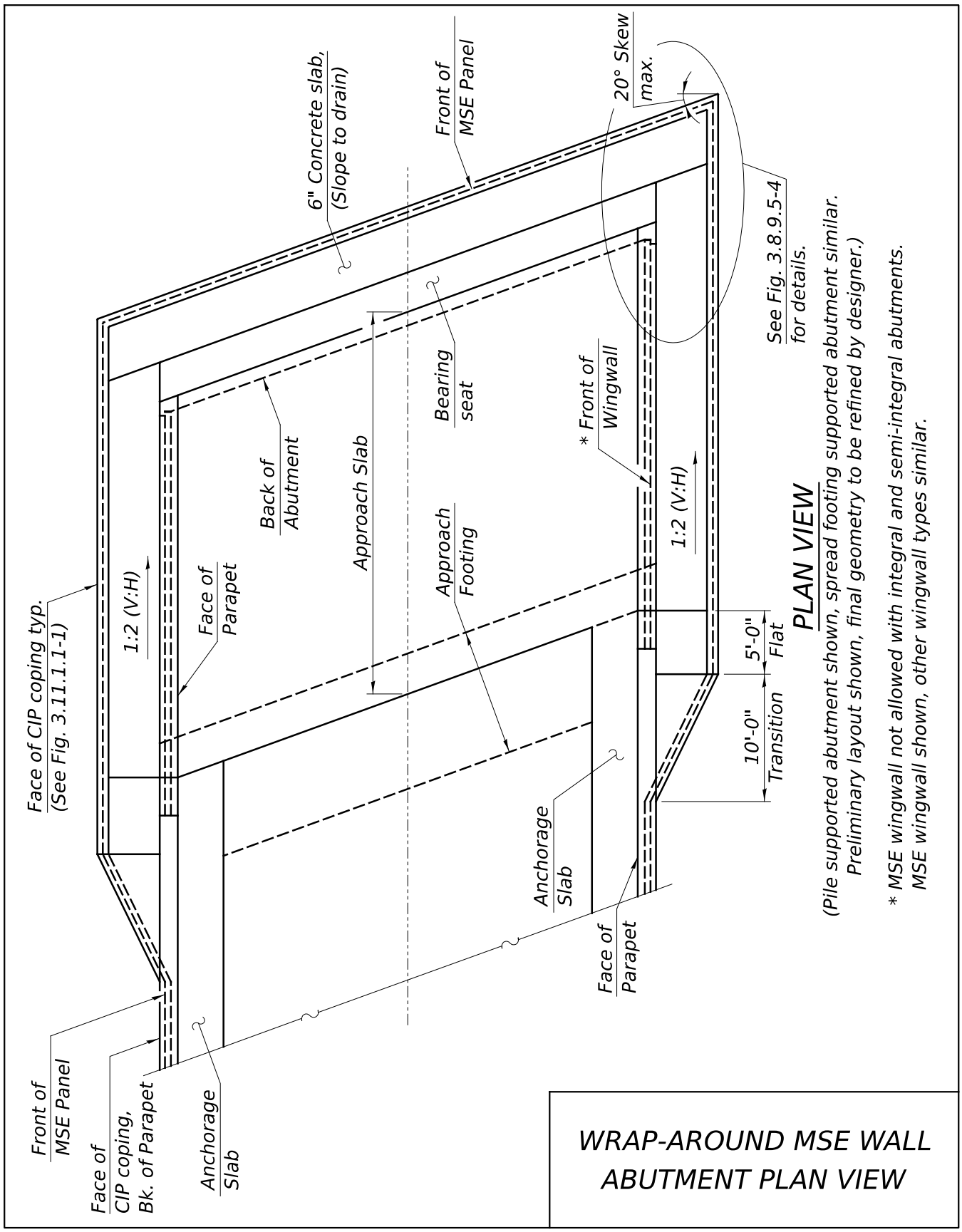


Figure 3.8.9.5-2

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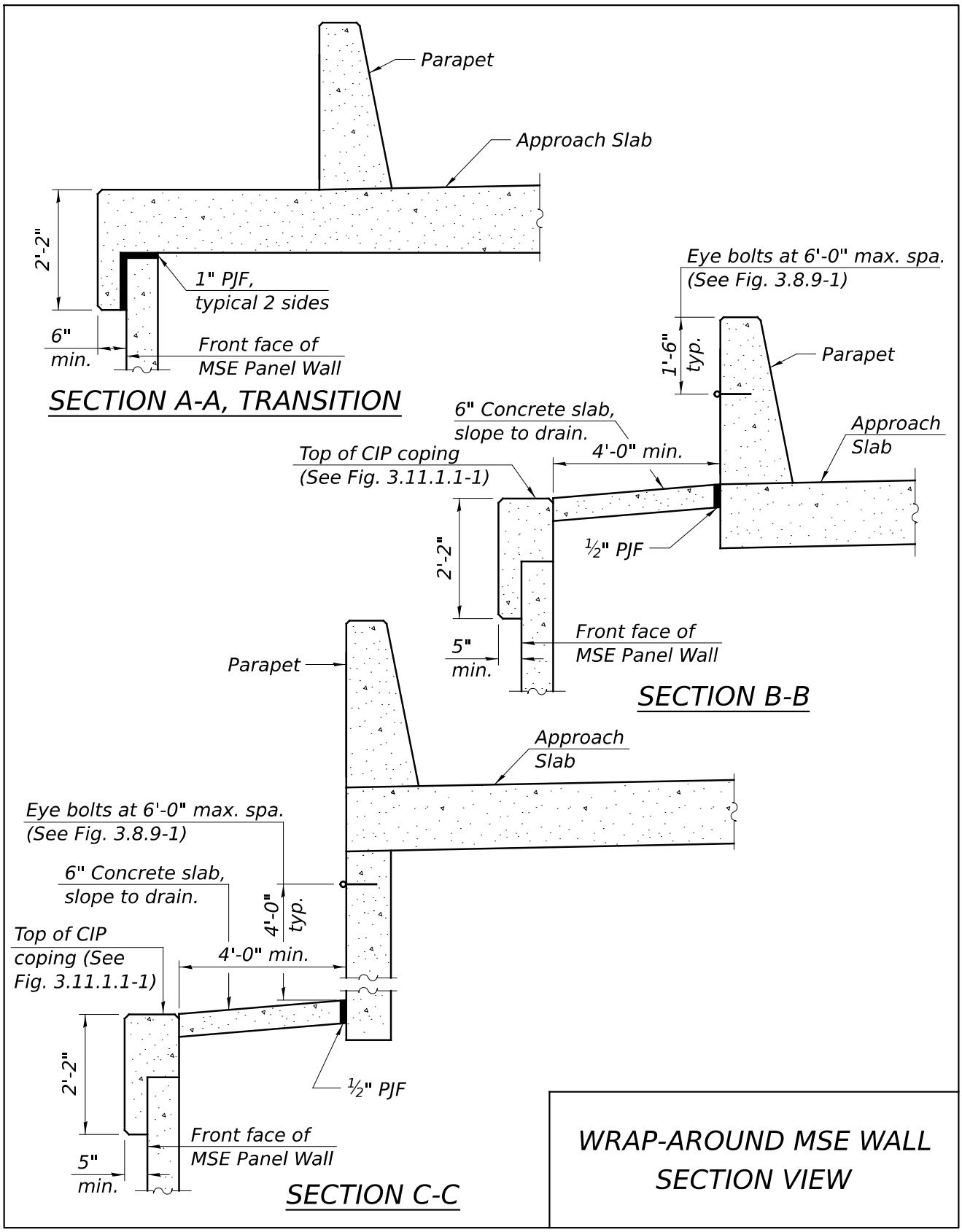
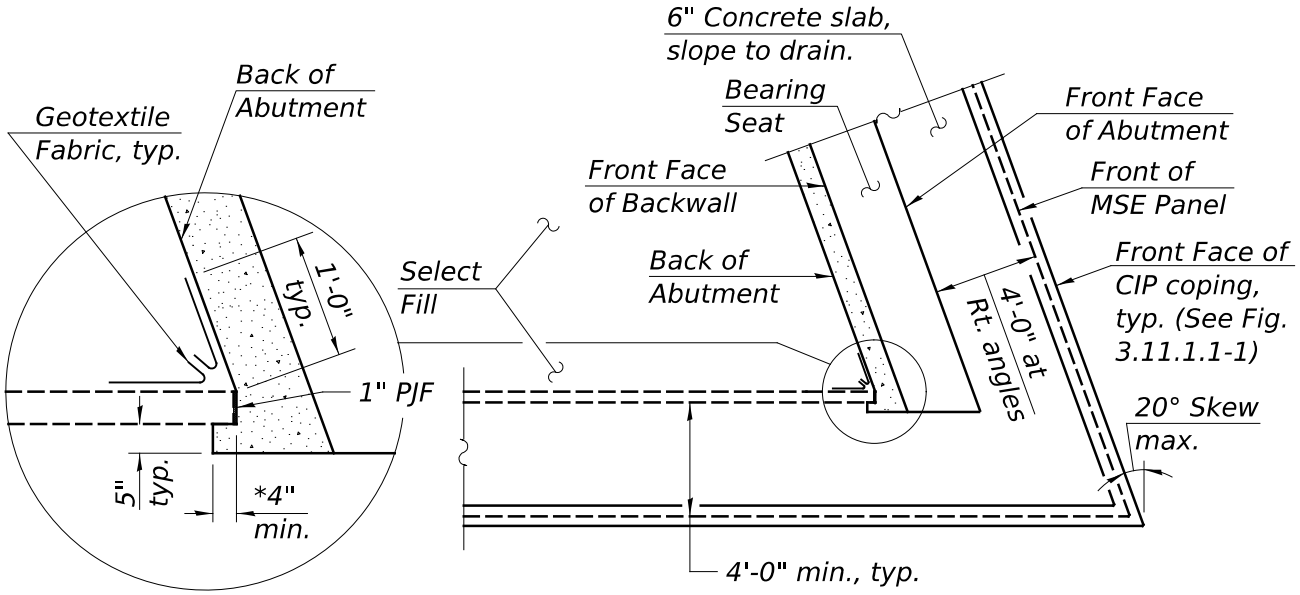
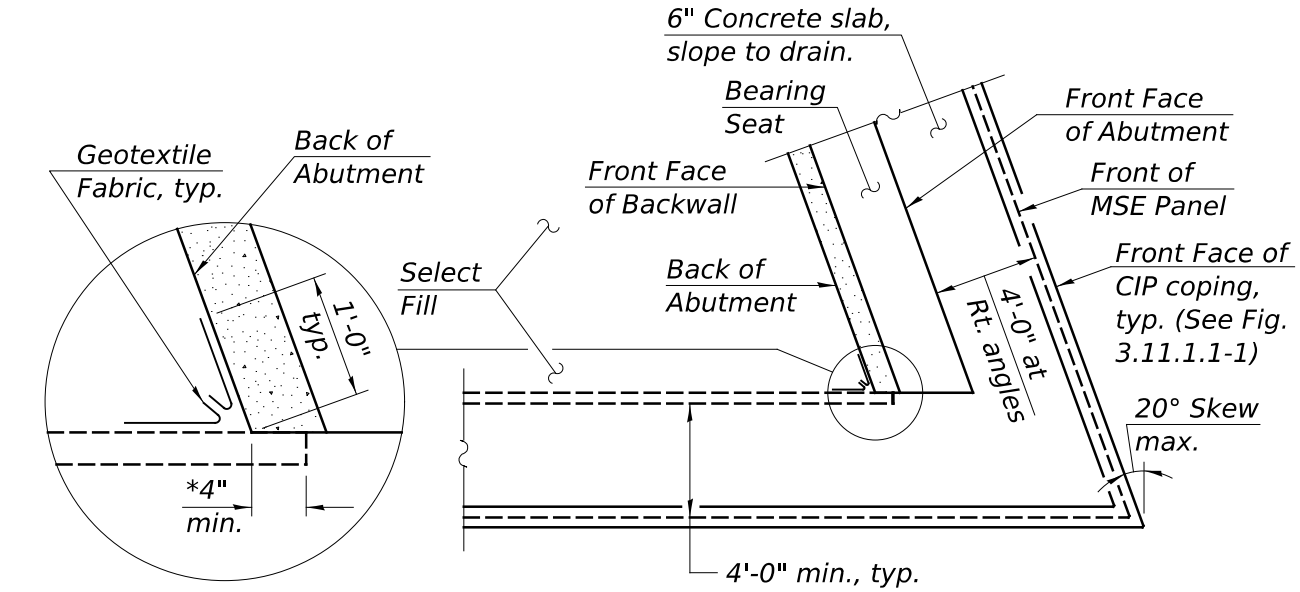


Figure 3.8.9.5-3

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EXPANSION ABUTMENT DETAIL



SEMI-INTEGRAL ABUTMENT DETAIL

Notes:

MSE wingwall detail shown. Other wingwall types are similar. See Section 3.8 for the appropriate wingwall to abutment connection detail.

** Limits of Bond Breaker membrane between panel and abutment.*

WRAP-AROUND MSE WALL CORNER DETAILS

Figure 3.8.9.5-4

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3.8.10 Approach Slab Support

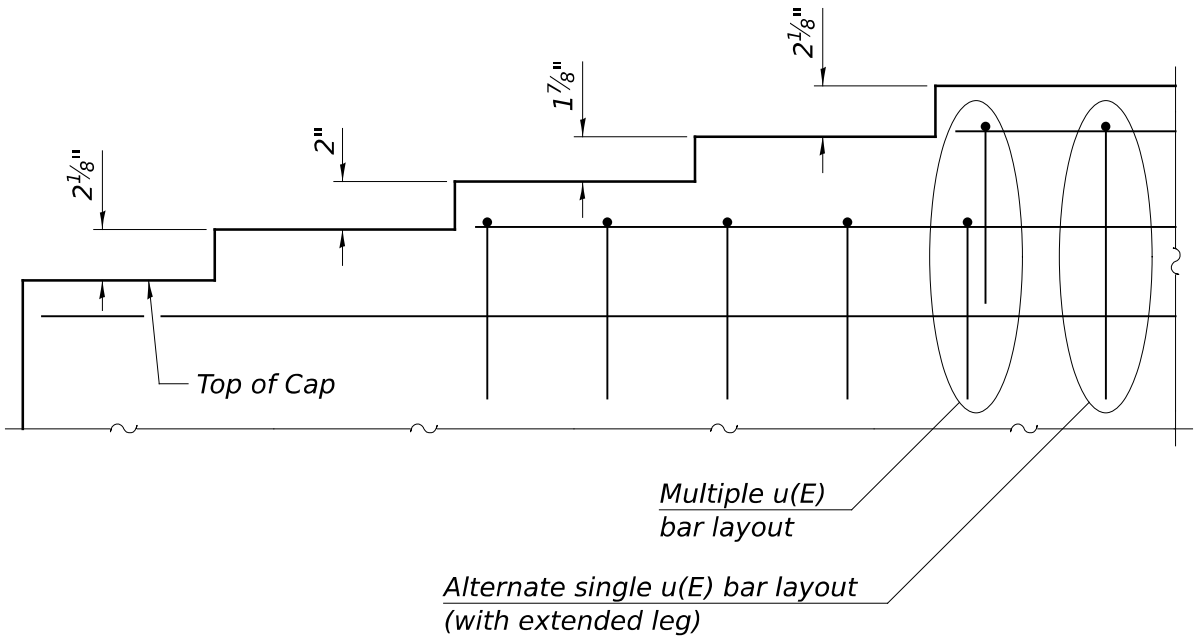
For integral, semi-integral, and stub abutment structures with cast-in-place approach slabs, the approach footing (e.g. the approach slab support) shall be constructed using top and bottom elevations as shown in the "Approach Slabs-Bridge Cast in Place" IDOT CADD Bridge Cell Library. The approach footing is generally constant depth to follow the bottom of the approach slab. The generic control points on Figures 3.8.3-9 and 3.8.5-8 simplify the approach slab layout for approach roadway/shoulder slopes to ensure that the approach slab thicknesses are met/surpassed and that the bottom of approach footing is prepared to the correct elevation.

The unfactored dead load reaction of the standard approach pavement without parapets shall be a minimum of 3.0 kips per ft. of width. The designer shall add the weight of parapets or railings to this minimum load (e.g. the addition of two 15 ft. parapets may increase the reaction to 3.4 kips per ft. for a 30 foot wide bridge). These reactions should not be considered if it reduces the load on the piles or drilled shafts. On all abutments, regardless of the type of planned approach roadway, a support shall be provided for a rigid-type approach pavement.

3.8.11 Bridge Seats

The bridge seats shall be constructed in steps poured monolithically with the abutment. The minimum step shall be ¾ in. For steel beams, steel fill plates shall be provided at bearings if steps are less than ¾ in. For concrete beams, if steps are less than ¾ in., the difference in top of deck elevations shall be incorporated into the beam fillets. The elevation of each seat shall be shown on the plans. Steps 4 in. or larger shall be reinforced (see Figure 3.8.11-1). All steps shall be perpendicular to the face of the abutment cap (with a possible exception at stage construction joints). In all cases, the bridge seats between the bearings shall be sloped to drain. Figure 3.8.11-1 presents a simple sketch of typical bridge seats. See Section 3.15.4.2 for minimum required seat widths for seismic design considerations.

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ELEVATION

BEAM SEAT
STEP HEIGHTS

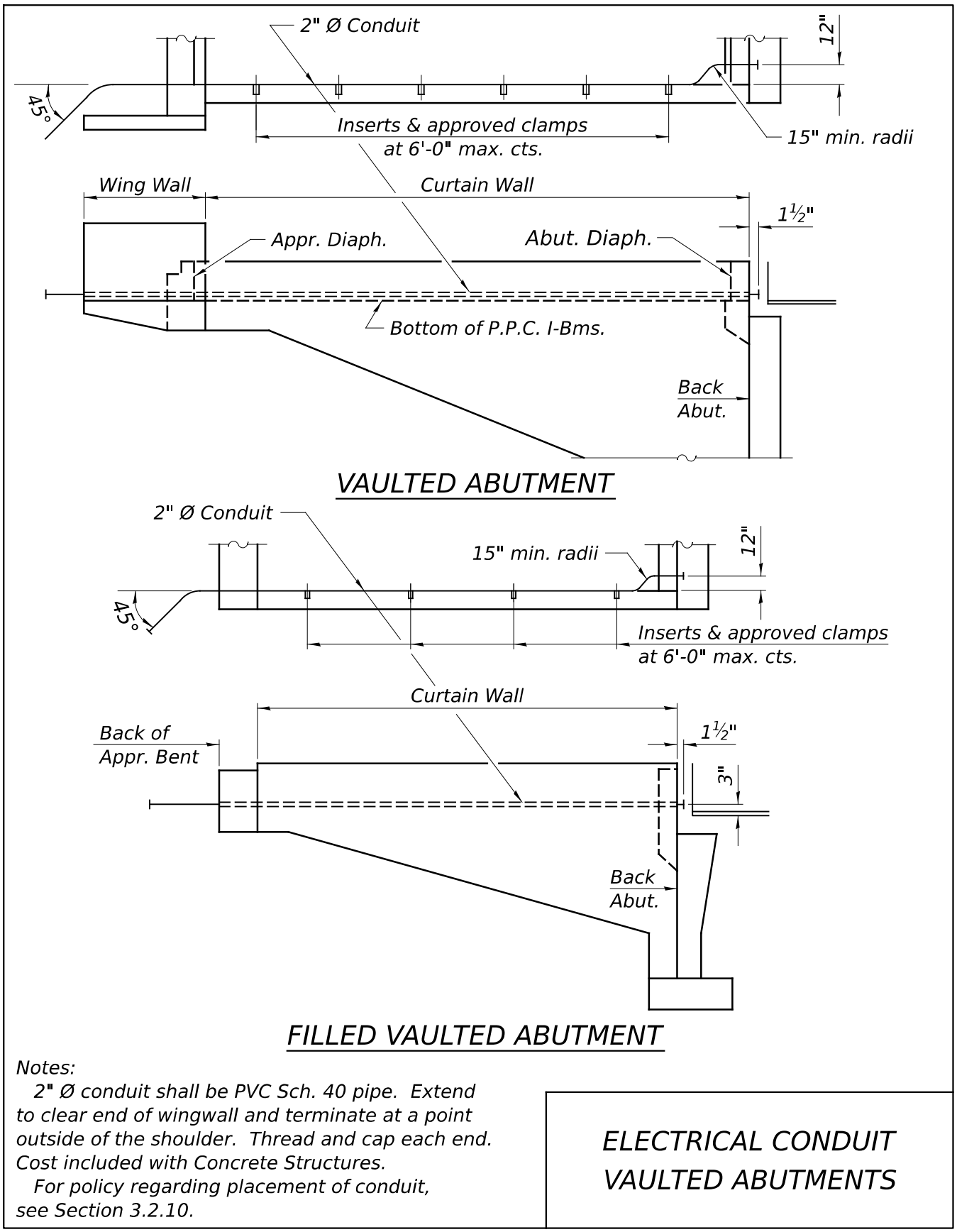
Figure 3.8.11-1

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3.8.12 Electrical Conduit in Abutments

Figure 3.8.12-1 presents details for electrical conduit in vaulted abutments. Details for electrical conduit in pile supported stub abutments are given in Figure 3.8.12-2.

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Notes:

2" Ø conduit shall be PVC Sch. 40 pipe. Extend to clear end of wingwall and terminate at a point outside of the shoulder. Thread and cap each end. Cost included with Concrete Structures.

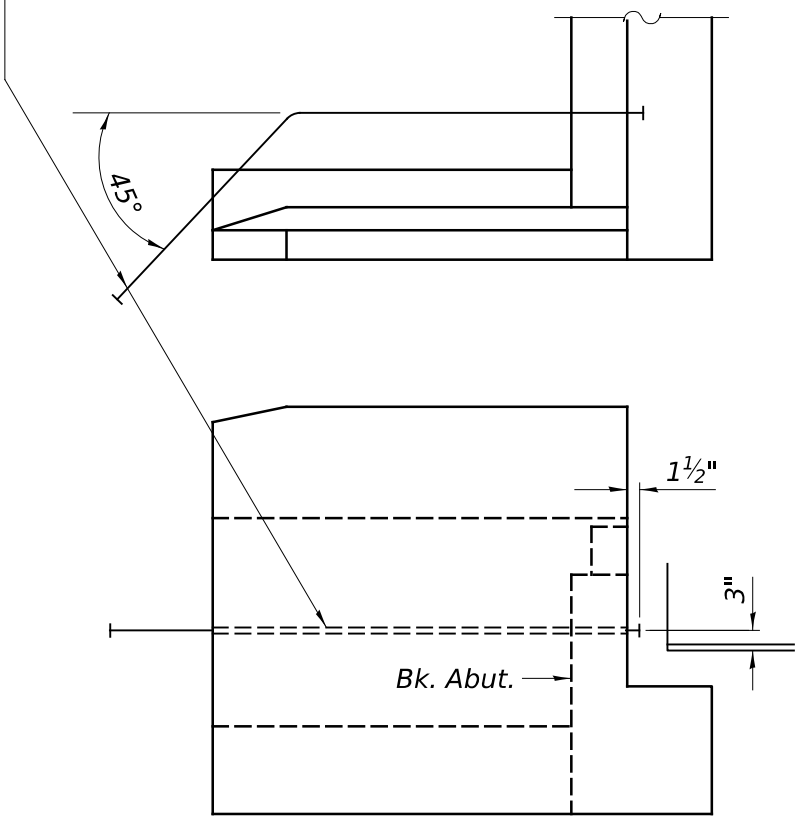
For policy regarding placement of conduit, see Section 3.2.10.

ELECTRICAL CONDUIT VAULTED ABUTMENTS

Figure 3.8.12-1

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Locate 2" Ø Conduit ±12" inside of fascia beam web and parallel to beam line. Extend to clear the wingwall and terminate at a point outside of shoulder. Thread and cap each end. Cost included with Concrete Structures.



STUB ABUTMENT

Note:
For policy regarding placement of conduit, see Section 3.2.10.

ELECTRICAL CONDUIT STUB ABUTMENTS

Figure 3.8.12-2

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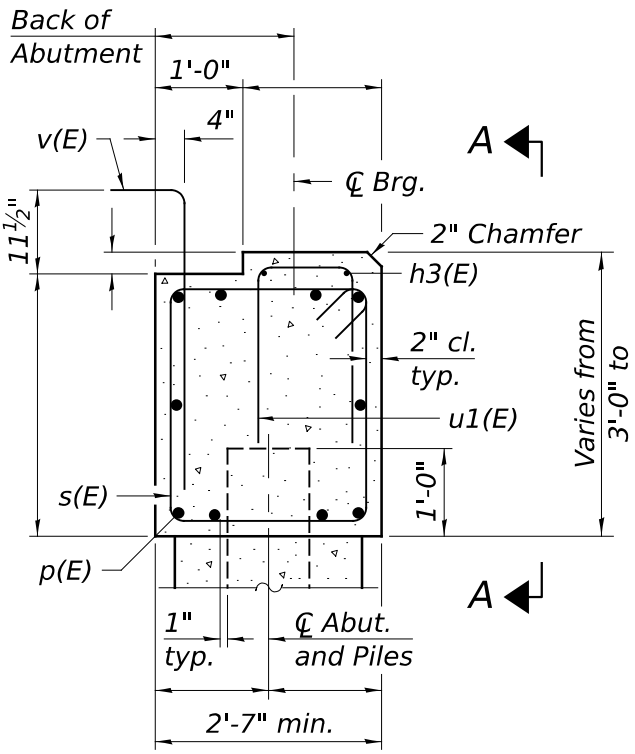
3.8.13 Abutments for Deck Beams

Abutment details for deck beam structures are shown in Figures 3.8.13-1 through 3.8.13-4. The general geometry is depicted for all depths of deck beams. A reinforcement scheme is also presented however the size of the reinforcement bars shall be determined by design.

Several factors shall be considered when determining the overall abutment cap length. The Standard Specifications note that the total width of the deck shall be the theoretical width plus ½ inch per deck beam joint. The cap shall ideally be an additional 6 inches wider than the deck on each end and 9 inches wider on each end when a side retainer is required at an expansion joint. See Section 3.5.7.5 for details of the side retainer. All of the aforementioned dimensions are at right angles to the centerline of the roadway and do not account for the additional cap length associated with skew. Note that the abutment cap ends are typically skewed parallel to the centerline of roadway as depicted on the base sheets.

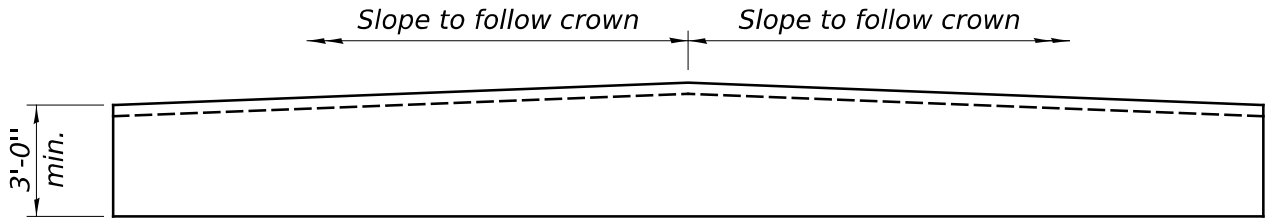
Abutments for deck beam structures require pile encasements.

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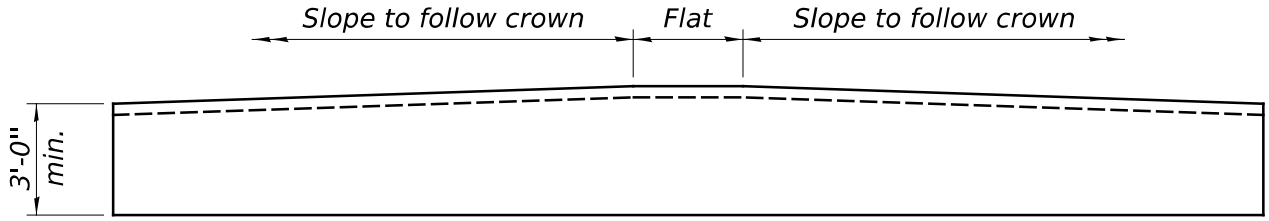
SECTION THRU ABUTMENT

(Dimensions are at Rt. ∠'s)



VIEW A-A

(For even number of beams)



VIEW A-A

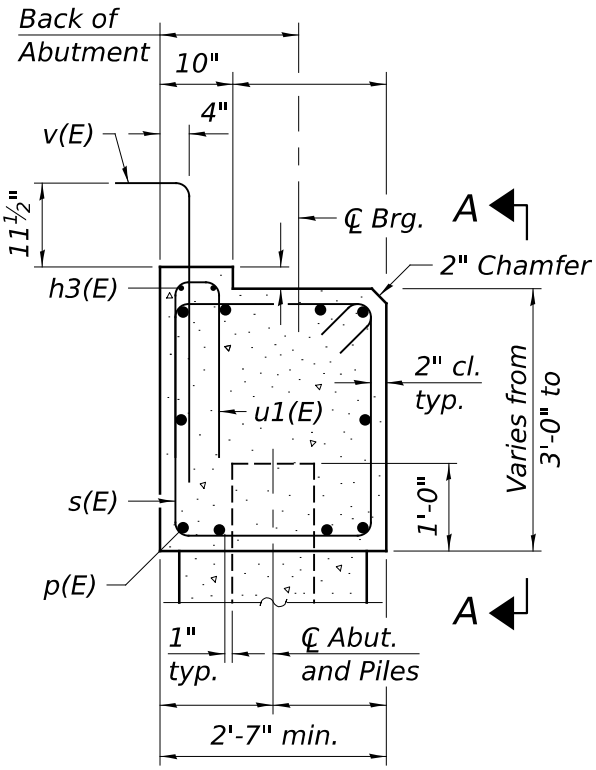
(For odd number of beams)

Notes:
Only applicable for 11" Deck Beams with HMAWS.

ABUTMENT DETAILS FOR DECK BEAMS WITH APPROACH PAVEMENT AND NO BACKWALL

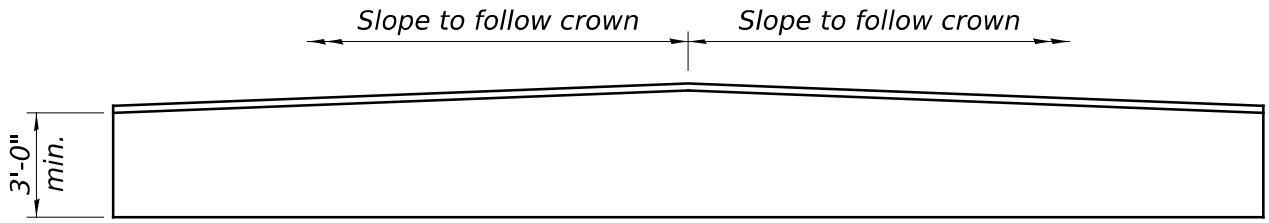
Figure 3.8.13-1

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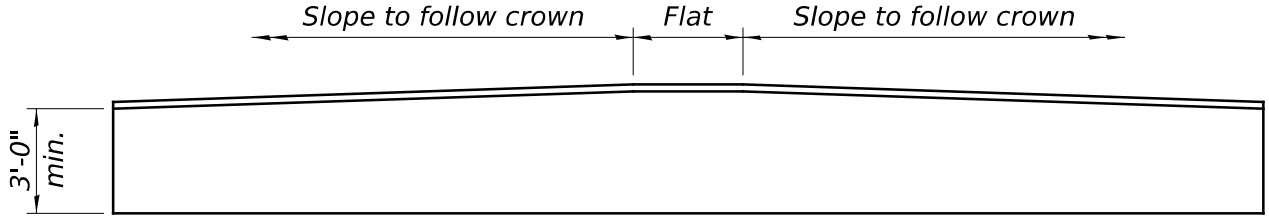
SECTION THRU ABUTMENT

(Dimensions are at Rt. ∠'s)



VIEW A-A

(For even number of beams)



VIEW A-A

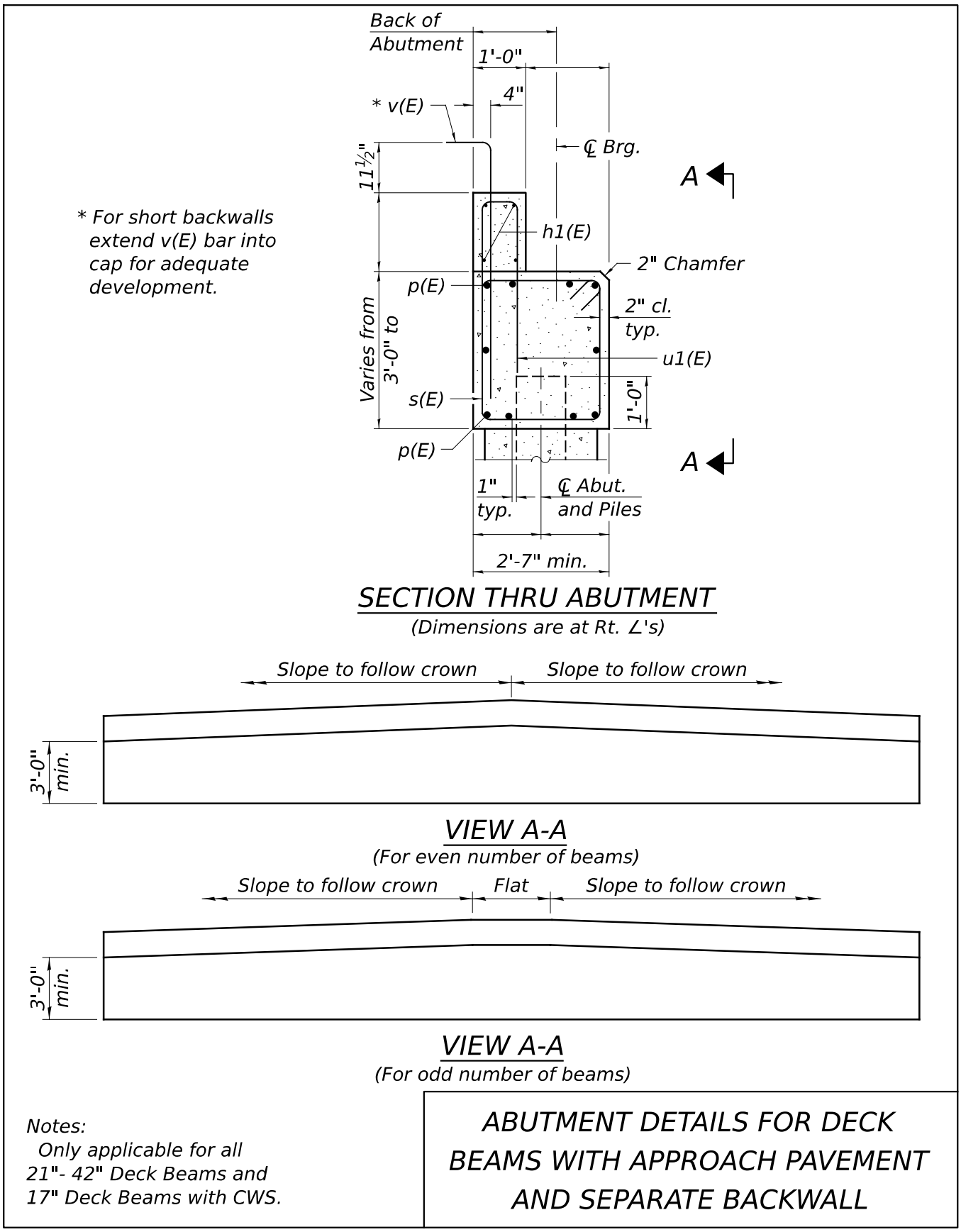
(For odd number of beams)

Notes:
Only applicable for
11" Deck Beams with CWS, and
17" Deck Beams with HMAWS.

ABUTMENT DETAILS FOR DECK BEAMS WITH APPROACH PAVEMENT AND MONOLITHIC BACKWALL

Figure 3.8.13-2

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SECTION THRU ABUTMENT
(Dimensions are at Rt. ∠'s)

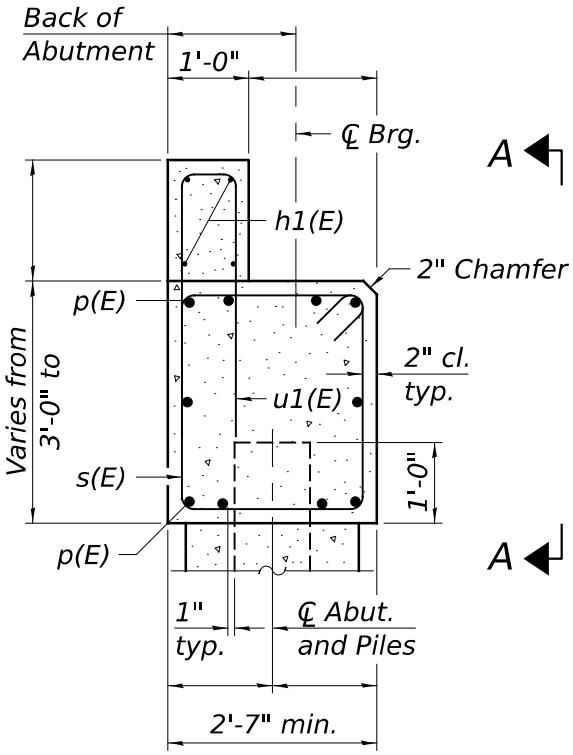
VIEW A-A
(For even number of beams)

VIEW A-A
(For odd number of beams)

ABUTMENT DETAILS FOR DECK BEAMS WITH APPROACH PAVEMENT AND SEPARATE BACKWALL

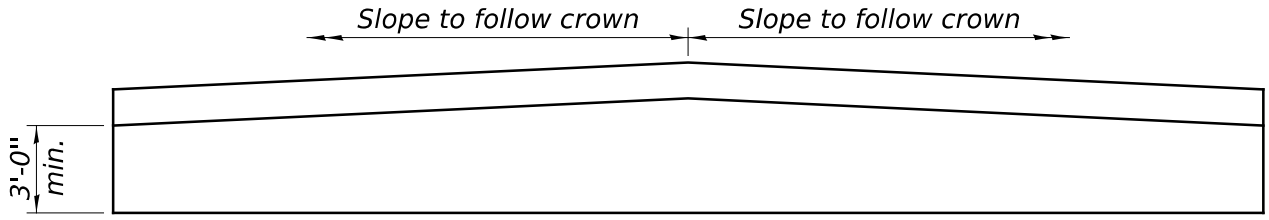
Figure 3.8.13-3

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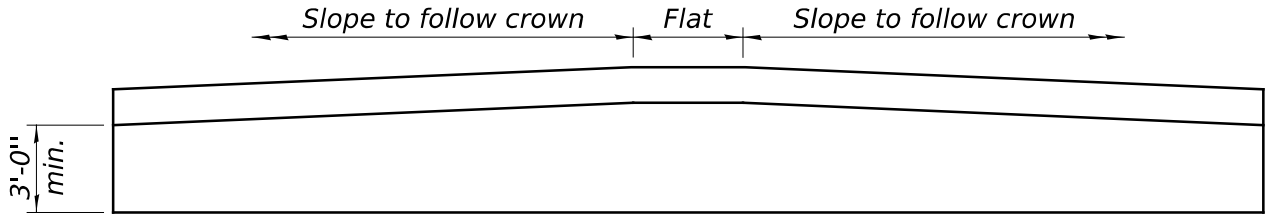
SECTION THRU ABUTMENT

(Dimensions are at Rt. ∠'s)



VIEW A-A

(For even number of beams)



VIEW A-A

(For odd number of beams)

ABUTMENT DETAILS FOR DECK BEAMS WITH SEPARATE BACKWALL AND NO APPROACH PAVEMENT

Figure 3.8.13-4

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3.8.14 Vaulted Abutments

The general configuration of the standard vaulted abutment utilizing precast, prestressed beams to support the abutment span is illustrated in Figure 3.8.14-1. This abutment is generally used when the abutment design span at right angles is greater than 21 ft. Access to the inside of the vault shall be provided for in this type of abutment.

The space provided between the curtain wall and the adjacent precast beam should be large enough to allow for inspection. The distance from the center of the curtain wall to the center of the adjacent precast beam may be as large as the center-to-center spacings of the precast beams. The curtain wall shall be designed to carry its share of vertical load and may be designed for the loadings specified for outside or exterior roadway beams.

Figures 3.8.14-2 and 3.8.14-3 provide the information necessary to determine the critical abutment dimensions.

Figure 3.8.14-4 provides details for end diaphragm reinforcement for PPC beams on vaulted abutments.

Figure 3.8.14-5 shows the general configuration and standard dimensions of the filled vaulted abutment using a reinforced concrete slab as the abutment slab. This abutment is generally used when the right angle design span is 21 ft. or less.

The vertical steel extending from the footing into the front wall of the sand filled vault shall be #7 bars at 12 in. cts. minimum. The vertical steel in the wall shall be #6 at 12 in. cts. minimum.

The main span bearing location for both filed and unfilled types of abutments depends on several beam clearance factors. See Section 2.3 for A_{bw} and A_{ba} layout equations, and contact the Department if additional guidance is required.

Vaulted abutment footings may be supported by piles or drilled shafts. See Sections 3.10.1 and 3.10.2, respectively, for more information. When the existing ground has adequate bearing capacity, the abutment may also be supported by a spread footing instead of using piles or drilled shafts. See Section 3.10.3 for guidance.

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The minimum transverse distance between the outside rows of piles in the footings of the sand filled vault shall be 6 ft. – 0 in. The footing shall be reinforced transversely top and bottom. The minimum reinforcement in the top of the footing shall be #6 bars at 12 in. cts. The bottom steel (t bars) shall be designed. The longitudinal reinforcement (w bars) in a pile supported footing shall be #5 bars at 12 in. cts. minimum placed between piles. See Section 3.10.1 for pile spacing requirements of pile supported footings. For the design of a pile supported footing in flexure and in shear, the pile load shall be distributed over a width equal to $0.8X + 3.75$ ft. but not wider than the longitudinal pile spacing, where X is the distance from the edge of the vertical wall to the center line of the pile under consideration.

Figure 3.8.14-6 presents drainage details for vaulted abutments.

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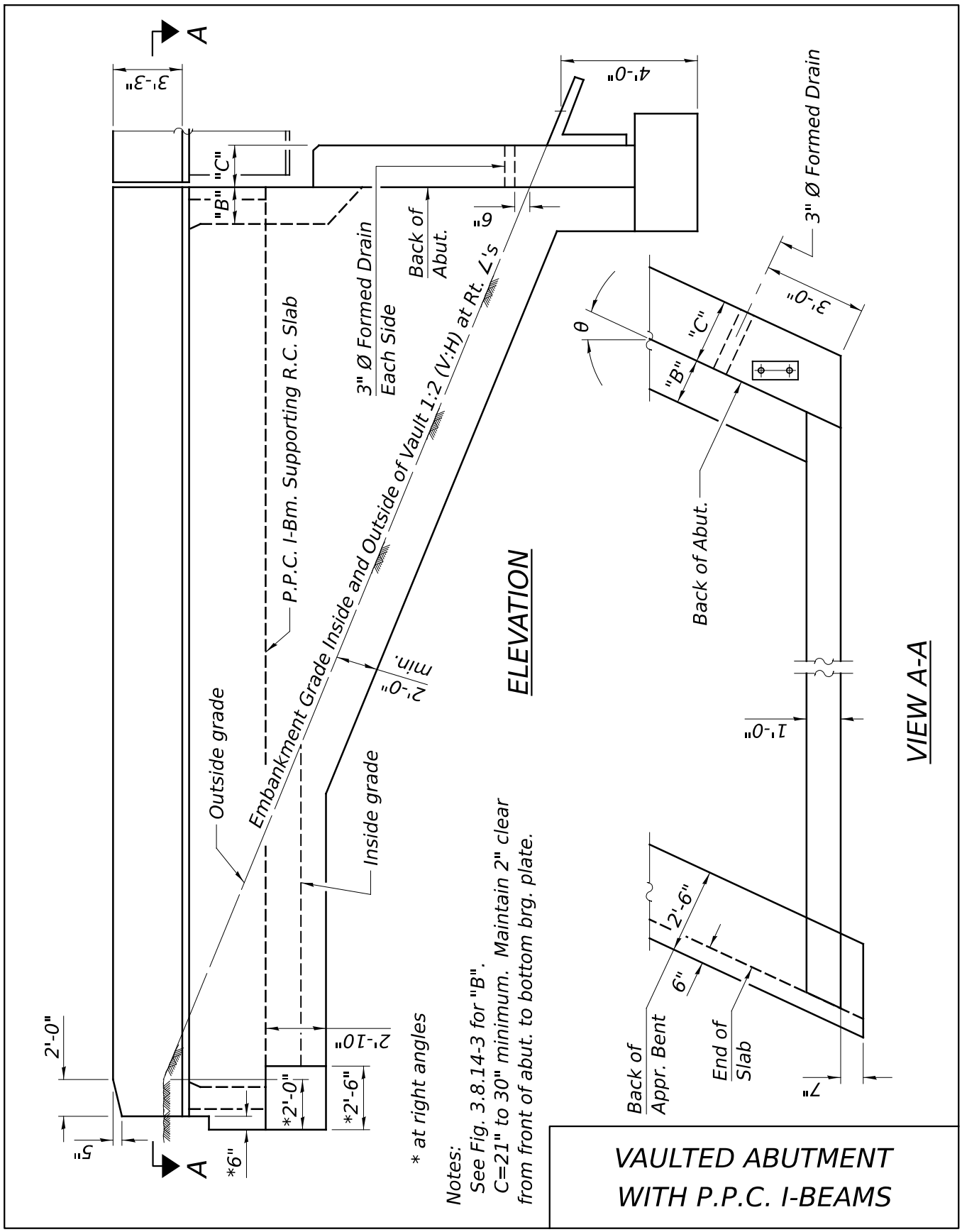


Figure 3.8.14-1

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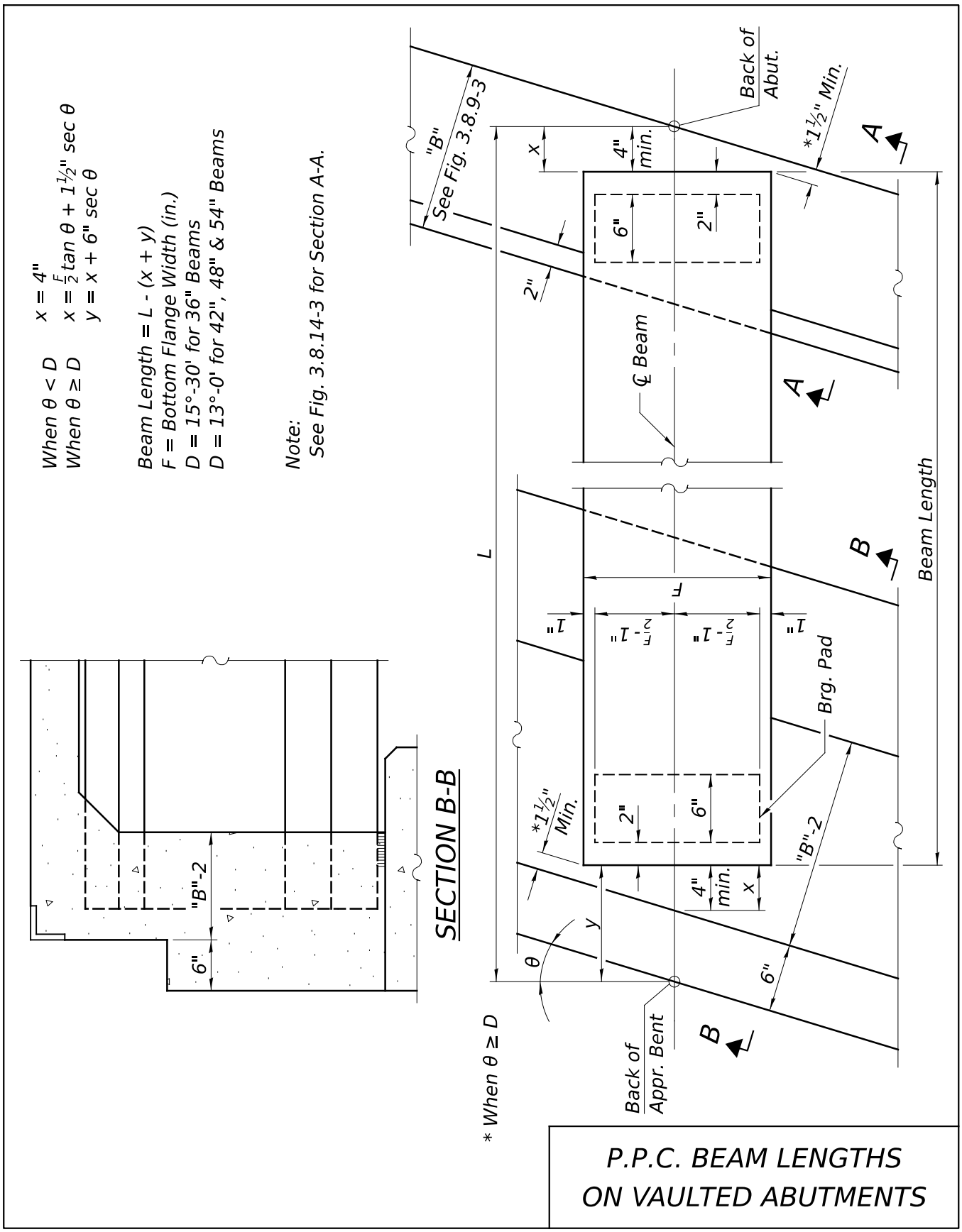


Figure 3.8.14-2

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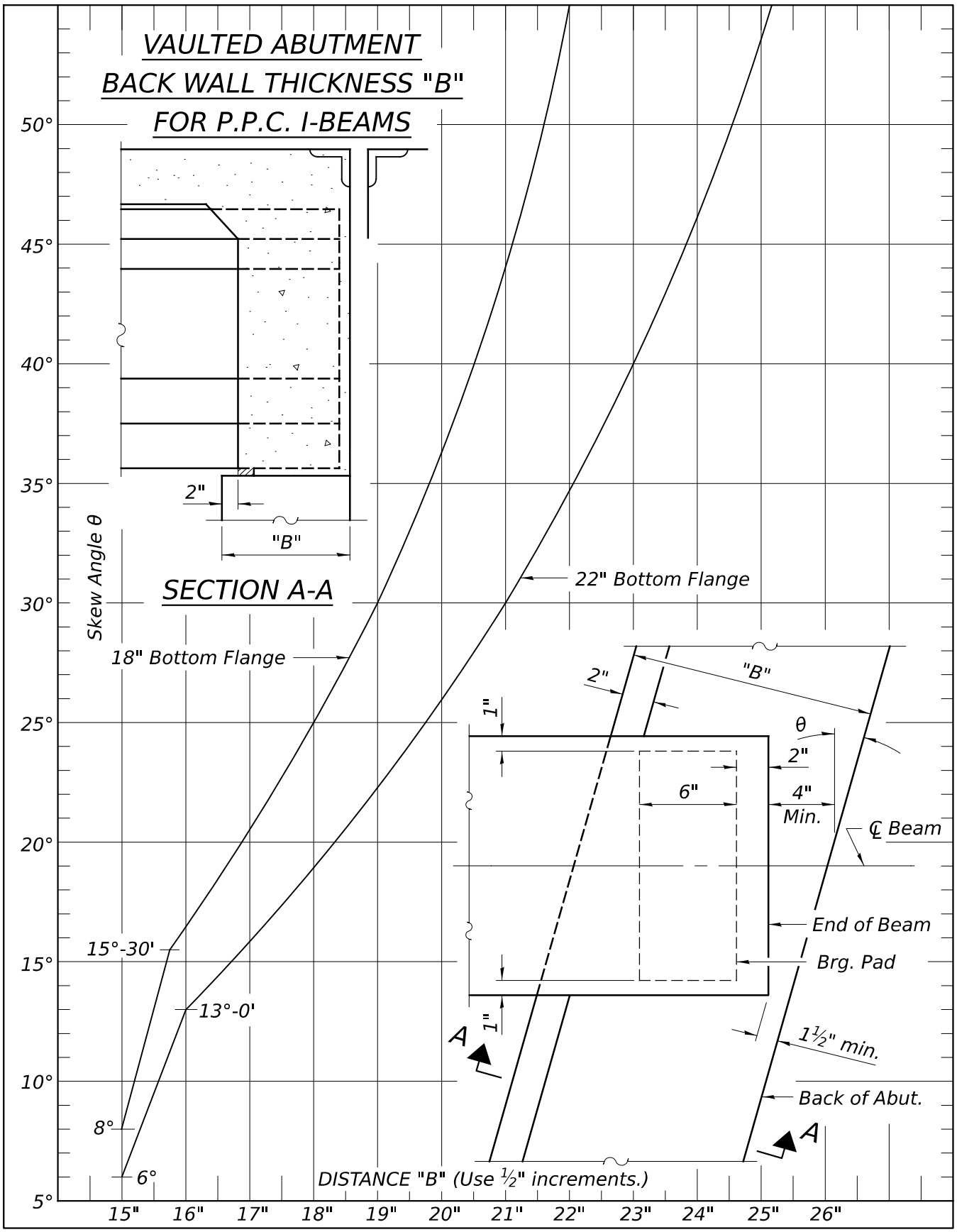
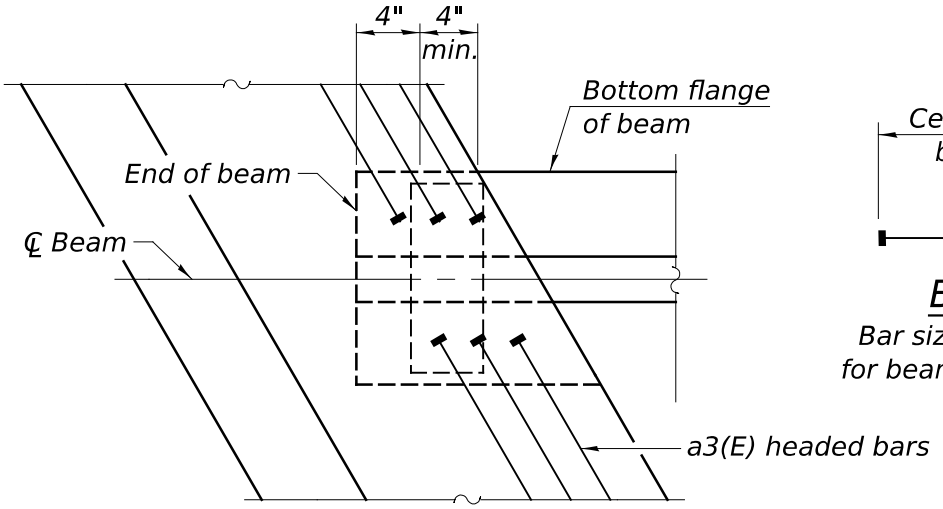


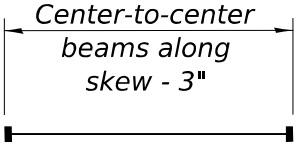
Figure 3.8.14-3

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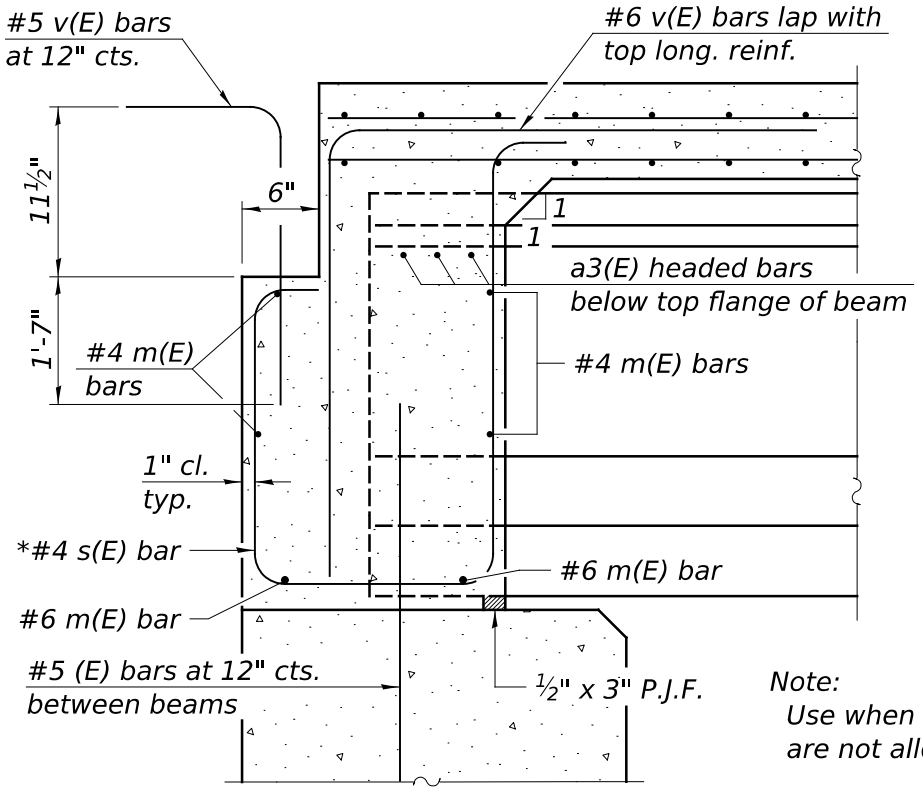
SKEWED DIAPHRAGM

(Deck and diaphragm reinforcement omitted)



BAR a3(E)

Bar size based on design for beam spacing and skew



Note:
Use when integral abutments are not allowed.

SECTION THRU ABUTMENT

** Place parallel to beams and space at right angles to beams.*

*** When total differential seat elevation is 8" to 1'-0", revise dimension accordingly to maintain required clearances.*

END DIAPHRAGM AT FIXED BEARING FOR PPC BEAMS ON VAULTED ABUTMENTS

Figure 3.8.14-4

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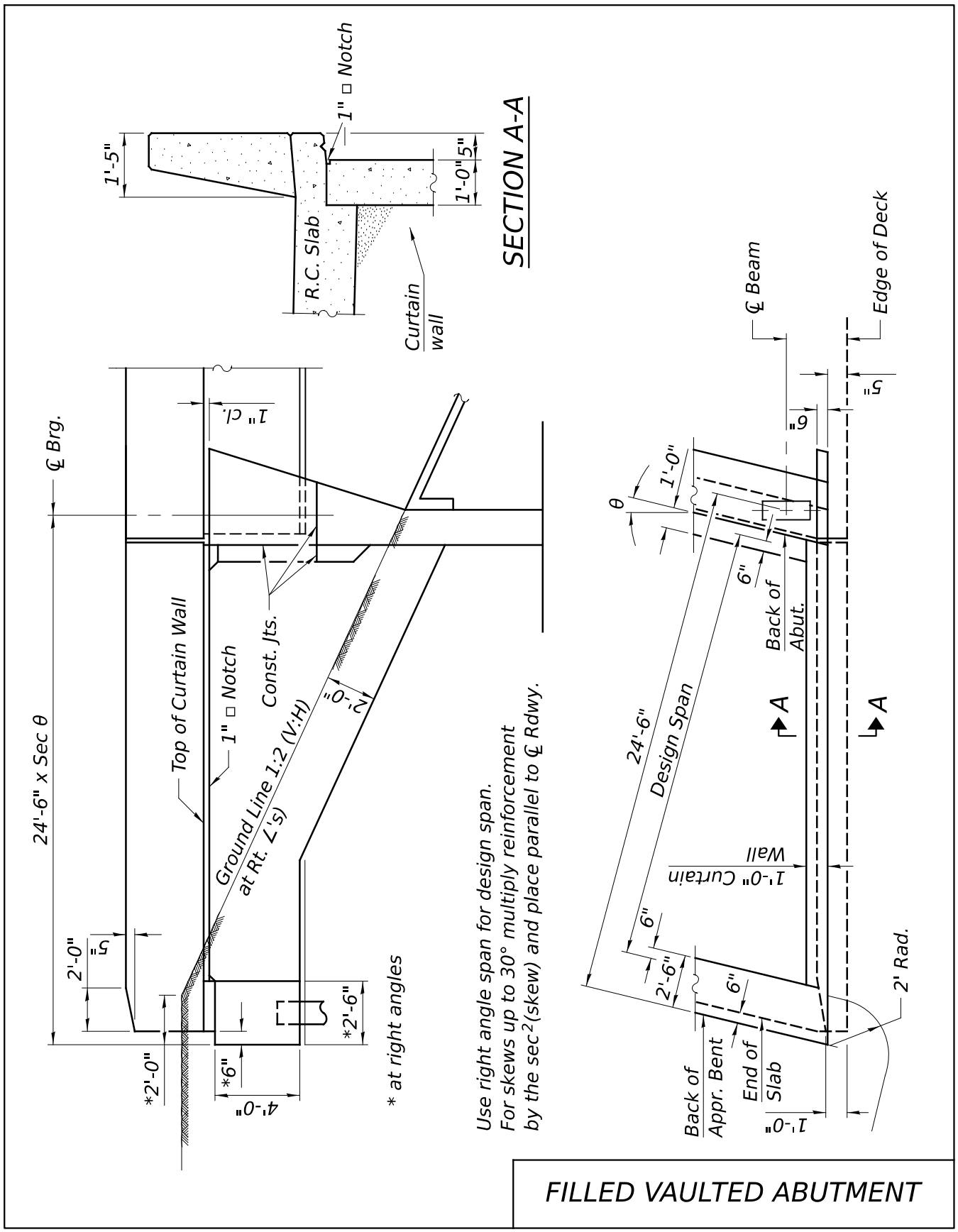


Figure 3.8.14-5

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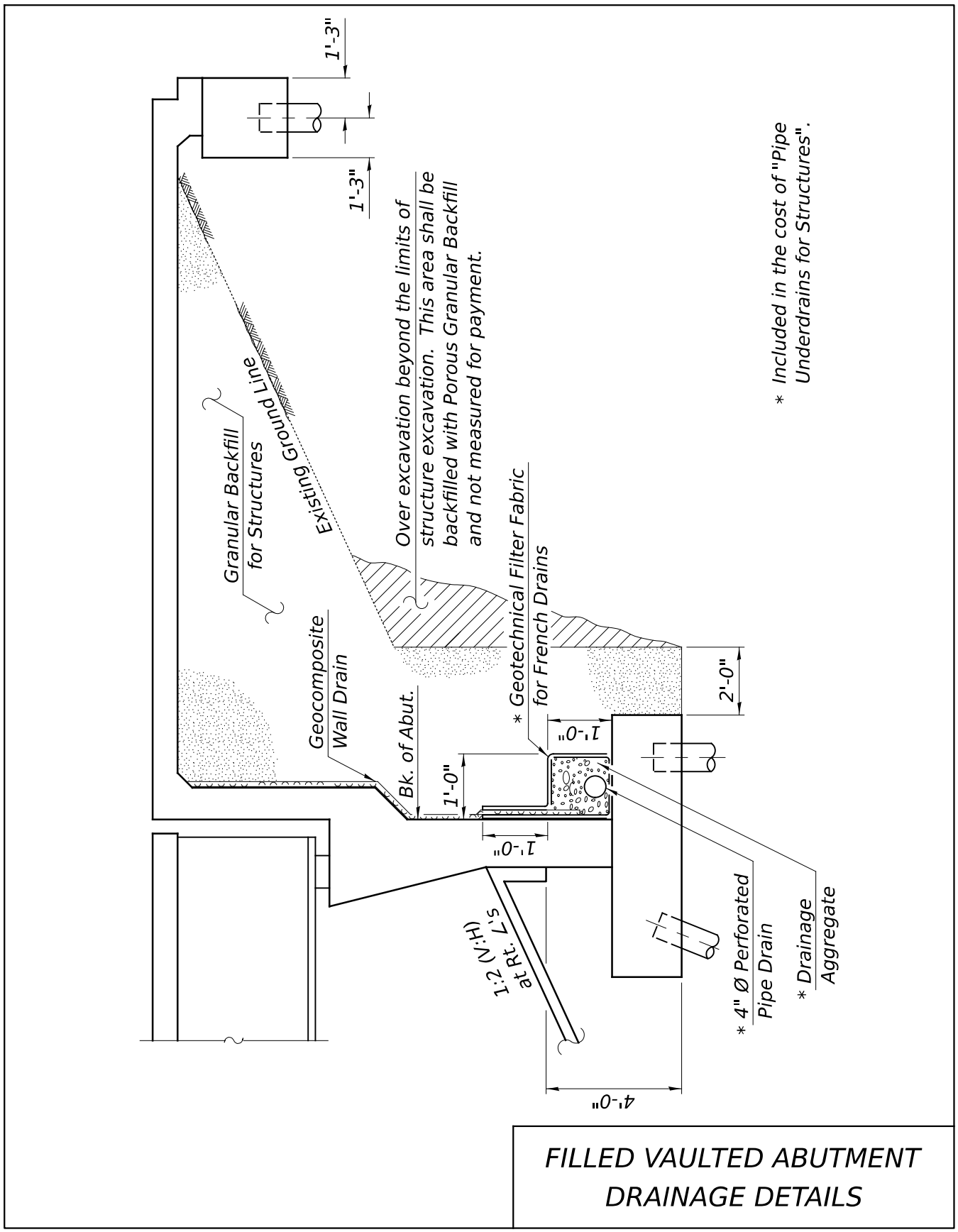


Figure 3.8.14-6

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3.9 Piers

This section covers the Department's policies related to substructure components and elements supporting the interior spans of a superstructure that transfer loads to the foundation piles, shafts, or footings. Detailing that meets the base sheet requirements and policies of the Department comprise the standard basis for design.

3.9.1 General

Three main categories of piers are built in Illinois: footing supported piers (on piles, drilled shafts, or spread footings); individually encased pile or drilled shaft bents; and solid wall encased pile or drilled shaft bents. Footing supported piers include multiple round and trapezoidal column bents with cap beams and crash walls, solid walls, and hammerheads. Pile types which can be individually encased or encased in a solid wall include H-pile and metal shell. For H-piles, the pile web shall be perpendicular to the centerline of the pier. See Figure 3.9.1-1. Precast concrete piles may also be encased in a solid wall. A variation on these pier types is individual drilled shafts with web walls. Section 2.3.6.2.2 provides guidance for pier selection and the basic geometric proportioning of piers. Appendix A1.2 provides electronic links to Base Sheets for typical pier configurations used in Illinois.

The following sections contain technical details, policies, and procedures for the design of the various pier types within each of the three primary categories described above.

Piers under deck joints or within 10 ft. of the outer edge of shoulder shall have all exposed surface areas treated with Concrete Sealer. For cases involving both existing and new concrete, such as structure widening, Concrete Sealer shall only be applied to new concrete. A film-forming Concrete Sealer shall be used for horizontal surfaces.

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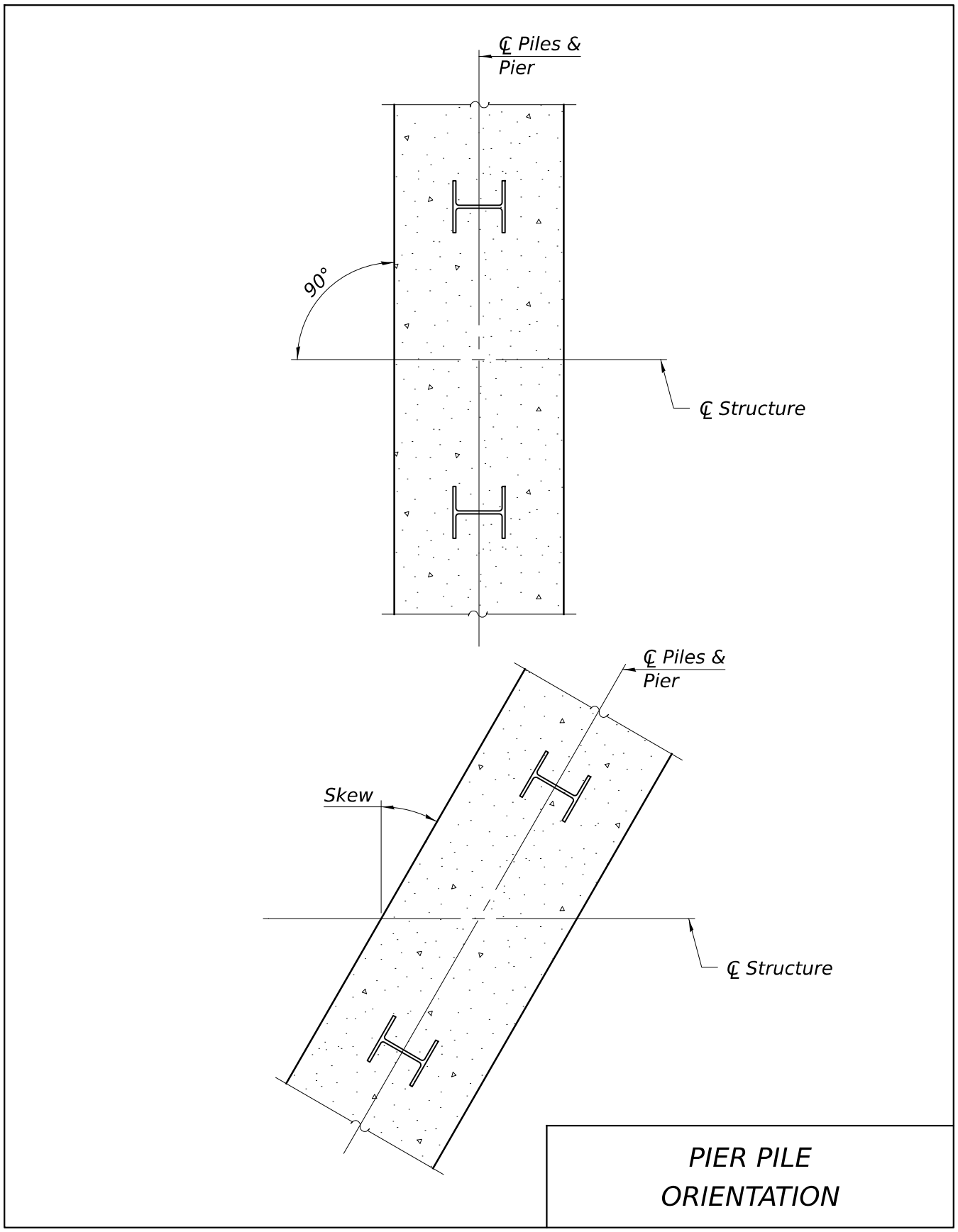


Figure 3.9.1-1

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3.9.2 Reinforcement

All reinforcement bars in pier elements of IDOT maintained structures, shall be epoxy coated unless an alternate protective system (ASTM 1035, TEC, Galvanized, etc.) is approved by the Department.

For pier caps continuously supported by solid wall encasement, distribution reinforcement in Article 5.6.7 of the AASHTO LRFD Bridge Design Specifications need not be checked because flexural tension is low, and the typical #7 p(E) bars have been sufficient from the Department's years of experience. Additional longitudinal cap reinforcement should be provided to address temperature and shrinkage requirements if the designer's calculated spacing exceeds 18".

Placement of reinforcement in pier caps shall be detailed so as not to interfere with anchor bolt locations. Multiple reinforcement layers should be considered to alleviate congestion.

3.9.3 Pier Design Loadings

Selected guidance, Departmental polices, and methods of analysis concerning the design loadings for piers are contained in this section. The AASHTO LRFD Bridge Design Specifications should also be referenced for more complete information on such loadings as vehicular live, braking, centrifugal, temperature gradient, friction, creep, wind, etc.

3.9.3.1 Dynamic Load Allowance (Impact)

The dynamic load allowance (impact) shall not be included in the design of elements below the bearing. Impact, however, shall be included in both the substructures and foundations when:

- the superstructure is poured monolithically with the substructure(s) on which it bears.
- there is a concrete diaphragm in the superstructure that sits on the substructure cap (see the base sheets of the "Diaphragms-Fixed Pier Prestressed Beams" IDOT CADD Bridge Cell Library).

3.9.3.2 Transmission of Transverse and Longitudinal Forces

Longitudinal forces transmitted from the superstructure to the substructure, shall be as specified by the AASHTO LRFD Bridge Design Specifications in magnitude but applied through the bearing. Transverse forces shall be as specified by AASHTO LRFD both in magnitude and points of application.

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3.9.3.3 Wind

Article 3.8 in the AASHTO LRFD Bridge Design Specifications contains the provisions for wind loadings on bridges. The longitudinal and transverse load transmission properties of elastomeric bearing types employed in Illinois should be considered when determining tributary areas for wind loadings on superstructures and forces in substructures. Reference Section 3.7 of this manual and AASHTO LRFD Bridge Design Specifications Article 14.6.3 for more information.

AASHTO LRFD Bridge Design Specifications Articles 3.8.1.2, 3.8.1.3 and 3.8.2 apply to pier design for most bridges built in Illinois. Wind on super and substructures is covered in Article 3.8.1.2. Article 3.8.1.3 contains the provisions for wind pressure on vehicles (wind on live load) and Article 3.8.2 details vertical wind pressure (overturning force). Note that overturning wind forces are not applied simultaneously with wind on live load in the AASHTO LRFD Bridge Design Specifications. Article 3.8.1.1 should only be used when bridges or parts of bridges are more than 30 ft. above ground or water level. Article 3.8.3 (aeroelastic instability) is not applicable for the construction of typical bridges.

3.9.3.4 Temperature, Shrinkage, Creep and Settlement

Consideration shall be given to stresses and movements resulting from variations in temperature, shrinkage and creep of concrete, and settlement. Typically, longitudinal movements are induced in piers by the superstructure expanding or contracting which can cause deflections and stresses in substructure elements. In the vertical direction; shrinkage, creep and settlement typically result in potential significant movement of piers. The Structure Geotechnical Report should provide the amount of foundation settlement if it is over ⅛ in. Insofar as shrinkage and temperature affect the design of a pier, the following criteria shall be followed:

1. The coefficient of expansion of reinforced concrete and structural steel shall be taken as 0.0000065/°F.
2. The coefficient of shrinkage for concrete shall be taken as 0.0002.

On a typical multi-column grade separation pier, the temperature differential between the cap and the crash wall may be considered negligible.

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3.9.3.5 Ice Forces

AASHTO LRFD Bridge Design Specifications Article 3.9 contains provisions for ice forces. The rivers on which ice forces shall be considered in pier designs are as follows:

1. Mississippi River
2. Illinois River
3. Rock River
4. Fox River
5. Kankakee River
6. Iroquois River

Ice forces shall also be considered in pier designs on any other river with identified ice problems.

The considered ice thickness shall be 18 in. for locations north of Peoria and 12 in. for locations south of Peoria. The height on the pier at which the ice forces are applied shall be the average of the low and high water elevations unless more precise data is available. The floating ice criteria above shall be applied to the ends of piers for each design project.

Forces on the sides of piers shall be applied only if the stream is navigable and the shore line is a bluff type. For this condition, every favorable feature such as skew, column action under an assumed condition of a receding water level, and the inability of ice to withstand tension shall be considered to provide a footing width within reasonable limits based on sound engineering judgment.

To resist any floating ice with splitting effects at those sites where heavy ice movements are known to occur or where relatively rapid stream flow exists, the upstream nose of the pier shall be beveled and reinforced with an 8 in. x 8 in. x ½ in. steel angle of ASTM A709 GR 36 or GR 50, galvanized in accordance with AASHTO M 111. The angle shall be anchored to the pier with 8 in. long stud shear connectors at 12 in. alternating centers. No encasements of piers with corrosion resistant steel plates or any similar treatment is required.

3.9.3.6 Vessel Collision Forces

AASHTO LRFD Bridge Design Specifications Article 3.14 contains provisions for vessel collision forces. Typically, piers for bridges over navigable waterways should be designed for vessel collision forces. A partial list of navigable waterways in Illinois is given in Section 2.3.9.3. If navigability or the requirement for consideration of vessel collision forces is in question, contact the BBS.

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3.9.3.7 Vehicle Collision Forces

Piers within the clear zone of interstate traffic and all piers supporting interstate traffic and within the clear zone of vehicular traffic are considered critical or essential. All other piers within the clear zone of vehicular traffic are considered typical. The clear zone is defined in Chapter 38 of the BDE Manual. All piers within the clear zone of vehicular traffic, regardless of classification, shall have a crash wall. Crash walls shall have a minimum height of 5 feet above the finished ground line at the pier. The top of the crash wall shall run continuously level.

New critical or essential piers shall be investigated for collision. Unless protected as specified in AASHTO LRFD Bridge Design Specifications Article 3.6.5.1, piers shall be designed to provide structural resistance to the vehicle collision force as detailed therein. When designing a pier for the vehicle collision force, all load and resistance factors shall be set to one (1.0). The BBS permits uplift on footings or tension in the piles if appropriate anchorage details are provided. It is seldom practical or economical to protect the pier in lieu of providing structural resistance, therefore the BBS recommends a cost-benefit analysis before implementing protection of the pier. With the approval of the BBS, piers may be considered to have structural resistance if they meet one of the listed criteria in AASHTO LRFD Bridge Design Specifications Article 3.6.5.1.

New typical piers may be exempt from providing structural resistance to vehicle collision if the annual frequency of bridge collapse (AFBC) is less than 0.001, as specified in AASHTO LRFD Bridge Design Specifications Article C3.6.5.1; otherwise, the same requirements of critical or essential piers apply.

Existing piers are exempt from the vehicle collision force provisions mentioned above; however, unprotected multi-column piers must be retrofitted with a crash wall. Guardrail, cable road guard, and impact attenuators are not considered adequate protection. To be exempt from the crash wall requirement, protection shall be as specified in AASHTO LRFD Bridge Design Specifications Article 3.6.5.1.

Piers adjacent to railways shall satisfy the crash wall height and train collision requirements as specified in the *AREMA Manual for Railway Engineering*.

Abutments are not required to be designed for vehicle collisions.

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3.9.3.8 Earthquake, Liquefaction and Scour

See Section 3.15 for information on earthquake forces. See Sections 2.3.6.3 and 3.10 for information on liquefaction and the two different scour events that are checked in pier design: the design scour and the check scour. Earthquake loading, with or without liquefaction, is considered an Extreme Event I loading, independent of Extreme Event II. Non-extreme event scour design criteria typically govern over extreme event scour design criteria at bridge piers. Where extreme event scour is a design consideration, it shall not be considered simultaneously with extreme event earthquake forces. Liquefaction (an extreme event associated with seismic loadings) considerations for structural design of piers and non-extreme event scour are also not required to be considered simultaneously. The following is a summary of the load combinations and scour depths to be used for each load combination:

Load Combination	**Earthquake**	**Liquefaction**	**Scour Event Considered**
Strength	NA	NA	Design Scour Depth
Extreme Event I	Yes	Yes	Degradation Scour Depth
Extreme Event II	NA	NA	Check Scour Depth

See Article C3.4.1 of the AASHTO LRFD Bridge Design Specifications for additional information.

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3.9.4.1 Circular and Trapezoidal Column Bents with Cap Beam and Crash wall

Reference Base Sheets (in the "Piers" IDOT CADD Bridge Cell Library) P-3 through P-7 and P-11 through P-13 for bents with trapezoidal columns. Base sheets P-24 and P-26 illustrate bents with circular columns. A pier type with drilled shaft foundations that can be considered in this pier category is shown in Base Sheet P-DSCW. It illustrates a multi-circular column bent with a crash wall.

All multi-column circular and trapezoidal piers should be designed as frames for loadings in the vertical and transverse directions. They should be treated as cantilevers for loadings in the longitudinal direction. The point of fixity for column design in the longitudinal and transverse direction should be at the top of the crash wall for grade separation piers. Caps may be designed as continuous beams which are pin supported at the columns. Simple finite element models employing only frame (beam-column) elements normally produce verifiable results using hand calculation methods. Design of multi-column bents in regions of moderate to high seismicity is addressed in Section 3.15. Analysis methods for un-cracked sections applicable to forces which are similar to seismic base shears at lower levels of acceleration (wind, braking, centrifugal) can also be found in Design Guide 3.15 on the primary Bridges & Structures page.

The minimum width of any pier cap at a grade separation shall be 2 ft. - 6 in. This minimum dimension shall be followed unless additional width is needed for the bearing seats. Extended seat widths (longitudinal and transverse) are typically a concern in regions of moderate to high seismicity. See Section 3.15. Extended seat widths also facilitate future bearing replacement.

If the length of a pier cap or crash wall is greater than 90 ft., a joint should be located between bearings near mid-length. On stage constructed pier caps and crash walls, a total width of 120 ft. or less may be permitted without requiring a joint. Joints required on pier caps shall be 1 in. open joints and joints required on crash walls shall be beveled bonded construction joints with reinforcement continuous through them.

All piers on grade separation structures shall be provided with a crash wall which extends 5 ft. minimum above finished ground. When a guardrail is to be installed running around the face of the pier, the ground elevation should be computed at the face of the guardrail. The top of the crash wall shall run continuously level. See Section 2.3.6.2.2.

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3.9.4.2 Solid Wall and Hammerhead Bents

Solid piers are shown on Base Sheets P-1, PB-1 or PC-1, in the "Piers" IDOT CADD Bridge Cell Library. It is preferable that walls remain straight. However, if required by design, walls may be battered as shown in Base Sheet PB-1. Batter may be required as a wall becomes very tall. The minimum width at the top of a solid pier shall be 2 ft. – 0 in. If the bearing seat requirements are such that more than 2 ft. – 0 in. in width is needed, consideration should be given to the use of a hammerhead grade separation pier as shown on Base Sheet P-2 or a modified hammerhead pier such as shown on Base Sheet P-10. The ends of pier stems shall be rounded when located in a stream.

Solid wall piers should be treated as cantilevers in the longitudinal direction. In the transverse direction, they should be treated as very stiff shear walls. It is permissible, however, to design solid wall piers as a single column if the height-to-width ratio of the column exceeds 2.5.

Hammerhead and modified hammerhead piers may be designed in a similar manner to solid wall piers. A hammerhead, as contrasted with the modified hammerhead, however, should be considered more like a column than a wall. The overhanging cantilevered portion in the transverse direction is also a primary structural design consideration for this pier type. Vertical open joints, when approved for wide piers, shall terminate above the footing.

3.9.4.3 Pier Footings

See Sections 3.10 and 2.3.6.3 for foundation aspects of footings and further information on requirements for pile supported, drilled shaft and spread footings. The longitudinal reinforcement (w bars) in a pile supported footing shall be #5's at 12 in. cts. minimum placed between piles. The transverse reinforcement (t bars) shall be designed.

To design a pile supported footing for flexure and shear, see the requirements of Article 5.12.8 of the AASHTO LRFD Bridge Design Specifications.

The vertical reinforcement extending from the footing into the wall for a fixed pier shall be designed to resist the moment created by the horizontal forces transmitted from the superstructure to the substructure as well as any horizontal forces which may act directly on the substructure. These forces shall be oriented to produce the maximum overturning at the pier. Note that special detailing is required for bridges located in regions of moderate to high seismicity, see Section 3.15.

Any construction joints in pier footings shall be bonded with continuous reinforcement.

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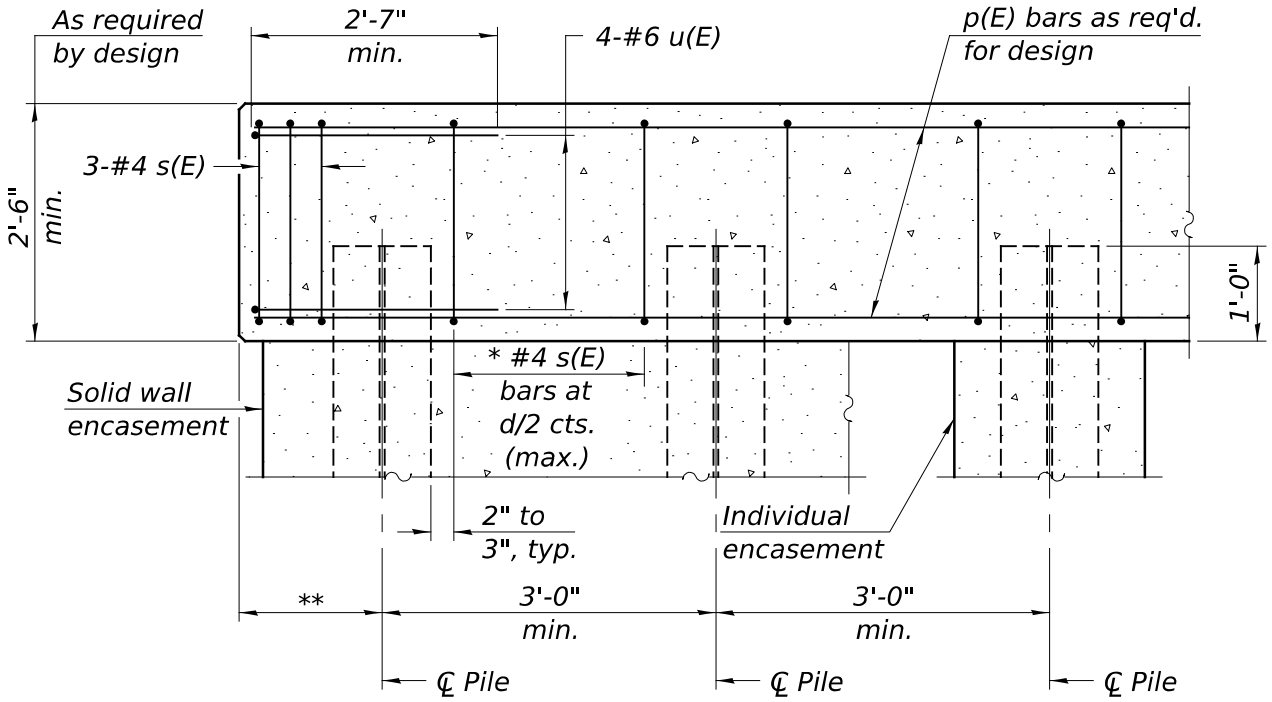
3.9.5 Individually Encased Pile Bent Piers and Column Drilled Shaft Bent Piers

Individual encasement details for H-piles can be found on Base Sheet F-HP and for metal shell piles they can be found on Base Sheet F-MS. For individual column drilled shaft piers (without web walls, encasement walls, etc.) refer to Base Sheet P-DS. Pile and drilled shaft base sheets may be found in the "Piles" and "Pier" IDOT CADD Bridge Cell Library, respectively.

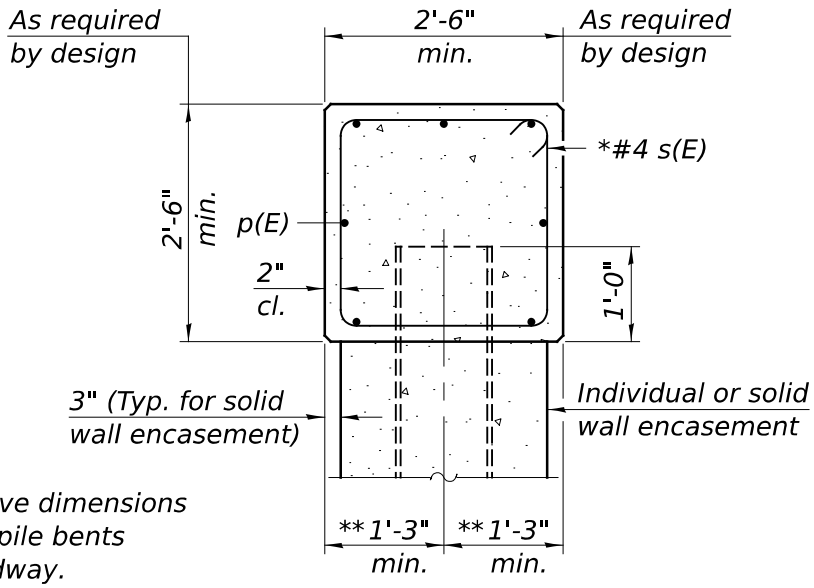
Individual pile encasements are not considered structural for design purposes. For longitudinal loadings, the piles and drilled shafts should be designed as cantilevers. In the transverse and vertical loading directions, the pier may either be modeled as a collection of individual cantilevers, pin supported just below the cap beam, or as a frame. Generally, cantilever type response of this pier type increases as the height of the piles or shafts above ground increases or the depth-of-fixity below ground becomes lower. See Section 3.10 for geotechnical aspects of design for pier types in this category. Figure 3.9.5-1 details a reinforced concrete cap for pile bents; Figure 3.9.5-2 shows the same pier but with alternate reveal and monolithic construction options. If each bearing is located above a pile (or drilled shaft), this cap may be used without designing the p(E) bars, but the minimum p(E) bar shall be a #7 bar. When the bearings are located other than directly above the piles (or drilled shafts), the p(E) bars may be designed assuming the cap is a continuous beam supported by the piles (or drilled shafts) only. Simple finite element models employing only frame (beam-column) elements normally produce results which can be verified using hand calculation methods. When required, development lengths and bar splice information is presented in Appendix A3.4.

A typical detail for a fixed slab bridge connection at a pier is shown in Figure 3.9.5-3.

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PARTIAL ELEVATION



Note:
In all cases give dimensions from ℄ to ℄ of pile bents along ℄ of roadway.

SECTION THRU INTERIOR BENT

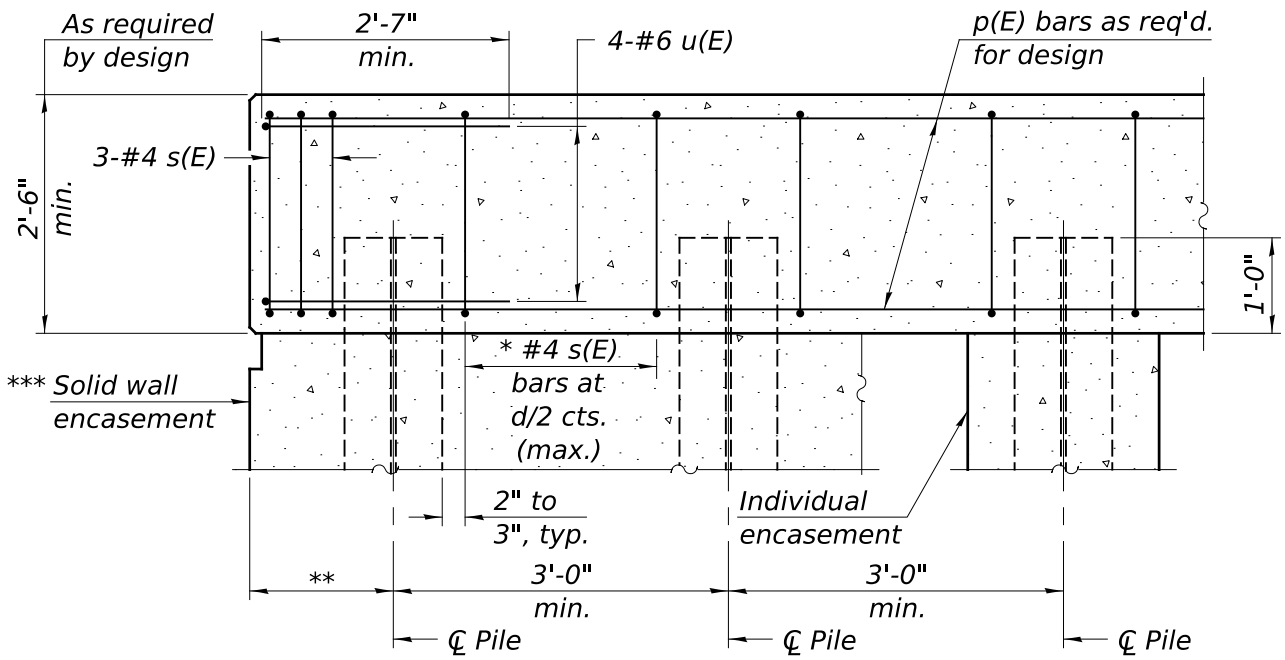
* Space s(E) bars to satisfy minimum shear requirements. d = effective depth

** Dimension also to satisfy minimum of ½ width of pile + 9"

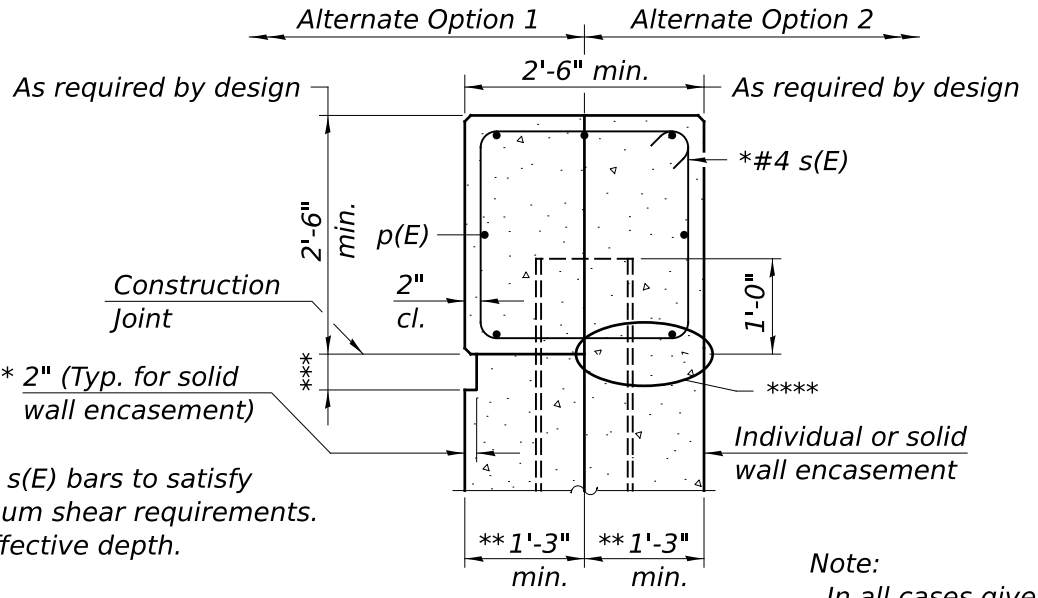
REINFORCED CONCRETE CAPS FOR INDIVIDUAL OR SOLID WALL ENCASED PILE BENT PIER

Figure 3.9.5-1

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PARTIAL ELEVATION



SECTION THRU INTERIOR BENT

* Space s(E) bars to satisfy minimum shear requirements. d = effective depth.

** Dimension also to satisfy minimum of ½ width of pile + 9".

*** Alternate Option 1: For solid wall encasements with construction joints, 2" wide x 4" tall reveal, each face.

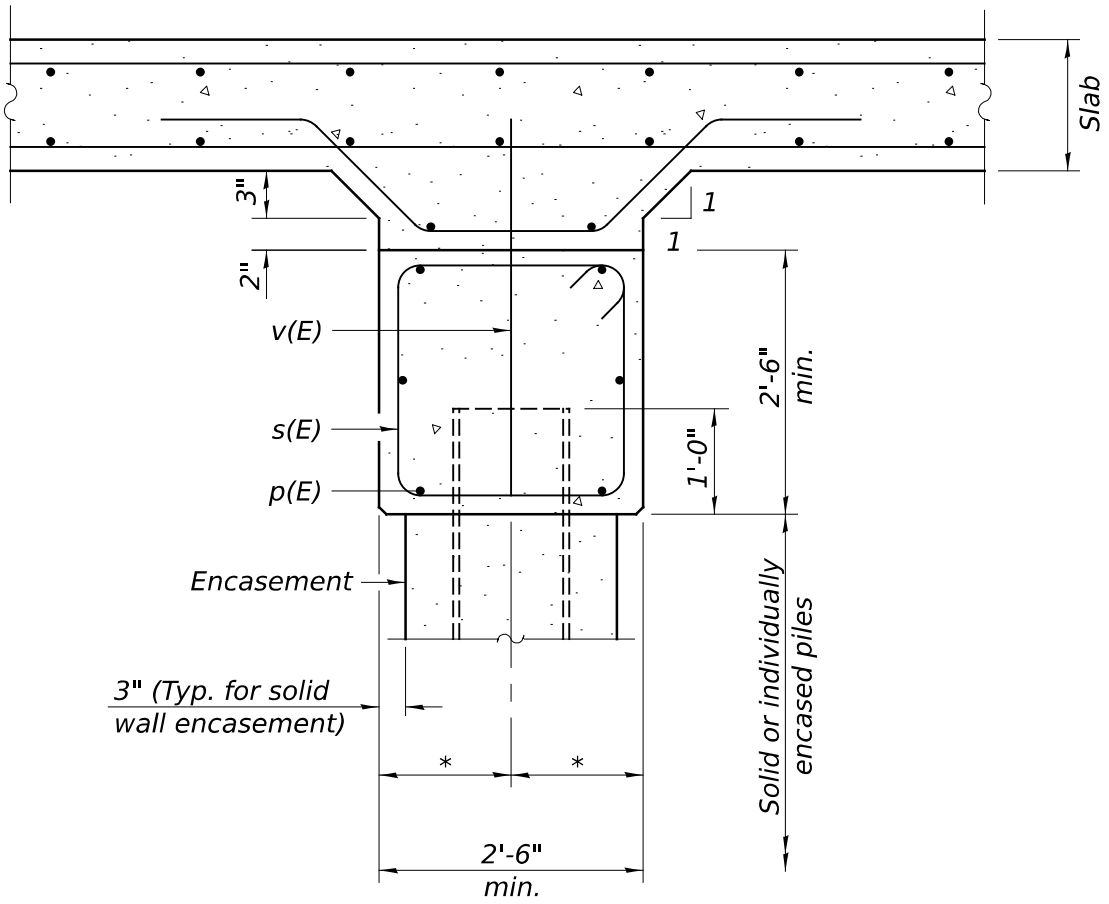
**** Alternate Option 2: For solid wall encasements without construction joints, pour wall flush with concrete cap, each face.

Note:
In all cases give dimensions from ℄ to ℄ of pile bents along ℄ of roadway.

ALTERNATE REINFORCED CONCRETE CAPS FOR INDIVIDUAL OR SOLID WALL ENCASED PILE BENT PIER

Figure 3.9.5-2

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Note:
The pier seat cross slope should match the slab deck cross slope.

** Dimension also to satisfy minimum of ½ width of pile + 9".*

PILE BENT PIER CONNECTION FOR SLAB BRIDGES

Figure 3.9.5-3

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3.9.6 Solid Wall Encased Pile and Drilled Shaft Bent Piers

Base Sheet P-DSSW, in the "Piers" IDOT CADD Bridge Cell Library, illustrates a solid wall encased drilled shaft bent pier. Solid wall encased metal shell, precast and H-pile bents are similar.

Figure 3.9.6-1 illustrates the standard IDOT reinforcement for a solid wall encased pier with H-piles, metal shell piles, or precast piles. These details shall be used for all pile bent piers with these types of piles regardless of seismic region.

Solid wall encasements are not considered structural but their weight and stiffness can influence how piers in this category should be designed. For longitudinal loadings, the piles and drilled shafts should be designed as cantilevers just as for individually encased pile bents or drilled shafts, but the wall portion should be considered essentially rigid. In the transverse and vertical loading directions, the wall should also be considered rigid with the piles or shafts below ground likely to exhibit frame action like response as opposed to cantilever behavior. Cap beams for this pier type may be designed in an analogous manner to individually encased pile or drilled shafts bents.

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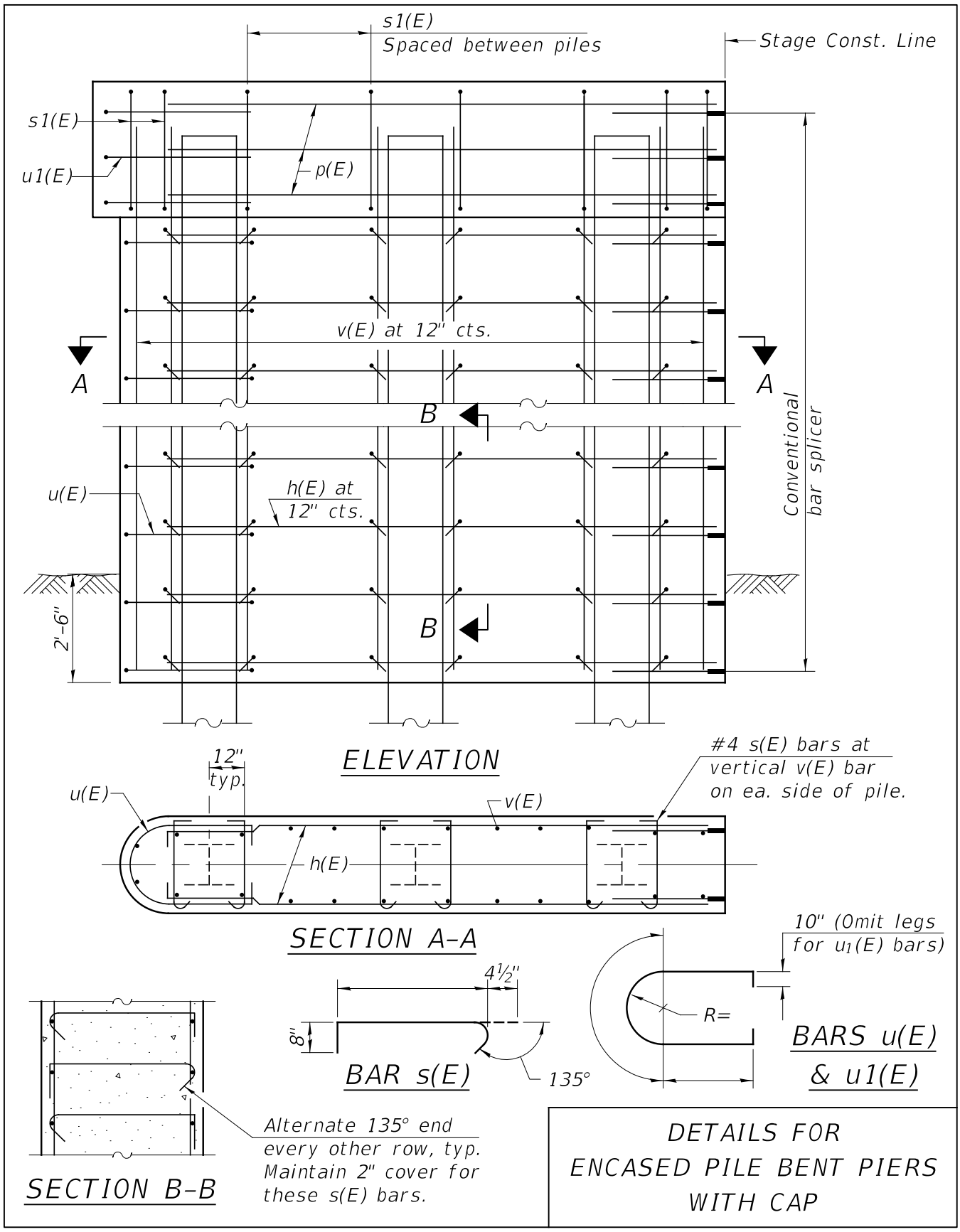


Figure 3.9.6-1

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3.9.7 Other Drilled Shaft Pier Types

3.9.7.1 Column-Web Wall Drilled Shaft Bent Pier

Base Sheet P-DSWW, in the "Piers" IDOT CADD Bridge Cell Library, illustrates a drilled shaft pier with web walls. Structurally, this pier type should be designed to behave as a hybrid of an individual drilled shaft pier and a solid wall encased drilled shaft pier. Hand analysis methods may be used during design, but verification with simple finite element models is recommended.

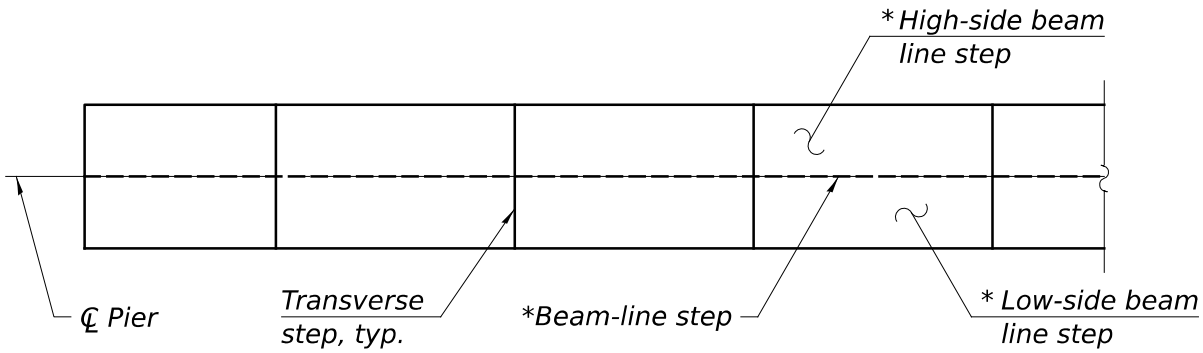
3.9.7.2 Transfer Beam Drilled Shaft Bent Pier

Base Sheet P-DSTB illustrates a drilled shaft pier with a transfer beam. The design and analysis of this pier type can be thought of as somewhat analogous to that of a two story building with bridge loadings. Vessel collisions on the transfer beam may require a dynamic analysis more complex than that for seismic loadings.

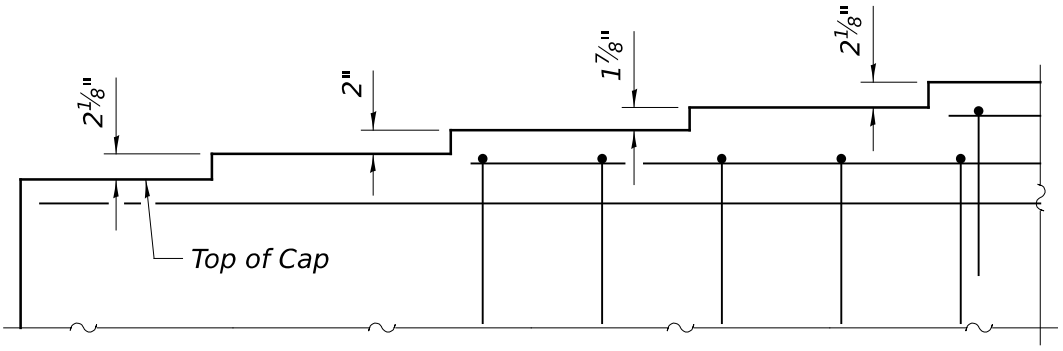
3.9.8 Bridge Seats

The bridge seats shall be constructed in steps poured monolithically with the pier cap. The minimum step shall be ¾ in. Provide steel fill plates at bearings if steps are less than ¾ in. The elevation of each seat shall be shown on the plans. Steps 4 in. or larger shall be reinforced (see Figure 3.9.8-1). All steps shall be perpendicular to the face of the pier cap (with a possible exception at stage construction joints). Figure 3.9.8-1 presents a simple sketch of typical bridge seats. See Section 3.15 for minimum required seat widths for seismic design considerations.

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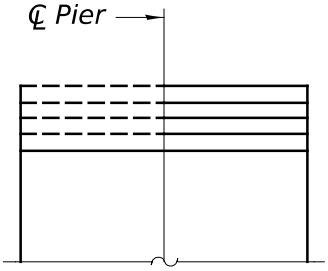


PARTIAL PLAN

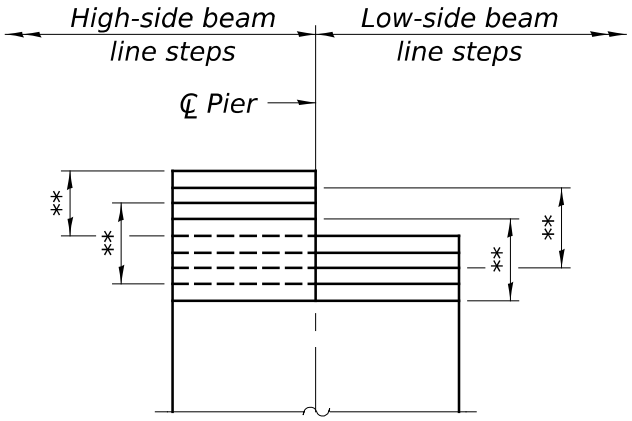


ELEVATION

Note:
Dashed seat lines indicate the following:
1 - *Final finish / strike-off (when no beam line steps)*
2 - *Bonded construction joint (for beam line steps)*



TRANSVERSE STEP (FINAL LAYOUT) OR BEAM-LINE STEP (PREPARATION)



BEAM-LINE STEPS (FINAL LAYOUT)

END VIEW

* *When required for design.*

** *6" min*

BRIDGE PIER SEATS AND STEPS

Figure 3.9.8-1

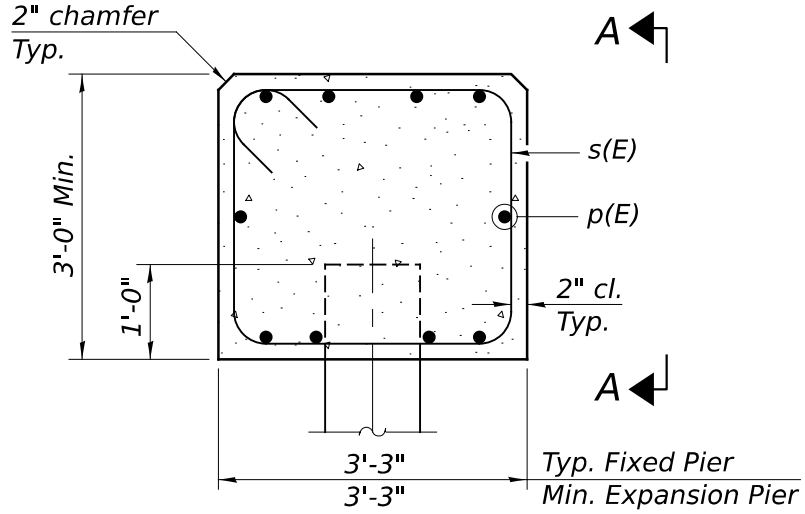
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3.9.9 Piers for Deck Beams

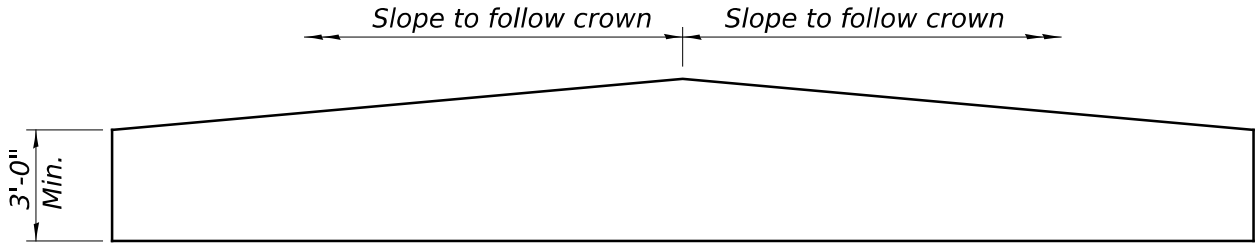
Pier details for deck beam structures are shown in Figure 3.9.9-1. The general geometry is depicted for all depths of deck beams. A reinforcement scheme is also presented however the size of the reinforcement bars shall be determined by design.

Several factors shall be considered when determining the overall pier cap length. The total width of the deck shall be the theoretical width plus ½ inch per deck beam joint. The cap shall ideally be an additional 6 inches wider from all points of the deck on each end and 9 inches wider from all points on the deck on each end when a side retainer is required at an expansion joint. See Section 3.5 for details of the side retainer. These measures account for fabrication and construction tolerances to ensure adequate pier lengths. Rounded pier ends, when required, shall also be added to the pier length described above. All of the aforementioned dimensions are at right angles to the centerline of the roadway and do not account for the additional cap length associated with skew.

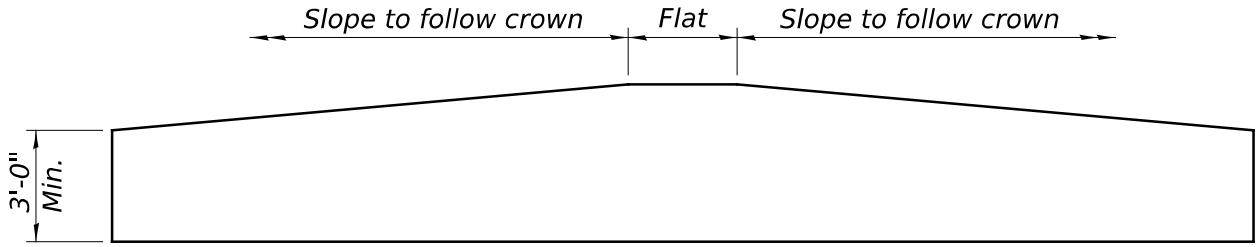
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SECTION THRU PIER
(Dimensions are at Rt. ∠'s)



VIEW A-A
(For even number of beams)



VIEW A-A
(For odd number of beams)

PIER DETAILS FOR
11"-42" DECK BEAMS

Figure 3.9.9-1

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3.10 Foundations

This section covers the Department's policies related to the design and detailing of piles, shafts, or footings to transfer AASHTO LRFD Bridge Design Specifications loads to soil or rock. Detailing that meets the subsurface investigation requirements and policies of the Department comprise the standard basis for design.

3.10.1 Piles

3.10.1.1 Introduction

Typically, piles are specified at the TSL phase when the soil conditions are not sufficient to support a spread footing and drilled shafts are found to be either too expensive or incompatible for a specific structure.

The Department generally utilizes four primary types of piles as foundation support for its structures. These are Steel, Metal Shell, Precast Concrete, and Timber. Details of piles, splices, encasement, pile shoes and other pertinent items for the first three listed pile types are given in Base Sheets F-HP, F-MS and F-PC, in the "Piles" IDOT CADD Bridge Cell Library.

An overview of pile design procedures and other pertinent design guidelines are given in following sections. AGMU Memo 10.2, found in the Design section on the primary Bridges & Structures Page, provides detailed information on the recommended pile design sequence and procedure. It clarifies the role of the geotechnical engineer, the development of the pile design table, and how it should be used by the structural engineer to complete the pile foundation design. The design guide and following sections use new terminology such as nominal required bearing, factored or allowable resistance available, geotechnical losses, and separate geotechnical and structural resistance factors to help assure proper design and construction of piles in Illinois.

References which should also be consulted include the IDOT Geotechnical Manual, found in the Foundations and Geotechnical section on the primary Bridges & Structures Page, and the AASHTO LRFD Bridge Design Specifications.

3.10.1.2 Nominal Required Bearing

The nominal required bearing of a pile is the geotechnical resistance it will develop during driving and is the sum of the nominal tip and side resistances the soil or rock provides to the pile. These values are

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not modified by an LRFD resistance factor, ϕ, and do not account for any geotechnical losses which decrease the geotechnical resistance during the life of the structure. Several nominal required bearing values are provided in the Structure Geotechnical Report (SGR) for various pile types and sizes determined to be appropriate for the subsurface conditions. A complete description of the geotechnical analyses required to use the equation is given in AGMU Memo 10.2.

3.10.1.2.1 Maximum Nominal Required Bearing

There is a maximum nominal required bearing, $R_{N\ Max}$, which can be specified for each of the standard pile types. $R_{N\ Max}$ is limited by the following empirical relationships which were developed to provide reasonable confidence that the dynamic stresses in piles during driving will not cause pile damage when driven in appropriate subsurface conditions.

Metal Shell $R_{N\ Max} = 0.85\ F_y\ A_s$

Where:

0.85	=	empirical dynamic driving coefficient
F_y	=	yield strength of shell (50 ksi)
A_s	=	area of shell

Steel $R_{N\ Max} = 0.54\ F_y\ A_s$

Where:

0.54	=	empirical dynamic driving coefficient
F_y	=	yield strength of steel pile (50 ksi)
A_s	=	area of steel pile

Precast $R_{N\ Max} = 0.3\ f'_c\ A_g$

Where:

0.3	=	empirical dynamic driving coefficient
f'_c	=	compressive strength of concrete (4.5 ksi precast, 5 ksi prestressed)
A_g	=	area of precast pile (14 in. x 14 in.)

Timber $R_{N\ Max} = 0.5\ F_{co}\ A_p$

Where:

0.5	=	empirical dynamic driving coefficient
F_{co}	=	base resistance of wood parallel to grain (2.7 ksi)
A_p	=	average area of pile (113 $in.^2$ assumed)

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Table 3.10.1.2.1-1 provides the maximum nominal required bearings for each pile type using the stresses and relationships provided above.

Pile Designation	Maximum Nominal Required Bearing (Kips)
Metal Shell 12”ϕ w/0.25” walls	392
Metal Shell 14”ϕ w/0.25” walls	459
Metal Shell 14”ϕ w/0.312” walls	570
Metal Shell 16”ϕ w/0.312” walls	654
Metal Shell 16”ϕ w/0.375” walls	782
Steel HP 8x36	286
Steel HP 10x42	335
Steel HP 10x57	454
Steel HP 12x53	418
Steel HP 12x63	497
Steel HP 12x74	589
Steel HP 12x84	664
Steel HP 14x73	578
Steel HP 14x89	705
Steel HP 14x102	810
Steel HP 14x117	929
Steel HP 16x121	967
Steel HP 16x141	1,126
Steel HP 16x162	1,288
Steel HP 16x183	1,461
Steel HP 18x135	1,077
Steel HP 18x157	1,247
Steel HP 18x181	1,436
Precast 14”x14”	265
Precast Prestressed14”x14”	294
Timber Pile	153

Table 3.10.1.2.1-1

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Note that these maxima don't apply to piles set in rock, as piles set in rock are not subject to drive stresses. The maximum nominal value for piles set in rock is equal to the yield strength of the pile times the cross-sectional area.

When determining the final H-pile size to be specified, normally the lowest weight section necessary which provides the factored or allowable geotechnical and structural resistance required should be selected. However, pile sections such as the HP 8 x 36, 10 x 57, 12 x 74, 12 x 84, 14 x 102, and 14 x 117 are rolled less frequently than other piling and thus can cause construction delays on some projects. Due to the limited supply of the above sections, designers are encouraged to utilize HP 10 x 42, 12 x 53, 12 x 63, 14 x 73 and 14 x 89 sections, when possible, to reduce pile acquisition time as well as eventually decrease cost as demand is increased on these sections. Similarly, designers may be required to use the larger HP 16 and HP 18 piles when cost-effective compared to more piles of smaller sizes, and supply/schedule/drivability of these piles may vary; contact the Bureau of Bridges & Structures for information about utilization of these larger piles.

3.10.1.3 Factored Resistance Available

The factored resistance available, R_F, represents the net long term geotechnical capacity available at the top of a pile which resists LRFD factored axial loadings. It reduces the nominal required bearing computed by applying the geotechnical resistance factor and accounts for any geotechnical losses such as downdrag, scour, and liquefaction. The LRFD equation for factored resistance available is given by:

$$R_F = \varphi_G R_N - \varphi_G \lambda_G \left(DD + Scour + Liq.\right) - \gamma_p DD$$

Where:

R_F	=	factored resistance available
R_N	=	nominal required bearing
DD	=	reduction from downdrag
Scour	=	reduction from scour
Liq.	=	reduction from liquefaction
ϕ_G	=	geotechnical resistance factor
λ_G	=	bias factor between WSDOT formula and IDOT estimated pile length equations (1.05)
γ_p	=	load factor for DD loading applied to pile

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See AGMU Memo 10.2 for more complete descriptions of the above equations for the LRFD case. IDOT uses a geotechnical resistance factor of 0.55. Extensive studies undertaken by the Department have shown that this value for resistance better represents the accuracy of the WSDOT formula for soils in Illinois.

The design resistance factor may be increased (or FS decreased) to produce shorter, more economical pile foundations when a load test is specified to be part of the construction control. The added cost of conducting a load test is significant and only justified when the calculated foundation savings is larger than the expense of a load test. When the number of piles or total lineal footage of piling on a project is significant, the designer should calculate the cost difference to determine if a load test is justified and obtain approval from the BBS Foundations and Geotechnical Unit.

When piles are to be driven in areas of moderate to high seismicity, the equations above for factored resistance available are identical except that ϕ_G for LRFD is taken as 1.0 when considering earthquake loadings. In addition, the Scour term should be set to zero and the Liq. term should only be used when calculating the factored or allowable resistance available (R_F or R_A) for earthquake loadings (an extreme event in LRFD). For most cases, initial SGR pile design tables should provide both non-seismic (or non-extreme event) factored or allowable resistance available and seismic resistance available (LRFD or ASD). If the seismic resistance available is not provided, the geotechnical engineer may need to be contacted to recalculate the pile design table based upon seismic considerations. See Section 3.15 and AGMU Memo 10.2 for further information and guidance on seismic considerations for pile design.

3.10.1.4 Structural Resistance

In many cases, the factored geotechnical resistance (factored resistance available, R_F) will control or is less than the factored structural resistance of the piles. However, in some cases, such as for seismic loadings, piles set in rock, and scour conditions, the factored structural resistance of the piles should be checked to assure adequate structural integrity. Seismic loadings and scour conditions typically involve combined lateral and axial loadings, and/or unbraced “column” lengths for piles. If the factored structural resistance ($R_{NS} \times \phi_S$) is less than the factored loads (Q_F), either the pile size or the number of piles should be increased such that $R_{NS} \times \phi_S \geq Q_F$.

1. If the pile size is increased, both the nominal required bearing (R_N) and the factored resistance available (R_F) shown on the plans should not change, since the Q_F demand per pile would not change.

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2. If the number of piles is increased, both the nominal required bearing (R_N) and the factored resistance available (R_F) shown on the plans should be decreased, since the Q_F demand per pile would decrease.

Both approaches, either an increase in pile size or an increase in the number of piles, should result in $R_{NS} \times \phi_S \approx Q_F$, $R_F \approx Q_F$ and $R_{N\ MAX} > R_N$. However, if the distance to rock is less than 20 feet, it is recommended that the piles be driven to their maximum nominal required bearing ($R_{N\ MAX}$) since the corresponding increase in geotechnical resistance can normally be obtained with minimal additional penetration/cost and may also be needed for future bridge rehabilitations. The modified load-resistance relationships would then be $R_{NS} \times \phi_S \approx Q_F$, $R_F > Q_F$ and $R_{N\ MAX} = R_N$.

It should also be noted that the factored loadings (Q_F) are at the top of piling and shall not include any factored downdrag (DD) which may be present. The LRFD strength load groups specify that the portion of DD which applies a loading to the pile be included with loadings from other applicable sources. However, it is IDOT's standard practice to require that the DD loading and DD reduction in resistance for a pile (as well as other reductions in resistance such as scour and liquefaction) be taken into account by the geotechnical engineer and incorporated in the SGR pile design table. Consequently, it shall not be added to the LRFD group loadings as suggested.

The minimum factored loading applied (Q_F) should normally be greater than zero (in compression). In cases where this cannot be accomplished using an economical pile layout (e.g. when seismic loadings are significant), the factored resistance available (R_F) in pullout should be calculated. This calculation will provide the minimum tip elevation to be included on the plans to ensure pullout resistance. See also Section 3.15.

When piles are to be driven in areas of moderate to high seismicity, the ϕ_S factors for LRFD are generally taken as 1.0 (H-piles and metal shells). The potential exposed lengths of piles due to liquefaction should be considered when investigating the structural capacity of piles under seismic loadings.

The scour case considered in Section 3.10.1.3 is typically not what would be considered an extreme event (see Section 3.10.1.5.1) and thus would only be considered under non-seismic load groups.

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3.10.1.5 Geotechnical losses (Scour, Downdrag and Liquefaction)

3.10.1.5.1 Scour

Design scour elevations shown on the TSL plan within the Design Scour Table are the basis for scour design at piers and abutments. When scour elevations provided on the TSL plan extend below pile encasements or bottoms of footings, the piles shall be designed, both geotechnically and structurally, to withstand the applied loadings without the presence of the foundation soils above the given elevations. Refer to Section 2.3.6.3.2 for additional background on determination of design scour elevations.

The Hydraulic Report is the basis of the theoretical scour prediction and normally considers the cumulative effects of long term aggradation/degradation, general contraction scour and local pier scour. The potential for lateral channel movement that could impact scour conditions at a pier or abutment is also assessed within the Hydraulic Report. Since the Hydraulic Report is often completed at an early stage of project development, the designer should verify that final pier features (location, width, skew) and the waterway opening configuration agree with that utilized in the Hydraulic Report. The scour prediction methods are empirical and mainly based on laboratory research in sand and the local scour equations are for live-bed scour in cohesionless sand-bed streams. Only limited soil data (D_{50} soil particle size) is required to be estimated (both upstream of the bridge and at the foundation) which does not significantly influence the final scour values. Furthermore, the scour equations do not account for the increased resistance to scour which exists in some cohesive soils and rock. Consequently, it is very possible that the calculated values may generate overly conservative scour depths in some cases. As directed in Section 2.3.6.3.2, the geotechnical engineer is charged with reducing predicted scour depths generated in the Hydraulic Report at bridge locations with non-granular streambeds. Based upon actual site subsurface conditions, this reduction or adjustment of predicted scour performed by the geotechnical engineer and contained within the SGR then becomes the basis of the design scour elevations shown in a table by the planner on the TSL plan. Since the pile design table values provided in the SGR are based on the scour elevations provided in the Hydraulics Report and any adjustments recommended in the SGR, further modifications in the planning or design phase requires coordination with the geotechnical engineer to provide revisions to the pile design table.

The 100 yr. flood event is used to calculate a pile's factored resistance available using the resistance factors provided above. The 200 yr. flood event should also be checked to assure that the applied loadings are less than the nominal resistance for extreme event loads per the AASHTO LRFD Bridge Design Specifications.

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The nominal value of Scour used in the factored and allowable resistance equations in Section 3.10.1.3 above is calculated by using the IDOT side resistance equations provided in AGMU Memo 10.2. This will reduce the resistance available to the designer below the nominal driven bearing by the portion of resistance provided by the layers against the pile expected to scour.

Section 10 of the IDOT Drainage Manual, found in the Hydraulics section on the primary Bridges & Structures Page, covers the issue of scour in detail and Section 11 provides discussion on scour countermeasures.

3.10.1.5.2 Downdrag

Downdrag (also known as negative skin friction) occurs when soil against a pile moves downward more than 0.4 in. after driving. This movement is most often caused by settlement of the foundation soils due to applied embankment loading, but may also occur due to other losses of embankment support such as liquefaction.

Downdrag is most often addressed by reducing the resistance available to the designer which, in effect, increases the pile size, the pile length or the number of piles such that they carry both structure and soil loadings. Although the term downdrag refers to the total loss of resistance available to support the structure loading at the top of the pile, it involves two distinct actions that are accounted for separately. In the factored or allowable resistance equations in Section 3.10.1.3 above, the nominal value of DD is used twice, once to reflect the loss of nominal driven bearing and again to account for the subsequent applied soil loading. Note that the nominal value of DD is only the resistance lost or the load applied (since they are of equal magnitude). DD may be computed using the IDOT side resistance equations provided in AGMU Memo 10.2 along the length of the pile applying load to the pile.

When the loss of net resistance from downdrag results in excessive added pile size, length, or number, the designer should evaluate other treatments to reduce or eliminate downdrag.

Precoring the pile locations may be specified and can be considered for design purposes to eliminate downdrag. This is most often a cost-effective solution when much of the downdrag loading is applied within 25 ft. of the foundation excavation and is caused by cohesive soils. Drilling to deeper depths in more problematic soils requires specialized equipment and shaft excavation support which becomes excessively expensive and time consuming. The piles can be driven to nominal required bearing (without resistance from the precored layers) and, since they are backfilled with loose dry sand, have almost no applied loading due to the subsequent embankment settlement. The plans shall specify the diameter of pre-coring (usually 18 in.) and the elevation that the precoring shall extend to. Precoring partial depth

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can decrease downdrag (but not eliminate it due to consolidation and settlement of layers below the precore elevations), which may be useful in reducing the cost of additional pile size, length or number. The SGR will provide the structure designer with recommendations regarding the diameter and elevation of precoring. The pile design table will be developed using these recommendations and, as such, it is critical that the Contract plans reflect this information provided in the SGR.

On some projects, it may be possible to schedule the embankment placement in an earlier contract to ensure that settlement will have finished prior to pile driving. Even when this is not possible, a waiting period (based on settlement amount or time) may be specified to ensure that the contractor cannot drive piles until most of the settlement has occurred. In either case, no downdrag need be included in the pile design since the settlement will be less than 0.4 in.

Applying bitumen coatings to the piles can be used to reduce downdrag. However, unless the pile is driven prior to embankment placement, it is difficult to ensure that the proper part of the pile is coated such that in the final driven condition the coated part is in the soil layers which are anticipated to cause downdrag.

3.10.1.5.3 Liquefaction

When the SGR indicates that a layer or layers of soil are expected to liquefy under the design level seismic event, the pile design table in the SGR will provide both the factored resistance available and the liquefaction reduced seismic resistance available during the seismic event.

When liquefaction occurs adjacent to a pile, the side resistance present (and reflected in the nominal driven bearing) is reduced to zero in those layers. As pore pressure in the layer is reduced to pre-static levels, a densification of the sand particles occurs resulting in a reduction in layer thickness and settlement of all layers above. When the settlement is larger than 0.4 in., it will also produce downdrag in the non-liquefied layers.

The nominal value of Liq. used in the factored or allowable resistance equations in Section 3.10.1.3 above would be equal to the loss of nominal driven bearing in the liquefying layers calculated using the IDOT side resistance equations provided in AGMU Memo 10.2. If non-liquefiable soils are present above liquefiable layers, downdrag is produced in the non-liquefied layers and the nominal value of DD shall be computed and used in both locations in the resistance equations to account for the loss of resistance DD and the applied loading DD.

See the IDOT Geotechnical Manual for detailed guidance on liquefaction analysis.

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3.10.1.6 Estimated Pile Lengths

The SGR provides pile design tables which contain a series of nominal required bearing values, R_N, the corresponding factored resistances available, and the estimated pile lengths. The structure designer shall select the estimated pile length from the appropriate pile design table such that the corresponding factored or allowable resistance available is greater than or equal to the factored loading applied. Contact the geotechnical engineer if the factored loading(s) are outside the pile design table.

AGMU Memo 10.2 provides the nominal side resistance and end bearing equations which are recommended for making pile length estimates. They have been used and improved over many years.

For most cases, the pile design tables provided in the SGR will include seismic considerations directly. Section 3.15 and AGMU Memo 10.2 provide guidance and methods for using the pile design tables presented in the SGR when seismic considerations are not included in the formulation.

Note that small H-piles should be avoided when estimated piles lengths are long due to a tendency for the pile to wander. As a general rule, the upper limit for HP 8 sections is 50 ft. total length, the limit for HP 10 sections is 75 ft. and the limit for HP 12 sections is 100 ft.

When H-piles are driven to their maximum nominal required bearing and the majority of resistance is developed in rock, the estimated length of the pile shall include the amount of penetration into rock. The penetration into rock is related to the amount of resistance required as well as the type and strength of the rock. Assuming minimal thickness of rock surface weathering, penetrations may be expected to be between 2.5 to 10 ft. in shale, 1.5 to 6 ft. in sandstone, and 0.5 to 3 ft. in limestone.

3.10.1.7 Test Piles

Test piles are specified to provide site specific pile bearing vs. length data which is used by the Department during construction to supplement the estimated plan length and boring information. With these data, a final itemized list of furnished pile lengths is determined for the contractor to supply. When no test piles are specified on a project, the list of furnished lengths is taken as the estimated plan length.

The need to utilize test piles is evaluated by the geotechnical engineer and provided to the structure designer as part of the SGR pile design recommendations. Several factors are considered when determining if test piles are required and which substructures should contain test piles. Consistency of both the subsurface soils/rock conditions and the nominal required bearing at adjacent substructures can provide the confidence necessary to reduce the number of test piles necessary since one test pile can

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be used to represent multiple locations. Existing pile driving records may also give additional information regarding how the piles will drive, and help determine the number and location of any test piles as well as "fine tune" the estimated lengths. When the confidence level in the required length is sufficient to reduce or eliminate test piles, the estimated pile lengths are normally slightly longer to assure sufficient length. However, the cost for this additional length should normally be less than the savings produced by avoiding the test pile expense.

H-piles not reaching rock are typically the most difficult to predict, followed by metal shell piles, with H-piles driven to rock having the least variation between estimated and driven lengths. When H-piles are estimated not to extend to rock, at least one test pile should be specified on a project and often one test pile will be shown at each substructure. Use of metal shell piles almost always requires at least one test pile and, depending on the consistency in nominal required bearing and subsurface soil strengths between substructures, may need test piles at every other substructure unit as a minimum. For either pile type, specifying at least one test pile allows for the possibility of adding test piles when necessary during construction at the established contract price. H-piles which are primarily end bearing may not require the use of a test pile if the rock surface is well defined, consistent between borings, and the borings are located near the substructures. However, where large differences in rock elevations are indicated and the penetration into the rock surface is not predictable, multiple test piles may be appropriate.

3.10.1.8 Pile Shoes

Pile shoes, as shown on Base Sheet F-HP and Base Sheet F-MS (in the "Piles" IDOT CADD Bridge Cell Library), are recommended when "hard driving" conditions are expected that would risk pile damage. In the case of H-piles, pile shoes should be used when piles are being driven to their maximum nominal required bearing and develop the majority of their capacity in hard rock (such as dolomite or limestone). Pile shoes may also be specified when piles are driven into rock at a steep batter or vertically into a sloping rock surface when there is concern for the piles to either bend a flange or not bite into the rock surface. Both metal shells and H-piles may require the use of shoes when there is evidence of dense layers, or either natural or man made obstructions which could cause damage to the pile. Metal shells are generally not as robust in negotiating difficult driving conditions as H-piles. However, H-piles do not allow inspection after driving to verify no damage as metal shells do. If damage is indicated in a metal shell, there are some repair options which can be employed.

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3.10.1.9 Corrosion Protection of Piles

In most cases, corrosion of piles is not a concern since they are embedded deeply below ground where insufficient oxygen exists to allow this process to develop. However, if piles are exposed or near the surface where aggressive soil conditions potentially exist, corrosion may result in substantial decreases in pile sections. For these cases, consideration shall be given for corrosion protection. Piles may be encased in concrete, oversized to allow sacrificial steel loss; and, in unique cases, utilize a paint system, be covered with a coal tar epoxy coating, or be galvanized to address corrosion.

The most common corrosion protection measure is the use of concrete encasement. It is used to encase the piles below a substructure to ensure any unprotected steel is at sufficient depth to limit the extent of corrosion. Piles supporting abutments placed in new fill are particularly susceptible to corrosion since, over time, embankment settlements expose the very tops of the piles. In addition, abutments often only have 2 ft. of riprap cover on one side and a pipe under drain system on the other so it is likely that the piles will become exposed to both moisture and oxygen which results in corrosion. Consequently, when pile supported abutments are located in areas with less than 4 ft. of soil cover and/or piles are subject to air and water exposure, concrete encasement shall be specified. The exception to this is integral abutments, in which the pile encasements are omitted in order to increase abutment flexibility and better accommodate thermal displacements. In the case of metal shells at abutments, a reinforcement cage is placed in the top 7 ft. of the piles instead of a concrete encasement. Base Sheet F-MS and Base Sheet F-HP show details of pile encasement at abutments and piers as well as metal shell pile reinforcement at abutments (which is equivalent to encasement). Since these Base Sheets are generic, use of concrete encasement shall be indicated on the plans at the required foundation location(s).

In some cases, concrete encasement is difficult to construct and/or an increase in pile section is determined to be cost effective. In these situations, pile steel thicknesses may be increased to provide sacrificial section areas as a means to address some of the expected corrosion.

Paint systems are often used when piles are proposed to be exposed as part of a substructure.

3.10.1.10 Design for Lateral Loading

Lateral loadings applied to pile foundations are typically resisted by battering selected piles, relying on the soil/structure interaction (also known as pile flexure or fixity), or a combination of these two methods.

Use of battered piles is the most common way to obtain lateral resistance with minimal lateral deflection. Batters up to 3 in. per ft. are typical and, in some cases, 4 in. per ft. has been used. In seismic regions,

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the use of battered piles is not normally desirable as they produce relatively stiff foundations (as compared to vertical piles) which perform in a less ductile manner during an earthquake. In some cases, site conditions will not permit the use of or limits the amount of batter. Regardless of the amount of batter specified, there is almost always some remaining lateral loading that the piles are required to resist in flexure (fixity). When the remaining factored lateral loading applied does not exceed 3 kips per pile, it may be assumed that the piles can provide that level of resistance without a more detailed analysis.

When the factored lateral loading per pile exceeds 3 kips, a more detailed soil structure interaction analysis shall be performed. This analysis shall determine actual pile moment and deflection such that the designer can evaluate pile adequacy. It also indicates the minimum embedment required to develop fixity and resist the applied lateral loads. In cases where the nominal required bearing can be obtained at depths less than this embedment, the minimum tip elevation should be specified in the pile data on the Contract plans. The SGR should provide some assistance in identifying the foundation soil's capacity to resist lateral pile loadings.

When piles are not expected to achieve adequate embedment to resist anticipated lateral loadings or to develop fixity, the piles may be specified to be set in rock. Although this is normally discussed in the SGR and may be specified on the TSL, it may only be identified during the final design process since necessary embedment is a function of lateral loading demands determined at this time. When piles are set in rock; the estimated top of rock, final diameter and socket depth are specified on the Final plans, and are determined with procedures which are similar to those used for drilled shafts. Note that the maximum nominal bearing may exceed Table 3.10.1.2.1-1 since the piles will not be driven. The diameter of the socket should be 18 inches if an HP8 or HP10 is used. The diameter of a socket should be 24 inches if an HP12 or HP14 is used.

Base Sheets F-HP and F-MS include splice details for H-piles and metal shell piles which develop the full moment capacity of pile sections in order to ensure the ability to carry design lateral loadings.

3.10.1.11 Pile Spacing

The designer shall make every effort to select a pile spacing at each substructure unit such that the ratio of the largest Nominal Required Bearing specified on a bridge to the smallest Nominal Required Bearing specified on a bridge not exceed 1.5. Keeping the Nominal Required Bearings at the various foundation elements within this range will help avoid the added expense of mobilizing more than one size pile hammer.

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The minimum pile spacing shall be 3 times the pile diameter. In cases where H-piles are used and the majority of pile resistance is in end bearing, a smaller spacing may be approved after discussions with the Bureau of Bridges & Structures. The maximum pile spacing shall be limited to 3.5 times the effective footing thickness plus 1 ft. - 0 in., not to exceed 8 ft. - 0 in. (integral abutments need not be limited to this 8 ft. - 0 in. spacing).

3.10.1.12 Pile Connection to Abutments, Piers and Footings

Piles supporting footings or non-integral abutments shall extend 12 in. into the structure. Integral abutment piling shall extend 2 ft. into the abutment to assure pile fixity. Reinforcement shall be placed to maintain 3 in. clearance from the bottom of the footing and arranged in such a manner to allow the pile to project into the footing or abutment. Pile bent piers shall have their piling extend through the individual or solid wall encasement 12 in. into the pier cap. Where piles are required to resist large lateral loadings by soil/structure interaction (also known as pile flexure or pile fixity), the piles may be embedded 2 ft. into the abutment or footing to ensure top of pile fixity and reduce deflection.

Overturning moments from seismic loads and uplift (tension) can create the need for a stronger lateral and/or axial connection between the piles and footing. See Section 3.15.

3.10.1.13 Pile Data Plan Information

Normally, all piles in a foundation unit are of the same size and are driven to the same nominal required bearing. The pile data to be included in the Contract plans shall include: 1. the pile type and size, 2. the nominal required bearing, 3. the factored or allowable resistance available, 4. the estimated pile length, 5. the number of production piles, and 6. the number of test piles. In some cases, other information shall be provided and is discussed.

1. The **Pile Type and Size** shall be provided so the contractor can bid and furnish the piles required at each foundation location. Examples of typical pile type and size callouts are as follows:

   Metal Shell –___ in. dia. x ____ in. walls ***with pile shoes***
   Steel – HP ___x ___ ***with pile shoes***
   Precast Concrete – 14 in. square ***prestressed***
   Timber – 12 in. dia. ***treated***

   Note the items in bold are examples of parameters which may or may not be specified for a project.

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2. The **Nominal Required Bearing** is provided in kips to instruct the contractor as to the driven bearing the production piles shall be installed to as well as assist the contractor in selecting a proper hammer size. When piles are set in rock, the Nominal Required Bearing shall be shown as “Set in Rock” on the plans.

3. The **Factored Resistance Available** shall be provided in kips. This value is not used by the contractor but documents the net long term axial factored or allowable pile capacity available at the top of the pile for the current and future design/rehabilitation work. It documents any reductions in geotechnical resistance that will occur after driving such as scour, downdrag, or liquefaction. It also reflects the resistance or safety factor which documents the accuracy in the method of construction control used at the time of installation.

4. The **Estimated Pile Length** is provided to give contractors a bid quantity, helps determine the length of the test pile, and when no test pile is specified, this length becomes the length furnished by the contractor. It also is used as a reference by the inspectors to identify when pile problems, such as lack of set up or improper hammer performance, are causing piles to stop short or run long. In some cases, a minimum tip elevation will be specified in addition to the estimated pile length. Normally, the minimum tip elevation will only be necessary when the piles have the potential to stop shorter than estimated resulting in inadequate lateral load strength or penetration below any geotechnical losses such as scour.

5. The **Number of Production Piles** is the total number of production piles required at the substructure or foundation covered by the pile data. When test piles are specified, the number of production piles shall be decreased by the number of test piles since they will be driven in production locations.

6. The **Number of Test Piles** shall always be stated. When no test piles are required, the designer shall specify zero test piles to document that the estimated pile length was made with sufficient confidence or added length.

7. When piles are set in rock, the **Estimated Top of Rock Elevation**, **Rock Socket Depth**, and **Rock Socket Diameter** shall be shown in the pile data on the plans.

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3.10.2 Drilled Shafts

In recent years, drilled shaft foundations have become more common on IDOT projects. They can be specified when spread footings are not feasible due to insufficient soil strength, or their advantages over pile foundations outweigh the disadvantages. Among the advantages of drilled shaft foundations over piles is their ability to address vertical and lateral capacity concerns resulting from large scour depths, potential liquefaction, low soil strengths and inadequate pile embedment. This section presents an overview and discussion of design considerations and Departmental policies for the design of drilled shaft foundations. The topics discussed include: geotechnical axial resistance, geotechnical lateral resistance, geotechnical losses, deflections, structural resistance and structural detailing.

To develop an optimal drilled shaft foundation configuration, several trial analyses and designs are normally required such that both geotechnical resistance and structural adequacy are addressed. Factored loadings shall be less than the factored resistances as required by the AASHTO LRFD Bridge Design Specifications. Additional technical guidance on drilled shaft design is available in the publication FHWA-IF-99-025 “Drilled Shafts: Construction Procedures and Design Methods.”

3.10.2.1 Geotechnical Axial Resistance

The approved SGR should provide the designer with the necessary guidance to determine the proper side resistance and/or end bearing values for drilled shafts at a planned project site such that the number, diameter, and embedment depths for the shafts can be selected for trial and final designs. If this information is not provided, the geotechnical engineer responsible for the SGR should be contacted to verify that the appropriate parameters and soil assumptions are being used for design.

Shafts founded entirely in soil shall be designed to utilize both side resistance and end bearing. Shafts extending into rock shall, in most cases, be designed utilizing only end bearing or side resistance in rock, whichever is larger, and neglect the overburden side resistance in soil. In some cases, the quality of the rock may be so poor or the unconfined compressive strength so low, the shafts shall be designed as an “intermediate geo-material” or even an equivalent soil mass.

Bells or enlarged bases for shafts on bridge projects are permitted in limited circumstances by the Department. The soil shall be a cohesive type and the angle of inclination of the bell from vertical shall be no greater than 30 degrees. In addition, the enlarged base of the shaft is only allowed to be considered 100% effective if the bell is dry, cleaned and inspected.

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As with piles, the weight of a shaft need not be considered an applied loading or a reduction in net geotechnical resistance available to support loadings applied to the top of a shaft.

The design resistance factor may be increased to produce shorter, more economical drilled shaft foundations when a load test is specified to be part of the construction control. The added cost of conducting a load test is significant and only justified when the calculated foundation savings is larger than the expense of a load test. When the number of shafts or total volume of shafts on a project is significant, the designer should calculate the cost difference to determine if a load test is justified and obtain approval from the BBS Foundations and Geotechnical Unit.

Shafts in rock do not require a service vertical deflection check and, in most cases, shafts founded in soils will only require a vertical deflection check when settlement tolerance is uncommonly tight, when the soils are not particularly stiff/dense, or when the applied loadings are uniquely high.

3.10.2.2 Geotechnical Losses (Scour, Downdrag and Liquefaction)

Accounting for geotechnical losses on drilled shafts is more easily accomplished than with pile foundations. The loss of side resistance from scour or liquefaction is taken account of by simply neglecting those layers when determining the required embedment to provide the necessary axial resistance. The side resistance of layers producing downdrag is subtracted from the side resistance of the remaining shaft. Note that for LRFD design, the load factor is 1.25 for downdrag on drilled shafts. Refer to Section 3.10.1.5 for more a detailed discussion of geotechnical losses.

3.10.2.3 Lateral Resistance

The SGR should provide guidance on the lateral load carrying capacity of drilled shaft foundations and, if not, the geotechnical engineer should be contacted to verify that the appropriate parameters and soil assumptions are being used in the design. Software (such as L-Pile and COM624) is required on most projects to accurately determine the required embedment depth to resist lateral loadings and the actual maximum moment as well as to calculate the anticipated shaft deflection. These programs account for the soil-structure interaction by computing the lateral soil resistance mobilized by the deflected shape of the drilled shaft. If the shaft deflection is excessive or if the embedment is inadequate to provide “fixity”, the shaft embedment can be increased to help address these issues. If deflection or rotation is still unacceptable, or if the maximum moment is more than the shaft can be designed to resist, the shaft diameter should be increased, or in some cases, more shafts can be added.

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3.10.2.4 Structural Resistance

Using the drilled shaft configuration which satisfies geotechnical axial and lateral resistance requirements, the shafts shall then be checked to verify that they can be designed with reasonable amounts of reinforcement steel. If the resulting reinforcement cage comprised of vertical bars and spirals is spaced too tightly, inadequate flow of concrete may result in structural defects. This circumstance would require another iteration of geotechnical axial and lateral analyses using larger diameter shafts or an increased number of shafts.

Drilled shafts are normally designed as columns with spiral reinforcement set at 6 in. centers to promote concrete flow which is critical to assure structural integrity considering challenging installation conditions and difficulties associated with inspection. In seismic areas, portions of the shaft above the ground and just below the finished grade will likely require a tighter spiral pitch since the soil may not provide adequate concrete confinement to achieve plastic moment capacity especially near the ground surface. See Section 3.15 for further guidance on the structural design and detailing of drilled shafts for seismic loadings.

The diameter of the reinforcement cage shall be the same throughout all portions of the shaft (above ground, through soil, and in rock). Shaft diameter in rock shall be specified 6 in. smaller than the portion in soil to produce no less than 2 in. cover in rock and 5 in. cover in soil. The 5 in. cover in soil is provided to account for a wide variety of wet, dry, and/or temporarily cased installation conditions which are less than ideal. For bridge piers with columns directly above the shafts, the columns are detailed 6 in. smaller in diameter than the shafts in soil. This allows the column cage to be the same diameter as the shaft cage which simplifies column to cage splicing and accommodates some field adjustment when shafts are constructed out of plan location.

3.10.2.5 Shafts for Abutments, Piers, and Walls

As described in Sections 2.3.6.2.2 and 3.9, and indexed in Appendix A1.2, drilled shaft Base Sheets are available for abutment and pier applications. These include open column piers, pier bents with web walls, and solid wall encased shaft piers (P-DS, P-DSWW, P-DSSW, P-DSTB, and P-DSCW). Note that at locations where there are significant seismic loads, special design considerations should be addressed. These include special reinforcement detailing as well as the potential for soil liquefaction. Drilled shafts supporting retaining walls shall utilize details taken from Base Sheet P-DSCW.

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The Final plans shall also provide the following items when applicable: estimated top of rock elevations, minimum bottom of permanent casing, and Estimated Water Surface Elevation (EWSE). See Section 2.3.6.4.2 for more guidance on determining the EWSE.

3.10.2.6 Shafts for Other Structures

Other structures under the jurisdiction of the Department also utilize drilled shaft foundations. These include high mast light towers, traffic signal mast arms, noise walls and sign structures. See the IDOT Design and Environment Manual (found on the primary Consultant Resources Page) Base Sheets covering high mast light tower and traffic signal mast arm shaft foundations and the IDOT Sign Structures Manual (found in the Fabrication section of the primary Bridges & Structures Page) for guidance and Base Sheets covering overhead, cantilever and monotube sign structure shaft foundations. Noise wall shaft foundations are typically designed by the vendor.

In each of these cases, the foundations act as single, independent, relatively short shafts for which the depths are typically controlled by lateral loads. Since these structures are less sensitive to service deflections, they are generally designed using Broms Method which uses ASD loadings and allowable lateral soil pressures reduced by a factor of safety typically between 2.0 and 2.5. See AGMU 11.1: "Light Tower, Traffic Signal, Sign Structure Foundation Design" containing links to a Design Guide and a spreadsheet.

3.10.3 Spread Footings

When the TSL plan specifies spread footing foundations, the final design configuration shall be developed to satisfy several geotechnical as well as structural requirements. Under the proper loading and site conditions, spread footing foundations are used to support semi-integral and stub abutments, bridge piers, wingwalls and retaining walls, overhead sign structures, approach slab and other structures.

Publication FHWA-IF-02-054 "Shallow Foundations" 2002, the AASHTO LRFD Bridge Design Specifications, and the IDOT Geotechnical Manual, found in the Foundations and Geotechnical section on the primary Bridges & Structures Page, should be referenced when designing spread footings. See Section 3.11 for guidance on the design of MSE walls supported by spread footing foundation soils.

The SGR should provide the designer with recommendations and foundation treatment guidance related to bearing capacity, footing elevations, sliding resistance, global stability, ground modification and other information.

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3.10.3.1 Bearing Resistance

Bearing resistance, also referred to as bearing capacity, is often the controlling parameter for which a footing is sized to assure adequate performance. The values provided in the SGR are normally sufficiently accurate for use in final design. However, the nominal bearing resistance is influenced by several factors including the footing length to width ratio, the embedment depth to width ratio as well as the applied loading inclination and eccentricity. The SGR may document some assumptions or qualifications on the use of the recommended values for design which do not fully take account of some of the factors discussed above for a particular project. As such, in cases where the design footing dimensions or loadings are not typical, the designer may wish to contact the geotechnical engineer to verify or recalculate the nominal bearing resistance using the proposed configuration and loads.

The footing should be shown to bear in soils or rock which can consistently provide the required strength throughout the footing area. This requires considerable geotechnical judgment and interpretations or extrapolations between all the soil borings or rock cores such that, upon excavation, the presence of expected foundation soils can be verified and the footing properly constructed. The SGR should provide assistance on these judgments and the approved TSL plan should provide approximate footing elevations. The actual performance of a spread footing is dependent on the soils within a minimum of one footing width below the foundation. Consequently, proper calculation of the nominal bearing resistance should include evaluation of the foundation material below the footing which is not normally verified during construction. In cases where conditions are not uniform, the excavation may be extended up to 2 ft. deeper to allow the footing to be placed on better material or the over excavation may be filled with concrete.

The nominal bearing resistance (Q_{ult}) is the resistance available from the soil to support the footing assuming it is uniformly loaded. Since a true uniform loading is rarely the case for design, an equivalent uniform bearing pressure (Q_{eubp}) is calculated and compared to bearing resistance to determine the required footing size. The bearing resistance should be computed using the formulation of the general bearing capacity equation(s) provided in the AASHTO LRFD Bridge Design Specifications including the effects of the various influence factors as appropriate. For LRFD, the form of the basic design equation includes the appropriate load and resistance factors and is given by,

$$\phi Q_{ult} \geq LF \times Q_{eubp}$$

The maximum or peak trapezoidal bearing pressure will typically exceed Q_{eubp}, but should not be used in place of Q_{eubp} to size the footing. The maximum applied service bearing pressure Q_{max} shall be shown

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on the Final Contract plans to provide construction inspection personnel some insight concerning the applied loading demands on the foundations which assists in field verification. As a rough guide or rule of thumb, the unconfined compressive strength, Q_u, of cohesive soils (shown on the boring logs or sometimes indicated during construction inspections) should be equal to or larger than Q_{max} shown on the plans.

3.10.3.2 Sliding Resistance

Sliding is relevant for spread footings which support inclined loads or are required to support substantial horizontal forces.

IDOT typically neglects the resistance to sliding supplied by passive pressure. This is because of uncertainties associated with future excavations, variability in the backfill material and compaction, and the fact that the lateral deflections to mobilize full passive pressure are usually larger than permitted to satisfy serviceability requirements. However, relying on some passive pressure may be warranted for very deep footings, or to resist some extreme event limit state loadings.

Sliding resistance is determined differently depending on whether the spread footing is setting on granular soil, cohesive soil, or rock. For granular soils, the sliding resistance is calculated as the vertical resultant, P, times the tangent of the friction angle for footings cast on in-place aggregate. Shear keys are not recommended for granular soils due to constructability concerns. For cohesive soils, sliding resistance is calculated as cohesion times the effective footing width B. Lower strength clays require special attention to ensure adequate sliding resistance and, in some cases, have successfully utilized shear keys. When shear keys are required, the following note should be added to the plans.

> *The shear key excavation shall be made with care to produce near-vertical sides as shown on the plans. The footing and shear key excavation shall be cleaned of loose material and the concrete poured against undisturbed in-place soils.*

Sliding resistance on rock is evaluated in a similar manner as that for a cohesive or a granular soil, depending on rock type and weathering. The designer should note that “horizontal planes of weakness” in rock can control the design. Keying the lower portion of the footing into rock can provide substantial added “passive pressure” resistance in addition to the friction or adhesion along the base of the footing. When spread footings are used or if additional sliding resistance is required from embedment in rock, the following note should be added to the plans.

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The bottom of footing elevation(s) shall be adjusted to ensure a minimum embedment of ____ inches in non-weathered rock. The rock excavation shall be made with near-vertical sides at the plan dimensions to allow the sides and base of the embedded portion of the footing to be cast against undisturbed rock surfaces.

Footings to be placed on shale or sensitive silts, which have the potential to degrade upon excavation and prolonged exposure to air and water, shall have a "mud slab" or "seal coat" concrete layer placed to maintain the deposit's integrity and sliding resistance. In these cases, the following note should be included on the plans.

The footing excavation(s) shall be undercut by 6 in. and immediately filled with seal coat concrete to prevent degradation of the exposed foundation material surface.

3.10.3.3 Settlement

While bearing and sliding resistance calculations are relatively straightforward and clearly indicate whether they will be adequate, the analysis and design to satisfy settlement is somewhat more subjective. Although the AASHTO and FHWA references noted previously provide extensive guidance on analysis of footing settlement, the designer ultimately is responsible to develop the necessary footing configuration to assure adequate short and long term performance, given the controlling foundation loadings and using the structures specific tolerance of settlement. The SGR should provide some assistance regarding expected settlement, but since the final foundation loadings, footing configuration, and deflection tolerance are not known during the development phase (TSL), the designer may need to contact the geotechnical engineer if questions or concerns arise during final design.

Most structures under IDOT's jurisdiction have some tolerance for settlement. Since accurately determining the tolerance may be somewhat subjective and the calculation of short and long term settlement, in most cases, is not exact, spread footings are normally used when a relatively high confidence in the predicted and actual settlements exists.

The load case specified in LRFD for settlement is Service I in which the load factors are 1.0. Consequently, LRFD analysis for settlement is, in effect, essentially an ASD type analysis. When the settlement analysis indicates concern for excessive foundation movement, uneven settlement, or rotation; the design may require ground improvement techniques such as removal and replacement, aggregate columns, pre-loading, waiting periods, or other project specific construction sequence requirements to assure long term performance.

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3.10.3.4 Structural Resistance

Structurally, considerations for spread footing design include design for flexure, flexural shear and punching shear as appropriate for each particular project.

The minimum thickness of a spread footing under an abutment or pier shall be 2 ft. unless design calculations indicate the necessity for a greater thickness.

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3.11 Retaining Walls

This section covers the Department's policies related to the design and detailing of walls and elements to resist lateral soil and surcharge demands specified herein and by the AASHTO LRFD Bridge Design Specifications. Detailing that meets the material and pre-qualification requirements of the Department comprise the standard basis for design.

There are several types of retaining walls commonly designed and constructed for/by the Department. These are mechanically stabilized earth (MSE), cast-in-place T-type, soldier pile, and permanent sheet pile. Soil nailed, gabion, precast modular, segmental block and other specialized wall types are also occasionally built.

When designing retaining walls, references which should be consulted in conjunction with a project's Structure Geotechnical Report (SGR) include:

1. IDOT Geotechnical Manual (found in the "Foundations and Geotechnical" section of the primary Bridges & Structures Page)
2. IDOT Culvert Manual (found in the "Design" section of the primary Bridges & Structures Page)
3. "Earth Retaining Structures Manual," FHWA-NHI-99-025, 1999
4. "Earth Retaining Systems," FHWA-SA-96-038, 1996
5. "Design and Construction of MSE Walls and Reinforced Soil Slopes," FHWA-NHI-10-025, 2009
6. "Manual for Design and Construction of Soil Nailed Walls," FHWA-SA-96-069R, 1996
7. AASHTO LRFD Bridge Design Specifications –Section 11

The following subsections present design overviews and some standard details for the wall types mentioned above.

3.11.1 Mechanically Stabilized Earth (MSE)

An MSE wall is a three-dimensional stabilized mass of compacted soil, soil reinforcement and wall facing elements which essentially behaves as a rigid body to resist earth pressure and other applied loadings by its own weight. The TSL planning engineer using the SGR is responsible for analyzing the applied loadings and foundation soils as well as specifying both the reinforced mass minimum dimensions and any foundation treatment necessary to assure that global and external stability is satisfied. The final plans designer will detail the vertical and horizontal limits of the wall (discussed below) as well as the extent of any ground improvement. The contractor shall select one of the approved MSE wall suppliers who is responsible for designing the internal stability of the reinforced mass. The design shall provide corrosion

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allowance to ensure a design life of at least 75 yrs. The Shop Drawings and internal stability design calculations submitted by the supplier are reviewed by the BBS Foundations and Geotechnical and Design Units to ensure compliance with the contract plan requirements and adequacy of the internal stability design. Section 522 of the Standard Specifications covers MSE walls.

Figures 3.11.1-1 and 3.11.1-2 present typical sections and standard details for MSE walls. These and other presented figures in following subsections are intended to be generic enough to allow for variations between approved wall systems. The designer shall be familiar with the Standard Specifications for MSE walls and delineate between the portions of the design that are the responsibility of the contractor and those which shall be detailed on the Contract plans. Base of MSE walls at grade separations are not required to be designed for vehicle collisions.

The “top of exposed panel line,” “finished grade line at front face of wall,” and “theoretical top of leveling pad line” elevations are to be shown on the Final plans (see Figures 3.11.1-1 and 3.11.1-2). These elevations define retention requirements, establish bid payment limits and are the basis of the wall supplier’s design. The reinforced mass of an MSE wall extends an orthogonal distance of 0.7*H* from the outside face unless the external stability design requires a wider base, where *H* is the actual height of wall at any section as defined in Figures 3.11.1-1 and 3.11.1-2. The height of the wall detailed on the plans is the distance between the theoretical top of leveling pad to the top of the anchorage slab, top of coping, or top of deck over the abutment depending on the situation.

External design considerations for MSE walls include bearing resistance, sliding, settlement and overturning/eccentricity. Global stability (overall slope stability) shall also be considered. The SGR shall be referenced for pertinent project design capacities and evaluations covering the considerations listed above. The geotechnical engineer responsible for the report and its evaluations and recommendations should be consulted by the wall designer when assistance is needed.

The designer shall verify the bearing resistance obtained from the SGR is adequate by comparing it to the equivalent uniform bearing pressure (Q_{eubp}) computed using all the external loadings and weight of the reinforced mass. Applicable loads can include, but are not limited to, traffic, railroad, noise wall, impact barrier, scour, seismic, abutment footings, backslope, and embankment. Since the reinforced mass is more flexible and forgiving than a reinforced concrete footing, the resistance factor or factor of safety used in design computations is less conservative. Thus, the foundation soil’s factored or allowable bearing resistance will not be the same for all wall types. Bearing resistance and settlement concerns are the two most common reasons why MSE walls are determined not to be feasible which results in other more expensive wall types being used. It is recommended that ground modification treatments be evaluated, and when cost effective, specified in the plans to address bearing resistance or settlement

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concerns as opposed to simply changing wall type during the design phase. Please note that the leveling pad is not a permanent structural element and does not need to have bearing pressure or settlement satisfied. The panels are supported long-term by the soil reinforcement in shear and the leveling pad is only a construction tool to support the first row of panels.

When checking sliding, the designer should pay particular attention to the friction angle of the soil material below the base of the stabilized mass. The friction angle of the reinforced mass may be assumed to be 34 degrees. Sliding calculations shall be performed following AASHTO recommendations and Section 3.10.

At least some settlement in MSE walls is usually expected because of their considerable weight. Some of the settlement will probably occur during construction and some afterwards. The designer is responsible for assessing the magnitude, horizontal limits, and time of settlement as well as the effects of any settlement on the wall, the infrastructure placed on top of the wall, and through the reinforced volume of soil. It should also be noted that MSE walls are well suited to handle settlements because of panel articulation and the ability of the semi-rigid soil mass to remain stable during relatively uniform movements. However, abrupt differential settlement or significant settlements occurring in walls can be problematic. As such, special wall and/or foundational treatments may be required for these cases.

The designer should note that removal and replacement of unsuitable soils, use of lightweight fill, stage construction with the use of a sacrificial fascia, wick drains, stone columns, a stabilized mass wider than 0.7*H* and other techniques can be utilized to address settlement, slope stability, bearing pressure or sliding deficiencies.

Regardless of whether ground modification is used, when total settlements (including after the approaches/roadway/anchorage slab are in place) are expected to exceed two inches, the following note should be added to the contract plans:

> *The MSE wall supplier is alerted to the fact that ___ inches of settlement are anticipated from Stations _____ to _____ and shall take appropriate measures to accommodate this settlement in the wall design.*

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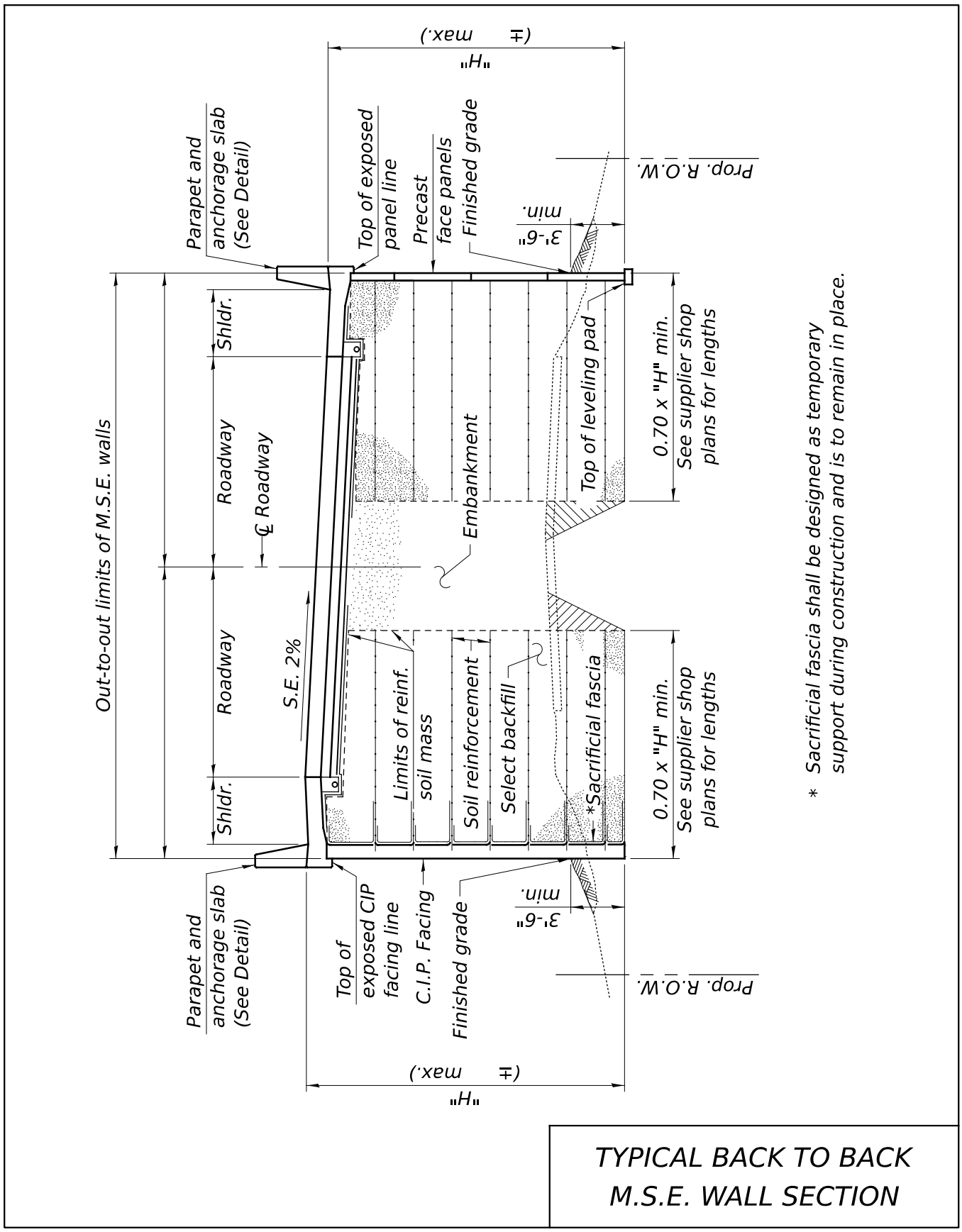


Figure 3.11.1-1

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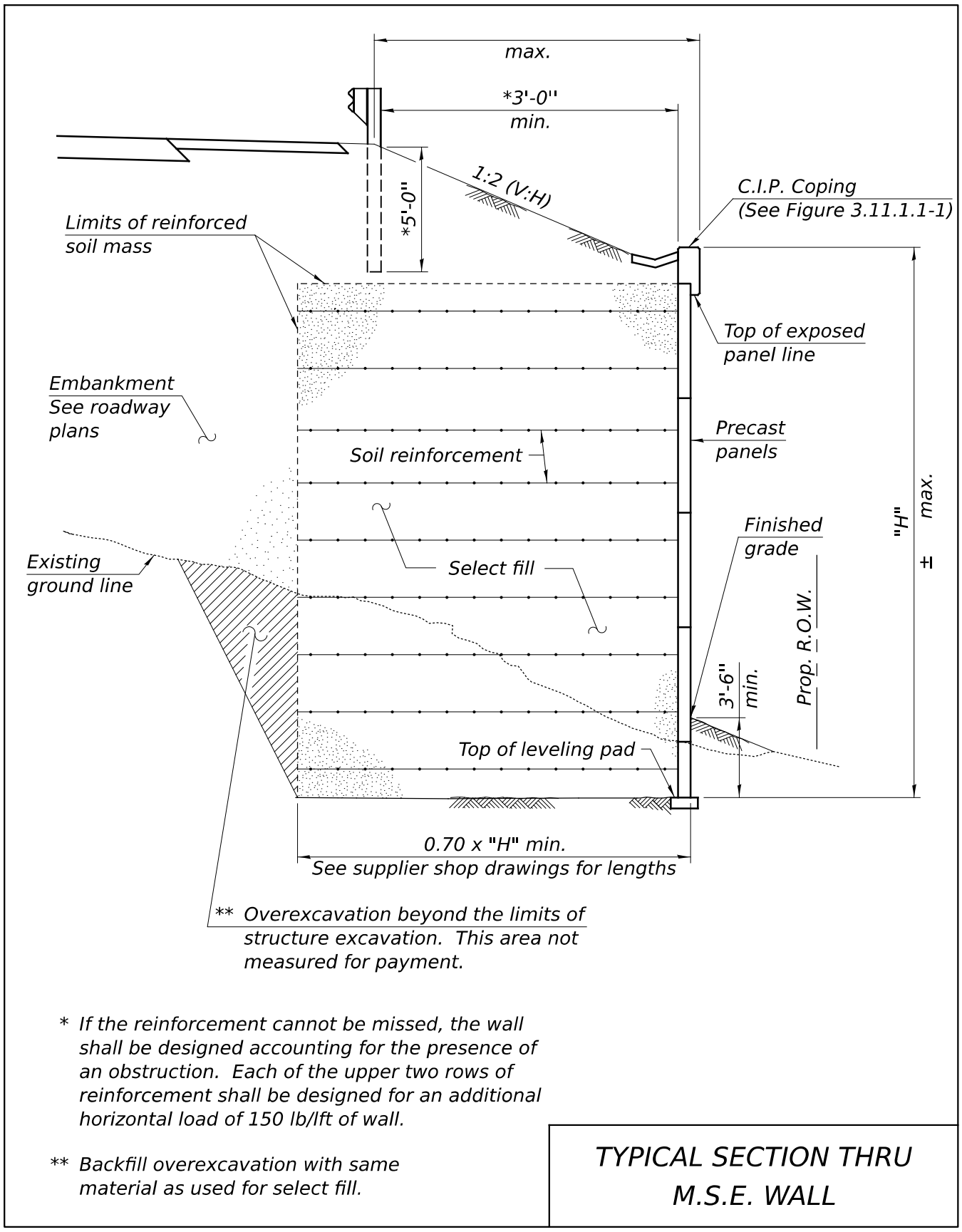


Figure 3.11.1-2

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3.11.1.1 Coping

A cast-in-place (CIP) coping, traffic barrier, or other capping design details are included on the plans to cover the panel's steps while maintaining a smooth bottom of coping line. This line on the plans is shown as "top of exposed panel line." Figure 3.11.1.1-1 presents standard details for CIP coping with precast panels. Figure 3.11.1.1-2 and 3.11.1.1-3 presents standard details for chain link fence mounted CIP coping with precast panels.

Whenever a fence is mounted to a CIP coping, the following notes shall be added to the plans:

1. *See IDOT Highway Standard drawing 664001 for fence details. Installation shall be per Section 664 of the Standard Specifications. All steel fabrication shall be per Article 505.04 of the Standard Specifications.*
2. *Fence shall be continuous over concrete coping expansion joints. Joint locations in the existing coping shall be measured in the field prior to fabrication of chain link fence details. Cost included with "Chain Link Fence, 4' Attached to Structure"*
3. *Edge distance from a construction or expansion joint in either existing or proposed copings to the closest expansion anchor shall be per manufacture's specifications. Expansion anchors shall have a minimum embedment of 6 inches or greater if specified by the manufacturer.*
4. *At all fence locations, the end posts shall be located 6" minimum and 12" maximum clear from the bottom flange of the bridge fascia beam.*
5. *Base plates and stiffeners shall be fabricated from material meeting the requirements of AASHTO M270 Grade 50. Base plates, stiffeners, and posts shall be Hot-Dipped Galvanized per AASHTO M111.*
6. *Drill drain hole to miss fillet weld and prior to galvanizing. Hole shall be drilled as close to the weld as possible.*

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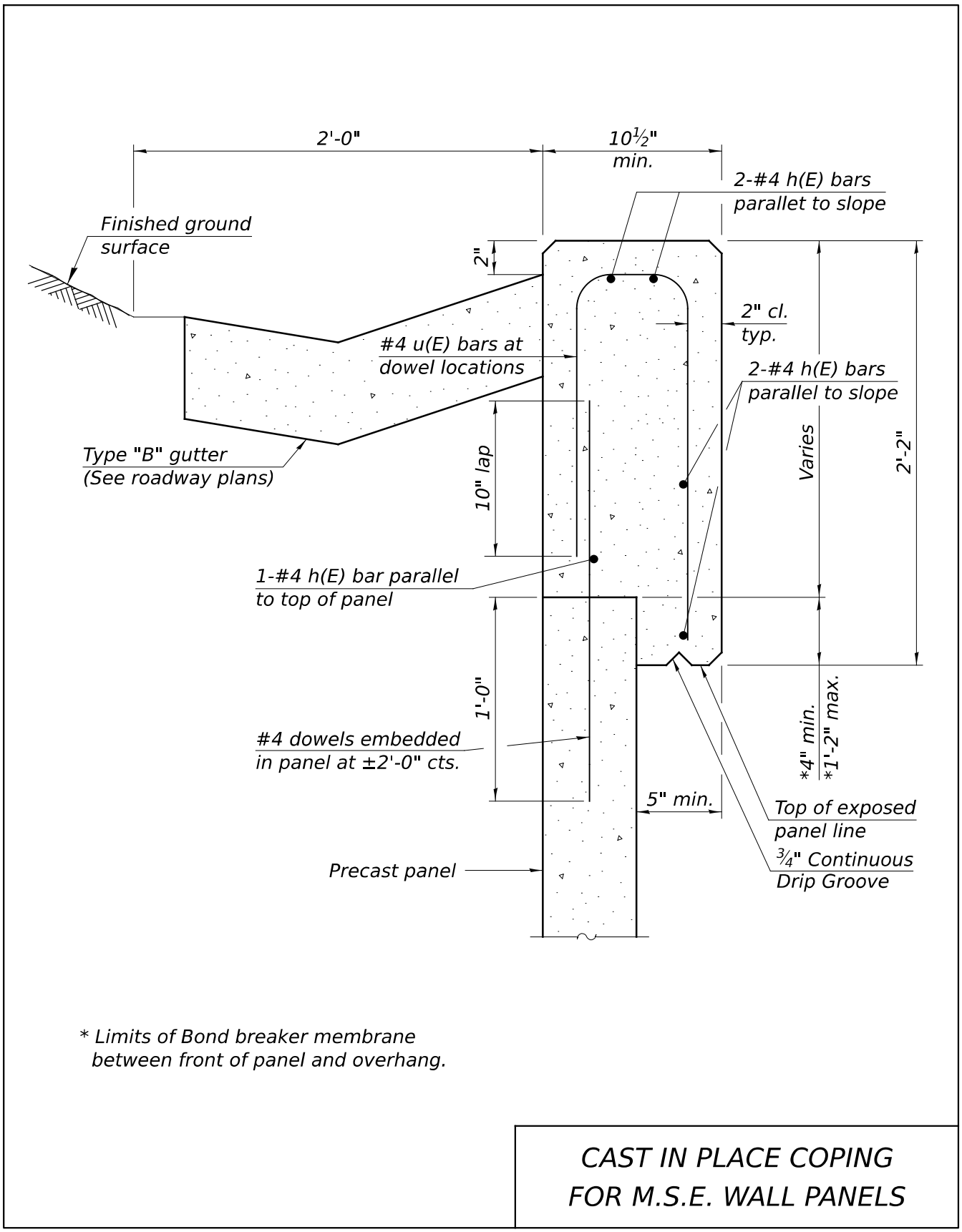
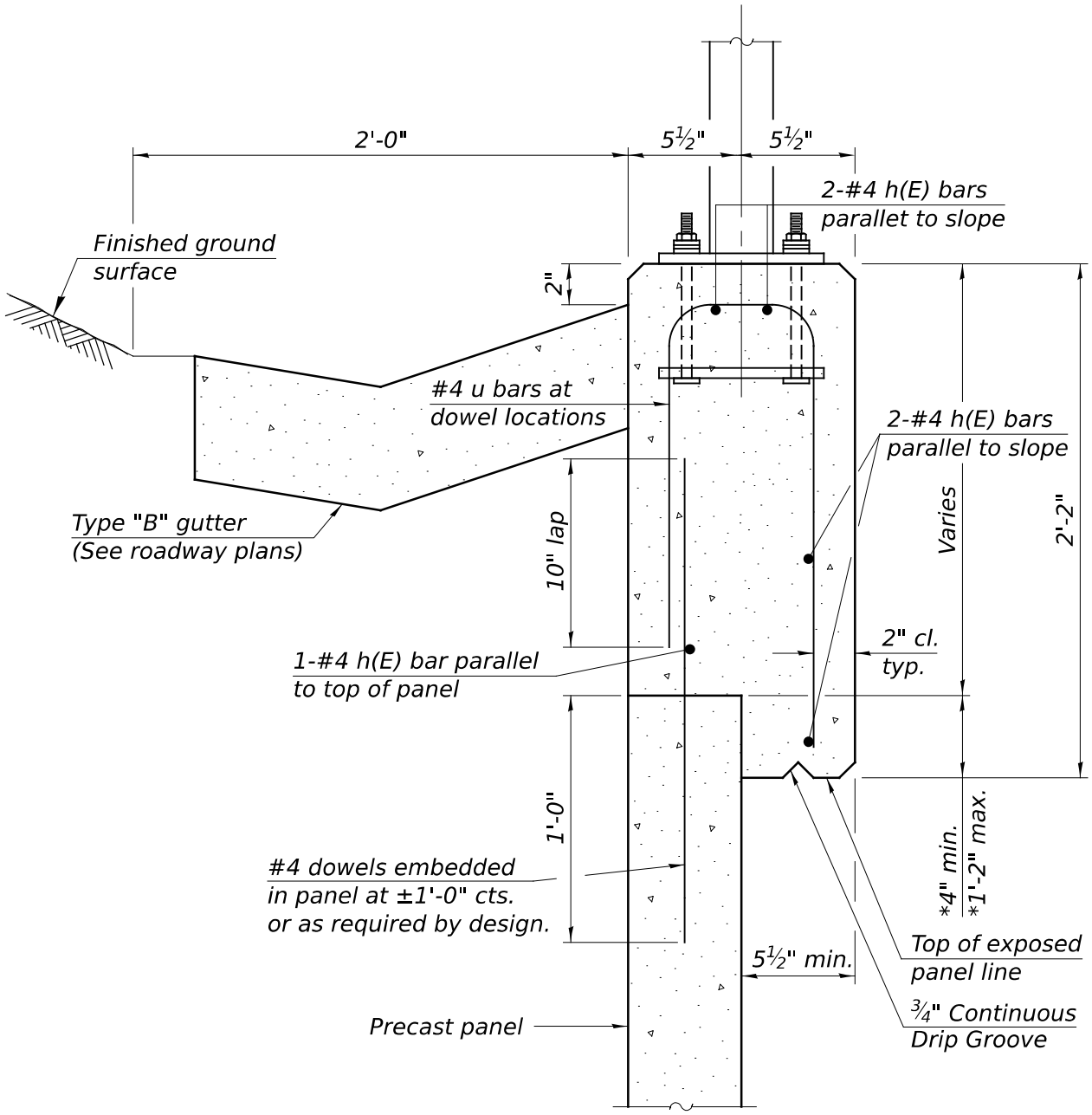


Figure 3.11.1.1-1

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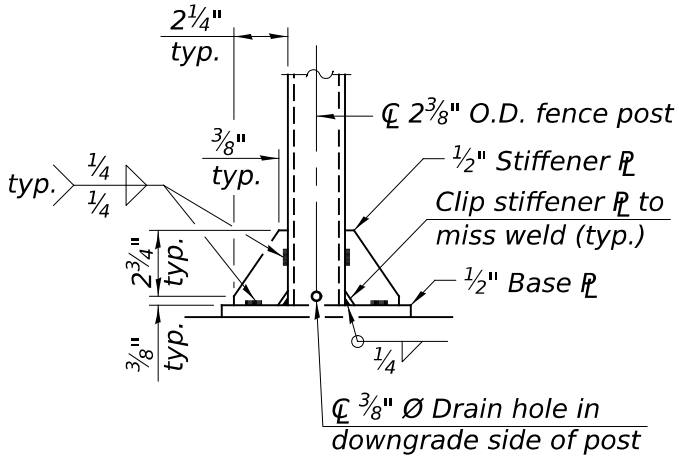
Notes:

When Precast Coping substitution is used, the connectin of the coping to the panel shall be desigend for the increased loading.

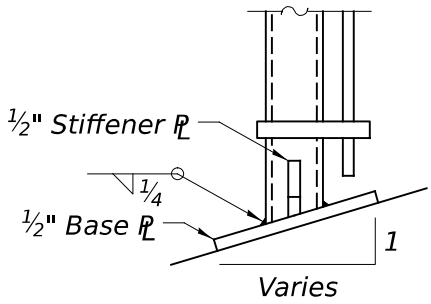
FENCE MOUNTED ON C.I.P. COPING

Figure 3.11.1.1-2

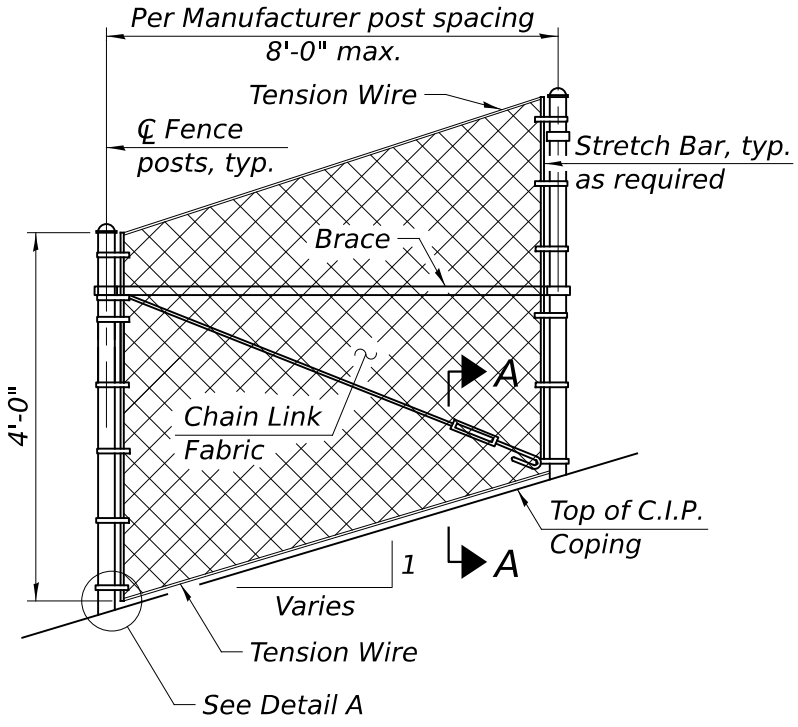
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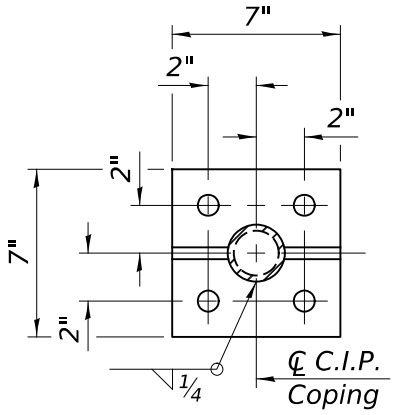
SECTION A-A
(Fence post base plate connection)



DETAIL A



CHAIN LINK FENCE ELEVATION



BASE PLATE PLAN

CHAIN LINK FENCE, ATTACHED TO STRUCTURE DETAILS

Figure 3.11.1.1-3

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3.11.1.2 Vertical and Horizontal Obstructions

It is not uncommon for drainage structures to be inside of the reinforced soil mass. Figure 3.11.1.2-1 presents 3 cases of generic details for catch basins with pipes within the region of reinforced soil. The MSE supplier is responsible for the design of the load transfer system around drainage structures as indicated in Figure 3.11.1.2-1.

Two standard details for pipes passing through cast-in-place wall panels are shown in Figure 3.11.1.2-2. As indicated in the figure, the wall supplier shall determine/design some of the required dimensions for the details in accordance with the specific situation.

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 3.0 ft from the wall face, driven 5.0 ft below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction. Each of the upper two rows of reinforcement shall be designed for an additional horizontal load of 150 lb per linear foot of wall, for a total additional load of 300 lb per linear foot of wall. See Figure 3.11.1-2, and see Figure 3.11.1.2-3 for guidance on horizontal obstructions.

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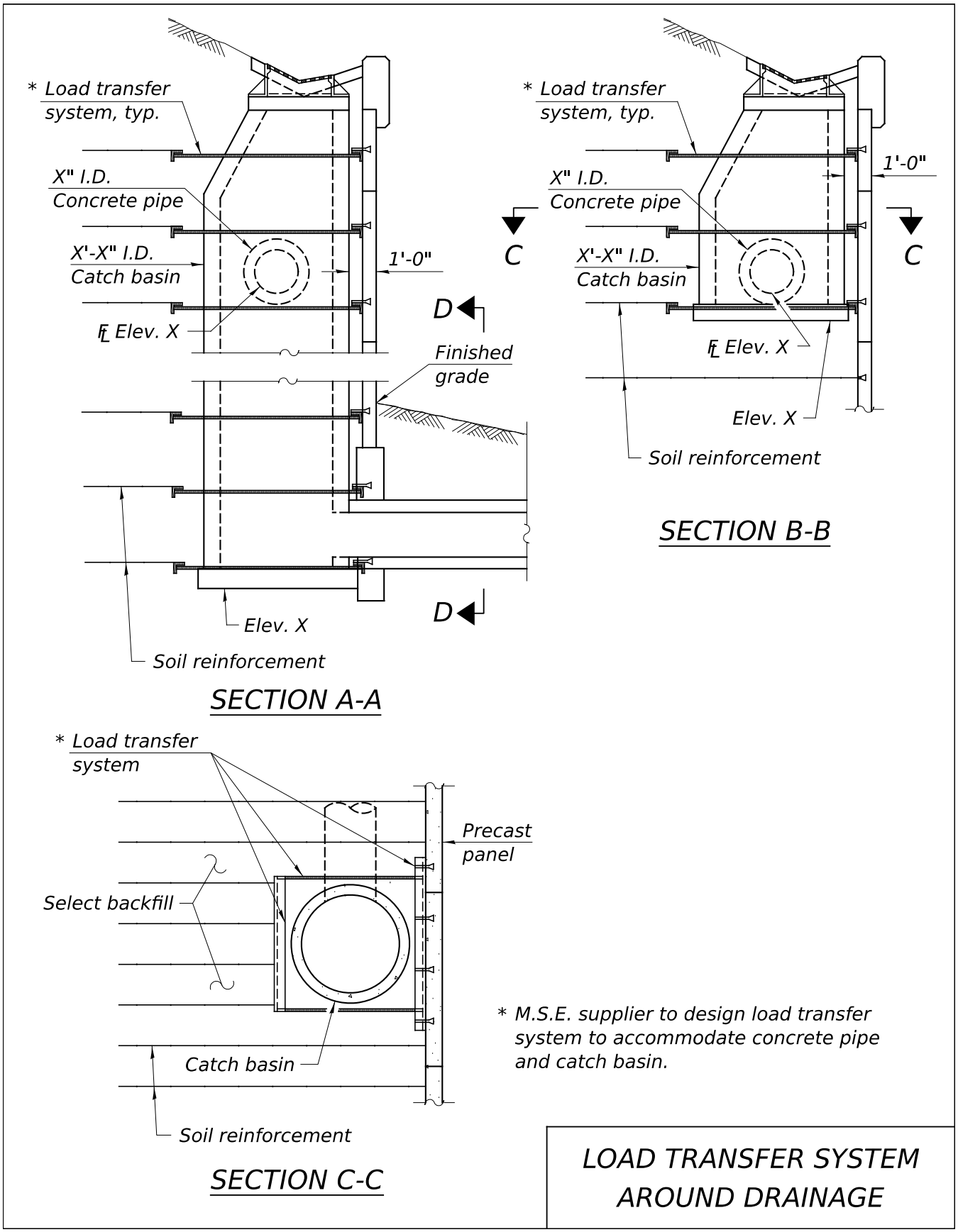


Figure 3.11.1.2-1

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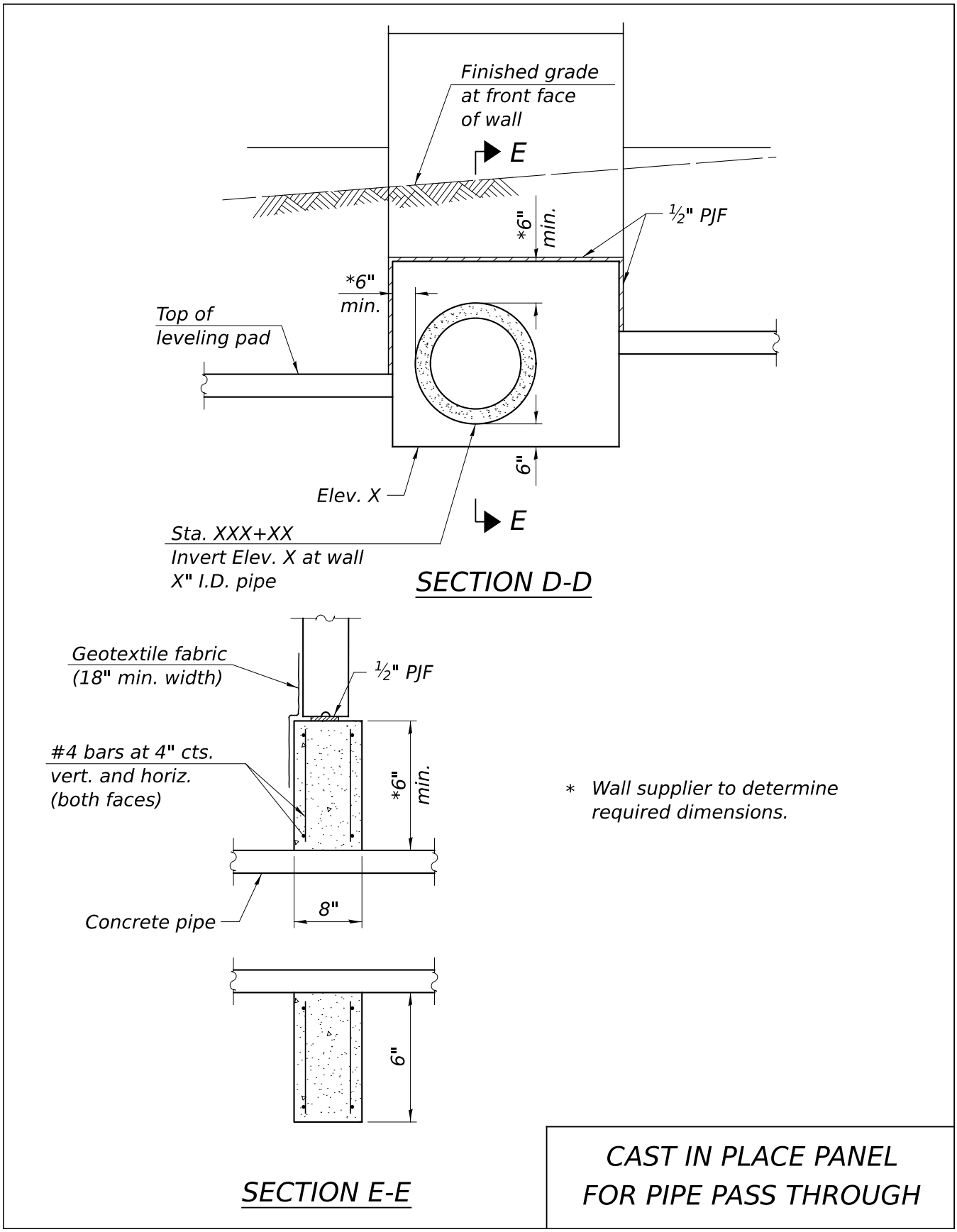
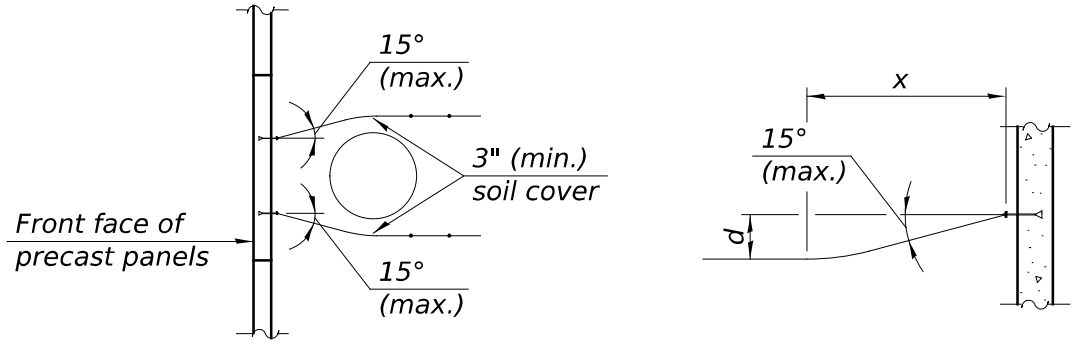


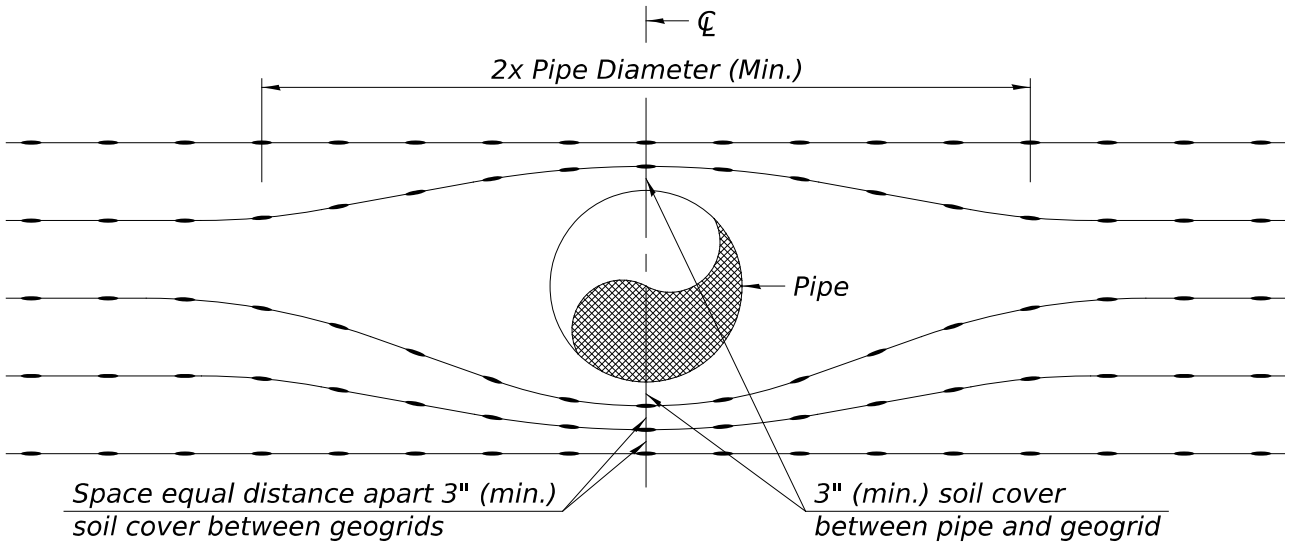
Figure 3.11.1.2-2

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Additional depth (d) required, in.	Required minimum distance (x) to achieve smooth bend, in.
3	27
6	39
9	48
12	60
15	72

INEXTENSIBLE REINFORCEMENT



EXTENSIBLE REINFORCEMENT

HORIZONTAL OBSTRUCTION IN M.S.E. WALL

Figure 3.11.1.2-3

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3.11.1.3 Anchorage Slab and Section Thru Parapet

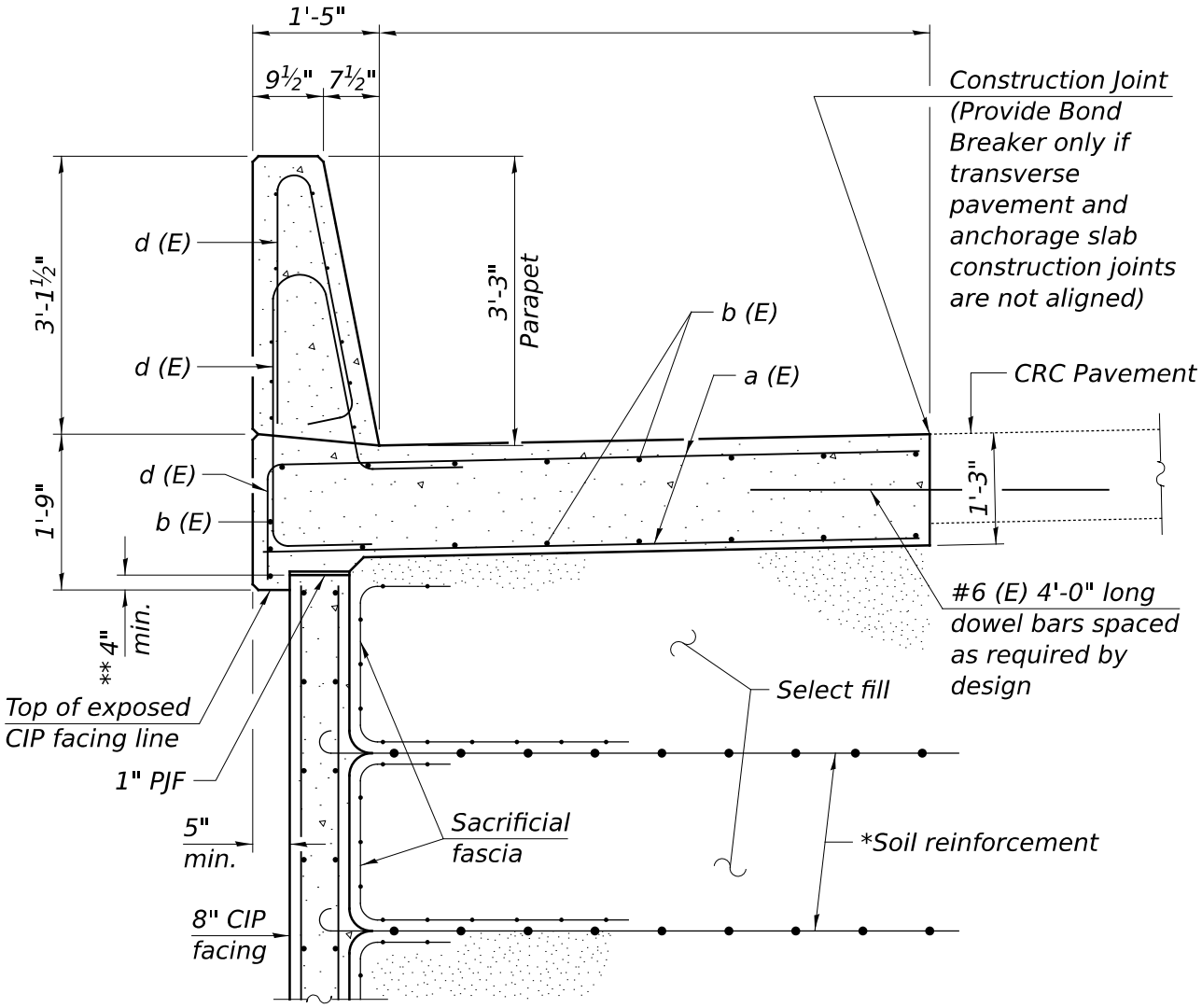
Figure 3.11.1.3-1 presents details for a section through parapet with an anchorage slab for cast-in-place facing. Figure 3.11.1.3-2 gives similar details, but with precast facing. A note specifying a bearing pressure surcharge of 1 ksf and horizontal sliding force of ½ kip/ft. of wall from the anchorage slab shall be provided on the plans as shown in Figures 3.11.1.3-1 and 3.11.1.3-2. The MSE wall supplier's internal stability design shall account for these forces. The anchorage slab, parapet, and the global stability of the MSE wall system shall also be designed by the engineer responsible for the Contract plans to resist a traffic impact loading on the parapet for a specified crash test level (TL). See Section 2.3.6.1.7 for more information on railings and TL requirements. The MSE wall designer shall be responsible for including an impact rupture load and an impact pullout load in the design of the top two layers of soil reinforcement, see Figures 3.11.1.3-1 and 3.11.1.3-2 for values.

The maximum width of the anchorage slabs shall be the width of the shoulder to avoid joints in the driving lanes. The slab may be narrower than the shoulder if structurally adequate. Anchorage slabs shall not be used on approach slabs.

When transverse joints are required in anchorage slabs, they shall be aligned with joints in parapets and, if possible, adjacent pavement joints.

Joints in parapets shall follow the details shown in Fig. 3.2.4-10, except that the bottom portion of the parapet shall also be preformed cork joint filler with a detail consistent with that of the top portion of the parapet.

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Soil Reinforcement Layer	Impact Rupture Load (lb./ft. of wall)	Impact Pullout Load (lb./ft. of wall)
Upper Layer	2,300	1,300
Second Layer	600	600

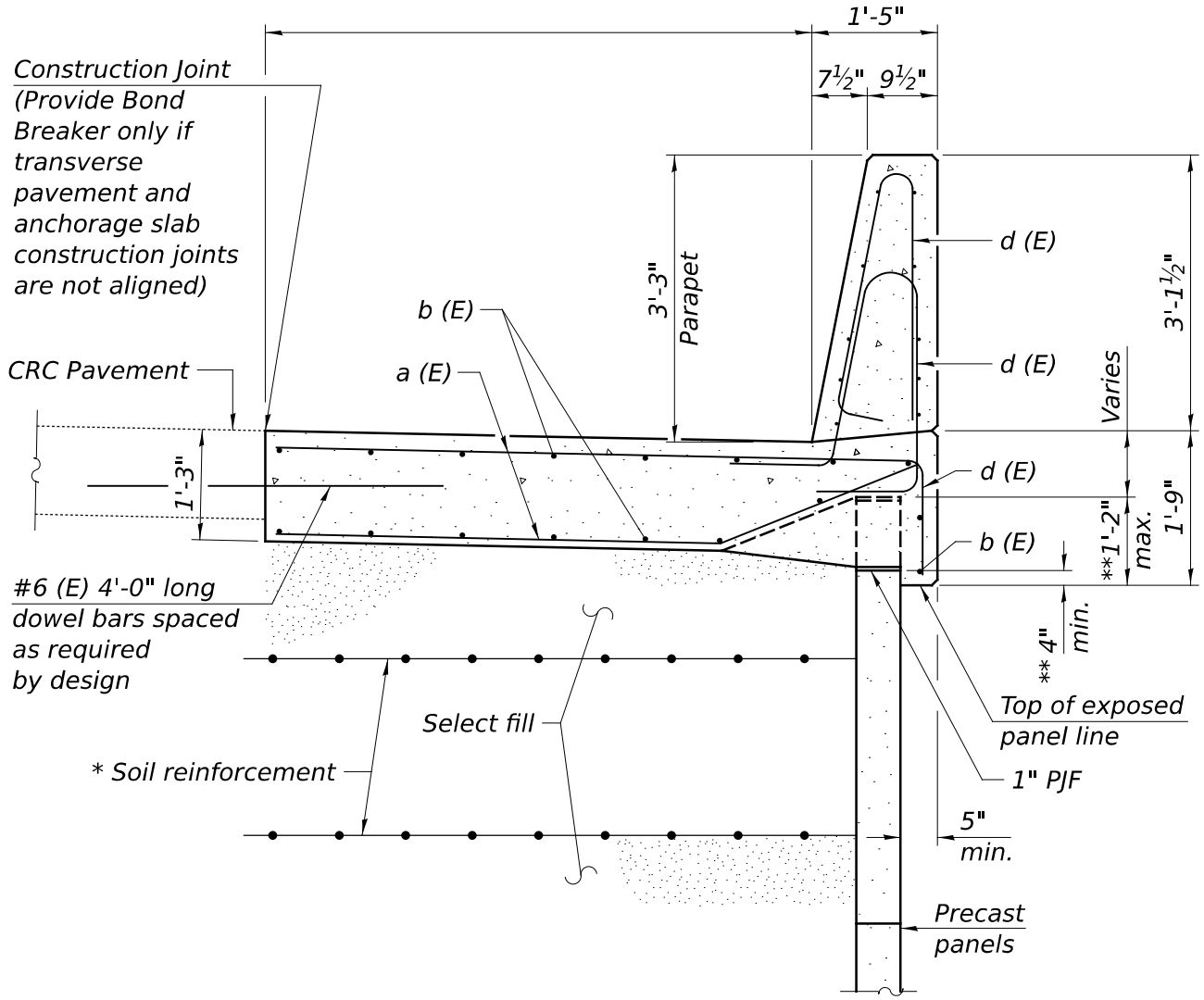
* The MSE wall supplier's internal stability design shall account for the anchorage slab's bearing pressure surcharge of 1.0 ksf and corresponding horizontal force with respect to reinforement layer location and load type.

** Limits of Bond breaker membrane between front of panel and overhang.

SECTION THRU CIP BARRIER AND ANCHORAGE SLAB ON CIP FACING

Figure 3.11.1.3-1

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Soil Reinforcement Layer	Impact Rupture Load (lb./ft. of wall)	Impact Pullout Load (lb./ft. of wall)
Upper Layer	2,300	1,300
Second Layer	600	600

* The MSE wall supplier's internal stability design shall account for the anchorage slab's bearing pressure surcharge of 1.0 ksf and corresponding horizontal force with respect to reinforement layer location and load type.

** Limits of Bond breaker membrane between front of panel and overhang.

SECTION THRU CIP BARRIER AND ANCHORAGE SLAB ON PRECAST PANELS

Figure 3.11.1.3-2

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3.11.2 Cast-In-Place T-Type

This wall is considered to have a 75 year design life. Figure 3.11.2-1 illustrates expansion and construction joint details for cast-in-place T-type walls.

T-type retaining walls can be supported by spread footings, piles, or drilled shafts. See Section 3.10 for additional guidance.

The SGR contains evaluations and recommendations for specific situations such as: bearing resistance, sliding, settlement, stability, pile design requirements, etc. The geotechnical engineer responsible for the SGR should be consulted by the wall designer when assistance is needed and when pile sizes outside of those addressed in the SGR or conditions beyond the SGR arise.

3.11.2.1 Geometry

The minimum stem thickness at the top of a T-type wall shall be 10 in. for concrete placement purposes. The minimum stem thickness of a T-type wall with a parapet mounted at any height of the wall shall be the thickness of the base of the parapet.

Stem thickness shall be shown in 0.5 inch increments.

No batter in the stem is provided when the stem thickness is equal to or less than 12 inches. When stem batter is required, the front (exposed) face should only be battered up to 0.5 inches per foot (H:V). Any further required batter should be provided on the back (in contact with earth) face of the stem.

For pile-supported footings, the minimum footing thickness shall be 1 ft. – 9 in. For pile-supported footings in seismic zones, piles shall either be embedded 24 inches into the footing or a positive connection between the piles and concrete as per Section 3.15.

Figure 3.11.2.1-1 shows minimum dimensions and reinforcement layout for a T-type wall on spread footings. Figure 3.11.2.1-2 shows minimum dimensions and reinforcement layout for a T-type wall on piles.

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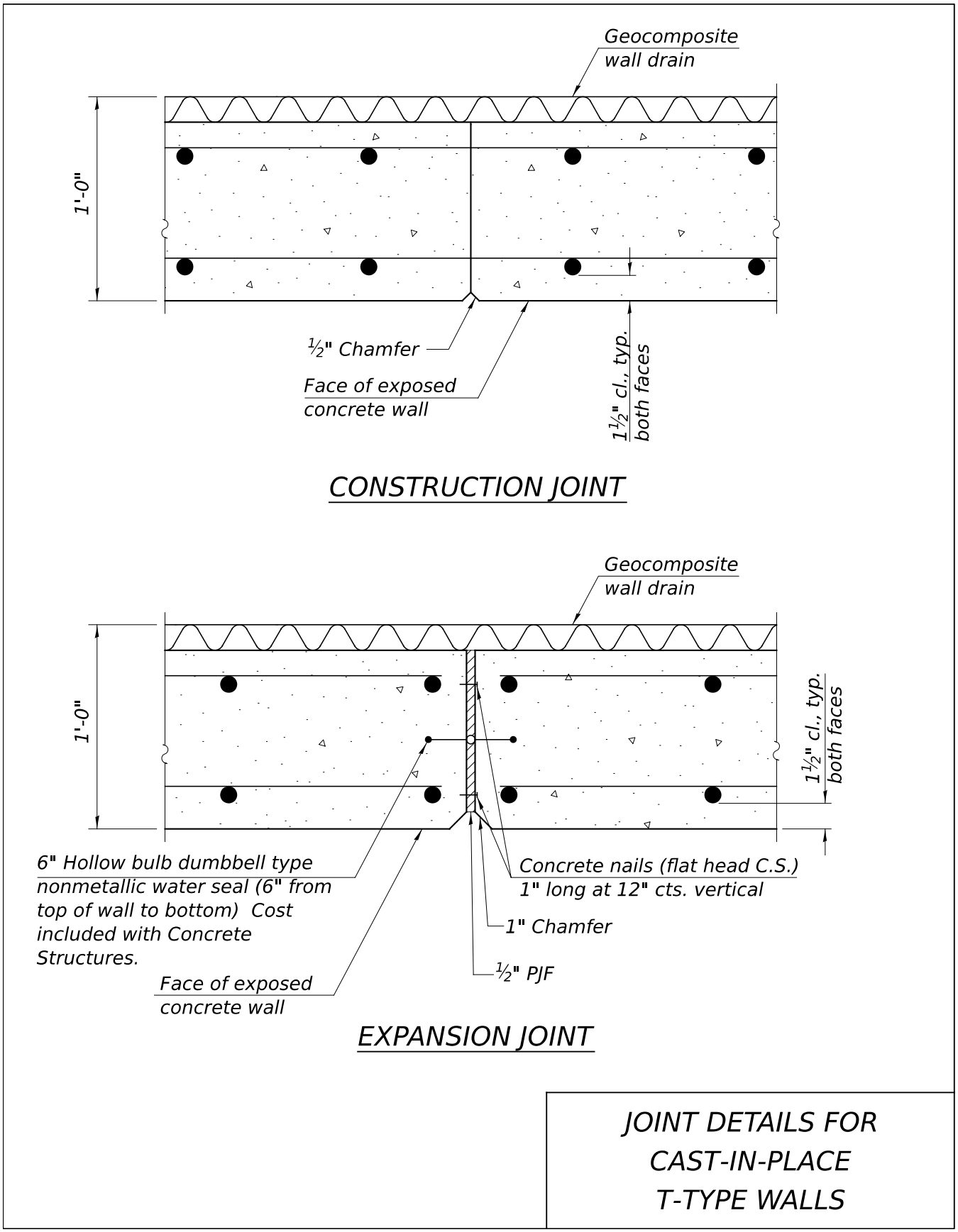


Figure 3.11.2-1

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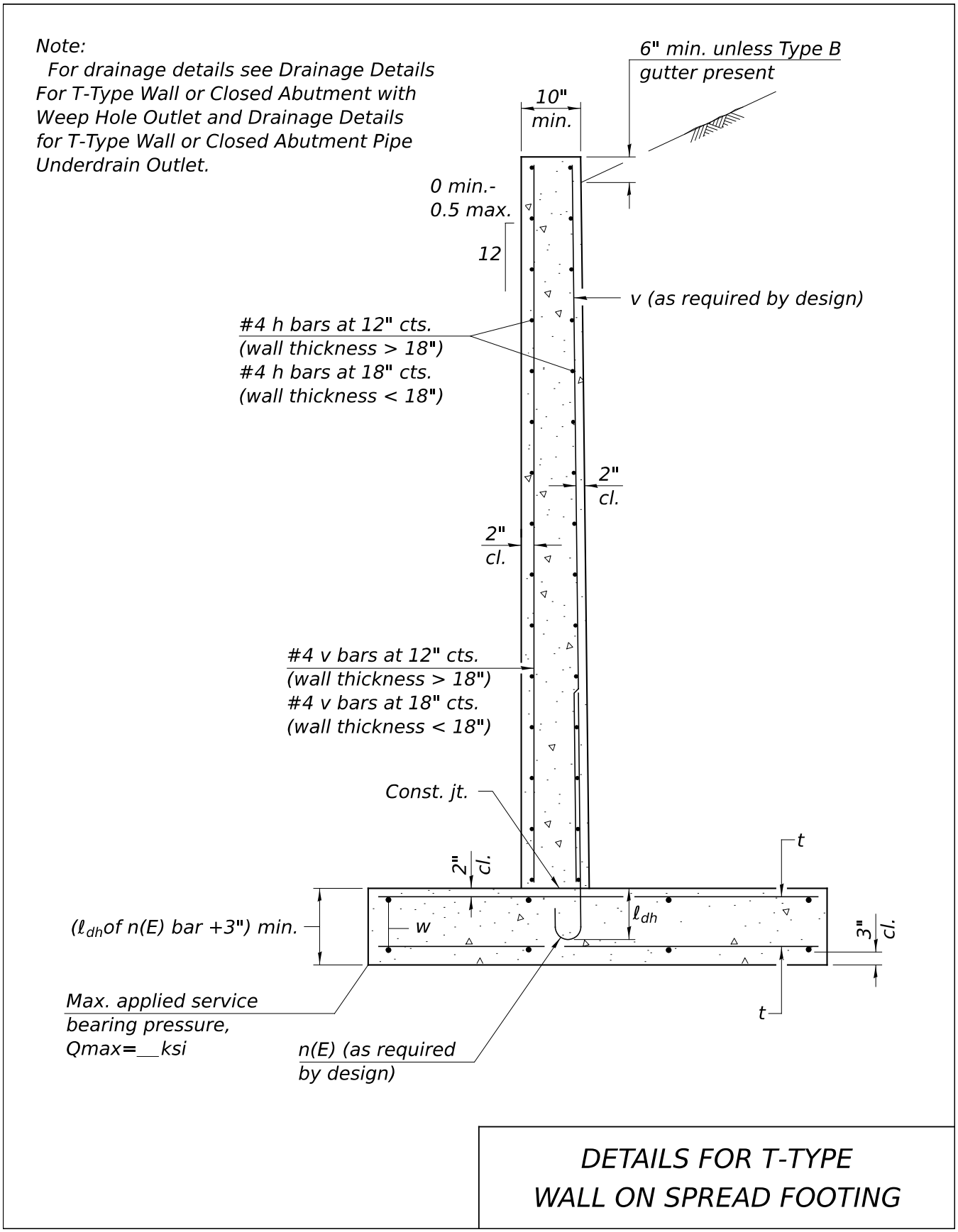


Figure 3.11.2.1-1

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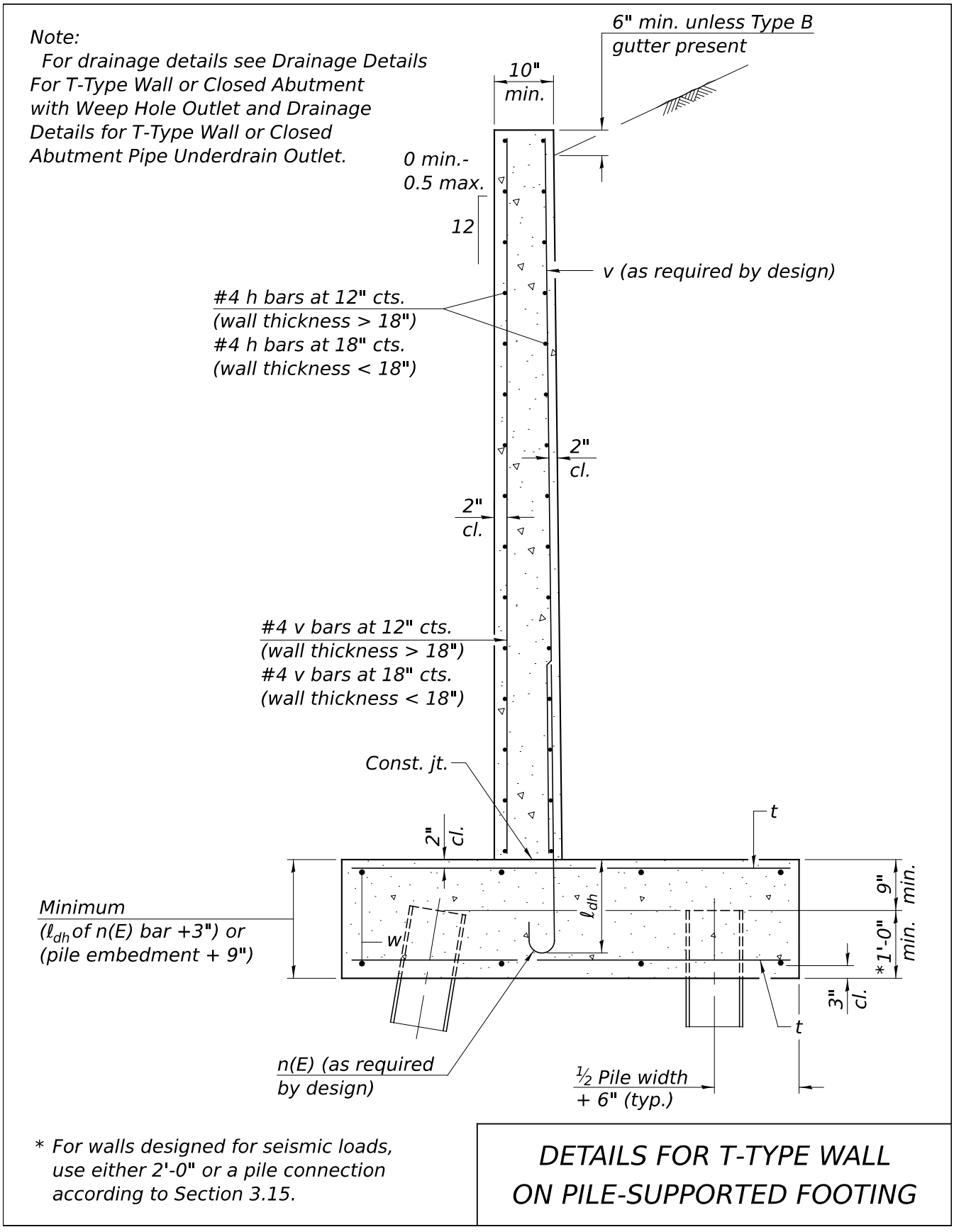


Figure 3.11.2.1-2

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3.11.2.2 Loads

The soils behind T-type walls may be assumed to have an angle of internal friction of 28 degrees. Most T-type walls founded on pile-supported footings or spread footings on soil will deflect enough to assume active soil conditions. However, walls founded on rock or with heavily battered piles may not deflect enough to warrant this assumption and should be designed assuming at-rest soil conditions.

IDOT T-type walls with granular backfill for structures and pipe underdrains may be assumed to be fully drained. IDOT T-type walls with weepholes may be assumed to drained at all levels above the level of the weepholes.

Crash loading on parapets may be assumed to be spread over a 1:1 slope to the base of the wall. Note that this loading cannot be distributed over an expansion joint.

3.11.2.3 Drainage and Backfill Details

Drainage and backfill details for T-type retaining walls are presented in Figures 3.11.2.3-1 and 3.11.2.3-2.

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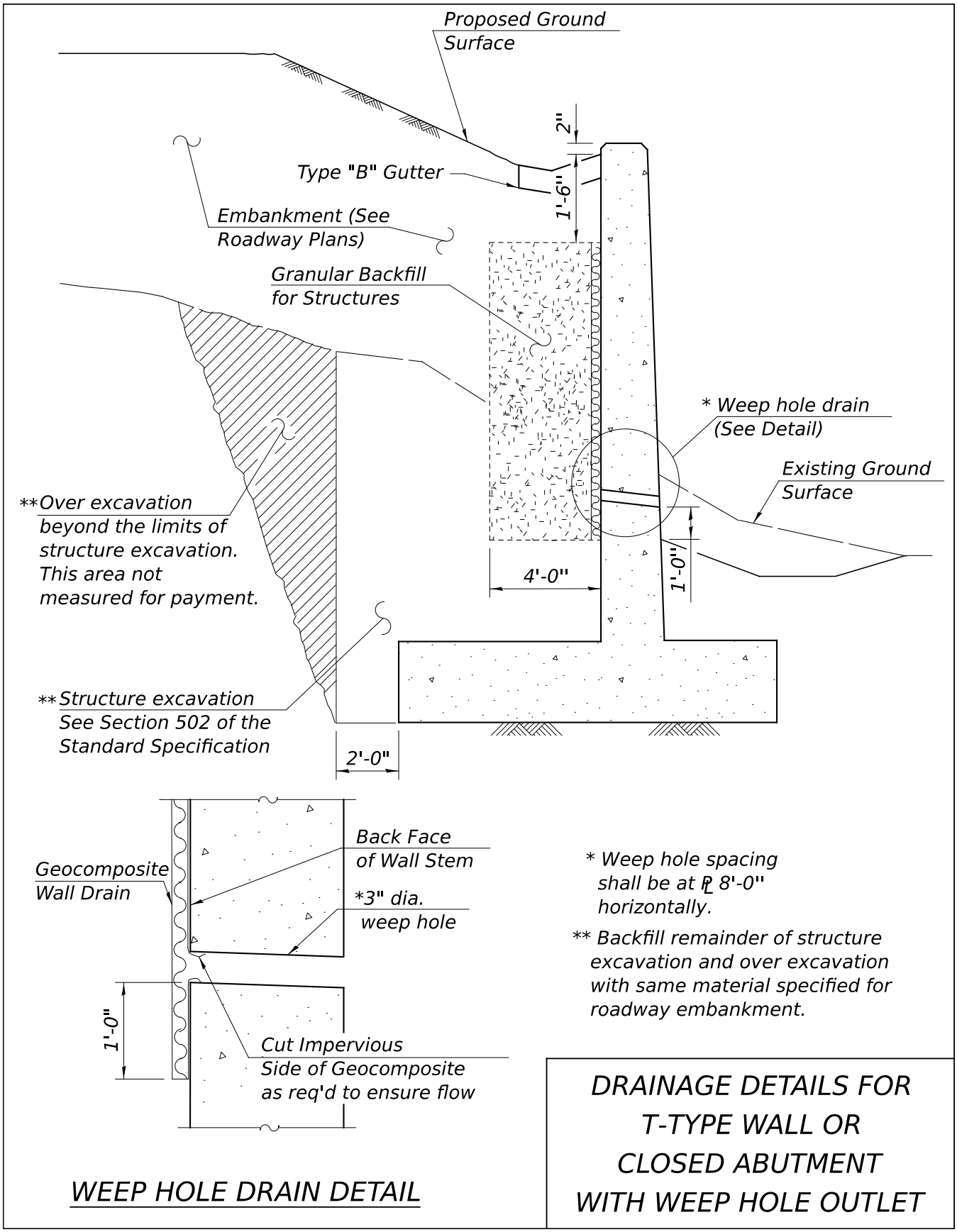


Figure 3.11.2.3-1

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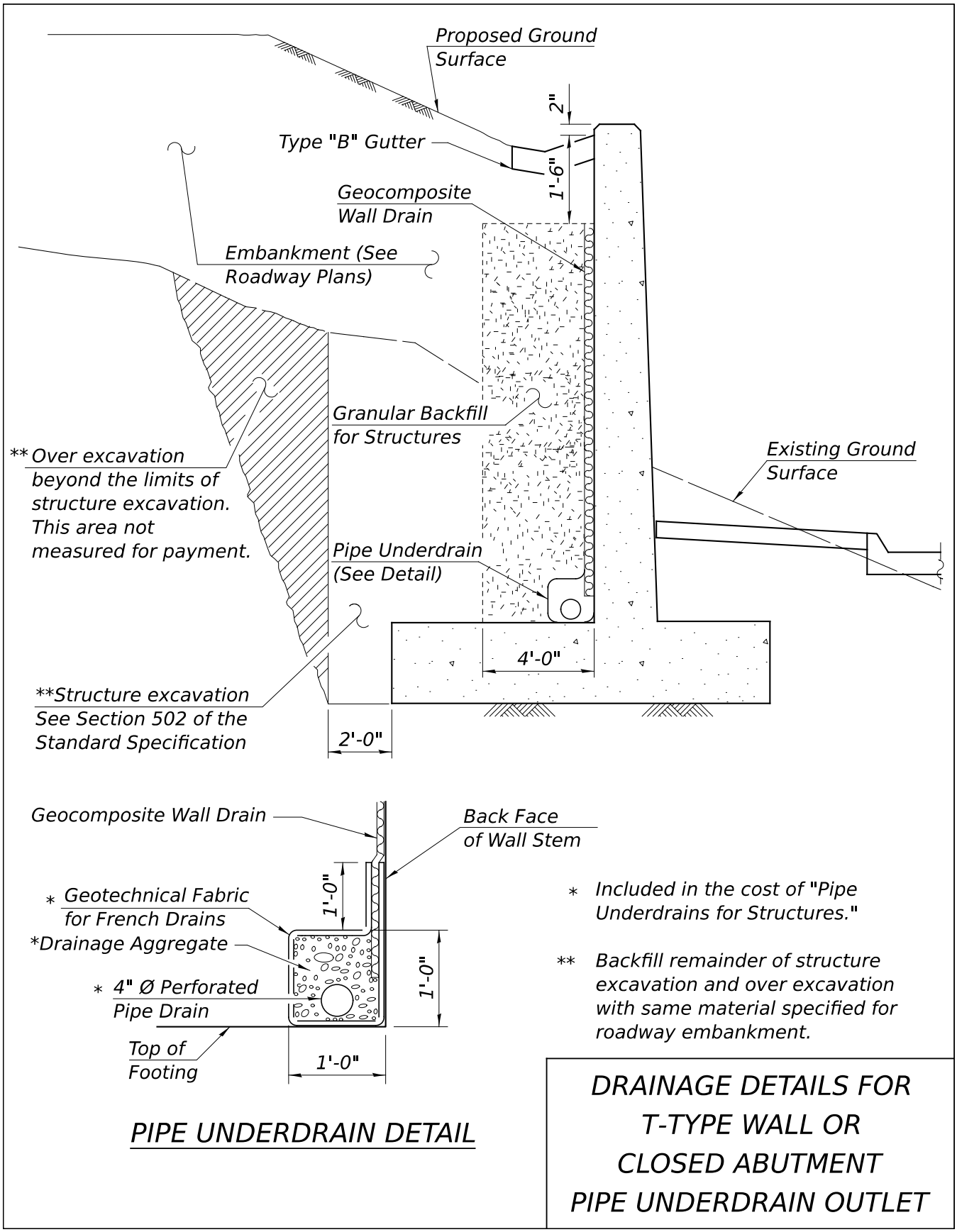


Figure 3.11.2.3-2

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3.11.3 Soldier Pile

Soldier pile walls are well suited for “cut wall” situations. Piles are installed in existing ground and, as soil is cut away in front of the wall, lagging between the piles is successively installed lower and lower until the final cut elevation is achieved. This method of construction allows for continuous undisturbed support of the ground and infrastructure immediately behind the wall. Soldier pile walls are also used in “fill” applications but this is not as common as for cut situations.

3.11.3.1 Driven or Drilled

Piles for soldier pile walls can either be driven to required tip elevations or placed in drilled holes and encased in concrete (“drilled piles”). Encasement normally uses full strength concrete from the pile tip to the bottom of the facing and controlled low strength material (CLSM) to the existing ground surface. Driven piles are limited to H sections while drilled piles can be larger W shapes, built-up plate sections or multiple W sections for longer cantilevered heights or to accommodate permanent ground anchor connections. Drilled piles can also provide larger soil bearing areas which allow greater spacings between piles, shorter embedment lengths, and possibly smaller sections. Drilled piles may also provide better corrosion protection. When the lagging and soldier piles are exposed permanently, the vertical plumb of the piles becomes more critical and can be better controlled when drilled as opposed to driven.

The drilled hole diameter for a soldier pile is determined by calculating the diagonal distance across the pile, adding 2 in. of clearance on each side and rounding up to the nearest standard auger size (18”, 24”, 30”, etc.).

The design of soldier pile walls shall follow the AASHTO LRFD Bridge Design Specifications. The surface of a soldier pile is assumed to bear laterally on a width of soil 3 times its drilled diameter or 3 times its flange width when driven. This distance, however, shall not be greater than the center-to-center distance between piles. Coulomb’s earth pressure coefficients should be used with the wall friction on the lagging assumed to be zero. For the portions of soldier piles below the wall facing, Coulomb’s earth pressure coefficients should be used. The first 3 ft. of passive pressure is normally neglected due to soil disturbance caused by installation of drainage measures and/or fascia, possible future changes in grade or utilities installation, and potential loss of soil strength due to freeze-thaw cycles.

Soldier piles are typically spaced at 8 ft. centers. However, spacings up to 14 ft. can be employed in wall portions with short retained heights. For special applications, e.g. where bending or deflection requirements are significant, drilled piles can be immediately adjacent to one another in order to maximize section modulus per ft. of wall.

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Soldier piles shall be sized to address strength and deflection requirements of the site, assuming a $\frac{1}{32}$ in. corrosion loss over the entire surface area for a 50 yr. design life. Deflection often controls the pile size in walls with taller exposed heights, soft foundation soils, or walls retaining settlement-sensitive structures. See the SGR for detailed geotechnical evaluations and information as well as the IDOT Geotechnical Manual, found in the Foundations and Geotechnical section of the primary Bridges & Structures Page, for further guidance.

3.11.3.2 Facing Alternatives

Facing alternatives for soldier pile walls include cast-in-place and precast concrete, and treated timber. The lagging is designed as a simple span between soldier piles for the earth pressure loading. The following subsections present some design details and discussions for the three primary facings used by the Department.

3.11.3.2.1 CIP Facing

Figures 3.11.3.2.1-1 and 3.11.3.2.1-2 present cast-in-place fascia soldier pile wall details for driven and drilled piles respectively. CIP facing requires temporary untreated timber lagging be placed with ¾ in. gaps between adjacent timbers to allow for ground water to enter the geocomposite wall drain attached to the lagging's outside face. Since the timber lagging is considered temporary, it is not required to support the full design earth pressure. It is the contractor's responsibility to size the timber lagging and, as such, the note below shall be included on the Final plans.

> *The Contractor is responsible for the design and performance of the lagging using no less than a 3 in. nominal rough-sawn thickness and timber with a minimum allowable bending stress of 1000 psi.*

The note is intended to require the contractor to evaluate the cut conditions and design the lagging to perform without excessive deflection or cracking. The use of CIP facing allows some variation in rear face alignment due to out of alignment soldier piles, pile deflections and lagging deflections.

The CIP facing design normally utilizes a 12 in. thick section. The reinforcement shall be designed for strength and serviceability. CIP concrete facing is connected to the soldier piles via ¾ in. welded studs. The length and spacing of the studs are determined by the geometry of the wall, and vertical and horizontal loadings. Typical plan sections of soldier pile walls with cast-in-place facing, which includes

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details for the welded studs, are presented in Figures 3.11.3.2.1-3 (showing single soldier piles) and 3.11.3.2.1-4 (showing immediately adjacent soldier piles to which tie backs can be attached). Figure 3.11.3.2.1-5 illustrates expansion and construction joint details for CIP facing. These joints should be placed at locations clear of soldier pile flanges. Clear cover shall be taken as the minimum dimension from the closest reinforcement to either (a) the geocomposite wall drain, or (b) the outside of the flange of the soldier pile. Since geocomposite wall drain thickness is unknown during design, it may be desirable to increase the clear cover in order to avoid inadequate cover should the Contractor choose a thicker geocomposite wall drain than that shown in design.

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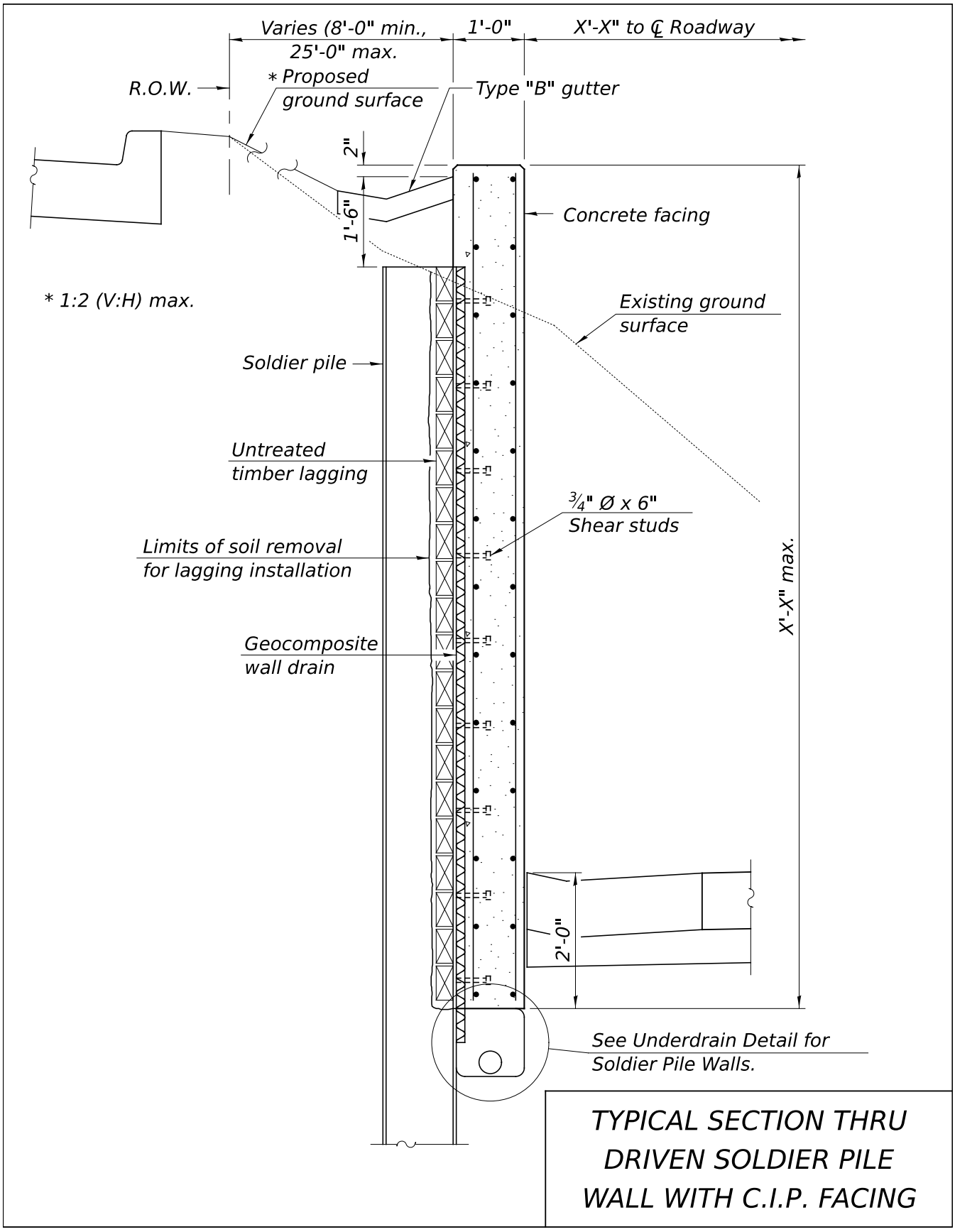


Figure 3.11.3.2.1-1

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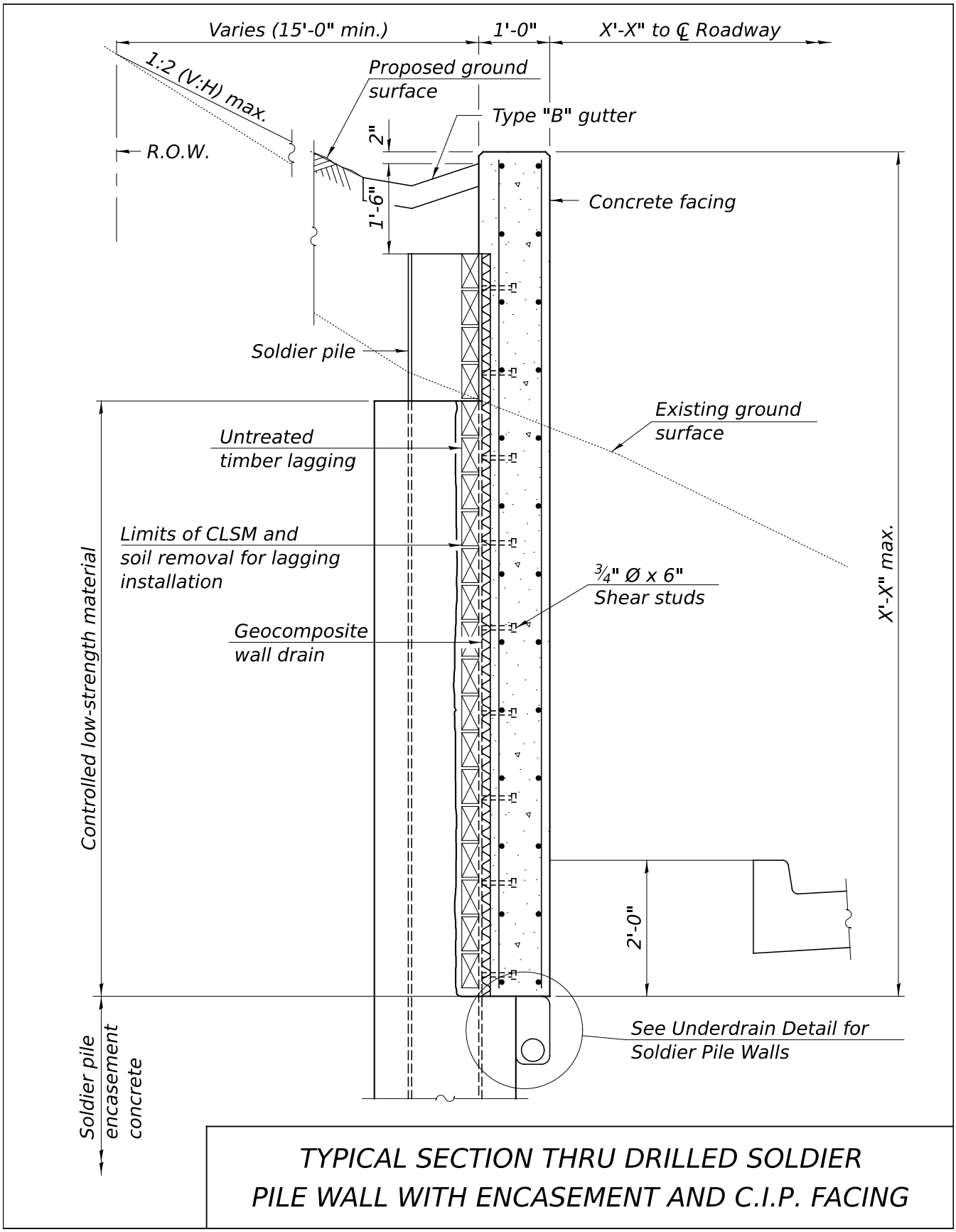


Figure 3.11.3.2.1-2

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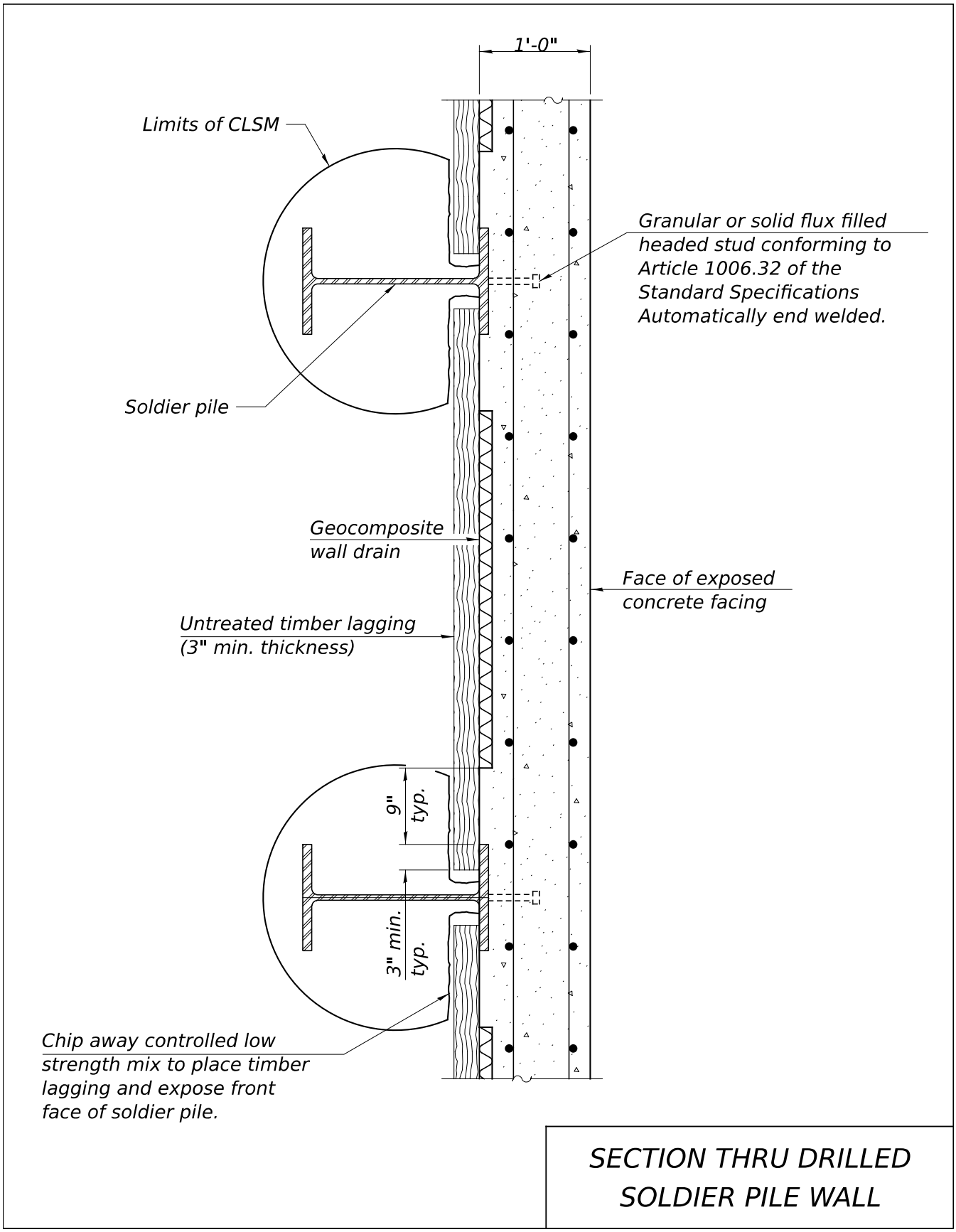


Figure 3.11.3.2.1-3

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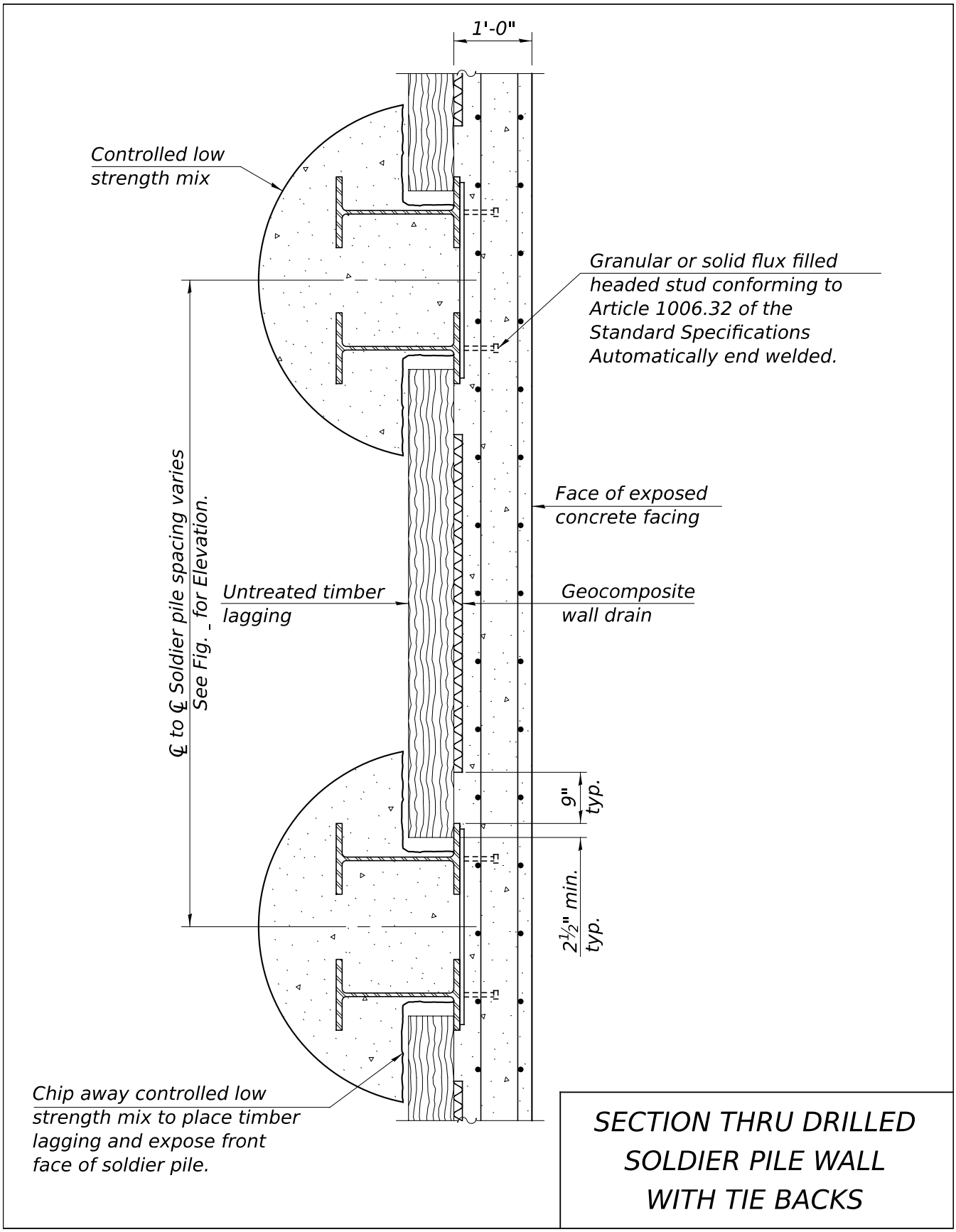


Figure 3.11.3.2.1-4

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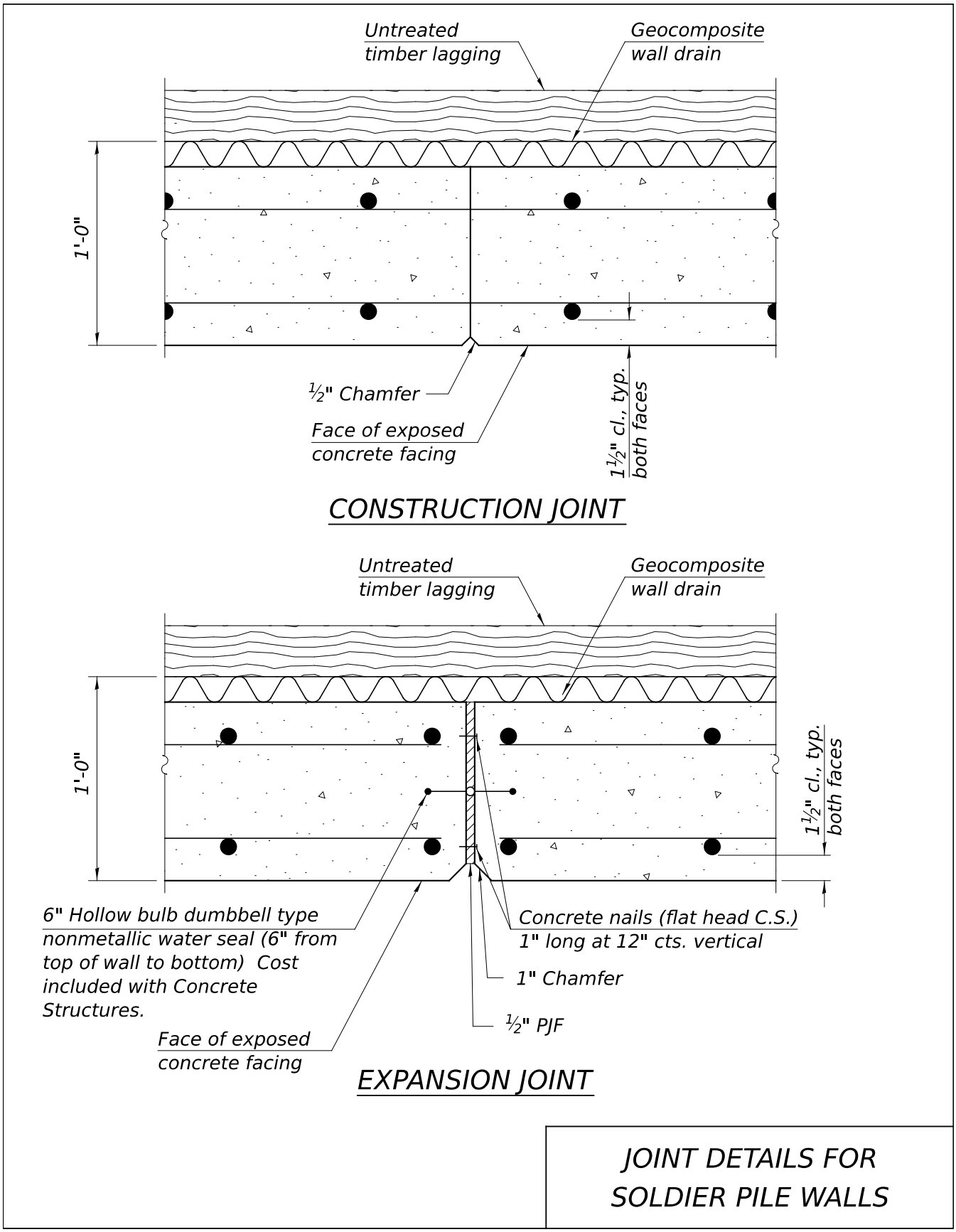


Figure 3.11.3.2.1-5

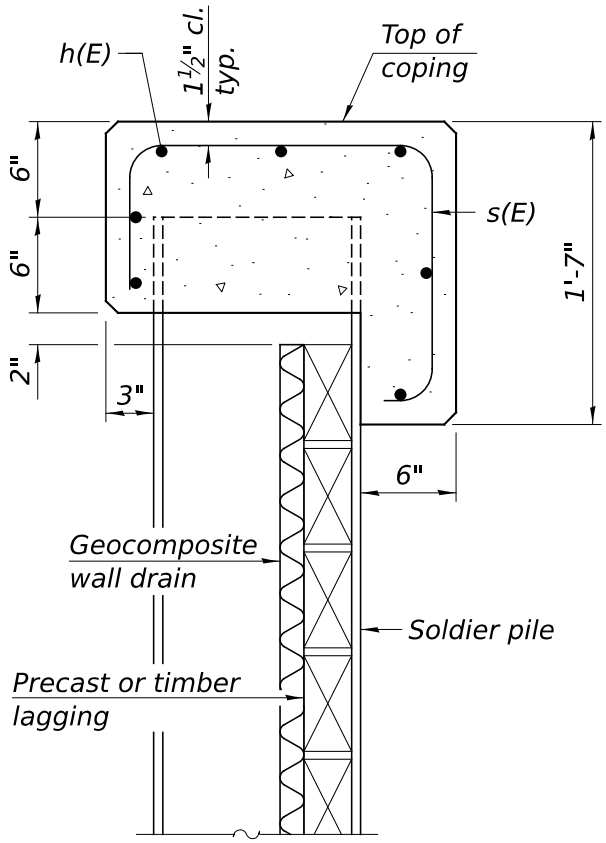
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3.11.3.2.2 Treated Timber

Exposed treated timber is normally used at locations in which the face is not typically viewed by the public or in bucolic settings. As is implied by the name, the timbers span between soldier piles and remain exposed. A CIP concrete cap or steel channel section is typically provided for treated timber soldier pile walls to cover the top of the lagging as well as the tops of the piles as shown in Figure 3.11.3.2.2-1 (which also notes that precast lagging may be utilized).

The plans shall note that the lagging is required to conform to Articles 507 and 1007.03 of the Standard Specifications. The timber shall be southern pine, Douglas fir (coast region), or other species listed in the AASHTO LRFD Bridge Design Specifications. Minimum bending strength (F_b) of the timber shall be shown on the plans and is normally specified at or above 1600 psi. Considering the lagging span, soil arching and other timber design factors, the required lagging thickness can be determined and specified.

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CONCRETE CAP SECTION FOR SOLDIER PILE WALL WITH EXPOSED LAGGING

Figure 3.11.3.2.2-1

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The following notes should be added to the Contract plans when exposed treated timber is used for a soldier pile wall.

> *The treated timber lagging shall be according to Article 507 and 1007 of the Standard Specifications.*
>
> *The drain shall be placed behind the lagging with the pervious side toward the soil according to Section 591 of the Standard Specifications and shall extend to within 6 in. of either side soldier pile flange. The drain shall be installed in stages as the excavation proceeds downward making sure that drain splices as well as the top side edges are covered as required to protect the drain core from soil intrusion.*

3.11.3.2.3 Precast Facing

Precast lagging and precast panels are used occasionally on Departmental projects. However, the expense of casting, shipping, handling and installation can be excessive for most locations in Illinois.

3.11.3.3 Anchorage Slab, Section Thru Parapet and Railing

Figure 3.11.3.3-1 presents a typical wall section for a soldier pile wall with CIP facing and an "independent" barrier or parapet with an anchorage slab. Figure 3.11.3.3-2 gives the details of the anchorage slab and parapet along with the interface between the wall and the anchorage slab.

The maximum width of the anchorage slabs shall be the width of the shoulder. The slab may be narrower than the shoulder if structurally adequate. Anchorage slabs shall not be used on approach slabs.

Figure 3.11.3.3-3 presents a typical wall section for a soldier pile wall with CIP facing and a "dependent" barrier or parapet. Note the absence of an anchorage slab in the figure. A typical wall section for a soldier pile wall with CIP facing and an attached rail is given in Figure 3.11.3.3-4.

See Section 3.11.1.3 for other related design criteria which includes traffic impact. Note that the "crash worthiness" of the details shown in Figures 3.11.3.3-3 and 3.11.3.3-4 may be inadequate for many situations without modification.

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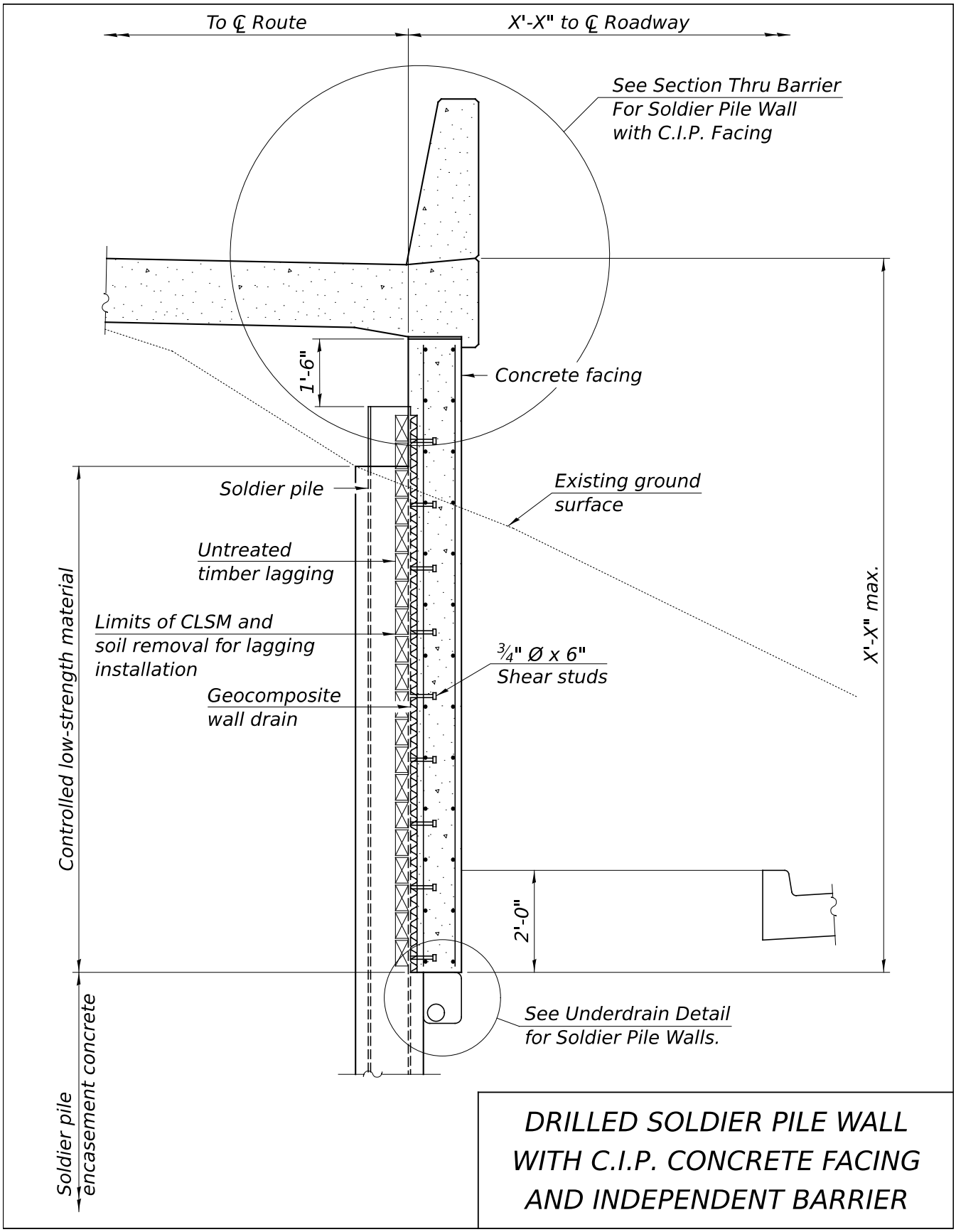


Figure 3.11.3.3-1

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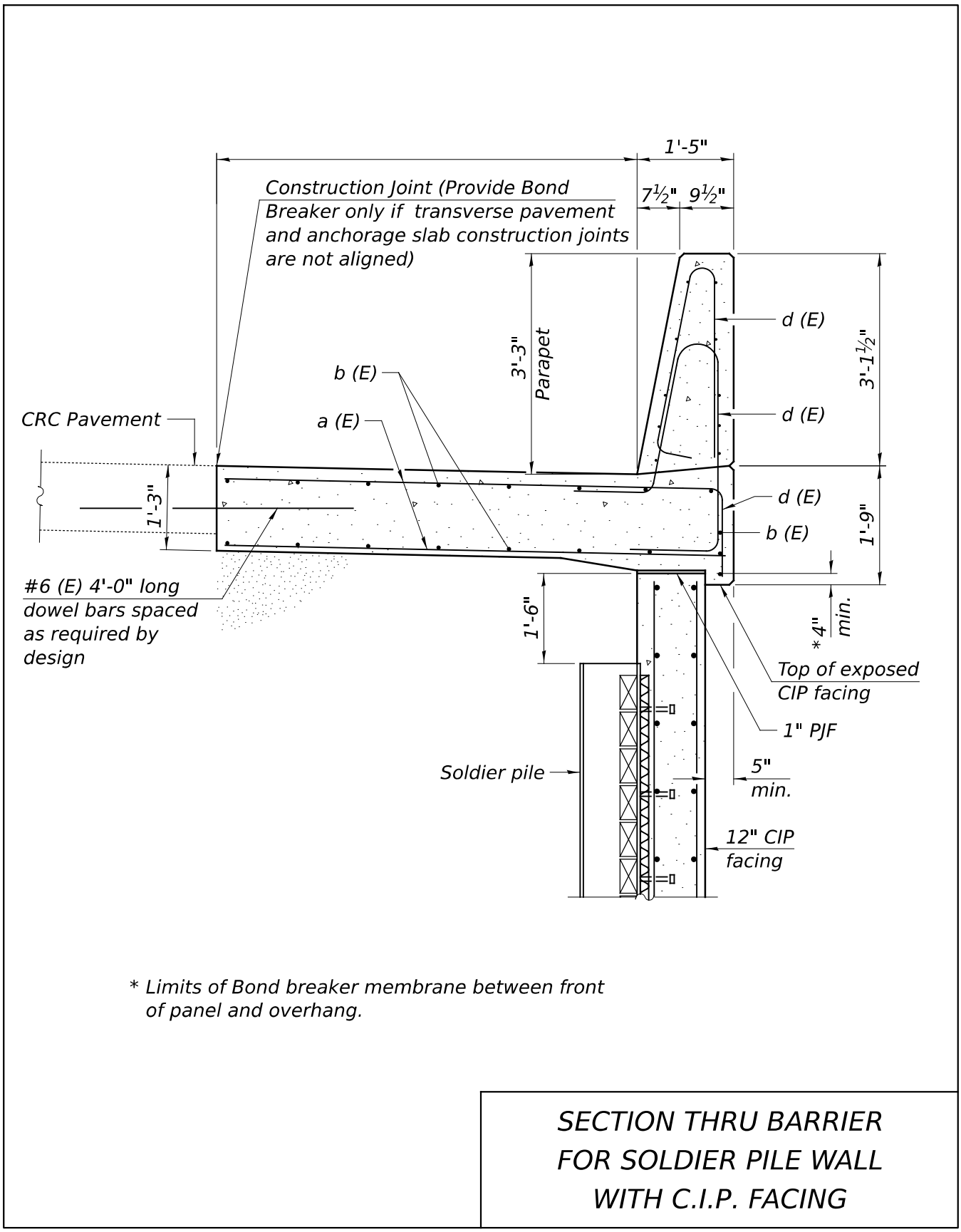


Figure 3.11.3.3-2

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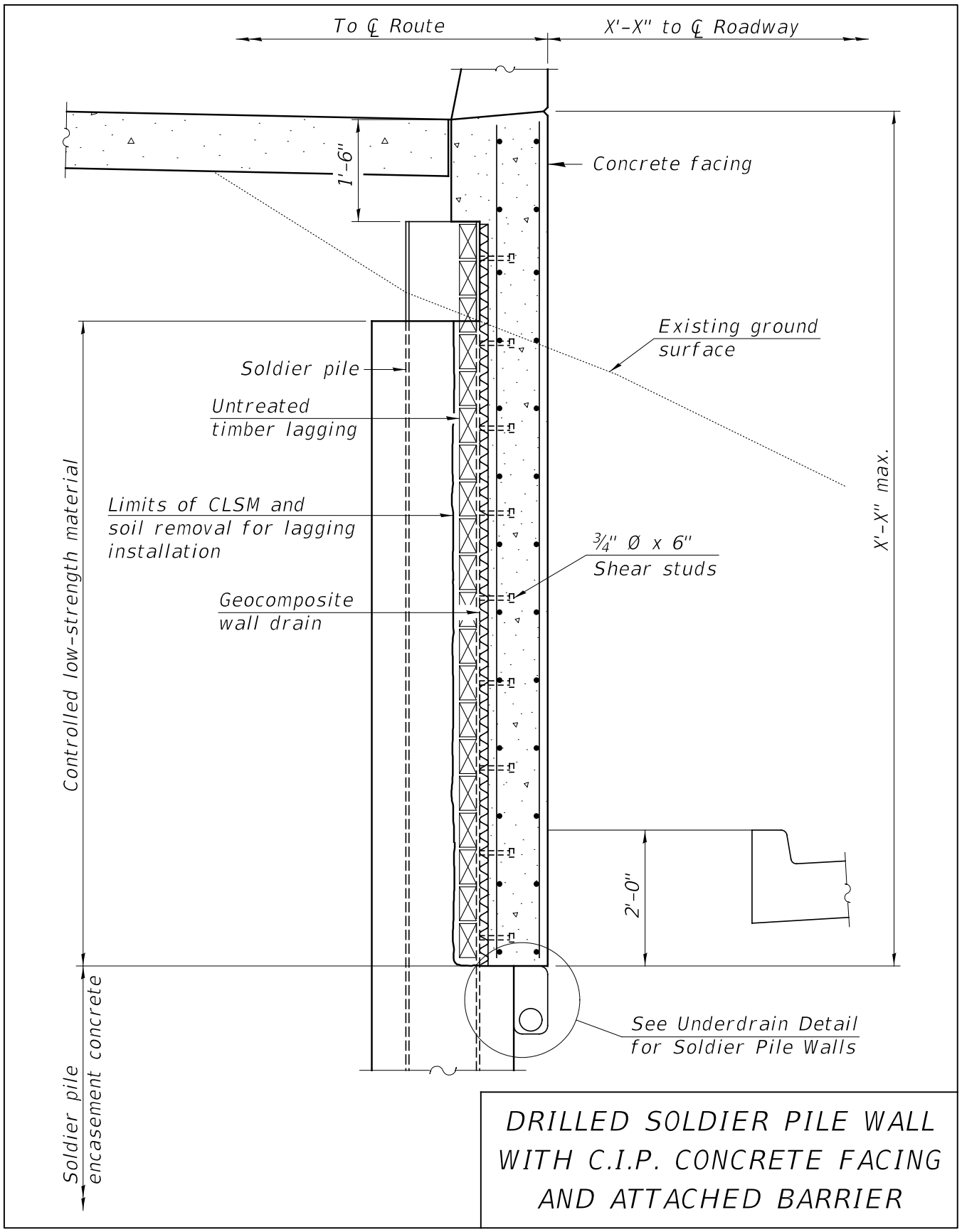


Figure 3.11.3.3-3

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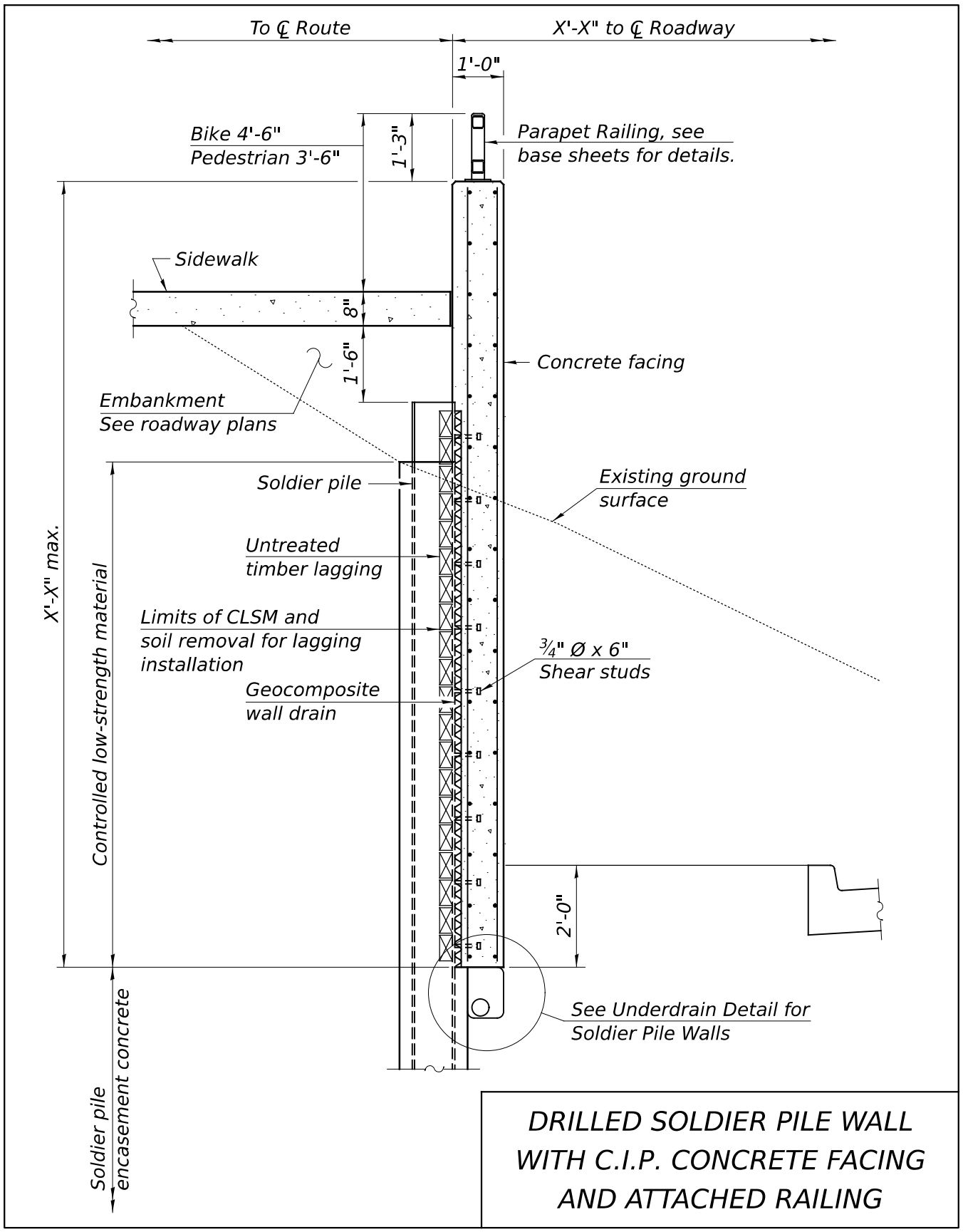


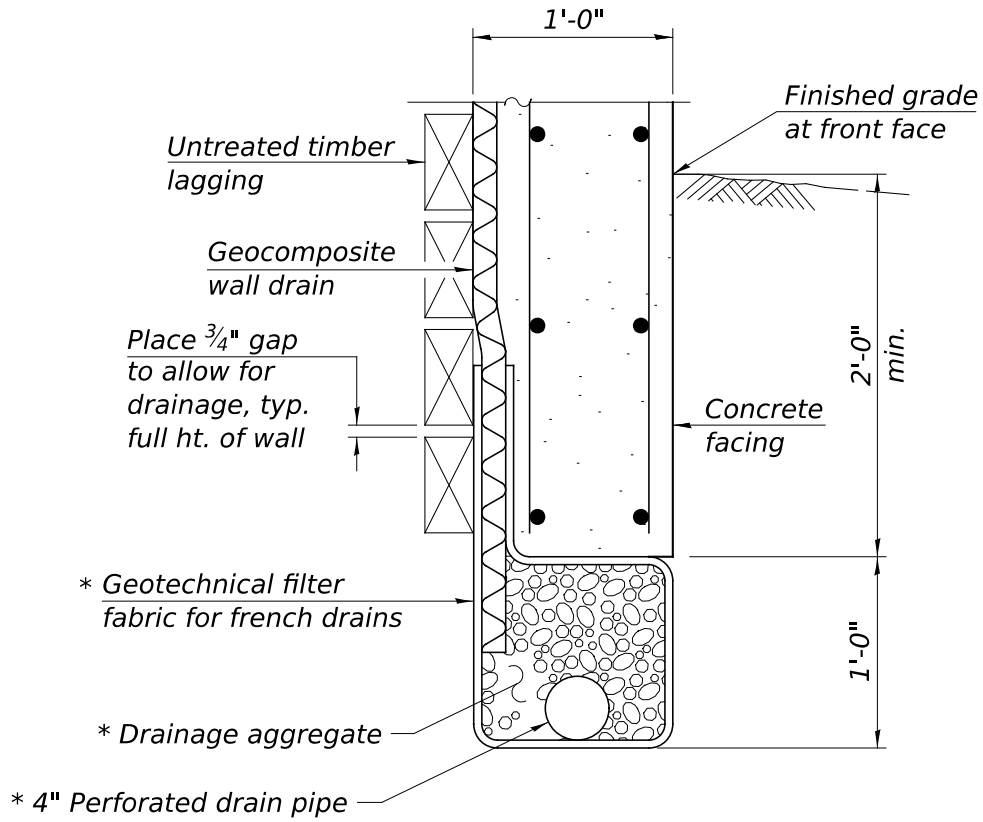
Figure 3.11.3.3-4

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3.11.3.4 Drainage

Some typical underdrain details for soldier piles walls are presented in Figures 3.11.3.4-1 through 3.11.3.4-2.

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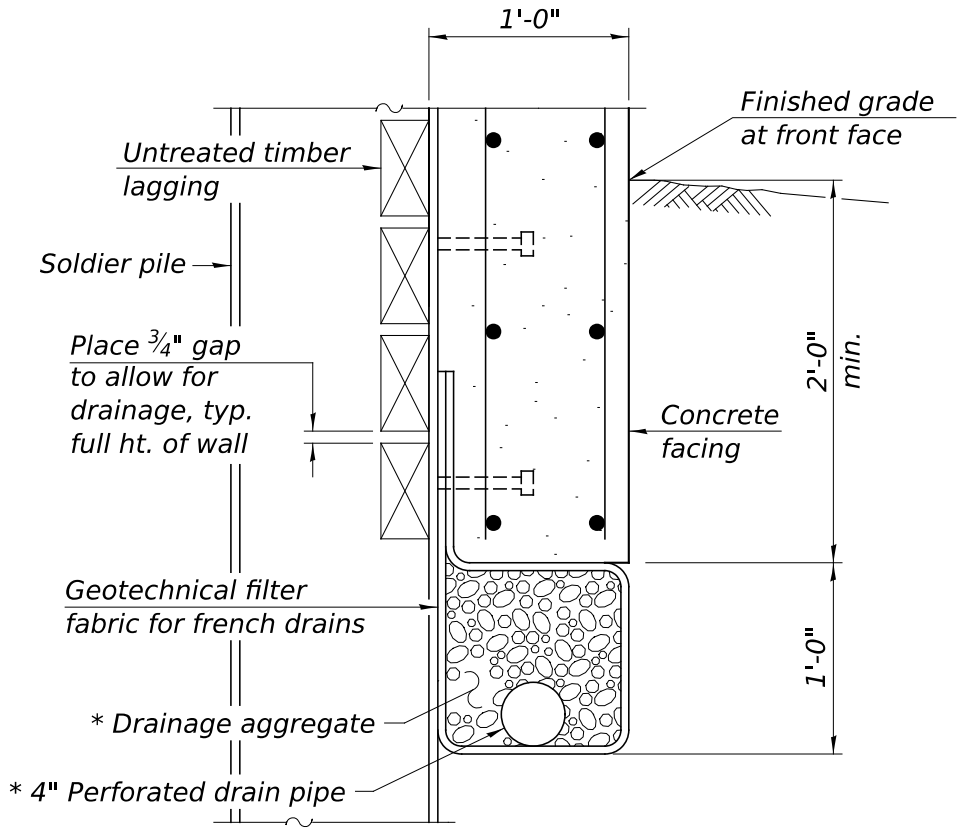
BETWEEN SOLDIER PILES

* Included in the cost of Pipe Underdrains for Structures.

UNDERDRAIN DETAIL FOR SOLDIER PILE WALLS

Figure 3.11.3.4-1

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AT SOLDIER PILES

* Included in the cost of Pipe Underdrains for Structures.

UNDERDRAIN DETAIL FOR SOLDIER PILE WALLS

Figure 3.11.3.4-2

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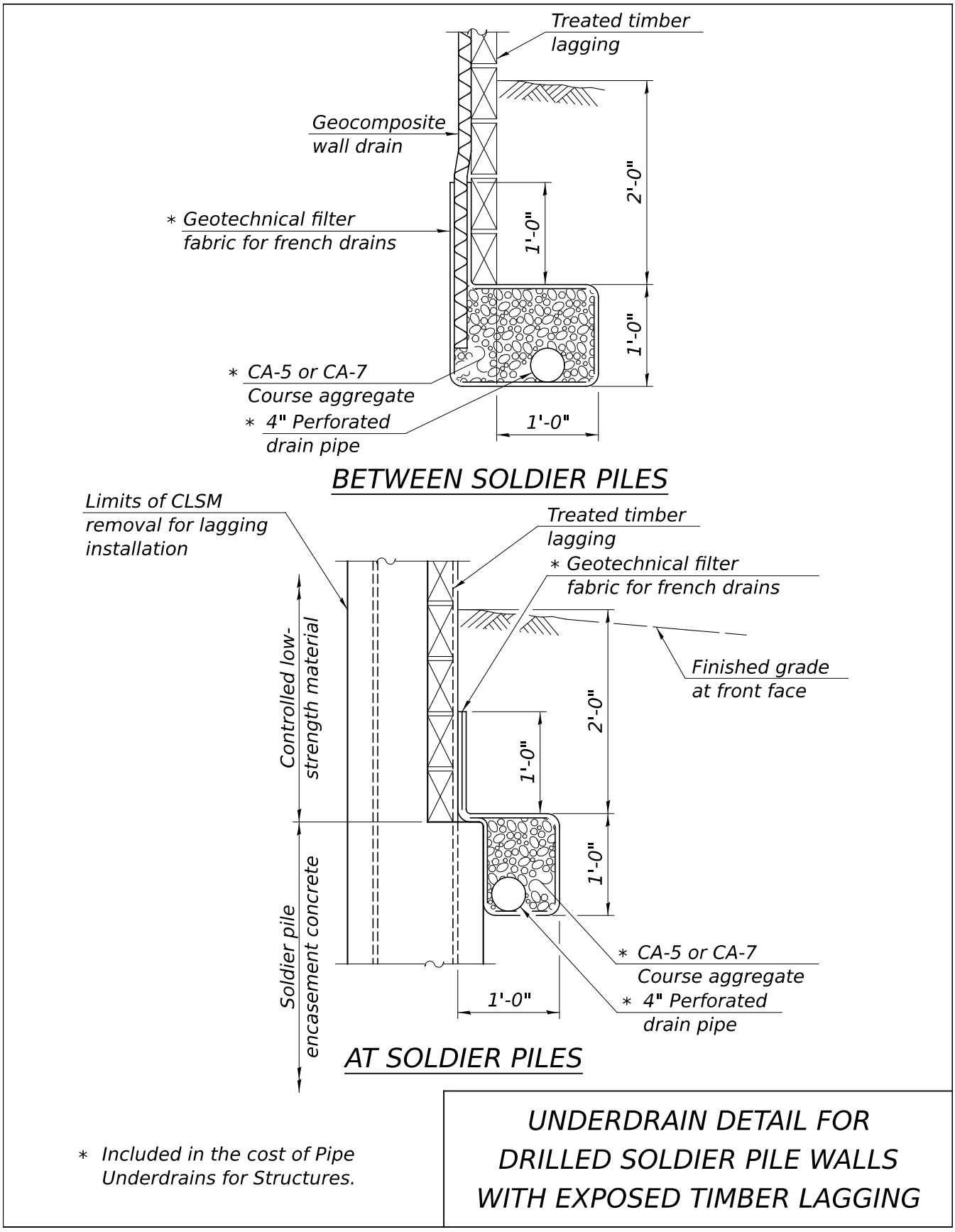


Figure 3.11.3.4-3

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3.11.4 Permanent Sheet Pile

Permanent cantilevered sheet pile retaining walls shall be designed for a 50 year design life unless they are traffic bearing or considered critical, in which case a 75 year design life shall be used.

Design considerations for this wall type include specifying section modulus, drivability of piles, deflection at the top of the wall, corrosion of the piling, and hydrostatic pressure on the wall. The SGR shall be referenced for recommendations and evaluations concerning the geotechnical aspects of the design. The geotechnical engineer providing the SGR should be consulted by the wall designer for assistance when needed.

The design of sheet piling is similar to soldier piles and shall follow the AASHTO LRFD Bridge Design Specifications which requires the tip elevation and section modulus be established using Coulomb's earth pressure coefficients and includes wall friction. The first 3 ft. of passive pressure is normally neglected due to soil disturbance caused by installation of drainage measures and/or facing, possible future changes in grade or utilities installation, and potential loss of soil strength due to freeze-thaw cycles.

Grade 50 steel (AASHTO M202) shall be considered the default material for sheet piling and shall be used when computing a preliminary minimum design section modulus.

Local buckling effects related to transverse stresses shall only be accounted for when the applied lateral pressure at point of maximum moment is more than 1000 psf. In such cases a reduction factor ρ_p shall be used to reduce the yield strength (F_y).

$$F_{yr} = \rho_p F_y$$

Where:

F_{yr} = reduced yield strength of steel (ksi)

ρ_p = reduction factor for local buckling effects related to transverse stresses, found in Table 3.11.4.1. Note: if the interlocks of the sheet piles are welded, ρ_p may be taken as 1.0.

F_y = yield strength of steel (ksi)

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D_P	$(b/t_{min})^* \varepsilon = 20.0$	$(b/t_{min})^* \varepsilon = 30.0$	$(b/t_{min})^* \varepsilon = 40.0$	$(b/t_{min})^* \varepsilon = 50.0$
1000	1.00	1.00	1.00	1.00
2000	0.99	0.97	0.95	0.87
3000	0.98	0.96	0.92	0.76
4000	0.98	0.94	0.88	0.60

Table 3.11.4.1

Where:

D_P = applied lateral pressure at the point of maximum moment (psf)

b = flange width (in.), but not less than 0.707c

c = slant height of web (in.)

t_{min} = minimum of t_f and t_w (in.)

t_f = flange thickness (in.)

t_w = web thickness (in.)

$\varepsilon = \sqrt{\frac{34}{F_y}}$

F_y = yield strength of steel (ksi)

Note: Intermediate values may be interpolated linearly.

Since using Table 3.11.4.1 requires information based on the specific sheet pile section to be used, which will not be determined until the contractor chooses a section, the designer shall assume the pile with the largest (b / t_{min}) that meets the minimum section modulus required for strength design.

The preliminary minimum section modulus (S_x) shall be calculated using F_{yr}, and shall be increased to account for applicable factors such as corrosion, driving stresses, and deflection controls.

Corrosion shall be accounted for by the designer using the minimum design section modulus (S_x) and the desired design life. Since each sheet has a slightly different corrosion reduction, the Department developed the following formulas to estimate the extra section modulus required to account for corrosion.

a.) For a 50 yr. design life, S_{req} (in^3/ft of wall)= $S_x(1.10) + 2.2$

b.) For a 75 yr. design life, S_{req} (in^3/ft of wall)= $S_x(1.12) + 4.0$

Deflection control and driving stresses may also control the design of the piles, causing S_{req} to be increased further.

The grade of steel, AASHTO specification, and S_{req} shall be shown on the plans.

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3.11.4.1 Facing Alternatives

Permanent sheet piling is usually installed with either a cast-in-place facing or the piles above ground left exposed. When the sheet pile is left exposed, the top of the sheets are typically encased with a concrete cap. In some cases, the sheeting can be painted or capped with a steel cap.

3.11.5 Wall Anchorage Systems

Wall anchorage systems are often used in combination with soldier pile and sheet pile walls to reduce overall wall cost, limit deflection and provide additional capacity to resist large horizontal design forces. However, they have been used to stabilize failing CIP T-type walls, as tie-down anchorage, and to address other specialized design needs.

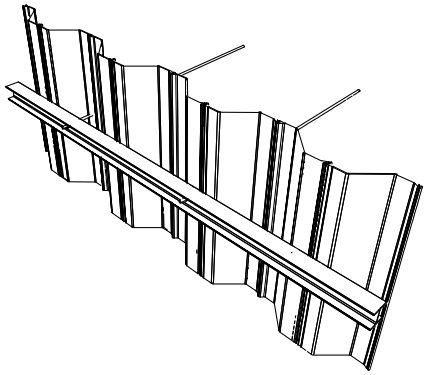
The Structure Geotechnical Report (SGR) shall be referenced for recommendations and evaluations concerning the geotechnical aspects of the design of wall anchorage systems. The geotechnical engineer providing the SGR should be consulted by the designer for assistance when needed. The IDOT Geotechnical Manual, found in the Foundations and Geotechnical section of the Bridges & Structures page, can also be referenced for more information and guidance on wall anchorage systems.

The following sub-sections give a brief overview of three common anchorage systems which are use by the Department. These are deadman, helical ground anchors and permanent ground anchors. A subsection with generic details for an anchor head assembly is also presented.

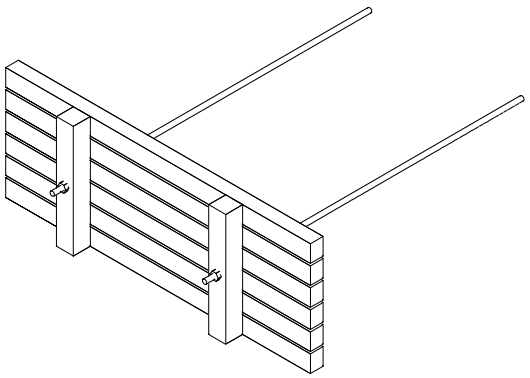
3.11.5.1 Deadman

Deadman anchors are low tech, fairly inexpensive and generally suited for low capacity applications requiring 10 to 60 kips of resistance. Galvanized and/or oversized rods are generally used to account for corrosion when connecting deadmen to the wall system. In most cases, the anchors are not load tested and only minor tie rod loading is developed as the rods are undergoing installation. Deadman anchors can consist of reinforced concrete which is cast in forms or in trenched excavations, or precast units can be used. Other deadmen types used by the Department include sheet pile, driven H-pile, treated timber lagging, or drilled shafts as shown in Figure 3.11.5.1-1. The design involves calculating the wall anchorage requirements in kips/ft., selecting an appropriate tie rod, and determining an appropriate deadman spacing. Although a continuous deadman can be specified, it is normally more cost effective to employ individual deadman elements which can be designed to utilize the passive pressure adjacent to each element.

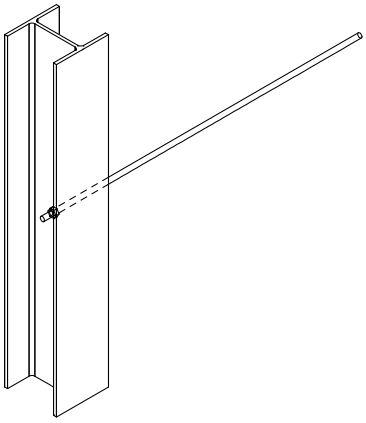
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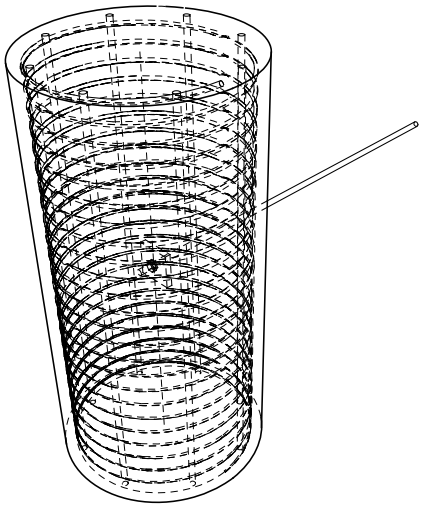
SHEET PILING



TIMBER



DRIVEN H-PILE



DRILLED SHAFT

DEADMAN
ANCHOR TYPES

Figure 3.11.5.1-1

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3.11.5.2 Helical Ground Anchors

Helical anchors have medium capacity, usually between 60 and 200 kips of nominal resistance. They can be installed at an angle or horizontally. Helical anchors are typically galvanized and oversized to address corrosion. The IDOT Special Provision requires selected anchors to be proof tested and the remaining anchors to develop sufficient installation resistance. In most applications, the anchors are not post tensioned and simply connected to walls in a manner which is similar to that for deadmen. The wall designer shall determine the most cost effective anchor spacing and specify the nominal loading on the plans. The anchor's angle of inclination, minimum extension length, and the estimated helical anchorage length shall also be specified. It is critical that the soil conditions are not too stiff or dense to prevent helical installation, and yet strong enough to allow the supplier to develop a feasible anchor design. The helical anchor supplier shall submit Shop Drawings proposing the number, diameter and embedment of the helical anchors which will be reviewed by the BBS Foundations and Geotechnical and Design Units. Various helical anchor contractors and vendors should be contacted to provide suggestions on both design/installation feasibility as well as comment on the Final Plan details.

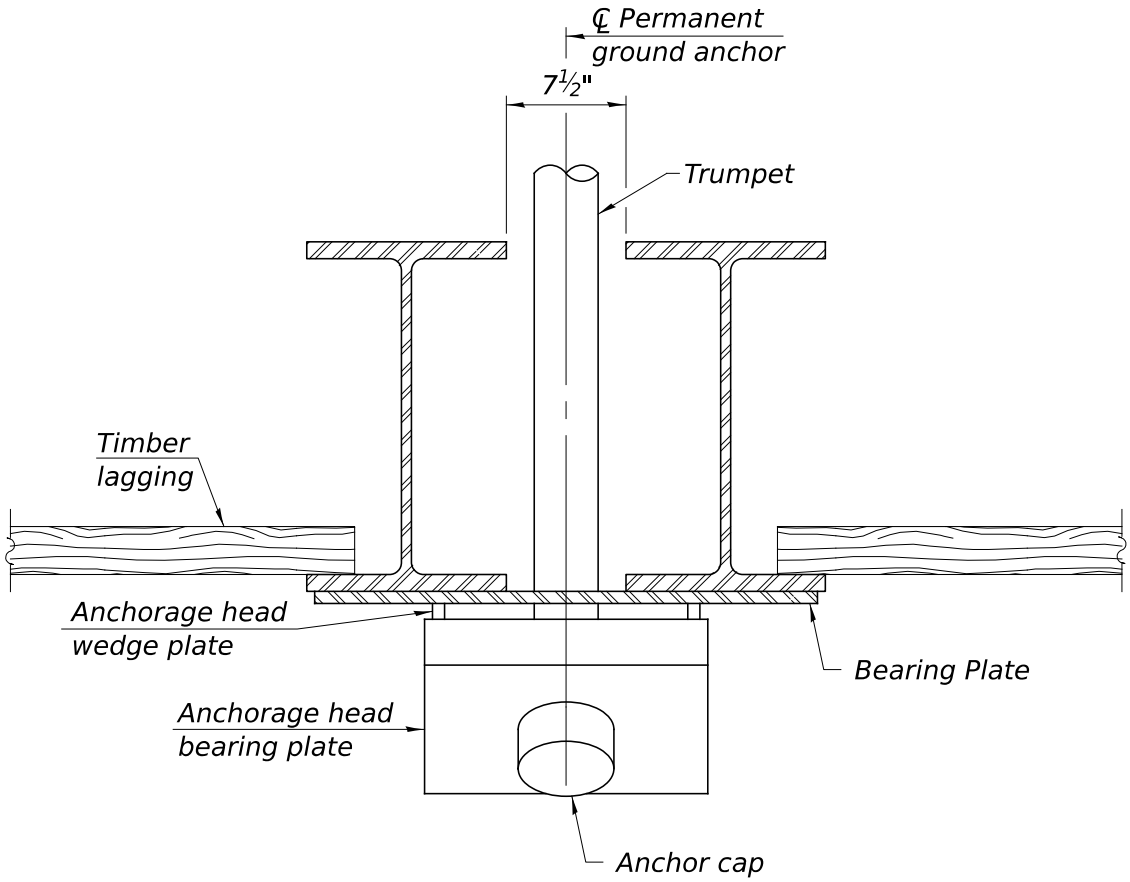
3.11.5.3 Permanent Ground Anchors

Permanent ground anchors are typically used for high capacity applications and designed to have a nominal resistance between 100 and 300 kips. They are double corrosion protected and proof tested to 1.33 times the specified nominal design loading. The designer shall also specify the anchor's angle of inclination, the minimum unbonded length, and the estimated bonded length. The specialty anchor sub-contractor shall submit their qualifications and experience during construction, and supply calculations and Shop Drawings indicating the installation method, drilled diameter, and bonded length per the Special Provision for Departmental approval.

3.11.5.4 Anchor Head Assemblies

Figures 3.11.5.4-1 and 3.11.5.4-2 present generic details for an anchor head assembly used on soldier pile walls with exposed timber lagging.

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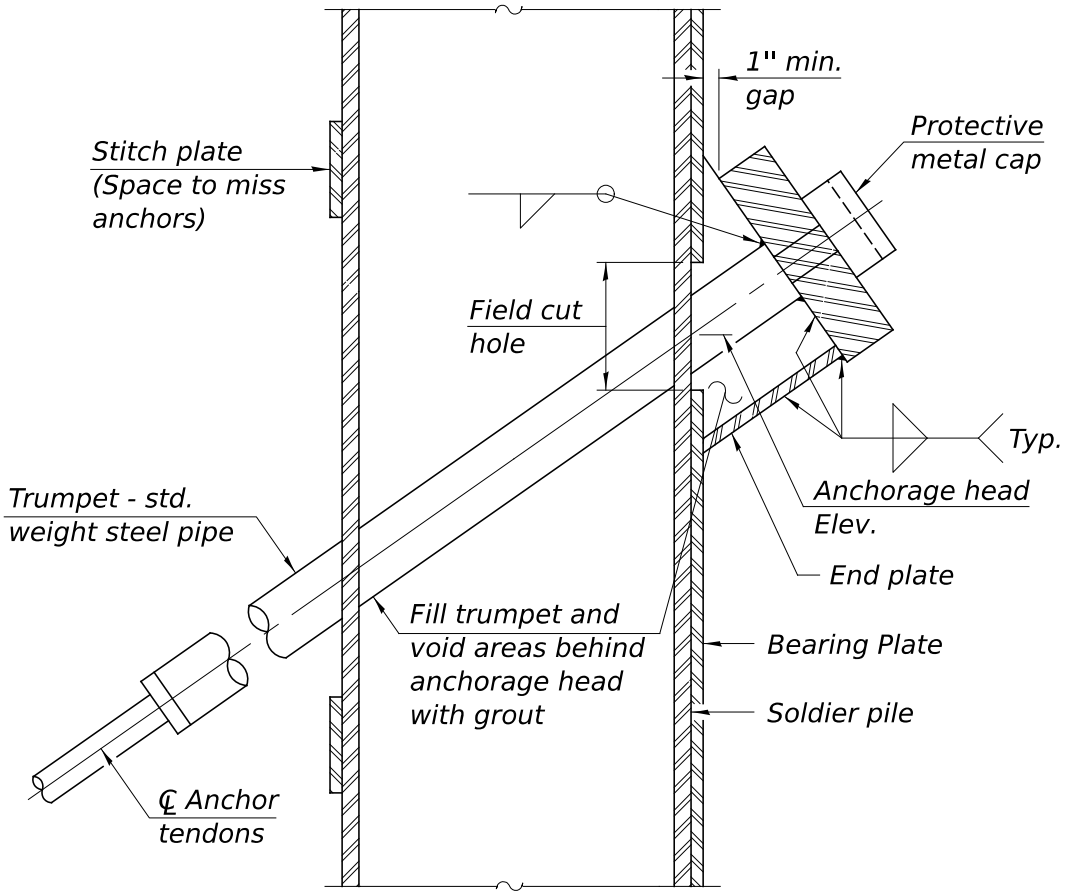


ANCHORAGE PLAN

ANCHORAGE HEAD ASSEMBLY

Figure 3.11.5.4-1

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SECTION THRU ANCHORAGE HEAD ASSEMBLY

Figure 3.11.5.4-2

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3.11.6 Soil Nailed and Other Specialized Wall Systems

Soil nailed walls are occasionally built in Illinois. They can be a viable option in low head room conditions. For this wall type, there should be enough right-of-way to install the nails to the full required length. Soil nailed walls should be avoided in granular soils or for installation below the water table. If a soil nailed or other specialized wall system is contemplated for a project, contact the BBS for concurrence and coordination.

Segmental Concrete block walls are utilized by the Department in a number of locations where their aesthetics are important and low initial cost is desirable. The Department has concerns with the dry-cast block freeze thaw durability and estimates the average design life at 30 years. Applications where the wall would be subjected to prolonged saturation such as around lakes or streams may shorten the design life even further. Block walls may be designed as gravity walls in lower retention applications, or may require soil reinforcement for taller walls. Generally, it is recommended that these walls not be utilized in critical applications, where a longer design life is required, or when exposure to excessive road salt applications is anticipated. This wall type is designed and contracted similar to MSE walls. The foundation soils should be evaluated to confirm design feasibility and adequate long term performance. Block walls are commonly used in retention applications requiring less than 7 ft. of exposed height. If unique conditions exist where taller wall heights or tiered wall configurations are desired, the District and BBS will provide site specific approval prior to TSL development. Section 522 of the Standard Specifications covers Segmental Concrete Block Walls.

The Department also makes use of gabion retaining walls, precast modular wall systems and other unique wall systems which lend themselves to site and project design constraints. Specifications, plan details, and other information may be available from the Department concerning these wall systems upon request.

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3.12 Noise Abatement Walls

This section covers the Department's policies related to the design, detailing, and submittal procedure requirements for ground-mounted and structure-mounted noise abatement walls. Detailing that meets the material and pre-qualification requirements of the Department comprise the standard basis for design.

Reference is made to Chapter 26 of the Bureau of Design & Environment (BDE) Manual and the IDOT Highway Traffic Noise Assessment Manual. The policies in the BDE Manual regarding noise analysis, wall placement location, wall materials, etc., should be examined during Phase I in advance of application of the policies herein. The following provides guidance on structural plan details, not to circumvent any policies in the Bureau of Design & Environment Manual or the IDOT Highway Traffic Noise Assessment Manual.

Noise abatement wall plans are evaluated by the Bureau of Bridges & Structures in two phases: the Phase II Design contract plan review phase, and the Phase III Construction/Fabrication shop drawing review phase.

The details and design procedures herein are consistent with precast reinforced concrete noise abatement wall panels with steel posts (ground-mounted or structure-mounted) or concrete posts (ground-mounted only). Other systems and materials are available and should be considered as applicable during the Phase I evaluations for the noise abatement wall.

3.12.1 Noise Abatement Wall Phase II Contract Plan Requirements

Issues involving noise abatement wall design and detailing are more easily resolved during Phase II design contract plan review, prior to letting and bidding. A Phase II submittal of the proposed noise abatement wall plans to the Bureau of Bridges & Structures is therefore required at the pre-final stage prior to letting and bidding, preferably six months in advance of the letting. The Bureau of Bridges & Structures will review the contract plans and special provisions for noise abatement walls that will be state-owned. The local agency owner will review the contract plans and special provisions for noise abatement walls that will be local agency owned.

3.12.1.1 Illinois Licensed Structural Seal Requirements

Depending upon whether the noise abatement wall is ground-mounted or structure-mounted, and the phase of the contract, the following structural seal requirements apply.

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Phase	Plan Type	SE Seal & Signature Required?
Phase II	Ground-Mounted NAW Contract Plans	No
	Structure-Mounted NAW Contract Plans	Yes
Phase III	Ground-Mounted NAW Shop Drawings	Yes
	Structure-Mounted NAW Shop Drawings	Yes

For ground-mounted noise abatement walls, no structural design is provided on the Phase II contract plans. The post, panel, and foundation designs for ground-mounted noise abatement walls are all provided during the Phase III shop drawing submittal. Therefore, an Illinois Licensed Structural Engineer seal and signature are not required for the Phase II contract plans for ground-mounted noise abatement walls but are required for the Phase III shop drawings for ground-mounted noise abatement walls.

For structure-mounted noise abatement walls, the effects of the noise abatement wall on the attached structure (bridge or anchorage slab), are considered part of the structural design of the wall and attached structure. The post sizes and connections shall be verified by the Engineer of Record for the noise abatement wall plans. Therefore, the Phase II contract plans for structure-mounted noise abatement walls require the seal and signature of an Illinois Licensed Structural Engineer.

For structure-mounted noise abatement walls, the Phase III shop drawings contain the structural design of wall elements such as panels, clip angles, etc. Therefore, the Phase III shop drawings for structure-mounted noise abatement walls require the seal and signature of an Illinois Licensed Structural Engineer.

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3.12.1.2 Detail Requirements

The intent of the Phase II contract documents, distributed for letting and bidding, is to ensure:

- adequate data is provided to the Contractor to accurately bid the noise abatement wall pay items, and then construct the noise abatement walls during the construction contract.
- adequate data is provided to the noise abatement wall supplier to design and detail the walls, and to provide shop drawings signed and sealed by an Illinois Licensed Structural Engineer.
- for structure-mounted noise abatement walls, the adequacy of the design details for the noise abatement wall mounted on the bridge or anchorage slab, and the structural adequacy of the bridge or anchorage slab itself.

In order to provide this data to the Contractor and wall supplier, the following details and specifications are required in the contract documents. These are listed using numerical indices below and discussed in more detail thereafter.

1. Design Specifications
2. Design Loading
3. Design Stresses
4. Aesthetic Requirements
5. General Plan and Elevation Views
6. Structural Details
7. Noise Reduction Data
8. Soil Data
9. Name Plate Requirements
10. Pay Items, Methods of Measurement, and Bases of Payment

1. Design Specifications

The most current version of the AASHTO LRFD Bridge Design Specifications shall be shown on the General Plan and Elevation sheet, under ***Design Specifications***. This provides a record of the design specifications for future engineers in the event of required wall repairs.

Structural design of noise abatement walls is governed by Section 15 of the AASHTO LRFD Bridge Design Specifications, titled Sound Barriers. Guide Bridge Special Provisions 101 and 102 alert the noise wall supplier that this code shall be used in design.

An example of specifying the design specifications is shown in Figure 3.12-1.

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2. Design Loading

Design loads shall be shown on the General Plan and Elevation sheet, under *Design Loads*.

Factored loadings shall be taken as 35 psf for Strength III or Strength V wind loading and 15 psf for Service I wind loading. These loads are derived from the wind loading formulas in Chapter 15 of the AASHTO LRFD Bridge Design Specifications, and are based upon Open Country wind exposure and a structure height of 33 ft. Use of the Open Country wind exposure category is sufficiently conservative that these loadings are applicable to noise walls in Urban or Suburban areas with heights far exceeding 33 ft., and recalculation of loads is not required.

For structure-mounted noise abatement walls, unfactored wall dead load shall be taken as 65 pounds per square foot. The unfactored 72 in. concrete barrier dead load is 0.98 kips per linear foot. On bridges, this loading shall be distributed by the slab over the three beams closest to the noise abatement wall. This additional loading shall be accounted for in the DC loading on the interior beam moment table.

When noise abatement walls are required to resist differential earth loading, maximum active earth pressure and live load surcharge loads shall be shown in the design loading. The maximum retained fill shall be 7 ft. Walls requiring greater than 7 ft. of retained fill are defined as retaining walls and will require additional planning, design, and detailing beyond the scope herein.

An example of specifying the design loading is shown in Figure 3.12-1. In addition:

For structure-mounted noise abatement walls, the designer shall add a third line under *Design Loads* as follows:

Designed Service Weight of Noise Abatement Wall: _ kips per lineal foot of wall

For ground-mounted noise abatement walls, when applicable, the designer shall add a third line under *Design Loads* as follows:

Unfactored Design Active Earth Pressure: 55 pcf equivalent fluid pressure
Minimum Live load Surcharge Pressure: 2 ft. additional earth pressure

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3. Design Stresses

The design stresses for materials shall be included on the General Plan and Elevation sheet, under Design Stresses. This will give the Contractor and wall supplier information on the types of materials required on the project, and their required design strengths.

Minimum required design strengths are provided below.

Field Units:

Drilled Shaft Concrete Foundations:	f'_c	=	4,000 psi
Structural Steel (Grade 36):	F_y	=	36,000 psi
Structural Steel (Grade 50):	F_y	=	50,000 psi
Reinforcement:	f_y	=	60,000 psi

Precast Units:

Precast Concrete:	f'_c	=	4,500 psi
Reinforcement:	f_y	=	60,000 psi
Welded Wire Reinforcement:	f_y	=	65,000 psi

An example of specifying the design stresses is shown in Figure 3.12-1.

4. Aesthetic Requirements

Aesthetic requirements, such as color, texture, or form liner patterns, shall be shown in the noise wall structure plans. When form liners are utilized, it is desirable to show a drawing of the required pattern on the contract plans to ensure the aesthetic requirements are clearly conveyed to the Contractor and wall supplier. These requirements may be too detailed to be placed on the General Plan and Elevation sheet and/or may be more appropriate to be placed in other locations in the noise abatement wall plans.

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5. General Plan and Elevation Sheets

For ground-mounted noise abatement walls, a General Plan and Elevation sheet for the noise abatement wall plans shall be provided.

For structure-mounted noise abatement walls, on either bridges or anchorage slabs, the wall is considered a separate structure and requires plans independent of the bridge or anchorage slab on which it is mounted. A General Plan and Elevation sheet for the noise abatement wall, independent of the bridge or anchorage slab, shall be provided. Structure-mounted noise abatement walls are not always constructed on the same contract as their supporting structures.

Figures 3.12-2 and 3.12-3 show Plan and Elevation details for ground-mounted noise abatement walls. Structure-mounted noise abatement wall General Plan and Elevation sheets are similar.

Plan Views

Plan views shall show stations and offsets of the beginning and end of the noise abatement wall, as well as the stations and offsets of any required overlapping wall sections, wall directional changes, adjacent retaining walls or culverts, obstructions, ground modifications such as drainage interference, all overhead and underground utilities, and lighting. The stations and offsets shown in the contract plans and in the shop drawings shall be tied to the project stationing and not to a separate stationing system.

Plan views shall show the station and offset of each boring log included in the plans.

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Elevation Views

Elevation views shall show the theoretical top and bottom of noise wall elevations at all necessary locations to ensure the proper shape of the wall is provided. This information, plus the station and offset, shall be provided for any location where there is a change in slope at either the top or the bottom of the wall or at the finished grade at the face of the wall. This information shall be provided for all locations shown in the Plan view as described in the section above. For long walls and/or when there are many locations to show, it is often useful to provide the information in a Data Table.

NOISE ABATEMENT WALL DATA TABLE

Segment # *SN # # # - N # # # #*

Station	Offset (Rt.) to Centerline Wall (ft)	Theoretical Top of NAW Elev.	[Road name] Profile Elev.	Finished Grade Elev. At Front Face of NAW	Finished Grade Elev. At Back Face of NAW	Existing Grade Elev. At Front Face of NAW	Theoretical Bottom of NAW Elev.	Theoretical Wall Height (ft)
716+96.85	401.93	655.00	642.81	638.03	639.34	637.93	637.36	17.64

Example: Ground Mounted Table

NOISE ABATEMENT WALL DATA TABLE

Segment # *SN # # # - N # # # #*

Post	Station	Offset to Centerline of Post (ft)	Theoretical Top of NAW Elev.	[Road name] PGL Elev.	Top of Parapet Elev.	Existing Grade Elev. At Front Face of NAW	Theoretical Bottom of NAW Elev.	Theoretical Wall Height (ft)
P14-01	492+28.83	64.27	609.73	594.92	600.67	595.75	600.17	9.56

Example: Structure Mounted Table

The theoretical top of wall elevations shown on the contract plans shall meet applicable noise wall height requirements. See Chapter 26 of the Bureau of Design & Environment Manual and the IDOT Highway Traffic Noise Assessment Manual for more information on noise wall height requirements. The noise wall height will be dependent upon the proposed ground surface elevations at the front face of the wall for ground-mounted walls, or the top of slab elevations for structure-mounted walls.

For ground-mounted walls, theoretical bottom of wall elevations shown on the contract plans should be assumed to be eight inches below the proposed ground surface. Theoretical bottom of wall elevations may be decreased as required to accommodate proposed ground conditions such as locations of utilities, earth retention, drainage, etc. The plans shall show both the existing ground elevations and proposed ground profile grade at the face of the wall.

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For structure-mounted walls, theoretical bottom-of-wall elevations shown on the plans shall be taken as six inches below the top of parapet elevation. See Figure 3.12-4.

For ground-mounted noise abatement walls, the theoretical top and bottom elevations are a minimum envelope to be provided by the supplier. Due to the discrete, stair-stepped nature of the final wall dimensions, the furnished top and bottom wall elevations will vary and exceed the dimensions shown on the plans. For structure-mounted noise abatement walls, the furnished height of the wall should closely match the dimensions shown on the plans such that the loading requirements are not exceeded.

On ground mounted noise abatement walls (NAWs), where the plans show stair-stepped lines for the wall or ground line, but no data for those points, the following note shall be added to the plans:

Theoretical Top of NAW Elev., Theoretical Bottom of NAW Elev., Finished Grade Elev. At Front Face of NAW, and Finished Grade Elev. At Back Face of NAW shall be taken as straight lines in the segments between each pair of stations shown in the NAW Data Table. For ground-mounted walls, the maximum post spacing of 20 ft. shall be shown on the elevation view. Exact post locations will be determined by the wall supplier and therefore are not shown on the plans; the intent of the plan view is to aid the supplier in locating available post locations, not to dictate exact post locations. See Figure 3.12-3 for an example.

For structure-mounted walls, the exact post locations shall be detailed on both the General Plan and Elevation sheet for the noise abatement wall plans and in the bridge or anchorage slab plans. The maximum post spacing for structure-mounted walls is given in Figure 3.12-4.

For ground-mounted walls, when proposed ground surface slopes indicate that ponding at the wall face is a concern, 4 in. diameter weep holes shall be provided at ground level in the bottom panel of the wall at a spacing of 8 ft. This is to be determined during Phase II and the weep holes shall be shown at the required locations on the plans.

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Section Thru Wall

When portions of ground-mounted walls are required to resist soil loading, a section thru the wall showing

i. the maximum height of retained fill
ii. centerline of roadway and/or profile grade line
iii. horizontal dimensions and cross slopes to get to the NAW

shall be provided; these may be provided as part of the "Section Thru Wall" or as a separate additional sketch. A separate profile grade sketch shall also be provided.

The section thru the wall shall show similar granular backfill and pipe underdrain details as a soldier pile retaining wall. See Chapter 3.11 of the Bridge Manual for applicable details. Walls requiring more than 7 ft. of soil retention will require design and detailing beyond the scope herein.

Details for sections thru walls for structure-mounted walls are given in Figures 3.12-4 and 3.12-7. Because noise walls require separate plan sheets than the structures they are mounted on, wall details on the bridge or anchorage slab sheets should be marked For Information Only.

Access door or panel locations shall be shown on the elevation view.

6. Structural Details

Figures 3.12-4 through 3.12-13 show details for structure-mounted noise abatement walls. These details shall be used for noise abatement walls mounted on bridges and anchorage slabs.

Figure 3.12-4 shows a standard configuration for a structure-mounted noise abatement wall on a bridge. The wall is mounted to the back of a 72 in. concrete barrier via a post connection bracket. The barrier is mounted on a thickened deck overhang.

The 72 in. concrete barrier for use with structure-mounted noise abatement walls consists of a standard 44 in. constant-slope barrier, with the addition of a vertical 28 in. parapet extension. The vertical extension reduces the rotation of a truck in an impact event, reducing the zone of intrusion of the truck. The vertical extension also reduces salt spray on the face of the noise wall, reducing wall damage due to chloride ingress.

The post connection bracket shape is a steel section that acts as a spacer, placing the front face of the wall 10 in. behind the back face of the 72 in. concrete barrier, further reducing the zone of intrusion of the truck.

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When used together, the 72 in. concrete barrier and offset noise wall position the wall such that it is outside the zone of intrusion of the truck. This configuration has been successfully crash-tested, and has been determined to be crashworthy for Test Level 5 criteria for the 2016 Manual for Assessing Safety Hardware.

The deck overhang thickness when a noise abatement wall is used shall be 11.5 in. This additional thickness is required to support the loading from the noise abatement wall and larger parapet. The fillet at the exterior beam section is also thickened such that there is a minimum of 11.5 in. of deck thickness over the exterior beam.

The maximum post spacing for a noise abatement wall mounted on a structure shall be 11 ft. 8 in. The maximum height for a noise abatement wall mounted on a bridge shall be 18 ft. above the deck elevation at the front of the parapet.

Structure-mounted noise abatement wall plans require the CL post locations to be as specified on the bridge or anchorage slab plans. This is to ensure that bridge or anchorage slab reinforcement details are consistent with the anchor rod assemblies and post connection locations on the wall plans. It also ensures that parapet joints are appropriately placed outside the minimum required distance from the CL post to the joint. Post locations for structure-mounted walls cannot be altered by the wall supplier.

Figure 3.12-4 also shows a section thru the noise abatement wall anchor rod assembly. This assembly consists of four L-shaped anchor rods, a template plate, and related hardware. More information is given in Figure 3.12-12 below.

Figure 3.12-5 shows deck overhang and parapet reinforcement details for use with the 72 in. concrete barrier. The reinforcement in the parapet, overhang, and adjacent slab are increased to support the noise abatement wall loading.

Parapet and deck overhang reinforcement used with the 72 in. concrete barrier is increased in “end regions” near full-depth joints and parapet ends. These “end regions” are shown in Figure 3.12-5 and are defined as regions within 15 ft. of full-depth joints and parapet ends. Regions outside of this distance are shown as “interior regions” and have lesser reinforcement requirements.

Figure 3.12-6 shows dimensions for the standard reinforcement used in the 72 in. concrete barrier and deck overhang.

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Figures 3.12-7, 3.12-8, and 3.12-9 show details for noise abatement walls mounted on anchorage slabs. These three figures show similar detail requirements to those required for bridges in Figures 3.12-4 through 3.12-6.

Figure 3.12-10 shows details for aluminum parapet joints and cork parapet joints, for use with the 72 in. concrete barrier.

Figure 3.12-11 shows details for an elastomeric debris shield. The intent of this shield is to prevent debris, such as snow and ice, from accumulating between the noise abatement wall and parapet.

Figure 3.12-12 shows a plan view of the post, post connection bracket, and parapet section. Dimensions for L-shaped anchor rods are shown in this figure, as well as an anchor rod template plate. The anchor rod template plate is embedded ¼ in. into the back face of the parapet, and maintains the positions of the anchor rods during the pouring of the parapet. The template plate will remain in place after the parapet forms are removed, with the connection bracket mounted on the back face of the template plate.

Figure 3.12-12 also shows required distance from posts to parapet joints.

Figure 3.12-13 shows post connection bracket details, and concrete connection details for post connection brackets.

Figure 3.12-14 shows details for Name Plates and name plate mounting locations.

7. Noise Reduction Data

A noise reduction data table shall be shown on the plans. The noise abatement wall supplier is required to meet noise reduction requirements. An example noise reduction table is found below, with instructions on how to fill in the blanks of the table.

Noise Wall Structure Number	**Face**	**From Sta.**	**To Sta.**	**Noise Reduction Coefficient**	**Comments**
	front face			Select Value	
	back face			Select Value	

Noise Reduction Data Table

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Noise Wall Structure Number:

A noise wall structure number will be issued by the Department and shall be provided in the table. For structure-mounted noise abatement walls, the noise wall structure number is independent of the bridge structure number. The noise wall structure number will have the format CCC-N####, where CCC is the county number, N is an indicator that it is a noise abatement wall, and #### is a four-digit index.

Face:

The intent of this column is to differentiate between the two faces of the wall. Because the two faces may require different treatments, it is important to differentiate between the two such that the contractor does not install a panel in the incorrect orientation.

The terms “front face” and “back face” are used as examples herein. The designer is encouraged to use as descriptive a term as possible to clarify to the

Contractor the correct installation orientation. Examples include “interstate face,” “frontage road face,” “residential face,” or use of actual roadway names such as “I-55 face”.

From and To Stations:

The noise reduction coefficients may change over the length of the wall. The intent of these columns is to show stations where these changes occur. Additional rows may be added as necessary.

Noise Reduction Coefficient:

The Noise Reduction Coefficient is a measure of how much noise is absorbed by absorptive noise walls. There are three options for this column: Reflective, Absorptive- 0.8 min., and Absorptive- 0.65 min. Instruction is given in BDE Manual Chapter 26 and IDOT Highway Traffic Noise Assessment Manual as to the applicability of each option.

Comments:

Any additional comments may be entered into this column.

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8. Soil Data

For ground-mounted noise abatement walls, approximate locations of each boring log included in the contract plans shall be shown in the plan view. Boring logs should be taken every 200 ft., or as close as feasible given issues such as terrain, utilities, right-of-way, etc. All boring logs shall be included in the contract plans. The wall designer will use the data in the boring logs to design the wall foundations. Conservative soil data assumptions are included in the Guide Bridge Special Provisions to ensure that the wall foundations may still be designed should boring logs be omitted or incomplete.

9. Name Plate Details

Name plates shall be provided for all noise abatement walls. Name plate location requirements and lettering details are provided in Figure 3.12-14.

10. Pay Items, Methods of Measurement, and Special Provisions

Anchor bolts used for the anchorage assembly for structure-mounted walls shall be paid for as NOISE ABATEMENT WALL ANCHOR ROD ASSEMBLY, with units of Each. This pay item is furnished with the parapet on which the noise abatement wall is mounted. Therefore, it shall be shown on the plans for the bridge or anchorage slab on which the wall is mounted. This pay item shall not be shown on the noise abatement wall plans. Each anchor rod assembly will consist of the four anchor rods, template plate, and associated hardware.

For ground-mounted noise abatement walls, the pay item NOISE ABATEMENT WALL, GROUND MOUNTED shall be used, with units of Sq. Ft. As specified in the special provision, this pay item includes the panels, posts, foundations, connections, and any other items associated with the wall.

For structure-mounted noise abatement walls, the pay item NOISE ABATEMENT WALL, STRUCTURE MOUNTED shall be used, with units of Sq. Ft. As specified in the special provision, this pay item includes the panels, posts, connections, debris shield, and any other items associated with the wall, with the exception of the anchor rod assemblies. This pay item shall be shown on the wall plans only. The bridge or anchorage slab plans shall not show these items. This is to avoid unintentionally doubling the quantities, and to allow for noise walls to be placed on separate contracts, independent of the contract containing the bridge or anchorage slab construction.

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Name plates will be paid for at the contract unit cost per each for NAME PLATES.

Methods of measurement for these pay items are found in the applicable Guide Bridge Special Provision.

Three Guide Bridge Special Provisions (GBSPs) shall be inserted into contracts as applicable. These are:

- GBSP 101 (Noise Abatement Wall, Ground Mounted)
- GBSP 102 (Noise Abatement Wall, Structure Mounted)
- GBSP 103 (Noise Abatement Wall, Anchor Rod Assembly)

3.12.2 Noise Abatement Wall Phase III Shop Drawing Requirements

The Phase III shop drawings submittal include the site-specific wall design calculations, details, working drawings, and fabrication shop drawings for the proposed noise abatement wall. The structural design includes required post sizes and locations, wall panel designs, baseplate designs, concrete shaft and shaft reinforcement details, steel connections, etc. The Bureau of Bridges & Structures or local agency owner will review the shop drawings submittal for structural and geotechnical adequacy, conformance with code requirements, and additional contract requirements.

Phase III shop drawings shall be sealed and signed by an Illinois Licensed Structural Engineer.

3.12.3 Other Wall Types

Noise abatement walls typically consist of precast reinforced concrete panels, with either steel or precast concrete posts. However, other wall types, such as acrylic and vinyl, have also been utilized by the Department for some applications. While use of these wall types is allowed, the standard details and special provisions outlined herein are not applicable. Selection of alternate noise wall types is performed during Phase I wall selection. For more information on these wall types, contact the Bureau of Bridges & Structures or the Bureau of Design & Environment.

3.12.3 Noise Wall Resources

CADD cells for many of the figures below are available in the “Details-Planning” and “Details-Design” IDOT CADD Bridge Cell Library.

Guide Bridge Special Provisions for Noise Walls are available on the primary Bridges & Structures Page.

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DESIGN SPECIFICATIONS

AASHTO LRFD Bridge Design Specifications, ___ Edition

DESIGN STRESSES

FIELD UNITS

f'c = 4,000 psi
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (Struct. Steel, M270 Grade 50, posts)
fy = 36,000 psi (Struct. Steel, M270 Grade 36, all other structural steel)

PRECAST UNITS

f'c = 4,500 psi
fy = 60,000 psi (Reinforcement)
fy = 65,000 psi (Welded Wire Reinforcement)

DESIGN LOADS

Strength III or V Wind: 35 psf
Service I Wind: 15 psf

NOISE ABATEMENT WALL SPECIFICATIONS, STRESSES, AND LOADS

Figure 3.12-1

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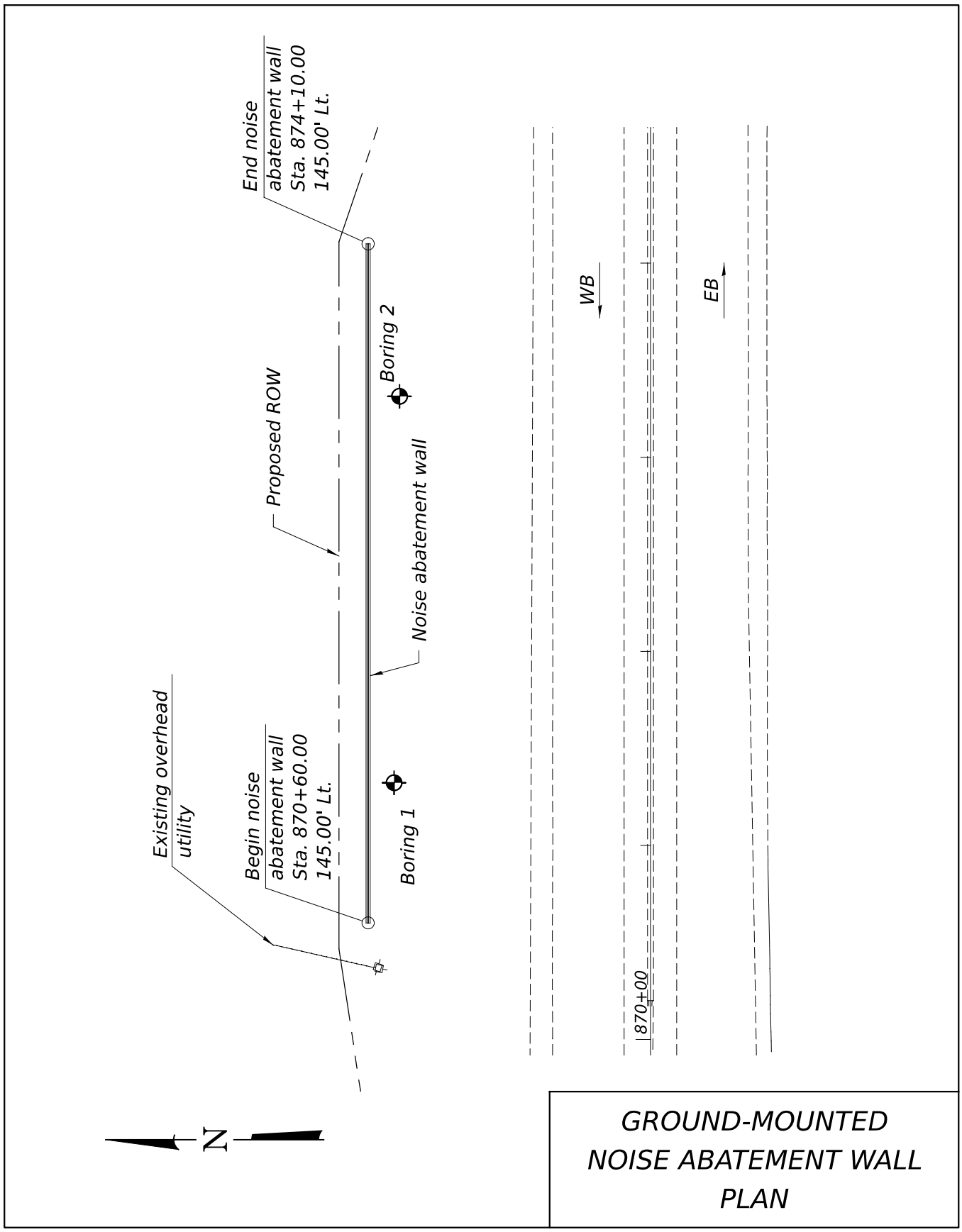


Figure 3.12-2

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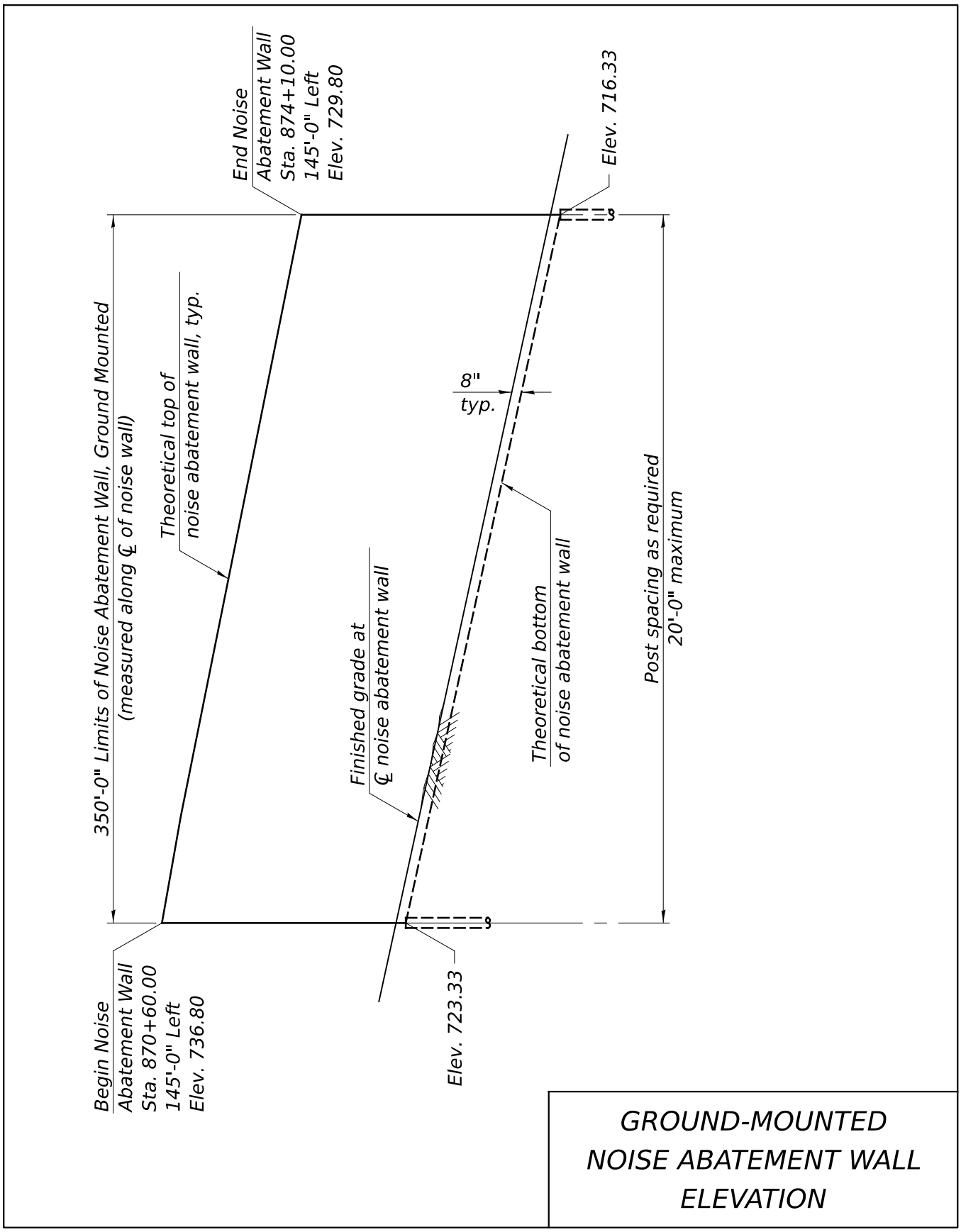
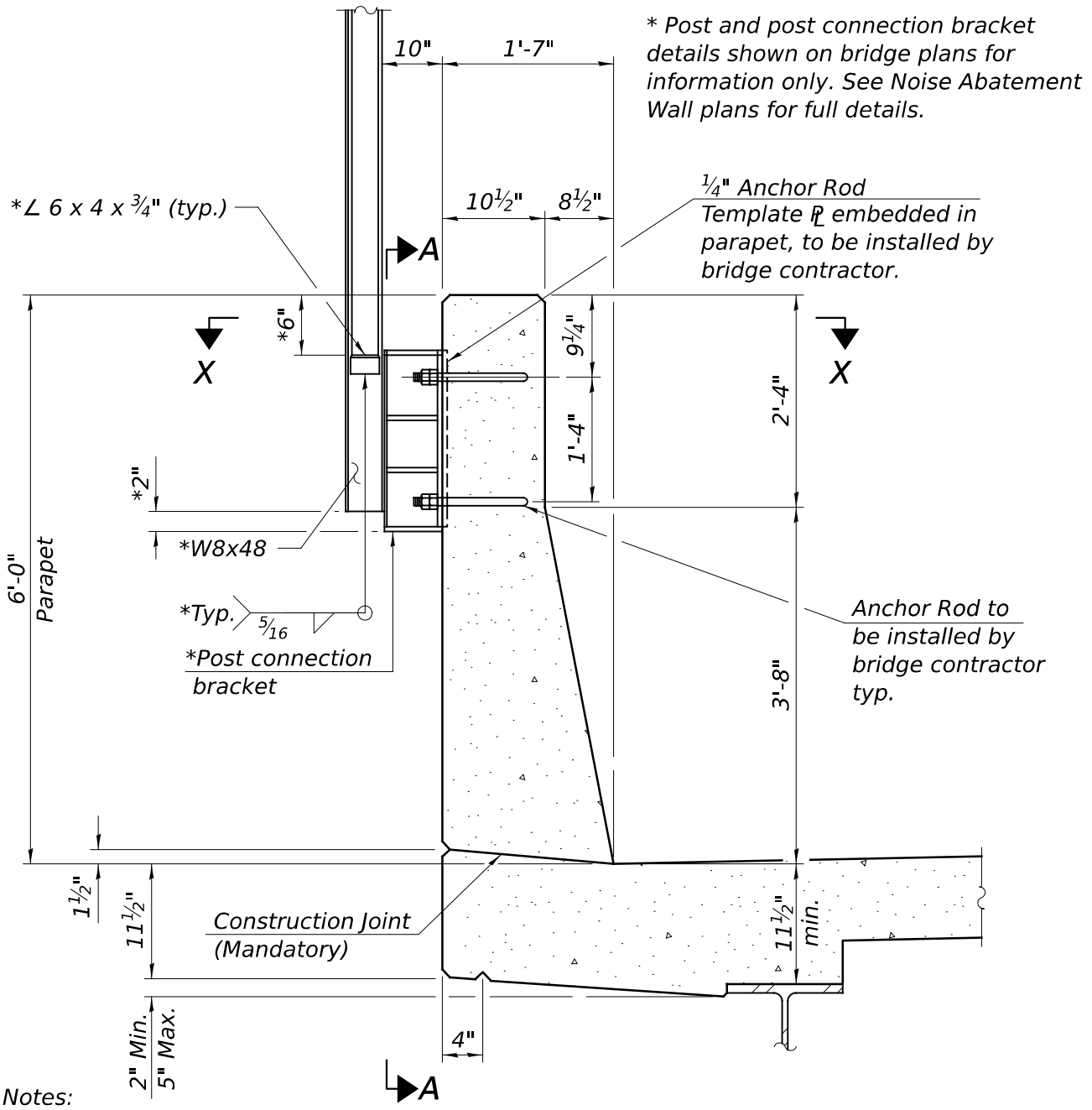


Figure 3.12-3

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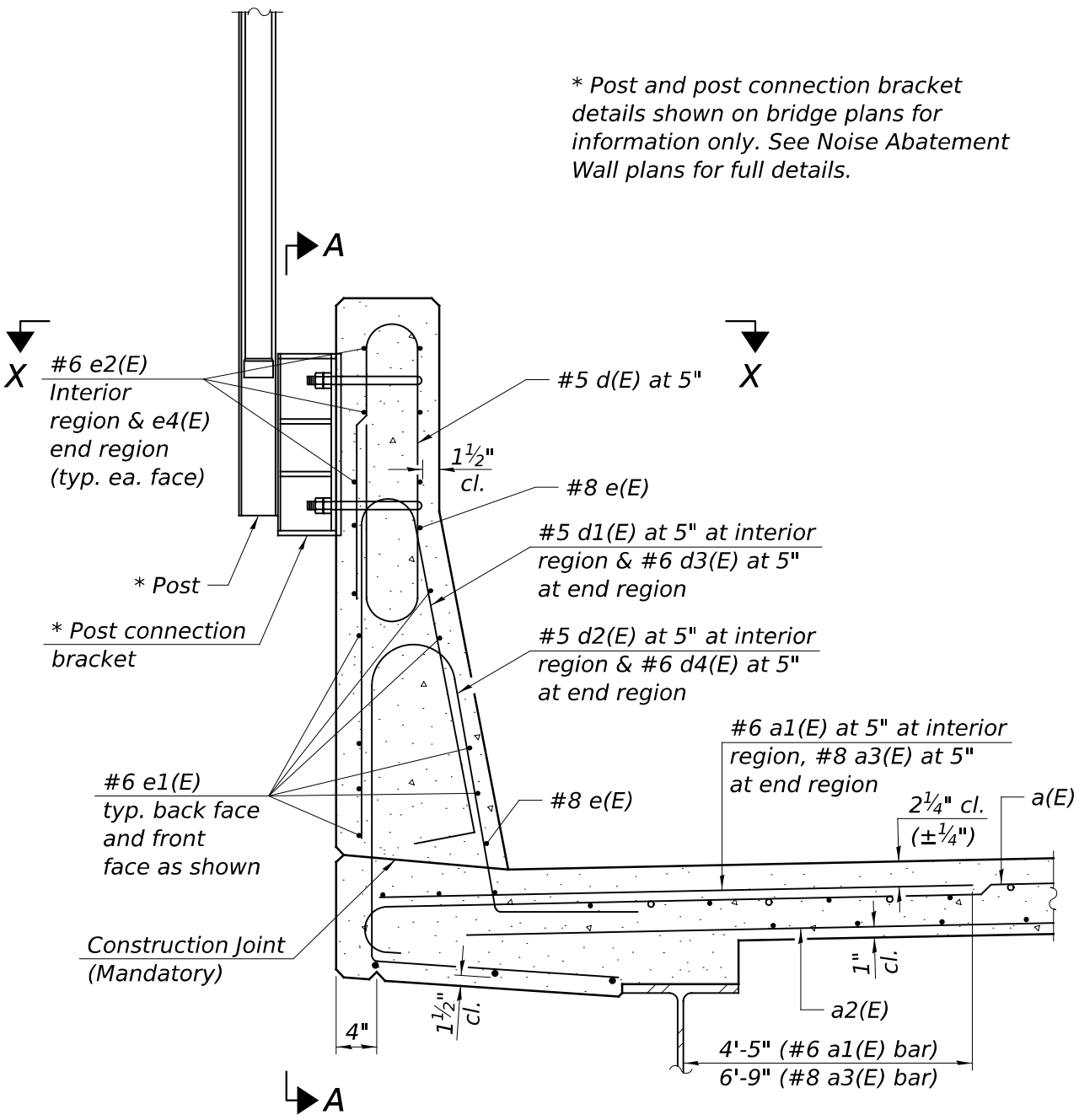
Notes:

1. Steel post maximum spacing is 11 ft. 8 in.
2. Maximum wall height is 18'-0", measured from the top of deck elevation at the front face of the parapet.
3. Slipforming of the parapet is not allowed.
4. For Section X-X, See Figure 3.12-12.
   For Section A-A, See Figure 3.12-13.
5. Cost of four anchor rods, template plate, and associated hardware included in the unit cost for one each of Noise Abatement Wall Anchor Rod Assembly

NOISE ABATEMENT WALL STRUCTURE MOUNTED ON BRIDGE CONFIGURATION

Figure 3.12-4

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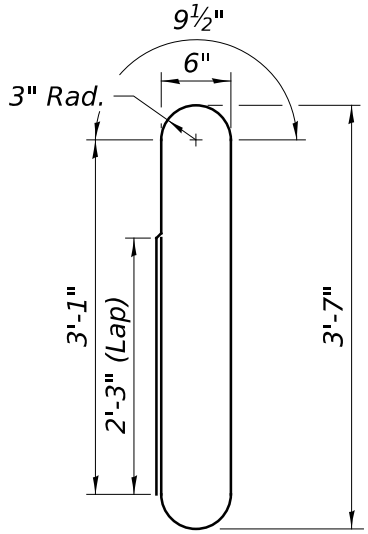
Notes:

1. *End regions are areas within 15' of full height aluminum joints or ends of parapet. All other areas are interior regions.*
2. *Slipforming of the parapet is not allowed.*
3. *For Section X-X, see Figure 3.12-12.*
4. *For Section A-A, see Figure 3.12-13.*

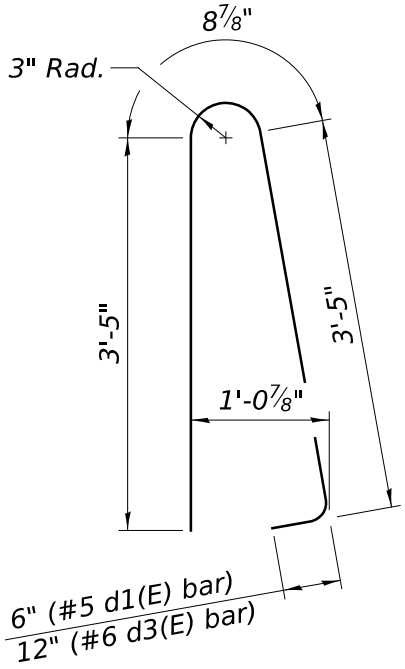
NOISE ABATEMENT WALL STRUCTURE MOUNTED ON BRIDGE DECK AND PARAPET REINFORCEMENT

Figure 3.12-5

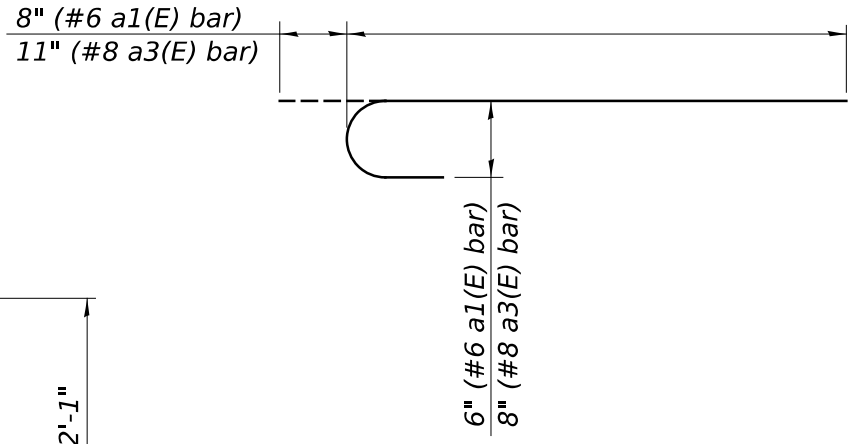
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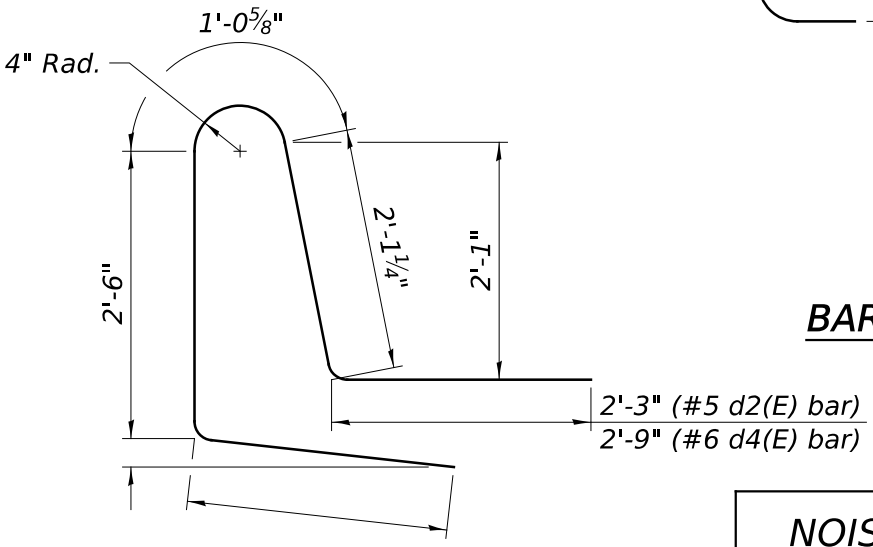
BAR d(E)



BAR d1(E) OR d3(E)



BAR a1(E) OR a3(E)



BAR d2(E) OR d4(E)

NOISE ABATEMENT WALL
STRUCTURE MOUNTED
ON BRIDGE
REINFORCEMENT DETAILS

Figure 3.12-6

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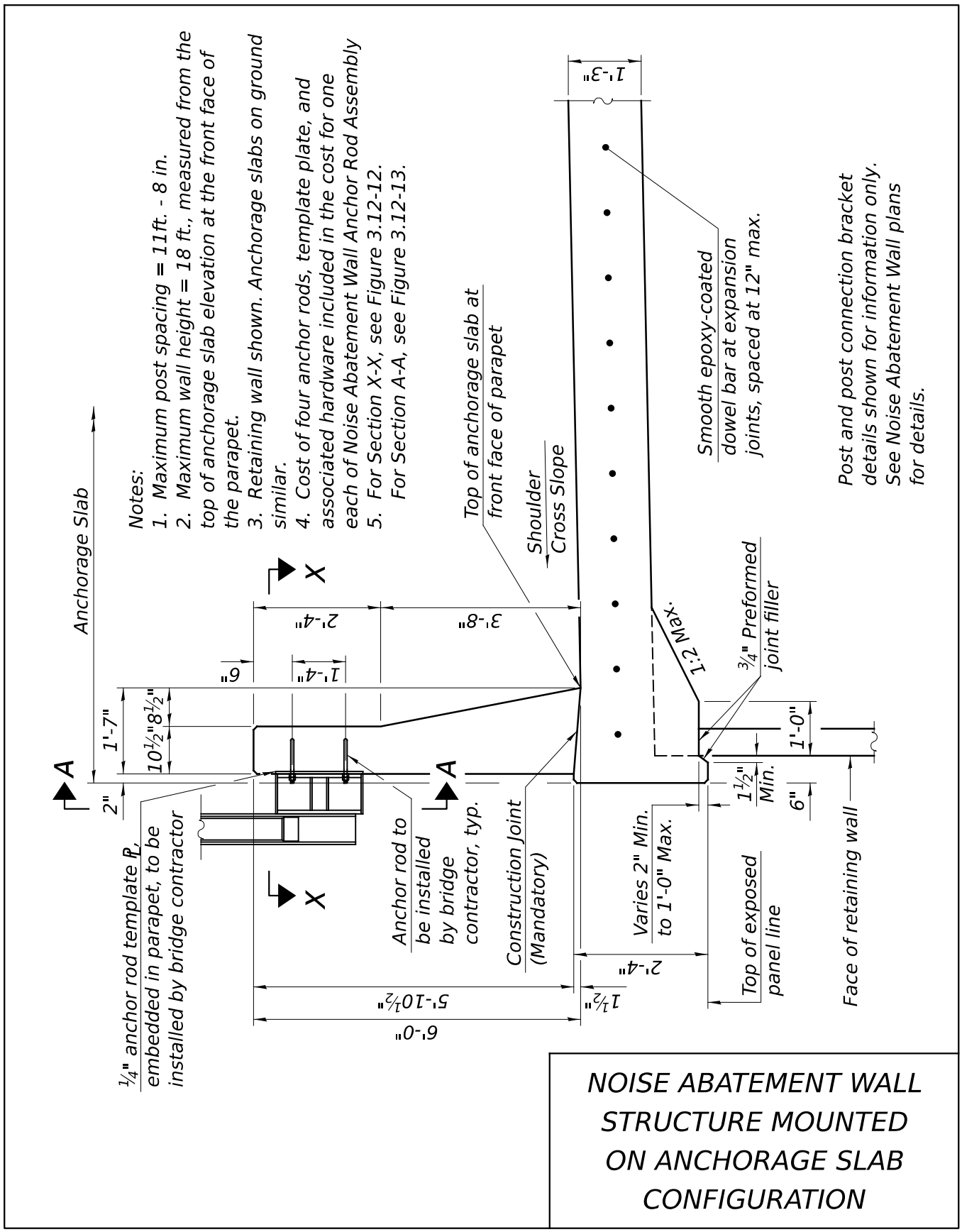


Figure 3.12-7

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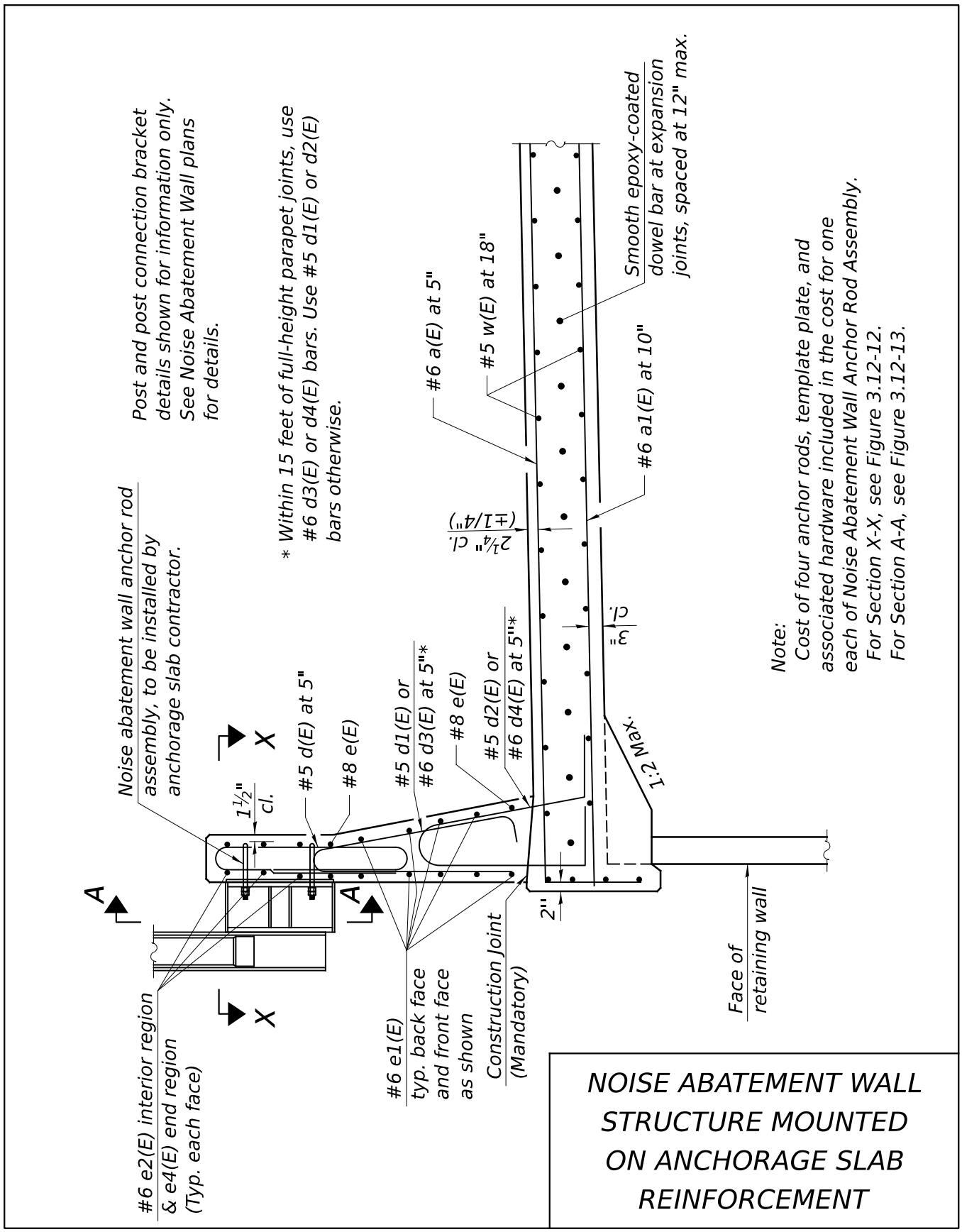
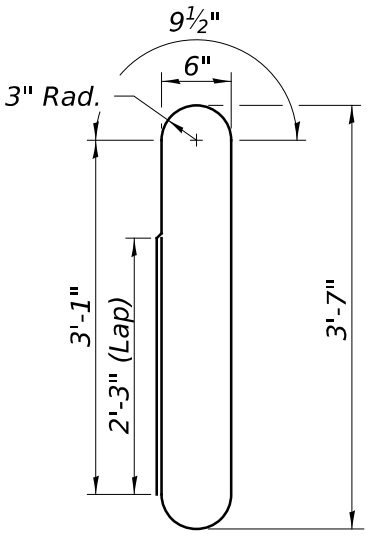
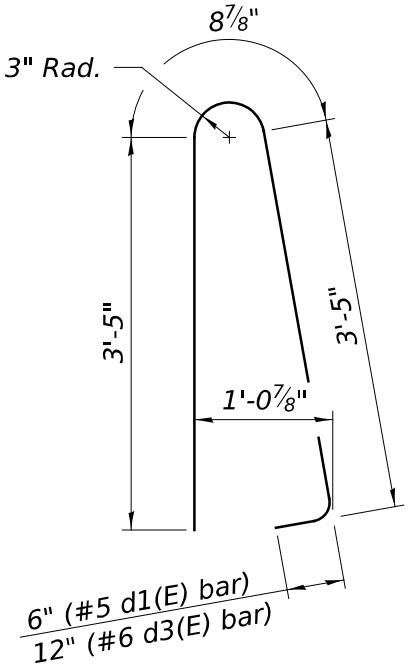


Figure 3.12-8

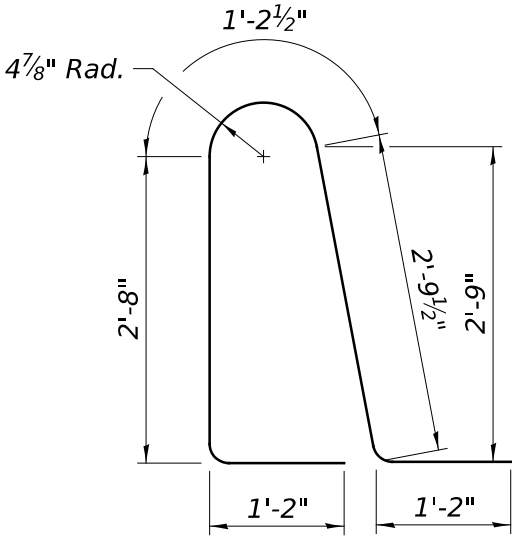
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BAR d(E)



BAR d1(E) OR d3(E)



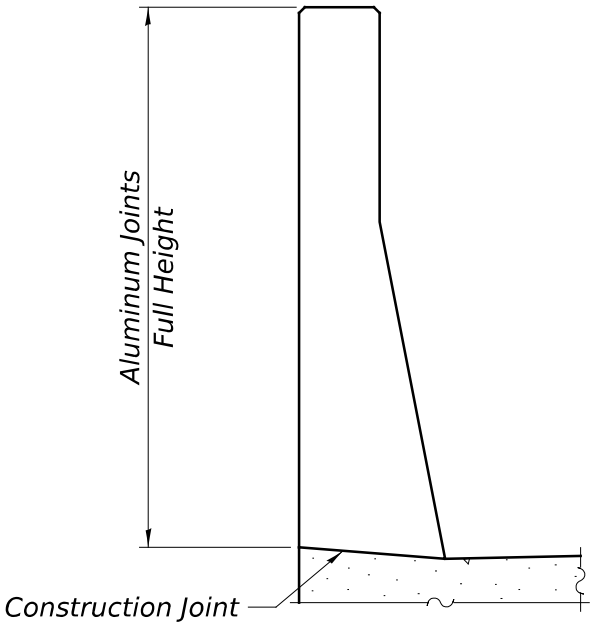
BAR d2(E) OR d4(E)

Back leg to extend 9½"into anchorage slab

NOISE ABATEMENT WALL
STRUCTURE MOUNTED
ON ANCHORAGE SLAB
REINFORCEMENT DETAILS

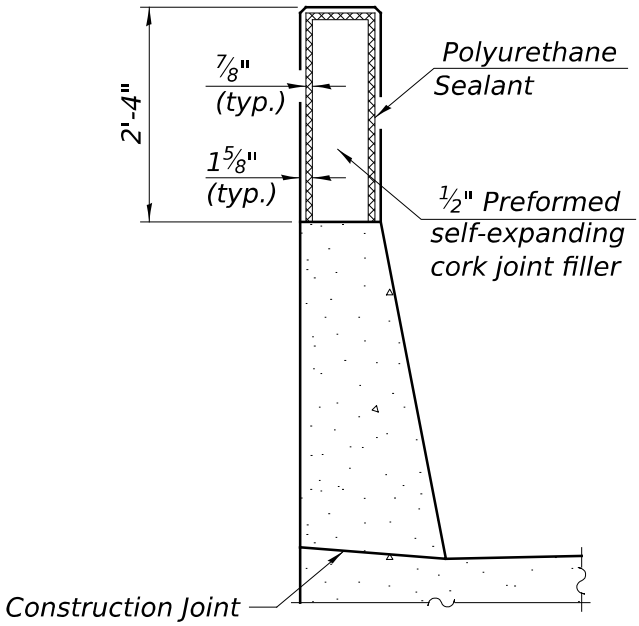
Figure 3.12-9

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ALUMINUM PARAPET JOINT DETAILS

Minimum joint spacing for aluminum parapet joints 11'-3".



CORK PARAPET JOINT DETAILS

Space cork parapet joints at 14'-0" to 20'-0" centers.

Notes:

1. *The 3/16" aluminum sheet shall be ASTM B 209 alloy 3303-H14 and coated to minimize reaction with wet concrete. Cost included with Concrete Superstructure.*

2. *The polyurethane sealent shall be according to Article 1050.04 of the Standard Specifications and the color shall be gray.*

NOISE ABATEMENT WALL STRUCTURE MOUNTED PARAPET JOINT DETAIL

Figure 3.12-10

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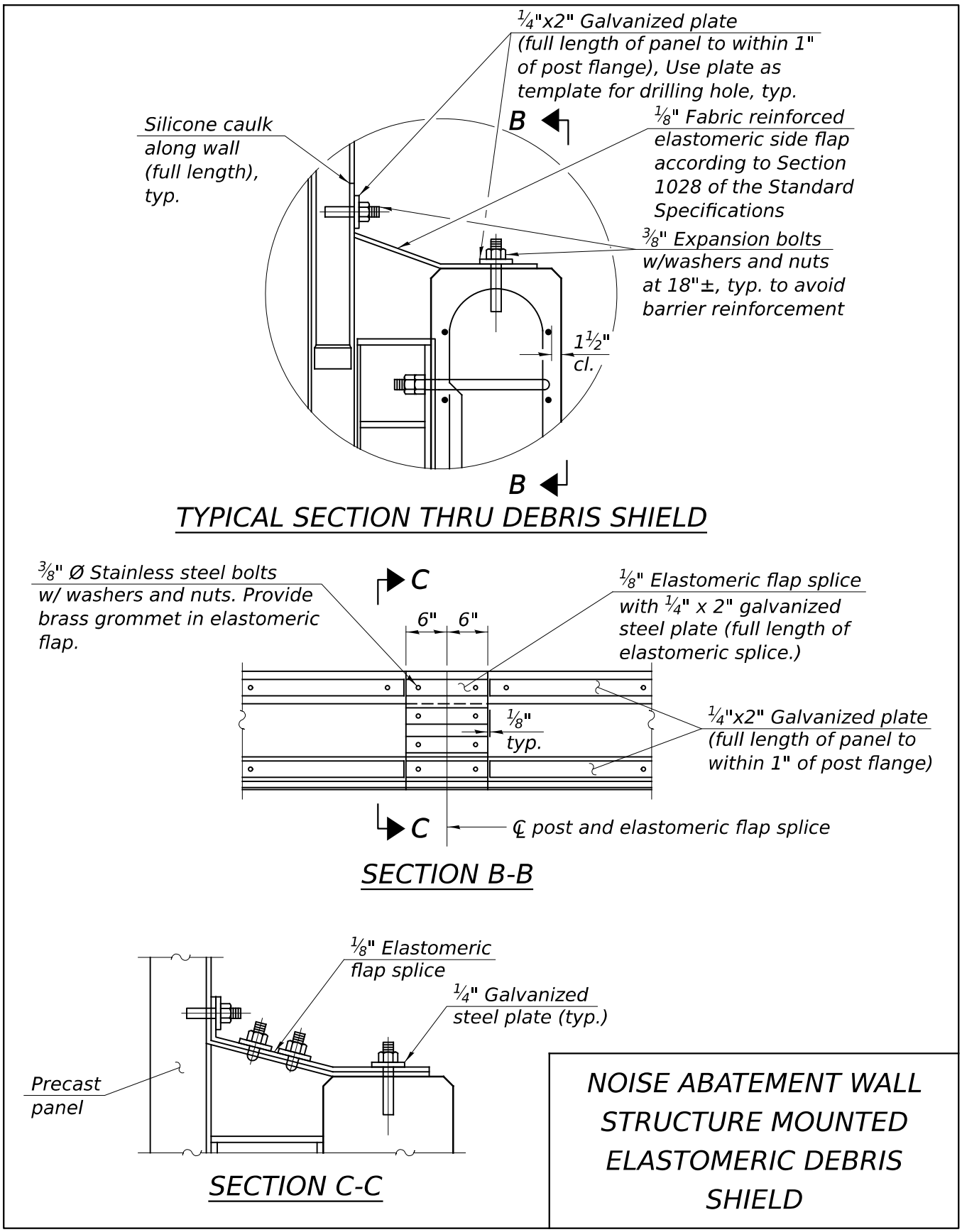


Figure 3.12-11

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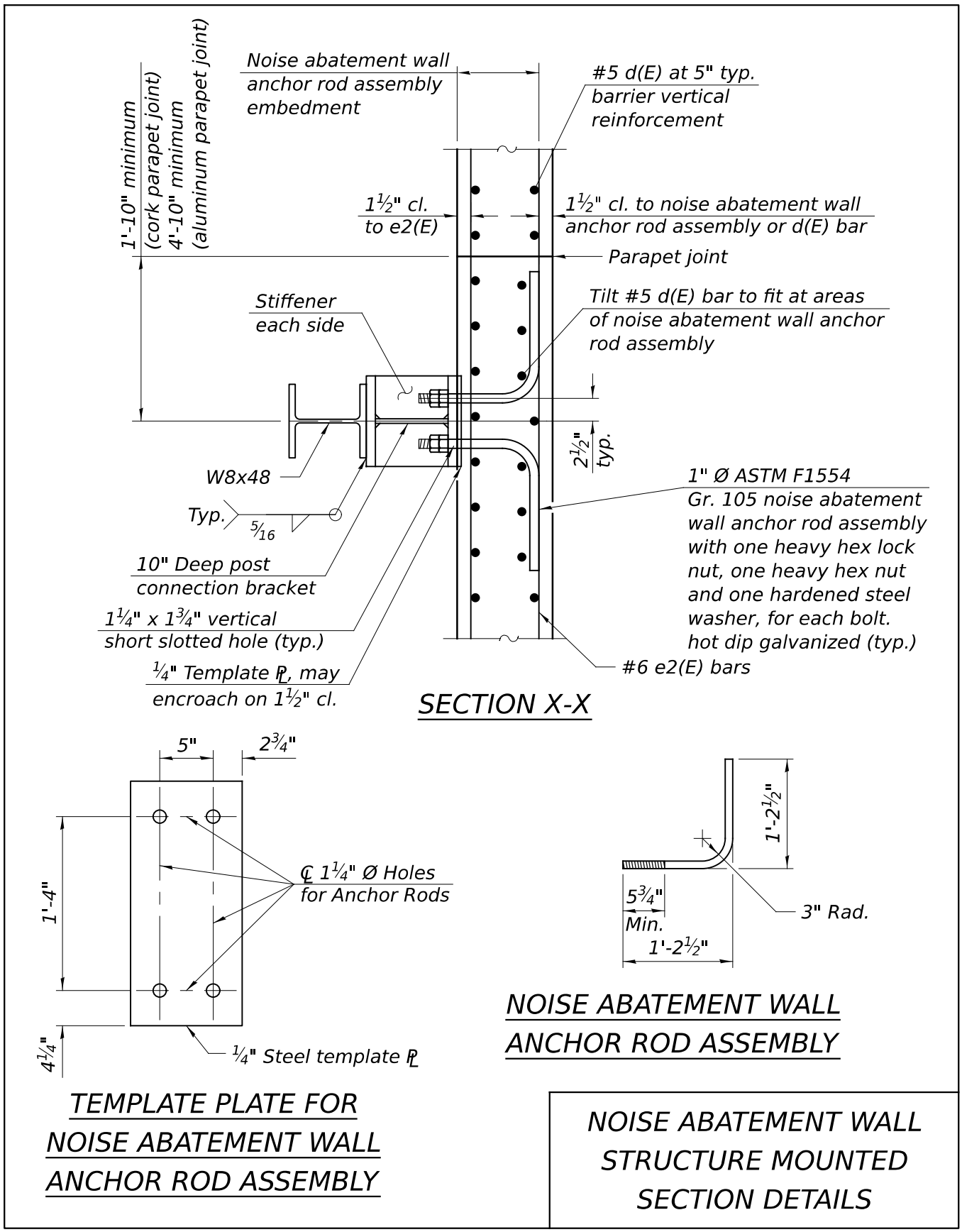
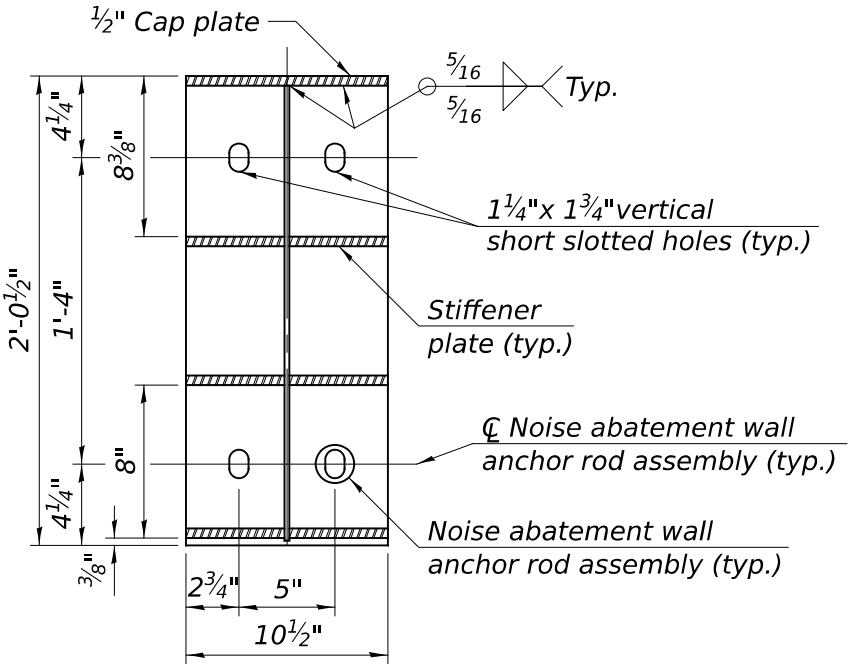


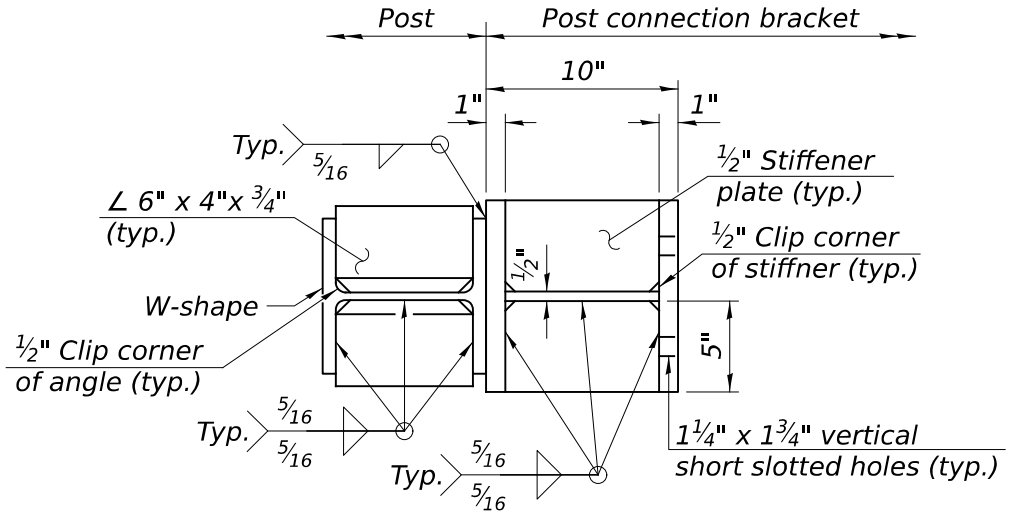
Figure 3.12-12

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SECTION A-A

(Showing post connection bracket)



SECTION THRU POST AND POST CONNECTION BRACKET

(Template ℄ not shown for clarity)

NOISE ABATEMENT WALL STRUCTURE MOUNTED SECTION DETAILS

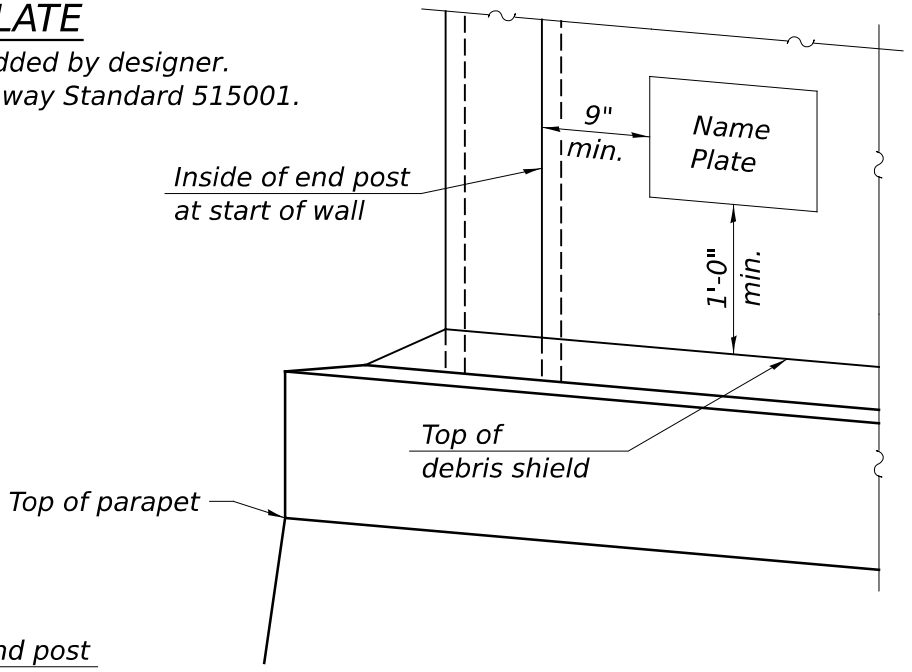
Figure 3.12-13

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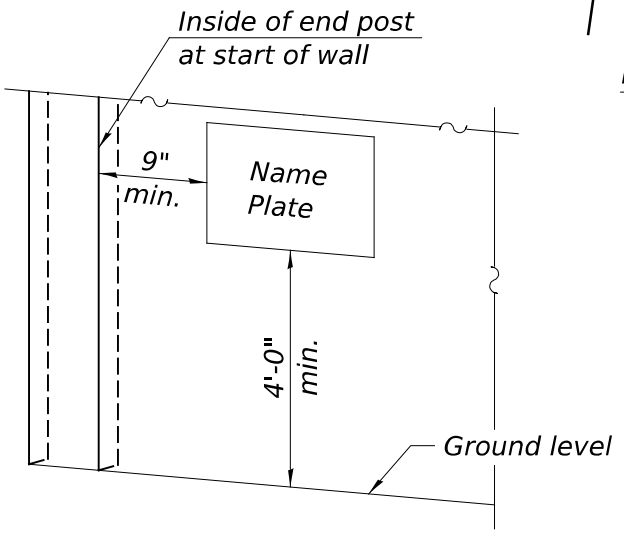
NOISE ABATEMENT WALL
BUILT (YEAR) BY
STATE OF ILLINOIS
(ROUTE NO. OR LOCAL AGENCY STREET NAME)
(SECTION NO.)
FROM STA. (STA. NO.) TO STA. (STA. NO.)
STRUCTURE NO. (STR. NO.)

NAME PLATE

Items in parentheses to be added by designer.
Refer all Name Plates to Highway Standard 515001.



FOR NOISE ABATEMENT WALL, STRUCTURE MOUNTED



FOR NOISE ABATEMENT WALL, GROUND MOUNTED

NAME PLATES FOR NOISE ABATEMENT WALLS

Figure 3.12-14

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3.13 Temporary Construction Works

This section covers the Department's policies related to the design and detailing of items during construction necessary to provide safe means and methods for resisting loads and demands that are transient or temporary due to staging requirements or existing environment limitations. Detailing that meets the Department's necessary licensing requirements and current "Standard Specifications for Road and Bridge Construction" comprise the standard basis for design.

There are a number of situations where temporary construction works are required when a structure is being built. They typically facilitate operations by retaining soil or water or both from an aspect of a structure (e.g. the foundation or at a stage construction line) which is under construction. The Department has developed policies and some straightforward methods for the design of temporary construction works. The following subsections present policy and methodology overviews for some commonly employed techniques. References are also provided for design guides that are available online.

The spectrum of significance (and cost) for temporary construction works is rather broad. Major river bridge crossings which require cofferdams with seal coats for the foundations and pylons or piers are the most costly while single span rural bridges where only temporary sheet piling is necessary at a stage construction line for abutments is on the lower end of cost. The middle of the spectrum for temporary construction works includes pay items for Temporary Soil Retention Systems, Temporary Geotextile Retaining Walls, and Temporary MSE Walls.

The Structure Geotechnical Report (SGR) should be referenced for information and recommendations concerning construction considerations. When necessary, the geotechnical engineer responsible for the report should be contacted for further information or clarification(s).

3.13.1 Temporary Sheet Piling and Temporary Soil Retention Systems

Temporary sheet piling is used for retaining soil in a "cut" situation during construction. Temporary Soil Retention Systems are shown schematically only in the contract plans and are left to the Contractor to determine the specific type and final design. Typically, it is a simple cantilevered design without wales or anchorages. When cantilevered sheet pile is not sufficient, a temporary soil retention system should be specified by the designer. The SGR should be referenced for recommendations concerning which method (or possibly a combination of methods) may be more appropriate for a project.

Temporary sheet piling shall be designed by the engineer responsible for the Contract plans. The contractor is responsible for retaining an Illinois Licensed Structural Engineer to detail the design of

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temporary soil retention systems. For temporary soil retention systems, it is the contractor's option as to the method or combination of methods by which the required soil will be retained. These designs shall be submitted to the Bureau of Bridges & Structures for review and approval. Redesigned temporary sheet piling designs that differ from the Contract plans are allowed and shall also be submitted for review.

Design Guide 3.13.1, in the Design section of the primary Bridges & Structures Page, gives the engineer step-by-step instructions, design aids and an example for the design of cantilevered temporary sheet piling systems. Figure 3.13.1-1 illustrates an example temporary sheet piling design. It includes typical details which shall be given on the Contract plans such as: minimum tip elevations, minimum required section moduli, ground surface/top of sheet piling elevations, retention limits (or exposed surface area), cut slopes required for construction (maximum excavation lines), removal of existing structures, stage lines, existing footings which may interfere with sheeting penetration, etc. Other details which should be shown if applicable but are not in Figure 3.13.1-1 include: buried utilities, overhead power lines, top of rock, etc. Also note that the horizontal dimensions and slopes shown on the Contract plan shall be along the face of the temporary sheeting or temporary soil retention system.

A temporary soil retention system should typically be used when a cantilevered sheet pile design is not feasible. Figure 3.13.1-2 presents an example temporary soil retention system specification as it should appear on the Contract plans. The figure shows how the example presented in Figure 3.13.1-1 would appear if a temporary soil retention system were specified instead of temporary sheet piling. Much of the same information is shown in both figures, but fewer details related to design are required for temporary soil retention systems. Specifically, do not show: wales, soldier piles, sheeting, deadmen, or any other potential retention system components on the Contract plans when a temporary soil retention system is specified. Showing specific items may lead to confusion in the bidding process as the contractor may believe they are part of the plans. In the plan view, only show the "zigzag" pattern usually associated with sheet piling and labeled as "Temporary Soil Retention System."

The quantity for payment on the Contract plan's Bill of Materials for temporary sheet piling and for temporary soil retention systems is specified in Section 522 of the Standard Specifications for Road and Bridge Construction.

For applicable plan notes related to temporary sheet piling and temporary soil retention systems, see Section 3.1.

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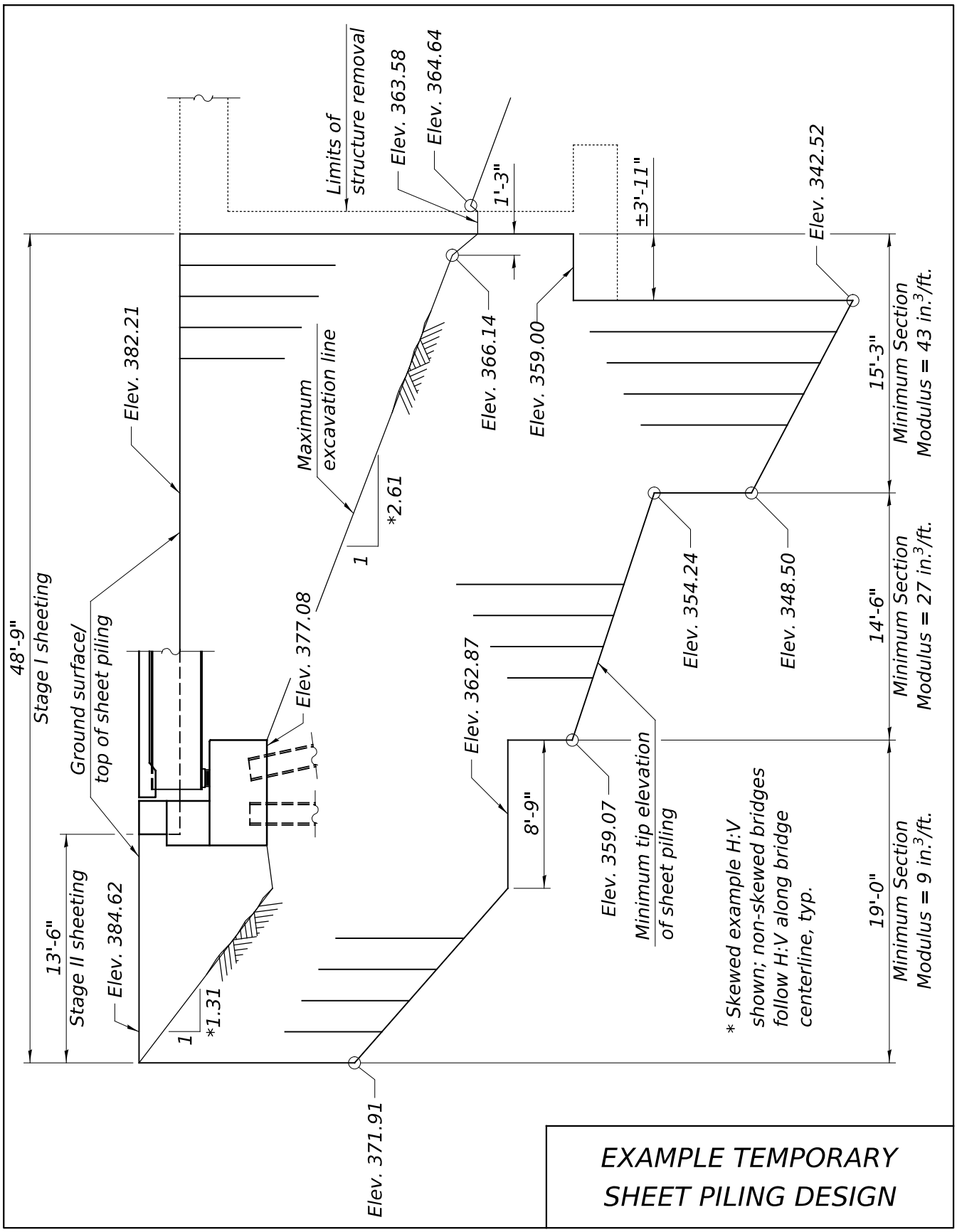


Figure 3.13.1-1

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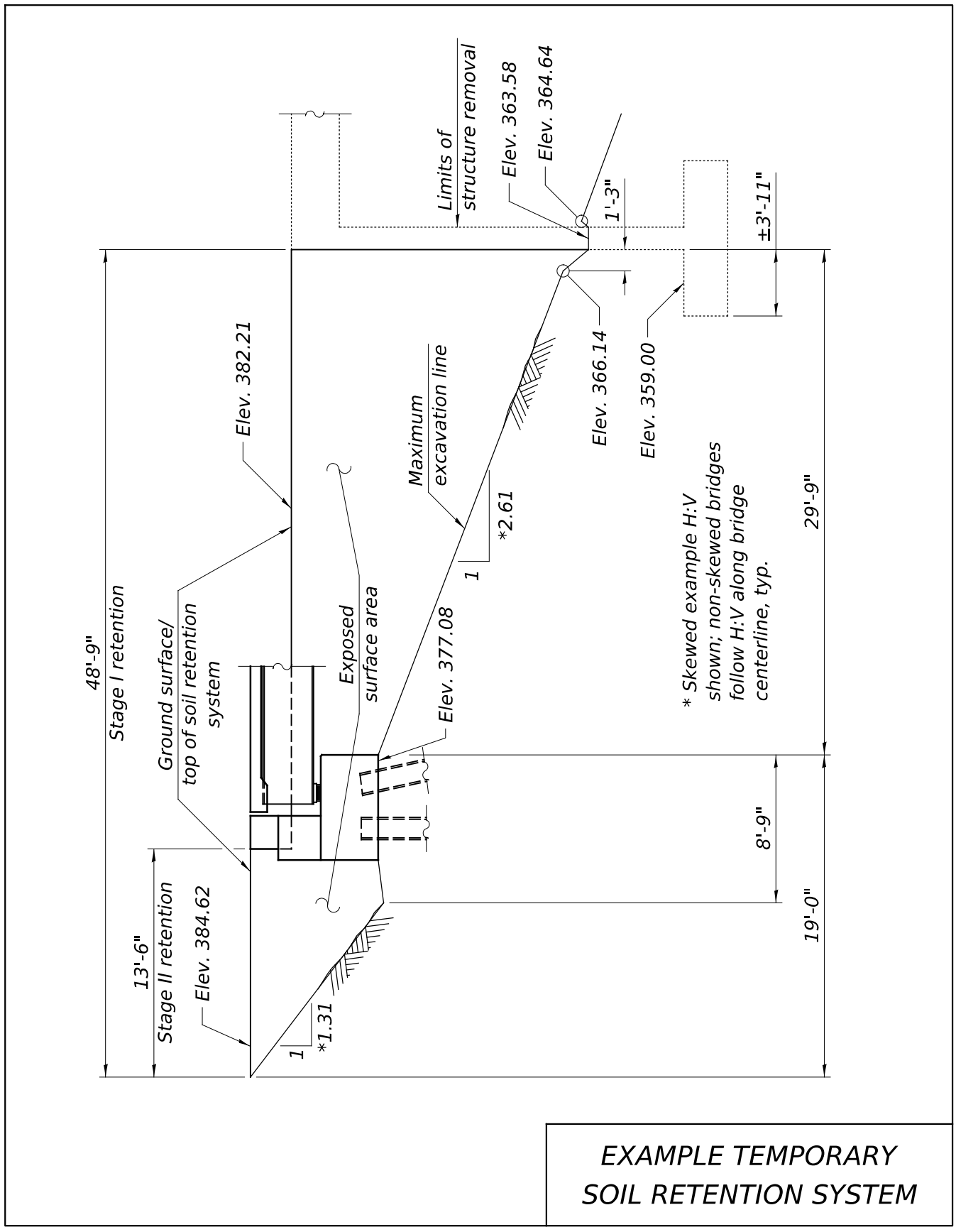


Figure 3.13.1-2

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3.13.2 Temporary Geotextile Walls and Temporary MSE Systems

Geotextile retaining walls are used as temporary construction works in “fill” situations. They can retain Stage I backfill over new box culverts where sloping the fill above the box is not practical or cost effective. Geotextile retaining walls can also be used for temporary surcharge retention and where bedrock or existing infrastructure prevents driving sheet piling. It is also not uncommon for geotextile retaining walls to be used in conjunction with temporary sheet piling or a temporary soil retention system. For “purely fill” applications, the Department prefers that the designer specify a temporary geotextile retaining wall over a temporary soil retention system (See Section 3.13.1) because it reduces cost and construction delays. Article 522.11 of the Standard Specifications for Road and Bridge Construction provides specifications for Geotextile Retaining Walls.

Temporary Mechanically Stabilized Earth (TMSE) systems can also be used in fill situations. See Section 3.11. Temporary MSE walls are typically used when a temporary geotextile retaining wall is not sufficient to retain the required soil for a particular project. For significant retained heights, large quantities of soil, or when project constraints are difficult, a temporary MSE wall may be warranted. Article 522.09 of the Standard Specifications for Road and Bridge Construction provides specifications for Temporary MSE Walls.

The Structural Geotechnical Report (SGR) will recommend the appropriate temporary retaining wall system. If a geotextile retaining wall is specified, it shall be designed and detailed on the final Contract plans. If a Temporary MSE wall is specified, it shall have the geometric limits detailed on the final Contract plans. The contractor is required to retain an Illinois Licensed Structural Engineer to perform the internal stability analysis and design of the temporary MSE wall and submit it to the Bureau of Bridges & Structures for review and approval. The procedure is similar to that required by the Department for permanent MSE walls. The Department’s website contains a Qualified Products List (QPL) of Temporary MSE wall systems that the contractor may choose from.

The design of geotextile retaining walls requires both internal and external stability analyses. Geotextile retaining wall external stability design is very similar to that of MSE walls. See Section 3.11. Considerations include bearing capacity, settlement, sliding, overturning, and global slope stability. Unlike MSE wall requirements, however, the designer responsible for the Contract plans is required to perform the internal stability analysis of the “reinforced soil mass” according to the AASHTO LRFD Bridge Design Specification’s provisions for MSE walls. Internal stability analysis and design involves determining the geotextile reinforcement length required to develop an adequate “pullout” factor of safety as well as the maximum service stress in the geotextile reinforcement. This maximum stress is specified on the plan as T_{min}. It is a necessary piece of information required by the contractor for building the wall

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as detailed on the plans. Design Guide 3.13.2, available on the primary Bridges & Structures Page, presents a detailed procedure for the design of temporary geotextile retaining walls.

Figure 3.13.2-1 presents a typical example of a geotextile retaining wall for a box culvert which is under construction. Shown are plan and typical section views.

The plan view should show: beginning and ending locations, any changes in the wall face alignment, and the approximate rear limits of the geotextile reinforcement. Existing and proposed structures as well as any stage construction sheet piling should also be indicated. When the sides of a fabric wall are constructed against or adjacent to a structure, the fabric should be shown to "lap back" against the structure 3 ft. to prevent soil leakage.

The typical section view should include: the geotextile reinforcement embedment length, the re-embedment lengths, the vertical spacing of the geotextile reinforcement layers, the dimensions to the stage construction and stage removal lines, and the distance between the wall face and the proposed pavement joint, normally at least 2 ft. Typically, the geotextile reinforcement length is set at a minimum ratio of 1.0 times the retained height. Re-embedment within the wall shall not be less than 4.0 ft. with the first 1 ft. making contact with the geotextile layer above while the remaining 3 ft. is embedded 3 in. into the select fill. The top re-embedment length should be shown extending to the rear limits of the geotextile reinforcement. Vertical spacing of geotextile reinforcement is typically 1 ft.

An elevation view should also be provided and indicate the exposed surface area including the top, bottom and any steps in the wall. When a geotextile wall is used with sheet piling, the fabric should be shown in this view to overlap or extend past the sheeting by 3 ft. to prevent soil leakage and allow for sheeting deflections as the backfill proceeds.

Figure 3.13.2-2 illustrates a standard construction sequence and Figure 3.13.2-3 presents a "form brace detail" used for construction of temporary geotextile retaining walls. The following instructions should accompany the construction sequence details shown in Figure 3.13.2-2:

1. Place form brace system on completed reinforcement level; back from the finished fabric face a distance of ⅓ to ½ the geotextile reinforcement spacing.
2. Position fabric so that the required geotextile re-embedment length extends over the top of the form brace and the design reinforcement width is placed with no slack against the previous level.
3. Compact select fill material in lifts to final lift height, create (± 3 in.) depression in zone where re-embedment length will be located and place additional height of compacted select fill against form brace.

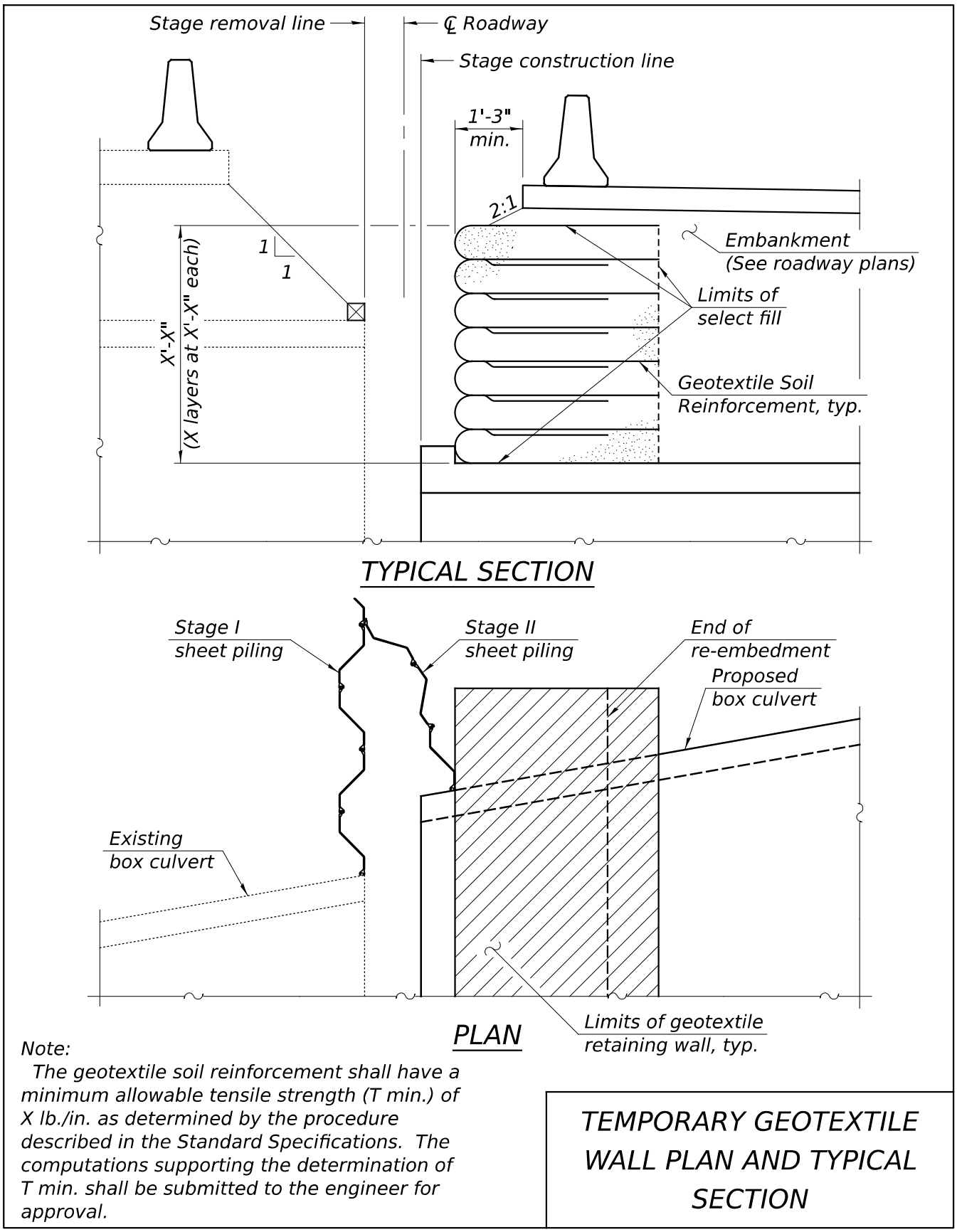
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4. Fold geotextile re-embedment length back over form brace into zone where depression was made in select fill and place additional select fill (± 3 in.) to embed geotextile and bring to final lift height.
5. Pull form brace outward allowing geotextile face to slightly readjust to form tight round face level with plan reinforcement spacing.

Providing drawings similar to Figures 3.13.2-1 to 3.13.2-3, and an elevation view along with the instructions above and the note below which specifies T_{min} is typically adequate detailing for the Contract plans which should all fit on 1 plan sheet.

> *The geotextile soil reinforcement shall have a minimum allowable tensile strength (T_{min}) of ___ lb./in. as determined by the procedure described in Section 522 of the Standard Specifications. The computations supporting the determination of T_{min} shall be submitted to the engineer for approval.*

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Note:
The geotextile soil reinforcement shall have a minimum allowable tensile strength (T min.) of X lb./in. as determined by the procedure described in the Standard Specifications. The computations supporting the determination of T min. shall be submitted to the engineer for approval.

TEMPORARY GEOTEXTILE WALL PLAN AND TYPICAL SECTION

Figure 3.13.2-1

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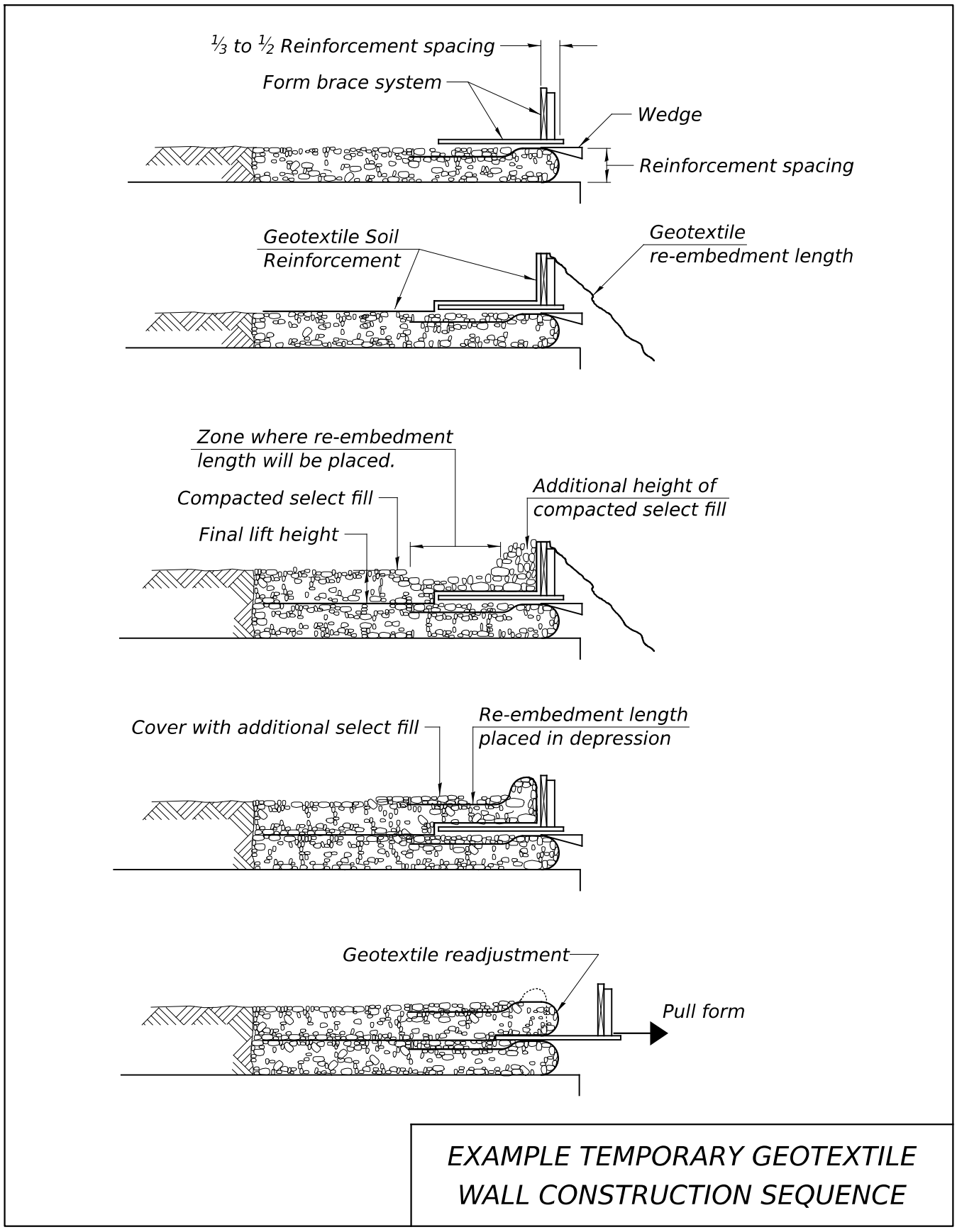
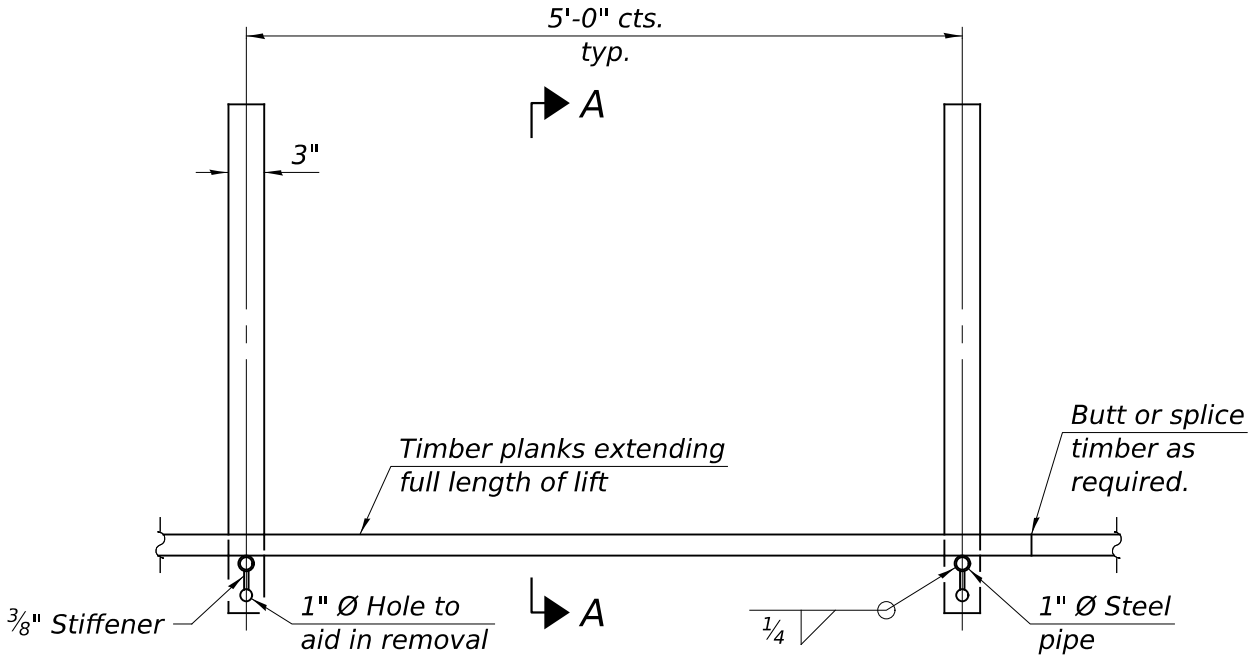
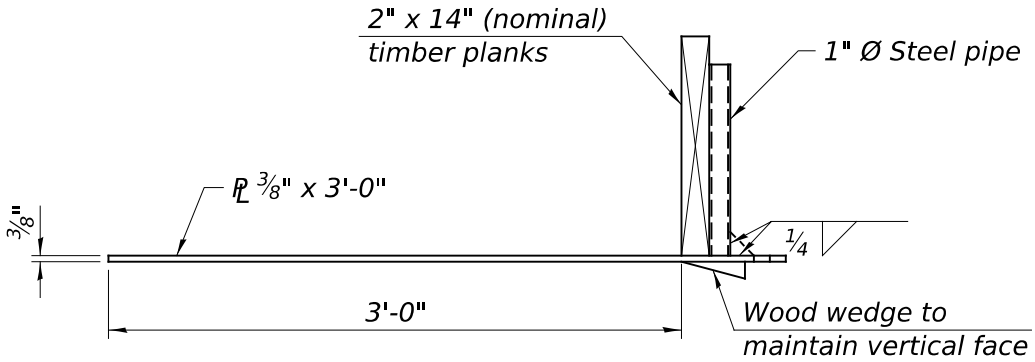


Figure 3.13.2-2

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FORM BRACE DETAIL



SECTION A-A

EXAMPLE TEMPORARY GEOTEXTILE WALL CONSTRUCTION SEQUENCE

Figure 3.13.2-3

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3.13.3 Cofferdams and Seal Coats

Cofferdams are employed in situations where water is anticipated to be present in areas of cast-in-place concrete construction. Exceptions to this policy are in Section 2.3.6.4.2;for these exceptions, tremieing concrete underwater into forms is permitted, and General Note 39 shall be added to the plans. The Department uses two types of cofferdams depending upon the anticipated depth of water above the bottom elevation of the substructure concrete. Type 1 cofferdams are used when the Estimated Water Surface Elevation (EWSE) is higher than the bottom elevation of the substructure by six feet or less and reasonable pumping efforts cannot be assumed to keep the excavation free from ground water. If the EWSE is more than six feet above the bottom elevation of the substructure, then a Type 2 cofferdam shall be used. All cofferdams are required to be dewatered. Any time the use of a seal coat is required to effectively dewater the cofferdam, then a Type 2 cofferdam shall be specified. When seal coat is specified, General Note 28 shall be added to the plans.

See Section 2.3.6.4.2 for more information on the EWSE. When type 2 cofferdams with a seal coat are necessary, the Cofferdam Design Water Elevation (CDWE) shall be shown on the contract plans. The CDWE is the top of cofferdam elevation which is 3 ft. above the EWSE. Seal coat, as depicted in Figure 3.13.3-1 is tremied underwater after excavation and pile driving are complete. It prevents water from seeping beneath sheet piling into a dewatered cofferdam. Typically, seal coats shall be 3 ft. thick minimum and are required when a significant amount of the soil at the site of a cofferdam is permeable (i.e. sandy).

The contractor shall submit Type 2 cofferdam design calculations and plans, prepared and sealed by an Illinois Licensed Structural Engineer, to the Bureau of Bridges & Structures for review and approval before any work on the cofferdams commences on a project.

Excavation required for both types of cofferdams shall be paid for as Cofferdam Excavation.

Typically, cofferdams are specified such that there is only one cofferdam pay item per substructure unit, regardless of whether or not stage construction is present. However, for widening projects where the pier is being widened on either side by differing lengths, two pay items may be used to account for the fact that the two cofferdams are different sizes and are completely separate from each other.

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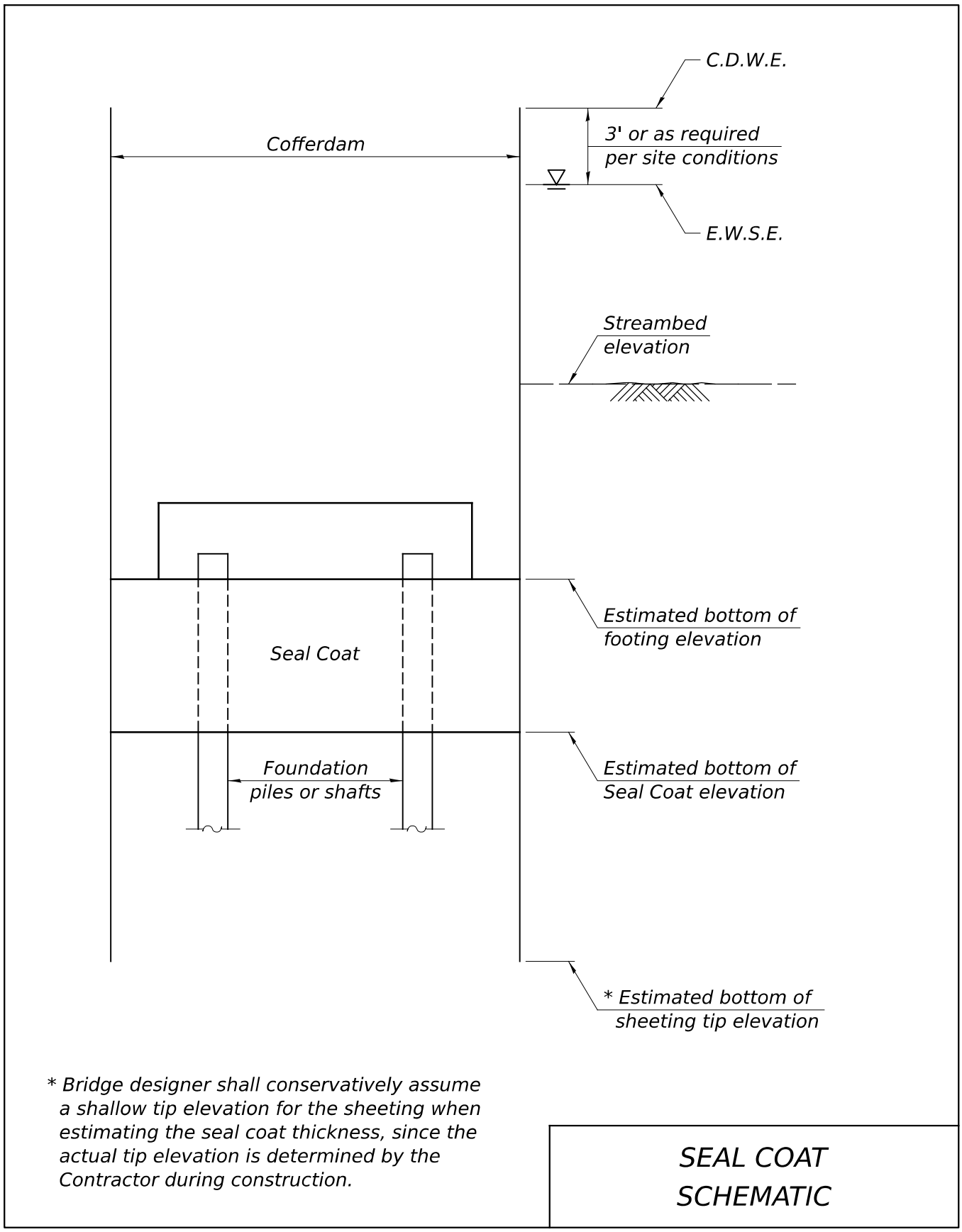


Figure 3.13.3-1

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Seal coats are designed to resist the hydrostatic buoyancy force (P_b) caused by the dewatering of a cofferdam. This design force is counteracted by the weight of seal coat concrete (P_{sc}), the resistance provided by the temporary cofferdam sheet piling (P_{sp}), and the resistance provided by the permanent foundation piles (P_{fp}). The factor of safety for the seal coat design is calculated as,

$$FS = \frac{P_{sc} + P_{sp} + P_{fp}}{P_b}$$

Detailed methods for seal coat design and analysis can be found online in Design Guide 3.13.3, which is available on the primary Bridges & Structures Page. This design guide should be referenced for seal coats designed for the Contract plans as well as those which are redesigned by the contractor. The designer is responsible for the initial seal coat estimate on the contract plans. This is used for establishing the seal coat quantities. The Contractor is responsible for the final seal coat design to be used in conjunction with the final cofferdam design.

Pay limits for cofferdams to be shown in the plans are given in the Illinois Standard Specifications for Road and Bridge Construction.

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3.14 Embankment and Slope Walls

This section covers the Department's policies related to the layout and protection of bridge abutments. Layout and detailing that meets the general subsurface investigation and preparation requirements of the Department's current "Standard Specifications for Road and Bridge Construction" comprise the standard basis for design.

3.14.1 Embankment and Berm

Figures 3.14.1-1 and 3.14.1-2 show embankment plans for single and dual bridge pile supported stub abutments. A construction procedure for embankment cones is detailed in Figure 3.14.1-3.

Bridge cone embankment quantities and the lengths of pipe culverts under the bridge cone shall not be shown on the Design plans. These quantities are supplied by the District and will be included in their "Summary of Quantities" for the complete Contract plans. The following General Note shall also be included on the Contract plans (repeated from Section 3.1):

> *The embankment configuration with 60 ft. berm shown shall be the minimum that must be placed and compacted prior to construction of the abutments.*

On the General Plan sheet, the end of approach slab at the abutment shall be indicated.

3.14.2 Slope Walls - General

For complicated slope wall configurations, an additional sheet should be included with the General Plan sheet showing sufficient dimensions and elevations to clarify the details.

3.14.3 Abutment Berms

Flattened berms at the top of abutment slope wall treatments are not required and may hinder inspection access of the abutment caps and diaphragms. However, when such berms are required to address bridge configuration issues, (see Figures 2.3.6.3.3-6, 2.3.6.3.3-7, and 3.14.4.1-3), a note shall be placed on the plans indicating that all abutment berms shall be sloped ½ in. per ft. to drain.

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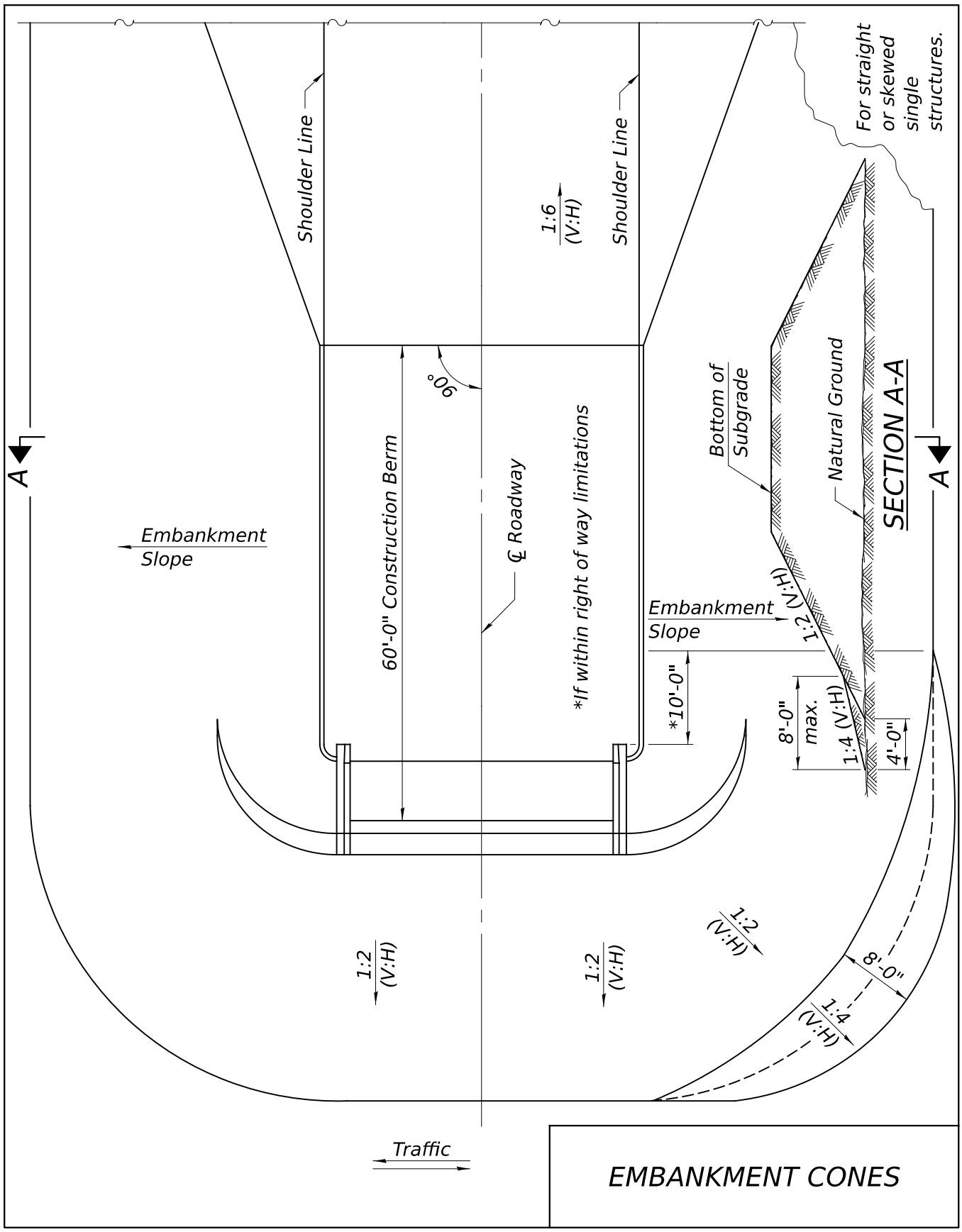


Figure 3.14.1-1

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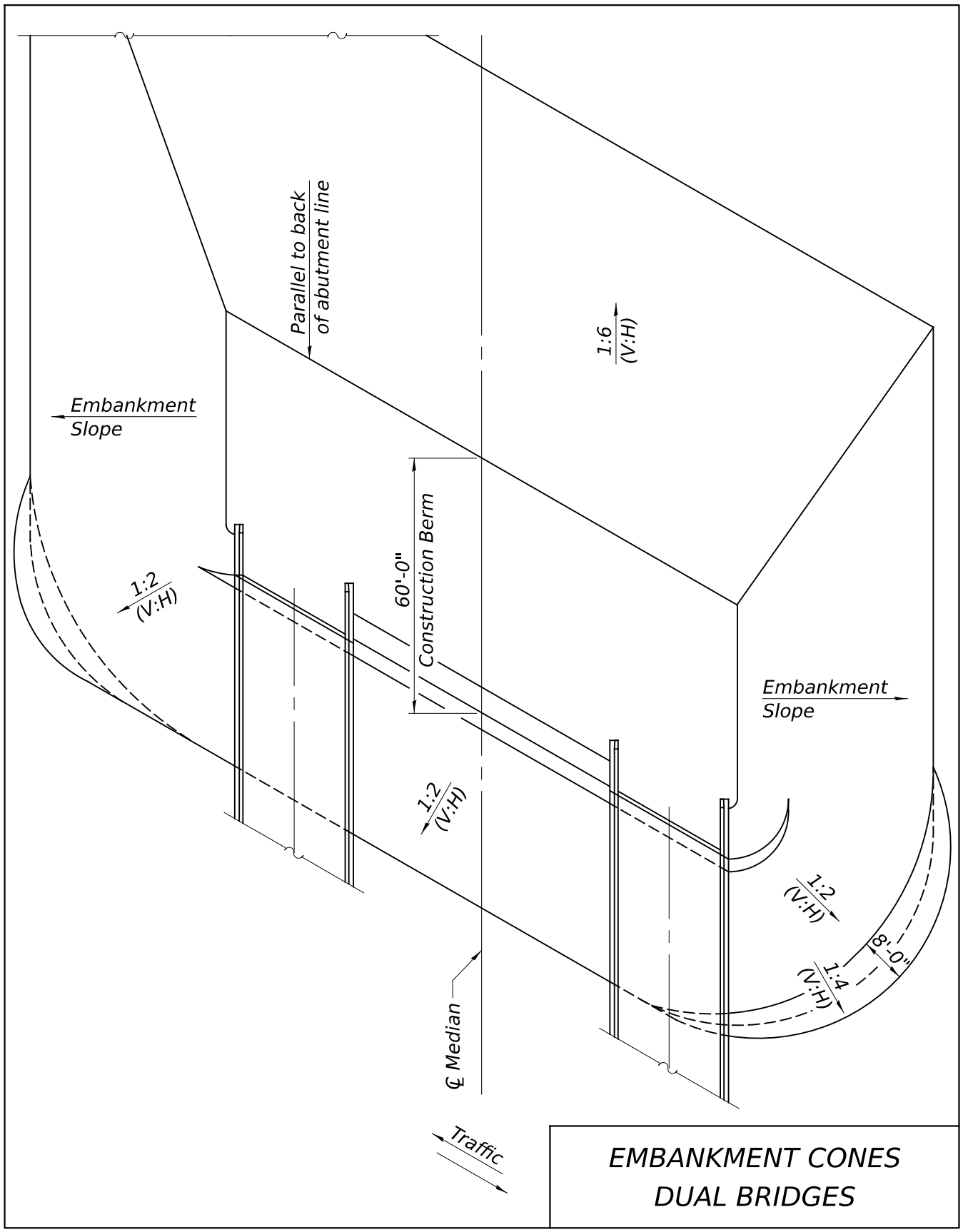


Figure 3.14.1-2

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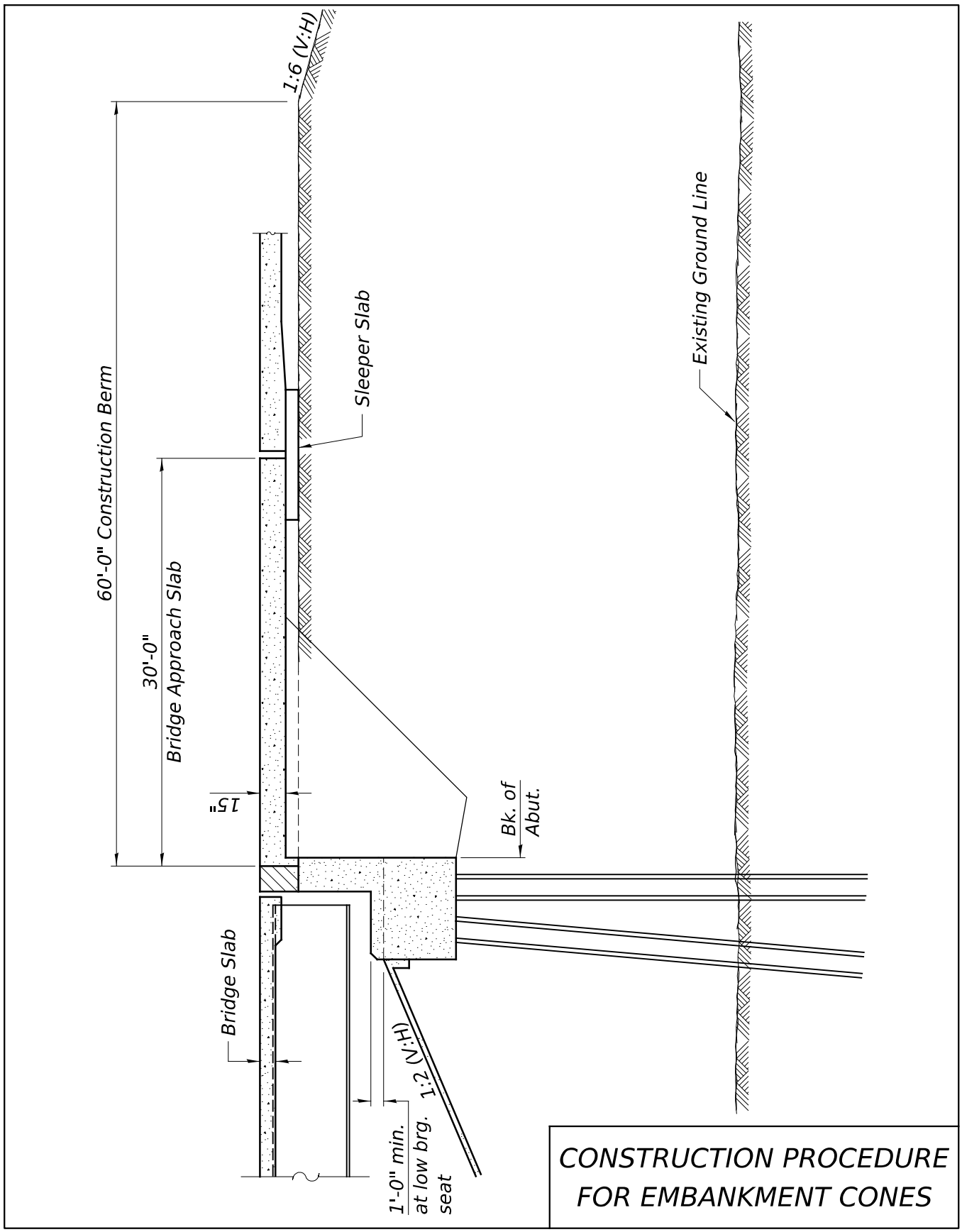


Figure 3.14.1-3

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3.14.4 Slope Walls (Grade Separations)

3.14.4.1 Concrete

The thickness of concrete slope walls shall be 4 in., and they shall be reinforced with welded wire reinforcement, 6 in. x 6 in. - W4.0, weighing 58 lbs. per 100 sq. ft. Cost of the welded wire reinforcement is included in the cost of the slope wall. Under a single structure, the slope wall shall be paved 2 ft. beyond the outside limits of the superstructure if the structure does not have drains above the slope wall and 5 ft. if the structure does have drains above the slope wall. The lateral edges shall be provided with cut-off walls for control of possible erosion. Cut-off walls shall be placed at approximately 10 foot centers, see Figure 3.14.4.1-1.

The embankment slope between dual structures shall not be paved when the distance between adjacent slope wall edges exceeds 10 ft.

Paved slope walls shall be separated from contact with a pier by a 2 in. preformed joint filler. This is detailed in Figure 3.14.4.1-1. No preformed joint filler is required between a paved berm and the front of an abutment cap except for integral structures as shown in Figure 3.14.4.1-2.

Details for a concrete slope wall with a paved berm are given in Figure 3.14.4.1-3.

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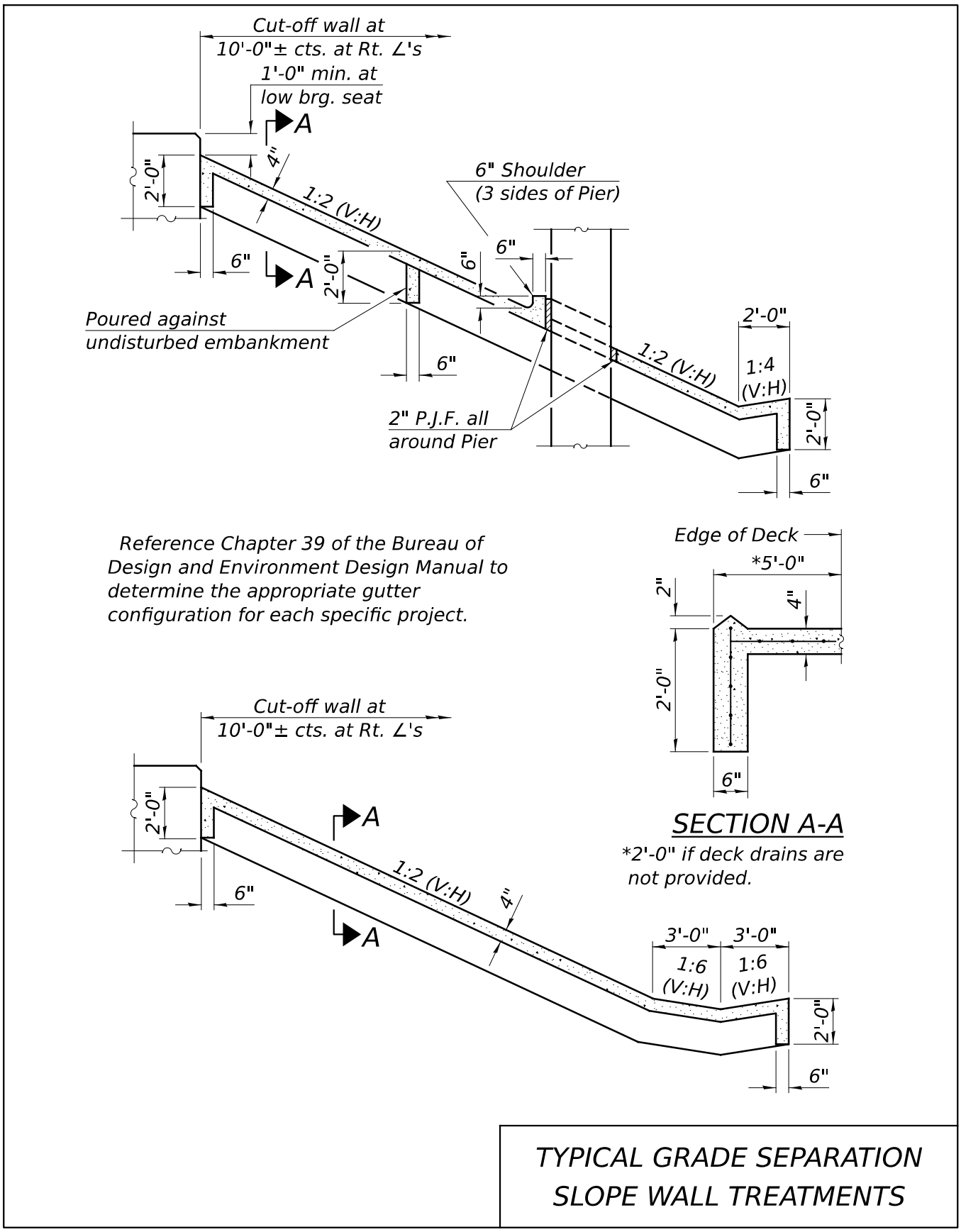
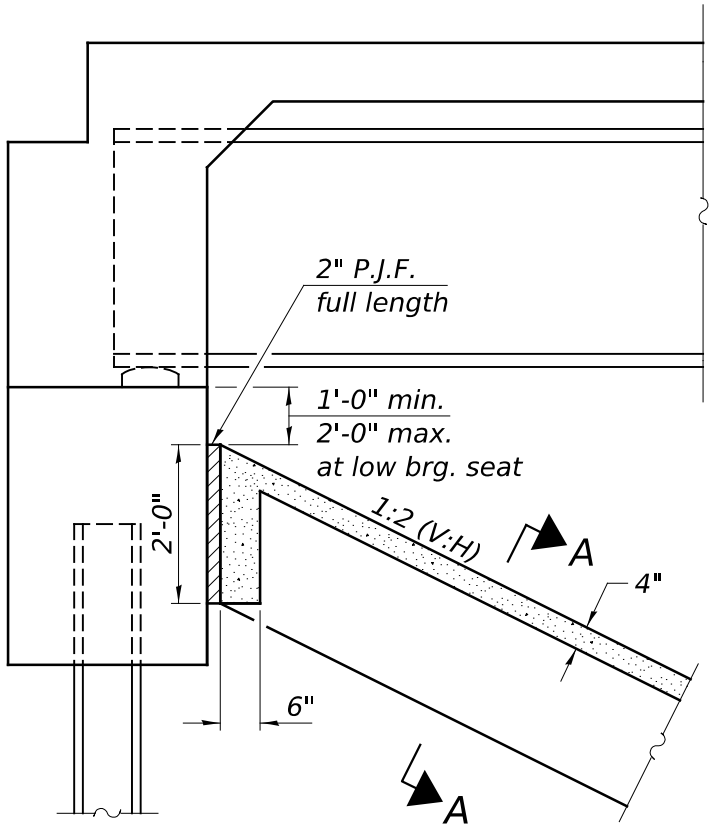
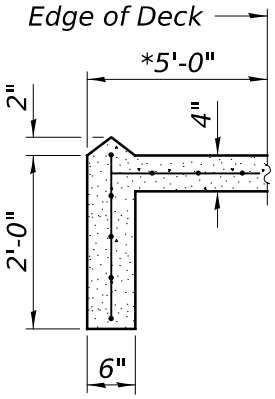


Figure 3.14.4.1-1

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INTEGRAL ABUTMENTS



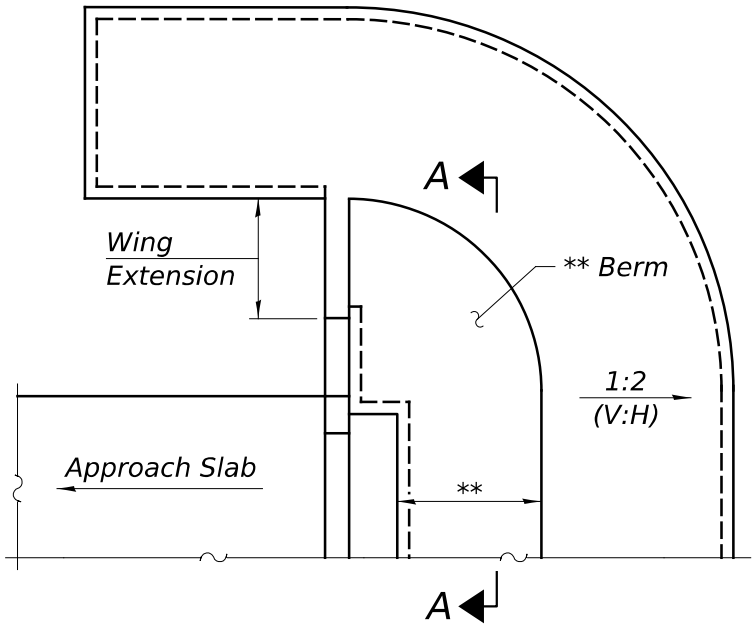
*2'-0" if deck drains are not provided.

SECTION A-A

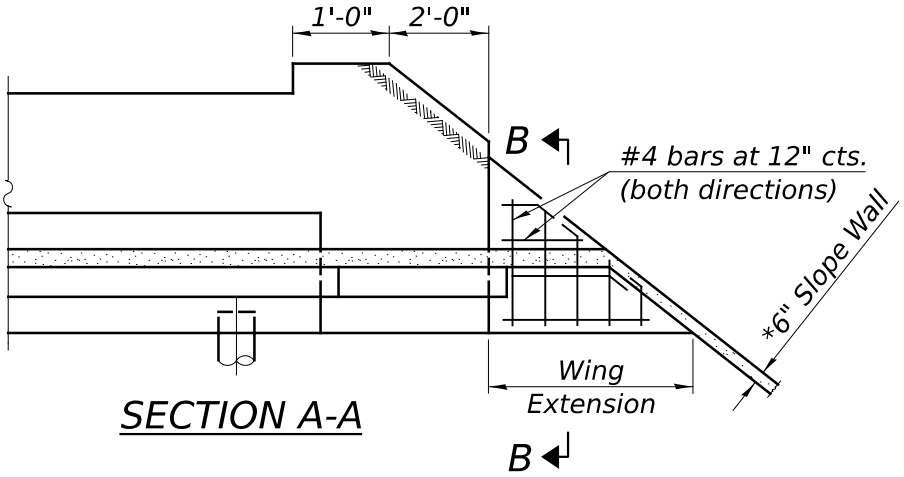
SLOPE WALL TREATMENTS
GRADE SEPARATIONS

Figure 3.14.4.1-2

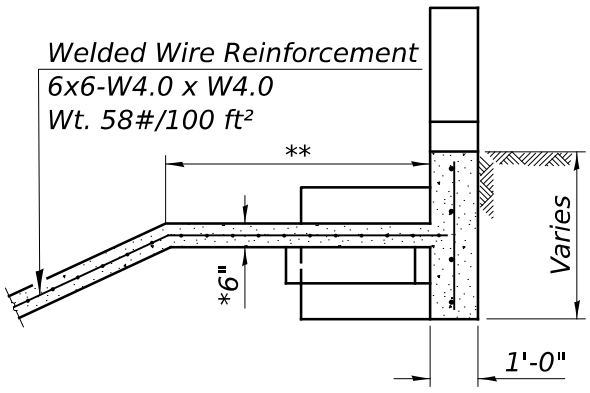
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PARTIAL PLAN OF SLOPE WALL
With wings at right angles to ℄ of Roadway



SECTION A-A



SECTION B-B

* 4" at Grade Separations

** Flat abutment berm shown. See Figure 3.14.4.1-1 & Figure 3.14.4.1-2 for preferred geometry at abutment face.

DETAILS OF SLOPE WALL WITH BERM

Figure 3.14.4.1-3

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3.14.4.2 Bituminous

Figure 3.14.4.2-1 presents details for a bituminous slope wall. Bituminous slope walls are typically used at railroad crossings; when warranted after considering constructability and cost, concrete slope walls may be used at railroad crossings if preferred for in-service maintenance/inspection accessibility.

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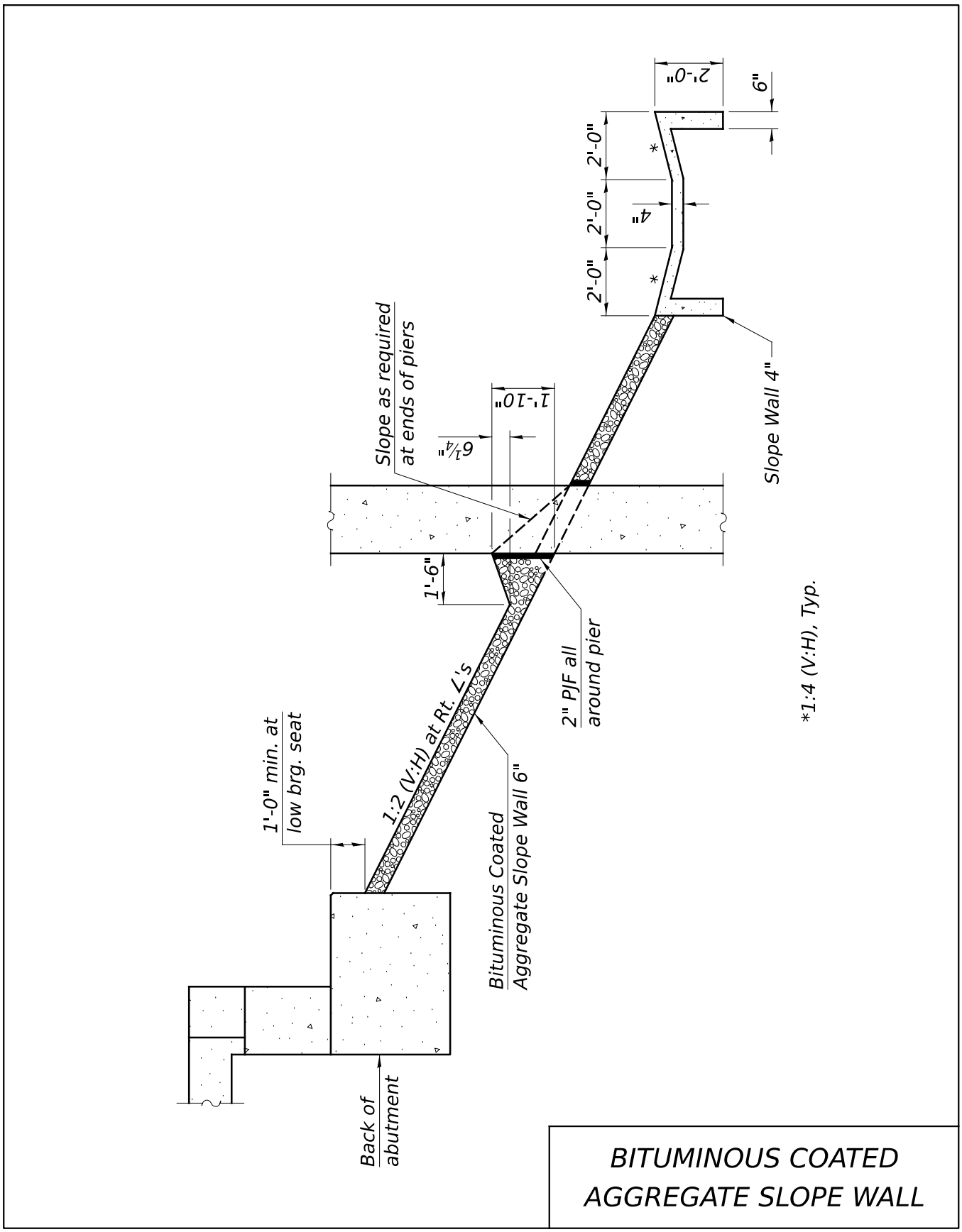


Figure 3.14.4.2-1

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3.14.5 Slope Protection Systems (Stream Crossing)

Typically, stone riprap is used for the protection of embankments of structures which cross streams. See Section 2.3.6.3.3 for riprap treatments at toes and flanks as well as proper layer thicknesses and dimensions for each class of riprap.

Concrete slope walls may be used to solve specific protection problems. The minimum thickness for a paved concrete slope wall subject to stream flow shall be 6 in. and it shall be reinforced the same as grade separation slope walls. The toe of the slope wall shall be detailed at a constant elevation.

The layout of a slope protection system will generally be as detailed in Figure 2.3.6.3.3-1. Figure 3.14.5-1 deals with specific details for a paved concrete slope wall at a stream crossing. Cut-off walls shall be placed every 10 feet.

The top of the slope protection may terminate at 2 ft. – 0 in. above design high water.

At stream crossings only, the following note shall be placed on the plans (repeated from Section 3.1):

> *Layout of the slope protection system may be varied to suit ground conditions in the field as directed by the Engineer.*

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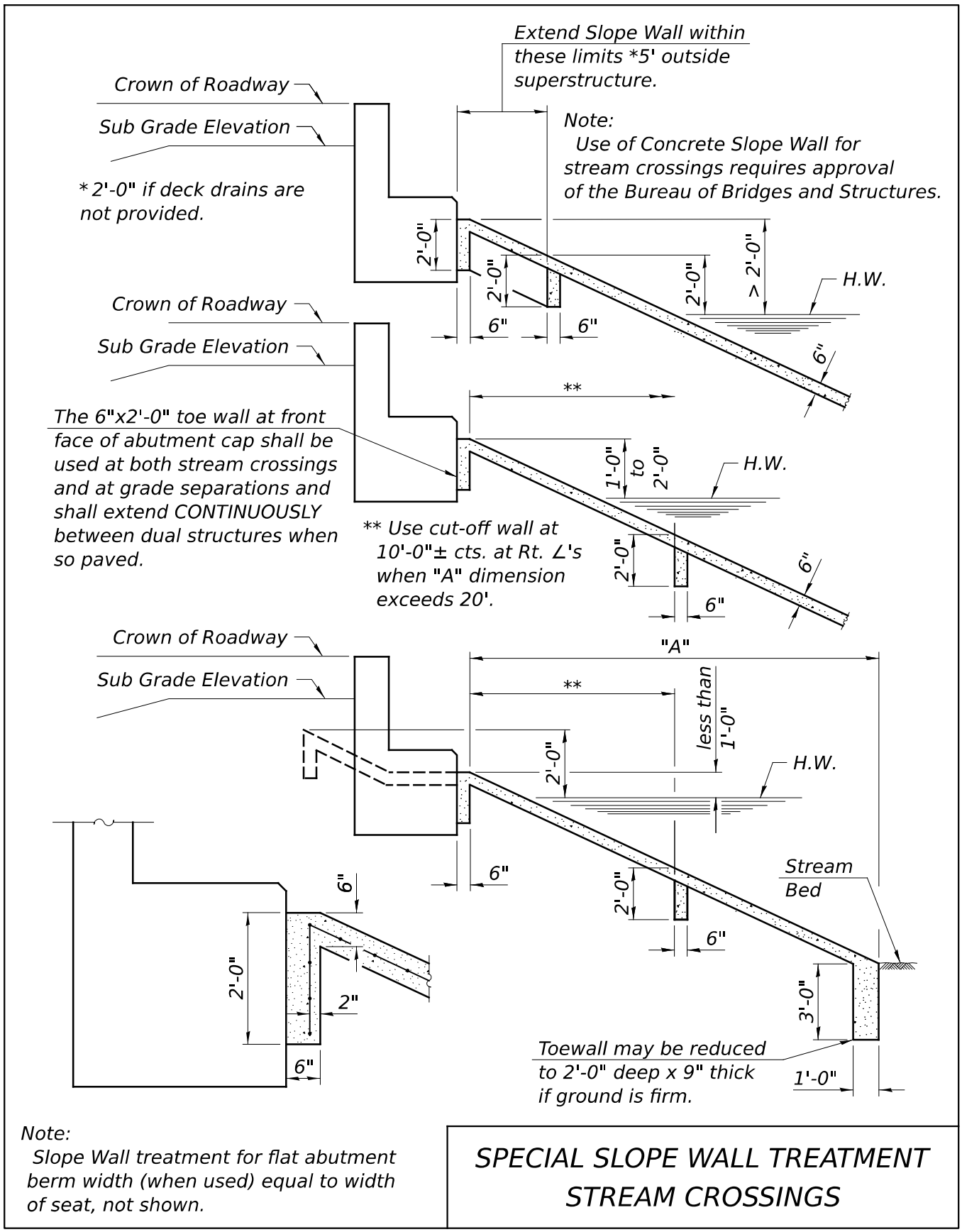


Figure 3.14.5-1

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3.15 Seismic Design (Force-Based)

For current Displacement-based seismic design approaches, and to scope whether a bridge project qualifies for a force-based or displacement-based design approach, refer to the latest seismic memorandums and manuals found at the primary Bridges & Structures Page.

The design guidance herein shall be applicable to all structures being designed using an LRFD force-based design approach when approved by the Department.

3.15.1 Introduction

The Department's primary objective when designing bridges to resist earthquake loadings (and other extreme event loading cases) is loss of span shall be prevented. Span loss equates to potential loss of life during a significant or moderate seismic event. However, damage to a bridge, which could be significant during an earthquake is expected and explicitly designed for. In order to achieve a balance between the prevention of loss of life or span and the practical reality of economics, the Department has devised a straightforward strategy for the seismic design of bridges in Illinois. The seismic design strategy for the State can also be termed a general plan or framework for Earthquake Resisting Systems (ERS's). There are several variations in the specifics of the plan which primarily depend upon bridge type.

The Department's ERS framework was primarily developed based upon seismic design provisions, options and concepts outlined in the AASHTO LRFD Bridge Design Specifications (but is also applicable to bridges retrofitted or designed according to previous non-LRFD Codes). Some minor modifications of the AASHTO LRFD Bridge Design Specifications and recommendations from the AASHTO Guide Specifications for LRFD Seismic Bridge Design, which are described below in more detail, are also incorporated into the ERS plan to accommodate bridge configurations and construction methods that are typical for Illinois.

3.15.2 Design Earthquake, and Seismic Performance Zones

New bridges on the State System and, as appropriate, new bridges on the Local System shall be designed according to the AASHTO LRFD Bridge Design Specifications for the 1000 yr. design return period earthquake (or a 7% probability of exceedance in 75 yrs.). Design accelerations shall be computed using the method found in Section 3 of the AASHTO LRFD Bridge Design Specifications . The full suite of 1000 yr. acceleration contour maps for Illinois can be found in Section 3 of the AASHTO LRFD Bridge

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Design Specifications . Derived approximate acceleration contour maps which include the effects of Soil Site Class for a 0.2 sec. period are given in Section 2.3.10. These derived maps should primarily be used for planning purposes to assist in the determination of the Seismic Performance Zone. Seismic Performance Zones (SPZ) are delineated according to Table 3.15.2-1 which is a reprint of Table 3.10.6-1 from the AASHTO LRFD Bridge Design Specifications . Refer to the "Seismic Design" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design for information on the calculation of a 1000 yr. design earthquake response spectrum and the determination of SPZ.

When feasible, bridges which are rehabilitated or reconstructed should, at a minimum, meet the requirements of the non LRFD Code for the 1000 yr. design event (especially on the Interstate System). However, there are many factors which contribute to the level of seismic retrofitting which would be required for any particular project. Further guidance on the required level of conformance to the Department's ERS Strategy for rehabilitated or reconstructed bridges can be found in Sections 2.1.2, 2.3.10 and 3.15.7.

LRFD Seismic Performance Zones
for 1000 yr. Design Return Period Earthquake

SPZ	$S_{D1} = F_vS_1$ (1 Sec. Accel.)
1	$F_vS_1 \le 0.15g$
2	$0.15g < F_vS_1 \le 0.30g$
3	$0.30g < F_vS_1 \le 0.50g$
4	$0.50g < F_vS_1$

Table 3.15.2-1

The load factor for live load, γ_{EQ}, reflects the probability that the HL-93 loads are applied to the bridge in the correct locations to produce maximum force effects at the exact same time of the earthquake. The probability that trucks and passenger vehicles will be in the exact locations to cause maximum force effects is very low, but can occur on roadways with large average daily traffic. Therefore, the following load factors shall apply:

- For bridges where ADTT > 1500, γ_{EQ} shall be taken as 0.5. This ADT corresponds to an average of one truck per minute on the structure. In a seismic event of duration of one minute, the truck and lane loading is assumed to have a 50% probability of generating HL-93 force effects.
- For bridges where ADTT ≤ 1500, γ_{EQ} should be taken as zero, at the discretion of the designer. Should the designer elect to use a nonzero value, a value of greater than 0.5 is not warranted.

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3.15.3 Earthquake Resisting Systems - General

3.15.3.1 Description

The basic IDOT philosophy for seismic bridge design is to allow certain levels of damage or element failures to occur at planned locations in a structure during an earthquake which will not cause a loss of span. Optimally, this is achieved through what can be termed as levels of seismic structural redundancy which dissipate seismic energy in key parts of bridges in succession as they fail and alter the response of the structure. The lowest level of redundancy is the “weakest link” or “lowest amperage fuse” in a bridge while successive levels are stronger links or fuses which do not have to resist the full energy of an earthquake.

The first fuse location in a bridge where damage is allowed to occur, i.e. the “weakest link”, is at the connections between the superstructure and the substructure. These connections shall only be designed to carry a nominal lateral force (percentage of seismic base shear) without regard to other potential failure modes. The nominal lateral force is determined as 20% of the dead load reaction (R-Factor = 1.0) for Seismic Performance Zones 1 to 4 (LRFD) and is independent of design earthquake return period. See Section 3.7.3 for details. Once the first fuse is “blown”, the seat widths at supports in both the transverse and longitudinal directions shall be adequate to prevent a span from falling. Adequate seat widths (support lengths) are the second tier of seismic redundancy in the Department’s ERS plan.

Once the superstructure has broken away from its connection to the substructure on one or more of its supports, the fundamental period of the bridge will, in all likelihood, increase and lateral seismic load will be transmitted to the substructure primarily through friction. If the structural period increases, the seismic energy and base shear typically decrease. Even if the structural period is not significantly increased after the connection between the superstructure and substructure fails, there could be parts of the superstructure that “go plastic” or buckle which will also help dissipate seismic energy. Bracing between steel girders in superstructures is an example. This type of damage to the superstructure is allowed according IDOT’s ERS strategy.

Seismic design of the substructures and foundations of a bridge encompass the third tier of seismic redundancy in IDOT’s general ERS plan. The “amperage level” of these fuses is somewhat higher than that of the first and second tier fuses. Generally, substructure types which have the capacity to form a plastic hinge without causing a span to collapse in combination with foundations which remain elastic during a significant seismic event are optimal. The best example of this would probably be a bridge with multiple round column bents and steel H-pile foundations for both the piers and abutments. However,

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there are examples of bridge types throughout the State which are not of "optimal seismic configuration". Strategies have been developed to help deal with these cases in the Department's ERS plan.

For a 1000 yr. design return period earthquake, IDOT's basic seismic design philosophy concerning substructures and foundations can be stated thus for typical bridges:

> *Once the first and second tier seismic redundancies have been fully engaged, the third tier will adequately resist a significant portion, but not all, of the original seismic energy transmitted to a bridge during an earthquake.*

3.15.3.2 Range of Applicability

The Department's ERS design strategy or plan is generally intended for "regular" Illinois bridges which can be designed assuming the first mode of vibration is the dominant response to a seismic loading. However, bridges which are "irregular" can and should still be designed according to the general principles of IDOT's ERS plan. These structures will usually require multi-modal analysis and may require some design and/or detailing techniques which are beyond the scope of this manual.

The definitions of regular bridges for seismic design in Illinois are given in LRFD Table 4.7.4.3.1-2 of the AASHTO LRFD Bridge Design Specifications. Regular bridges are defined the same regardless of design earthquake return period. Generally, bridges are regular if there are 2 to 6 spans (single span bridges are always regular), do not have span length ratios which are significantly large, do not have pier stiffness ratios which are significantly large, and are not overly curved. Multi-unit bridges, i.e. those structures with at least two distinct sets of continuous spans which share at least one simple support at a pier, are considered irregular by the Department and require multi-modal analysis in Zones 2, 3 and 4.

Most bridges on the State System and many on the Local System should be classified as "Recovery" (formerly "Essential" in past LRFD Specifications). Some Local Bridges may be classified as "Ordinary" (formerly "Other" in past LRFD Specifications). Very few bridges should be classified as "Critical". AASHTO LRFD Bridge Design Specifications Table 4.7.4.3.1-1 prescribes that multi-modal analysis is required for regular bridges classified as Recovery in Zones 3 and 4. For regular bridges classified as Ordinary in Zones 2, 3 & 4 and regular bridges classified as Recovery in Zone 2, only a single mode analysis is required according to the AASHTO LRFD Bridge Design Specifications. IDOT, however, generally does not require multi-mode analysis of regular bridges classified as Recovery in Zones 3 & 4. A single mode analysis for a regular bridge in Illinois is typically more than adequate for design purposes, regardless of Seismic Zone, unless the shape of the computed first mode in the transverse direction does not reasonably resemble a half sine wave.

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The methods and strategies of the Department's ERS plan presented in subsequent sections are intended to be applicable to all Seismic Zones regardless of the design earthquake return period.

Major and/or critical bridges in highly seismic regions of Illinois are typically designed for an earthquake return period of 2500 years. As such, they require the highest level of sophistication in their planning and design for seismic effects and are beyond the scope of this manual. The design methods and provisions of the AASHTO Guide Specifications for LRFD Seismic Bridge Design should be considered for the design of major and/or critical bridges. This guide specification encompasses a "displacement based" design approach which may be appropriate for these bridges as well as some which are not considered major and/or critical structures in far Southern Illinois (SPZ 4). Consult the BBS if there is a question as to whether the AASHTO Guide Specifications for LRFD Seismic Bridge Design should be considered for a particular project.

If a temporary bridge, expected to be in service for less than five years, is required for a project which lies in a region of moderate to high seismicity, it should typically be designed as per Section 513 of the IDOT Standard Specifications for Road and Bridge Construction.

3.15.4 Earthquake Resisting Systems - Specifics

The subsections below present an overview of the design considerations and details which are required to implement the Department's general ERS plan for specific situations. It is well acknowledged that seismic design can be somewhat of an intuitive form of structural engineering lacking in precise numerical solutions. This is reflected in the provisions of the current AASHTO Specifications. The Department's detailed concept of tiered seismic redundancy is intended to aid bridge designers in producing designs capable of achieving a consistent level of seismic resistance.

When used in conjunction with the relatively simple dynamic and static analysis techniques given in Section 3.15.4.3, Section 3.15.6, and the "Seismic Design" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design; the prescribed practices given in Sections 3.15.4.1, 3.15.4.2 and 3.15.5 concerning the detailing of superstructure to substructure connections, seat width (support length) requirements, and the detailing of columns, walls and foundations should produce a reasonably high probability that span loss is prevented during a moderate or major earthquake.

All single span bridges, and multi-span bridges located in Seismic Performance Zone (SPZ) 1 with $S_{D1} < 0.10$ are only required to implement Sections 3.15.4.1 and 3.15.4.2 to fulfill the Department's ERS strategy due to their lower seismic risk. Multi-span bridges in SPZ 1 with $S_{D1} \geq 0.10$ and $S_{D1} \leq 0.15$ shall

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meet the requirements of the sections noted directly above as well as the minimum requirements (without seismic force analysis) for the detailing of plastic hinge regions in concrete members according to Articles 5.11.4.1.4 and 5.11.4.1.5 of the AASHTO LRFD Bridge Design Specifications with the exception that the maximum spacing of confinement reinforcement shall be 6 in. These requirements in the AASHTO LRFD Bridge Design Specifications were added in 2008 because the design accelerations for the upper portion of SPZ 1 are similar to the previous SPZ 2 when the previous design earthquake return periods were 500 yrs. Increased embedment of piles into caps and footings is also required. The concepts and figures presented in Section 3.15.5 should be used as a guide for meeting the minimum detailing requirements of the upper portion of SPZ 1.

3.15.4.1 Superstructure to Substructure Connections: Level 1 Redundancy

Section 3.7.3 presents the details for designing the “nominal” or “fuse” connections between the superstructure and substructure for elastomeric and fixed bearings. Side retainers are thought of as part of this connection or fuse. The lateral earthquake design force for the connections, as prescribed in Section 3.7.3, is 20% of the dead load reaction of the superstructure (R-Factor = 1.0) at each support regardless of the SPZ or earthquake return period.

The methodology for the determination of the lateral design forces is the same for bridges with either prestressed concrete I-beam superstructures or steel superstructures. IDOT standard drawings for prestressed concrete construction along with Departmental Base Sheets present the majority of the details for the connections between the superstructure and substructures of PPC I-beam bridges. Side retainers, anchor bolts and pintles used for bearings on prestressed concrete I-beams and bulb-T’s shall all also be designed according to Section 3.7.3. At fixed piers, the IDOT standard side retainer details for prestressed concrete fascia beams shall still be specified due to constructability reasons but shall be neglected in calculating seismic resistance. The #8 dowel bars used to connect the diaphragm to the cap beam at fixed piers shall be designed using Figure 3.4.9-4 and Figure 3.4.9-5.

There are no special seismic detailing requirements for superstructure to substructure connections for integral abutments with either steel or prestressed concrete superstructures.

There are no special seismic detailing requirements for superstructure to substructure connections for slab bridges.

Concrete deck beam bridges shall use the standard details found in Section 3.5. These details are sufficient for all deck beam bridges regardless of span length or beam size.

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3.15.4.2 Support Lengths: Level 2 Redundancy

In lieu of using the equation below to obtain an exact seat width or support length requirement, the values found in Table 3.15.4.2-1 may be used for all bridges, regardless of return period or location. The values presented in the table for the 1000 yr. event were calculated using the required seat width equation recommended by the NCHRP 12-49 project. The equation exhibits greater compatibility with Illinois' ERS strategy. The values in the table may be more conservative than those obtained by use of the equation below or than that of the AASHTO LRFD Bridge Design Specifications. Pier heights of 25 ft. (which includes crash wall and cap beam, top of pier to depth-of-fixity in soil, etc. as appropriate) and a skew of 30 degrees were used to generate the table. Other assumptions are listed in the footnotes. For more precise determinations or for structures beyond the limits assumed for Table 3.15.4.2-1, required seat widths shall be calculated. For the 1000 yr. event, required support width, N in in., should be calculated with the equation below.

$$N = \left[3.94 + 0.0204L + 0.084H + 1.087\sqrt{H}\sqrt{1+\left(2\frac{B}{L}\right)^2} \right] \frac{1+1.25F_vS_1}{\cos\alpha} \qquad (3.15.4.2\text{-}1)$$

Where:

L	=	Typically the length between expansion joints (ft.).
H	=	Height of tallest substructure unit between expansion joints, including units at the joints (ft.).
B	=	Out-to-out width of superstructure (ft.).
α	=	Skew angle (°).
F_vS_1	=	One second period spectral response coefficient modified for Site Class.
B/L	=	Not to be taken greater than ⅜.

The required seat width, N, is measured along the beam from the edges of piers or abutments to the end of the beam in the longitudinal direction. In the transverse direction, N is measured from the edges of piers or abutments to the centerline of the fascia beam. According to the AASHTO LRFD Bridge Design Specifications, only the longitudinal seat widths are required to be provided. However, to implement the Department's ERS strategy, both directions shall be provided with adequate seat widths such that the superstructure has enough room to "ride out" an earthquake on its supports once the bearings and their connections fail.

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Seat Width Requirements at Piers and Abutments (1000 yr. event)*				
Length (Exp. Joint to Exp. Joint) (ft)	LRFD Zone 1 N Req. (in)	LRFD Zone 2 N Req. (in)	LRFD Zone 3 N Req. (in)	LRFD Zone 4 N Req. (in)
L ≤ 100	20	24	28	32
100 < L ≤ 200	23	27	32	37
200 < L ≤ 300	26	30	36	41
300 < L ≤ 400	29	33	39	45

*Notes:
1. Piers Heights ≤ 25 ft.
2. Skew Angle ≤ 30°
3. B/L Ratio of 0.375
4. F_vS_1 at Upper Zone Boundary for Zones 1 to 3
5. F_vS_1 = 0.7 for Zone 4

Table 3.15.4.2-1

For single span bridges, L is the span length of the structure. (There are no seat width requirements if the abutments are integral for single span structures.) For continuous multi-span integral or semi-integral abutment bridges, the required seismic seat widths at piers in the transverse direction may be divided by one and one-half (1½) with no seat width requirements at the abutments. Approach pavements are not included in the length of the structure for integral or semi-integral abutment bridges. The added stiffness provided by the combination of integral abutments and the superstructure allows this reduction. For piers supporting precast prestressed concrete I and Bulb-T beams, regardless of abutment type, the overall pile cap width may be limited to 48 in.

There are no special seismic seat width requirements for slab bridges.

There are no special seismic seat width requirements for deck beam bridges. The standard anchor bolts provide a resistance much greater than that required to assume fuse behavior. As the rods cannot reasonably be expected to fuse, the connection need not satisfy minimum seat width requirements in any direction.

3.15.4.3 Substructures and Foundations: Level 3 Redundancy

Substructure units shall be designed for the full effects of the earthquake loading. Applicable response modification factors are described below.

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Response Modification Factors (R-Factors) for seismic design are given in AASHTO LRFD Bridge Design Specifications Table 3.10.7.1-1. Typically, R-Factors are only applied to elastic design moments from an equivalent static earthquake analysis. Guidance on the design of typical substructure and foundation configurations is given below. Included are R-Factors and ϕ factors to employ as well as suggested general analysis techniques. Seismic detailing requirements and other guidance are given in Sections 3.15.5 and 3.10. Note that the R-Factors given below are based upon typical bridges in Illinois being classified as "Recovery" (Operational Category, formerly "Importance Category") in the AASHTO LRFD Bridge Design Specifications. Some local bridges may be classified as "Ordinary". See Section 3.15.8 for further information. Generally, most bridges in Illinois would not be classified as "Critical". The BBS should be contacted if there is a question about a bridge's Operational Category.

3.15.4.3.1 ϕ Factors

For a 1000 yr. design return period earthquake, an LRFD extreme event load case, it is permissible to use the full resisting nominal capacity of certain elements to achieve practical and economical designs that will satisfy the Department's primary objective of preventing span loss. Columns, walls, and drilled shafts may be designed with a ϕ factor of 1.0 for axial force and moment. The geotechnical resistance factor (ϕ) for pullout of piles in tension may also be set to 1.0 for the 1000 yr. design return period earthquake. The use of these higher ϕ factors is considered acceptable based upon the amount of damage permissible to the Department for a seismic event and the level of isolation anticipated between the superstructure and substructure as a result of Level 1 of the ERS framework. For shear, a ϕ factor of 0.9 should be used for columns and walls (as is currently specified in the AASHTO LRFD Bridge Design Specifications) and drilled shafts.

3.15.4.3.2 Foundation Issues

The pile design table provided in the Structure Geotechnical Report (SGR) should provide the geotechnical factored or allowable resistances available under axial compression for seismic loadings. For LRFD, the geotechnical extreme event resistance factor for pile strength is 1.0.

The simplest (and most conservative) method to convert the tabulated factored or allowable resistances available for non-seismic pile design to seismic pile design is to multiply the non-seismic design values by 2.0 for LRFD. This method, however, amplifies the effect of the downdrag loading which may be included in the determination of geotechnical resistance. It is more accurate to only amplify the required bearing and geotechnical losses. Conversion by either method is acceptable.

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Even though seismic loads may govern the design of the foundation, the factored or allowable resistances available reported on the Contract plans should not be based upon the seismic resistances. The factored or allowable resistances available reported on the Contract plans should be based upon the geotechnical factored or allowable resistances reported in the SGR for non-seismic loadings.

When there is a potential for liquefaction at a bridge site, the guidance provided below should still generally be adhered to although some modifications, depending on the severity to which the soil has the potential to liquefy, may be required. As noted in Section 2.3.10, the potential for liquefaction shall be evaluated for all projects according to the requirements of their SPZ. See also Section 3.10 for further guidance.

3.15.4.3.3 Circular and Trapezoidal Column Bents with Cap Beam, Crash wall, Footing and Piles

This substructure and foundation combination is considered optimal in regions of moderate to high seismicity by the Department as any plastic deformation that may occur due to an earthquake is typically above ground and “inspectable”. The columns will theoretically form plastic hinges just above the crash wall and just below the cap beam if detailed properly. An R-Factor of 3.5 should be used for the flexural design of the columns. The piles and crash wall (for bending about the weak axis at the wall-footing interface) may be designed for the lesser of R/2 for Zone 2, the full elastic base shear force (R = 1.0) for Zones 3 and 4, or the shear required to cause the columns to form plastic hinges.

For a 1000 yr. design earthquake, the effective moment of inertia of the columns should be taken as half of the gross moment of inertia (i.e. cracked).

The piles, either H-piles or metal shell, are typically assumed to deform in reverse curvature (longitudinally and transversely) for analysis and design purposes with the length from the depth-of-fixity to the bottom of the footing serving as the “column height.” H-piles are considered optimal in regions of moderate to high seismicity.

3.15.4.3.4 Solid Wall and Hammerhead Bents with Footing and Piles

For analysis and design purposes in the transverse direction, solid walls should be considered as very stiff shear walls or “rigid links” to the weaker (more flexible) piles. In the longitudinal direction, they should be modeled and designed as cantilevers. The weak link (fuse, plastic hinge) should be designed to occur in the wall just above the footing and not in the piles. In the transverse direction, an R-Factor of 1.0

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should be used for design of the wall while an R-Factor of 1.5 is appropriate for the designing the wall in the longitudinal direction. The piles may be designed for the full elastic base shear force (R = 1.0) or the shear required to cause plastic hinging in the wall stems.

The design of hammerhead piers can be somewhat different than that of a solid wall which spans the full transverse width of the superstructure. Because of the narrower width of the stem inherent to hammerhead piers, the axial stresses in the stem (particularly at the top of the stem or base of the "hammerhead") may be considerably higher than for a conventional wall type pier warranting it to be designed as a compression member subjected to biaxial bending. An R-Factor of 1.5 may be used for the stem for both the longitudinal and transverse directions. The piles should be designed according to that prescribed above for wall solid wall piers.

For a 1000 yr. design earthquake, the effective moment of inertia of the wall or stem in the longitudinal direction should be taken as half of the gross moment of inertia (i.e. cracked).

3.15.4.3.5 Individually Encased and Solid Wall Pile Bent Piers

The seismic design and analysis methods for individually encased pile bents and solid wall encased pile bents are very similar. Individually encased bents, however, are more flexible than solid wall bents in both the transverse and longitudinal directions. The theoretical plastic hinges for both types can tend to form in areas which are not readily inspected after an earthquake has occurred. As such, they should be designed with an R-Factor of 2.0 for both the longitudinal and transverse directions.

3.15.4.3.6 Drilled Shaft Piers

There are five basic types of drilled shaft piers typically built in Illinois. These are

i. Individual column drilled shaft bents
ii. Solid wall encased drilled shaft bents
iii. Individual column drilled shaft bents with web walls
iv. Individual column drilled shaft bents with crash wall
v. Drilled shaft bents with transfer beam

and base sheets for these types (P-DS, P-DSSW, P-DSWW, P-DSCW, P-DSTB, respectively) are found in the "Piers" IDOT CADD Bridge Cell Library.

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For a 1000 yr. design earthquake, the effective moment of inertia of the columns and/or shafts, as appropriate, should be taken as half of the gross moment of inertia (i.e. cracked).

For individual column drilled shaft bents, plastic hinges may theoretically occur in the drilled shaft portion of the pier (i.e., below ground) which may not be inspectable. Consequently, an R-Factor of 2.0 should be used when designing this pier type. If analysis shows that plastic hinges are likely to form above ground, an R-Factor of 3.5 may be used.

Solid wall encased drilled shaft bents (P-DSSW) are likely to develop plastic hinges below the ground line. As such, an R-Factor of 2.0 should be used for design.

Individual column drilled shaft bents with web walls (P-DSWW) are expected to behave similar to solid wall encased drilled shafts (P-DSSW). The upper web walls, because they are connected to the columns, will behave like a semi-rigid wall. In the longitudinal direction, the stiffness of the lower web walls should be ignored since they are not connected to the shafts but should be considered in the transverse direction as they will inherently restrain deflection of the shafts. An R-Factor of 2.0 is recommended for this pier type as it is anticipated that plastic hinges will occur in areas that are not readily accessible. However, a value of 3.5 is also acceptable if the analysis shows that plastic hinges are likely to develop near the base of the upper web walls.

Individual column drilled shaft bents with crash walls (P-DSCW) or transfer beams should be analyzed and designed similar to multiple column bents with footings, crash walls and piles. An R-Factor of 3.5 should be used for the column design. An R-Factor of 1.0 should be used for flexural design of crash walls and transfer beams. The designer should investigate whether it is possible for plastic hinges to form in the drilled shafts (below ground) due to seismic loadings in both the transverse and longitudinal directions. The drilled shafts should be designed with enough strength to ensure that the plastic hinges form above the crash wall or transfer beam.

In any of the above cases involving a column in the upper portion and a drilled shaft in the lower portion of the pier, it is also permissible to design the drilled shafts for the forces corresponding to plastic hinging of the columns to achieve economical and practical designs.

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3.15.4.3.7 Abutments

The foundations of abutments should typically be designed for the full elastic base shear (R = 1.0). The passive pressure of the soil bearing against the abutment and wingwalls may be utilized at the designer's discretion.

3.15.5 Overview of Seismic Detailing Requirements for Substructures and Foundations

An overview of seismic detailing requirements for substructures and foundations is given in this section. There are no special seismic detailing requirements for prestressed concrete and steel superstructures according to the Department's ERS plan. Detailing requirements for the connections of the superstructure to the substructures are given in Section 3.7.3. Seat width requirements are given in Section 3.15.4.2.

AASHTO LRFD Bridge Design Specifications Articles 5.11.2 through 5.11.4 contains a number of detailing and design provisions for reinforced concrete members. These should also be referenced for further guidance.

3.15.5.1 Circular and Trapezoidal Column Bents with Cap Beam and Crash wall

Figures 3.15.5.1-1 through 3.15.5.1-4 illustrate the types of special seismic details required in columns, caps beams and crash walls for circular and trapezoidal column bents. The areas in and around where plastic hinging will occur in columns are termed "seismic critical". Plastic hinges will form at the top and bottom of columns just below the cap beam and just above the crash wall. Shear and confinement reinforcement is required to be carefully detailed in these seismic critical areas. Lap splicing of spiral and longitudinal reinforcement is typically prohibited anywhere within the columns but may be allowed in special circumstances with the approval of the BBS. When splicing of bars is necessary, they shall preferably be mechanically spliced. Mechanical splices for longitudinal bars shall be staggered according to AASHTO LRFD Bridge Design Specifications Article 5.11.4.1.6.

Typical grade separation structures in IL that utilize such substructure units often have relatively short columns. For detailing simplicity, spiral and tie spacing in these circular and trapezoidal columns should typically be constant. The governing center-to-center design spacing should be that required for confinement at plastic hinging regions. Tie and spiral spacings should not be less than 3 in. nor greater than 6 in. center-to-center. Bar sizes greater than #5 (for ties or spirals) should be avoided in columns when possible. The 6 in. maximum allowable spacing for spirals and ties is per the AASHTO Guide

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Specifications for LRFD Seismic Bridge Design and is considered permissible for use with the AASHTO LRFD Bridge Design Specifications. If a spacing less than 3 in. is required, the BBS should be contacted. For taller columns, it is permissible to increase the vertical tie spacing outside of the plastic hinging region to the lesser of the least dimension of the member or 12 in. However, the amount of lateral reinforcement provided by the larger vertical spacing should not be less than 50% of the lateral reinforcement required in the plastic hinging region. Ties within columns should be detailed so that no longitudinal bar is more than 12 in. from a bar restrained by a corner tie within the plastic hinging region or 24 in. from a bar restrained by a corner tie outside of the plastic hinging region. Lateral reinforcement shall be the greater of that amount required for shear strength or the minimum prescriptive detailing requirements that are provided. When investigating shear, it is simplest to determine the required tie or spiral spacing by assuming the concrete has no shear strength ($V_c = 0.0$).

Spirals and ties shall be extended into cap beams and crash walls not less than half the largest column dimension or 15 in., whichever is greater. As shown in Figures 3.15.5.1-2 and 3.15.5.1-4, “seismic hoops” are an alternative for shear reinforcement extensions into cap beams and crash walls for circular columns. The extra 1 ½ turns at the ends of column spirals shall be lap welded together to form an equivalent seismic hoop as a transition. As an alternative, a 135° standard hook into the core at the end of the extra 1 ½ spiral turns may be provided.

For tall columns, splicing of spiral reinforcement may be required for constructability purposes. Splicing of reinforcement within plastic moment regions is prohibited. Therefore, the plastic moment regions shall be shown as “no splice zones” on the plans to alert the Contractor. See Figures 3.15.5.1-1 and 3.15.5.1-2 for examples.

To ensure that spirals are spliced using equivalent seismic hoops, the following note shall be placed on the plans:

> *When splicing of spiral reinforcement is necessary, the spirals shall be provided with 1 ½ extra turns at the ends to be spliced. These additional turns shall either be welded together according to AWS D1.4, or shall both terminate with a 135° standard hook.*

Because use of seismic details for trapezoidal columns results in difficult detailing and placement of reinforcement, the use of round columns instead of trapezoidal columns in regions of Illinois where seismic detailing is required is strongly encouraged. Regardless, the detailing of ties in trapezoidal columns is illustrated in Figures 3.15.5.1-3 and 3.15.5.1-4. In regions of Illinois where no special seismic detailing is required, pairs of lap spliced U-shaped bars are normally detailed as shear reinforcement in the columns. Figure 3.15.5.1-4 gives an example of how this basic detailing method has been modified

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and built upon to satisfy confinement and shear strength requirements. U-bars which have 90° standard bends at the ends, a rectangular or square tie in the center of the column, and cross ties which lap across the long (transverse) but do not lap across the short (longitudinal) direction of the columns may all be used in various combinations to meet seismic requirements. More detailed requirements, guidance and illustrations can be found in appropriate articles of the AASHTO LRFD Bridge Design Specifications.

Complex detailing of confinement reinforcement extending into cap beams and crash walls for trapezoidal column bents is not required. Only “seismic rectangular hoops” or simple closed hoops with constant dimension are required. No cross ties or U-bars should be specified in these regions as shown in Figures 3.15.5.1-3 and 3.15.5.1-4.

Minimum confinement steel according to Figures 3.15.5.2-1 and 3.15.5.2-2 shall be provided in crash walls which exceed 6 ft. in height or are similar in height to the columns.

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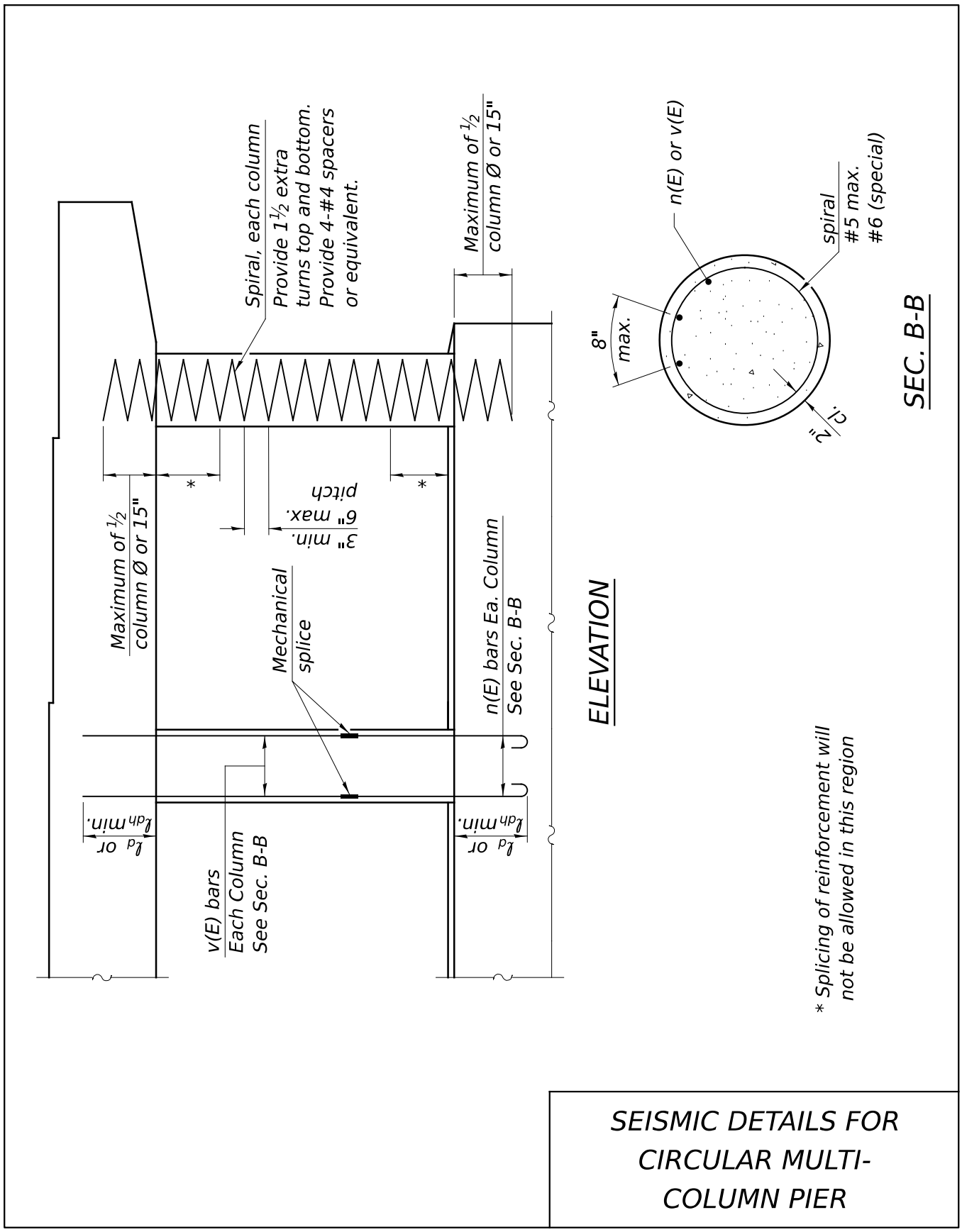
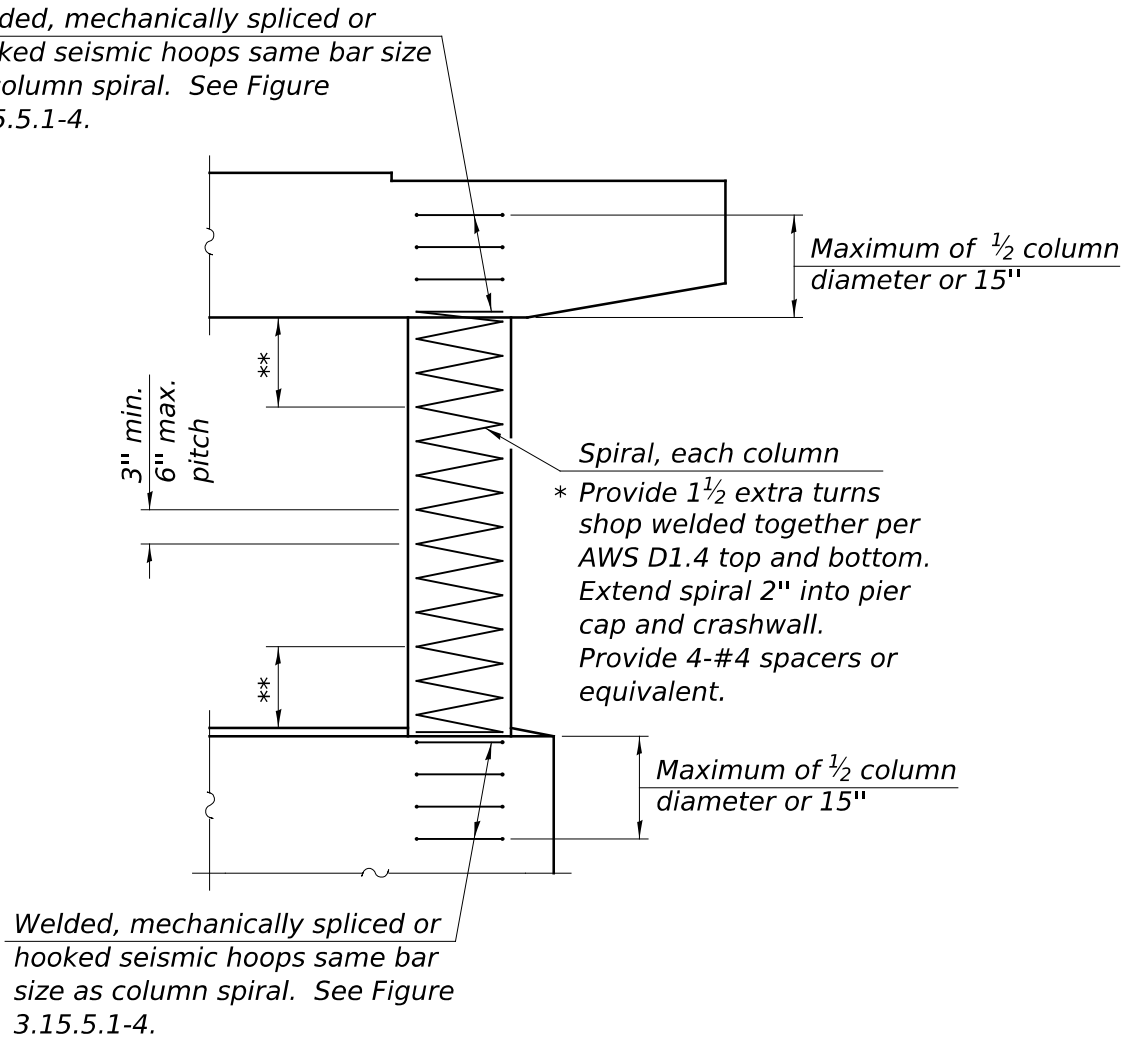


Figure 3.15.5.1-1

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ELEVATION

* Allowable substitution:
Provide 1½ extra turns top and bottom with 135° standard hook into core at ends of spiral

** Splicing of reinforcement will not be allowed in this region

ALTERNATE SEISMIC DETAILS FOR CONFINEMENT OF CIRCULAR COLUMNS

Figure 3.15.5.1-2

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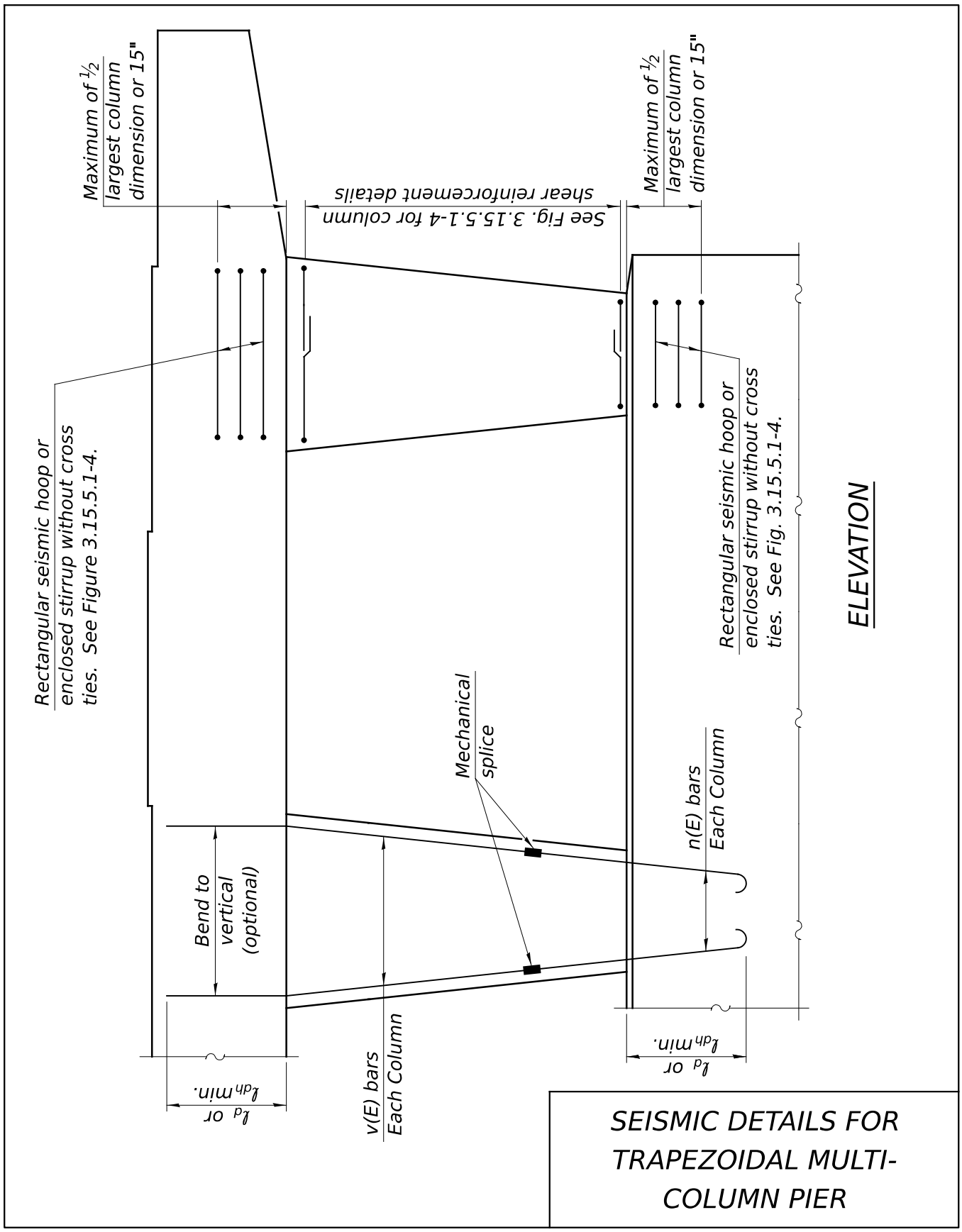
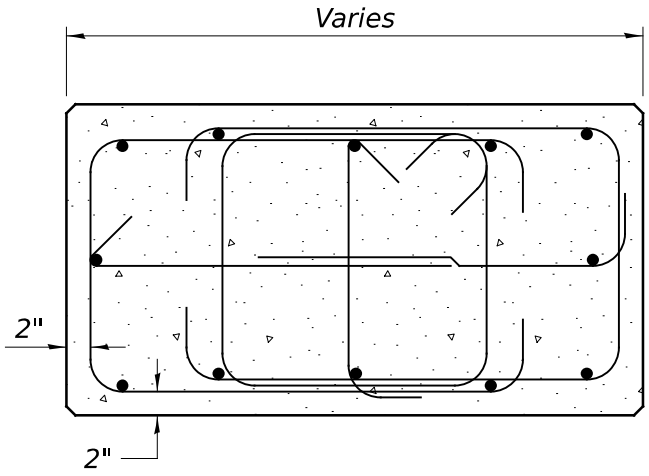


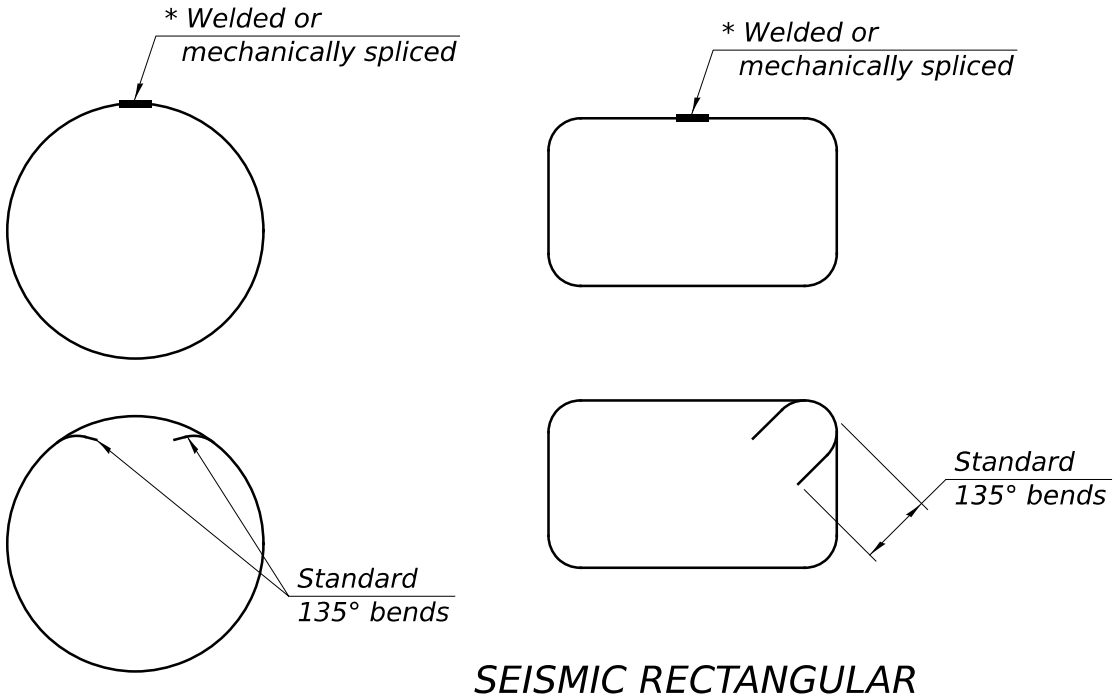
Figure 3.15.5.1-3

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SECTION THRU COLUMN

Section intended to convey example techniques for seismic tie reinforcement in trapezoidal columns. Individual designs will vary. See AASHTO Specifications for further guidance.



SEISMIC HOOP

SEISMIC RECTANGULAR HOOP

* Shop welded per AWS D1.4.

SEISMIC CONFINEMENT DETAILS

Figure 3.15.5.1-4

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Vertical column reinforcement shall extend at least the development length, l_d (not hooked) or l_{dh} (hooked), into cap beams as crash walls. Hooked bars are preferred by the Department as they provide increased ductility. The provisions for calculating development lengths are similar in the AASHTO LRFD Bridge Design Specifications. When detailing hooked bars, constructability and bar congestion shall be considered.

3.15.5.2 Solid Wall and Hammerhead Bents with Footings

Figure 3.15.5.2-1 illustrates typical seismic detailing requirements which are applicable to solid wall, modified hammerhead, and hammerhead piers with footings. The focus of the figure is on the primary seismic critical location in and around the wall-footing interface (plastic hinging region). The theoretical plastic hinging region extends from the bottom of the wall to a distance that is at least equal to the thickness of the wall, one-sixth the height of the wall or 18 in., whichever is greater. Lateral reinforcement, in the form of cross ties (typically #3's or #4's) and horizontal perimeter reinforcement, shall be detailed in an arrangement that satisfies the criteria outlined for tied columns in Section 3.15.5.1. In addition, a single layer of cross ties shall be provided across the top layer of footing reinforcement at the same horizontal spacing used for the plastic hinging region of the stem.

Figure 3.15.5.2-2 provides seismic detailing requirements outside of the plastic hinging region and also conceptually illustrates suggested details for staged construction. Similar to that mentioned above, a nominal amount of lateral reinforcement in the form of cross ties and horizontal perimeter reinforcement shall be detailed in accordance with the criteria outlined for tied columns in Section 3.15.5.1 to ensure a nominal amount of ductility outside the plastic hinging region. In addition, perimeter reinforcement shall satisfy the minimum ratio prescribed in AASHTO LRFD Bridge Design Specifications Article 5.11.4.2.

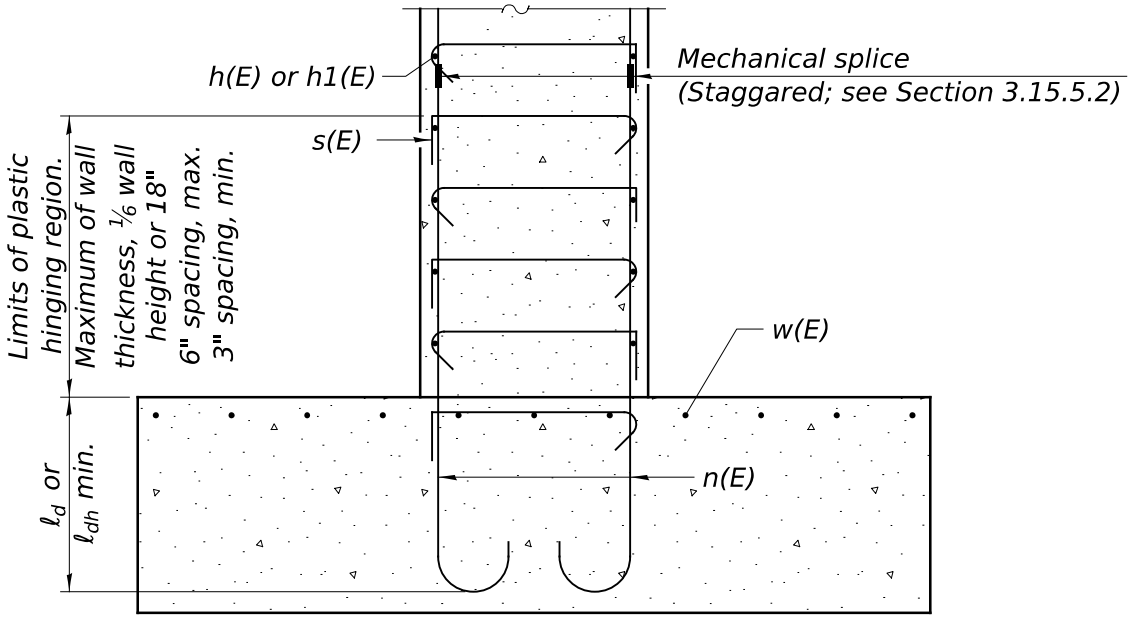
The normal lateral reinforcement around the perimeter of the wall on which the cross ties set should be joined together by mechanical splicers or welding to form a hoop in the plastic hinging region. The maximum bar size should typically be #6. Beyond the seismic critical area, shear reinforcement around the wall perimeter need not be joined by mechanical splicers or welding. Providing at least 90° bends (standard hooks) at the ends of u1(E) bars pointing towards the wall interior is an acceptable alternative equivalent outside of the plastic hinging region as depicted in Figure 3.15.5.2-2.

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For staged construction, as shown in Figure 3.15.5.2-2, the horizontal perimeter reinforcement should protrude from the finished Stage I construction in the plastic hinging region near the base of the wall, while the horizontal perimeter reinforcement outside of this region may be spliced conventionally. Designers should review literature from the manufacturers listed on the Department's approved supplier list for mechanical splices in determining how far the bars should protrude from the finished Stage I construction to accommodate an array of possible mechanical splices. Consideration should be given to any possible stage construction clearance issues when investigating the projection length.

When splicing of vertical (longitudinal) bars is necessary, they should be mechanically spliced. Lap splicing of vertical reinforcement in plastic hinging regions is not permitted and is strongly discouraged outside of the plastic hinging region. If lap splices are desired, the BBS should be contacted. Mechanical splices for vertical bars shall be staggered according to AASHTO LRFD Bridge Design Specifications Article 5.11.4.1.6. Vertical bars shall extend at least the development length, l_d (not hooked) or l_{dh} (hooked), into the footing. For solid walls (where applicable) and modified hammerhead piers, the vertical bars should extend the same development length into the caps which do not have any other special seismic detailing. Hooked bars are preferred by the Department as they provide increased ductility. When detailing hooked bars, constructability and bar congestion shall be considered.

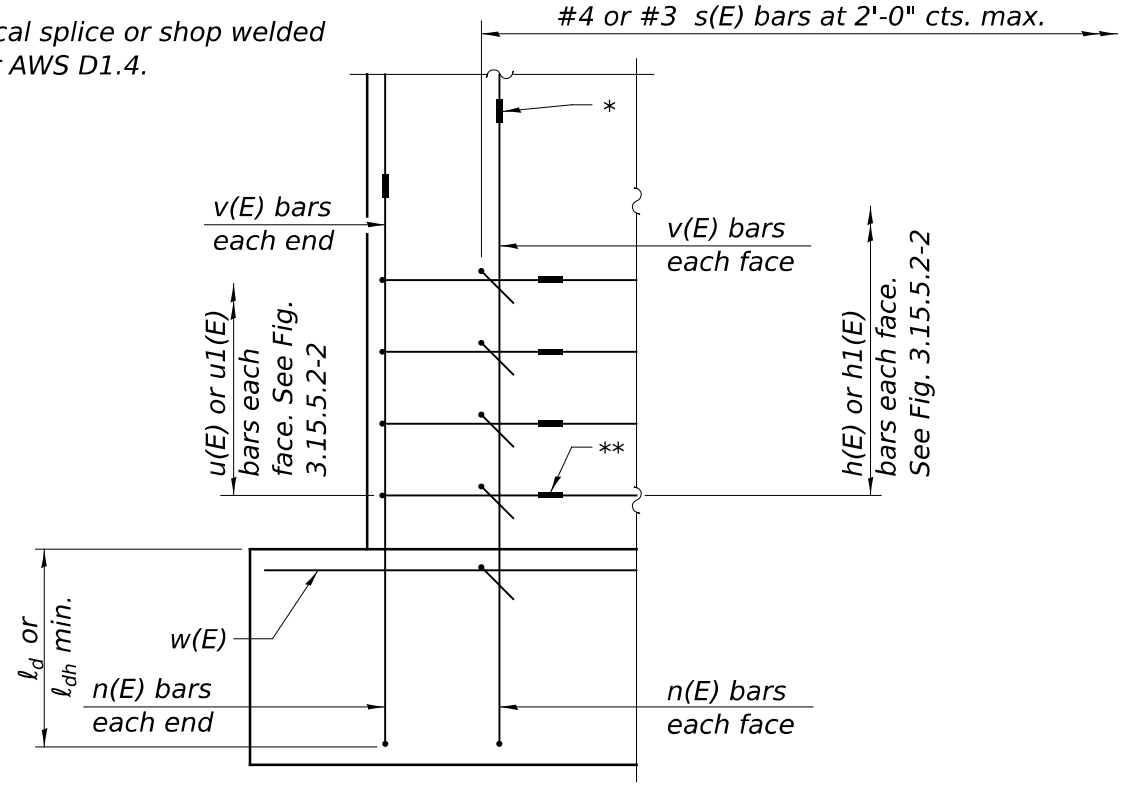
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SECTION THRU PIER

* Mechanical splice

** Mechanical splice or shop welded splice per AWS D1.4.



ELEVATION

SEISMIC DETAILS FOR SOLID STEM PIER WITH FOOTING - PLASTIC HINGING REGION

Figure 3.15.5.2-1

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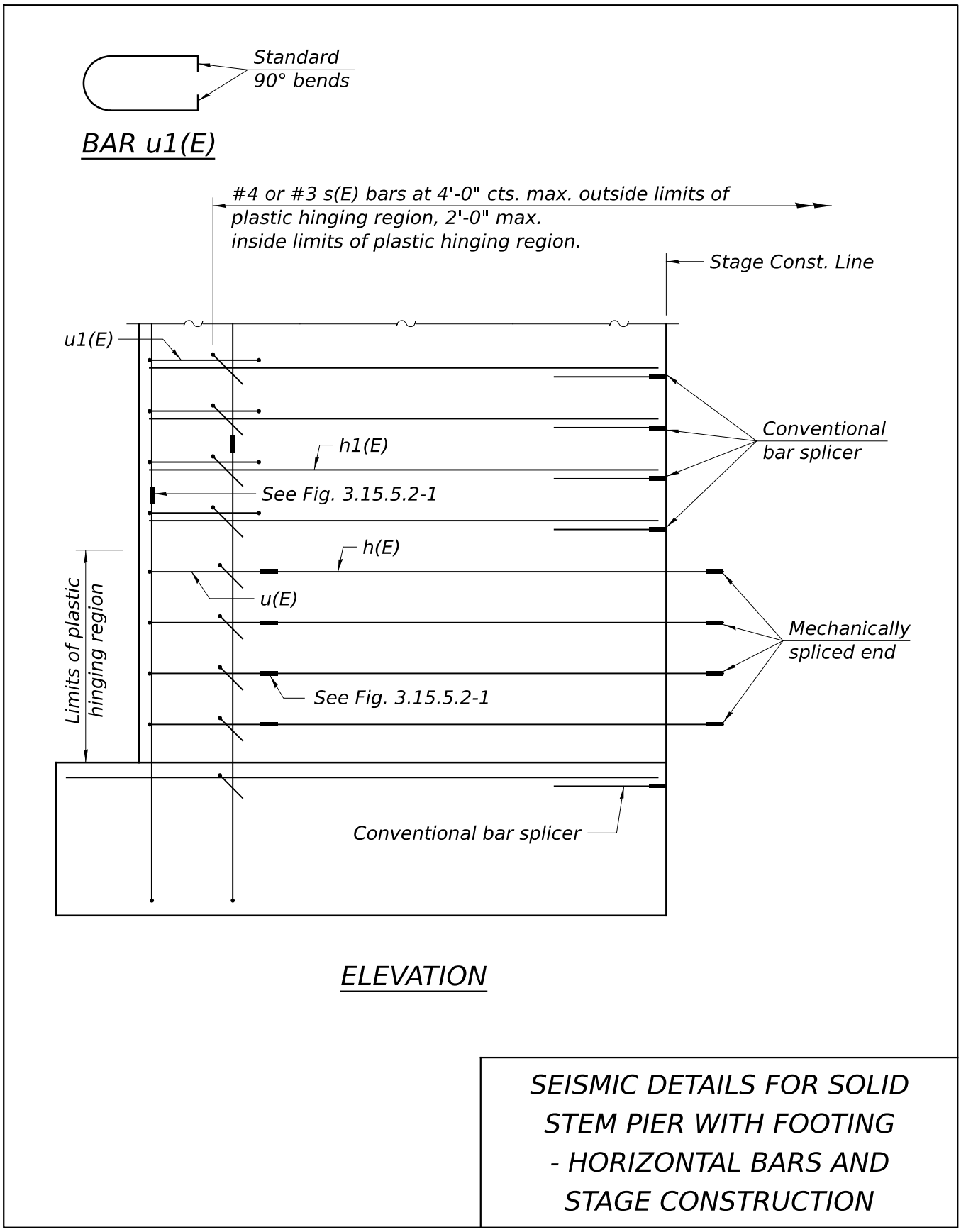


Figure 3.15.5.2-2

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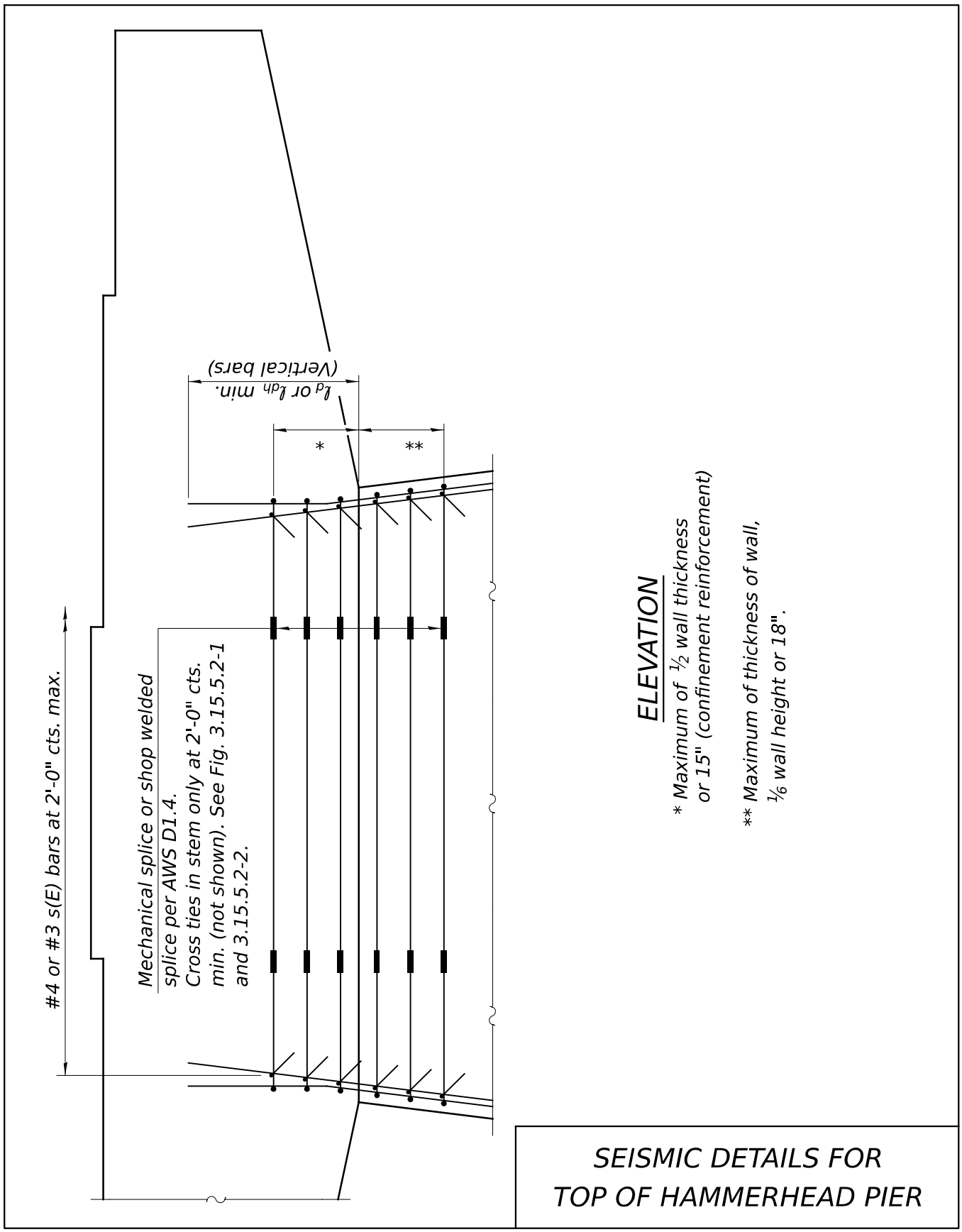


Figure 3.15.5.2-3

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As discussed in Section 3.15.4.3.4, hammerhead (not modified hammerhead) piers should typically be designed more like a column than a wall. Details similar to those for a trapezoidal column may be more suitable around the hammerhead (analogous to a cap beam on a single column) and wall (stem) interface portion of the pier depending upon how the pier has been dimensionally proportioned. Hoops, either mechanically spliced or welded (as described above for the bases of walls), with constant dimensions should be provided for a distance of at least ½ the thickness of the wall or 15 in., whichever is greater, into the hammerhead cap. Either lapped, closed rectangular hoops with cross ties or horizontal perimeter reinforcement with cross ties should be provided over the height of the wall and satisfy the same prescriptive requirement previously mentioned. At a minimum, the vertical bars in the walls should extend l_d or l_{dh} into the hammerhead (cap). Hooked bars are preferred by the Department as they provide increased ductility see Figure 3.15.5.2-3. When detailing hooked bars, constructability and bar congestion shall be considered.

3.15.5.3 Individually Encased and Solid Wall Pile Bent Piers

For individually encased and solid wall pile bents, piles should be embedded into cap beams a minimum of 2 ft. – 0 in. Note that a deeper cap may be needed to fulfill this requirement.

Reinforcement for solid wall pile bent piers shall be according to Figure 3.9.6-1. This reinforcement shall be used for all solid wall pile bent piers regardless of seismic region.

If the designer considers individual pile encasements to be structural, the encasement reinforcement shall be detailed in accordance with 3.15.5.1.

3.15.5.4 Drilled Shaft Piers

Seismic details for drilled shaft piers are shown in Figures 3.15.5.4-1 through 3.15.5.4-5. Many of the same types of details used for other classes of piers are also applicable to drilled shafts. Some of the primary differences as well as similarities are emphasized in this section.

All column and cap beam interface region seismic details are similar to those for circular column bents for shear and vertical steel. The alternative options for shear reinforcement extensions into cap beams (using ties instead of spiral extensions) are also permitted (See Figures 3.15.5.1-2 and 3.15.5.1-4). Vertical bars in the columns should extend l_d or l_{dh} into caps. Hooked bars are preferred by the Department as they provide increased ductility. When detailing hooked bars, constructability and bar congestion shall be considered.

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Lap splices for spirals and longitudinal (vertical) bars are strictly prohibited in theoretical plastic hinging regions and near column-shaft interfaces as these are considered potential plastic hinging locations. Lap splicing of spiral and longitudinal reinforcement is also strongly discouraged in all other regions of columns and shafts whenever possible. For tall shafts, splicing of spiral reinforcement may be required for constructability purposes. As splicing of reinforcement within plastic moment regions is prohibited, these regions shall be shown on the plans. See Figures 3.15.5.1-1 and 3.15.5.1-2 for examples.

When splicing of longitudinal bars is necessary, they should be mechanically spliced. Mechanical splices for vertical bars shall be staggered according to AASHTO LRFD Bridge Design Specifications Article 5.11.4.1.6. An exception to using mechanical splices is when nominal extensions of the drilled shaft cages are required in construction due to variable field conditions. In most instances, the cage can be lengthened by lap splicing additional bars at the base of the cage as the moment demand in this area is greatly diminished. This should be handled on a project by project basis.

In “transition” regions, such as column-shaft interfaces or when spiral center-to-center spacing changes, the extra 1 ½ turns at spiral terminations shall be welded together to form an equivalent seismic hoop, or alternatively, a 135° standard hook into the spiral core may be provided.

To ensure that spirals are spliced using equivalent seismic hoops, the following note shall be placed on the plans:

> *When splicing of spiral reinforcement is necessary, the spirals shall be provided with 1 ½ extra turns at the ends to be spliced. These additional turns shall either be welded together according to AWS D1.4, or shall both terminate with a 135° standard hook.*

Typically, spirals with pitches not greater than 6 in. (as recommended in the AASHTO Guide Specifications for LRFD Seismic Bridge Design) and not less than 3 in. are required in drilled shafts a minimum distance of 3 shaft diameters below ground, wall encasements, etc. The only exception is drilled shaft piers with crash walls for which the minimum is the greater of 1 shaft diameter or 30 in. below the crash wall (See Figure 3.15.5.4-4). Below these points, the spiral spacing shall be 6 in. as is currently detailed on the Departmental Base Sheets for drilled shafts.

Exterior reinforcement within crash walls shall be mechanically spliced. This is considered to be adequate confinement for crash walls and additional tie bars are unnecessary.

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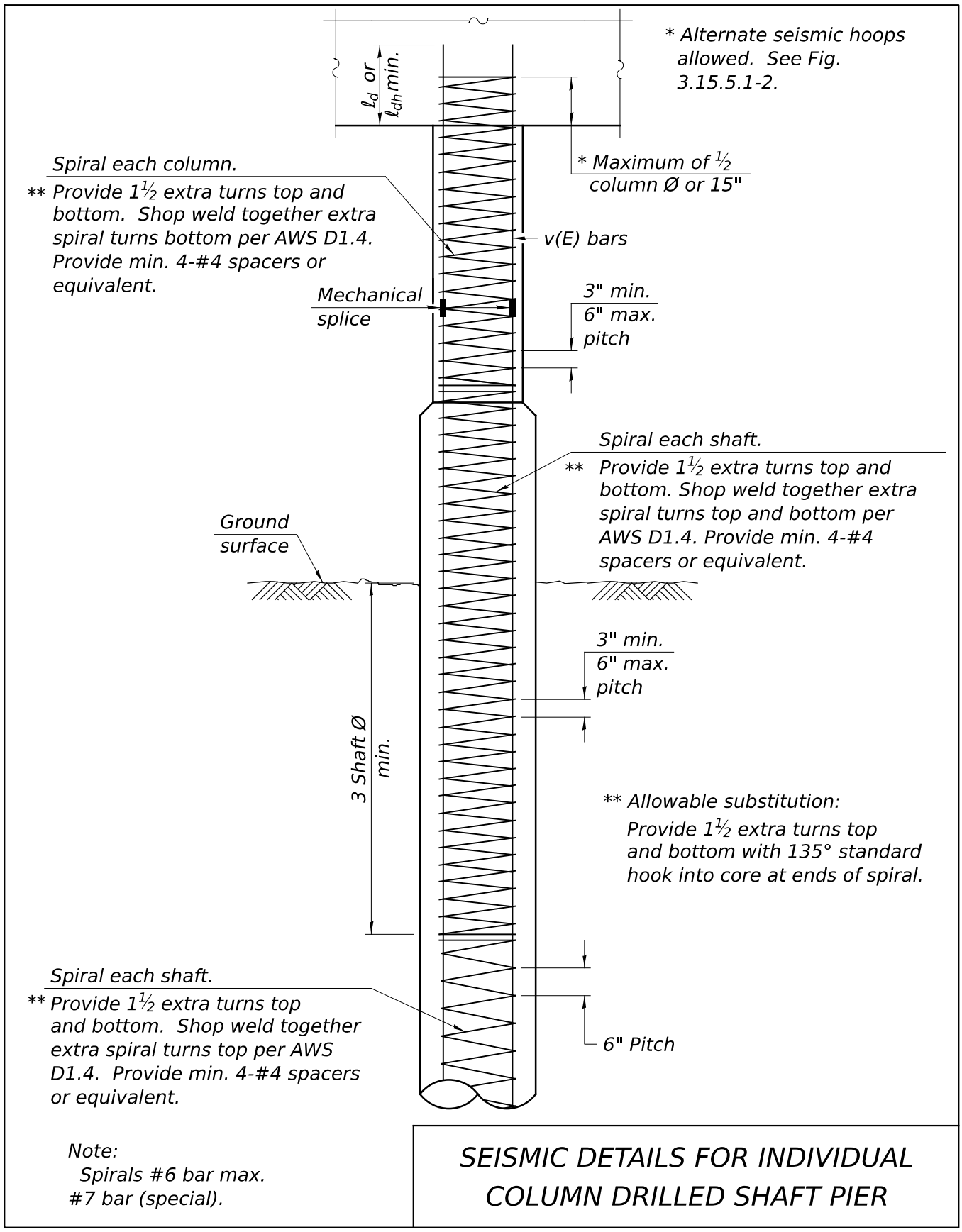


Figure 3.15.5.4-1

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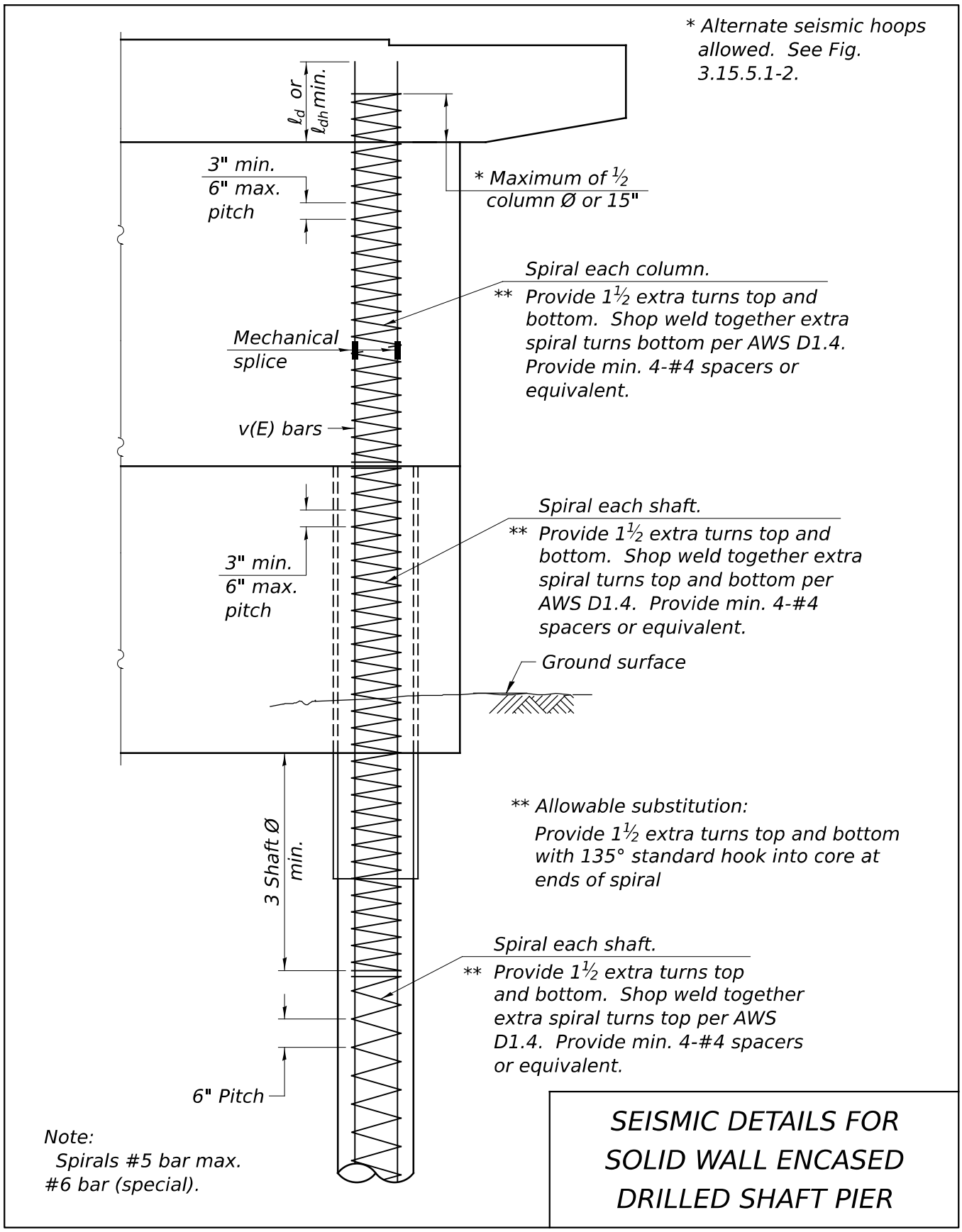


Figure 3.15.5.4-2

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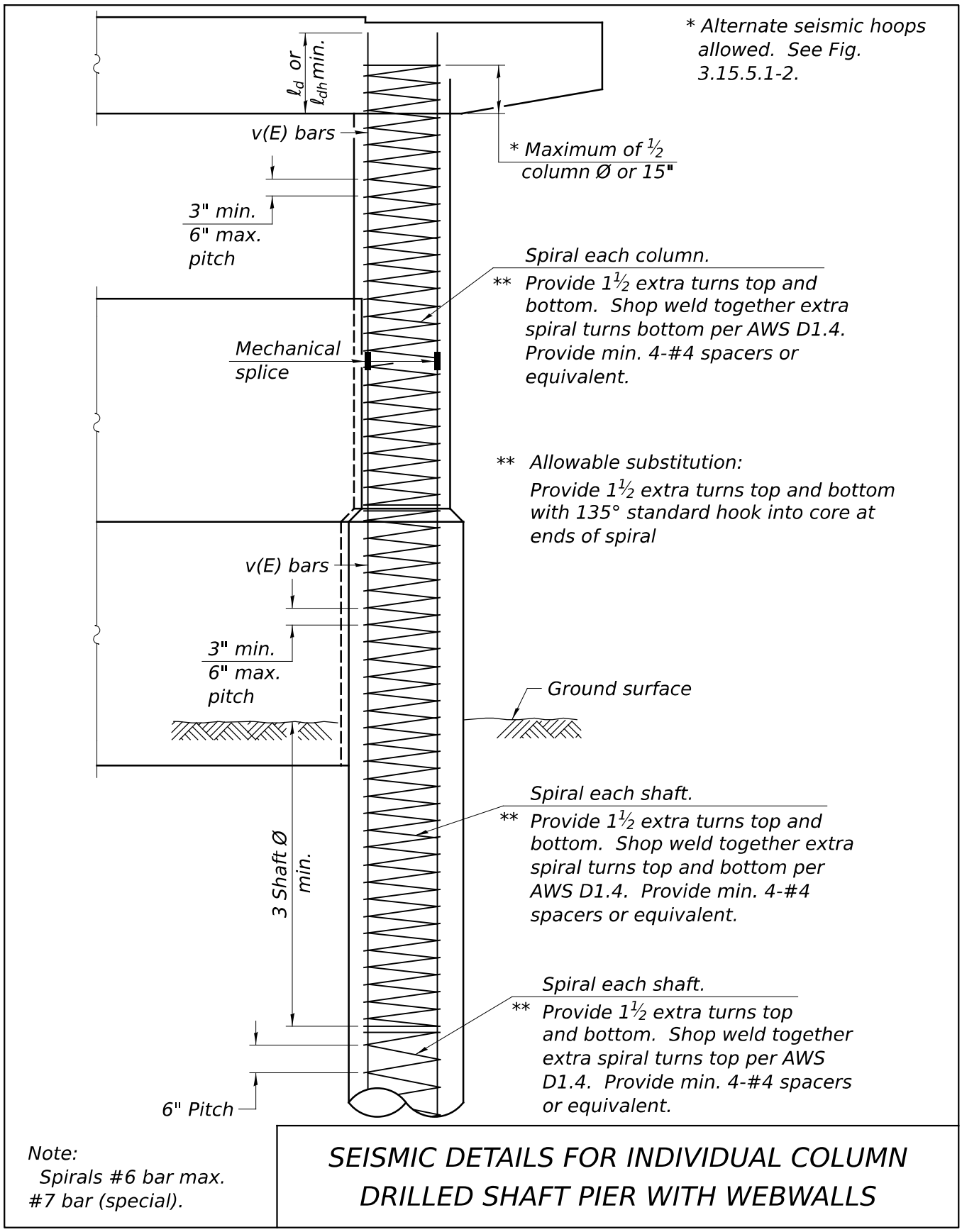


Figure 3.15.5.4-3

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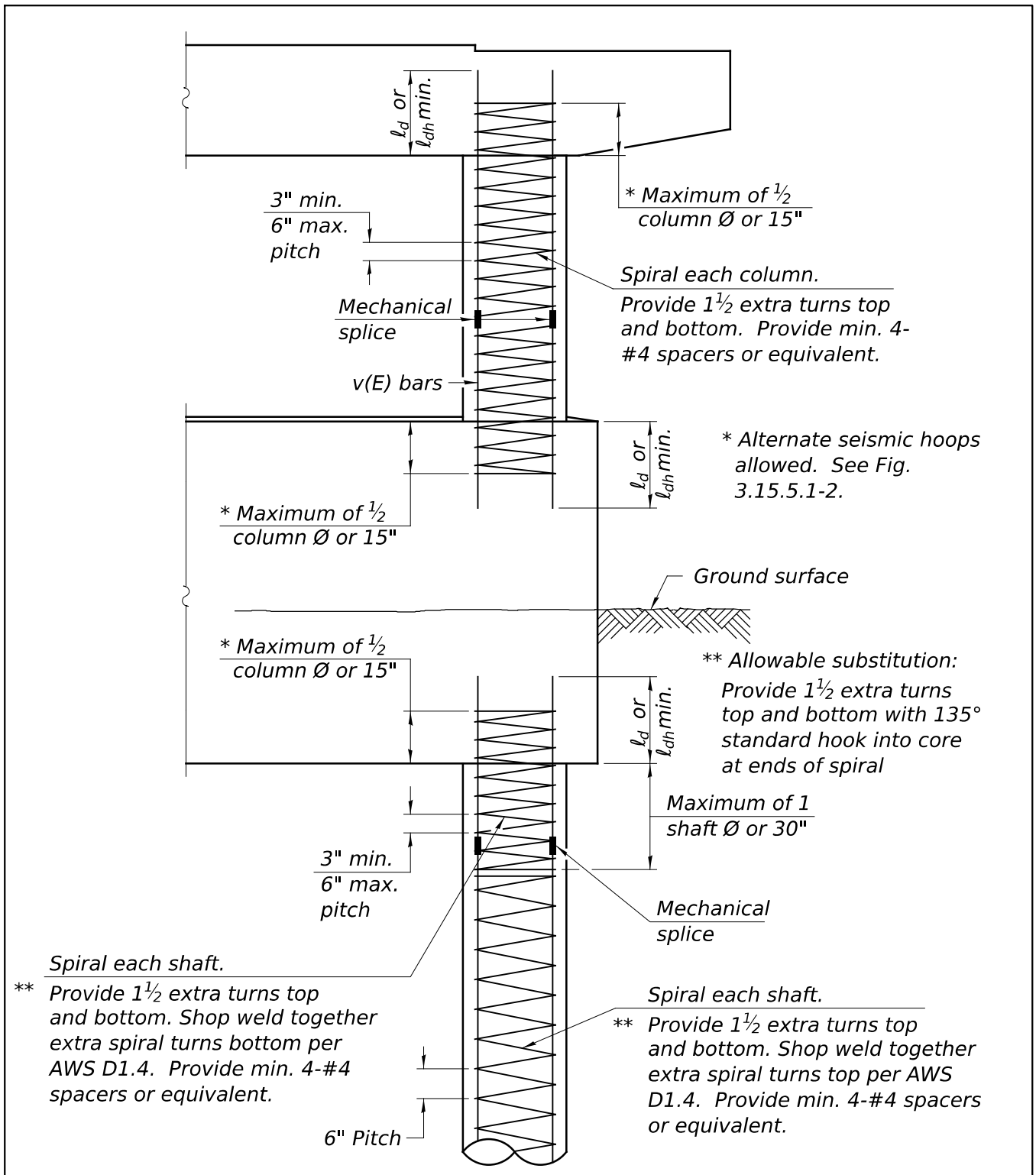


Figure 3.15.5.4-4

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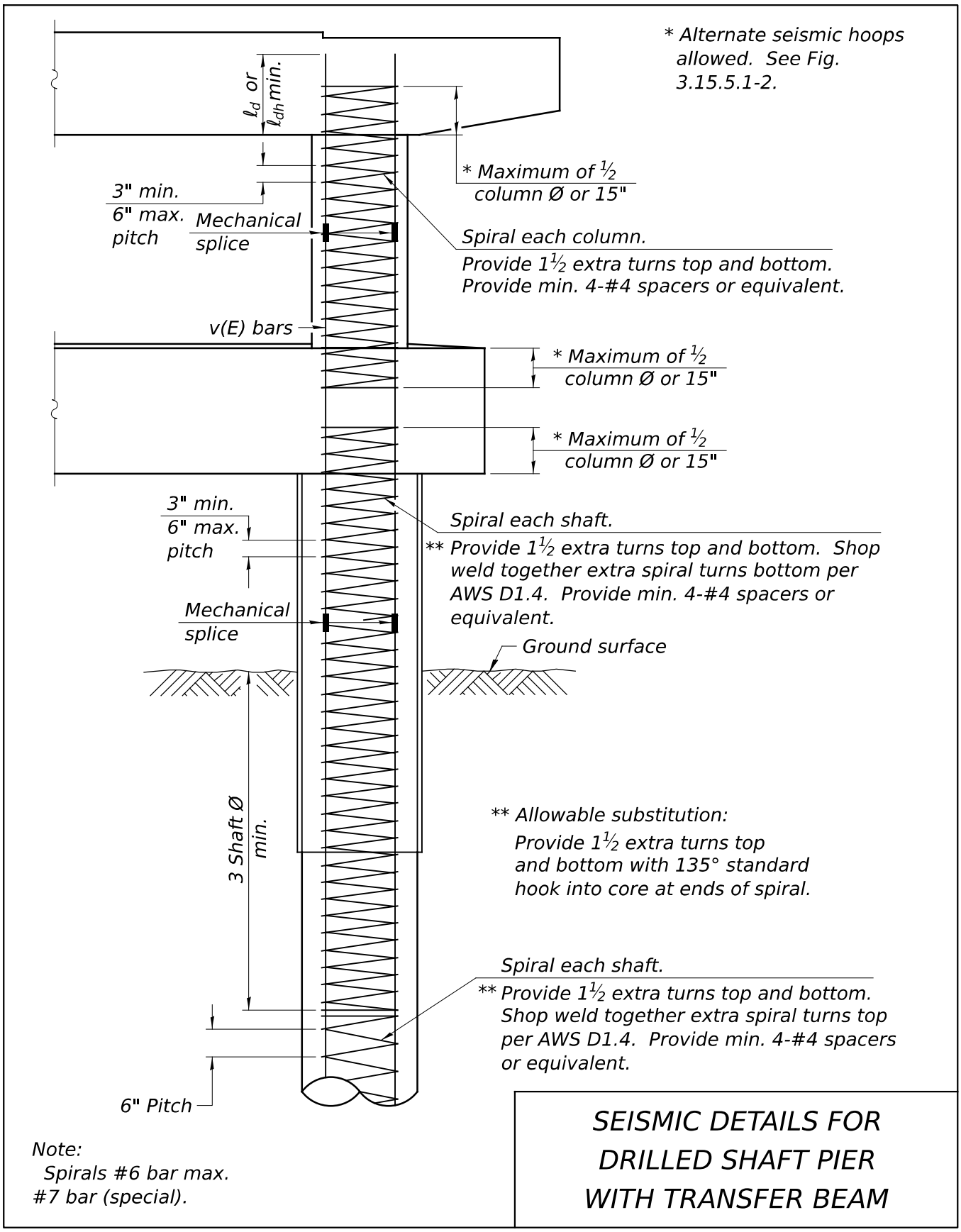


Figure 3.15.5.4-5

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3.15.5.5 Piles

Piles at abutments, pier footings, and pier cap beams should typically be embedded a minimum of 2 ft. – 0 in. to ensure a fixed boundary condition. If a 2 ft. – 0 in. minimum embedment is not feasible or the piles are subject to tension, the details illustrated in Figure 3.15.5.5-1 should be used for steel H-piles. For metal shell (MS) piles, the details for pile reinforcement at abutments (in the "Piles" IDOT CADD Bridge Cell Library) can be conceptually considered as equivalent to those in Figure 3.15.5.5-1 with regards to ensuring fixity and providing pullout resistance when the pile embedment is 2 ft. – 0 in. If pile embedment is less than 2 ft. – 0 in., the tops of the reinforcement bars should be extended so that the total embedment of the MS pile and reinforcement into the concrete member is not less than 2 ft. – 0 in. The top of the longitudinal bars for the MS piles shall be hooked 90°. Details of this type may also be provided at piers if required by design. See also Section 3.10.1 for further guidance on the geotechnical and structural design of piles.

In SPZ 2 through 4, the use of battered piles is discouraged due to the amount of stiffness they introduce into the structure and the difficulty in predicting their behavior during a seismic event. It is recommended that vertical piles, designed for combined axial and bending loads, be used whenever practical.

Structurally, metal shell piles should be designed as reinforced concrete columns in soil with the shell assumed to act compositely with the concrete. Note that, in general, the details for reinforcement provided inside of metal shell piles at abutments are not considered supplemental columnar reinforcement for resistance to seismic loadings. The reinforcement shown on the base sheets is intended to provide a nominal amount of shear and flexural capacity near the top of the piles should the metal shell corrode and deteriorate. The designer has the option of using additional reinforcement to obtain higher pile capacities if necessary.

Refer to the examples and appendices in the "Seismic Design" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design for additional information on the design of piles to resist seismic loadings. Guidance on pile fixity depths and axial strength versus moment strength column curves for metal shell piles are provided in Appendix C of the "Seismic Design" Design Guide. Examples 1 and 4 provide guidance and complete calculations for the structural design of piles to resist seismic loadings. With the exception of pile bent piers, when designing piles an R-factor of 1.0 shall be used.

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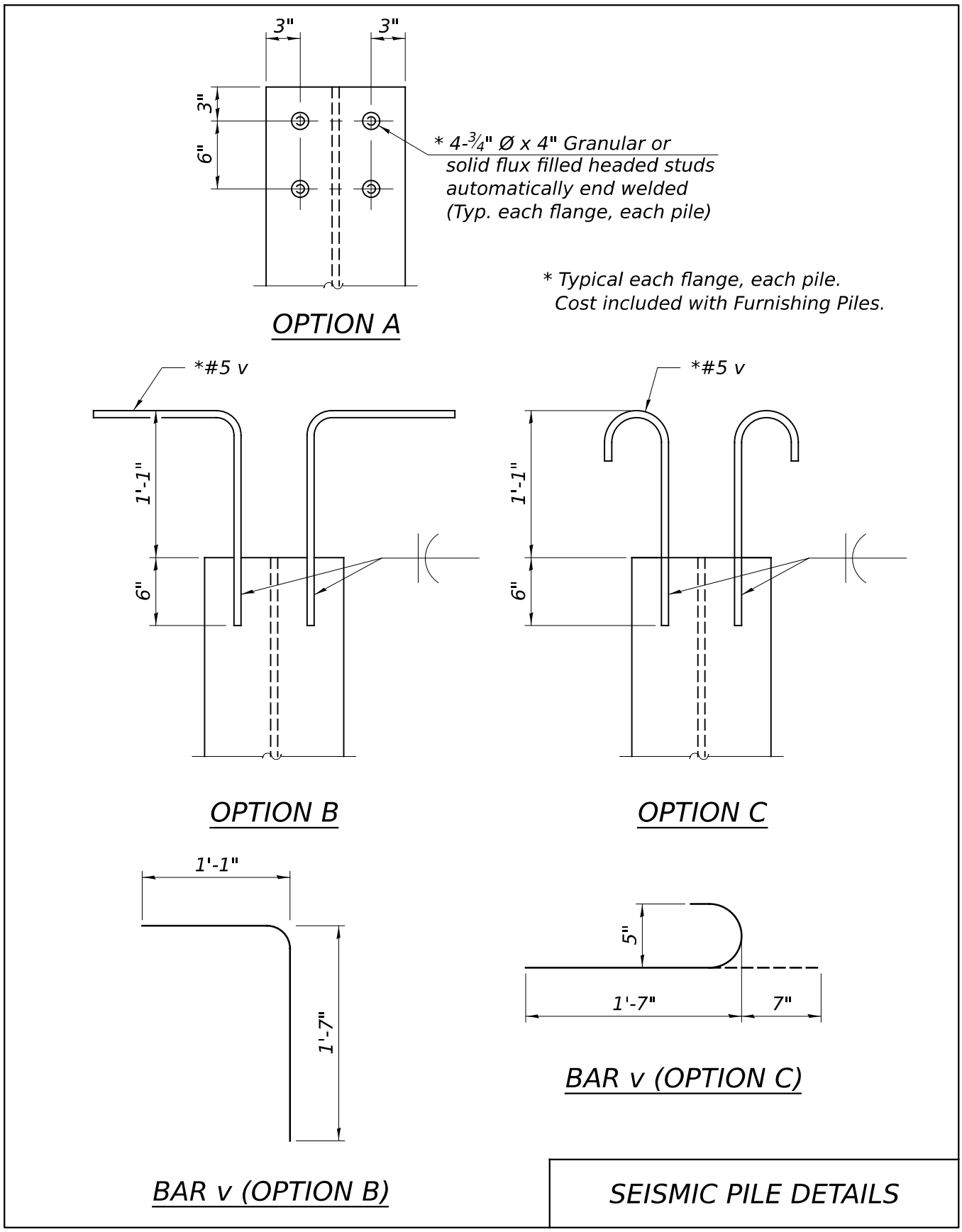


Figure 3.15.5.5-1

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3.15.5.6 Spread Footings

Spread footings may be used in instances where competent rock exists within a reasonable distance from the ground surface or suitable soils are present where strength degradation under dynamic cyclical loading is not expected. Spread footings located on soils which are susceptible to liquefaction are strictly prohibited unless mitigated by ground improvements and approved by the BBS.

Articles 5.3 and 6.3 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design may be referenced for information on the seismic design of spread footings. When designing spread footings, an R-factor of 1.0 shall be used.

3.15.5.7 Closed Abutments

When bars are required to be spliced in stems of closed abutments, they should be mechanically spliced. Mechanical splices for vertical bars shall be staggered according to AASHTO LRFD Bridge Design Specifications Article 5.11.4.1.6.

3.15.6 Overview of Global Analysis and Design Methods

Most typical bridges built in Illinois can be analyzed and designed for seismic loadings with straightforward methods. Finite element models with only frame and spring type elements or hand calculation methods without the assistance of a computer can normally be used to achieve reasonably accurate results for design purposes. It is usually good practice, though, to use a combination of these two approaches for verification of results. This section contains an overview of the basics of global structural modeling and simple structural dynamics which can be used as tools for analyzing and designing typical bridges in Illinois for seismic loadings. More complete guidance and examples can be found by referencing the “Seismic Design” Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design.

Typically, it is useful to divide seismic structural analysis of a bridge into separate global and local models. A global model encompasses the entire bridge while local models focus on certain parts of a structure such as a pier. Forces obtained from analysis of the global model are used to load local models.

3.15.6.1 Fundamental Period and Base Shears

Global models are typically used to determine the structural periods and design “base shears” in the longitudinal and transverse directions. The base shear in either the transverse or longitudinal direction

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is a percentage of a bridge's weight (or mass) which is used as a seismic design loading for the whole structure. The fundamental periods are a measure of the flexibility of a structure and are directly related to the level of design acceleration. In turn, the base shear is a function of the fundamental period and design acceleration. Generally, as structural flexibility increases, the design acceleration level decreases for a given soil type and design earthquake (or the percentage of the mass of the structure which is "effective" decreases).

For regular bridges, the first mode shape of a multimodal analysis is typically the controlling mode shape. The deflected shape of the first mode for a regular bridge as defined in the AASHTO LRFD Bridge Design Specifications approximates a half sine wave in the transverse direction. The deflected shape in the longitudinal direction is usually that of a cantilever at every fixed pier (and perhaps at abutments in the case of integral abutments) in a bridge.

To avoid the unnecessary rigors of a multimodal analysis, the AASHTO LRFD Bridge Design Specifications suggest two simpler methods for calculating the fundamental periods and base shears for the first mode shape of a bridge, the Uniform Load Method and the Single Mode Spectral Analysis Method. The Uniform Load Method is considered the simpler of the two and, as such, is recommended. The key to the method is the determination of an "equivalent stiffness" of a bridge which is obtained by taking a total uniform load applied to a global model and dividing by the maximum computed deflection. AASHTO LRFD Bridge Design Specifications Article 4.7.4.3.2c outlines the Uniform Load Method procedure.

The weight (mass) of the bridge to use in seismic analyses should be, at a minimum, the entire superstructure including the deck, future wearing surface (at the designer's discretion and engineering judgment), beams, parapets, cross-bracing, and diaphragms as appropriate; the cap beams at piers; and half of the pier columns or walls. Half the weight of the abutments may be included as well at the designer's discretion.

3.15.6.2 Transverse Models

A simple "generic" global model used for seismic analysis and design of the transverse direction for a three span bridge is illustrated in Figure 3.15.6.2-1. This basic model form can be adapted to a number of situations which are typical in Illinois. It can also be expanded upon by including more spans (up to 6), or by using more complex representations of the superstructure, substructures and foundations with finite elements.

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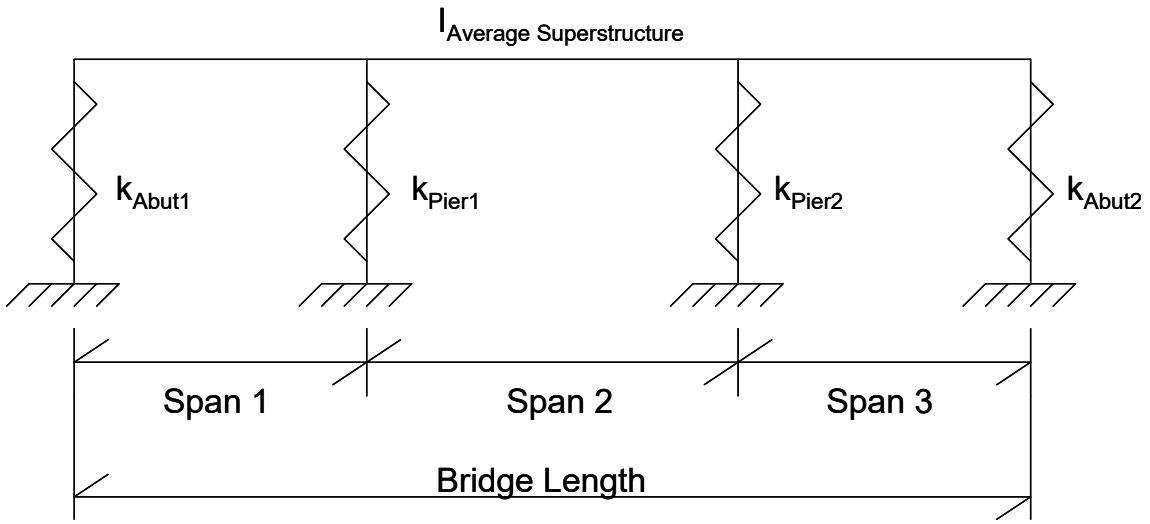


Figure 3.15.6.2-1

If a bridge exhibits a high degree of symmetry, it can be analyzed by hand methods (see the "Seismic Design" Design Guide at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design). Otherwise, the simplest finite element model for a 2 to 6 span bridge consisting of linear beam elements for the superstructure spans and substructure components may be used. Equivalent springs may be substituted for column elements and other substructure components. Spring element stiffnesses for global models can be determined from local models by applying a point load to a local model (100 kips, for example), finding the maximum deflection (0.5 in., for example) and calculating the ratio of applied load to maximum deflection to obtain a value for "k" (200 kips/in., for example).

If the superstructure does not have a uniform cross section along the entire length of the bridge (for example plate girder structures), a weighted average transverse moment of inertia may be used in most situations. It is acceptable to either count or discount the parapet's contribution to the transverse moment of inertia of the superstructure. Considering the parapets as half effective may be the most realistic assumption. Typically, the superstructure of bridges built in Illinois is very stiff in relation to the substructures and foundations. As such, some assumptions related to a superstructure's moment of inertia do not adversely affect the calculated period. Beams, either PPC-I or steel, should only be counted as half-effective when determining the transverse superstructure moment of inertia to account for shear lag. The term shear lag refers to the assumption that only about the upper halves of the beams near the deck are fully engaged or effective.

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Abutments can be modeled as pinned (infinite spring stiffness), but this is not recommended. The Uniform Load Method tends to overestimate the amount of the design base shear which "flows" to abutments. Consequently, pinned supports will amplify this effect which is not desirable. Reasonable modeling of the relative stiffnesses between support points (i.e. at substructures and foundations) is an aspect of credible modeling which should not be discounted.

For all support locations, regardless of whether the bearing is considered fixed or expansion, the connections between the superstructure and the substructures should be considered fully effective for transverse analyses.

3.15.6.3 Longitudinal Models

Typically, for global models in the longitudinal direction, the superstructure is modeled as infinitely rigid or as a rigid link between the substructures and foundations. At non-integral abutments, it is acceptable to assume that no longitudinal seismic force is transmitted from the superstructure at expansion bearings. The fixed piers, instead, are designed to carry the entire longitudinal base shear if both the bearings at the abutments are expansion type. According to the philosophy of the Illinois' ERS strategy, however, it is recommended to assume that some force is transmitted to abutments regardless of bearing type, fixed or expansion. At integral abutments and fixed piers, fully effective connections should be assumed.

3.15.7 Seismic Retrofitting of Existing Bridges

Sections 3.15.1 to 3.15.6 should all be used for reference when seismic retrofitting is considered for an existing bridge. Generally, the level of seismic retrofitting which would be required for a particular rehabilitation or reconstruction project depends upon several factors including, but not limited to, ADT; Operational Category of the bridge; age of the bridge; economic considerations; if the bridge is on the Interstate, State, or Local System; etc. The range of standards the Department may deem appropriate for retrofitting a structure can vary from none, to minimal, to that for a 1000 yr. Event and will be established on a project by project basis. See Section 2.3.10 for more information and discussion.

The FHWA Seismic Retrofitting Manual for Highway Structures (2006) should be referenced as needed. However, designers should note that some of the presented analysis and design methods are considered somewhat impractical and overly complex by the Department for typical situations. When analyzing an existing structure, for the 1000 yr. event, the LRFD Code methods should be used. In addition, the Seismic Hazard Level (SHL) shall be determined using the table below in lieu of Table 1-5 given in the 2006 Manual.

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Hazard Level	Using $S_{D1} = FvS_1$
I	$S_{D1} \leq 0.15$
II	$0.15 < S_{D1} \leq 0.25$
III	$0.25 < S_{D1} \leq 0.40$
IV	$0.40 < S_{D1}$

Table 3.15.7-1

There are certain types of retrofitting goals that are considered more important than others by the Department. The level of importance stems from the IDOT ERS Strategy and the primary objective to prevent span loss. The relative importance of these goals should be viewed as more critical when achievement of the seismic capacity is not a viable option to the designer. When a 1000 yr. seismic retrofit capacity is specified by the Department, the relative importance of the goals discussed below is somewhat diminished and details should be provided that satisfy the IDOT ERS Strategy.

Providing adequate seat widths is a high priority for an existing superstructure over other potential retrofit measures when options for the designer are limited. It is a key component of the Department's ERS strategy and objective to prevent span loss. If attainment of adequate seat widths is not a practical option, "equivalent seat widths" can be obtained using such measures as isolation bearings or longitudinal restrainer cables. Transversely, concrete blocks rigidly attached to pier or abutment caps on both sides of beams may be installed to significantly restrain movement during an earthquake and prevent loss of span.

When transverse and/or longitudinal restraint retrofit measures are used to achieve equivalent seat widths, the existing capacity of the substructure and foundation of a bridge becomes a more important consideration when attempting to attain a certain level of seismic capacity which is less than that required for the 1000 yr. seismic design event. This is because there is a greater potential for the full energy of the design earthquake to be transmitted to the substructure and foundations. In these cases, it may become necessary to employ techniques such as fiber wraps, tensioned strands, or steel jackets in plastic hinging regions of columns and/or retrofit measures for the foundations. However, when retrofitting a bridge for the 1000 yr. design earthquake, all reasonable measures should be explored for possible implementation to achieve each level of seismic redundancy in the IDOT ERS strategy.

Figures 3.15.7-1 to 3.15.7-4 illustrate some techniques which the Department uses for seismic retrofitting of bridges. The methods presented in these figures, however, should not be considered as all inclusive. Chapters 8, 9 and 10 of the FHWA Seismic Retrofitting Manual for Highway Structures can also be referenced for state-of-the-art retrofitting techniques which may be acceptable to the Department.

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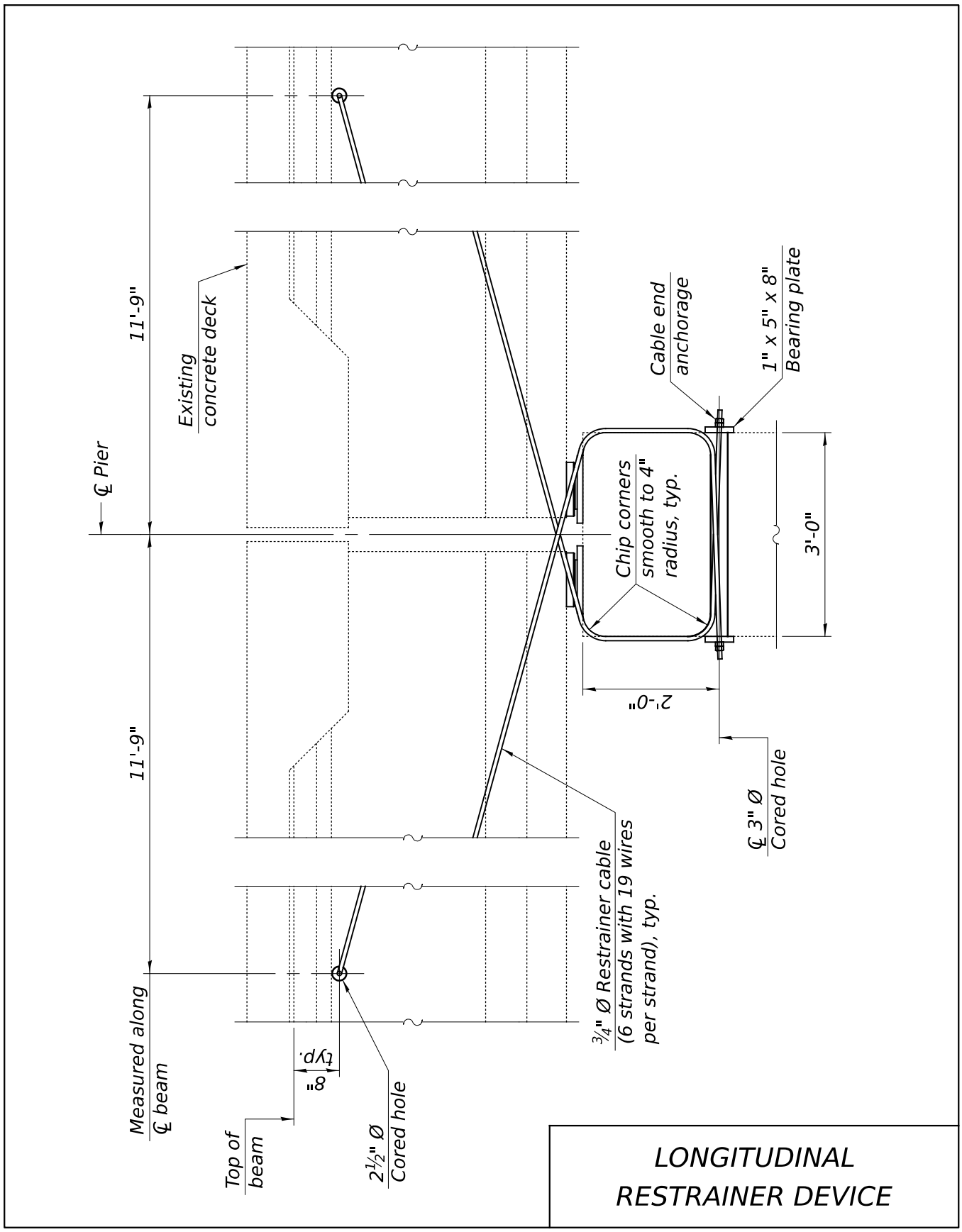
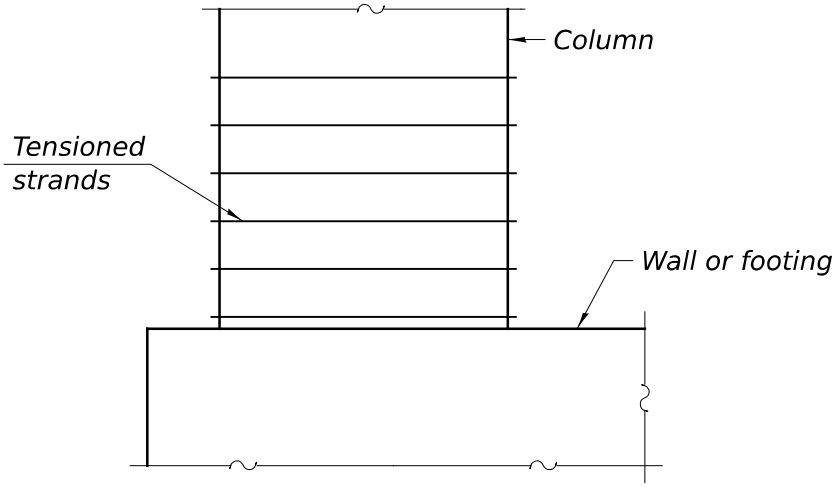
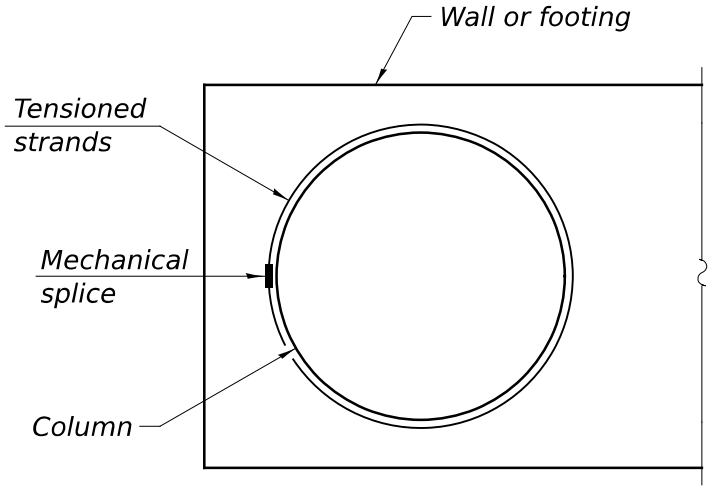


Figure 3.15.7-1

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ELEVATION

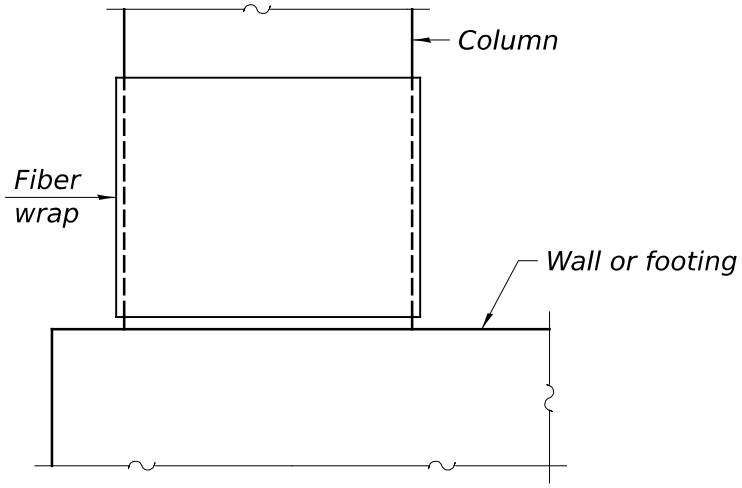


PLAN

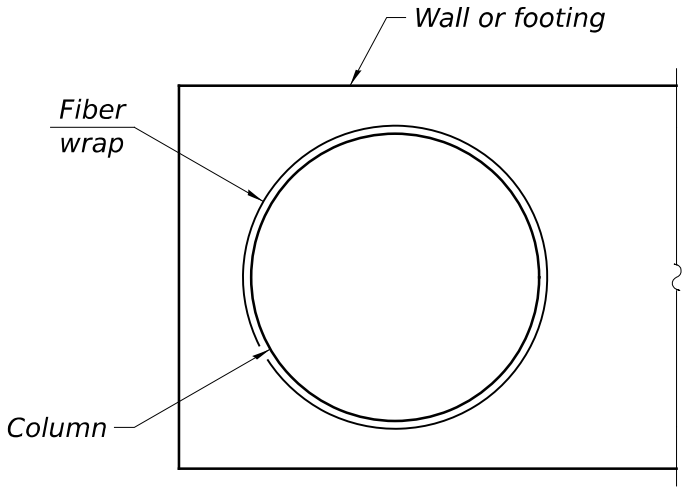
TENSIONED STRANDS

Figure 3.15.7-2

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ELEVATION

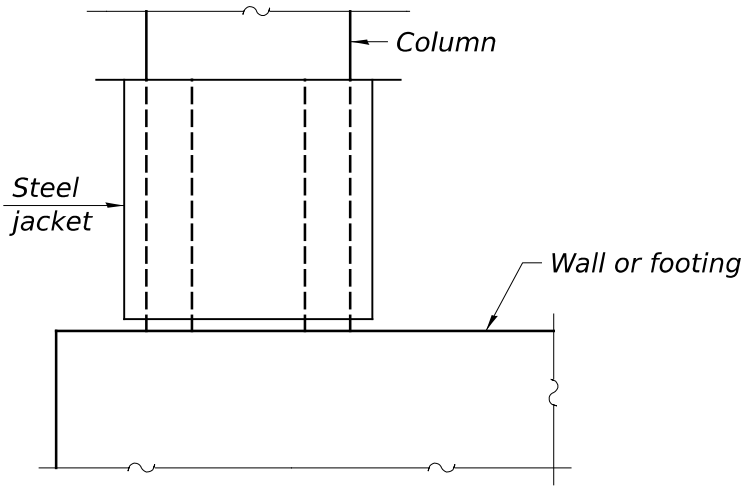


PLAN

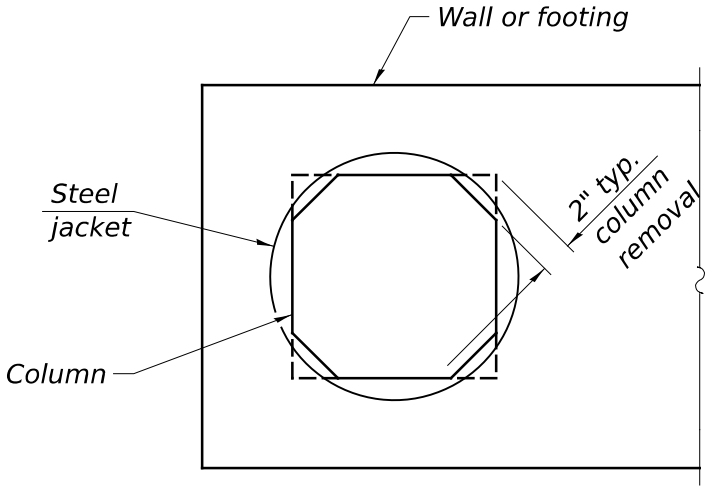
FIBER WRAP

Figure 3.15.7-3

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ELEVATION



PLAN

STEEL JACKET

Figure 3.15.7-4

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3.15.8 Local Bridge Flexible Design Option for 1000 Year Return Period Earthquake

A “flexible” approach for the 1000 yr. design return period seismic event using the AASHTO LRFD Bridge Design Specifications and the Department’s ERS strategy may be permitted for some local agency owned bridges in primarily rural areas that are part of a redundant roadway network. Roadways that may be completely closed to traffic during reconstruction are generally considered to be part of a redundant network of roadways. The term “flexible” primarily refers to an allowance for increased plasticity in substructures and foundations by employing larger R-Factors than those prescribed in Section 3.15.4.3.3 through 3.15.4.3.7. The generally less conservative, but still adequate, seat width requirements contained in AASHTO LRFD Bridge Design Specifications Article 4.7.4.4 may also be used in lieu of Section 3.15.4.2.

This flexible design option is permitted on structures satisfying the following criteria:

- Simply supported non-composite PPC deck beam or concrete channel beam superstructures with less than six spans
- Steel wide flange (but not plate girder) or PPC I-beams (but not Bulb-T) superstructures with less than four spans
- Maximum total bridge length of 400 feet
- Pile bent/stub abutments with no bridge approach slab connected to the abutments
- All slab structures
- Structures not on the National Highway System

The following recommended R-Factors for the 1000 yr. design return period earthquake are based upon bridges being classified as “Ordinary” according to the AASHTO LRFD Bridge Design Specifications.

1. Circular and trapezoidal column bents with cap beam, crash wall, footing and piles
    a. Columns: R = 5.0
    b. Piles: R = 1.5
2. Solid wall and hammerhead bents with footing and piles
    a. Transverse direction of solid walls: R = 1.5
    b. Longitudinal direction of solid walls: R = 2.5
    c. Transverse direction of hammerheads: R = 2.5
    d. Longitudinal direction of hammerheads: R = 2.5
    e. Piles: R = 1.5
3. Individually encased and solid wall pile bent piers
    a. Piles: R = 3.5

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4. Drilled Shaft Piers
    a. Individual column drilled shaft bents with plastic hinges below ground: R = 3.5
    b. Individual column drilled shaft bents with plastic hinges above ground: R = 5.0
    c. Solid wall encased drilled shaft bents: R = 3.5
    d. Individual column drilled shaft bent with web walls: R = 3.5 or 5.0
    e. Individual column drilled shaft bents with crash wall (columns): R = 5.0
    f. Individual column drilled shaft bents with crash wall (shafts): R = 1.5
5. Integral abutments
    a. Piles: R = 1.5
6. Pile supported stub abutments
    a. Piles: R = 1.5

Example 4 in the “Seismic Design” Design Guide (at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Design) details the design of a multi-span PPC deck beam bridge using the flexible approach. The bridge type in the example is commonly built on the Local Bridge System in the Southern regions of Illinois where seismicity is moderate to high. The bridge configuration consists of a set of simple spans supported by stub abutments each with a single row of piles, and individually encased pile bent piers. In addition, the superstructure overlay does not provide continuity over piers (which is typical). Multi-span PPC deck beam bridges primarily constructed on the Local System are essentially a special class of structures for seismic design compared to those discussed above.

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Appendix

A1 General Aids

The BBS distributes various general aids to the planning and design community through the IDOT Website. Future updates to the Department website or file-hosting procedures may modify or change direct hyperlinks. The Department encourages all who use BBS documents to register for all applicable "Stay Connected" and "Subscribe" options found at applicable IDOT webpages and to keep all required information (i.e. email address) updated with the necessary subscription service/instructions.

A1.1 External Resources

The Department distributes all BBS documents through the primary Bridges & Structures Page. BBS manual users will be informed of Department updates via the IDOT BBS Subscription Service found in the "Stay Connected" link at the Bridges & Structures Page.

A1.2 Base Sheets

The Department distributes all BBS Base Sheets, CADD Downloads, CADD Guidelines, and Example Plans through the primary IDOT CADD page. BBS manual users will be informed of Base Sheet and/or Cell Detail updates via the IDOT BBS Subscription Service found in the "Stay Connected" link at the primary IDOT CADD page. See Section 1.1 for more information.

Base Sheets are intended to be used without revision except for the addition of some items which have intentionally been left blank. Some Base Sheets specify alternate details. Typically, not all alternates will pertain to the specific job to which the Base Sheet was attached. However, in most cases, details and notes located elsewhere in the plans should clarify which alternate is applicable. In most cases it is not necessary to cross out details which do not pertain.

In some cases, it may be necessary to modify details or add details to the Base Sheets. When this is done, the name and date of the Base Sheet in the lower left corner should be removed. This alerts the person reviewing the plans that the Base Sheet has been revised and that it will need to be reviewed. This also maintains the integrity of the Base Sheets by preventing the revised Base Sheet from being used on future jobs as if it was the original.

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A1.3 Approved Type Size and Location (TSLPlans

The Department provides a list of approved TSL Drawings saved as “Type Size and Location Drawings (TSL)” at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Planning. Organized by Structure Number, County, Superstructure Type, Substructure Type and Date, this list is updated regularly and can provide planners/designers with the latest Bridge approvals under the latest codes and presentation standards.

A1.4 Shop Drawing Procedures

The Department provides instructions for state projects requiring fabrication shop drawing review under the “Shop Drawings Procedures” Memorandum at the primary Bridges & Structures Page **>** Specific Scope of Services **>** Fabrication. Requirements, review procedures/flowcharts, and information sheets are included in this memorandum along with the contact information for key Department Bureaus.

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A2 Planning Aids

A2.1 County Information

Prefix	County	District	Region	Prefix	County	District	Region
001	Adams	6	4	052	Lee	2	2
002	Alexander	9	5	053	Livingston	3	2
003	Bond	8	5	054	Logan	6	4
004	Boone	2	2	055	McDonough	4	3
005	Brown	6	4	056	McHenry	1	1
006	Bureau	3	2	057	McLean	5	3
007	Calhoun	8	5	058	Macon	7	4
008	Carroll	2	2	059	Macoupin	6	4
009	Cass	6	4	060	Madison	8	5
010	Champaign	5	3	061	Marion	8	5
011	Christian	6	4	062	Marshall	4	3
012	Clark	7	4	063	Mason	6	4
013	Clay	7	4	064	Massac	9	5
014	Clinton	8	5	065	Menard	6	4
015	Coles	7	4	066	Mercer	4	3
016	Cook	1	1	067	Monroe	8	5
017	Crawford	7	4	068	Montgomery	6	4
018	Cumberland	7	4	069	Morgan	6	4
019	DeKalb	3	2	070	Moultrie	7	4
020	DeWitt	5	3	071	Ogle	2	2
021	Douglas	5	3	072	Peoria	4	3
022	DuPage	1	1	073	Perry	9	5
023	Edgar	5	3	074	Piatt	5	3
024	Edwards	7	4	075	Pike	6	4
025	Effingham	7	4	076	Pope	9	5
026	Fayette	7	4	077	Pulaski	9	5
027	Ford	3	2	078	Putnam	4	3
028	Franklin	9	5	079	Randolph	8	5
029	Fulton	4	3	080	Richland	7	4
030	Gallatin	9	5	081	Rock Island	2	2
031	Greene	8	5	082	St Clair	8	5
032	Grundy	3	2	083	Saline	9	5
033	Hamilton	9	5	084	Sangamon	6	4
034	Hancock	6	4	085	Schuyler	6	4
035	Hardin	9	5	086	Scott	6	4
036	Henderson	4	3	087	Shelby	7	4
037	Henry	2	2	088	Stark	4	3
038	Iroquois	3	2	089	Stephenson	2	2
039	Jackson	9	5	090	Tazewell	4	3
040	Jasper	7	4	091	Union	9	5
041	Jefferson	9	5	092	Vermilion	5	3
042	Jersey	8	5	093	Wabash	7	4
043	JoDaviess	2	2	094	Warren	4	3
044	Johnson	9	5	095	Washington	8	5
045	Kane	1	1	096	Wayne	7	4
046	Kankakee	3	2	097	White	9	5
047	Kendall	3	2	098	Whiteside	2	2
048	Knox	4	3	099	Will	1	1
049	Lake	1	1	100	Williamson	9	5
050	LaSalle	3	2	101	Winnebago	2	2
051	Lawrence	7	4	102	Woodford	4	3

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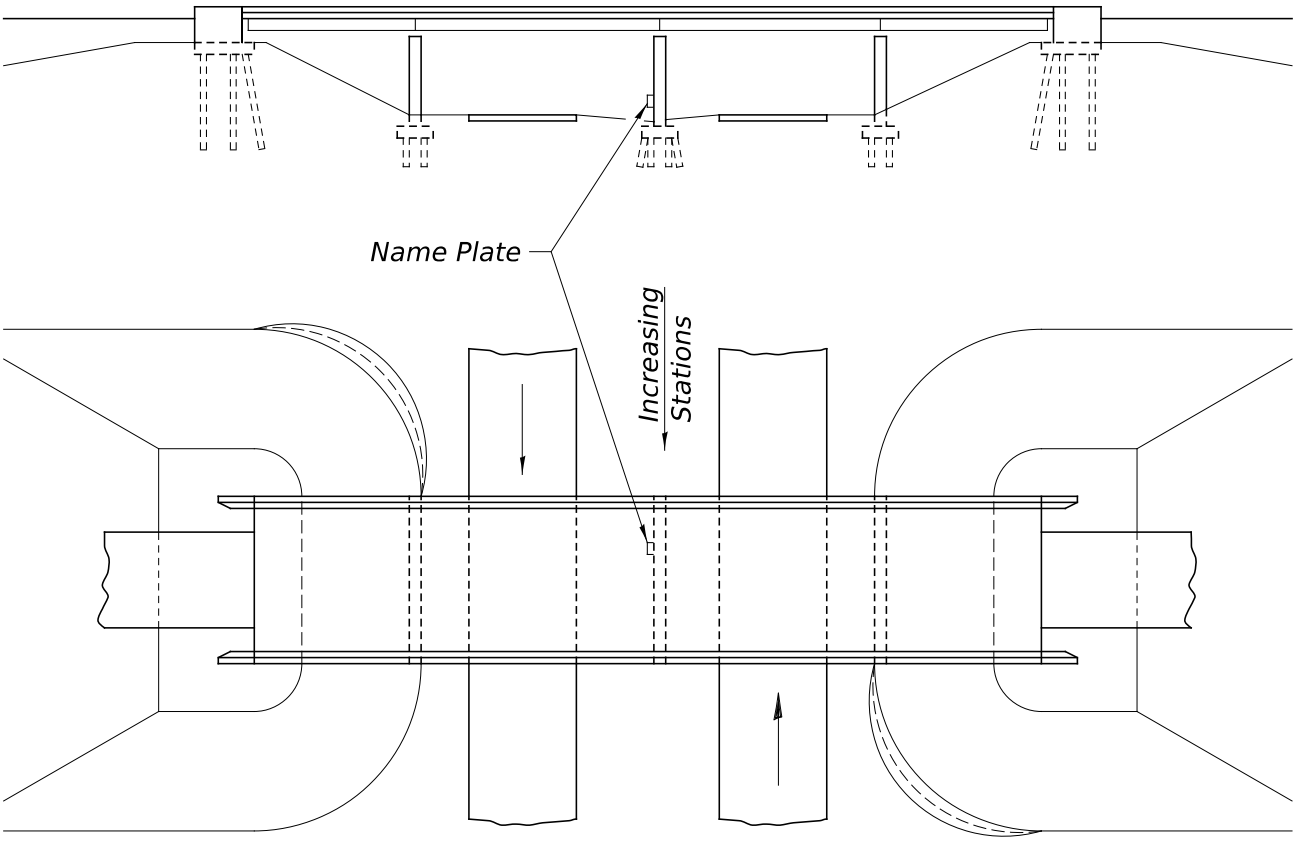
A2.2 Suffixes to Denote Type of Work

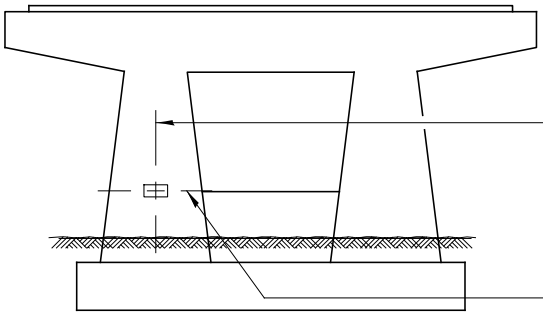
A	-	Grading
AC	-	Access Control – Frontage Roads or other features of access control except bridges
B	-	Bridges (Complete Structures or Substructures only)
BR	-	Bridge Reconstruction
BY	-	Bridge Widening
D	-	Bridge Floors
E	-	Steel Erection
F	-	Steel Fabrication
FL	-	Railroad Crossing Protection
HB	-	Highway Grade Separation
HVB	-	Highway Railroad Grade Separation
I	-	Miscellaneous
K	-	Interchange Work except bridges
K-HB	-	Interchange Grade Separations
K-HVB	-	Interchange Grade Separation over Highway and Railroad
L	-	Lighting
P	-	Painting
R	-	Reconstruction
RS	-	Resurfacing
SB	-	Subway (Railroad)
T	-	Storm Sewers or Deficient Drainage Correction
TS	-	Traffic Signals
VB	-	Viaducts (Railroad)
W	-	Pavement Widening
Y	-	Widening Shoulders and Ditches
Z	-	City Pavement

No suffix is to be used for pavement sections

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A2.3 Name Plate Location for FAI Routes





COLUMN TYPE OF PIERS

℄ of Column, Column nearest approaching traffic

SOLID PIERS

±3'-0" in from pier end, face that is facing approaching traffic

ALL PIERS

±4'-0" above crown of Roadway Elev.

GENERAL POLICY

For FAI Routes Under: Use one name plate on center pier.
For FAI Routes Over: Use one name plate on parapet. See Figure A2.5.
For Rebuilt Structures: Locate new name plate next to existing name plate.

NAME PLATE LOCATION FOR FAI ROUTES

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A2.4 Lettering for Name Plates

Refer all Name Plates to Std. 515001

1. F.A.I. Routes:

STA. 215+25
BUILT BY
STATE OF ILLINOIS
F.A.I. RT. 57 SEC. 24-IHB-2
LOADING
STR. NO. -

2. F.A. & S.B.I. Routes:

STA. 17+63
BUILT BY
STATE OF ILLINOIS
F.A. RT. 157 SEC. 3B-2
LOADING
STR. NO. -

3. Local Agency (Federal-aid):

NAME OF STREAM
BUILT BY
ADAMS COUNTY
SEC. 83-00127-00-BR
F.A. RT. 1414 STA. 25+78
STR. NO. - LOADING

4. City or County M.F.T. :

NAME OF STREAM
BUILT BY
CITY OF QUINCY
SEC. 80-00014-00-BR
STA. 29+42
STR. NO. - LOADING

5. Township M.F.T. :

NAME OF STREAM
BUILT BY
COTTON HILL ROAD DISTRICT
SANGAMON COUNTY
SEC. 83-09021-00-BR
STA. 17+68
STR. NO. - LOADING

6. R.R. Overhead and Subway:

C. & N.W. R.R.
BUILT BY
STATE OF ILLINOIS
F.A.I. RT. 57 SEC. 24-IVB-3
STA. 226+42 LOADING
STR. NO. -

7. Retaining Wall:

STA. 123+50
BUILT 20 BY
STATE OF ILLINOIS
F.A.P. RTE. SEC. 12-3456-78-RP
LOADING HL-93
STR. NO. 123-4567

Notes:
Refer to plans for correct station, loading (HL93, HS20, HS20 & alt., etc.), etc.

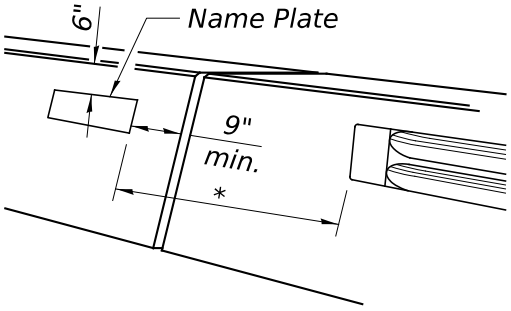
LETTERING FOR NAME PLATES

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A2.5 Name Plate Location on Superstructures and Culverts

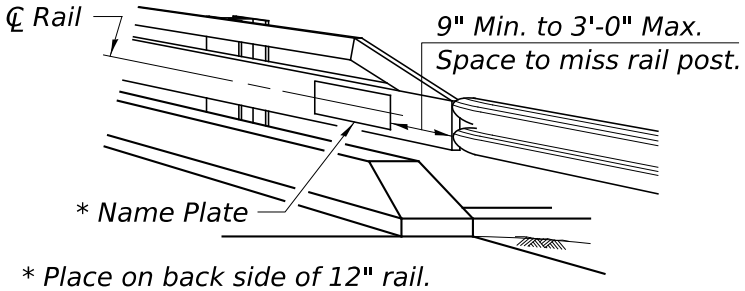
GENERAL POLICY

On one way traffic structures - Place name plate on right side of approach end.
On two way traffic structures - Place name plate on right side of approach end while looking in the direction of increasing stationing.



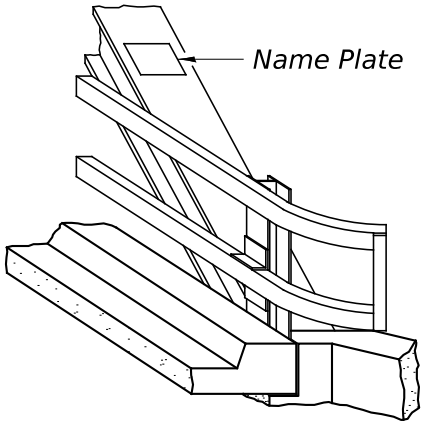
FOR PARAPETS

(*9" min. if no joint)

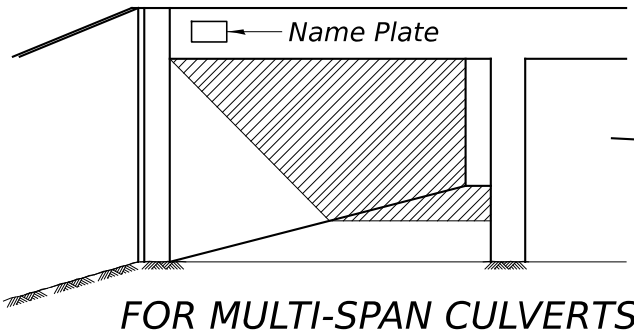


* Place on back side of 12" rail.

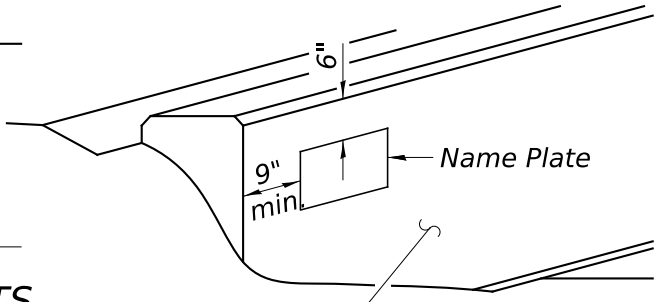
FOR STEEL RAILS



FOR TRUSSES



FOR MULTI-SPAN CULVERTS



FOR WALLS

Notes:

Unless otherwise noted on the plans, Name Plates are required for structures 20 ft. or greater in length.

On dry land bridges, place name plate on parapet. If there is no parapet, embed name plate into curb or sidewalk section.

NAME PLATE LOCATION ON SUPERSTRUCTURES AND CULVERTS

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A3 Design Aids

The following figures and tables have been provided as general references to aid in design.

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INCHES TO DECIMALS OF A FOOT

Fractions of an Inch	Decimals of a Foot											
	0 In.	1 In.	2 In.	3 In.	4 In.	5 In.	6 In.	7 In.	8 In.	9 In.	10 In.	11 In.
0	0.0000	0.0833	0.1667	0.2500	0.3333	0..4167	0.5000	0.5833	0.6667	0.7500	0.8333	0.9167
1/16	0.0052	0.0885	0.1719	0.2552	0.3385	0.4219	0.5052	0.5885	0.6719	0.7552	0.8385	0.9219
	0.0104	0.0938	0.1771	0.2604	0.3438	0.4271	0.5104	0.5938	0.6771	0.7604	0.8438	0.9271
3/16	0.0156	0.0990	0.1823	0.2656	0.3490	0.4323	0.5156	0.5990	0.6823	0.7656	0.8490	0.9323
1/4	0.0208	0.1042	0.1875	0.2708	0.3542	0.4375	0.5208	0.6042	0.6875	0.7708	0.8542	0.9375
5/16	0.0260	0.1094	0.1927	0.2760	0.3594	0.4427	0.5260	0.6094	0.6927	0.7760	0.8594	0.9427
3/8	0.0313	0.1146	0.1979	0.2813	0.3646	0.4479	0.5313	0.6146	0.6979	0.7813	0.8646	0.9479
	0.0365	0.1198	0.2031	0.2865	0.3698	0.4531	0.5365	0.6198	0.7031	0.7865	0.8698	0.9531
1/2	0.0417	0.1250	0.2083	0.2917	0.3750	0.4583	0.5417	0.6250	0.7083	0.7917	0.8750	0.9583
	0.0469	0.1302	0.2135	0.2969	0.3802	0.4635	0.5469	0.6302	0.7135	0.7969	0.8802	0.9635
5/8	0.0521	0.1354	0.2188	0.3021	0.3854	0.4688	0.5521	0.6354	0.7188	0.8021	0.8854	0.9688
	0.0573	0.1406	0.2240	0.3073	0.3906	0.4740	0.5573	0.6406	0.7240	0.8073	0.8906	0.9740
	0.0625	0.1458	0.2292	0.3125	0.3958	0.4792	0.5625	0.6458	0.7292	0.8125	0.8958	0.9792
13/16	0.0677	0.1510	0.2344	0.3177	0.4010	0.4844	0.5677	0.6510	0.7344	0.8177	0.9010	0.9844
	0.0729	0.1563	0.2396	0.3229	0.4063	0.4896	0.5729	0.6563	0.7396	0.8229	0.9063	0.9896
	0.0781	0.1615	0.2448	0.3281	0.4115	0.4948	0.5781	0.6615	0.7448	0.8281	0.9115	0.9948

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REINFORCEMENT BARS
AREAS, WEIGHTS, PERIMETERS & SPACING PER ONE FT. SECTION

Size	Area Sq. in.	Wt. per ft. Lbs.	AREAS - As, given in **bold** type (top) in sq. inches. PERIMETERS - Σo, given in light type (bottom) in inches.													
			4"	4½"	5"	5½"	6"	6½"	7"	7½"	8"	8½"	9"	10"	11"	12"
2	.0490	.167	**.147**	**.131**	**.118**	**.107**	**.098**	**.090**	**.084**	**.078**	**.074**	**.069**	**.065**	**.059**	**.053**	**.049**
			2.36	2.10	1.89	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	.943	.858	.786
3	.1104	.376	**.33**	**.29**	**.27**	**.24**	**.22**	**.20**	**.19**	**.18**	**.17**	**.16**	**.15**	**.13**	**.12**	**.11**
			3.54	3.14	2.83	2.57	2.36	2.18	2.02	1.89	1.77	1.66	1.57	1.41	1.29	1.18
4	.1963	.668	**.59**	**.52**	**.47**	**.43**	*.39*	**.36**	**.34**	**.31**	**.29**	**.28**	**.26**	**.24**	**.21**	**.20**
			4.71	4.19	3.77	3.43	3.14	2.90	2.69	2.51	2.36	2.22	2.09	1.88	1.71	1.57
5	.3068	1.043	**.92**	**.82**	**.74**	**.67**	**.61**	**.57**	**.53**	**.49**	**.46**	**.43**	**.41**	**.37**	**.33**	**.31**
			5.89	5.24	4.71	4.28	3.93	3.62	3.36	3.14	2.94	2.77	2.62	2.36	2.14	1.96
6	.4418	1.502	**1.32**	**1.18**	**1.06**	**.96**	**.88**	**.82**	**.76**	**.71**	**.66**	**.62**	**.59**	**.53**	*.48*	**.44**
			7.07	6.28	5.66	5.14	4.71	4.35	4.04	3.77	3.53	3.33	3.14	2.83	2.57	2.36
7	.6013	2.044	**1.80**	**1.60**	**1.44**	**1.31**	**1.20**	**1.11**	**1.03**	**.96**	**.90**	**.85**	**.80**	**.72**	**.66**	**.60**
			8.25	7.33	6.60	6.00	5.50	5.07	4.71	4.40	4.12	3.88	3.67	3.30	3.00	2.75
8	.7854	2.670	**2.36**	**2.09**	**1.88**	**1.71**	**1.57**	**1.45**	**1.35**	**1.26**	**1.18**	**1.11**	**1.05**	**.94**	**.86**	**.79**
			9.42	8.38	7.54	6.86	6.28	5.80	5.39	5.03	4.71	4.44	4.19	3.77	3.43	3.14
9	1.000	3.400	**3.00**	**2.67**	**2.40**	**2.18**	**2.00**	**1.85**	**1.71**	**1.60**	**1.50**	**1.41**	**1.33**	**1.20**	**1.09**	**1.00**
			10.63	9.45	8.51	7.73	7.09	6.54	6.08	5.67	5.32	5.00	4.73	4.25	3.87	3.54
10	1.2667	4.303	**3.80**	**3.38**	**3.04**	**2.76**	**2.53**	**2.34**	**2.17**	**2.03**	**1.90**	**1.79**	**1.69**	**1.52**	**1.38**	**1.27**
			11.96	10.63	9.58	8.71	7.98	7.37	6.84	6.39	5.99	5.64	5.32	4.79	4.35	3.99
11	1.5615	5.313	**4.69**	**4.17**	**3.75**	**3.41**	**3.13**	**2.89**	**2.68**	**2.50**	**2.34**	**2.21**	**2.08**	**1.87**	**1.70**	**1.56**
			13.28	11.80	10.63	9.67	8.86	8.18	7.60	7.09	6.65	6.26	5.91	5.32	4.83	4.43

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180° END HOOK

90° END HOOK

STIRRUP & TIE HOOKS

180° END HOOK

All Grades

Bar Size	Hook A or G	J
#4	6"	4"
#5	7"	5"
#6	8"	6"
#7	10"	7"
#8	11"	8"
#9	1'-3"	11¾"
#10	1'-5"	1'-1¼"
#11	1'-7"	1'-2¾"
#14	2'-3"	1'-9¾"
#18	3'-0"	2'-4½"

90° END HOOK

All Grades

Bar Size	Hook A or G	Radius to Bar Center R
#4	8"	1¾"
#5	10"	2 3/16"
#6	1'-0"	2⅝"
#7	1'-2"	3 1/16"
#8	1'-4"	3½"
#9	1'-7"	5 5/16"
#10	1'-10"	6"
#11	2'-0"	6⅝"
#14	2'-7"	10"
#18	3'-5"	13⅛"

STIRRUP & TIE HOOKS

Grades 40-50-60 ksi

Bar Size	90° Hook	135° Hook
	Hook A or G	Hook A or G
#4	4½"	4½"
#5	6"	5½"
#6	1'-0"	8"

REINFORCEMENT BAR HOOK DIMENSIONS

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A3.4 Reinforcement Bar Tension Splices (Normal Weight Concrete)

A3.4.1 General

Reinforcement Bar Splices
(for Coated and Uncoated Bars)

Tension Splices (AASHTO LRFD Bridge Design Specifications, 10th Ed., 5.10.8.4.3a)

I. Regions of high tensile stress or
Reinforcement provided < twice that required for strength.

1. Use Class B splice

II. Regions of low tensile stress or
Reinforcement provided > twice that required for strength.

1. No more than 1/2 of the bars are lap spliced within a required lap length.
Use Class A splice

2. More than 1/2 of the bars are lap spliced within a required lap length.
Use Class B splice

III. Assume all temperature and shrinkage reinforcement fully stressed. A Class B splice shall be used.

Notation of Variables

l_d = Modified tension development length of a deformed bar (in)

= $l_{db} \times (\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc})$

l_{db} = Basic development length (in)

$$= 0.17 d_b \left(\frac{\lambda_{er} f_y - \frac{F_h}{A_b}}{1.97 \lambda f_c^{'0.25}} \right)^2$$

s = Spacing of reinforcement in a concrete-encased shape (in)

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A3.4.2 Epoxy Coated, f'c = 3,500 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 3500 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-10"	1'-4"	2'-4"	1'-8"
	1"	2'-2"	2'-2"	2'-9"	2'-9"
#5	2"	2'-9"	1'-7"	3'-7"	2'-1"
	1.5"	2'-9"	2'-4"	3'-7"	3'-0"
	1"	3'-2"	3'-2"	4'-1"	4'-1"
#6	2.5"	4'-0"	1'-11"	5'-2"	2'-6"
	2"	4'-0"	2'-6"	5'-2"	3'-3"
	1.5"	4'-0"	3'-2"	5'-2"	4'-2"
	1"	4'-4"	4'-4"	5'-8"	5'-8"
#7	2.5"	4'-8"	2'-10"	6'-0"	3'-8"
	2"	4'-8"	3'-4"	6'-0"	4'-4"
	1.5"	4'-8"	4'-2"	6'-0"	5'-5"
	1"	5'-8"	5'-8"	7'-4"	7'-4"
#8	2.5"	6'-1"	3'-7"	7'-10"	4'-7"
	2"	6'-1"	4'-3"	7'-10"	5'-6"
	1.5"	6'-1"	5'-4"	7'-10"	6'-11"
	1"	7'-1"	7'-1"	9'-2"	9'-2"
#9	2.5"	7'-8"	4'-6"	10'-0"	5'-10"
	2"	7'-8"	5'-3"	10'-0"	6'-10"
	1.5"	7'-8"	6'-6"	10'-0"	8'-6"
	1"	8'-7"	8'-7"	11'-2"	11'-2"
#10	2.5"	8'-6"	5'-8"	11'-1"	7'-5"
	2"	8'-6"	6'-6"	11'-1"	8'-5"
	1.5"	8'-6"	8'-0"	11'-1"	10'-5"
	1"	10'-5"	10'-5"	13'-7"	13'-7"
#11	2.5"	10'-6"	7'-0"	13'-8"	9'-1"
	2"	10'-6"	7'-9"	13'-8"	10'-1"
	1.5"	10'-6"	9'-7"	13'-8"	12'-5"
	1"	12'-4"	12'-4"	16'-0"	16'-0"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	2'-0"	1'-8"	2'-8"	2'-2"
	1"	2'-5"	2'-5"	3'-2"	3'-2"
#5	2"	3'-2"	2'-1"	4'-1"	2'-9"
	1.5"	3'-2"	2'-7"	4'-1"	3'-5"
	1"	3'-7"	3'-7"	4'-8"	4'-8"
#6	2.5"	4'-6"	2'-6"	5'-10"	3'-3"
	2"	4'-6"	2'-10"	5'-10"	3'-9"
	1.5"	4'-6"	3'-7"	5'-10"	4'-8"
	1"	4'-11"	4'-11"	6'-5"	6'-5"
#7	2.5"	5'-3"	3'-2"	6'-10"	4'-1"
	2"	5'-3"	3'-10"	6'-10"	4'-11"
	1.5"	5'-3"	4'-9"	6'-10"	6'-2"
	1"	6'-5"	6'-5"	8'-4"	8'-4"
#8	2.5"	6'-10"	4'-0"	8'-11"	5'-3"
	2"	6'-10"	4'-10"	8'-11"	6'-3"
	1.5"	6'-10"	6'-0"	8'-11"	7'-10"
	1"	8'-0"	8'-0"	10'-5"	10'-5"
#9	2.5"	8'-9"	5'-1"	11'-4"	6'-8"
	2"	8'-9"	6'-0"	11'-4"	7'-9"
	1.5"	8'-9"	7'-5"	11'-4"	9'-7"
	1"	9'-9"	9'-9"	12'-8"	12'-8"
#10	2.5"	9'-8"	6'-6"	12'-7"	8'-5"
	2"	9'-8"	7'-4"	12'-7"	9'-7"
	1.5"	9'-8"	9'-1"	12'-7"	11'-9"
	1"	11'-10"	11'-10"	15'-4"	15'-4"
#11	2.5"	11'-11"	7'-11"	15'-6"	10'-4"
	2"	11'-11"	8'-10"	15'-6"	11'-5"
	1.5"	11'-11"	10'-10"	15'-6"	14'-0"
	1"	14'-0"	14'-0"	18'-2"	18'-2"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Epoxy Coated, f'c = 4,000 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 4000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-8"	1'-3"	2'-2"	1'-7"
	1"	2'-0"	2'-0"	2'-7"	2'-7"
#5	2"	2'-7"	1'-6"	3'-5"	2'-0"
	1.5"	2'-7"	2'-2"	3'-5"	2'-10"
	1"	3'-0"	3'-0"	3'-10"	3'-10"
#6	2.5"	3'-9"	1'-10"	4'-10"	2'-4"
	2"	3'-9"	2'-5"	4'-10"	3'-1"
	1.5"	3'-9"	3'-0"	4'-10"	3'-11"
	1"	4'-1"	4'-1"	5'-3"	5'-3"
#7	2.5"	4'-4"	2'-8"	5'-8"	3'-5"
	2"	4'-4"	3'-2"	5'-8"	4'-1"
	1.5"	4'-4"	3'-11"	5'-8"	5'-1"
	1"	5'-3"	5'-3"	6'-10"	6'-10"
#8	2.5"	5'-8"	3'-4"	7'-4"	4'-4"
	2"	5'-8"	4'-0"	7'-4"	5'-2"
	1.5"	5'-8"	5'-0"	7'-4"	6'-5"
	1"	6'-7"	6'-7"	8'-7"	8'-7"
#9	2.5"	7'-2"	4'-3"	9'-4"	5'-6"
	2"	7'-2"	4'-11"	9'-4"	6'-5"
	1.5"	7'-2"	6'-1"	9'-4"	7'-11"
	1"	8'-1"	8'-1"	10'-6"	10'-6"
#10	2.5"	8'-0"	5'-4"	10'-4"	6'-11"
	2"	8'-0"	6'-1"	10'-4"	7'-11"
	1.5"	8'-0"	7'-6"	10'-4"	9'-9"
	1"	9'-9"	9'-9"	12'-8"	12'-8"
#11	2.5"	9'-10"	6'-7"	12'-9"	8'-6"
	2"	9'-10"	7'-3"	12'-9"	9'-6"
	1.5"	9'-10"	8'-11"	12'-9"	11'-7"
	1"	11'-6"	11'-6"	15'-0"	15'-0"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-11"	1'-7"	2'-6"	2'-0"
	1"	2'-3"	2'-3"	2'-11"	2'-11"
#5	2"	2'-11"	2'-0"	3'-10"	2'-6"
	1.5"	2'-11"	2'-5"	3'-10"	3'-2"
	1"	3'-4"	3'-4"	4'-4"	4'-4"
#6	2.5"	4'-3"	2'-4"	5'-6"	3'-0"
	2"	4'-3"	2'-8"	5'-6"	3'-6"
	1.5"	4'-3"	3'-5"	5'-6"	4'-5"
	1"	4'-7"	4'-7"	6'-0"	6'-0"
#7	2.5"	4'-11"	3'-0"	6'-5"	3'-10"
	2"	4'-11"	3'-7"	6'-5"	4'-7"
	1.5"	4'-11"	4'-5"	6'-5"	5'-9"
	1"	6'-0"	6'-0"	7'-9"	7'-9"
#8	2.5"	6'-5"	3'-9"	8'-4"	4'-11"
	2"	6'-5"	4'-6"	8'-4"	5'-10"
	1.5"	6'-5"	5'-8"	8'-4"	7'-4"
	1"	7'-6"	7'-6"	9'-9"	9'-9"
#9	2.5"	8'-2"	4'-9"	10'-7"	6'-2"
	2"	8'-2"	5'-7"	10'-7"	7'-3"
	1.5"	8'-2"	6'-11"	10'-7"	9'-0"
	1"	9'-2"	9'-2"	11'-10"	11'-10"
#10	2.5"	9'-1"	6'-1"	11'-9"	7'-10"
	2"	9'-1"	6'-11"	11'-9"	8'-11"
	1.5"	9'-1"	8'-6"	11'-9"	11'-0"
	1"	11'-1"	11'-1"	14'-4"	14'-4"
#11	2.5"	11'-2"	7'-5"	14'-6"	9'-8"
	2"	11'-2"	8'-3"	14'-6"	10'-9"
	1.5"	11'-2"	10'-1"	14'-6"	13'-2"
	1"	13'-1"	13'-1"	17'-0"	17'-0"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Epoxy Coated, f'c = 4,500 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 4500 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-7"	1'-2"	2'-1"	1'-6"
	1"	1'-11"	1'-11"	2'-5"	2'-5"
#5	2"	2'-6"	1'-5"	3'-2"	1'-10"
	1.5"	2'-6"	2'-1"	3'-2"	2'-8"
	1"	2'-10"	2'-10"	3'-8"	3'-8"
#6	2.5"	3'-6"	1'-9"	4'-7"	2'-3"
	2"	3'-6"	2'-3"	4'-7"	2'-11"
	1.5"	3'-6"	2'-10"	4'-7"	3'-8"
	1"	3'-10"	3'-10"	5'-0"	5'-0"
#7	2.5"	4'-1"	2'-6"	5'-4"	3'-3"
	2"	4'-1"	3'-0"	5'-4"	3'-10"
	1.5"	4'-1"	3'-9"	5'-4"	4'-10"
	1"	5'-0"	5'-0"	6'-6"	6'-6"
#8	2.5"	5'-4"	3'-2"	6'-11"	4'-1"
	2"	5'-4"	3'-9"	6'-11"	4'-10"
	1.5"	5'-4"	4'-8"	6'-11"	6'-1"
	1"	6'-3"	6'-3"	8'-1"	8'-1"
#9	2.5"	6'-10"	4'-0"	8'-10"	5'-2"
	2"	6'-10"	4'-8"	8'-10"	6'-0"
	1.5"	6'-10"	5'-9"	8'-10"	7'-6"
	1"	7'-7"	7'-7"	9'-10"	9'-10"
#10	2.5"	7'-6"	5'-0"	9'-9"	6'-6"
	2"	7'-6"	5'-9"	9'-9"	7'-5"
	1.5"	7'-6"	7'-1"	9'-9"	9'-2"
	1"	9'-3"	9'-3"	12'-0"	12'-0"
#11	2.5"	9'-3"	6'-2"	12'-1"	8'-1"
	2"	9'-3"	6'-10"	12'-1"	8'-11"
	1.5"	9'-3"	8'-5"	12'-1"	10'-11"
	1"	10'-11"	10'-11"	14'-2"	14'-2"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-10"	1'-6"	2'-4"	1'-11"
	1"	2'-2"	2'-2"	2'-9"	2'-9"
#5	2"	2'-9"	1'-10"	3'-7"	2'-5"
	1.5"	2'-9"	2'-4"	3'-7"	3'-0"
	1"	3'-2"	3'-2"	4'-1"	4'-1"
#6	2.5"	4'-0"	2'-3"	5'-2"	2'-10"
	2"	4'-0"	2'-6"	5'-2"	3'-3"
	1.5"	4'-0"	3'-2"	5'-2"	4'-2"
	1"	4'-4"	4'-4"	5'-8"	5'-8"
#7	2.5"	4'-8"	2'-10"	6'-0"	3'-8"
	2"	4'-8"	3'-4"	6'-0"	4'-4"
	1.5"	4'-8"	4'-2"	6'-0"	5'-5"
	1"	5'-8"	5'-8"	7'-4"	7'-4"
#8	2.5"	6'-1"	3'-7"	7'-10"	4'-7"
	2"	6'-1"	4'-3"	7'-10"	5'-6"
	1.5"	6'-1"	5'-4"	7'-10"	6'-11"
	1"	7'-1"	7'-1"	9'-2"	9'-2"
#9	2.5"	7'-8"	4'-6"	10'-0"	5'-10"
	2"	7'-8"	5'-3"	10'-0"	6'-10"
	1.5"	7'-8"	6'-6"	10'-0"	8'-6"
	1"	8'-7"	8'-7"	11'-2"	11'-2"
#10	2.5"	8'-6"	5'-8"	11'-1"	7'-5"
	2"	8'-6"	6'-6"	11'-1"	8'-5"
	1.5"	8'-6"	8'-0"	11'-1"	10'-5"
	1"	10'-5"	10'-5"	13'-7"	13'-7"
#11	2.5"	10'-6"	7'-0"	13'-8"	9'-1"
	2"	10'-6"	7'-9"	13'-8"	10'-1"
	1.5"	10'-6"	9'-6"	13'-8"	12'-5"
	1"	12'-4"	12'-4"	16'-0"	16'-0"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Epoxy Coated, f'c = 5,000 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 5000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-6"	1'-1"	1'-11"	1'-5"
	1"	1'-10"	1'-10"	2'-4"	2'-4"
#5	2"	2'-4"	1'-4"	3'-0"	1'-9"
	1.5"	2'-4"	1'-11"	3'-0"	2'-6"
	1"	2'-8"	2'-8"	3'-5"	3'-5"
#6	2.5"	3'-4"	1'-8"	4'-4"	2'-1"
	2"	3'-4"	2'-2"	4'-4"	2'-9"
	1.5"	3'-4"	2'-8"	4'-4"	3'-6"
	1"	3'-8"	3'-8"	4'-9"	4'-9"
#7	2.5"	3'-11"	2'-4"	5'-1"	3'-1"
	2"	3'-11"	2'-10"	5'-1"	3'-8"
	1.5"	3'-11"	3'-6"	5'-1"	4'-7"
	1"	4'-9"	4'-9"	6'-2"	6'-2"
#8	2.5"	5'-1"	3'-0"	6'-7"	3'-10"
	2"	5'-1"	3'-7"	6'-7"	4'-8"
	1.5"	5'-1"	4'-5"	6'-7"	5'-9"
	1"	5'-11"	5'-11"	7'-8"	7'-8"
#9	2.5"	6'-5"	3'-9"	8'-4"	4'-11"
	2"	6'-5"	4'-5"	8'-4"	5'-9"
	1.5"	6'-5"	5'-6"	8'-4"	7'-1"
	1"	7'-3"	7'-3"	9'-4"	9'-4"
#10	2.5"	7'-2"	4'-9"	9'-3"	6'-2"
	2"	7'-2"	5'-5"	9'-3"	7'-1"
	1.5"	7'-2"	6'-8"	9'-3"	8'-8"
	1"	8'-9"	8'-9"	11'-4"	11'-4"
#11	2.5"	8'-10"	5'-11"	11'-5"	7'-8"
	2"	8'-10"	6'-6"	11'-5"	8'-6"
	1.5"	8'-10"	8'-0"	11'-5"	10'-4"
	1"	10'-4"	10'-4"	13'-5"	13'-5"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-8"	1'-5"	2'-2"	1'-10"
	1"	2'-0"	2'-0"	2'-8"	2'-8"
#5	2"	2'-8"	1'-9"	3'-5"	2'-3"
	1.5"	2'-8"	2'-2"	3'-5"	2'-10"
	1"	3'-0"	3'-0"	3'-11"	3'-11"
#6	2.5"	3'-9"	2'-1"	4'-11"	2'-9"
	2"	3'-9"	2'-5"	4'-11"	3'-1"
	1.5"	3'-9"	3'-0"	4'-11"	3'-11"
	1"	4'-2"	4'-2"	5'-4"	5'-4"
#7	2.5"	4'-5"	2'-8"	5'-9"	3'-5"
	2"	4'-5"	3'-2"	5'-9"	4'-1"
	1.5"	4'-5"	4'-0"	5'-9"	5'-2"
	1"	5'-4"	5'-4"	7'-0"	7'-0"
#8	2.5"	5'-9"	3'-4"	7'-6"	4'-4"
	2"	5'-9"	4'-0"	7'-6"	5'-3"
	1.5"	5'-9"	5'-0"	7'-6"	6'-6"
	1"	6'-8"	6'-8"	8'-8"	8'-8"
#9	2.5"	7'-4"	4'-3"	9'-6"	5'-7"
	2"	7'-4"	5'-0"	9'-6"	6'-6"
	1.5"	7'-4"	6'-2"	9'-6"	8'-1"
	1"	8'-2"	8'-2"	10'-7"	10'-7"
#10	2.5"	8'-1"	5'-5"	10'-6"	7'-0"
	2"	8'-1"	6'-2"	10'-6"	8'-0"
	1.5"	8'-1"	7'-7"	10'-6"	9'-10"
	1"	9'-11"	9'-11"	12'-10"	12'-10"
#11	2.5"	10'-0"	6'-8"	12'-11"	8'-8"
	2"	10'-0"	7'-5"	12'-11"	9'-7"
	1.5"	10'-0"	9'-1"	12'-11"	11'-9"
	1"	11'-8"	11'-8"	15'-2"	15'-2"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Epoxy Coated, f'c = 6,000 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 6000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-5"	1'-0"	1'-9"	1'-4"
	1"	1'-8"	1'-8"	2'-2"	2'-2"
#5	2"	2'-2"	1'-3"	2'-9"	1'-7"
	1.5"	2'-2"	1'-9"	2'-9"	2'-4"
	1"	2'-5"	2'-5"	3'-2"	3'-2"
#6	2.5"	3'-1"	1'-6"	4'-0"	1'-11"
	2"	3'-1"	1'-11"	4'-0"	2'-6"
	1.5"	3'-1"	2'-5"	4'-0"	3'-2"
	1"	3'-4"	3'-4"	4'-4"	4'-4"
#7	2.5"	3'-7"	2'-2"	4'-7"	2'-9"
	2"	3'-7"	2'-7"	4'-7"	3'-4"
	1.5"	3'-7"	3'-3"	4'-7"	4'-2"
	1"	4'-4"	4'-4"	5'-7"	5'-7"
#8	2.5"	4'-8"	2'-9"	6'-0"	3'-6"
	2"	4'-8"	3'-3"	6'-0"	4'-3"
	1.5"	4'-8"	4'-1"	6'-0"	5'-3"
	1"	5'-5"	5'-5"	7'-0"	7'-0"
#9	2.5"	5'-11"	3'-5"	7'-8"	4'-6"
	2"	5'-11"	4'-0"	7'-8"	5'-3"
	1.5"	5'-11"	5'-0"	7'-8"	6'-6"
	1"	6'-7"	6'-7"	8'-7"	8'-7"
#10	2.5"	6'-6"	4'-4"	8'-6"	5'-8"
	2"	6'-6"	5'-0"	8'-6"	6'-5"
	1.5"	6'-6"	6'-1"	8'-6"	7'-11"
	1"	8'-0"	8'-0"	10'-4"	10'-4"
#11	2.5"	8'-0"	5'-4"	10'-5"	7'-0"
	2"	8'-0"	5'-11"	10'-5"	7'-9"
	1.5"	8'-0"	7'-4"	10'-5"	9'-6"
	1"	9'-5"	9'-5"	12'-3"	12'-3"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-7"	1'-4"	2'-0"	1'-8"
	1"	1'-10"	1'-10"	2'-5"	2'-5"
#5	2"	2'-5"	1'-7"	3'-2"	2'-1"
	1.5"	2'-5"	2'-0"	3'-2"	2'-7"
	1"	2'-9"	2'-9"	3'-7"	3'-7"
#6	2.5"	3'-6"	1'-11"	4'-6"	2'-6"
	2"	3'-6"	2'-2"	4'-6"	2'-10"
	1.5"	3'-6"	2'-9"	4'-6"	3'-7"
	1"	3'-9"	3'-9"	4'-11"	4'-11"
#7	2.5"	4'-0"	2'-5"	5'-3"	3'-2"
	2"	4'-0"	2'-11"	5'-3"	3'-9"
	1.5"	4'-0"	3'-8"	5'-3"	4'-9"
	1"	4'-11"	4'-11"	6'-4"	6'-4"
#8	2.5"	5'-3"	3'-1"	6'-10"	4'-0"
	2"	5'-3"	3'-8"	6'-10"	4'-9"
	1.5"	5'-3"	4'-7"	6'-10"	6'-0"
	1"	6'-1"	6'-1"	7'-11"	7'-11"
#9	2.5"	6'-8"	3'-11"	8'-8"	5'-1"
	2"	6'-8"	4'-7"	8'-8"	5'-11"
	1.5"	6'-8"	5'-8"	8'-8"	7'-4"
	1"	7'-6"	7'-6"	9'-8"	9'-8"
#10	2.5"	7'-5"	4'-11"	9'-7"	6'-5"
	2"	7'-5"	5'-7"	9'-7"	7'-4"
	1.5"	7'-5"	6'-11"	9'-7"	9'-0"
	1"	9'-0"	9'-0"	11'-9"	11'-9"
#11	2.5"	9'-1"	6'-1"	11'-10"	7'-11"
	2"	9'-1"	6'-9"	11'-10"	8'-9"
	1.5"	9'-1"	8'-3"	11'-10"	10'-9"
	1"	10'-8"	10'-8"	13'-10"	13'-10"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Epoxy Coated, f'c = 7,000 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 7000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-3"	1'-0"	1'-8"	1'-4"
	1"	1'-6"	1'-6"	2'-0"	2'-0"
#5	2"	2'-0"	1'-2"	2'-7"	1'-6"
	1.5"	2'-0"	1'-8"	2'-7"	2'-2"
	1"	2'-3"	2'-3"	2'-11"	2'-11"
#6	2.5"	2'-10"	1'-5"	3'-8"	1'-9"
	2"	2'-10"	1'-10"	3'-8"	2'-4"
	1.5"	2'-10"	2'-3"	3'-8"	2'-11"
	1"	3'-1"	3'-1"	4'-0"	4'-0"
#7	2.5"	3'-4"	2'-0"	4'-3"	2'-7"
	2"	3'-4"	2'-5"	4'-3"	3'-1"
	1.5"	3'-4"	3'-0"	4'-3"	3'-10"
	1"	4'-0"	4'-0"	5'-2"	5'-2"
#8	2.5"	4'-4"	2'-6"	5'-7"	3'-3"
	2"	4'-4"	3'-0"	5'-7"	3'-11"
	1.5"	4'-4"	3'-9"	5'-7"	4'-11"
	1"	5'-0"	5'-0"	6'-6"	6'-6"
#9	2.5"	5'-6"	3'-2"	7'-1"	4'-2"
	2"	5'-6"	3'-9"	7'-1"	4'-10"
	1.5"	5'-6"	4'-8"	7'-1"	6'-0"
	1"	6'-1"	6'-1"	7'-11"	7'-11"
#10	2.5"	6'-1"	4'-1"	7'-10"	5'-3"
	2"	6'-1"	4'-7"	7'-10"	6'-0"
	1.5"	6'-1"	5'-8"	7'-10"	7'-4"
	1"	7'-5"	7'-5"	9'-7"	9'-7"
#11	2.5"	7'-5"	5'-0"	9'-8"	6'-6"
	2"	7'-5"	5'-6"	9'-8"	7'-2"
	1.5"	7'-5"	6'-9"	9'-8"	8'-9"
	1"	8'-9"	8'-9"	11'-4"	11'-4"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-5"	1'-2"	1'-10"	1'-7"
	1"	1'-9"	1'-9"	2'-3"	2'-3"
#5	2"	2'-3"	1'-6"	2'-11"	1'-11"
	1.5"	2'-3"	1'-10"	2'-11"	2'-5"
	1"	2'-7"	2'-7"	3'-4"	3'-4"
#6	2.5"	3'-2"	1'-9"	4'-2"	2'-4"
	2"	3'-2"	2'-0"	4'-2"	2'-8"
	1.5"	3'-2"	2'-7"	4'-2"	3'-4"
	1"	3'-6"	3'-6"	4'-6"	4'-6"
#7	2.5"	3'-9"	2'-3"	4'-10"	2'-11"
	2"	3'-9"	2'-8"	4'-10"	3'-6"
	1.5"	3'-9"	3'-5"	4'-10"	4'-5"
	1"	4'-6"	4'-6"	5'-11"	5'-11"
#8	2.5"	4'-10"	2'-10"	6'-4"	3'-8"
	2"	4'-10"	3'-5"	6'-4"	4'-5"
	1.5"	4'-10"	4'-3"	6'-4"	5'-6"
	1"	5'-8"	5'-8"	7'-4"	7'-4"
#9	2.5"	6'-2"	3'-7"	8'-0"	4'-8"
	2"	6'-2"	4'-3"	8'-0"	5'-6"
	1.5"	6'-2"	5'-3"	8'-0"	6'-10"
	1"	6'-11"	6'-11"	9'-0"	9'-0"
#10	2.5"	6'-10"	4'-7"	8'-11"	5'-11"
	2"	6'-10"	5'-3"	8'-11"	6'-9"
	1.5"	6'-10"	6'-5"	8'-11"	8'-4"
	1"	8'-4"	8'-4"	10'-10"	10'-10"
#11	2.5"	8'-5"	5'-8"	10'-11"	7'-4"
	2"	8'-5"	6'-3"	10'-11"	8'-1"
	1.5"	8'-5"	7'-8"	10'-11"	9'-11"
	1"	9'-11"	9'-11"	12'-10"	12'-10"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Epoxy Coated, f'c = 8,500 psi, fy = 60,000 psi

Lap Splices in Tension for Epoxy Coated Reinforcement in Normal Weight Concrete

(f'c = 8500 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-2"	1'-0"	1'-6"	1'-4"
	1"	1'-5"	1'-5"	1'-10"	1'-10"
#5	2"	1'-10"	1'-1"	2'-4"	1'-4"
	1.5"	1'-10"	1'-6"	2'-4"	1'-11"
	1"	2'-1"	2'-1"	2'-8"	2'-8"
#6	2.5"	2'-7"	1'-3"	3'-4"	1'-7"
	2"	2'-7"	1'-8"	3'-4"	2'-1"
	1.5"	2'-7"	2'-1"	3'-4"	2'-8"
	1"	2'-10"	2'-10"	3'-8"	3'-8"
#7	2.5"	3'-0"	1'-10"	3'-11"	2'-4"
	2"	3'-0"	2'-2"	3'-11"	2'-10"
	1.5"	3'-0"	2'-9"	3'-11"	3'-6"
	1"	3'-8"	3'-8"	4'-9"	4'-9"
#8	2.5"	3'-11"	2'-4"	5'-1"	3'-0"
	2"	3'-11"	2'-9"	5'-1"	3'-7"
	1.5"	3'-11"	3'-5"	5'-1"	4'-5"
	1"	4'-7"	4'-7"	5'-11"	5'-11"
#9	2.5"	4'-11"	2'-11"	6'-5"	3'-9"
	2"	4'-11"	3'-5"	6'-5"	4'-5"
	1.5"	4'-11"	4'-3"	6'-5"	5'-6"
	1"	5'-7"	5'-7"	7'-2"	7'-2"
#10	2.5"	5'-6"	3'-8"	7'-2"	4'-9"
	2"	5'-6"	4'-2"	7'-2"	5'-5"
	1.5"	5'-6"	5'-2"	7'-2"	6'-8"
	1"	6'-9"	6'-9"	8'-9"	8'-9"
#11	2.5"	6'-9"	4'-6"	8'-9"	5'-10"
	2"	6'-9"	5'-0"	8'-9"	6'-6"
	1.5"	6'-9"	6'-2"	8'-9"	8'-0"
	1"	7'-11"	7'-11"	10'-3"	10'-3"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-4"	1'-1"	1'-8"	1'-5"
	1"	1'-7"	1'-7"	2'-0"	2'-0"
#5	2"	2'-0"	1'-4"	2'-8"	1'-9"
	1.5"	2'-0"	1'-8"	2'-8"	2'-2"
	1"	2'-4"	2'-4"	3'-0"	3'-0"
#6	2.5"	2'-11"	1'-7"	3'-9"	2'-1"
	2"	2'-11"	1'-10"	3'-9"	2'-5"
	1.5"	2'-11"	2'-4"	3'-9"	3'-0"
	1"	3'-2"	3'-2"	4'-1"	4'-1"
#7	2.5"	3'-5"	2'-1"	4'-5"	2'-8"
	2"	3'-5"	2'-5"	4'-5"	3'-2"
	1.5"	3'-5"	3'-1"	4'-5"	4'-0"
	1"	4'-1"	4'-1"	5'-4"	5'-4"
#8	2.5"	4'-5"	2'-7"	5'-9"	3'-4"
	2"	4'-5"	3'-1"	5'-9"	4'-0"
	1.5"	4'-5"	3'-10"	5'-9"	5'-0"
	1"	5'-2"	5'-2"	6'-8"	6'-8"
#9	2.5"	5'-7"	3'-3"	7'-3"	4'-3"
	2"	5'-7"	3'-10"	7'-3"	5'-0"
	1.5"	5'-7"	4'-9"	7'-3"	6'-2"
	1"	6'-3"	6'-3"	8'-2"	8'-2"
#10	2.5"	6'-3"	4'-2"	8'-1"	5'-5"
	2"	6'-3"	4'-9"	8'-1"	6'-2"
	1.5"	6'-3"	5'-10"	8'-1"	7'-7"
	1"	7'-7"	7'-7"	9'-10"	9'-10"
#11	2.5"	7'-8"	5'-1"	9'-11"	6'-8"
	2"	7'-8"	5'-8"	9'-11"	7'-4"
	1.5"	7'-8"	6'-11"	9'-11"	9'-0"
	1"	9'-0"	9'-0"	11'-8"	11'-8"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 3,500 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 3500 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-3"	1'-1"	1'-7"	1'-5"
	1"	1'-5"	1'-5"	1'-10"	1'-10"
#5	2"	1'-10"	1'-4"	2'-5"	1'-9"
	1.5"	1'-10"	1'-7"	2'-5"	2'-0"
	1"	2'-2"	2'-2"	2'-9"	2'-9"
#6	2.5"	2'-8"	1'-7"	3'-6"	2'-1"
	2"	2'-8"	1'-8"	3'-6"	2'-2"
	1.5"	2'-8"	2'-2"	3'-6"	2'-9"
	1"	2'-11"	2'-11"	3'-9"	3'-9"
#7	2.5"	3'-1"	1'-11"	4'-0"	2'-5"
	2"	3'-1"	2'-3"	4'-0"	2'-11"
	1.5"	3'-1"	2'-10"	4'-0"	3'-8"
	1"	3'-9"	3'-9"	4'-11"	4'-11"
#8	2.5"	4'-1"	2'-5"	5'-3"	3'-1"
	2"	4'-1"	2'-10"	5'-3"	3'-8"
	1.5"	4'-1"	3'-7"	5'-3"	4'-7"
	1"	4'-9"	4'-9"	6'-2"	6'-2"
#9	2.5"	5'-2"	3'-0"	6'-8"	3'-11"
	2"	5'-2"	3'-6"	6'-8"	4'-7"
	1.5"	5'-2"	4'-4"	6'-8"	5'-8"
	1"	5'-9"	5'-9"	7'-6"	7'-6"
#10	2.5"	5'-8"	3'-10"	7'-5"	4'-11"
	2"	5'-8"	4'-4"	7'-5"	5'-8"
	1.5"	5'-8"	5'-4"	7'-5"	6'-11"
	1"	7'-0"	7'-0"	9'-1"	9'-1"
#11	2.5"	7'-0"	4'-8"	9'-1"	6'-1"
	2"	7'-0"	5'-2"	9'-1"	6'-9"
	1.5"	7'-0"	6'-5"	9'-1"	8'-3"
	1"	8'-3"	8'-3"	10'-8"	10'-8"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-7"	1'-5"	2'-0"	1'-10"
	1"	1'-10"	1'-10"	2'-5"	2'-5"
#5	2"	2'-5"	1'-9"	3'-2"	2'-3"
	1.5"	2'-5"	2'-0"	3'-2"	2'-7"
	1"	2'-9"	2'-9"	3'-7"	3'-7"
#6	2.5"	3'-6"	2'-1"	4'-6"	2'-9"
	2"	3'-6"	2'-2"	4'-6"	2'-10"
	1.5"	3'-6"	2'-9"	4'-6"	3'-7"
	1"	3'-9"	3'-9"	4'-11"	4'-11"
#7	2.5"	4'-0"	2'-5"	5'-3"	3'-2"
	2"	4'-0"	2'-11"	5'-3"	3'-9"
	1.5"	4'-0"	3'-8"	5'-3"	4'-9"
	1"	4'-11"	4'-11"	6'-4"	6'-4"
#8	2.5"	5'-3"	3'-1"	6'-10"	4'-0"
	2"	5'-3"	3'-8"	6'-10"	4'-9"
	1.5"	5'-3"	4'-7"	6'-10"	6'-0"
	1"	6'-2"	6'-2"	7'-11"	7'-11"
#9	2.5"	6'-8"	3'-11"	8'-8"	5'-1"
	2"	6'-8"	4'-7"	8'-8"	5'-11"
	1.5"	6'-8"	5'-8"	8'-8"	7'-4"
	1"	7'-6"	7'-6"	9'-8"	9'-8"
#10	2.5"	7'-5"	4'-11"	9'-7"	6'-5"
	2"	7'-5"	5'-8"	9'-7"	7'-4"
	1.5"	7'-5"	6'-11"	9'-7"	9'-0"
	1"	9'-1"	9'-1"	11'-9"	11'-9"
#11	2.5"	9'-1"	6'-1"	11'-10"	7'-11"
	2"	9'-1"	6'-9"	11'-10"	8'-9"
	1.5"	9'-1"	8'-3"	11'-10"	10'-9"
	1"	10'-8"	10'-8"	13'-11"	13'-11"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 4,000 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 4000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-2"	1'-0"	1'-6"	1'-4"
	1"	1'-4"	1'-4"	1'-9"	1'-9"
#5	2"	1'-9"	1'-3"	2'-3"	1'-8"
	1.5"	1'-9"	1'-5"	2'-3"	1'-11"
	1"	2'-0"	2'-0"	2'-7"	2'-7"
#6	2.5"	2'-6"	1'-6"	3'-3"	2'-0"
	2"	2'-6"	1'-7"	3'-3"	2'-1"
	1.5"	2'-6"	2'-0"	3'-3"	2'-7"
	1"	2'-9"	2'-9"	3'-6"	3'-6"
#7	2.5"	2'-11"	1'-9"	3'-9"	2'-3"
	2"	2'-11"	2'-1"	3'-9"	2'-9"
	1.5"	2'-11"	2'-8"	3'-9"	3'-5"
	1"	3'-6"	3'-6"	4'-7"	4'-7"
#8	2.5"	3'-10"	2'-3"	4'-11"	2'-11"
	2"	3'-10"	2'-8"	4'-11"	3'-6"
	1.5"	3'-10"	3'-4"	4'-11"	4'-4"
	1"	4'-5"	4'-5"	5'-9"	5'-9"
#9	2.5"	4'-10"	2'-10"	6'-3"	3'-8"
	2"	4'-10"	3'-4"	6'-3"	4'-3"
	1.5"	4'-10"	4'-1"	6'-3"	5'-4"
	1"	5'-5"	5'-5"	7'-0"	7'-0"
#10	2.5"	5'-4"	3'-7"	6'-11"	4'-8"
	2"	5'-4"	4'-1"	6'-11"	5'-3"
	1.5"	5'-4"	5'-0"	6'-11"	6'-6"
	1"	6'-6"	6'-6"	8'-6"	8'-6"
#11	2.5"	6'-7"	4'-5"	8'-6"	5'-8"
	2"	6'-7"	4'-10"	8'-6"	6'-4"
	1.5"	6'-7"	6'-0"	8'-6"	7'-9"
	1"	7'-8"	7'-8"	10'-0"	10'-0"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-6"	1'-4"	1'-11"	1'-8"
	1"	1'-9"	1'-9"	2'-3"	2'-3"
#5	2"	2'-3"	1'-8"	2'-11"	2'-1"
	1.5"	2'-3"	1'-11"	2'-11"	2'-5"
	1"	2'-7"	2'-7"	3'-4"	3'-4"
#6	2.5"	3'-3"	2'-0"	4'-2"	2'-6"
	2"	3'-3"	2'-1"	4'-2"	2'-8"
	1.5"	3'-3"	2'-7"	4'-2"	3'-4"
	1"	3'-6"	3'-6"	4'-7"	4'-7"
#7	2.5"	3'-9"	2'-3"	4'-11"	2'-11"
	2"	3'-9"	2'-9"	4'-11"	3'-6"
	1.5"	3'-9"	3'-5"	4'-11"	4'-5"
	1"	4'-7"	4'-7"	5'-11"	5'-11"
#8	2.5"	4'-11"	2'-11"	6'-5"	3'-9"
	2"	4'-11"	3'-6"	6'-5"	4'-6"
	1.5"	4'-11"	4'-4"	6'-5"	5'-7"
	1"	5'-9"	5'-9"	7'-5"	7'-5"
#9	2.5"	6'-3"	3'-8"	8'-1"	4'-9"
	2"	6'-3"	4'-3"	8'-1"	5'-7"
	1.5"	6'-3"	5'-4"	8'-1"	6'-11"
	1"	7'-0"	7'-0"	9'-1"	9'-1"
#10	2.5"	6'-11"	4'-8"	9'-0"	6'-0"
	2"	6'-11"	5'-3"	9'-0"	6'-10"
	1.5"	6'-11"	6'-6"	9'-0"	8'-5"
	1"	8'-6"	8'-6"	11'-0"	11'-0"
#11	2.5"	8'-6"	5'-8"	11'-1"	7'-5"
	2"	8'-6"	6'-4"	11'-1"	8'-2"
	1.5"	8'-6"	7'-9"	11'-1"	10'-1"
	1"	10'-0"	10'-0"	13'-0"	13'-0"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 4,500 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 4500 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-1"	1'-0"	1'-5"	1'-4"
	1"	1'-3"	1'-3"	1'-8"	1'-8"
#5	2"	1'-8"	1'-2"	2'-2"	1'-7"
	1.5"	1'-8"	1'-5"	2'-2"	1'-9"
	1"	1'-11"	1'-11"	2'-5"	2'-5"
#6	2.5"	2'-4"	1'-5"	3'-1"	1'-10"
	2"	2'-4"	1'-6"	3'-1"	1'-11"
	1.5"	2'-4"	1'-11"	3'-1"	2'-5"
	1"	2'-7"	2'-7"	3'-4"	3'-4"
#7	2.5"	2'-9"	1'-8"	3'-7"	2'-2"
	2"	2'-9"	2'-0"	3'-7"	2'-7"
	1.5"	2'-9"	2'-6"	3'-7"	3'-3"
	1"	3'-4"	3'-4"	4'-4"	4'-4"
#8	2.5"	3'-7"	2'-1"	4'-8"	2'-9"
	2"	3'-7"	2'-6"	4'-8"	3'-3"
	1.5"	3'-7"	3'-2"	4'-8"	4'-1"
	1"	4'-2"	4'-2"	5'-5"	5'-5"
#9	2.5"	4'-7"	2'-8"	5'-11"	3'-5"
	2"	4'-7"	3'-1"	5'-11"	4'-0"
	1.5"	4'-7"	3'-10"	5'-11"	5'-0"
	1"	5'-1"	5'-1"	6'-7"	6'-7"
#10	2.5"	5'-0"	3'-4"	6'-6"	4'-4"
	2"	5'-0"	3'-10"	6'-6"	5'-0"
	1.5"	5'-0"	4'-9"	6'-6"	6'-2"
	1"	6'-2"	6'-2"	8'-0"	8'-0"
#11	2.5"	6'-2"	4'-2"	8'-1"	5'-5"
	2"	6'-2"	4'-7"	8'-1"	6'-0"
	1.5"	6'-2"	5'-8"	8'-1"	7'-4"
	1"	7'-3"	7'-3"	9'-5"	9'-5"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-5"	1'-3"	1'-9"	1'-7"
	1"	1'-8"	1'-8"	2'-2"	2'-2"
#5	2"	2'-2"	1'-7"	2'-9"	2'-0"
	1.5"	2'-2"	1'-9"	2'-9"	2'-4"
	1"	2'-5"	2'-5"	3'-2"	3'-2"
#6	2.5"	3'-1"	1'-10"	4'-0"	2'-5"
	2"	3'-1"	1'-11"	4'-0"	2'-6"
	1.5"	3'-1"	2'-5"	4'-0"	3'-2"
	1"	3'-4"	3'-4"	4'-4"	4'-4"
#7	2.5"	3'-7"	2'-2"	4'-7"	2'-9"
	2"	3'-7"	2'-7"	4'-7"	3'-4"
	1.5"	3'-7"	3'-3"	4'-7"	4'-2"
	1"	4'-4"	4'-4"	5'-7"	5'-7"
#8	2.5"	4'-8"	2'-9"	6'-0"	3'-6"
	2"	4'-8"	3'-3"	6'-0"	4'-3"
	1.5"	4'-8"	4'-1"	6'-0"	5'-3"
	1"	5'-5"	5'-5"	7'-0"	7'-0"
#9	2.5"	5'-11"	3'-5"	7'-8"	4'-6"
	2"	5'-11"	4'-0"	7'-8"	5'-3"
	1.5"	5'-11"	5'-0"	7'-8"	6'-6"
	1"	6'-7"	6'-7"	8'-7"	8'-7"
#10	2.5"	6'-6"	4'-4"	8'-6"	5'-8"
	2"	6'-6"	5'-0"	8'-6"	6'-5"
	1.5"	6'-6"	6'-2"	8'-6"	7'-11"
	1"	8'-0"	8'-0"	10'-4"	10'-4"
#11	2.5"	8'-1"	5'-5"	10'-5"	7'-0"
	2"	8'-1"	6'-0"	10'-5"	7'-9"
	1.5"	8'-1"	7'-4"	10'-5"	9'-6"
	1"	9'-5"	9'-5"	12'-3"	12'-3"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 5,000 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 5000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\ l_d$)		Class B Lap ($1.3\ l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-0"	1'-0"	1'-4"	1'-4"
	1"	1'-3"	1'-3"	1'-7"	1'-7"
#5	2"	1'-7"	1'-2"	2'-0"	1'-6"
	1.5"	1'-7"	1'-4"	2'-0"	1'-8"
	1"	1'-9"	1'-9"	2'-4"	2'-4"
#6	2.5"	2'-3"	1'-4"	2'-11"	1'-9"
	2"	2'-3"	1'-5"	2'-11"	1'-10"
	1.5"	2'-3"	1'-10"	2'-11"	2'-4"
	1"	2'-5"	2'-5"	3'-2"	3'-2"
#7	2.5"	2'-7"	1'-7"	3'-5"	2'-1"
	2"	2'-7"	1'-11"	3'-5"	2'-5"
	1.5"	2'-7"	2'-4"	3'-5"	3'-1"
	1"	3'-2"	3'-2"	4'-1"	4'-1"
#8	2.5"	3'-5"	2'-0"	4'-5"	2'-7"
	2"	3'-5"	2'-5"	4'-5"	3'-1"
	1.5"	3'-5"	3'-0"	4'-5"	3'-10"
	1"	4'-0"	4'-0"	5'-2"	5'-2"
#9	2.5"	4'-4"	2'-6"	5'-7"	3'-3"
	2"	4'-4"	2'-11"	5'-7"	3'-10"
	1.5"	4'-4"	3'-8"	5'-7"	4'-9"
	1"	4'-10"	4'-10"	6'-3"	6'-3"
#10	2.5"	4'-9"	3'-2"	6'-2"	4'-2"
	2"	4'-9"	3'-8"	6'-2"	4'-9"
	1.5"	4'-9"	4'-6"	6'-2"	5'-10"
	1"	5'-10"	5'-10"	7'-7"	7'-7"
#11	2.5"	5'-11"	3'-11"	7'-8"	5'-1"
	2"	5'-11"	4'-4"	7'-8"	5'-8"
	1.5"	5'-11"	5'-4"	7'-8"	6'-11"
	1"	6'-11"	6'-11"	8'-11"	8'-11"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\ l_d$)		Class B Lap ($1.3\ l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-4"	1'-2"	1'-8"	1'-6"
	1"	1'-7"	1'-7"	2'-0"	2'-0"
#5	2"	2'-0"	1'-6"	2'-8"	1'-11"
	1.5"	2'-0"	1'-8"	2'-8"	2'-2"
	1"	2'-4"	2'-4"	3'-0"	3'-0"
#6	2.5"	2'-11"	1'-9"	3'-9"	2'-3"
	2"	2'-11"	1'-10"	3'-9"	2'-5"
	1.5"	2'-11"	2'-4"	3'-9"	3'-0"
	1"	3'-2"	3'-2"	4'-1"	4'-1"
#7	2.5"	3'-5"	2'-1"	4'-5"	2'-8"
	2"	3'-5"	2'-5"	4'-5"	3'-2"
	1.5"	3'-5"	3'-1"	4'-5"	4'-0"
	1"	4'-1"	4'-1"	5'-4"	5'-4"
#8	2.5"	4'-5"	2'-7"	5'-9"	3'-4"
	2"	4'-5"	3'-1"	5'-9"	4'-0"
	1.5"	4'-5"	3'-10"	5'-9"	5'-0"
	1"	5'-2"	5'-2"	6'-8"	6'-8"
#9	2.5"	5'-7"	3'-3"	7'-3"	4'-3"
	2"	5'-7"	3'-10"	7'-3"	5'-0"
	1.5"	5'-7"	4'-9"	7'-3"	6'-2"
	1"	6'-3"	6'-3"	8'-1"	8'-1"
#10	2.5"	6'-2"	4'-2"	8'-1"	5'-5"
	2"	6'-2"	4'-9"	8'-1"	6'-1"
	1.5"	6'-2"	5'-10"	8'-1"	7'-7"
	1"	7'-7"	7'-7"	9'-10"	9'-10"
#11	2.5"	7'-8"	5'-1"	9'-11"	6'-7"
	2"	7'-8"	5'-8"	9'-11"	7'-4"
	1.5"	7'-8"	6'-11"	9'-11"	9'-0"
	1"	8'-11"	8'-11"	11'-7"	11'-7"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 6,000 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 6000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\ l_d$)		Class B Lap ($1.3\ l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-0"	1'-0"	1'-4"	1'-4"
	1"	1'-1"	1'-1"	1'-5"	1'-5"
#5	2"	1'-5"	1'-1"	1'-10"	1'-4"
	1.5"	1'-5"	1'-2"	1'-10"	1'-7"
	1"	1'-8"	1'-8"	2'-1"	2'-1"
#6	2.5"	2'-1"	1'-3"	2'-8"	1'-7"
	2"	2'-1"	1'-4"	2'-8"	1'-8"
	1.5"	2'-1"	1'-8"	2'-8"	2'-2"
	1"	2'-3"	2'-3"	2'-11"	2'-11"
#7	2.5"	2'-5"	1'-5"	3'-1"	1'-10"
	2"	2'-5"	1'-9"	3'-1"	2'-3"
	1.5"	2'-5"	2'-2"	3'-1"	2'-10"
	1"	2'-11"	2'-11"	3'-9"	3'-9"
#8	2.5"	3'-1"	1'-10"	4'-0"	2'-4"
	2"	3'-1"	2'-2"	4'-0"	2'-10"
	1.5"	3'-1"	2'-9"	4'-0"	3'-6"
	1"	3'-7"	3'-7"	4'-8"	4'-8"
#9	2.5"	3'-11"	2'-4"	5'-1"	3'-0"
	2"	3'-11"	2'-8"	5'-1"	3'-6"
	1.5"	3'-11"	3'-4"	5'-1"	4'-4"
	1"	4'-5"	4'-5"	5'-9"	5'-9"
#10	2.5"	4'-4"	2'-11"	5'-8"	3'-9"
	2"	4'-4"	3'-4"	5'-8"	4'-4"
	1.5"	4'-4"	4'-1"	5'-8"	5'-4"
	1"	5'-4"	5'-4"	6'-11"	6'-11"
#11	2.5"	5'-4"	3'-7"	7'-0"	4'-8"
	2"	5'-4"	4'-0"	7'-0"	5'-2"
	1.5"	5'-4"	4'-11"	7'-0"	6'-4"
	1"	6'-4"	6'-4"	8'-2"	8'-2"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\ l_d$)		Class B Lap ($1.3\ l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-2"	1'-1"	1'-7"	1'-5"
	1"	1'-5"	1'-5"	1'-10"	1'-10"
#5	2"	1'-10"	1'-4"	2'-5"	1'-9"
	1.5"	1'-10"	1'-7"	2'-5"	2'-0"
	1"	2'-1"	2'-1"	2'-9"	2'-9"
#6	2.5"	2'-8"	1'-7"	3'-5"	2'-1"
	2"	2'-8"	1'-8"	3'-5"	2'-2"
	1.5"	2'-8"	2'-2"	3'-5"	2'-9"
	1"	2'-11"	2'-11"	3'-9"	3'-9"
#7	2.5"	3'-1"	1'-10"	4'-0"	2'-5"
	2"	3'-1"	2'-3"	4'-0"	2'-11"
	1.5"	3'-1"	2'-10"	4'-0"	3'-7"
	1"	3'-9"	3'-9"	4'-10"	4'-10"
#8	2.5"	4'-0"	2'-4"	5'-3"	3'-1"
	2"	4'-0"	2'-10"	5'-3"	3'-8"
	1.5"	4'-0"	3'-6"	5'-3"	4'-7"
	1"	4'-8"	4'-8"	6'-1"	6'-1"
#9	2.5"	5'-1"	3'-0"	6'-8"	3'-11"
	2"	5'-1"	3'-6"	6'-8"	4'-6"
	1.5"	5'-1"	4'-4"	6'-8"	5'-8"
	1"	5'-9"	5'-9"	7'-5"	7'-5"
#10	2.5"	5'-8"	3'-9"	7'-4"	4'-11"
	2"	5'-8"	4'-4"	7'-4"	5'-7"
	1.5"	5'-8"	5'-4"	7'-4"	6'-11"
	1"	6'-11"	6'-11"	9'-0"	9'-0"
#11	2.5"	7'-0"	4'-8"	9'-1"	6'-1"
	2"	7'-0"	5'-2"	9'-1"	6'-8"
	1.5"	7'-0"	6'-4"	9'-1"	8'-3"
	1"	8'-2"	8'-2"	10'-7"	10'-7"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 7,000 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 7000 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-0"	1'-0"	1'-4"	1'-4"
	1"	1'-0"	1'-0"	1'-4"	1'-4"
#5	2"	1'-4"	1'-0"	1'-9"	1'-4"
	1.5"	1'-4"	1'-1"	1'-9"	1'-5"
	1"	1'-6"	1'-6"	2'-0"	2'-0"
#6	2.5"	1'-11"	1'-2"	2'-6"	1'-6"
	2"	1'-11"	1'-3"	2'-6"	1'-7"
	1.5"	1'-11"	1'-6"	2'-6"	2'-0"
	1"	2'-1"	2'-1"	2'-8"	2'-8"
#7	2.5"	2'-3"	1'-4"	2'-10"	1'-9"
	2"	2'-3"	1'-7"	2'-10"	2'-1"
	1.5"	2'-3"	2'-0"	2'-10"	2'-7"
	1"	2'-8"	2'-8"	3'-6"	3'-6"
#8	2.5"	2'-11"	1'-8"	3'-9"	2'-2"
	2"	2'-11"	2'-0"	3'-9"	2'-7"
	1.5"	2'-11"	2'-6"	3'-9"	3'-3"
	1"	3'-4"	3'-4"	4'-4"	4'-4"
#9	2.5"	3'-8"	2'-2"	4'-9"	2'-9"
	2"	3'-8"	2'-6"	4'-9"	3'-3"
	1.5"	3'-8"	3'-1"	4'-9"	4'-0"
	1"	4'-1"	4'-1"	5'-4"	5'-4"
#10	2.5"	4'-1"	2'-9"	5'-3"	3'-6"
	2"	4'-1"	3'-1"	5'-3"	4'-0"
	1.5"	4'-1"	3'-10"	5'-3"	4'-11"
	1"	4'-11"	4'-11"	6'-5"	6'-5"
#11	2.5"	5'-0"	3'-4"	6'-6"	4'-4"
	2"	5'-0"	3'-8"	6'-6"	4'-9"
	1.5"	5'-0"	4'-6"	6'-6"	5'-10"
	1"	5'-10"	5'-10"	7'-7"	7'-7"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap ($1.0\,l_d$)		Class B Lap ($1.3\,l_d$)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-1"	1'-0"	1'-5"	1'-4"
	1"	1'-4"	1'-4"	1'-9"	1'-9"
#5	2"	1'-9"	1'-3"	2'-3"	1'-7"
	1.5"	1'-9"	1'-5"	2'-3"	1'-10"
	1"	2'-0"	2'-0"	2'-6"	2'-6"
#6	2.5"	2'-6"	1'-6"	3'-2"	1'-11"
	2"	2'-6"	1'-7"	3'-2"	2'-0"
	1.5"	2'-6"	2'-0"	3'-2"	2'-7"
	1"	2'-8"	2'-8"	3'-6"	3'-6"
#7	2.5"	2'-10"	1'-9"	3'-9"	2'-3"
	2"	2'-10"	2'-1"	3'-9"	2'-8"
	1.5"	2'-10"	2'-7"	3'-9"	3'-4"
	1"	3'-6"	3'-6"	4'-6"	4'-6"
#8	2.5"	3'-9"	2'-2"	4'-10"	2'-10"
	2"	3'-9"	2'-7"	4'-10"	3'-5"
	1.5"	3'-9"	3'-3"	4'-10"	4'-3"
	1"	4'-4"	4'-4"	5'-8"	5'-8"
#9	2.5"	4'-9"	2'-9"	6'-2"	3'-7"
	2"	4'-9"	3'-3"	6'-2"	4'-2"
	1.5"	4'-9"	4'-0"	6'-2"	5'-3"
	1"	5'-4"	5'-4"	6'-10"	6'-10"
#10	2.5"	5'-3"	3'-6"	6'-10"	4'-7"
	2"	5'-3"	4'-0"	6'-10"	5'-2"
	1.5"	5'-3"	4'-11"	6'-10"	6'-5"
	1"	6'-5"	6'-5"	8'-4"	8'-4"
#11	2.5"	6'-6"	4'-4"	8'-5"	5'-7"
	2"	6'-6"	4'-9"	8'-5"	6'-3"
	1.5"	6'-6"	5'-10"	8'-5"	7'-7"
	1"	7'-7"	7'-7"	9'-10"	9'-10"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.4.2 Uncoated, f'c = 8,500 psi, fy = 60,000 psi

Lap Splices in Tension for Unprotected Reinforcement in Normal Weight Concrete

(f'c = 8500 psi; fy = 60000 psi)

Modified Tension Development Lengths					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-0"	1'-0"	1'-4"	1'-4"
	1"	1'-0"	1'-0"	1'-4"	1'-4"
#5	2"	1'-3"	1'-0"	1'-7"	1'-4"
	1.5"	1'-3"	1'-0"	1'-7"	1'-4"
	1"	1'-5"	1'-5"	1'-9"	1'-9"
#6	2.5"	1'-9"	1'-1"	2'-3"	1'-4"
	2"	1'-9"	1'-1"	2'-3"	1'-5"
	1.5"	1'-9"	1'-5"	2'-3"	1'-10"
	1"	1'-11"	1'-11"	2'-5"	2'-5"
#7	2.5"	2'-0"	1'-3"	2'-7"	1'-7"
	2"	2'-0"	1'-5"	2'-7"	1'-11"
	1.5"	2'-0"	1'-10"	2'-7"	2'-4"
	1"	2'-5"	2'-5"	3'-2"	3'-2"
#8	2.5"	2'-7"	1'-7"	3'-5"	2'-0"
	2"	2'-7"	1'-10"	3'-5"	2'-5"
	1.5"	2'-7"	2'-4"	3'-5"	3'-0"
	1"	3'-1"	3'-1"	3'-11"	3'-11"
#9	2.5"	3'-4"	1'-11"	4'-4"	2'-6"
	2"	3'-4"	2'-3"	4'-4"	2'-11"
	1.5"	3'-4"	2'-10"	4'-4"	3'-8"
	1"	3'-9"	3'-9"	4'-10"	4'-10"
#10	2.5"	3'-8"	2'-6"	4'-9"	3'-2"
	2"	3'-8"	2'-10"	4'-9"	3'-8"
	1.5"	3'-8"	3'-5"	4'-9"	4'-6"
	1"	4'-6"	4'-6"	5'-10"	5'-10"
#11	2.5"	4'-6"	3'-0"	5'-10"	3'-11"
	2"	4'-6"	3'-4"	5'-10"	4'-4"
	1.5"	4'-6"	4'-1"	5'-10"	5'-4"
	1"	5'-4"	5'-4"	6'-10"	6'-10"

Horizontal Lap w/>12" Concrete Cast Below					
Bar Size	Min. Cover	Class A Lap (1.0 l_d)		Class B Lap (1.3 l_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
#4	1.5"	1'-0"	1'-0"	1'-4"	1'-4"
	1"	1'-3"	1'-3"	1'-7"	1'-7"
#5	2"	1'-7"	1'-2"	2'-0"	1'-6"
	1.5"	1'-7"	1'-4"	2'-0"	1'-8"
	1"	1'-9"	1'-9"	2'-4"	2'-4"
#6	2.5"	2'-3"	1'-4"	2'-11"	1'-9"
	2"	2'-3"	1'-5"	2'-11"	1'-10"
	1.5"	2'-3"	1'-10"	2'-11"	2'-4"
	1"	2'-5"	2'-5"	3'-2"	3'-2"
#7	2.5"	2'-7"	1'-7"	3'-4"	2'-0"
	2"	2'-7"	1'-11"	3'-4"	2'-5"
	1.5"	2'-7"	2'-4"	3'-4"	3'-1"
	1"	3'-2"	3'-2"	4'-1"	4'-1"
#8	2.5"	3'-5"	2'-0"	4'-5"	2'-7"
	2"	3'-5"	2'-5"	4'-5"	3'-1"
	1.5"	3'-5"	3'-0"	4'-5"	3'-10"
	1"	3'-11"	3'-11"	5'-1"	5'-1"
#9	2.5"	4'-4"	2'-6"	5'-7"	3'-3"
	2"	4'-4"	2'-11"	5'-7"	3'-10"
	1.5"	4'-4"	3'-8"	5'-7"	4'-9"
	1"	4'-10"	4'-10"	6'-3"	6'-3"
#10	2.5"	4'-9"	3'-2"	6'-2"	4'-2"
	2"	4'-9"	3'-8"	6'-2"	4'-8"
	1.5"	4'-9"	4'-6"	6'-2"	5'-10"
	1"	5'-10"	5'-10"	7'-7"	7'-7"
#11	2.5"	5'-10"	3'-11"	7'-7"	5'-1"
	2"	5'-10"	4'-4"	7'-7"	5'-8"
	1.5"	5'-10"	5'-4"	7'-7"	6'-11"
	1"	6'-10"	6'-10"	8'-11"	8'-11"

Notes: Tables based on AASHTO LRFD Bridge Design Specifications, 10th Ed.
Confinement reinforcement was conservatively assumed to be zero.
Larger covers for the smaller bars were intentionally omitted because additional cover does not reduce the lap length.

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A3.5 Weight of Rectangular Steel Sections

The following coefficient factors shall be applied in accordance with ASTM A6.

A3.5.1 Thickness 0.0625 in to 0.5 in & Widths 6.25 in to 23.50 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0300	1.0300	1.0300	1.0300	1.0250	1.0225	1.0200	1.0200
				Thickness (in.)				
	0.0625	0.125	0.1875	0.25	0.3125	0.375	0.4375	0.5
Width (in.)								
6.25	1.37	2.74	4.11	5.48	6.81	8.15	9.49	10.85
6.50	1.42	2.85	4.27	5.70	7.08	8.48	9.87	11.28
6.75	1.48	2.96	4.44	5.91	7.36	8.81	10.25	11.71
7.00	1.53	3.07	4.60	6.13	7.63	9.13	10.63	12.15
7.25	1.59	3.18	4.76	6.35	7.90	9.46	11.01	12.58
7.50	1.64	3.29	4.93	6.57	8.17	9.79	11.39	13.02
7.75	1.70	3.40	5.09	6.79	8.45	10.11	11.77	13.45
8.00	1.75	3.50	5.26	7.01	8.72	10.44	12.15	13.88
8.25	1.81	3.61	5.42	7.23	8.99	10.76	12.53	14.32
8.50	1.86	3.72	5.59	7.45	9.26	11.09	12.91	14.75
8.75	1.92	3.83	5.75	7.67	9.54	11.42	13.29	15.18
9.00	1.97	3.94	5.91	7.89	9.81	11.74	13.67	15.62
9.25	2.03	4.05	6.08	8.10	10.08	12.07	14.05	16.05
9.50	2.08	4.16	6.24	8.32	10.35	12.40	14.43	16.49
9.75	2.14	4.27	6.41	8.54	10.63	12.72	14.81	16.92
10.00	2.19	4.38	6.57	8.76	10.90	13.05	15.18	17.35
10.25	2.25	4.49	6.74	8.98	11.17	13.37	15.56	17.79
10.50	2.30	4.60	6.90	9.20	11.44	13.70	15.94	18.22
10.75	2.35	4.71	7.06	9.42	11.72	14.03	16.32	18.66
11.00	2.41	4.82	7.23	9.64	11.99	14.35	16.70	19.09
11.25	2.46	4.93	7.39	9.86	12.26	14.68	17.08	19.52
11.50	2.52	5.04	7.56	10.08	12.53	15.00	17.46	19.96
11.75	2.57	5.15	7.72	10.30	12.81	15.33	17.84	20.39
12.00	2.63	5.26	7.89	10.51	13.08	15.66	18.22	20.83
12.50	2.74	5.48	8.21	10.95	13.62	16.31	18.98	21.69
13.00	2.85	5.70	8.54	11.39	14.17	16.96	19.74	22.56
13.50	2.96	5.91	8.87	11.83	14.71	17.61	20.50	23.43
14.00	3.07	6.13	9.20	12.27	15.26	18.27	21.26	24.30
14.50	3.18	6.35	9.53	12.71	15.80	18.92	22.02	25.16
15.00	3.29	6.57	9.86	13.14	16.35	19.57	22.78	26.03
15.50	3.40	6.79	10.19	13.58	16.89	20.22	23.54	26.90
16.00	3.50	7.01	10.51	14.02	17.44	20.88	24.30	27.77
16.50	3.61	7.23	10.84	14.46	17.98	21.53	25.06	28.63
17.00	3.72	7.45	11.17	14.90	18.53	22.18	25.81	29.50
17.50	3.83	7.67	11.50	15.33	19.07	22.83	26.57	30.37
18.00	3.94	7.89	11.83	15.77	19.62	23.49	27.33	31.24
18.50	4.05	8.10	12.16	16.21	20.16	24.14	28.09	32.11
19.00	4.16	8.32	12.49	16.65	20.71	24.79	28.85	32.97
19.50	4.27	8.54	12.81	17.09	21.25	25.44	29.61	33.84
20.00	4.38	8.76	13.14	17.52	21.80	26.10	30.37	34.71
20.50	4.49	8.98	13.47	17.96	22.34	26.75	31.13	35.58
21.00	4.60	9.20	13.80	18.40	22.89	27.40	31.89	36.44
21.50	4.71	9.42	14.13	18.84	23.43	28.05	32.65	37.31
22.00	4.82	9.64	14.46	19.28	23.98	28.70	33.41	38.18
22.50	4.93	9.86	14.79	19.71	24.52	29.36	34.17	39.05
23.00	5.04	10.08	15.11	20.15	25.07	30.01	34.93	39.91
23.50	5.15	10.30	15.44	20.59	25.61	30.66	35.68	40.78

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A3.5.2 Thickness 0.0625 in to 0.5 in & Widths 24.00 in to 59.00 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0300	1.0300	1.0300	1.0300	1.0250	1.0225	1.0200	1.0200
				Thickness (in.)				
	0.0625	0.125	0.1875	0.25	0.3125	0.375	0.4375	0.5
Width (in.)								
24.00	5.26	10.51	15.77	21.03	26.16	31.31	36.44	41.65
25.00	5.48	10.95	16.43	21.91	27.25	32.62	37.96	43.39
26.00	5.70	11.39	17.09	22.78	28.34	33.92	39.48	45.12
27.00	5.91	11.83	17.74	23.66	29.43	35.23	41.00	46.86
28.00	6.13	12.27	18.40	24.53	30.52	36.53	42.52	48.59
29.00	6.35	12.71	19.06	25.41	31.61	37.84	44.04	50.33
30.00	6.57	13.14	19.71	26.29	32.70	39.14	45.55	52.06
31.00	6.79	13.58	20.37	27.16	33.79	40.45	47.07	53.80
32.00	7.01	14.02	21.03	28.04	34.88	41.75	48.59	55.53
33.00	7.23	14.46	21.69	28.92	35.97	43.06	50.11	57.27
34.00	7.45	14.90	22.34	29.79	37.06	44.36	51.63	59.00
35.00	7.67	15.33	23.00	30.67	38.15	45.67	53.15	60.74
36.00	7.89	15.77	23.66	31.54	39.24	46.97	54.67	62.48
37.00	8.10	16.21	24.31	32.42	40.33	48.28	56.18	64.21
38.00	8.32	16.65	24.97	33.30	41.42	49.58	57.70	65.95
39.00	8.54	17.09	25.63	34.17	42.51	50.89	59.22	67.68
40.00	8.76	17.52	26.29	35.05	43.60	52.19	60.74	69.42
41.00	8.98	17.96	26.94	35.92	44.69	53.49	62.26	71.15
42.00	9.20	18.40	27.60	36.80	45.78	54.80	63.78	72.89
43.00	9.42	18.84	28.26	37.68	46.87	56.10	65.30	74.62
44.00	9.64	19.28	28.92	38.55	47.96	57.41	66.81	76.36
45.00	9.86	19.71	29.57	39.43	49.05	58.71	68.33	78.09
46.00	10.08	20.15	30.23	40.31	50.14	60.02	69.85	79.83
47.00	10.30	20.59	30.89	41.18	51.23	61.32	71.37	81.56
48.00	10.51	21.03	31.54	42.06	52.32	62.63	72.89	83.30
COEFF.	1.035	1.035	1.035	1.03	1.03	1.025	1.0225	1.02
49.00	10.79	21.57	32.36	42.93	53.67	64.09	74.59	85.04
50.00	11.01	22.01	33.02	43.81	54.76	65.40	76.11	86.77
51.00	11.23	22.45	33.68	44.69	55.86	66.71	77.63	88.51
52.00	11.45	22.89	34.34	45.56	56.95	68.01	79.15	90.24
53.00	11.67	23.33	35.00	46.44	58.05	69.32	80.68	91.98
54.00	11.89	23.77	35.66	47.32	59.14	70.63	82.20	93.71
55.00	12.11	24.21	36.32	48.19	60.24	71.94	83.72	95.45
56.00	12.33	24.65	36.98	49.07	61.34	73.24	85.24	97.18
57.00	12.55	25.09	37.64	49.94	62.43	74.55	86.77	98.92
58.00	12.77	25.53	38.30	50.82	63.53	75.86	88.29	100.65
59.00	12.99	25.97	38.96	51.70	64.62	77.17	89.81	102.39

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A3.5.3 Thickness 0.5625 in to 1.0 in & Widths 6.25 in to 23.50 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0200	1.0200	1.0200	1.0175	1.0175	1.0175	1.0175	1.0175
				Thickness (in.)				
	0.5625	0.625	0.6875	0.75	0.8125	0.875	0.9375	1.0
Width (in.)								
6.25	12.20	13.56	14.91	16.23	17.58	18.93	20.29	21.64
6.50	12.69	14.10	15.51	16.88	18.29	19.69	21.10	22.51
6.75	13.18	14.64	16.11	17.53	18.99	20.45	21.91	23.37
7.00	13.67	15.18	16.70	18.18	19.69	21.21	22.72	24.24
7.25	14.15	15.73	17.30	18.83	20.40	21.96	23.53	25.10
7.50	14.64	16.27	17.90	19.48	21.10	22.72	24.34	25.97
7.75	15.13	16.81	18.49	20.12	21.80	23.48	25.16	26.83
8.00	15.62	17.35	19.09	20.77	22.51	24.24	25.97	27.70
8.25	16.11	17.90	19.69	21.42	23.21	24.99	26.78	28.56
8.50	16.59	18.44	20.28	22.07	23.91	25.75	27.59	29.43
8.75	17.08	18.98	20.88	22.72	24.61	26.51	28.40	30.30
9.00	17.57	19.52	21.48	23.37	25.32	27.27	29.21	31.16
9.25	18.06	20.07	22.07	24.02	26.02	28.02	30.02	32.03
9.50	18.55	20.61	22.67	24.67	26.72	28.78	30.84	32.89
9.75	19.04	21.15	23.27	25.32	27.43	29.54	31.65	33.76
10.00	19.52	21.69	23.86	25.97	28.13	30.30	32.46	34.62
10.25	20.01	22.24	24.46	26.62	28.83	31.05	33.27	35.49
10.50	20.50	22.78	25.06	27.27	29.54	31.81	34.08	36.35
10.75	20.99	23.32	25.65	27.92	30.24	32.57	34.89	37.22
11.00	21.48	23.86	26.25	28.56	30.94	33.32	35.71	38.09
11.25	21.96	24.40	26.84	29.21	31.65	34.08	36.52	38.95
11.50	22.45	24.95	27.44	29.86	32.35	34.84	37.33	39.82
11.75	22.94	25.49	28.04	30.51	33.05	35.60	38.14	40.68
12.00	23.43	26.03	28.63	31.16	33.76	36.35	38.95	41.55
12.50	24.40	27.12	29.83	32.46	35.16	37.87	40.57	43.28
13.00	25.38	28.20	31.02	33.76	36.57	39.38	42.20	45.01
13.50	26.36	29.29	32.21	35.06	37.98	40.90	43.82	46.74
14.00	27.33	30.37	33.41	36.35	39.38	42.41	45.44	48.47
14.50	28.31	31.45	34.60	37.65	40.79	43.93	47.07	50.20
15.00	29.29	32.54	35.79	38.95	42.20	45.44	48.69	51.93
15.50	30.26	33.62	36.99	40.25	43.60	46.96	50.31	53.67
16.00	31.24	34.71	38.18	41.55	45.01	48.47	51.93	55.40
16.50	32.21	35.79	39.37	42.85	46.42	49.99	53.56	57.13
17.00	33.19	36.88	40.57	44.14	47.82	51.50	55.18	58.86
17.50	34.17	37.96	41.76	45.44	49.23	53.02	56.80	60.59
18.00	35.14	39.05	42.95	46.74	50.64	54.53	58.43	62.32
18.50	36.12	40.13	44.14	48.04	52.04	56.05	60.05	64.05
19.00	37.09	41.22	45.34	49.34	53.45	57.56	61.67	65.78
19.50	38.07	42.30	46.53	50.64	54.86	59.08	63.30	67.52
20.00	39.05	43.39	47.72	51.93	56.26	60.59	64.92	69.25
20.50	40.02	44.47	48.92	53.23	57.67	62.11	66.54	70.98
21.00	41.00	45.55	50.11	54.53	59.08	63.62	68.16	72.71
21.50	41.98	46.64	51.30	55.83	60.48	65.14	69.79	74.44
22.00	42.95	47.72	52.50	57.13	61.89	66.65	71.41	76.17
22.50	43.93	48.81	53.69	58.43	63.30	68.16	73.03	77.90
23.00	44.90	49.89	54.88	59.73	64.70	69.68	74.66	79.63
23.50	45.88	50.98	56.08	61.02	66.11	71.19	76.28	81.36

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A3.5.4 Thickness 0.5625 in to 1.0 in & Widths 24.00 in to 59.00 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0200	1.0200	1.0200	1.0175	1.0175	1.0175	1.0175	1.0175
				Thickness (in.)				
	0.5625	0.625	0.6875	0.75	0.8125	0.875	0.9375	1.0
Width (in.)								
24.00	46.86	52.06	57.27	62.32	67.52	72.71	77.90	83.10
25.00	48.81	54.23	59.65	64.92	70.33	75.74	81.15	86.56
26.00	50.76	56.40	62.04	67.52	73.14	78.77	84.39	90.02
27.00	52.71	58.57	64.43	70.11	75.95	81.80	87.64	93.48
28.00	54.67	60.74	66.81	72.71	78.77	84.83	90.89	96.95
29.00	56.62	62.91	69.20	75.31	81.58	87.86	94.13	100.41
30.00	58.57	65.08	71.59	77.90	84.39	90.89	97.38	103.87
31.00	60.52	67.25	73.97	80.50	87.21	93.92	100.62	107.33
32.00	62.48	69.42	76.36	83.10	90.02	96.95	103.87	110.79
33.00	64.43	71.59	78.74	85.69	92.83	99.97	107.12	114.26
34.00	66.38	73.76	81.13	88.29	95.65	103.00	110.36	117.72
35.00	68.33	75.92	83.52	90.89	98.46	106.03	113.61	121.18
36.00	70.28	78.09	85.90	93.48	101.27	109.06	116.85	124.64
37.00	72.24	80.26	88.29	96.08	104.09	112.09	120.10	128.11
38.00	74.19	82.43	90.68	98.68	106.90	115.12	123.35	131.57
39.00	76.14	84.60	93.06	101.27	109.71	118.15	126.59	135.03
40.00	78.09	86.77	95.45	103.87	112.53	121.18	129.84	138.49
41.00	80.05	88.94	97.83	106.47	115.34	124.21	133.08	141.96
42.00	82.00	91.11	100.22	109.06	118.15	127.24	136.33	145.42
43.00	83.95	93.28	102.61	111.66	120.97	130.27	139.58	148.88
44.00	85.90	95.45	104.99	114.26	123.78	133.30	142.82	152.34
45.00	87.86	97.62	107.38	116.85	126.59	136.33	146.07	155.80
46.00	89.81	99.79	109.77	119.45	129.40	139.36	149.31	159.27
47.00	91.76	101.96	112.15	122.05	132.22	142.39	152.56	162.73
48.00	93.71	104.13	114.54	124.64	135.03	145.42	155.80	166.19
COEFF.	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.0175
49.00	95.66	106.29	116.92	127.55	138.18	148.81	159.44	169.65
50.00	97.62	108.46	119.31	130.16	141.00	151.85	162.70	173.12
51.00	99.57	110.63	121.70	132.76	143.82	154.89	165.95	176.58
52.00	101.52	112.80	124.08	135.36	146.64	157.92	169.20	180.04
53.00	103.47	114.97	126.47	137.97	149.46	160.96	172.46	183.50
54.00	105.43	117.14	128.85	140.57	152.28	164.00	175.71	186.97
55.00	107.38	119.31	131.24	143.17	155.10	167.03	178.96	190.43
56.00	109.33	121.48	133.63	145.78	157.92	170.07	182.22	193.89
57.00	111.28	123.65	136.01	148.38	160.74	173.11	185.47	197.35
58.00	113.24	125.82	138.40	150.98	163.56	176.14	188.73	200.81
59.00	115.19	127.99	140.79	153.58	166.38	179.18	191.98	204.28

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A3.5.5 Thickness 1.125 in to 2.0 in & Widths 6.25 in to 23.50 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175
				Thickness (in.)				
	1.125	1.25	1.375	1.50	1.625	1.75	1.875	2.0
Width (in.)								
6.25	24.34	27.05	29.75	32.46	35.16	37.87	40.57	43.28
6.50	25.32	28.13	30.94	33.76	36.57	39.38	42.20	45.01
6.75	26.29	29.21	32.13	35.06	37.98	40.90	43.82	46.74
7.00	27.27	30.30	33.32	36.35	39.38	42.41	45.44	48.47
7.25	28.24	31.38	34.52	37.65	40.79	43.93	47.07	50.20
7.50	29.21	32.46	35.71	38.95	42.20	45.44	48.69	51.93
7.75	30.19	33.54	36.90	40.25	43.60	46.96	50.31	53.67
8.00	31.16	34.62	38.09	41.55	45.01	48.47	51.93	55.40
8.25	32.13	35.71	39.28	42.85	46.42	49.99	53.56	57.13
8.50	33.11	36.79	40.47	44.14	47.82	51.50	55.18	58.86
8.75	34.08	37.87	41.66	45.44	49.23	53.02	56.80	60.59
9.00	35.06	38.95	42.85	46.74	50.64	54.53	58.43	62.32
9.25	36.03	40.03	44.04	48.04	52.04	56.05	60.05	64.05
9.50	37.00	41.12	45.23	49.34	53.45	57.56	61.67	65.78
9.75	37.98	42.20	46.42	50.64	54.86	59.08	63.30	67.52
10.00	38.95	43.28	47.61	51.93	56.26	60.59	64.92	69.25
10.25	39.92	44.36	48.80	53.23	57.67	62.11	66.54	70.98
10.50	40.90	45.44	49.99	54.53	59.08	63.62	68.16	72.71
10.75	41.87	46.53	51.18	55.83	60.48	65.14	69.79	74.44
11.00	42.85	47.61	52.37	57.13	61.89	66.65	71.41	76.17
11.25	43.82	48.69	53.56	58.43	63.30	68.16	73.03	77.90
11.50	44.79	49.77	54.75	59.73	64.70	69.68	74.66	79.63
11.75	45.77	50.85	55.94	61.02	66.11	71.19	76.28	81.36
12.00	46.74	51.93	57.13	62.32	67.52	72.71	77.90	83.10
12.50	48.69	54.10	59.51	64.92	70.33	75.74	81.15	86.56
13.00	50.64	56.26	61.89	67.52	73.14	78.77	84.39	90.02
13.50	52.58	58.43	64.27	70.11	75.95	81.80	87.64	93.48
14.00	54.53	60.59	66.65	72.71	78.77	84.83	90.89	96.95
14.50	56.48	62.75	69.03	75.31	81.58	87.86	94.13	100.41
15.00	58.43	64.92	71.41	77.90	84.39	90.89	97.38	103.87
15.50	60.37	67.08	73.79	80.50	87.21	93.92	100.62	107.33
16.00	62.32	69.25	76.17	83.10	90.02	96.95	103.87	110.79
16.50	64.27	71.41	78.55	85.69	92.83	99.97	107.12	114.26
17.00	66.22	73.57	80.93	88.29	95.65	103.00	110.36	117.72
17.50	68.16	75.74	83.31	90.89	98.46	106.03	113.61	121.18
18.00	70.11	77.90	85.69	93.48	101.27	109.06	116.85	124.64
18.50	72.06	80.07	88.07	96.08	104.09	112.09	120.10	128.11
19.00	74.01	82.23	90.45	98.68	106.90	115.12	123.35	131.57
19.50	75.95	84.39	92.83	101.27	109.71	118.15	126.59	135.03
20.00	77.90	86.56	95.21	103.87	112.53	121.18	129.84	138.49
20.50	79.85	88.72	97.59	106.47	115.34	124.21	133.08	141.96
21.00	81.80	90.89	99.97	109.06	118.15	127.24	136.33	145.42
21.50	83.75	93.05	102.36	111.66	120.97	130.27	139.58	148.88
22.00	85.69	95.21	104.74	114.26	123.78	133.30	142.82	152.34
22.50	87.64	97.38	107.12	116.85	126.59	136.33	146.07	155.80
23.00	89.59	99.54	109.50	119.45	129.40	139.36	149.31	159.27
23.50	91.54	101.71	111.88	122.05	132.22	142.39	152.56	162.73

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A3.5.6 Thickness 1.125 in to 2.0 in & Widths 24.00 in to 26.00 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175
				Thickness (in.)				
Width (in.)	1.125	1.25	1.375	1.50	1.625	1.75	1.875	2.0
24.00	93.48	103.87	114.26	124.64	135.03	145.42	155.80	166.19
24.50	95.43	106.03	116.64	127.24	137.84	148.45	159.05	169.65
25.00	97.38	108.20	119.02	129.84	140.66	151.48	162.30	173.12
25.50	99.32	110.36	121.40	132.43	143.47	154.51	165.54	176.58
26.00	101.27	112.53	123.78	135.03	146.28	157.54	168.79	180.04

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A3.5.7 Thickness 2.125 in to 3.0 in & Widths 6.25 in to 23.50 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175
				Thickness (in.)				
	2.125	2.25	2.375	2.50	2.625	2.75	2.875	3.0
Width (in.)								
6.25	45.98	48.69	51.39	54.10	56.80	59.51	62.21	64.92
6.50	47.82	50.64	53.45	56.26	59.08	61.89	64.70	67.52
6.75	49.66	52.58	55.51	58.43	61.35	64.27	67.19	70.11
7.00	51.50	54.53	57.56	60.59	63.62	66.65	69.68	72.71
7.25	53.34	56.48	59.62	62.75	65.89	69.03	72.17	75.31
7.50	55.18	58.43	61.67	64.92	68.16	71.41	74.66	77.90
7.75	57.02	60.37	63.73	67.08	70.44	73.79	77.14	80.50
8.00	58.86	62.32	65.78	69.25	72.71	76.17	79.63	83.10
8.25	60.70	64.27	67.84	71.41	74.98	78.55	82.12	85.69
8.50	62.54	66.22	69.90	73.57	77.25	80.93	84.61	88.29
8.75	64.38	68.16	71.95	75.74	79.53	83.31	87.10	90.89
9.00	66.22	70.11	74.01	77.90	81.80	85.69	89.59	93.48
9.25	68.06	72.06	76.06	80.07	84.07	88.07	92.08	96.08
9.50	69.90	74.01	78.12	82.23	86.34	90.45	94.56	98.68
9.75	71.74	75.95	80.17	84.39	88.61	92.83	97.05	101.27
10.00	73.57	77.90	82.23	86.56	90.89	95.21	99.54	103.87
10.25	75.41	79.85	84.29	88.72	93.16	97.59	102.03	106.47
10.50	77.25	81.80	86.34	90.89	95.43	99.97	104.52	109.06
10.75	79.09	83.75	88.40	93.05	97.70	102.36	107.01	111.66
11.00	80.93	85.69	90.45	95.21	99.97	104.74	109.50	114.26
11.25	82.77	87.64	92.51	97.38	102.25	107.12	111.98	116.85
11.50	84.61	89.59	94.56	99.54	104.52	109.50	114.47	119.45
11.75	86.45	91.54	96.62	101.71	106.79	111.88	116.96	122.05
12.00	88.29	93.48	98.68	103.87	109.06	114.26	119.45	124.64
12.50	91.97	97.38	102.79	108.20	113.61	119.02	124.43	129.84
13.00	95.65	101.27	106.90	112.53	118.15	123.78	129.40	135.03
13.50	99.33	105.17	111.01	116.85	122.70	128.54	134.38	140.22
14.00	103.00	109.06	115.12	121.18	127.24	133.30	139.36	145.42
14.50	106.68	112.96	119.23	125.51	131.78	138.06	144.34	150.61
15.00	110.36	116.85	123.35	129.84	136.33	142.82	149.31	155.80
15.50	114.04	120.75	127.46	134.17	140.87	147.58	154.29	161.00
16.00	117.72	124.64	131.57	138.49	145.42	152.34	159.27	166.19
16.50	121.40	128.54	135.68	142.82	149.96	157.10	164.24	171.39
17.00	125.08	132.43	139.79	147.15	154.51	161.86	169.22	176.58
17.50	128.76	136.33	143.90	151.48	159.05	166.62	174.20	181.77
18.00	132.43	140.22	148.01	155.80	163.59	171.39	179.18	186.97
18.50	136.11	144.12	152.13	160.13	168.14	176.15	184.15	192.16
19.00	139.79	148.01	156.24	164.46	172.68	180.91	189.13	197.35
19.50	143.47	151.91	160.35	168.79	177.23	185.67	194.11	202.55
20.00	147.15	155.80	164.46	173.12	181.77	190.43	199.08	207.74
20.50	150.83	159.70	168.57	177.44	186.32	195.19	204.06	212.93
21.00	154.51	163.59	172.68	181.77	190.86	199.95	209.04	218.13
21.50	158.19	167.49	176.80	186.10	195.41	204.71	214.02	223.32
22.00	161.86	171.39	180.91	190.43	199.95	209.47	218.99	228.51
22.50	165.54	175.28	185.02	194.76	204.49	214.23	223.97	233.71
23.00	169.22	179.18	189.13	199.08	209.04	218.99	228.95	238.90
23.50	172.90	183.07	193.24	203.41	213.58	223.75	233.92	244.09

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A3.5.8 Thickness 2.125 in to 3.0 in & Widths 24.00 in to 26.00 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175
				Thickness (in.)				
	2.125	2.25	2.375	2.50	2.625	2.75	2.875	3.0
Width (in.)								
24.00	176.58	186.97	197.35	207.74	218.13	228.51	238.90	249.29
24.50	180.26	190.86	201.46	212.07	222.67	233.27	243.88	254.48
25.00	183.94	194.76	205.58	216.40	227.22	238.03	248.85	259.67
25.50	187.61	198.65	209.69	220.72	231.76	242.79	253.83	264.87
26.00	191.29	202.55	213.80	225.05	236.30	247.56	258.81	270.06

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A3.5.9 Thickness 3.125 in to 4.0 in & Widths 6.25 in to 23.50 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0150
				Thickness (in.)				
	3.125	3.25	3.375	3.50	3.625	3.75	3.875	4.0
Width (in.)								
6.25	67.62	70.33	73.03	75.74	78.44	81.15	83.85	86.35
6.50	70.33	73.14	75.95	78.77	81.58	84.39	87.21	89.80
6.75	73.03	75.95	78.88	81.80	84.72	87.64	90.56	93.25
7.00	75.74	78.77	81.80	84.83	87.86	90.89	93.92	96.71
7.25	78.44	81.58	84.72	87.86	90.99	94.13	97.27	100.16
7.50	81.15	84.39	87.64	90.89	94.13	97.38	100.62	103.61
7.75	83.85	87.21	90.56	93.92	97.27	100.62	103.98	107.07
8.00	86.56	90.02	93.48	96.95	100.41	103.87	107.33	110.52
8.25	89.26	92.83	96.40	99.97	103.55	107.12	110.69	113.98
8.50	91.97	95.65	99.33	103.00	106.68	110.36	114.04	117.43
8.75	94.67	98.46	102.25	106.03	109.82	113.61	117.39	120.88
9.00	97.38	101.27	105.17	109.06	112.96	116.85	120.75	124.34
9.25	100.08	104.09	108.09	112.09	116.10	120.10	124.10	127.79
9.50	102.79	106.90	111.01	115.12	119.23	123.35	127.46	131.25
9.75	105.49	109.71	113.93	118.15	122.37	126.59	130.81	134.70
10.00	108.20	112.53	116.85	121.18	125.51	129.84	134.17	138.15
10.25	110.90	115.34	119.77	124.21	128.65	133.08	137.52	141.61
10.50	113.61	118.15	122.70	127.24	131.78	136.33	140.87	145.06
10.75	116.31	120.97	125.62	130.27	134.92	139.58	144.23	148.51
11.00	119.02	123.78	128.54	133.30	138.06	142.82	147.58	151.97
11.25	121.72	126.59	131.46	136.33	141.20	146.07	150.94	155.42
11.50	124.43	129.40	134.38	139.36	144.34	149.31	154.29	158.88
11.75	127.13	132.22	137.30	142.39	147.47	152.56	157.64	162.33
12.00	129.84	135.03	140.22	145.42	150.61	155.80	161.00	165.78
12.50	135.25	140.66	146.07	151.48	156.89	162.30	167.71	172.69
13.00	140.66	146.28	151.91	157.54	163.16	168.79	174.41	179.60
13.50	146.07	151.91	157.75	163.59	169.44	175.28	181.12	186.51
14.00	151.48	157.54	163.59	169.65	175.71	181.77	187.83	193.41
14.50	156.89	163.16	169.44	175.71	181.99	188.26	194.54	200.32
15.00	162.30	168.79	175.28	181.77	188.26	194.76	201.25	207.23
15.50	167.71	174.41	181.12	187.83	194.54	201.25	207.96	214.14
16.00	173.12	180.04	186.97	193.89	200.81	207.74	214.66	221.04
16.50	178.53	185.67	192.81	199.95	207.09	214.23	221.37	227.95
17.00	183.94	191.29	198.65	206.01	213.37	220.72	228.08	234.86
17.50	189.35	196.92	204.49	212.07	219.64	227.22	234.79	241.77
18.00	194.76	202.55	210.34	218.13	225.92	233.71	241.50	248.68
18.50	200.17	208.17	216.18	224.19	232.19	240.20	248.21	255.58
19.00	205.58	213.80	222.02	230.24	238.47	246.69	254.91	262.49
19.50	210.99	219.42	227.86	236.30	244.74	253.18	261.62	269.40
20.00	216.40	225.05	233.71	242.36	251.02	259.67	268.33	276.31
20.50	221.81	230.68	239.55	248.42	257.29	266.17	275.04	283.21
21.00	227.22	236.30	245.39	254.48	263.57	272.66	281.75	290.12
21.50	232.63	241.93	251.24	260.54	269.85	279.15	288.46	297.03
22.00	238.03	247.56	257.08	266.60	276.12	285.64	295.16	303.94
22.50	243.44	253.18	262.92	272.66	282.40	292.13	301.87	310.84
23.00	248.85	258.81	268.76	278.72	288.67	298.63	308.58	317.75
23.50	254.26	264.44	274.61	284.78	294.95	305.12	315.29	324.66

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A3.5.10 Thickness 3.125 in to 4.0 in & Widths 24.00 in to 26.00 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0175	1.0150
				Thickness (in.)				
	3.125	3.25	3.375	3.50	3.625	3.75	3.875	4.0
Width (in.)								
24.00	259.67	270.06	280.45	290.84	301.22	311.61	322.00	331.57
24.50	265.08	275.69	286.29	296.89	307.50	318.10	328.70	338.47
25.00	270.49	281.31	292.13	302.95	313.77	324.59	335.41	345.38
25.50	275.90	286.94	297.97	309.01	320.05	331.08	342.12	352.29
26.00	281.31	292.57	303.82	315.07	326.32	337.58	348.83	359.20

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A3.5.11 Thickness 4.125 in to 5.0 in & Widths 6.25 in to 23.50 in

Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0150	1.0150	1.0150	1.0150	1.0150	1.0150	1.0150	1.0150
				Thickness (in.)				
	4.125	4.25	4.375	4.50	4.625	4.75	4.875	5.0
Width (in.)								
6.25	89.04	91.74	94.44	97.14	99.84	102.54	105.23	107.93
6.50	92.61	95.41	98.22	101.02	103.83	106.64	109.44	112.25
6.75	96.17	99.08	102.00	104.91	107.82	110.74	113.65	116.57
7.00	99.73	102.75	105.77	108.80	111.82	114.84	117.86	120.88
7.25	103.29	106.42	109.55	112.68	115.81	118.94	122.07	125.20
7.50	106.85	110.09	113.33	116.57	119.80	123.04	126.28	129.52
7.75	110.41	113.76	117.11	120.45	123.80	127.14	130.49	133.84
8.00	113.98	117.43	120.88	124.34	127.79	131.25	134.70	138.15
8.25	117.54	121.10	124.66	128.22	131.78	135.35	138.91	142.47
8.50	121.10	124.77	128.44	132.11	135.78	139.45	143.12	146.79
8.75	124.66	128.44	132.22	135.99	139.77	143.55	147.33	151.10
9.00	128.22	132.11	135.99	139.88	143.77	147.65	151.54	155.42
9.25	131.78	135.78	139.77	143.77	147.76	151.75	155.75	159.74
9.50	135.35	139.45	143.55	147.65	151.75	155.85	159.96	164.06
9.75	138.91	143.12	147.33	151.54	155.75	159.96	164.16	168.37
10.00	142.47	146.79	151.10	155.42	159.74	164.06	168.37	172.69
10.25	146.03	150.46	154.88	159.31	163.73	168.16	172.58	177.01
10.50	149.59	154.13	158.66	163.19	167.73	172.26	176.79	181.33
10.75	153.16	157.80	162.44	167.08	171.72	176.36	181.00	185.64
11.00	156.72	161.47	166.22	170.96	175.71	180.46	185.21	189.96
11.25	160.28	165.14	169.99	174.85	179.71	184.56	189.42	194.28
11.50	163.84	168.81	173.77	178.74	183.70	188.66	193.63	198.59
11.75	167.40	172.48	177.55	182.62	187.69	192.77	197.84	202.91
12.00	170.96	176.14	181.33	186.51	191.69	196.87	202.05	207.23
12.50	178.09	183.48	188.88	194.28	199.67	205.07	210.47	215.86
13.00	185.21	190.82	196.44	202.05	207.66	213.27	218.89	224.50
13.50	192.33	198.16	203.99	209.82	215.65	221.48	227.30	233.13
14.00	199.46	205.50	211.55	217.59	223.63	229.68	235.72	241.77
14.50	206.58	212.84	219.10	225.36	231.62	237.88	244.14	250.40
15.00	213.71	220.18	226.66	233.13	239.61	246.08	252.56	259.04
15.50	220.83	227.52	234.21	240.90	247.60	254.29	260.98	267.67
16.00	227.95	234.86	241.77	248.68	255.58	262.49	269.40	276.31
16.50	235.08	242.20	249.32	256.45	263.57	270.69	277.82	284.94
17.00	242.20	249.54	256.88	264.22	271.56	278.90	286.24	293.57
17.50	249.32	256.88	264.43	271.99	279.54	287.10	294.65	302.21
18.00	256.45	264.22	271.99	279.76	287.53	295.30	303.07	310.84
18.50	263.57	271.56	279.54	287.53	295.52	303.50	311.49	319.48
19.00	270.69	278.90	287.10	295.30	303.50	311.71	319.91	328.11
19.50	277.82	286.24	294.65	303.07	311.49	319.91	328.33	336.75
20.00	284.94	293.57	302.21	310.84	319.48	328.11	336.75	345.38
20.50	292.06	300.91	309.76	318.61	327.47	336.32	345.17	354.02
21.00	299.19	308.25	317.32	326.39	335.45	344.52	353.58	362.65
21.50	306.31	315.59	324.87	334.16	343.44	352.72	362.00	371.29
22.00	313.43	322.93	332.43	341.93	351.43	360.92	370.42	379.92
22.50	320.56	330.27	339.99	349.70	359.41	369.13	378.84	388.55
23.00	327.68	337.61	347.54	357.47	367.40	377.33	387.26	397.19
23.50	334.80	344.95	355.10	365.24	375.39	385.53	395.68	405.82

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A3.5.12 Thickness 4.125 in to 5.0 in & Widths 24.00 in to 26.00 in

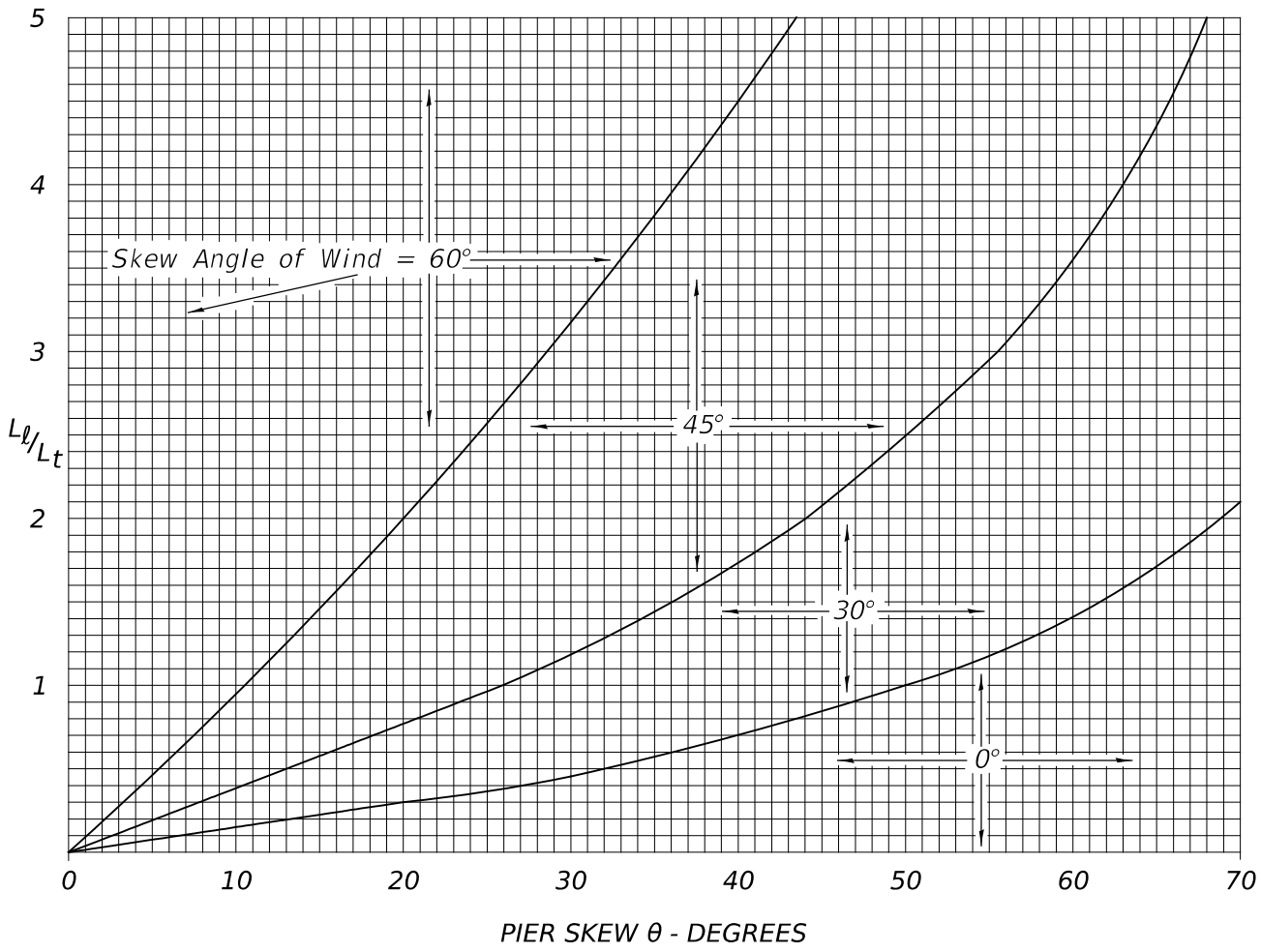
Weight of Rectangular Steel Sections

(Pounds per Linear Foot)

Coeff.	1.0150	1.0150	1.0150	1.0150	1.0150	1.0150	1.0150	1.0150
				Thickness (in.)				
	4.125	4.25	4.375	4.50	4.625	4.75	4.875	5.0
Width (in.)								
24.00	341.93	352.29	362.65	373.01	383.37	393.74	404.10	414.46
24.50	349.05	359.63	370.20	380.78	391.36	401.94	412.51	423.09
25.00	356.18	366.97	377.76	388.55	399.35	410.14	420.93	431.73
25.50	363.30	374.30	385.32	396.32	407.33	418.34	429.35	440.36
26.00	370.42	381.65	392.87	404.10	415.32	426.55	437.77	449.00

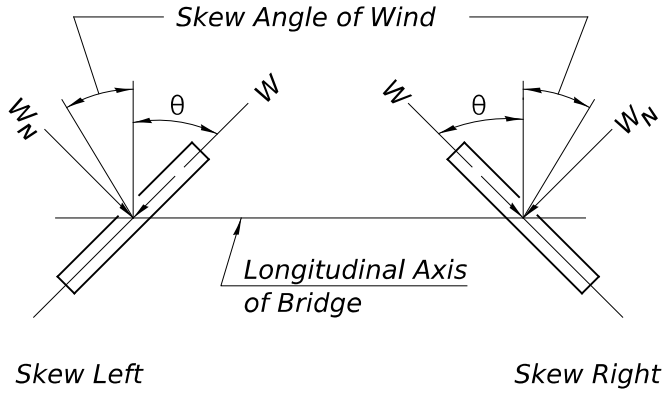
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A3.6 Skew Angle of Wind Girder Spans (W_N Max)



L_ℓ = Loaded length, longitudinal wind on superstructure

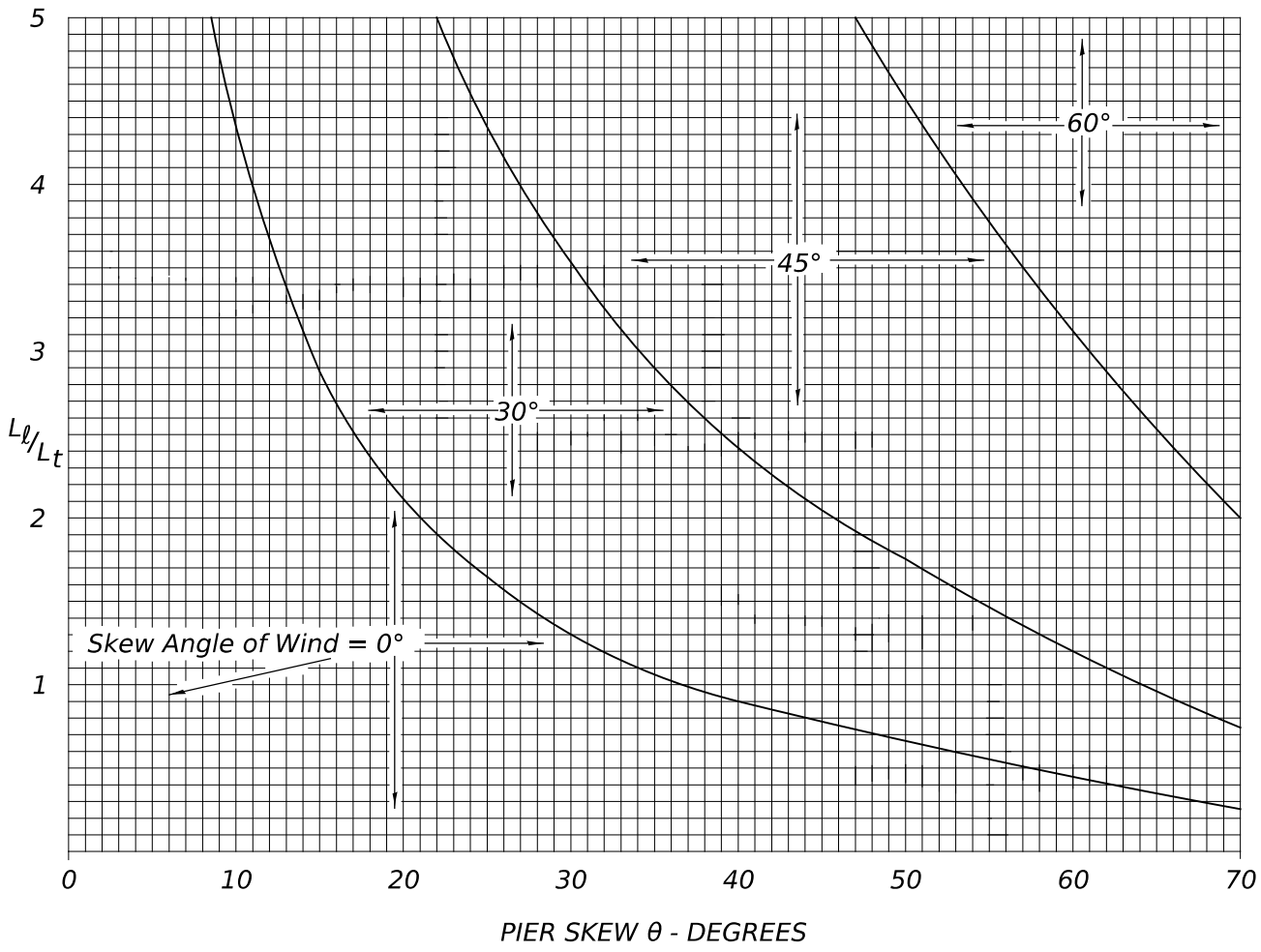
L_t = Loaded length, transverse wind on superstructure



SKEW ANGLE OF WIND GIRDER SPANS W_N MAX.

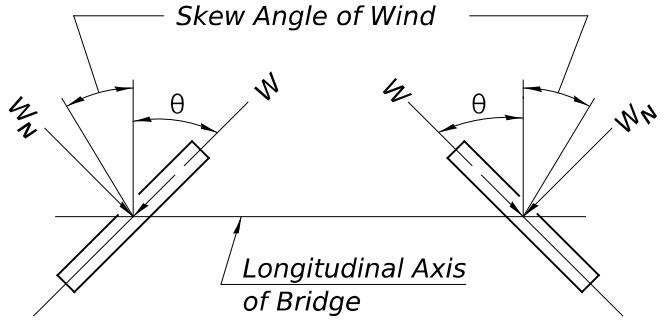
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A3.7 Skew Angle of Wind Girder Spans (W Max)



L_ℓ = Loaded length, longitudinal wind on superstructure

L_t = Loaded length, transverse wind on superstructure



SKEW ANGLE OF WIND
GIRDER SPANS
W MAX.

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A3.8 Table of Maximum Live Load Moments, Shears and Reactions For Simple Spans

The values in this table were generated using the HL-93 live loading specified in AASHTO LRFD Article 3.6.1.2 for one lane. The moments provided include the dynamic load allowance (IM). The location at which the maximum moment occurs is also given. The shears/reactions provided are given with and without (IM).

A3.8.1 Spans 1 ft through 70 ft

	Moment		End Shear/Reaction			Moment		End Shear/Reaction	
Span (ft.)	Location	M(LL+IM) (k-ft.)	R(LL) (kips)	R(LL+IM) (kips)	Span (ft.)	Location	M(LL+IM) (k-ft.)	R(LL) (kips)	R(LL+IM) (kips)
1	0.500	10.7	32.3	42.9	36	0.472	637.2	64.9	82.5
2	0.500	21.6	32.6	43.2	37	0.473	659.6	65.7	83.4
3	0.500	32.6	33.0	43.5	38	0.474	682.2	66.5	84.4
4	0.500	43.8	33.3	43.8	39	0.474	704.9	67.2	85.3
5	0.500	55.2	33.6	44.2	40	0.475	727.8	68.0	86.2
6	0.500	66.7	35.3	46.3	41	0.443	754.6	68.7	87.1
7	0.500	78.4	38.0	49.7	42	0.444	784.9	69.4	87.9
8	0.500	90.2	40.1	52.4	43	0.446	815.3	70.1	88.7
9	0.500	102.2	41.8	54.6	44	0.447	845.9	70.8	89.5
10	0.500	114.4	43.2	56.4	45	0.448	876.7	71.5	90.3
11	0.409	131.8	44.4	57.9	46	0.449	907.7	72.1	91.1
12	0.417	149.7	45.5	59.3	47	0.450	938.9	72.7	91.8
13	0.423	167.9	46.5	60.4	48	0.451	970.2	73.4	92.5
14	0.429	186.4	47.3	61.5	49	0.452	1001.6	74.0	93.2
15	0.433	205.0	48.1	62.4	50	0.453	1033.3	74.6	93.9
16	0.438	223.8	48.9	63.3	51	0.454	1065.1	75.1	94.6
17	0.441	242.8	49.6	64.1	52	0.455	1097.1	75.7	95.2
18	0.444	262.0	50.2	64.9	53	0.456	1129.2	76.3	95.9
19	0.447	281.4	50.8	65.6	54	0.457	1161.6	76.8	96.5
20	0.450	301.0	51.4	66.3	55	0.458	1194.0	77.4	97.1
21	0.452	320.8	52.0	66.9	56	0.458	1226.7	77.9	97.7
22	0.455	340.7	52.5	67.5	57	0.459	1259.5	78.5	98.3
23	0.457	360.8	53.0	68.1	58	0.460	1292.5	79.0	98.9
24	0.458	381.0	53.5	68.6	59	0.460	1325.6	79.5	99.5
25	0.460	401.5	54.1	69.3	60	0.461	1358.9	80.0	100.1
26	0.462	422.1	55.1	70.5	61	0.462	1392.4	80.5	100.6
27	0.463	442.8	56.0	71.7	62	0.462	1426.1	81.0	101.2
28	0.464	463.8	57.0	72.8	63	0.463	1459.9	81.5	101.7
29	0.466	484.9	58.1	74.2	64	0.464	1493.8	82.0	102.3
30	0.467	506.1	59.2	75.6	65	0.464	1528.0	82.5	102.8
31	0.468	527.6	60.2	76.8	66	0.465	1562.3	82.9	103.3
32	0.469	549.2	61.2	78.1	67	0.465	1596.7	83.4	103.9
33	0.470	570.9	62.2	79.2	68	0.466	1631.4	83.9	104.4
34	0.471	592.9	63.1	80.4	69	0.466	1666.2	84.3	104.9
35	0.471	615.0	64.0	81.4	70	0.467	1701.1	84.8	105.4

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A3.8.1 Spans 71 ft through 150 ft

	Moment		End Shear/Reaction			Moment		End Shear/Reaction	
Span (ft.)	Location	M(LL+IM) (k-ft.)	R(LL) (kips)	R(LL+IM) (kips)	Span (ft.)	Location	M(LL+IM) (k-ft.)	R(LL) (kips)	R(LL+IM) (kips)
71	0.467	1736.2	85.3	105.9	111	0.479	3273.6	101.5	123.2
72	0.468	1771.5	85.7	106.4	112	0.479	3315.3	101.8	123.6
73	0.468	1806.9	86.2	106.9	113	0.479	3357.2	102.2	124.0
74	0.468	1842.5	86.6	107.4	114	0.480	3399.3	102.6	124.4
75	0.469	1878.3	87.0	107.8	115	0.480	3441.5	103.0	124.8
76	0.469	1914.2	87.5	108.3	116	0.480	3483.9	103.3	125.2
77	0.470	1950.3	87.9	108.8	117	0.480	3526.4	103.7	125.6
78	0.470	1986.6	88.3	109.3	118	0.480	3569.1	104.1	125.9
79	0.470	2023.0	88.8	109.7	119	0.480	3612.0	104.4	126.3
80	0.471	2059.6	89.2	110.2	120	0.481	3655.0	104.8	126.7
81	0.471	2096.3	89.6	110.6	121	0.481	3698.2	105.2	127.1
82	0.472	2133.2	90.0	111.1	122	0.481	3741.5	105.5	127.5
83	0.472	2170.3	90.5	111.6	123	0.481	3785.0	105.9	127.9
84	0.472	2207.5	90.9	112.0	124	0.481	3828.7	106.3	128.2
85	0.473	2244.9	91.3	112.4	125	0.481	3872.5	106.6	128.6
86	0.473	2282.4	91.7	112.9	126	0.481	3916.5	107.0	129.0
87	0.473	2320.2	92.1	113.3	127	0.482	3960.7	107.3	129.4
88	0.473	2358.0	92.5	113.8	128	0.482	4005.0	107.7	129.7
89	0.474	2396.1	92.9	114.2	129	0.482	4049.4	108.1	130.1
90	0.474	2434.3	93.3	114.6	130	0.482	4094.1	108.4	130.5
91	0.474	2472.6	93.7	115.1	131	0.482	4138.9	108.8	130.9
92	0.475	2511.1	94.1	115.5	132	0.482	4183.8	109.1	131.2
93	0.475	2549.8	94.5	115.9	133	0.482	4228.9	109.5	131.6
94	0.475	2588.6	94.9	116.3	134	0.483	4274.2	109.9	132.0
95	0.475	2627.6	95.3	116.8	135	0.483	4319.6	110.2	132.3
96	0.476	2666.8	95.7	117.2	136	0.483	4365.2	110.6	132.7
97	0.476	2706.1	96.1	117.6	137	0.483	4411.0	110.9	133.1
98	0.476	2745.6	96.5	118.0	138	0.483	4456.9	111.3	133.4
99	0.476	2785.3	96.9	118.4	139	0.483	4502.9	111.6	133.8
100	0.477	2825.1	97.3	118.8	140	0.483	4549.2	112.0	134.2
101	0.477	2865.0	97.7	119.2	141	0.483	4595.6	112.4	134.5
102	0.477	2905.2	98.1	119.6	142	0.484	4642.1	112.7	134.9
103	0.477	2945.5	98.4	120.0	143	0.484	4688.8	113.1	135.3
104	0.478	2985.9	98.8	120.4	144	0.484	4735.7	113.4	135.6
105	0.478	3026.5	99.2	120.8	145	0.484	4782.8	113.8	136.0
106	0.478	3067.3	99.6	121.2	146	0.484	4829.9	114.1	136.4
107	0.478	3108.2	100.0	121.6	147	0.484	4877.3	114.5	136.7
108	0.478	3149.3	100.3	122.0	148	0.484	4924.8	114.8	137.1
109	0.479	3190.6	100.7	122.4	149	0.484	4972.5	115.2	137.4
110	0.479	3232.0	101.1	122.8	150	0.484	5020.3	115.5	137.8

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A3.9 Design Guides

Design guides are available under the Design section on the primary Bridges & Structures Page. These guides are provided as supplements to this manual. The following design guides are available for viewing and download.

Design Guides	Reference
Bridge Scupper Placement	BM 2.3.6.1.8
LRFD Deck Design	BM 3.2.1
LRFD Slab Bridge Design	BM 3.2.11
LRFD Composite Steel Beam Design (for Straight Bridges)	BM 3.3.4
LRFD Stud Shear Connector Design (for Straight Bridges)	BM 3.3.9
Camber Design	BM 3.3.12
LRFD Bolted Splice Design	BM 3.3.21
LRFD PPC I-, Bulb-T, and IL-Beam Design	BM 3.4
LRFD PPC Deck Beam Design	BM 3.5
Temporary Sheet Piling Design	BM 3.13.1
Temporary Geotextile Retaining Wall Design	BM 3.13.2
Cofferdam Seal Coat Design	BM 3.13.3
Seismic Design	BM 3.15
Seismic Site Class Definition	AGMU Memo 09.1
Liquefaction Analysis Procedure	AGMU Memo 10.1
Geotechnical Pile Design Procedure	AGMU Memo 10.2
Shaft Overturning and Torsional Analysis	AGMU Memo 11.1
LRFD Culvert Design Loads	CM 3.4
LRFD Culvert Flexure Design	CM 3.5
LRFD Culvert Shear Design	CM 3.5.12

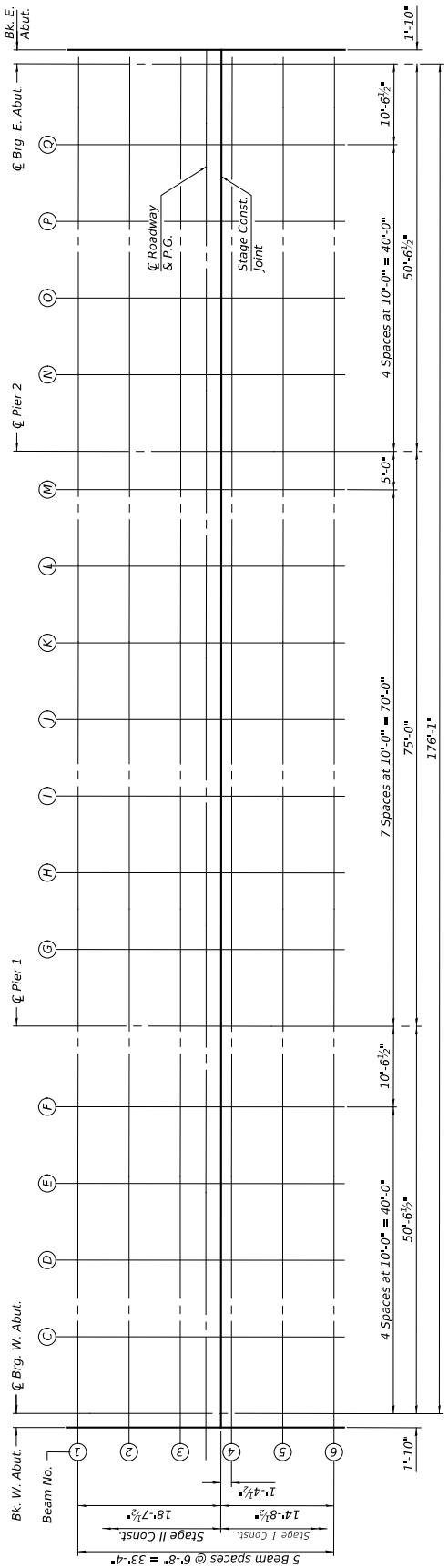
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A3.10 Design Layout Examples – Top of Slab Elevations

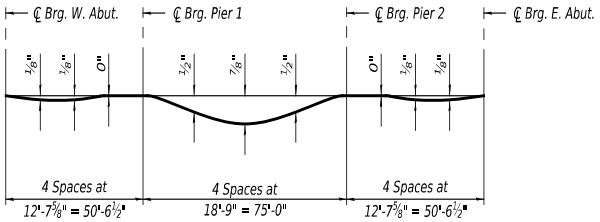
The following figures are reproduced for information only. See Section 3.1.9 for additional discussion.

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A3.10.1 Top of Slab Elevations – Plan View



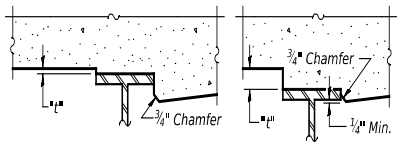
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DEAD LOAD DEFLECTION DIAGRAM

(Includes weight of concrete only.)

Note:
The above deflections are not to be used in the field if the engineer is working from the grade elevations adjusted for dead load deflections and grinding as shown below.



At Minimum Fillet *At Maximum Fillet*

To determine "t": After all structural steel has been erected, elevations of the top flanges of the beams shall be taken at intervals shown below. These elevations subtracted from the "Theoretical Grade Elevations Adjusted for Dead Load Deflection and Grinding" shown below, minus the initial slab thickness prior to grinding, equals the fillet heights "t" above top flange of beams.

The slab is to be ground after curing to achieve smoothness, but the slab is not to be ground to elevations below the "Theoretical Grade Elevations" shown below. For grinding the deck, see Special Provisions.

FILLET HEIGHTS

BEAM 1

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	-16.67	453.56	453.58
℄ Brg. W. Abut.	1070+86.96	-16.67	453.55	453.57
C	1070+96.96	-16.67	453.45	453.48
D	1071+06.96	-16.67	453.36	453.39
E	1071+16.96	-16.67	453.27	453.30
F	1071+26.96	-16.67	453.18	453.21
℄ Pier 1	1071+37.50	-16.67	453.10	453.12
G	1071+47.50	-16.67	453.01	453.06
H	1071+57.50	-16.67	452.93	453.00
I	1071+67.50	-16.67	452.86	452.94
J	1071+77.50	-16.67	452.78	452.87
K	1071+87.50	-16.67	452.71	452.79
L	1071+97.50	-16.67	452.64	452.70
M	1072+07.50	-16.67	452.58	452.61
℄ Pier 2	1072+12.50	-16.67	452.54	452.56
N	1072+22.50	-16.67	452.48	452.50
O	1072+32.50	-16.67	452.42	452.45
P	1072+42.50	-16.67	452.36	452.39
Q	1072+52.50	-16.67	452.31	452.33
℄ Brg. E. Abut.	1072+63.04	-16.67	452.25	452.27
Bk. E. Abut.	1072+64.88	-16.67	452.24	452.26

BEAM 2

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	-10.00	453.69	453.71
℄ Brg. W. Abut.	1070+86.96	-10.00	453.67	453.69
C	1070+96.96	-10.00	453.58	453.60
D	1071+06.96	-10.00	453.48	453.52
E	1071+16.96	-10.00	453.39	453.42
F	1071+26.96	-10.00	453.31	453.33
℄ Pier 1	1071+37.50	-10.00	453.22	453.24
G	1071+47.50	-10.00	453.14	453.18
H	1071+57.50	-10.00	453.06	453.12
I	1071+67.50	-10.00	452.98	453.06
J	1071+77.50	-10.00	452.91	453.00
K	1071+87.50	-10.00	452.83	452.91
L	1071+97.50	-10.00	452.77	452.82
M	1072+07.50	-10.00	452.70	452.73
℄ Pier 2	1072+12.50	-10.00	452.67	452.69
N	1072+22.50	-10.00	452.60	452.62
O	1072+32.50	-10.00	452.54	452.57
P	1072+42.50	-10.00	452.48	452.52
Q	1072+52.50	-10.00	452.43	452.46
℄ Brg. E. Abut.	1072+63.04	-10.00	452.37	452.39
Bk. E. Abut.	1072+64.88	-10.00	452.36	452.38

BEAM 3

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	-3.33	453.79	453.81
℄ Brg. W. Abut.	1070+86.96	-3.33	453.77	453.79
C	1070+96.96	-3.33	453.68	453.70
D	1071+06.96	-3.33	453.58	453.62
E	1071+16.96	-3.33	453.49	453.52
F	1071+26.96	-3.33	453.41	453.43
℄ Pier 1	1071+37.50	-3.33	453.32	453.34
G	1071+47.50	-3.33	453.24	453.28
H	1071+57.50	-3.33	453.16	453.22
I	1071+67.50	-3.33	453.08	453.16
J	1071+77.50	-3.33	453.01	453.10
K	1071+87.50	-3.33	452.93	453.01
L	1071+97.50	-3.33	452.87	452.92
M	1072+07.50	-3.33	452.80	452.83
℄ Pier 2	1072+12.50	-3.33	452.77	452.79
N	1072+22.50	-3.33	452.70	452.72
O	1072+32.50	-3.33	452.64	452.67
P	1072+42.50	-3.33	452.58	452.62
Q	1072+52.50	-3.33	452.53	452.56
℄ Brg. E. Abut.	1072+63.04	-3.33	452.47	452.49
Bk. E. Abut.	1072+64.88	-3.33	452.46	452.48

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℄ ROADWAY & PG

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	0.00	453.84	453.86
℄ Brg. W. Abut.	1070+86.96	0.00	453.82	453.84
C	1070+96.96	0.00	453.73	453.75
D	1071+06.96	0.00	453.63	453.67
E	1071+16.96	0.00	453.54	453.57
F	1071+26.96	0.00	453.46	453.48
℄ Pier 1	1071+37.50	0.00	453.37	453.39
G	1071+47.50	0.00	453.29	453.33
H	1071+57.50	0.00	453.21	453.27
I	1071+67.50	0.00	453.13	453.21
J	1071+77.50	0.00	453.06	453.15
K	1071+87.50	0.00	452.98	453.06
L	1071+97.50	0.00	452.92	452.97
M	1072+07.50	0.00	452.85	452.88
℄ Pier 2	1072+12.50	0.00	452.82	452.84
N	1072+22.50	0.00	452.75	452.77
O	1072+32.50	0.00	452.69	452.72
P	1072+42.50	0.00	452.63	452.67
Q	1072+52.50	0.00	452.58	452.61
℄ Brg. E. Abut.	1072+63.04	0.00	452.52	452.54
Bk. E. Abut.	1072+64.88	0.00	452.51	452.53

STAGE CONSTRUCTION JOINT

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	1.96	453.81	453.83
℄ Brg. W. Abut.	1070+86.96	1.96	453.79	453.81
C	1070+96.96	1.96	453.70	453.73
D	1071+06.96	1.96	453.60	453.64
E	1071+16.96	1.96	453.52	453.54
F	1071+26.96	1.96	453.43	453.45
℄ Pier 1	1071+37.50	1.96	453.34	453.36
G	1071+47.50	1.96	453.26	453.30
H	1071+57.50	1.96	453.18	453.24
I	1071+67.50	1.96	453.10	453.18
J	1071+77.50	1.96	453.03	453.12
K	1071+87.50	1.96	452.96	453.03
L	1071+97.50	1.96	452.89	452.94
M	1072+07.50	1.96	452.82	452.85
℄ Pier 2	1072+12.50	1.96	452.79	452.81
N	1072+22.50	1.96	452.72	452.74
O	1072+32.50	1.96	452.66	452.69
P	1072+42.50	1.96	452.60	452.64
Q	1072+52.50	1.96	452.55	452.58
℄ Brg. E. Abut.	1072+63.04	1.96	452.49	452.51
Bk. E. Abut.	1072+64.88	1.96	452.48	452.50

BEAM 4

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	3.33	453.79	453.81
℄ Brg. W. Abut.	1070+86.96	3.33	453.77	453.79
C	1070+96.96	3.33	453.68	453.70
D	1071+06.96	3.33	453.58	453.62
E	1071+16.96	3.33	453.49	453.52
F	1071+26.96	3.33	453.41	453.43
℄ Pier 1	1071+37.50	3.33	453.32	453.34
G	1071+47.50	3.33	453.24	453.28
H	1071+57.50	3.33	453.16	453.22
I	1071+67.50	3.33	453.08	453.16
J	1071+77.50	3.33	453.01	453.10
K	1071+87.50	3.33	452.93	453.01
L	1071+97.50	3.33	452.87	452.92
M	1072+07.50	3.33	452.80	452.83
℄ Pier 2	1072+12.50	3.33	452.77	452.79
N	1072+22.50	3.33	452.70	452.72
O	1072+32.50	3.33	452.64	452.67
P	1072+42.50	3.33	452.58	452.62
Q	1072+52.50	3.33	452.53	452.56
℄ Brg. E. Abut.	1072+63.04	3.33	452.47	452.49
Bk. E. Abut.	1072+64.88	3.33	452.46	452.48

BEAM 5

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	10.00	453.69	453.71
℄ Brg. W. Abut.	1070+86.96	10.00	453.67	453.69
C	1070+96.96	10.00	453.58	453.60
D	1071+06.96	10.00	453.48	453.52
E	1071+16.96	10.00	453.39	453.42
F	1071+26.96	10.00	453.31	453.33
℄ Pier 1	1071+37.50	10.00	453.22	453.24
G	1071+47.50	10.00	453.14	453.18
H	1071+57.50	10.00	453.06	453.12
I	1071+67.50	10.00	452.98	453.06
J	1071+77.50	10.00	452.91	453.00
K	1071+87.50	10.00	452.83	452.91
L	1071+97.50	10.00	452.77	452.82
M	1072+07.50	10.00	452.70	452.73
℄ Pier 2	1072+12.50	10.00	452.67	452.69
N	1072+22.50	10.00	452.60	452.62
O	1072+32.50	10.00	452.54	452.57
P	1072+42.50	10.00	452.48	452.52
Q	1072+52.50	10.00	452.43	452.46
℄ Brg. E. Abut.	1072+63.04	10.00	452.37	452.39
Bk. E. Abut.	1072+64.88	10.00	452.36	452.38

BEAM 6

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Dead Load Deflection and Grinding
Bk. W. Abut.	1070+85.13	16.67	453.56	453.58
℄ Brg. W. Abut.	1070+86.96	16.67	453.55	453.57
C	1070+96.96	16.67	453.45	453.48
D	1071+06.96	16.67	453.36	453.39
E	1071+16.96	16.67	453.27	453.30
F	1071+26.96	16.67	453.18	453.21
℄ Pier 1	1071+37.50	16.67	453.10	453.12
G	1071+47.50	16.67	453.01	453.06
H	1071+57.50	16.67	452.93	453.00
I	1071+67.50	16.67	452.86	452.94
J	1071+77.50	16.67	452.78	452.87
K	1071+87.50	16.67	452.71	452.79
L	1071+97.50	16.67	452.64	452.70
M	1072+07.50	16.67	452.58	452.61
℄ Pier 2	1072+12.50	16.67	452.54	452.56
N	1072+22.50	16.67	452.48	452.50
O	1072+32.50	16.67	452.42	452.45
P	1072+42.50	16.67	452.36	452.39
Q	1072+52.50	16.67	452.31	452.33
℄ Brg. E. Abut.	1072+63.04	16.67	452.25	452.27
Bk. E. Abut.	1072+64.88	16.67	452.24	452.26

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A3.10.4 Top of Slab Elevations – West Approach

NORTH SHOULDER LINE

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of W. Appr. Slab	1070+56.13	-18.00	453.83	453.85
A	1070+66.13	-18.00	453.72	453.74
B	1070+76.13	-18.00	453.62	453.64
E. End of W. Appr. Slab	1070+86.13	-18.00	453.53	453.55

NORTH EDGE OF ROADWAY

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of W. Appr. Slab	1070+56.13	-12.00	453.95	453.97
A	1070+66.13	-12.00	453.84	453.86
B	1070+76.13	-12.00	453.74	453.76
E. End of W. Appr. Slab	1070+86.13	-12.00	453.65	453.67

℄ ROADWAY & PG

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of W. Appr. Slab	1070+56.13	0.00	454.13	454.15
A	1070+66.13	0.00	454.02	454.04
B	1070+76.13	0.00	453.92	453.94
E. End of W. Appr. Slab	1070+86.13	0.00	453.83	453.85

STAGE CONSTRUCTION JOINT

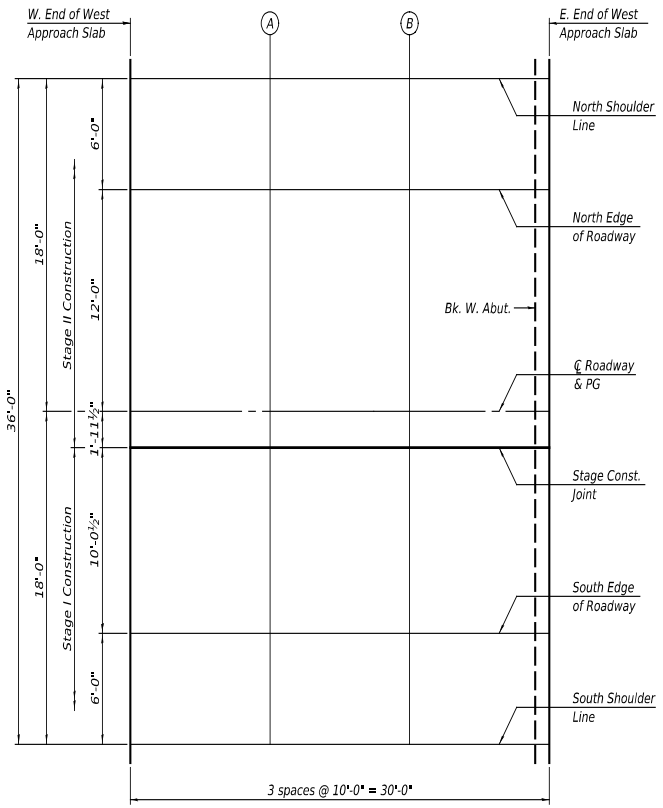
Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of W. Appr. Slab	1070+56.13	1.96	454.10	454.12
A	1070+66.13	1.96	454.00	454.02
B	1070+76.13	1.96	453.90	453.92
E. End of W. Appr. Slab	1070+86.13	1.96	453.80	453.82

SOUTH EDGE OF ROADWAY

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of W. Appr. Slab	1070+56.13	12.00	453.95	453.97
A	1070+66.13	12.00	453.84	453.86
B	1070+76.13	12.00	453.74	453.76
E. End of W. Appr. Slab	1070+86.13	12.00	453.65	453.67

SOUTH SHOULDER LINE

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of W. Appr. Slab	1070+56.13	18.00	453.83	453.85
A	1070+66.13	18.00	453.72	453.74
B	1070+76.13	18.00	453.62	453.64
E. End of W. Appr. Slab	1070+86.13	18.00	453.53	453.55



PLAN
West Approach (E.B.)

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A3.10.5 Top of Slab Elevations – East Approach

NORTH SHOULDER LINE

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of E. Appr. Slab	1072+63.88	-18.00	452.22	452.24
R	1072+73.88	-18.00	452.17	452.19
S	1072+83.88	-18.00	452.12	452.14
E. End of E. Appr. Slab	1072+93.88	-18.00	452.07	452.09

NORTH EDGE OF ROADWAY

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of E. Appr. Slab	1072+63.88	-12.00	452.34	452.36
R	1072+73.88	-12.00	452.29	452.31
S	1072+83.88	-12.00	452.24	452.26
E. End of E. Appr. Slab	1072+93.88	-12.00	452.19	452.21

℄ ROADWAY & PG

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of E. Appr. Slab	1072+63.88	0.00	452.52	452.54
R	1072+73.88	0.00	452.47	452.49
S	1072+83.88	0.00	452.42	452.44
E. End of E. Appr. Slab	1072+93.88	0.00	452.37	452.39

STAGE CONSTRUCTION JOINT

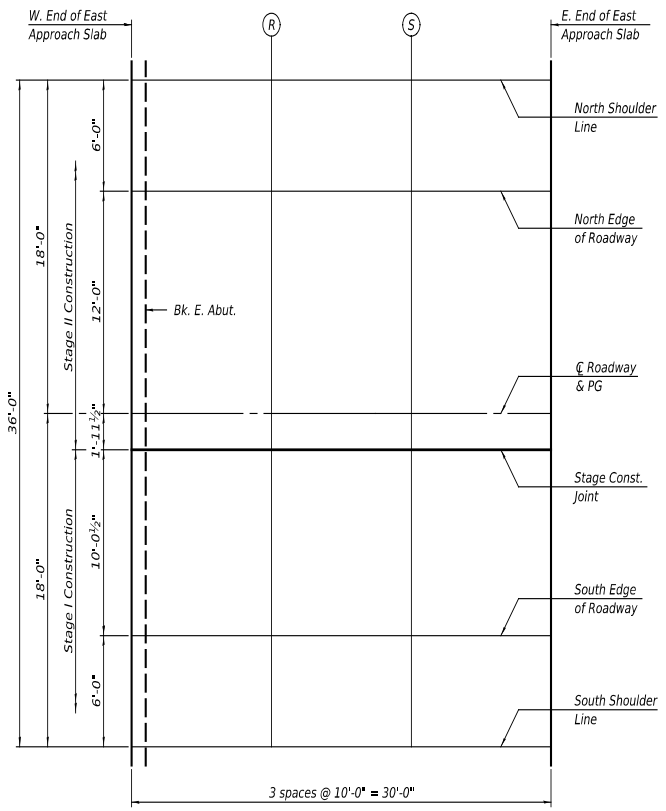
Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of E. Appr. Slab	1072+63.88	1.96	452.49	452.51
R	1072+73.88	1.96	452.44	452.46
S	1072+83.88	1.96	452.39	452.41
E. End of E. Appr. Slab	1072+93.88	1.96	452.34	452.36

SOUTH EDGE OF ROADWAY

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of E. Appr. Slab	1072+63.88	12.00	452.34	452.34
R	1072+73.88	12.00	452.29	452.29
S	1072+83.88	12.00	452.24	452.24
E. End of E. Appr. Slab	1072+93.88	12.00	452.19	452.19

SOUTH SHOULDER LINE

Location	Station	Offset	Theoretical Grade Elevations	Theoretical Grade Elevations Adjusted For Grinding
W. End of E. Appr. Slab	1072+63.88	18.00	452.22	452.24
R	1072+73.88	18.00	452.17	452.19
S	1072+83.88	18.00	452.12	452.14
E. End of E. Appr. Slab	1072+93.88	18.00	452.07	452.09



PLAN
East Approach (E.B.)



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