5. CONCRETE STRUCTURES

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

5.1 Materials

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

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

5.1.1 Concrete

MnDOT Spec. 2461 identifies and describes concrete mix types. Based on their strength, location of application, and durability properties, different mixes are used for various structural concrete components. Table 5.1.1.1 identifies the standard MnDOT concrete mix types to be used for different bridge components.

The four or five characters used to identify a concrete mix provide information on the properties of the mix. The first character designates the type of concrete (with or without air entrainment requirements). The second character identifies the grade of concrete. Each letter is associated with a different cement-void ratio. The third character in the label is the upper limit for the slump in inches. The fourth character identifies the coarse aggregate gradation. The fifth character, if present, identifies the type of coarse aggregate to be used. Note that there are two exceptions to the above: job mixes (JM) for box girders, and high performance concrete (HPC) mixes for bridge decks and slabs.

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

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

Table 5.1.1.1 Design Concrete Mix Summary

Location/Element	MnDOT Concrete Mix Designation	Design Compressive Strength (ksi)	Maximum Aggregate Size (in)
Cofferdam seals	1X62	5.0	1
Cast-in-place concrete piles and spread footing leveling pads	1P62	3.0	2
Drilled shafts	1X62 3X62	5.0 5.0	1 1
Footings and pile caps	1G52	4.0	1 ½ *
Abutment stems, wingwalls, cast-in-place wall stems, pier columns, and pier caps	3B52	4.0	1 ½ *
Integral abutment diaphragms and pier continuity diaphragms	Same mix as used in deck	4.0	1
Pretensioned superstructures	1W82 or 3W82	5.0 – 9.0 at final 4.5 – 7.5 at initial	1
Cast-in-place and precast box girders	3JM	6.0 or higher	1
Monolithic decks and slabs	3YHPC-M, 3YLCHPC-M or 3Y42-M	4.0	1
Decks and slabs that will receive a 2 inch concrete wearing course	3YHPC-S, 3YLCHPC-S or 3Y42-S	4.0	1
Barriers, parapets, medians, and sidewalks	3S52	4.0	1
Concrete wearing course	3U17A	4.0	$^{5}/_{8}$
MSE wall panels, PMBW blocks, and noisewall panels	3Y82	4.0	1
Precast box culverts, arches, and 3-sided structures	3W82	5.0 or higher	1*

* For determination of s_{xe} per LRFD 5.8.3.4.2, use max aggregate size a_g = ¾"

Reinforced Concrete Sections

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

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

[5.4.2.4-1] For reinforced concrete elements, use: $E_c = 33{,}000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$

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

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

Prestressed Concrete Elements

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

For pretensioned beams (M, MN, MW, and RB) fabricated with high-strength concrete (greater than 6.0 ksi), compute the modulus of elasticity with the ACI 363 equation below:

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

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

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

Table 5.1.1.2 summarizes concrete properties for analysis and design:

Table 5.1.1.2
Concrete Properties

Parameter	Equation/Value
Unit Weight	Reinforced Concrete Elements: $w_c = 0.145$ kcf for calculation of E_c $w_c = 0.150$ kcf for dead load calculation Pretensioned and Post-tensioned Elements: $w_c = 0.150$ kcf for calc. of E_c (except pretensioned beams) $w_c = 0.155$ kcf for dead load calculation
Modulus of Elasticity	Pretensioned Beams: E_c (ksi) $= 33{,}000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ where $f'_c \le 6$ ksi E_c (ksi) $= 1265 \cdot \sqrt{f'_c} + 1000$ where $f'_c > 6$ ksi All Other Concrete Elements: E_c (ksi) $= 33{,}000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$
Thermal Coefficient	$\alpha_C = 6.0 \times 10^{-6} = \text{in/in/}^\circ\text{F}$
Shrinkage Strain	Reinf. Conc.: $\varepsilon_{sh} = 0.0002$ @ 28 days and 0.0005 @ 1 year Prestressed Concrete: per LRFD Art. 5.4.2.3
Poisson's ratio	$\nu = 0.2$

5.1.2 Reinforcing Steel

Reinforcing bars shall satisfy MnDOT Spec 3301. ASTM A615 Grade 60 deformed bars (black or epoxy coated) should be used in most circumstances. In some cases, Grade 75 stainless steel bars will be required in the bridge deck and barrier (see Tech. Memo No. 11-15-B-06 *Policy on the Use of Stainless Steel Reinforcement in Bridge Decks & Barriers*). Use f_y = 75 ksi when designing with stainless steel bars. Always use stainless steel (either Grade 60 or 75 is adequate for this situation) for the connecting bar between approach panel and end diaphragm at integral and semi-integral abutments.

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

The modulus of elasticity for mild steel reinforcing (E_S) is 29,000 ksi.

All reinforcement bars, except stainless steel bars and bars that are entirely embedded in footings, shall be epoxy coated.

5.1.3 Reinforcement Bar Couplers

Contractors select reinforcement bar couplers that meet the requirements stated in MnDOT Spec. 2472.3.D.2. In general, the couplers need to:

- Provide a capacity that is 125% of the nominal bar capacity.
- Be epoxy coated.
- Satisfy fatigue testing requirements of NCHRP Project 10-35 (12 ksi).

5.1.4 Prestressing Steel

Uncoated low-relaxation 7-wire strand or uncoated deformed, high-strength bars are acceptable prestressing steels. Strands shall conform to ASTM A416. Bars shall conform to ASTM A722.

Use the following properties for prestressing steel:

Tensile strength:	f_{pu} = 270 ksi for strands
	f_{pu} = 250 ksi for bars
Yield strength:	f_{py} = 243 ksi for strands
	f_{py} = 120 ksi for bars
Elastic Modulus:	E_p = 28,500 ksi for strands
	E_p = 30,000 ksi for bars

Standard 7-wire prestressing strand area, A_{ps}:

$^3/_8$" diameter strand:	0.085 in^2/strand
$^1/_2$" diameter strand:	0.153 in^2/strand
0.6" diameter strand:	0.217 in^2/strand

5.1.5 Post-tensioning Hardware

For post-tensioned concrete bridges, open ducts must be used for tendon passageways through the superstructure. Longitudinal ducts are typically 3 to 4 inches in diameter and must be sufficiently rigid to withstand the loads imposed upon them. The preferred material for longitudinal ducts is corrugated plastic (HDPE). Transverse ducts are typically smaller, containing from 1 to 4 strands. Because the transverse ducts are relatively close to the top of the deck with heavy applications of corrosive de-icing chemicals, corrugated plastic ducts are required. The anchor head is typically galvanized or epoxy coated based on project needs. Discuss the protection requirements with the State Bridge Design Engineer.

Tendon anchorage devices are required at the ends of each duct. Anchorages should be shown and indicated on the drawings. Detailing is unnecessary because the post-tensioning supplier will provide these details in the shop drawings for the post-tensioning system. Designers must consider the local zone anchorage reinforcement (typically spiral reinforcement) provided by potential suppliers to allow adequate room for the general zone reinforcement designed and detailed in the bridge plans.

5.2 Reinforcement Details

Practices for detailing a variety of reinforced concrete elements are presented in this section. These include standard concrete cover and bar spacing dimensions, plus a variety of specific design and detailing instructions.

Reinforcing details are intended to provide a durable structure with straightforward details. Details must be constructible, allowing steel to be placed without undue effort, and provide adequate clear cover and adequate space between reinforcement to permit the placement of concrete.

5.2.1 Minimum Clear Cover and Clear Spacing

The minimum clear cover dimension to reinforcement varies with the location in the bridge. It varies with how the component is constructed (precast, cast in forms, cast against earth) and the exposure the element has to de-icing salts. In general, minimum covers increase as control over concrete placement decreases and as the anticipated exposure to de-icing salts increases. Following is a list of structural components and the corresponding minimum clear cover. For components that are not listed, a 2" minimum clear cover is required unless it is shown differently in the Bridge Office standards.

Foundations

Top Bars

- Minimum clear cover is 3 inches.

Bottom Bars, Spread Footing

- Minimum clear cover to the bottom concrete surface is 5 inches.
- Minimum clear cover to the side concrete surface is 3 inches.

Bottom Bars, Pile Cap w/ Pile Embedded 1 foot

- Rest directly on top of trimmed pile.

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

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

Abutments and Piers

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

Decks and Slabs

Top Bars, Roadway Bridge Deck or Slab

- Minimum clear cover to the top concrete surface is 3 inches.
- Minimum horizontal clear cover is 2 inches.

Top Bars, Pedestrian Bridge Deck

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

Bottom Bars, Deck

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

Bottom Bars, Slab

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

5.2.2 Reinforcing Bar Lists

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

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

A – 6 – 03 – E

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

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

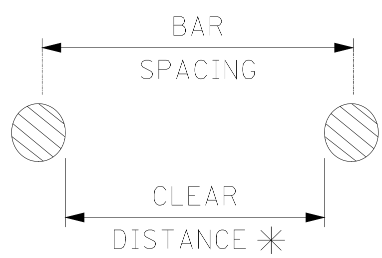
Table 5.2.2.1
Reinforcing Steel Sizes and Properties

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

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

Table 5.2.2.2
Average Area per Foot Width Provided by Various Bar Spacings (in^2/ft)

Bar Size Number	Nominal Diameter (in)	Spacing of Bars in Inches												
		3	3.5	4	4.5	5	5.5	6	7	8	9	10	11	12
3	0.375	0.44	0.38	0.33	0.29	0.26	0.24	0.22	0.19	0.17	0.15	0.13	0.12	0.11
4	0.500	0.80	0.69	0.60	0.53	0.48	0.44	0.40	0.34	0.30	0.27	0.24	0.22	0.20
5	0.625	1.24	1.06	0.93	0.83	0.74	0.68	0.62	0.53	0.47	0.41	0.37	0.34	0.31
6	0.750	1.76	1.51	1.32	1.17	1.06	0.96	0.88	0.75	0.66	0.59	0.53	0.48	0.44
7	0.875	2.40	2.06	1.80	1.60	1.44	1.31	1.20	1.03	0.90	0.80	0.72	0.65	0.60
8	1.000	3.16	2.71	2.37	2.11	1.90	1.72	1.58	1.35	1.19	1.05	0.95	0.86	0.79
9	1.128	4.00	3.43	3.00	2.67	2.40	2.18	2.00	1.71	1.50	1.33	1.20	1.09	1.00
10	1.270	---	4.35	3.81	3.39	3.05	2.77	2.54	2.18	1.91	1.69	1.52	1.39	1.27
11	1.410	---	---	4.68	4.16	3.74	3.40	3.12	2.67	2.34	2.08	1.87	1.70	1.56



* Per LRFD 5.10.3.1.1, the minimum clear distance between bars in a layer shall be the greatest of:

1) 1.5 times the nominal diameter of the bar
2) 1.5 times the maximum size of the coarse aggregate **
3) 1.5 inches

** Per the current edition of *MnDOT Standard Specifications for Construction*

The weight of spiral reinforcement on a per foot basis is provided in Table 5.2.2.3. The standard spiral reinforcement is $^1/_2$ inch diameter with a 3 inch pitch. When selecting the size of round columns, use outside dimensions that are consistent with cover requirements and standard spiral outside diameters.

Figure 5.2.2.1 through 5.2.2.5 contain development length (Class A lap) and tension lap splice design tables for epoxy coated, plain uncoated, and stainless steel reinforcement bars. Knowing the bar size, location, concrete cover, bar spacing, and class of splice, designers can readily find the appropriate lap length. The tables are based on 4 ksi concrete.

Figure 5.2.2.6 contains development length tables for bars with standard hooks. Values are provided for epoxy coated, plain uncoated, and stainless steel reinforcement bars. Standard hook dimensions are also included.

Figure 5.2.2.7 contains graphics that illustrate acceptable methods for anchoring or lapping stirrup reinforcement. Open stirrups must have the "open" end anchored in the compression side of the member. This anchorage consists of development of the bar or hook prior to reaching a depth of $^d/_2$ or placing the hooks around longitudinal reinforcement. Detail closed double stirrups with a Class B lap. Also included in Figure 5.2.2.7 are stirrup and tie hook dimensions and a table showing minimum horizontal bar spacings for various concrete mixes.

Table 5.2.2.3
Weight of Spiral Reinforcement

O.D. SPIRAL (in)	WEIGHTS IN POUNDS PER FOOT OF HEIGHT			
	$^3/_8$" DIA. ROD		$^1/_2$" DIA. ROD	
	6" PITCH (lb/ft)	F (lb)	3" PITCH (lb/ft)	F (lb)
24	4.72	7.1	16.79	12.60
26	5.12	7.7	18.19	13.65
28	5.51	8.3	19.59	14.70
30	5.91	8.9	20.99	15.75
32	6.30	9.5	22.38	16.80
34	6.69	10.1	23.78	17.85
36	7.09	10.7	25.18	18.90
38	7.48	11.2	26.58	20.00
40	7.87	11.8	27.98	21.00
42	8.27	12.4	29.38	22.00
44	8.66	13.0	30.78	23.10
46	9.06	13.6	32.18	24.10
48	9.45	14.2	33.58	25.20
50	9.84	14.8	34.98	26.20
52	10.24	15.4	36.38	27.30
54	10.63	15.9	37.77	28.30
56	11.02	16.5	39.17	29.40
58	11.42	17.1	40.57	30.40
60	11.81	17.7	41.97	31.50
62	12.21	18.3	43.37	32.50
64	12.60	18.9	44.77	33.60
66	12.99	19.5	46.17	34.60
68	13.39	20.1	47.57	35.70

For more complete coverage, see *CRSI Design Handbook*.

Total weight = (wt. per ft x height) + F

F = weight to add for finishing
(this includes $1^1/_2$ turns at the top and $1^1/_2$ turns at the bottom of spiral)

For additional information see MnDOT 2472 and AASHTO LRFD 5.10.6.2

TENSION LAP SPLICES FOR EPOXY COATED BARS WITH >12″ CONCRETE CAST BELOW

f_y=60 ksi f_c'=4 ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing															
		4″		5″		5 1/2″		6″		6 1/2″		7″		7 1/2″		≥ 8″	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
2"	**3**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**
	4	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	5	**2'-7"**	**3'-4"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**
	6	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"
	7	**3'-11"**	**5'-1"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**
	8	5'-2"	6'-8"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"
	9	**6'-6"**	**8'-6"**	**5'-3"**	**6'-9"**	**5'-1"**	**6'-7"**	**5'-1"**	**6'-7"**	**5'-1"**	**6'-7"**	**5'-1"**	**6'-7"**	**5'-1"**	**6'-7"**	**5'-1"**	**6'-7"**
	10	8'-3"	10'-9"	6'-7"	8'-7"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"
	11	**10'-2"**	**13'-3"**	**8'-2"**	**10'-7"**	**7'-6"**	**9'-9"**	**7'-6"**	**9'-9"**	**7'-6"**	**9'-9"**	**7'-6"**	**9'-9"**	**7'-6"**	**9'-9"**	**7'-6"**	**9'-9"**
	14	N/A	N/A	11'-9"	15'-3"	10'-8"	13'-10"	10'-4"	13'-5"	10'-4"	13'-5"	10'-4"	13'-5"	10'-4"	13'-5"	10'-4"	13'-5"
$2\frac{3"}{8}$	**3**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**
	4	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	5	**2'-7"**	**3'-4"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**
	6	3'-1"	4'-0"	3'-1"	4'-0"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"
	7	**3'-11"**	**5'-1"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**
	8	5'-2"	6'-8"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"
	9	**6'-6"**	**8'-6"**	**5'-3"**	**6'-9"**	**4'-9"**	**6'-2"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**
	10	8'-3"	10'-9"	6'-7"	8'-7"	6'-0"	7'-10"	5'-6"	7'-2"	5'-6"	7'-2"	5'-6"	7'-2"	5'-6"	7'-2"	5'-6"	7'-2"
	11	**10'-2"**	**13'-3"**	**8'-2"**	**10'-7"**	**7'-5"**	**9'-8"**	**6'-10"**	**8'-10"**	**6'-8"**	**8'-7"**	**6'-8"**	**8'-7"**	**6'-8"**	**8'-7"**	**6'-8"**	**8'-7"**
	14	N/A	N/A	11'-9"	15'-3"	10'-8"	13'-10"	9'-9"	12'-9"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"
≥ 3″	**3**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**	**1'-5"**	**1'-10"**
	4	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	5	**2'-7"**	**3'-4"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**	**2'-5"**	**3'-1"**
	6	3'-1"	4'-0"	3'-1"	4'-0"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"
	7	**3'-11"**	**5'-1"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-7"**	**4'-8"**	**3'-4"**	**4'-4"**	**3'-4"**	**4'-4"**	**3'-4"**	**4'-4"**	**3'-4"**	**4'-4"**
	8	5'-2"	6'-8"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"
	9	**6'-6"**	**8'-6"**	**5'-3"**	**6'-9"**	**4'-9"**	**6'-2"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**	**4'-8"**	**6'-0"**
	10	8'-3"	10'-9"	6'-7"	8'-7"	6'-0"	7'-10"	5'-6"	7'-2"	5'-3"	6'-9"	5'-3"	6'-9"	5'-3"	6'-9"	5'-3"	6'-9"
	11	**10'-2"**	**13'-3"**	**8'-2"**	**10'-7"**	**7'-5"**	**9'-8"**	**6'-10"**	**8'-10"**	**6'-3"**	**8'-2"**	**5'-10"**	**7'-7"**	**5'-10"**	**7'-6"**	**5'-10"**	**7'-6"**
	14	N/A	N/A	11'-9"	15'-3"	10'-8"	13'-10"	9'-9"	12'-9"	9'-0"	11'-9"	8'-5"	10'-11"	7'-10"	10'-2"	7'-8"	9'-11"

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete, and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of A_s spliced within required lap length	
$A_{s,\ provided}/A_{s,\ required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where: $A_{s,\ provided}$ = Area of reinforcement provided and $A_{s,\ required}$ = Area of reinforcement required by analysis

Figure 5.2.2.1
Reinforcement Data

TENSION LAP SPLICES FOR EPOXY COATED BARS WITH ≤ 12″ CONCRETE CAST BELOW

f_y=60 ksi f_c'=4 ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing															
		4″		5″		5 1/2″		6″		6 1/2″		7″		7 1/2″		≥ 8″	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
1″	3	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"
	4	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
	5	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"
	6	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"
	7	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"
	8	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"
	9	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"
	10	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"
	11	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"
	14	N/A	N/A	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"
$1\frac{1"}{2}$	3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
	4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	5	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	6	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
	7	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"
	8	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"
	9	5'-9"	7'-6"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"
	10	7'-4"	9'-6"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"
	11	9'-0"	11'-8"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"
	14	N/A	N/A	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"
2″	3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
	4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	5	2'-3"	3'-0"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
	6	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
	7	3'-6"	4'-6"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"
	8	4'-6"	5'-11"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"
	9	5'-9"	7'-6"	4'-7"	6'-0"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"
	10	7'-4"	9'-6"	5'-10"	7'-7"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"
	11	9'-0"	11'-8"	7'-2"	9'-4"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"
	14	N/A	N/A	10'-4"	13'-5"	9'-5"	12'-3"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"
$2\frac{3"}{8}$	3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
	4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	5	2'-3"	3'-0"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
	6	2'-9"	3'-7"	2'-9"	3'-7"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"
	7	3'-6"	4'-6"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"
	8	4'-6"	5'-11"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"
	9	5'-9"	7'-6"	4'-7"	6'-0"	4'-2"	5'-5"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"
	10	7'-4"	9'-6"	5'-10"	7'-7"	5'-4"	6'-11"	4'-11"	6'-4"	4'-10"	6'-4"	4'-10"	6'-4"	4'-10"	6'-4"	4'-10"	6'-4"
	11	9'-0"	11'-8"	7'-2"	9'-4"	6'-7"	8'-6"	6'-0"	7'-10"	5'-10"	7'-7"	5'-10"	7'-7"	5'-10"	7'-7"	5'-10"	7'-7"
	14	N/A	N/A	10'-4"	13'-5"	9'-5"	12'-3"	8'-8"	11'-3"	8'-1"	10'-5"	8'-1"	10'-5"	8'-1"	10'-5"	8'-1"	10'-5"
≥ 3″	3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
	4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	5	2'-3"	3'-0"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
	6	2'-9"	3'-7"	2'-9"	3'-7"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"
	7	3'-6"	4'-6"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	2'-7"	3'-4"	2'-7"	3'-4"	2'-7"	3'-4"	2'-7"	3'-4"
	8	4'-6"	5'-11"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	2'-11"	3'-9"	2'-11"	3'-9"	2'-11"	3'-9"
	9	5'-9"	7'-6"	4'-7"	6'-0"	4'-2"	5'-5"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"
	10	7'-4"	9'-6"	5'-10"	7'-7"	5'-4"	6'-11"	4'-11"	6'-4"	4'-7"	6'-0"	4'-7"	6'-0"	4'-7"	6'-0"	4'-7"	6'-0"
	11	9'-0"	11'-8"	7'-2"	9'-4"	6'-7"	8'-6"	6'-0"	7'-10"	5'-7"	7'-2"	5'-2"	6'-8"	5'-1"	6'-8"	5'-1"	6'-8"
	14	N/A	N/A	10'-4"	13'-5"	9'-5"	12'-3"	8'-8"	11'-3"	8'-0"	10'-4"	7'-5"	9'-7"	6'-11"	9'-0"	6'-9"	8'-9"

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete, and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of A_s spliced within required lap length	
$A_{s,\ provided}/A_{s,\ required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where: $A_{s,\ provided}$ = Area of reinforcement provided and $A_{s,\ required}$ = Area of reinforcement required by analysis

Figure 5.2.2.2
Reinforcement Data

TENSION LAP SPLICES FOR PLAIN UNCOATED BARS WITH >12" CONCRETE CAST BELOW

f_y=60 ksi f_c'=4 ksi

Conc. Cover	Bar Size	Reinforcement Spacing															
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
2"	3	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"
	4	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"
	5	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
	6	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"
	7	3'-0"	3'-11"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
	8	3'-11"	5'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"
	9	5'-0"	6'-6"	4'-0"	5'-2"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"
	10	6'-4"	8'-3"	5'-1"	6'-7"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"
	11	7'-10"	10'-1"	6'-3"	8'-1"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"
	14	N/A	N/A	9'-0"	11'-8"	8'-2"	10'-7"	7'-11"	10'-3"	7'-11"	10'-3"	7'-11"	10'-3"	7'-11"	10'-3"	7'-11"	10'-3"
≥ 3"	3	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"
	4	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"
	5	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
	6	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"
	7	3'-0"	3'-11"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
	8	3'-11"	5'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"
	9	5'-0"	6'-6"	4'-0"	5'-2"	3'-8"	4'-9"	3'-7"	4'-7"	3'-7"	4'-7"	3'-7"	4'-7"	3'-7"	4'-7"	3'-7"	4'-7"
	10	6'-4"	8'-3"	5'-1"	6'-7"	4'-7"	6'-0"	4'-3"	5'-6"	4'-0"	5'-2"	4'-0"	5'-2"	4'-0"	5'-2"	4'-0"	5'-2"
	11	7'-10"	10'-1"	6'-3"	8'-1"	5'-8"	7'-4"	5'-3"	6'-9"	4'-10"	6'-3"	4'-6"	5'-10"	4'-5"	5'-9"	4'-5"	5'-9"
	14	N/A	N/A	9'-0"	11'-8"	8'-2"	10'-7"	7'-6"	9'-9"	6'-11"	9'-0"	6'-5"	8'-4"	6'-0"	7'-10"	5'-10"	7'-7"

TENSION LAP SPLICES FOR PLAIN UNCOATED BARS WITH ≤ 12" CONCRETE CAST BELOW

f_y=60 ksi f_c'=4 ksi

Conc. Cover	Bar Size	Reinforcement Spacing															
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
2"	3	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"
	4	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"
	5	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	6	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
	7	2'-4"	3'-0"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"
	8	3'-0"	3'-11"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"
	9	3'-10"	5'-0"	3'-1"	4'-0"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	10	4'-11"	6'-4"	3'-11"	5'-1"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"
	11	6'-0"	7'-10"	4'-10"	6'-3"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"
	14	N/A	N/A	6'-11"	9'-0"	6'-4"	8'-2"	6'-1"	7'-11"	6'-1"	7'-11"	6'-1"	7'-11"	6'-1"	7'-11"	6'-1"	7'-11"
≥ 3"	3	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"
	4	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"
	5	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	6	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
	7	2'-4"	3'-0"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"
	8	3'-0"	3'-11"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"
	9	3'-10"	5'-0"	3'-1"	4'-0"	2'-10"	3'-8"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
	10	4'-11"	6'-4"	3'-11"	5'-1"	3'-7"	4'-7"	3'-3"	4'-3"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"
	11	6'-0"	7'-10"	4'-10"	6'-3"	4'-5"	5'-8"	4'-0"	5'-3"	3'-9"	4'-10"	3'-5"	4'-6"	3'-5"	4'-5"	3'-5"	4'-5"
	14	N/A	N/A	6'-11"	9'-0"	6'-4"	8'-2"	5'-9"	7'-6"	5'-4"	6'-11"	4'-11"	6'-5"	4'-8"	6'-0"	4'-6"	5'-10"

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of A_s spliced within required lap length	
$A_{s,\ provided}/A_{s,\ required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where: $A_{s,\ provided}$ = Area of reinforcement provided and $A_{s,\ required}$ = Area of reinforcement required by analysis

Figure 5.2.2.3

Reinforcement Data

TENSION LAP SPLICES FOR STAINLESS STEEL BARS WITH >12″ CONCRETE CAST BELOW

f_y=75 ksi f_c'=4 ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing															
		4″		5″		5 1/2″		6″		6 1/2″		7″		7 1/2″		≥ 8″	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
2″	3	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	4	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
	5	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"
	6	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"
	7	3'-9"	4'-11"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"
	8	4'-11"	6'-5"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"
	9	6'-3"	8'-1"	5'-0"	6'-6"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"
	10	7'-11"	10'-3"	6'-4"	8'-3"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"
	11	9'-9"	12'-8"	7'-10"	10'-1"	7'-2"	9'-4"	7'-2"	9'-4"	7'-2"	9'-4"	7'-2"	9'-4"	7'-2"	9'-4"	7'-2"	9'-4"
	14	N/A	N/A	11'-3"	14'-7"	10'-2"	13'-3"	9'-10"	12'-10"	9'-10"	12'-10"	9'-10"	12'-10"	9'-10"	12'-10"	9'-10"	12'-10"
$2\frac{3"}{8}$	3	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	4	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
	5	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"
	6	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"
	7	3'-9"	4'-11"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"
	8	4'-11"	6'-5"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"
	9	6'-3"	8'-1"	5'-0"	6'-6"	4'-7"	5'-11"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"
	10	7'-11"	10'-3"	6'-4"	8'-3"	5'-9"	7'-6"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"
	11	9'-9"	12'-8"	7'-10"	10'-1"	7'-1"	9'-2"	6'-6"	8'-5"	6'-4"	8'-3"	6'-4"	8'-3"	6'-4"	8'-3"	6'-4"	8'-3"
	14	N/A	N/A	11'-3"	14'-7"	10'-2"	13'-3"	9'-4"	12'-2"	8'-9"	11'-4"	8'-9"	11'-4"	8'-9"	11'-4"	8'-9"	11'-4"
≥ 3″	3	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
	4	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
	5	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"
	6	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"
	7	3'-9"	4'-11"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"
	8	4'-11"	6'-5"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"
	9	6'-3"	8'-1"	5'-0"	6'-6"	4'-7"	5'-11"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"
	10	7'-11"	10'-3"	6'-4"	8'-3"	5'-9"	7'-6"	5'-3"	6'-10"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"
	11	9'-9"	12'-8"	7'-10"	10'-1"	7'-1"	9'-2"	6'-6"	8'-5"	6'-0"	7'-10"	5'-7"	7'-3"	5'-6"	7'-2"	5'-6"	7'-2"
	14	N/A	N/A	11'-3"	14'-7"	10'-2"	13'-3"	9'-4"	12'-2"	8'-8"	11'-3"	8'-0"	10'-5"	7'-6"	9'-9"	7'-4"	9'-6"

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of A_s spliced within required lap length	
$A_{s,\ provided}/A_{s,\ required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where: $A_{s,\ provided}$ = Area of reinforcement provided and $A_{s,\ required}$ = Area of reinforcement required by analysis

Figure 5.2.2.4
Reinforcement Data

TENSION LAP SPLICES FOR STAINLESS STEEL BARS WITH ≤ 12″ CONCRETE CAST BELOW

f_y=75 ksi f_c'=4 ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing															
		4″		5″		5 1/2″		6″		6 1/2″		7″		7 1/2″		≥ 8″	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
1″	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"
	6	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"
	7	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"
	8	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"
	9	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"
	10	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"
	11	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"
	14	N/A	N/A	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"
$1\frac{1}{2}$"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	8	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"
	9	4'-10"	6'-3"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"
	10	6'-1"	7'-11"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"
	11	7'-6"	9'-9"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"
	14	N/A	N/A	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"
2″	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	2'-11"	3'-9"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"
	8	3'-9"	4'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	9	4'-10"	6'-3"	3'-10"	5'-0"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"
	10	6'-1"	7'-11"	4'-11"	6'-4"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"
	11	7'-6"	9'-9"	6'-0"	7'-10"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"
	14	N/A	N/A	8'-8"	11'-3"	7'-10"	10'-2"	7'-7"	9'-10"	7'-7"	9'-10"	7'-7"	9'-10"	7'-7"	9'-10"	7'-7"	9'-10"
$2\frac{3}{8}$"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	2'-11"	3'-9"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"
	8	3'-9"	4'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	9	4'-10"	6'-3"	3'-10"	5'-0"	3'-6"	4'-7"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"
	10	6'-1"	7'-11"	4'-11"	6'-4"	4'-5"	5'-9"	4'-1"	5'-3"	4'-1"	5'-3"	4'-1"	5'-3"	4'-1"	5'-3"	4'-1"	5'-3"
	11	7'-6"	9'-9"	6'-0"	7'-10"	5'-6"	7'-1"	5'-0"	6'-6"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"
	14	N/A	N/A	8'-8"	11'-3"	7'-10"	10'-2"	7'-2"	9'-4"	6'-9"	8'-9"	6'-9"	8'-9"	6'-9"	8'-9"	6'-9"	8'-9"
≥ 3″	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	2'-11"	3'-9"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"
	8	3'-9"	4'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	9	4'-10"	6'-3"	3'-10"	5'-0"	3'-6"	4'-7"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"
	10	6'-1"	7'-11"	4'-11"	6'-4"	4'-5"	5'-9"	4'-1"	5'-3"	3'-10"	5'-0"	3'-10"	5'-0"	3'-10"	5'-0"	3'-10"	5'-0"
	11	7'-6"	9'-9"	6'-0"	7'-10"	5'-6"	7'-1"	5'-0"	6'-6"	4'-8"	6'-0"	4'-4"	5'-7"	4'-3"	5'-6"	4'-3"	5'-6"
	14	N/A	N/A	8'-8"	11'-3"	7'-10"	10'-2"	7'-2"	9'-4"	6'-8"	8'-8"	6'-2"	8'-0"	5'-9"	7'-6"	5'-8"	7'-4"

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of A_s spliced within required lap length	
$A_{s,\ provided}/A_{s,\ required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where: $A_{s,\ provided}$ = Area of reinforcement provided and $A_{s,\ required}$ = Area of reinforcement required by analysis

Figure 5.2.2.5
Reinforcement Data

DEVELOPMENT LENGTH FOR STANDARD HOOKS IN TENSION

For plain and epoxy bars, f_y = 60 ksi f_c'= 4ksi
For stainless steel bars, f_y = 75 ksi

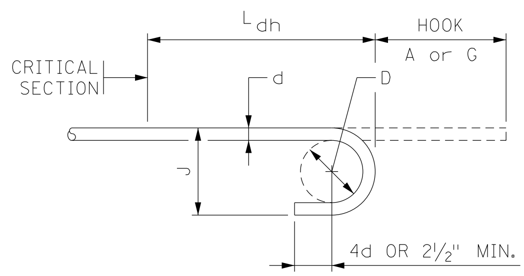
Hooked Bar Development Length, L_{dh}
When: Side Cover ≥ 2.5" AND
For 90° Hooks, Concrete Cover ≥ 2"
in the direction of bar extension

Bar Size	Plain Uncoated Bars	Epoxy Coated Bars	Stainless Steel Bars
3	6"	7"	8″
4	8"	10"	10″
5	10"	1'-0"	1′-0″
6	1'-0"	1'-2"	1'-3"
7	1'-2"	1'-4"	1'-5"
8	1'-4"	1'-7"	1'-7"
9	1'-6"	1'-9"	1'-10"
10	1'-8"	2'-0"	2'-1"
11	1'-10"	2'-2"	2'-3"
14	2'-9"	3'-3"	3'-5"

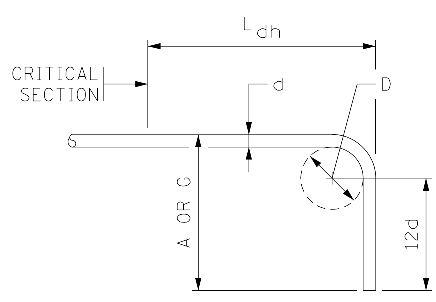
Hooked Bar Development Length, L_{dh}
When: Side Cover < 2.5" OR
For 90° Hooks, Concrete Cover < 2"
in the direction of bar extension

Bar Size	Plain Uncoated Bars	Epoxy Coated Bars	Stainless Steel Bars
3	8"	9"	9"
4	10"	1'-0"	1'-0"
5	1'-0"	1'-3"	1'-3"
6	1'-3"	1'-6"	1'-6"
7	1'-5"	1'-8"	1'-9"
8	1'-7"	1'-11"	2'-0"
9	1'-10"	2'-2"	2'-3"
10	2'-1"	2'-5"	2'-7"
11	2'-3"	2'-9"	2'-10"
14	2'-9"	3'-3"	3'-5"

Tables include modification factors per LRFD Art. 5.11.2.4.2 for normal weight concrete, bar coating, and reinforcement confinement. The reinforcement confinement factor is not applicable to bars larger than No. 11 bars. Note that MnDOT allows use of No. 14 bar standard hooks for concrete strengths up to 10 ksi.



180° BEND

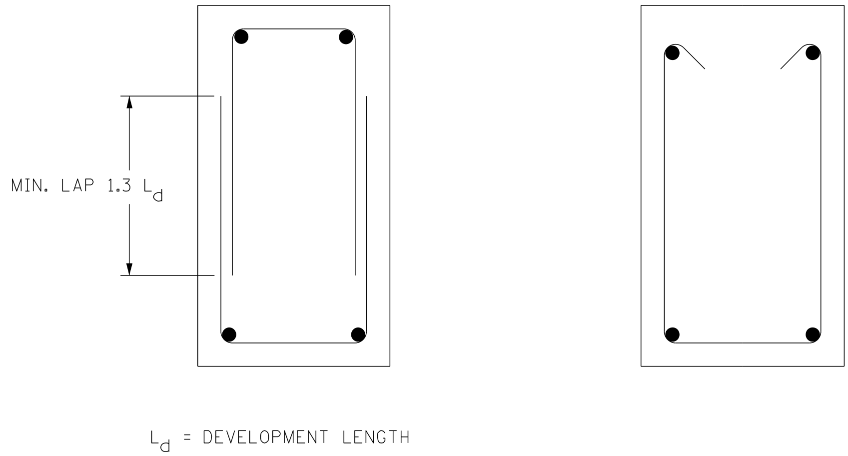


90° BEND

STANDARD HOOK DIMENSIONS

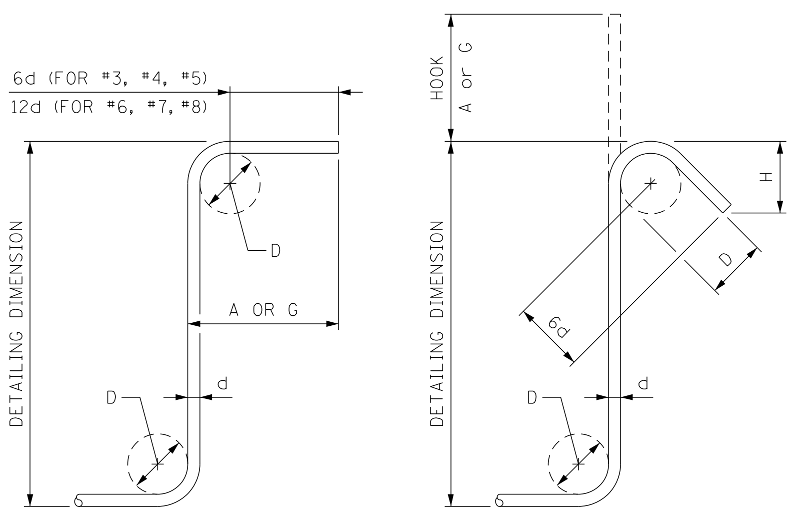
BAR SIZE	D	180° HOOKS		90° HOOKS
		A OR G	J	A OR G
3	2¼"	5"	3"	6"
4	3"	6"	4"	8"
5	3¾"	7"	5"	10"
6	4½"	8"	6"	1'-0"
7	5¼"	10"	7"	1'-2"
8	6"	11"	8"	1'-4"
9	9½"	1'-3"	11¾"	1'-7"
10	10¾"	1'-5"	1'-1¼"	1'-10"
11	12"	1'-7"	1'-2¾"	2'-0"
14	18¼"	2'-3"	1'-9¾"	2'-7"

Figure 5.2.2.6
Reinforcement Data



METHODS FOR ANCHORAGE OF SHEAR REINFORCEMENT

SEE AASHTO LRFD 5.11.2.6.2 AND 5.11.2.6.4



90° BEND

135° BEND

STIRRUP AND TIE HOOK DIMENSIONS				
BAR SIZE	D	90° HOOKS	135° HOOKS	
		A OR G	A OR G	H *
3	1½"	4"	4"	2½"
4	2"	4½"	4½"	3"
5	2½"	6"	5½"	3¾"
6	4½"	1'-0"	8"	4½"
7	5¼"	1'-2"	9"	5¼"
8	6"	1'-4"	10½"	6"

MINIMUM HORIZONTAL BAR SPACING (℄ TO ℄)		
BAR SIZE	CONCRETE MIX: 1G52 AND 3B52	CONCRETE MIX: 3JM, 3YHPC-M, 3YHPC-S, 3YLCHPC-M, 3YLCHPC-S, 3Y42, 3S52, 3Y82 AND 3W82
3	2⅝"	1⅞"
4	2¾"	2"
5	2⅞"	2⅛"
6	3"	2¼"
7	3⅛"	2⅜"
8	3¼"	2½"
9	3½"	2⅞"
10	3⅝"	3¼"
11	3¾"	3⅝"
14	4¼"	4¼"

SEE AASHTO LRFD 5.10.3.1

NOTE: MINIMUM HORIZONTAL BAR SPACING SHALL ALSO APPLY TO THE DISTANCE FROM A CONTACT LAP SPLICE TO ADJACENT SPLICES OR BARS.

Figure 5.2.2.7
Reinforcement Data

5.2.3 General Reinforcement Practices

Reinforcement practices follow those shown by the Concrete Reinforcing Steel Institute (C.R.S.I.) in the Manual of Standard Practice. These practices include:

1) For bent bars, omit the last length dimension on reinforcement bar details.
2) Use standard length bars for all but the last bar in long bar runs.
3) Use a maximum length of 60 feet for #4 deck or slab bars and 40 feet for other applications.
4) Use a maximum length of 60 feet for bars #5 and larger.
5) Recognize that bar cutting and bending tolerances are ±1 inch for bars and that this tolerance is important for long straight bars that do not have lap splices to provide dimensional flexibility. Refer to MnDOT document *Suggested Reinforcement Detailing Practices*, which is located at http://www.dot.state.mn.us/bridge/standards.html, for more guidance on rebar detailing to account for tolerances.
6) Reinforcement bars longer than 60 feet or larger than #11 are available only on special order, and should be avoided. Designers should check with the State Bridge Design Engineer before using special order sizes or lengths.

5.2.4 Reinforcement Bar Couplers

Reinforcement bar couplers are expensive compared to conventional lap splices. Where lap splices cannot be readily used (bridge widening projects, staged construction, large river pier longitudinal bars—anywhere that the available space for a rebar projection is limited), couplers should be considered. Where possible, stagger reinforcement bar couplers in order to distribute the stiffness of the couplers. There are numerous coupler types and sizes. For members that require couplers, consider the coupler outside diameter and length when detailing reinforcement, in order to avoid congestion and clear cover issues.

5.2.5 Adhesive Anchors

Similar to bar couplers, adhesive anchors are expensive. Adhesive anchors are typically used to attach secondary structural members to new concrete or primary structural members to existing (old) concrete. A typical use is to attach a metal rail to a concrete base.

See Article 13.3.2 of this manual for an adhesive anchor design example.

Adhesive anchors shall not be used for constant tension applications.

5.2.6 Shrinkage and Temperature Reinforcement [5.10.8]

Follow the requirements for shrinkage and temperature reinforcement given in LRFD 5.10.8. An exception to this is that shrinkage and temperature reinforcement is not required in buried footings of typical bridges.

5.3 Concrete Slabs

In many bridge engineering documents the terms "concrete slab" and "concrete deck" are used interchangeably. Within this manual, "concrete slab" will refer to a superstructure type without supporting beam elements. In most cases, the primary reinforcement for slabs is parallel to the centerline of roadway. Likewise, within this manual "concrete decks" will refer to the superstructure element placed on top of beams or girders. In most cases, the primary reinforcement for a concrete deck is transverse to the centerline of roadway. Practices for concrete decks are described in Section 9 of this manual.

5.3.1 Geometry

The maximum span lengths for concrete slabs are as follows:

Number of Spans	Without Haunches	With Haunches
1	30 ft	40 ft
2	40 ft	50 ft
3 or 4	50 ft	60 ft

End spans should be approximately 80% of the center span length to balance moments and prevent uplift.

LRFD Table 2.5.2.6.3-1 provides guidance for recommended minimum structure depth as a function of span length for slab superstructures without haunches.

When haunches are required, use linear haunches in accordance with the following:

Minimum slab depth at pier = $1.33 \cdot \left[\frac{S+10}{30}\right]$

(includes wear course if present)

Minimum slab depth in non-haunched area = $0.8 \cdot \left[\frac{S+10}{30}\right]$
(includes wear course if present)

Minimum haunch length L = $0.15 \cdot S$
(where S is the length of longest span)

5.3.2 Design/Analysis

Skew can be ignored for slab bridges with skew angles of 20° or less.

- Place transverse reinforcement parallel to substructures.

For slab bridges with skew angles between 20° and 45°

- Perform a two-dimensional plate analysis.
- Place transverse reinforcement normal to the bridge centerline.

Slab type bridges are not allowed for bridges with skew angles greater than 45°.

Slab bridges curved in plan may be designed as if straight. Designers should consider and investigate the need for providing additional reinforcement in the portion of the slab outside of chord lines connecting substructure units.

Do not include the concrete wearing course in section properties when performing strength and serviceability checks. This will ensure that the slab has adequate capacity if traffic is carried on the bridge during operations associated with milling off the old wearing course and the placement of a new wearing course. An exception to this is when checking the top reinforcement in the negative moment region for flexural crack control.

[5.7.3.4]

When checking crack control for slabs and decks, use the Class 2 exposure condition (γ_e = 0.75). Although the actual clear cover may exceed 2 inches for the slab/deck top bars, calculate d_c using a maximum clear concrete cover equal to 2 inches.

Determine reinforcement bar cutoff points based on strength, serviceability, and minimum reinforcement requirements.

[5.14.4.1]

Although not required by AASHTO, MnDOT requires a check of one-way shear in slab bridges. For determination of the live load distribution factor for shear, assume that the live load is distributed over the entire width of the superstructure. Load all lanes and use the appropriate multiple presence factor. For determination of factored shear resistance, use $\beta = 2.0$. If shear reinforcement is needed, try thickening the slab to eliminate it. If shear reinforcement must be used, calculate the appropriate β and σ values using LRFD Article 5.8.3.4.2.

5.3.3 Exterior Strip [4.6.2.1.4b]

Outside edges of slab bridges contain the exterior strip or edge beam. At a minimum, the exterior strip reinforcement must match that of the interior portions of the bridge.

Special consideration for the design of edge beams is required for bridges with sidewalks. Separately poured sidewalks may be considered to act compositely with the slab when adequate means of shear transfer at the interface is provided.

5.3.4 Reinforcement Layout

Use the following guidelines for layout of reinforcement in a simple span slab bridge (see example in Figure 5.3.4.1):

Interior strip reinforcement
- Top longitudinal – 1 spacing, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Exterior strip reinforcement
- Top longitudinal – 1 spacing, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Transverse reinforcement – 1 spacing, 1 bar size

Use the following guidelines for layout of reinforcement in a continuous slab bridge:

Option 1 (see example in Figure 5.3.4.2):

Interior strip reinforcement
- Top longitudinal – 2 spacings, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Exterior strip reinforcement
- Top longitudinal – 2 spacings, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Transverse reinforcement – 1 spacing, 1 bar size

Option 2 (see example in Figure 5.3.4.3):

Interior strip reinforcement
- Top longitudinal – 2 spacings, 2 bar sizes
- Bottom longitudinal – 2 spacings, 2 bar sizes

Exterior strip reinforcement
- Top longitudinal – 2 spacings, 2 bar sizes
- Bottom longitudinal – 2 spacings, 2 bar sizes

Transverse reinforcement - 1 spacing, 1 bar size

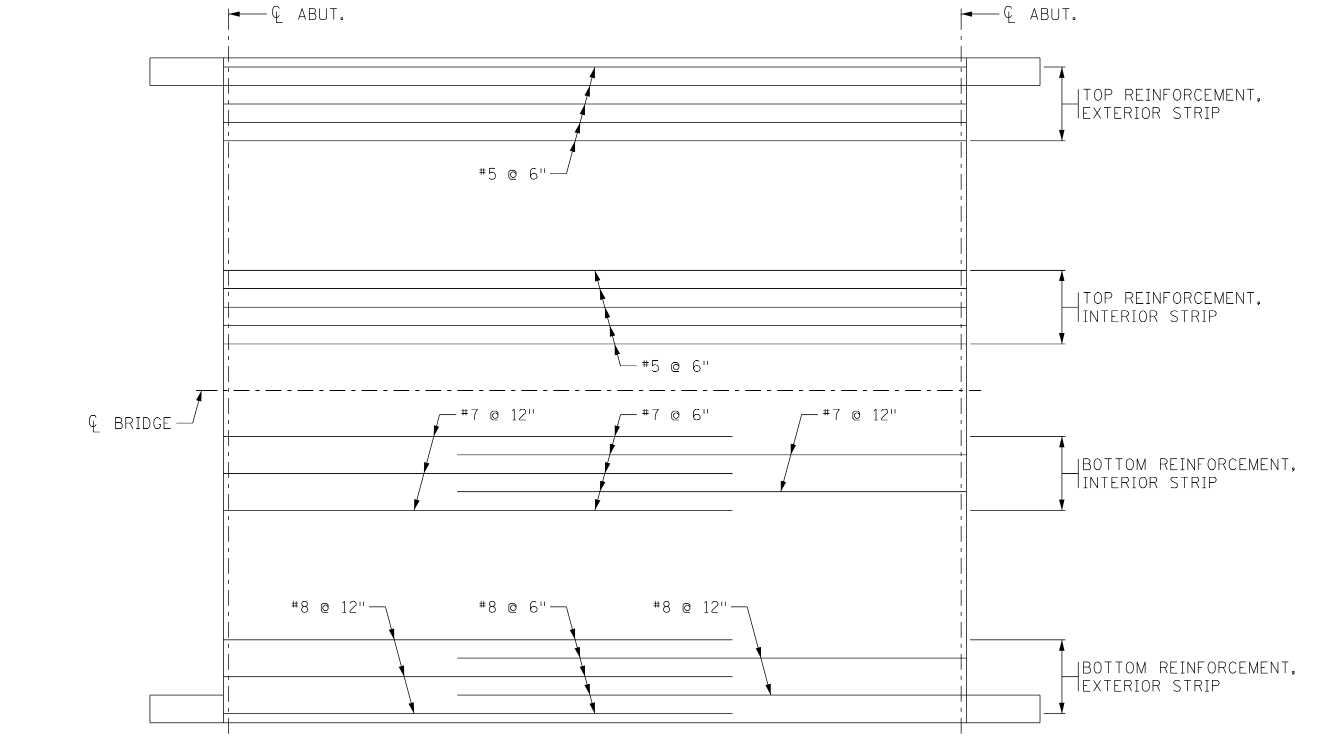


Figure 5.3.4.1

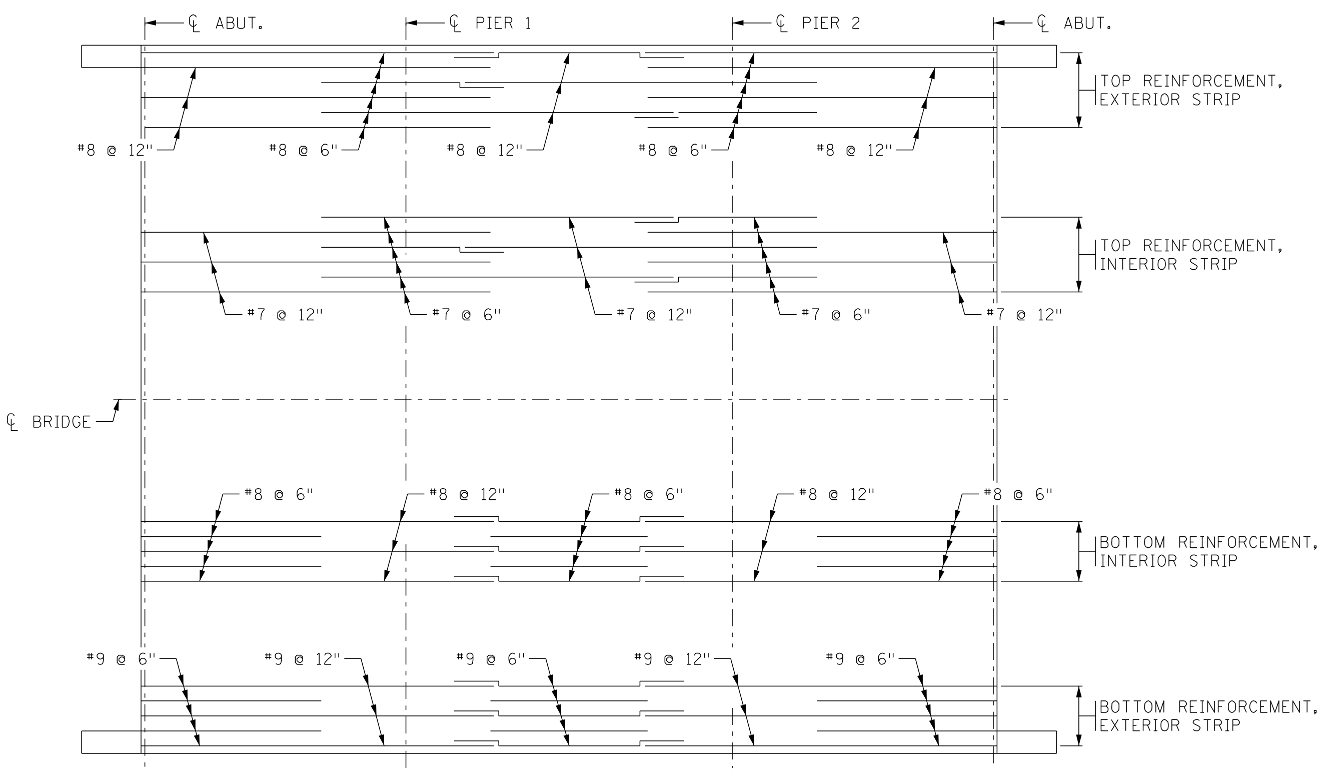


Figure 5.3.4.2

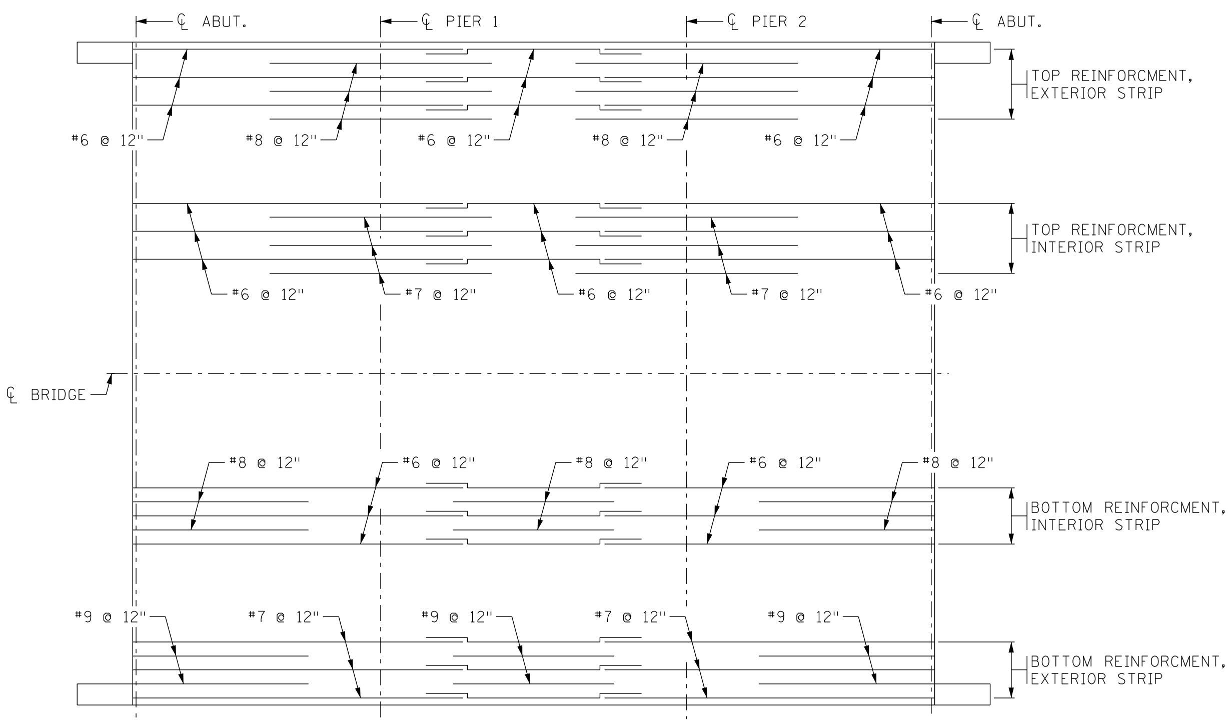


Figure 5.3.4.3

To simplify placement, detail reinforcement such that top bars are positioned over bottom bars where possible. For example, if the design requires bottom longitudinal bars spaced on 10 inch centers, top longitudinal bars might be spaced on 10 inch centers in positive moment regions and 5 inch centers in negative moment regions.

Extend railing dowel bars to the bottom layer of slab reinforcement and provide a horizontal leg for ease of placement.

5.3.5 Camber and Deflections

[5.7.3.6.2]

In order to obtain the best rideability over the life of the structure, camber concrete slab bridges for the immediate dead load deflection plus one half of the long-term deflection. Use gross section properties for dead load deflection calculations and a long-term creep multiplier of 4.0.

Railings, sidewalks, medians, and wearing courses are not placed while the slab is supported on falsework. Assume that only the slab carries the dead load of these elements.

Check live load deflections using the effective moment of inertia. The effective moment of inertia may be approximated as one half of the gross moment of inertia. The maximum live load deflection is $^{L}/_{800}$ for vehicular bridges that do not carry pedestrians and $^{L}/_{1000}$ for vehicular bridges that carry pedestrians.

Consider the concrete wearing course to be functioning compositely with the slab for live load deflection. Assume the riding surface has lost $^{1}/_{2}$ inch of thickness due to wear.

Use a live load distribution factor equal to the number of lanes times the multiple presence factor and divide by the width of the slab for the deflection check.

5.4 Pretensioned Concrete

The details of pretensioned concrete beams are presented on standard Bridge Details Part II sheets incorporated into a set of plans. Prepare a separate sheet for each type of beam in the project. Beams are identical if they have the same cross-section, strand layout, concrete strengths, and a similar length. To simplify fabrication and construction, try to minimize the number of beam types incorporated into a project. Design exterior beams with a strength equal to or greater than the interior beams.

5.4.1 Geometry

Provide a minimum stool along centerline of beam that is based on $1^{1}/_{2}$ inches of minimum stool at edge of flange. For dead load computations assume an average stool height equal to the minimum stool height plus 1 inch. Deck cross slopes, horizontal curves, and vertical curves all impact the stool height.

There are several Bridge Office practices regarding the type and location of diaphragms or cross frames for prestressed beam bridges:

1) Design prestressed I-beam bridges without continuity over the piers, except in the following situations:
    a) Bridge is over water with pile bent piers supported by unstable soils such as fat clay.
    b) Bridge is over water with pile bent piers at risk for large ice or debris loading and pier does not have an encasement wall.
2) Intermediate diaphragms are not required for 14RB, 18RB, 22RB, and 27M beams. For all other beam sizes, the following applies. Intermediate diaphragms are not required for single spans of 45'-0" or less. Provide one diaphragm per every 45 feet of span length, spaced evenly along the span as stated in Table 5.4.1.1.

Table 5.4.1.1

Span length (ft)	Base number of intermediate diaphragms
Less than 45′-0″	0
45′-0″ to 90′-0″	1 located at midspan
90′-0″ to 135′-0″	2 located at the third points
135′-0″ to 180′-0″	3 located at the quarter points
Greater than 180′-0″	4 plus an additional diaphragm for each additional 45 ft of span length greater than 180′-0″

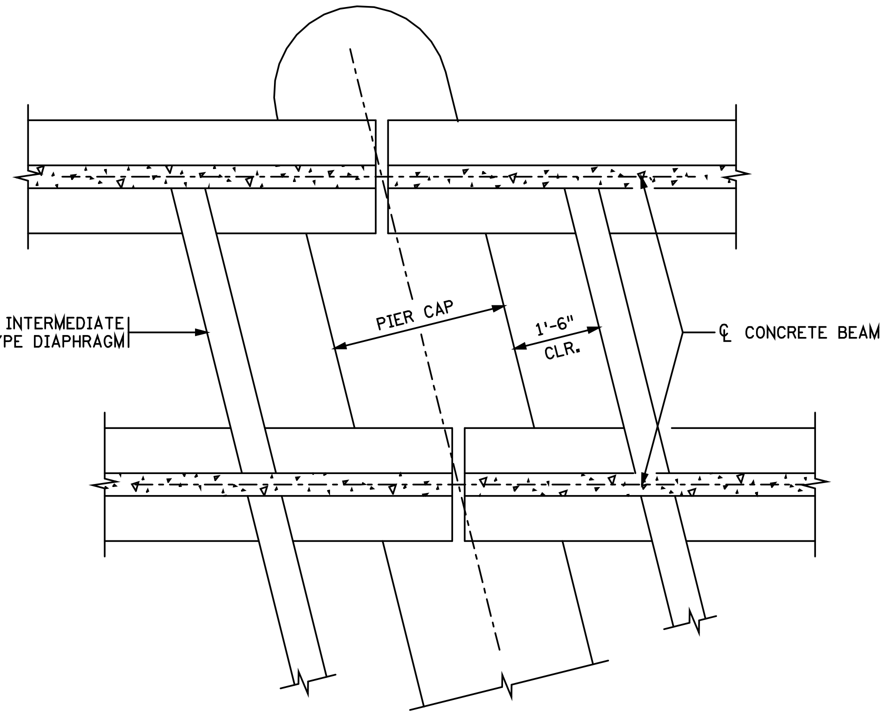
For spans over traffic, place additional diaphragms in the fascia bay approached by traffic to provide bracing against impact from over-height traffic loads. For two-lane roadways, place one diaphragm approximately over each shoulder. For additional lanes, space additional diaphragms at intervals of about 25'-0" over the roadway.

3) Figure 5.4.1.1 illustrates the typical layout of intermediate diaphragms at piers for bridges without continuity over the piers.

Locate the centerline of bearing $7^{1}/_{2}$ inches from the end of the beam for RB, M, and MN shapes. Locate the centerline of bearing $8^{1}/_{2}$ inches from the end of the beam for MW shapes. For MW shapes, this dimension can be adjusted if used with higher movement bearings, as opposed to the typical curved plate elastomeric bearings shown in Section 14 of this manual. However, if the $8^{1}/_{2}$ inch dimension is exceeded, a special design for the bearing, sole plate, and beam end region must be completed.

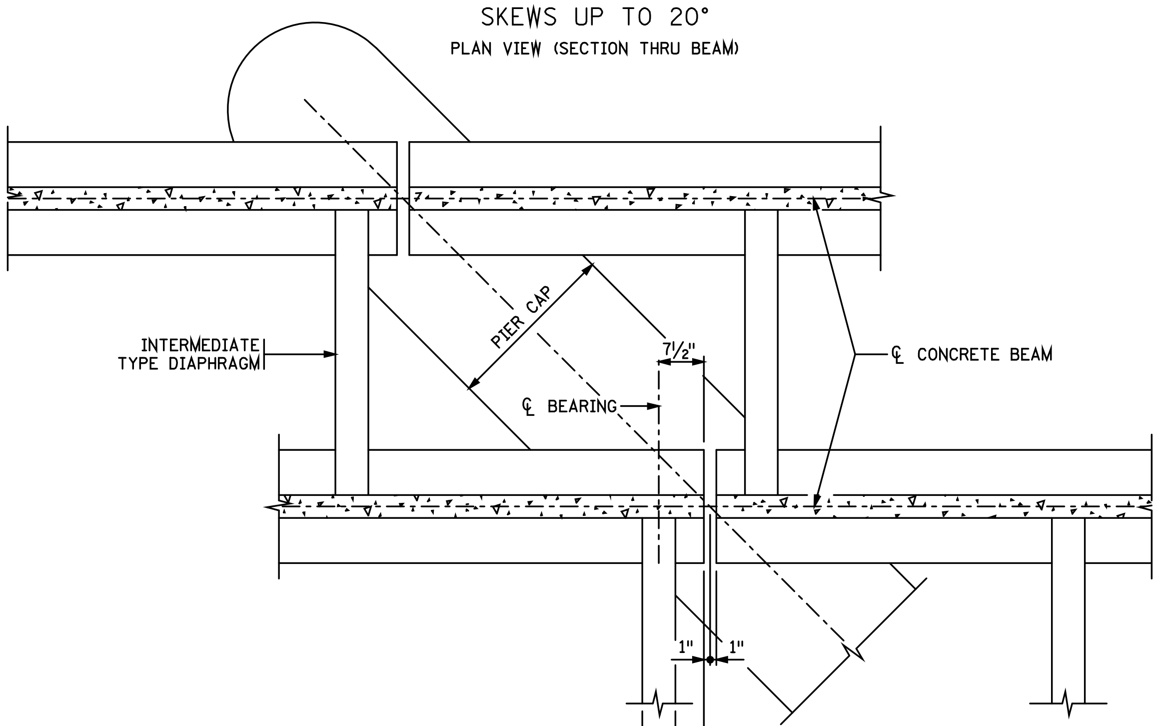
At piers of two span bridges, provide 2 inches of clearance between the ends of RB, M, and MN beams. Provide 3 inches clearance for structures with three or more spans. Provide 4 inches of clearance between the ends of MW beams regardless of the number of spans. Note that the fabrication length tolerance for pretensioned I-beams is $\pm^{1}/_{8}$" per 10 feet of length. It may be necessary to cope beam flanges at piers for bridges with tight horizontal curves or at skewed abutments.

For bridges on significant grades $(\geq 3\%)$ the sloped length of the beam will be significantly longer than the horizontal length between substructure units. If the sloped length is $^{1}/_{2}$ inch or more than the horizontal length, identify the sloped length dimension on the beam detail plan sheets.



SKEWS UP TO 20°

PLAN VIEW (SECTION THRU BEAM)



SKEW OVER 20°

PLAN VIEW (SECTION THRU BEAM)

Figure 5.4.1.1

Typical Diaphragm Layout at Piers for Prestressed Concrete Beam Bridge With Continuous Deck Over Piers

5.4.2 Stress Limits [5.9.3] [5.9.4]

Similar to the Standard Specifications, the LRFD Specifications identify service load stress limits for different elements and locations.

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

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

$$0.0948 \cdot \sqrt{f'_{ci}} \leq 0.2 \text{ ksi} \qquad (\text{where } f'_{ci} \text{ is in ksi})$$

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

$$0.19 \cdot \sqrt{f'_{c}} \qquad (\text{where } f'_{c} \text{ is in ksi})$$

5.4.3 Design/Analysis

Use the "approximate method" provided in LRFD Article 5.9.5.3 to compute prestress losses.

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

At the time of prestress transfer (initial), the minimum required concrete strength (f'_{ci}) is 4.5 ksi and the maximum is limited to 7.5 ksi. At the termination of the curing period (final), the minimum concrete strength (f'_{c}) is 5 ksi and the maximum strength is 9 ksi. Higher initial or final strengths may be used with approval from the Bridge Design Engineer. An initial concrete strength greater than 7 ksi may add cost to the beam. The fabricator cannot remove the beam from the bed until a cylinder break indicates the concrete has reached its specified initial strength. With strengths higher than 7 ksi, the fabricator may have to leave the beam in the bed longer than the normal 16-18 hours or add increased amounts of superplastizer and cement, thereby increasing the cost of the beam.

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

If the calculated initial or final strengths differ by more than 0.3 ksi from those used in the analysis, reanalyze the beam with the new values. Reanalysis is needed because changes to the concrete strengths f'_{ci} and f'_c affect the concrete modulus, which affects the prestress losses and the composite beam section modulus.

Straight strands must be arranged in a 2 inch grid pattern. See standard beam sheets for possible strand locations. Arrange draped strands in a 2 inch grid pattern independent of the straight strands. Use draped strands to reduce the initial required strength f'_{ci} at the end of the beam. Straight strands may be placed in the draped area at 2 inches from the bottom of the beam. Draped strands must start at 3 inches minimum from the bottom at the hold-downs and 3 inches minimum from the top at the end of the beam. Straight strands should be used in place of draped strands whenever possible.

In order to easily allow the fabricator to place and secure the stirrups in the bottom of the beam, always include strands in the outermost locations of rows 1 and 3 for RB, M, and MN shapes. Include strands in the outermost locations of rows 1 and 2 for MW shapes. Rows numbers are measured up from the bottom of the beam. See Figure 5.4.3.1.

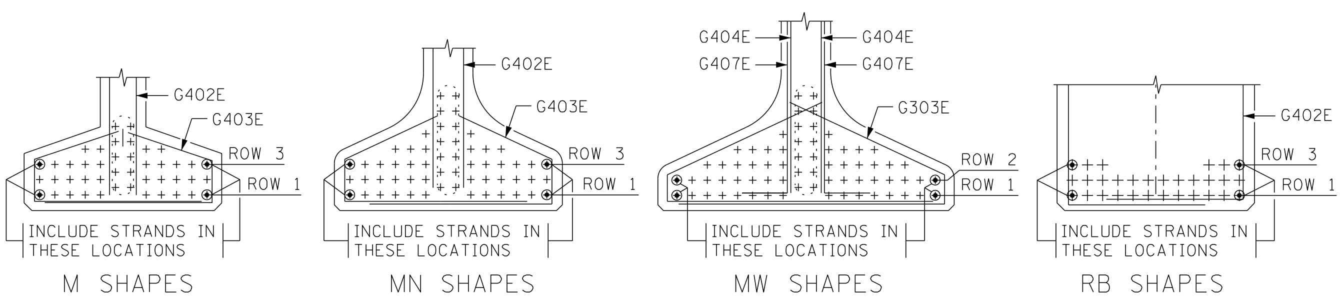


Figure 5.4.3.1

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

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

The following guidance is provided to designers to evaluate initial and final stresses to optimize their designs:

Final Stresses

Midpoint Strength at Bottom of Beam

If tension stress is greater than $0.19 \cdot \sqrt{f'_c}$ (0.570 ksi for 9 ksi concrete), lower the stress by:

1) Add 2 strands to the bottom row of straight and move the draped strands up 1 inch at midpoint (bottom row of draped at 4 inches).
2) Add 2 strands to the second row of straight and move the draped strands up 2 inches at midpoint (bottom row of draped at 6 inches) or add 2 draped strands (bottom row of draped at 4 inches).
3) Continue to add strands as stated above until 6 straight and 4 draped have been added. If the tension stress is still greater than 0.570 ksi, consider adding another line of beams to the bridge. If the tension stress is less than 0.570 ksi, two strands (either straight or draped) may be removed and the beam reanalyzed. If the stress becomes greater than 0.570 ksi, return to the original number of strands.

Initial Stresses

Midpoint Strength at Bottom of Beam

If the required initial strength is greater than 7.0 ksi:

1) Move the center of gravity of the strands up at midpoint of the beam until either the final concrete strength becomes 9.0 ksi or the initial strength is 0.5 to 1.0 ksi lower than the final strength.
2) Remove 2 strands (preferably draped) from the beam and reanalyze. Keep in mind that changes will affect the required final strength. If the removing of strands increases the final concrete strength above 9.0 ksi, do not remove the strands but consider other changes in the strand pattern.

End Strength at Bottom of Beam

If the required initial strength is greater than 7.0 ksi and greater than that calculated at the midpoint:

1) Strands may be draped to decrease the required strength. Keep in mind that changes to strand locations at the end of the beam may affect the mid-beam stresses.
2) If the initial strength is lower than calculated at the midpoint, draped strands may be placed straight thereby decreasing the

hold-down force and the number of draped strands required. Keep in mind that changes may affect the mid-beam stresses.

End Strength at Top of Beam

If the required initial strength is greater than 7.0 ksi, raise the center of gravity of the strands at the end of the beam. This can be accomplished by draping strands that were previously straight or increasing the height of the draped strands.

Midpoint Strength at Top of Beam

If the required initial strength is greater than calculated at the bottom end or midpoint:

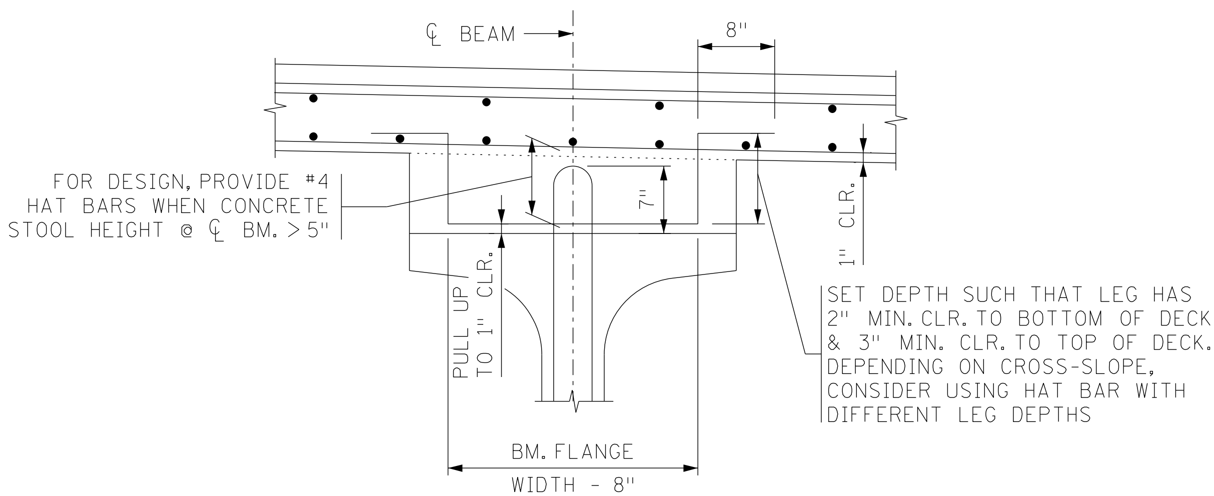
1) The center of gravity of the strands may be moved higher at the center.
2) The number of strands may be reduced to decrease the required strength.

If the guidance above results in an initial concrete strength greater than 7.0 ksi, the initial strength may be increased up to a maximum value of 7.5 ksi. Note that this will likely increase the beam cost.

Ensure that adequate shear and splitting reinforcement is provided in the ends of beams. The maximum size for stirrup bars is #5. Based on the concrete mix used for prestressed beams, the minimum stirrup spacing is $2^1/_2$ inches. If the required amount of reinforcement cannot be provided within $^h/_4$ of the end of the beam, provide the remainder at a $2^1/_2$ inch spacing.

Design shear reinforcement using the "General Procedure" provisions given in LRFD Article 5.8.3.4.2.

Horizontal shear reinforcement must be provided to ensure composite action between the beam and deck. MnDOT standard beam sheets accomplish this by extending the beam stirrups into the deck (G402E & G508E with 7" projection for RB, M, MN shapes and G404E & G508E with 7¼" projection for MW shapes). In order to ensure composite action, the shear reinforcement must extend into the deck far enough to engage the deck bottom mat of reinforcement. Check the stool heights over the length of the beams. For regions where stool heights are found greater than 5 inches at beam centerline, do not increase the stirrup length or pull up the stirrups, but rather provide #4 "hat" shaped bars as shown in Figure 5.4.3.2. Set the leg depth to provide 2 inches minimum clear to the bottom of deck and 3 inches clear to the top of deck for the upper hook, and 1 inch clear from the bottom of the bar to the beam flange. In cases where field personnel report excessive stools not anticipated in the bridge plan, discuss with them whether one "hat" bar or two "Z" bars would be better for rebar placement.



HAT BAR FOR BEAMS WITH LARGE STOOL HEIGHTS

Figure 5.4.3.2

Due to the height of the "MW" series beams, investigate whether a deck pour sequence is needed to reduce the effects of beam end rotation on the end region of the deck.

5.4.4 Detailing/ Reinforcement

Identify the beam type on the beam sheet by depth in inches and length rounded to the next highest foot. In the superstructure quantities list, identify the beam type by depth. For example, an MN45 beam, 72'-4" long would be "MN45-73" on the beam sheet and "MN45" in the quantities list. Group beams of similar lengths with the same strand pattern into one type on a beam sheet. The pay item quantity will be the total length of beams (of each height) in feet.

On the framing plan, show the beam and diaphragm spacing, staging, type of diaphragms, centerline of piers, centerline of abutment and pier bearings, working points, beam marks (B1, B2 etc.), the "X" end of beams, and the type and location of bearings. One end of each beam is labeled the "X" end after fabrication. This is used during erection to ensure that the beams are properly placed. Many times diaphragm inserts are not symmetric and beams can only be placed one way.

5.4.5 Camber and Deflection

The standard beam sheets contain a camber diagram where designers are to provide camber information. Knowing the deflection values associated with prestressing and different dead load components, camber values can be obtained.

MnDOT camber multipliers are used to approximately convert the prestress and selfweight deflections at the time of prestress transfer to the deflections at the time of erection. Use a camber multiplier of 1.40 for the prestress deflection component. Use a camber multiplier of 1.40 for the selfweight of the member. No multiplier is used for diaphragm dead loads, deck and stool dead loads or parapet and median dead loads. These camber multipliers differ from the PCI multipliers as they are based on research specific to MnDOT beams. They are based on a time lapse of 30 to 180 days between the time of prestress transfer and the time of beam erection for deck placement.

Use of the MnDOT camber multipliers does not apply to the "MW" series beams. Complete a refined camber analysis using an appropriate creep model for "MW" series camber determination. Then report the estimated camber values for various girder ages in the bridge plan.

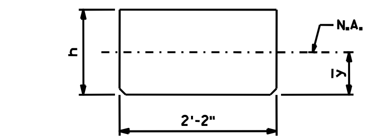
The "Erection Camber" is the camber of the beam at the time of erection after the diaphragms are in place. The "Est. Dead Load Deflection" is the sum of deflections associated with the placement of the deck, railings, sidewalks, and stool. Do not include the weight of the future wearing surface when computing the dead load deflection.

5.4.6 Standard I-Beams

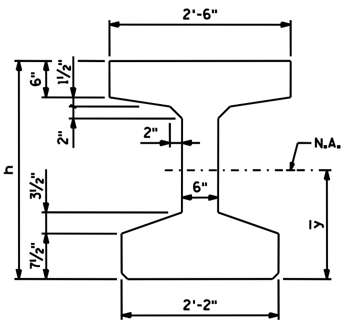
I-beam sections available for use in Minnesota include the "M" series, "MN" series, and "MW" series. The "M" series sections range in depth from 27 inches to 36 inches and have identical top and bottom flange dimensions along with a 6 inch thick web. The "MN" series sections range in depth from 45 inches to 63 inches. The "MN" series sections are more efficient than the "M" series due to wider top and bottom flanges and a $6^1/_2$ inch thick web. Due to the development of the MN45, MN54, MN63, and 82MW shapes, most of the M shapes (45M, 54M, 63M, 72M, and 81M) have been archived. The 27M and 36M shapes continue to be available as there is no corresponding MN shape at those depths. The "MW" series sections allow for spans in the range of 150 to 200 feet. Figures 5.4.6.1 through 5.4.6.4 contain section properties and preliminary beam spacing vs. span length curves for all standard I-beam shapes.

5.4.7 Rectangular Beams

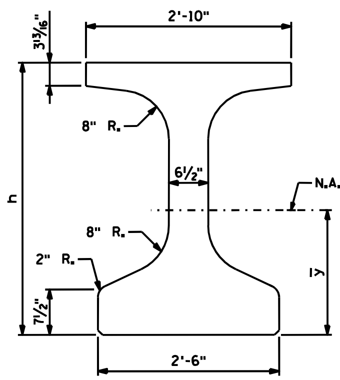
Solid rectangular prestressed beams may be used on short span bridges. These units are most appropriate for short span structures requiring a low profile or where construction of falsework for a slab structure would be difficult or unwanted. Figure 5.4.6.1 and 5.4.6.2 contain section properties and preliminary beam spacing vs. span length curves for the standard rectangular beams.



RECTANGULAR BEAM



"M" SERIES I-BEAM



"MN" SERIES I-BEAM

DESIGN ASSUMPTIONS FOR PRESTRESSED CONCRETE BEAM CHART:

2012 AASHTO LRFD Bridge Design Specifications, 6th Edition.

HL-93 Live Load

Beam Concrete: $f'_c = 9.0$ ksi $f'_{ci} = 7.5$ ksi $w_{bm} = 0.155$ kips/ft^3

$E_c = 1265\sqrt{f'_c} + 1000$ ksi

Deck Concrete: $f'_c = 4.0$ ksi $E_c = 3644$ ksi

w_c = 0.145 kcf for E_c computation

w_c = 0.150 kcf for dead load computation

0.6" diameter low relaxation strands, $E_s = 28{,}500$ ksi

$f_{pu} = 270$ ksi with initial pull of 0.75 f_{pu}

Simple supports with six beams and deck without wearing course.
Deck carries two F-Rails with no sidewalk or median, skew = 0 degrees.

Effective deck thickness is total deck thickness minus $^1/_2$" of wear.

$1^1/_2$" stool height used for composite beam section properties.
$2^1/_2$" average stool height used for dead load calculations.

Rail dead load applied equally to all beams.
Dead load includes 0.020 ksf future wearing course.

Approximate long term losses are used per LRFD 5.9.5.3.

Service Concrete Tensile Stress Limits:

After Initial Losses: $0.094\sqrt{f'_{ci}} \le 0.2$ ksi

After All Losses: $0.19\sqrt{f'_c}$

Beam Properties

BEAM	h (in)	SHAPE	AREA (in^2)	W ① (lb/ft)	$\bar{y}$ (in)	I (in^4)	S_B (in^3)	A_c ② (in^2)
14RB	14	Rect.	364	392	7.00	5,945	849	312
18RB	18	Rect.	468	504	9.00	12,640	1,404	364
22RB	22	Rect.	572	616	11.00	23,070	2,097	416
27M	27	I-Beam	516	555	13.59	43,080	3,170	296
36M	36	I-Beam	570	614	17.96	93,530	5,208	323
MN45	45	I-Beam	690	743	20.58	178,780	8,687	427
MN54	54	I-Beam	749	806	24.63	285,230	11,580	457
MN63	63	I-Beam	807	869	28.74	421,750	14,670	486

① Based on 155 pounds per cubic foot.

② Based on a 9" slab with $^1/_2$" of wear and $1^1/_2$" stool. See LRFD 5.8.3.4.2 for A_c definition.

Figure 5.4.6.1
Precast Prestressed Concrete Beam Data (RB, M, MN)

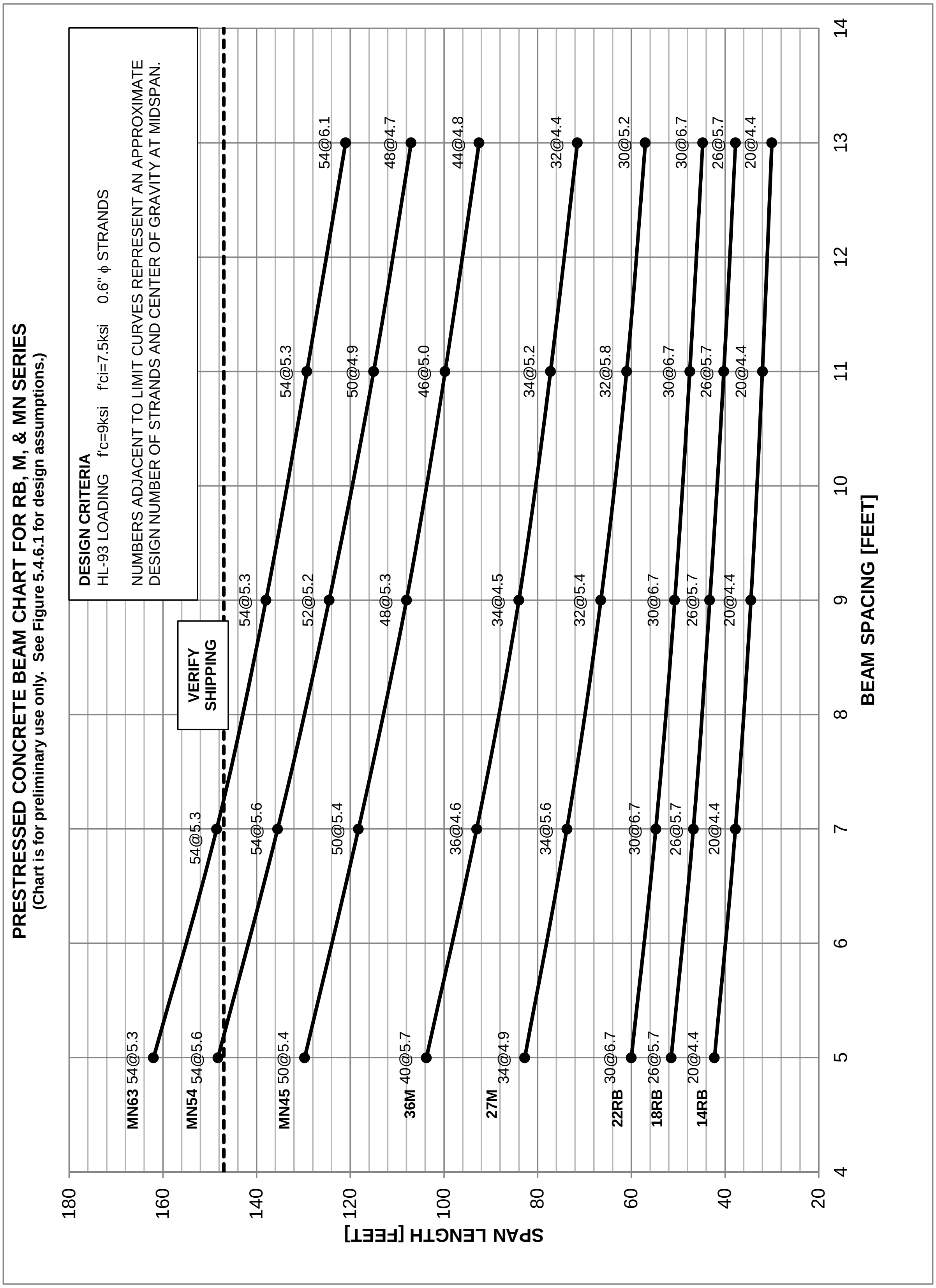
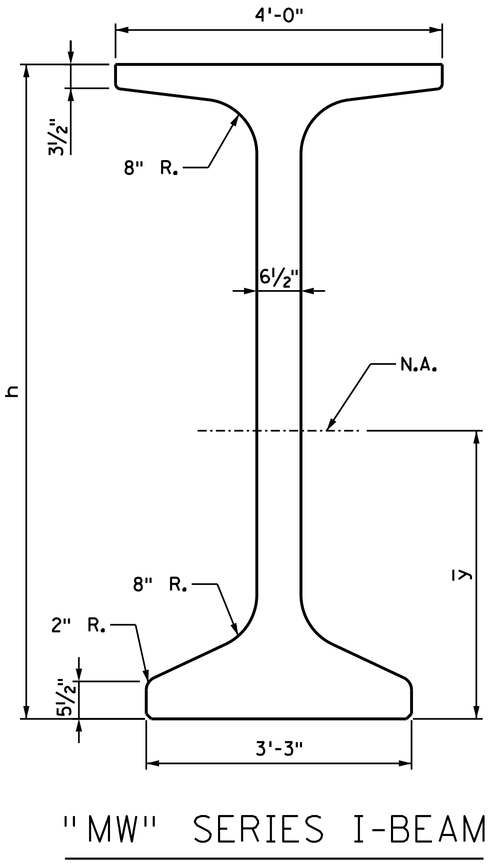

PRESTRESSED CONCRETE BEAM CHART FOR RB, M, & MN SERIES
(Chart is for preliminary use only. See Figure 5.4.6.1 for design assumptions.)
DESIGN CRITERIA
HL-93 LOADING f'c=9ksi f'ci=7.5ksi 0.6" ϕ STRANDS
NUMBERS ADJACENT TO LIMIT CURVES REPRESENT AN APPROXIMATE DESIGN NUMBER OF STRANDS AND CENTER OF GRAVITY AT MIDSPAN.
VERIFY SHIPPING
MN63 54@5.3
54@5.3
54@5.3
54@5.3
54@6.1
MN54 54@5.6
54@5.6
52@5.2
50@4.9
48@4.7
MN45 50@5.4
50@5.4
48@5.3
46@5.0
44@4.8
36M 40@5.7
36@4.6
34@4.5
34@5.2
32@4.4
27M 34@4.9
34@5.6
32@5.4
32@5.8
30@5.2
22RB 30@6.7
30@6.7
30@6.7
30@6.7
30@6.7
18RB 26@5.7
26@5.7
26@5.7
26@5.7
26@5.7
14RB 20@4.4
20@4.4
20@4.4
20@4.4
20@4.4
SPAN LENGTH [FEET]
180
160
140
120
100
80
60
40
20
BEAM SPACING [FEET]
4
5
6
7
8
9
10
11
12
13
14

Figure 5.4.6.2



DESIGN ASSUMPTIONS FOR PRESTRESSED CONCRETE BEAM CHART:

2012 AASHTO LRFD Bridge Design Specifications, 6th Edition.

HL-93 Live Load

Beam Concrete: $f'_c = 9.0$ ksi $f'_{ci} = 7.5$ ksi $w_{bm} = 0.155$ kips/ft^3

$E_c = 1265\sqrt{f'_c} + 1000$ ksi

Deck Concrete: $f'_c = 4.0$ ksi $E_c = 3644$ ksi

w_c = 0.145 kcf for E_c computation

w_c = 0.150 kcf for dead load computation

0.6" diameter low relaxation strands, $E_s = 28{,}500$ ksi

$f_{pu} = 270$ ksi with initial pull of $0.75\ f_{pu}$

Simple supports with six beams and deck without wearing course.
Deck carries two F-Rails with no sidewalk or median, skew = 0 degrees.

Effective deck thickness is total deck thickness minus $^1/_2$" of wear.

$1^1/_2$" stool height used for composite beam section properties.
$2^1/_2$" average stool height used for dead load calculations.

Rail dead load applied equally to all beams.
Dead load includes 0.020 ksf future wearing course.

Approximate long term losses are used per LRFD 5.9.5.3.

Service Concrete Tensile Stress Limits:

After Initial Losses: $0.094\sqrt{f'_{ci}} \leq 0.2$ ksi

After All Losses: $0.19\sqrt{f'_c}$

Beam Properties

BEAM	h (in)	SHAPE	AREA (in^2)	W ① (lb/ft)	$\overline{y}$ (in)	I (in^4)	S_B (in^3)	A_c ② (in^2)
82MW	82	I-Beam	1062	1143	38.37	1,010,870	26,345	609
96MW	96	I-Beam	1153	1241	45.02	1,486,510	33,019	655

① Based on 155 pounds per cubic foot.
② Based on a 9" slab with $^1/_2$" of wear and $1^1/_2$" stool. See LRFD 5.8.3.4.2 for A_c definition.

Figure 5.4.6.3
Precast Prestressed Concrete Beam Data for MW Series

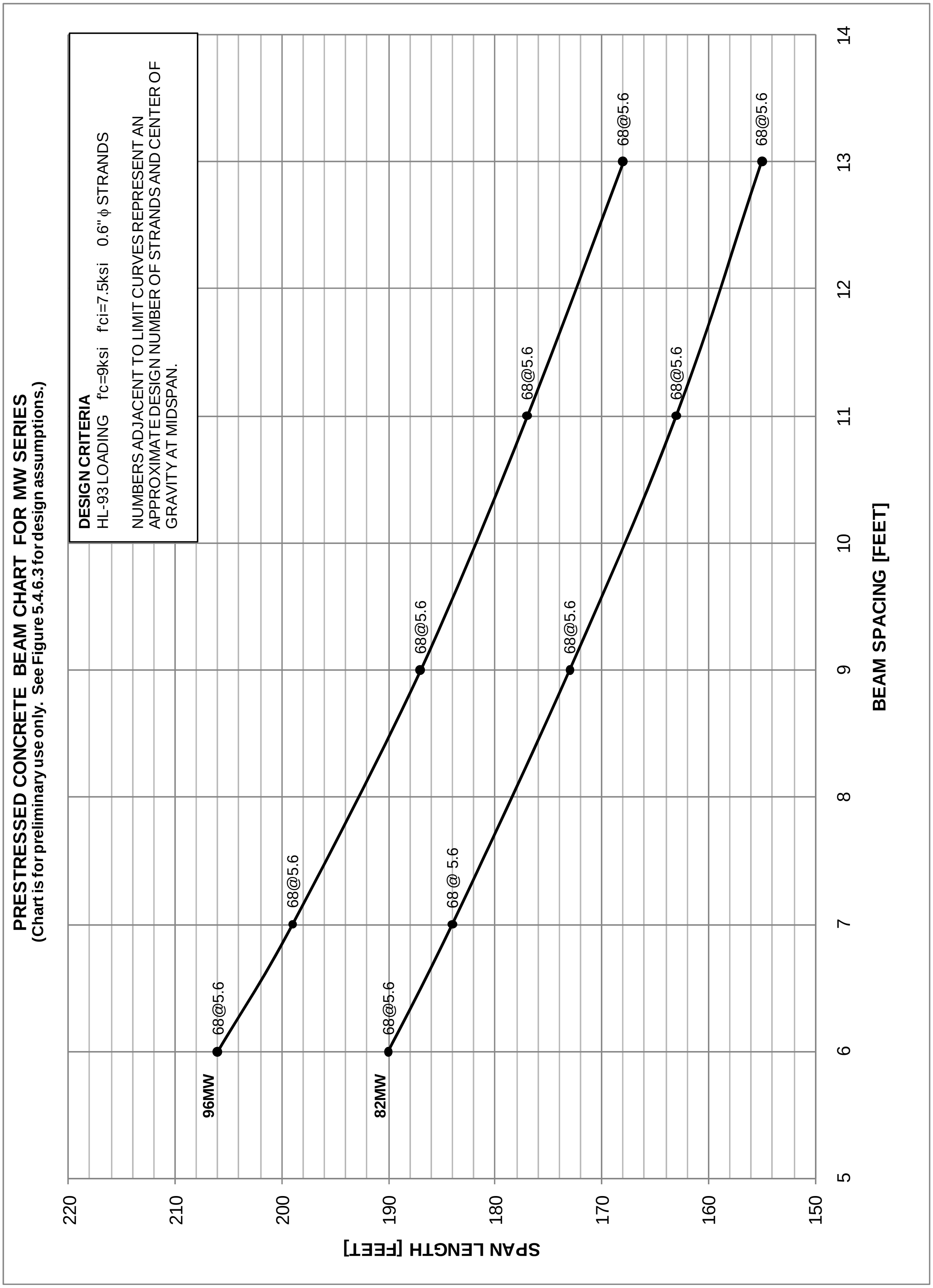

PRESTRESSED CONCRETE BEAM CHART FOR MW SERIES
(Chart is for preliminary use only. See Figure 5.4.6.3 for design assumptions.)
DESIGN CRITERIA
HL-93 LOADING f'c=9ksi f'ci=7.5ksi 0.6" ϕ STRANDS
NUMBERS ADJACENT TO LIMIT CURVES REPRESENT AN APPROXIMATE DESIGN NUMBER OF STRANDS AND CENTER OF GRAVITY AT MIDSPAN.
96MW
82MW
68@5.6
SPAN LENGTH [FEET]
BEAM SPACING [FEET]
220
210
200
190
180
170
160
150
5
6
7
8
9
10
11
12
13
14

Figure 5.4.6.4

5.4.8 Double-Tee Beams

Pretensioned double-tees are not used anymore on Minnesota bridges. The standard Bridge Details Part II Figures 5-397.525 and 5-397.526 were archived in 2009.

5.5 Post-Tensioned Concrete

Post-tensioned (PT) concrete structures have their prestressing steel stressed after the concrete has been placed and partially cured.

Post-tensioned concrete bridges are specialty structures. Poor detailing and poor construction practices can greatly reduce the service life of these structures. Designers should follow current practices recommended by the American Segmental Bridge Institute (ASBI) and the Post-Tensioning Institute (PTI).

Design segmental box girders and post-tensioned concrete slab bridges for zero tension under service loads.

5.5.1 PT Slab Bridges

Post-tensioned concrete slab bridges are used for projects requiring spans longer than those efficiently accommodated with conventionally reinforced concrete slabs. The drawback to post-tensioned slabs is that they are more complex to design and construct. Elastic shortening and secondary bending moments due to post-tensioning are important design parameters for post-tensioned slab bridges. During construction a number of additional components are involved. They include the installation of post-tensioning ducts and anchorages, the pushing or pulling of strands through the ducts, the jacking of tendons, and grouting operations.

5.5.2 PT I-Girders

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

5.5.3 PT Precast or Cast-In-Place Box Girders

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

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

5.6 Concrete Finishes and Coatings

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

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

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

5.7 Design Examples

Three design examples complete Section 5. The examples consist of a three-span reinforced concrete slab superstructure, a prestressed I-beam superstructure, and a three-span post-tensioned slab superstructure.

5.7.1 Three-Span Haunched Reinforced Concrete Slab

This example illustrates the design of a haunched reinforced concrete slab bridge. The three continuous spans are 44'-0", 55'-0", and 44'-0" in length. The roadway width is 44'-0" with MnDOT Type F barrier railings for a total out-to-out width of 47'-4". The bridge is skewed 10 degrees. A plan view and typical sections of the bridge are shown in Figures 5.7.1.1 and 5.7.1.2.

After determining live load distribution factors, dead and live loads are computed at span tenth points. Next the live load deflection and the shear capacity of the section is checked. Then using Strength I, Service I, and Fatigue design moments the flexural reinforcement is sized. This is accomplished by:

- Providing adequate steel for strength
- Verifying that crack control checks are satisfied
- Checking fatigue stresses in the reinforcement
- Verifying that minimum reinforcement checks are satisfied

Finally, distribution and shrinkage and temperature reinforcement is sized.

Material and design parameters used in this example are:

Concrete Strength at 28 Days, $f'_c = 4.0$ ksi

Concrete Unit Weight, $w_c = 0.150$ kcf (dead loads)
$w_c = 0.145$ kcf (modulus)

Reinforcing Bars:
Yield Strength, $f_y = 60$ ksi
Modulus of Elasticity, $E_s = 29{,}000$ ksi

Weight of Future Wearing Surface = 20 psf
Weight of concrete rail = 0.439 kip/ft

For simplicity, the wearing course is assumed to extend from out-to-out of deck.

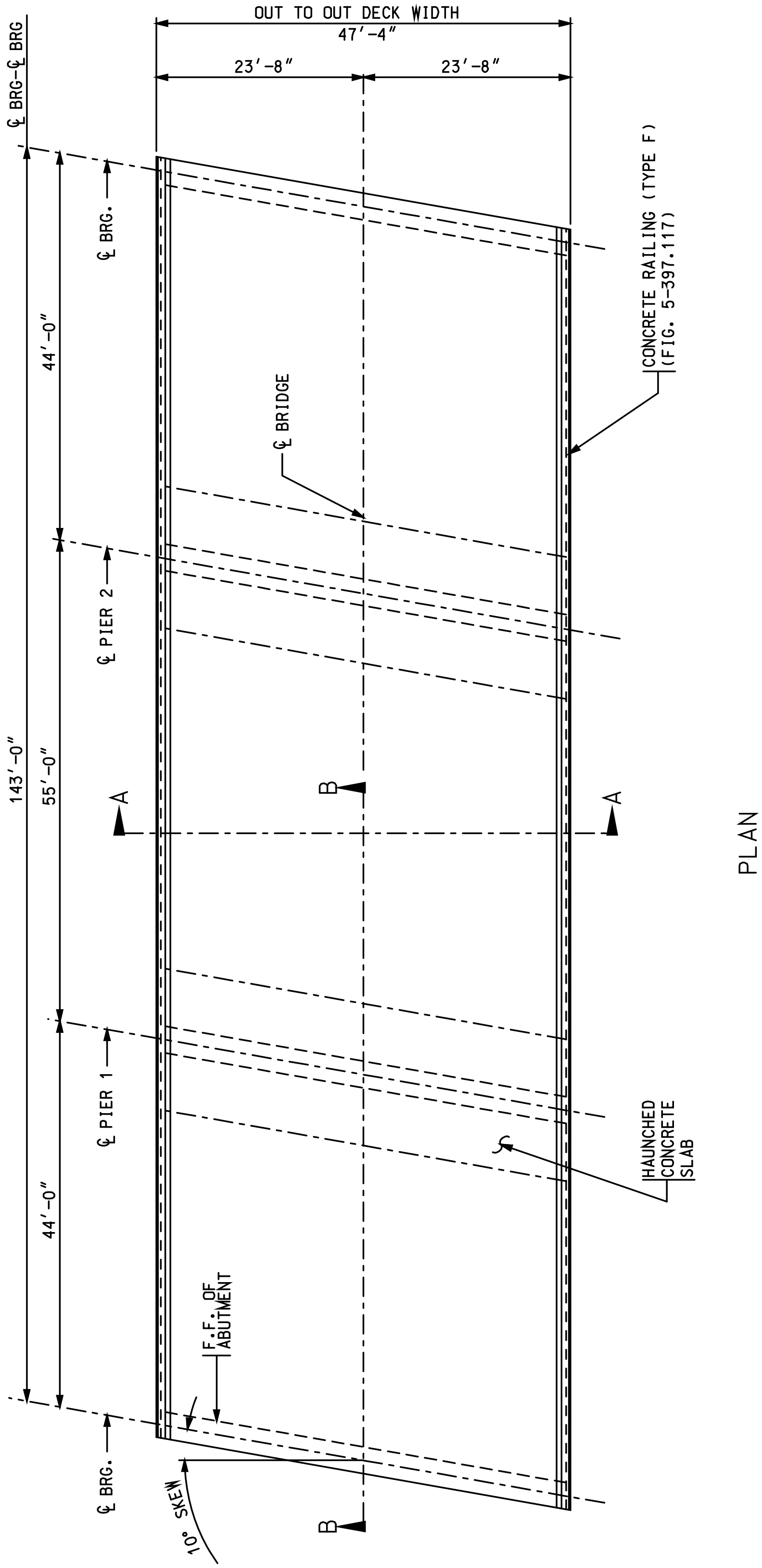

OUT TO OUT DECK WIDTH
47'-4"
23'-8"
23'-8"
℄ BRG-℄ BRG
℄ BRG.
44'-0"
℄ PIER 2
143'-0"
55'-0"
℄ PIER 1
44'-0"
℄ BRG.
10° SKEW
F.F. OF ABUTMENT
℄ BRIDGE
A
A
B
B
CONCRETE RAILING (TYPE F) (FIG. 5-397.117)
HAUNCHED CONCRETE SLAB
PLAN

Figure 5.7.1.1

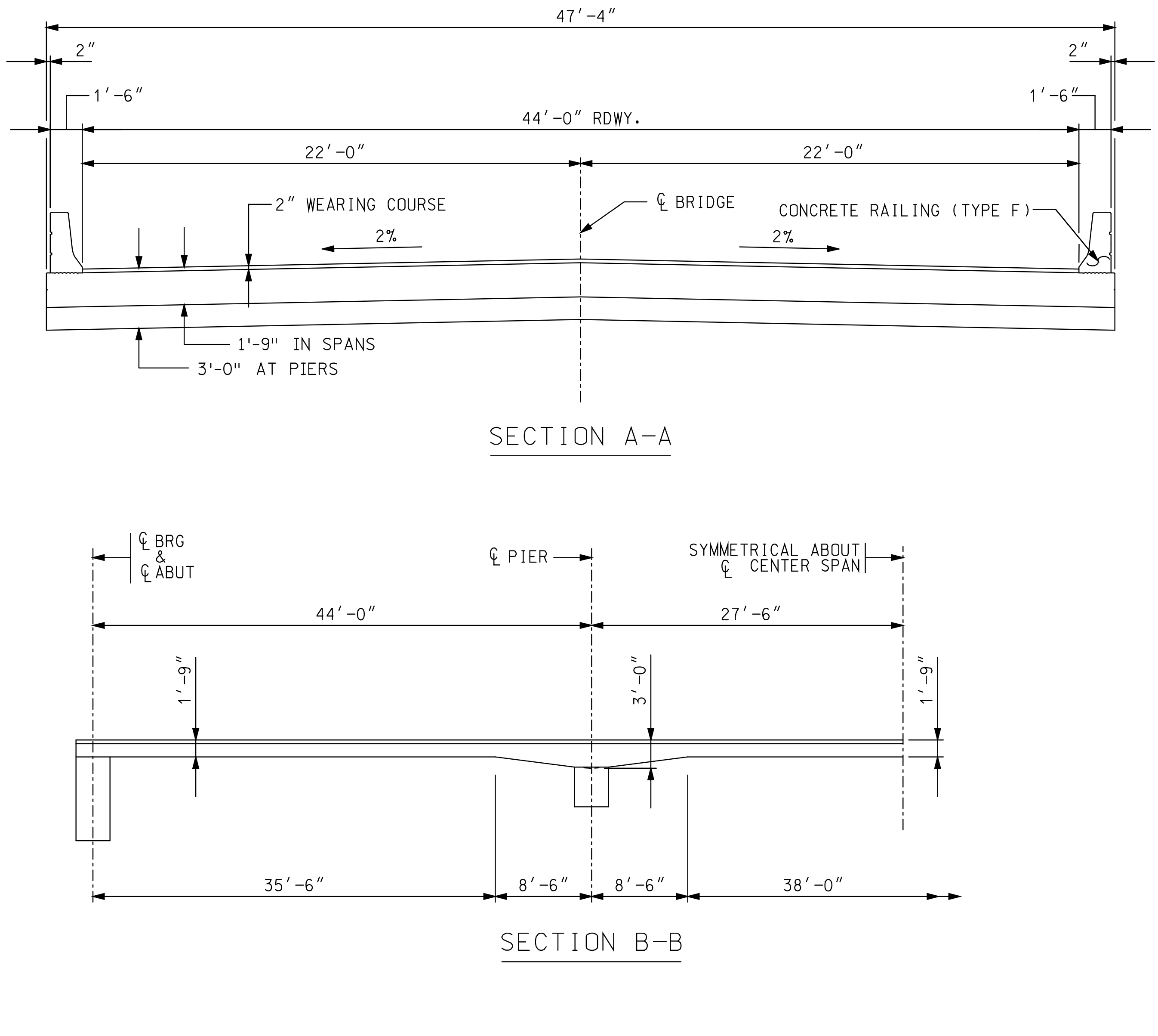

47'-4"
2"
2"
1'-6"
1'-6"
44'-0" RDWY.
22'-0"
22'-0"
2" WEARING COURSE
℄ BRIDGE
CONCRETE RAILING (TYPE F)
2%
2%
1'-9" IN SPANS
3'-0" AT PIERS
SECTION A-A
℄ BRG & ℄ ABUT
℄ PIER
SYMMETRICAL ABOUT ℄ CENTER SPAN
44'-0"
27'-6"
1'-9"
3'-0"
1'-9"
35'-6"
8'-6"
8'-6"
38'-0"
SECTION B-B

Figure 5.7.1.2

A. Determine Slab Depths

MnDOT's practice is to use linear haunches, with the haunch length equal to 15 % of the longest span.

Haunch Length $= 0.15 \cdot 55 = 8.25$ ft <u>Use 8.5 ft</u>

The minimum slab depth at midspan (h_{min}) is also determined with the length of the longest span (S):

$$0.80 \cdot \frac{(S + 10)}{30} = 0.80 \cdot \frac{(55 + 10)}{30} = 1.73 \text{ ft}$$ <u>Use $h_{min} = 1.75$ ft</u>

The depth of the slab required at the pier (h_{max}) is determined with an equation based on the length of the longest span:

$$1.33 \cdot \frac{(S + 10)}{30} = 1.33 \cdot \frac{(55 + 10)}{30} = 2.88 \text{ ft}$$ <u>Use $h_{max} = 3.00$ ft</u>

The slab depth (h) includes the 2 inch wearing course.

B. Determine Interior Live Load Strip Width [4.6.2.3] [3.6.1.1.1]

The LRFD Specifications contain equations to determine the strip width that carries a lane of live load. Slab designs are performed on a strip one foot wide. The strip widths found with the LRFD equations are inverted to arrive at the live load distribution factor for a 1 foot wide strip (LLDF).

For interior strips multiple equations are evaluated to determine whether one or multiple live load lanes govern.

Flexure – One Lane Loaded

Multiple Presence Factors have been incorporated into the LRFD equations per LRFD C3.6.1.1.2.

[Eqn. 4.6.2.3-1] Equivalent strip width (in), $E = 10.0 + 5.0 \cdot \sqrt{L_1 \cdot W_1}$

Where:

L_1 is the modified span length.
It is equal to the span length, but can be no greater than 60.

W_1 is the modified bridge width.
It is the minimum bridge width, but can be no greater than 30.

For the 44 ft side spans:

$$E = 10.0 + 5.0 \cdot \sqrt{44 \cdot 30} = 191.7 \text{ in/lane}$$

Therefore $LLDF_{SL} = \dfrac{1}{\left(\dfrac{191.7}{12}\right)} = 0.063$ lanes/ft <u>Governs</u>

For the 55 ft center span:

$$E = 10.0 + 5.0 \cdot \sqrt{55 \cdot 30} = 213.1 \text{ in/lane}$$

$$\text{Therefore } LLDF_{SL} = \frac{1}{\left(\frac{213.1}{12}\right)} = 0.056 \text{ lanes/ft}$$

Flexure – Fatigue Loading

Divide the one lane LLDF by 1.2 to remove the Multiple Presence Factor

For the 44 ft side spans:

$$LLDF_{FAT} = \frac{0.063}{1.2} = 0.052 \text{ lanes/ft}$$ <u>Governs</u>

For the 55 ft center span:

$$LLDF_{FAT} = \frac{0.056}{1.2} = 0.047 \text{ lanes/ft}$$

Flexure – Multiple Lanes Loaded

[Eqn. 4.6.2.3-2] Equivalent strip width (in), $E = 84.0 + 1.44 \cdot \sqrt{L_1 \cdot W_1} \le \frac{12 \cdot W}{N_L}$

L_1 = 44 ft or 55 ft

W_1 = 47.33 ft

W is the actual bridge width = 47.33 ft

$N_L = \frac{44}{12} = 3.7$ <u>Use 3</u>

The upper limit on the equivalent strip width is:

$$\frac{12.0 \cdot W}{N_L} = \frac{12.0 \cdot 47.33}{3} = 189.3 \text{ in/lane}$$

For the 44 ft side spans:

$$E = 84.0 + 1.44 \cdot \sqrt{44 \cdot 47.33} = 149.7 \le 189.3 \text{ in/lane}$$

$$\text{Therefore the } LLDF_{ML} = \frac{1}{\left(\frac{149.7}{12}\right)} = 0.080 \text{ lanes/ft}$$ <u>Governs</u>

For the 55 ft center span:

$$E = 84.0 + 1.44 \cdot \sqrt{55 \cdot 47.33} = 157.5 \leq 189.3 \text{ in/lane}$$

$$\text{Therefore the } LLDF_{ML} = \frac{1}{\left(\frac{157.5}{12}\right)} = 0.076 \text{ lanes/ft}$$

To simplify the process of arriving at design forces, the maximum distribution factor (0.080 lanes/ft) will be used for all locations.

Shear and Deflection

[5.14.4.1] Although not required by AASHTO, MnDOT requires that slab type bridges be checked for shear using the same live load distribution factor calculated for deflection.

[2.5.2.6.2] All design lanes should be loaded and the entire slab assumed to resist the loads.

$N_L = 3$ $\quad$ $m = 0.85$ $\quad$ $IM = 33\%$

Dynamic load allowance (IM) is applied only to the truck portion of the live load. The distribution factor for the lane portion of the live load is:

$$LLDF_{s\Delta} = \frac{(\#\text{ of lanes}) \cdot (MPF)}{(\text{deck width})} = \frac{3 \cdot 0.85}{47.33} = 0.054 \text{ lanes/ft}$$

The distribution factor for the truck portion is:

$$LLDF_{s\Delta} \cdot (1 + IM) = 0.054 \cdot (1 + 0.33) = 0.072 \text{ lanes/ft}$$

Reduction for Skew

[Eqn. 4.6.2.3-3] $r = 1.05 - 0.25 \cdot \tan\theta = 1.05 - 0.25 \cdot \tan(10^\circ) = 1.006$ <u>No Reduction</u>

C. Determine Exterior Live Load Strip Width

[4.6.2.1.4]

The exterior strip is assumed to carry one wheel line and a tributary portion of lane load.

Check if the equivalent strip is less than the maximum width of 72 inches.

$$E = (\text{Distance from edge to inside of barrier}) + 12 + \left(\frac{\text{smallest int. } E}{4}\right)$$

$$= 20 + 12 + \frac{149.7}{4} = 69.4 < 72.0 \text{ in}$$ <u>Use 69.4 in</u>

Compute the distribution factor associated with one truck wheel line:

$$LLDF_{EXTT} = \frac{1 \text{ wheelline} \cdot MPF}{(2 \text{ wheellines/lane}) \cdot (E/12)} = \frac{1 \cdot 1.2}{(2) \cdot (69.4/12)}$$

$$= 0.104 \text{ lanes/ft}$$

Compute the distribution factor associated with lane load on a 69.4 inch wide exterior strip:

$$LLDF_{EXTL} = \frac{\left(\dfrac{\text{deck width loaded}}{10 \text{ ft load width}}\right) \cdot MPF}{(\text{exterior strip width})} = \frac{\left(\dfrac{69.4/12 - 20/12}{10}\right) \cdot 1.2}{(69.4/12)}$$

$$= 0.085 \text{ lanes/ft}$$

For simplicity, the larger value (0.104 lanes/ft) is used for both load types when assembling design forces.

D. Resistance Factors and Load Modifiers [5.5.4.2.1] [1.3.3-1.3.5]

The following resistance factors will be used for this example:

$\phi = 0.90$ for flexure and tension (assumed, must be checked)

$\phi = 0.90$ for shear and torsion

The following load modifiers will be used for this example:

		Strength	Service	Fatigue
Ductility	η_D	1.0	1.0	1.0
Redundancy	η_R	1.0	1.0	1.0
Importance	η_I	1.0	n/a	n/a
	$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0	1.0

E. Select Applicable Load Combinations and Load Factors [3.4.1]

Three load combinations will be considered for the design example. STRENGTH I - Will be considered with a standard HL-93 loading.

$$U = 1.0 \cdot [1.25 \cdot (DC) + 1.75 \cdot (LL + IM)]$$

SERVICE I - Will be used primarily for crack control checks.

$$U = 1.0 \cdot (DC) + 1.0 \cdot (LL + IM)$$

FATIGUE - Will be used to evaluate the reinforcing steel.

$$U = 0.75 \cdot (LL + IM)$$

F. Calculate Live Load Force Effects [3.6.1]

The LRFD Specifications contain several live load components that are combined and scaled to generate design live loads. The components include: HL-93 design truck, lane loading, tandem axles, a truck train, and a fatigue truck with fixed axle spacings.

For this example the following combinations will be investigated:

Design Truck + Design Lane
Design Tandem + Design Lane
0.90 (Truck Train + Design Lane) (Neg. Moment Regions)
Fatigue Truck

The dynamic load allowance (IM) has the following values:

IM = 15% when evaluating fatigue and fracture.
IM = 33% when evaluating all other limit states.

It is not applied to the lane live load.

G. Calculate Force Effects from Other Loads

The dead load from the barriers is conservatively assumed to be fully carried by both interior and exterior strips. Since the slab thickness varies, the load effect due to the slab is kept separate.

Interior Strip (1'-0" Wide)

Slab, wearing course, and barrier dead loads

$$W_{DC} = (1.0 \cdot 0.150 \cdot h) + \frac{2 \cdot (0.439)}{47.33} = 0.150 \cdot h + 0.019 \quad \text{(kip/ft)}$$

Future wearing surface

$$W_{DW} = (1.0 \cdot 0.020) = 0.020 \quad \text{(kip/ft)}$$

(included with DC loads in load tables)

Exterior Strip (1'-0" Wide)

Slab, wearing course, and barrier dead loads

$$W_{DC} = (1.0 \cdot 0.150 \cdot h) + \frac{0.439}{\left(\frac{69.4}{12}\right)} = 0.150 \cdot h + 0.076 \quad \text{(kip/ft)}$$

Future wearing surface

$$W_{DW} = \frac{\left(\frac{69.4}{12} - 1.67\right) \cdot 0.020}{\left(\frac{69.4}{12}\right)} = 0.014 \quad \text{(kip/ft)}$$

(included with DC loads in load tables)

H. Summary of Analysis Results

From this point forward, the design of an interior strip (subject to dead and live loads) will be presented. The design procedure for the exterior strip is similar. A computer analysis was performed with a three-span continuous beam model. The model included the stiffening effect of the haunches.

Bending moment summaries obtained at different span locations are presented in Tables 5.7.1.1 through 5.7.1.4. These tables also contain truck live load deflections and dead load deflections due to slab selfweight, wearing course, and two barriers. Shear information is presented in Tables 5.7.1.5 through 5.7.1.7.

Loads and deflections that appear later in the example are identified with bold numbers.

Table 5.7.1.1
Moment Summary – One Lane

Span Point	Lane (kip-ft)	Truck (kip-ft)	Tandem (kip-ft)	Truck Tr (kip-ft)	+ Fatigue (kip-ft)	- Fatigue (kip-ft)
1.0	0	0	0	-	0	0
1.1	50	194	178	-	140	-25
1.2	87	316	299	-	225	-49
1.3	112	373	368	-	285	-74
1.4	124	390	388	-	**299**	**-98**
1.5	124	374	374	-	285	-123
1.6	112/-76	333/-220	329/-178	-	253	-147
1.7	86/-87	244/-254	249/-204	-253	187	-172
1.8	-104	-289	-233	-292	128	-196
1.9	-149	-325	-263	-337	67	-258
2.0	-221	-378	-292	-383	**75**	**-387**
2.1	-129	-267	-229	-284	79	-228
2.2	44/-75	157/-226	190/-193	-226	151	-151
2.3	78/-64	284/-187	288/-163	-	223	-123
2.4	107	360	350	-	275	-95
2.5	117	378	368	-	284	-66

Table 5.7.1.2

Moment Summary – Interior Strip (per foot width)

Span Point	M_{DC} (kip-ft)	* Truck + Lane (kip-ft)	* Tandem + Lane (kip-ft)	* .9 (Truck Tr + Lane) (kip-ft)
1.0	0	0	0	-
1.1	17.2	25	23	-
1.2	28.9	41	39	-
1.3	34.3	49	48	-
1.4	**34.3**	51	51	-
1.5	28.9	50	50	-
1.6	17.1	44/-29	44/-25	-
1.7	-1.1	33/-34	34/-29	-31
1.8	-23.6	-39	-33	-35
1.9	-53.6	-47	-40	-43
2.0	**-90.9**	-58	-49	-53
2.1	-48.2	-39	-35	-37
2.2	-16.0	21/-30	24/-27	-27
2.3	7.5	37/-25	37/-22	-
2.4	20.4	47	46	-
2.5	25.7	50	49	-

* Includes Dynamic Load Allowance (IM) and Live Load Distribution Factor.

Table 5.7.1.3

Moment Summary – Exterior Strip (per foot width)

Span Point	M_{DC} (kip-ft)	* Truck + Lane (kip-ft)	* Tandem + Lane (kip-ft)	* 0.9 (Truck Tr + Lane) (kip-ft)
1.0	0	0	0	-
1.1	20.0	32	30	-
1.2	33.3	53	51	-
1.3	40.6	63	62	-
1.4	40.6	67	67	-
1.5	33.3	64	64	-
1.6	19.8	58/-38	57/-33	-
1.7	-1.0	44/-44	45/-37	-40
1.8	-28.1	-51	-43	-46
1.9	-62.5	-60	-52	-56
2.0	-105.2	-75	-63	-69
2.1	-55.2	-50	-45	-48
2.2	-18.7	28/-40	31/-34	-35
2.3	8.3	48/-33	48/-29	-
2.4	24.0	61	59	-
2.5	29.2	64	63	-

*Includes Dynamic Load Allowance (IM) and Live Load Distribution Factor.

Table 5.7.1.4

Moment Load Combinations and Deflections

Span Point	Service I		Strength I		* Lane LL Deflection (in)	* Truck LL Deflection (in)	** Dead Ld Deflection (in)
	Interior (kip-ft)/ft	Exterior (kip-ft)/ft	Interior (kip-ft)/ft	Exterior (kip-ft)/ft			
1.0	0	0	0	0	0.000	0.000	0.000
1.1	42	52	65	81	0.047	0.172	0.089
1.2	70	86	107	134	0.087	0.310	0.162
1.3	83	104	128	162	0.118	0.414	0.209
1.4	**86**	107	**133**	167	0.137	0.466	0.224
1.5	79	98	123	154	**0.141**	**.0475**	0.208
1.6	**62/-12**	78/-18	**99/-35**	127/-49	0.131	0.430	0.166
1.7	**32/-35**	42/-45	**58/-61**	77/-78	0.108	0.344	0.110
1.8	**-63**	-79	**-98**	-124	0.076	0.242	0.056
1.9	**-100**	-123	**-148**	-184	0.038	0.120	0.019
2.0	**-149**	-180	**-215**	-263	0.000	0.000	0.000
2.1	**-87**	-105	**-128**	-156	0.046	0.156	0.002
2.2	**8/-46**	13/-58	**28/-73**	37/-93	0.072	0.328	0.031
2.3	**44/-18**	56/-25	**74/-37**	94/-50	0.138	0.500	0.085
2.4	67	85	108	137	0.167	0.586	0.130
2.5	75	94	**119**	149	**0.178**	**0.653**	0.147

* Based on $I_{effective} = {}^{1}/_{2}\ I_{gross}$. Includes LL distribution factor.

**Includes selfweight, wearing course, and barriers.

Table 5.7.1.5

Shear Summary – One Lane

Span Point	Lane (kips)	Truck (kips)	Tandem (kips)
1.0	12.7	52.8	47.0
1.1	10.3	44.1	40.4
1.2	7.8	35.8	34.0
1.3	5.9	28.2	27.8
1.4	5.8	21.1	22.8
1.5	7.3	27.6	28.5
1.6	9.2	34.9	33.9
1.7	11.4	42.3	38.6
1.8	13.9	49.3	42.6
1.9	16.4	55.6	46.0
2.0	19.9	61.2	48.8
2.1	16.6	54.4	45.0
2.2	13.4	46.9	40.4
2.3	10.6	38.8	35.0
2.4	8.2	30.5	29.0
2.5	6.2	22.4	22.7

Table 5.7.1.6
Shear Summary (per foot width)

Span Point	V_{DC} (kips)	* Truck + Lane (kips)	* Tandem + Lane (kips)
1.0	4.6	4.5	4.1
1.1	3.3	3.7	3.5
1.2	2.0	3.0	2.9
1.3	0.7	2.3	2.3
1.4	0.7	1.8	2.0
1.5	2.0	2.4	2.4
1.6	3.3	3.0	2.9
1.7	4.7	3.7	3.4
1.8	6.0	4.3	3.8
1.9	7.5	4.9	4.2
2.0	9.5	5.5	4.6
2.1	6.7	4.8	4.1
2.2	5.0	4.1	3.6
2.3	3.3	3.4	3.1
2.4	1.7	2.6	2.5
2.5	0.0	1.9	2.0

* Includes Dynamic Load Allowance (IM) and 0.054 Distribution Factor.

Table 5.7.1.7
Shear Summary – Load Combinations

Span Point	SERVICE I (kips)	STRENGTH I (kips)
1.0	9.1	13.6
1.1	7.0	10.6
1.2	5.0	7.7
1.3	3.0	5.0
1.4	2.7	4.3
1.5	4.4	6.8
1.6	6.3	9.4
1.7	8.4	12.3
1.8	10.3	**15.0**
1.9	12.4	**17.9**
2.0	15.0	21.4
2.1	11.5	16.8
2.2	9.1	13.4
2.3	6.7	10.0
2.4	4.3	6.7
2.5	2.0	3.4

I. Live Load Deflection [2.5.2.6]

To prevent serviceability problems, a limit is placed on the maximum live load deflections. The limit is:

$$\Delta_{LL+I} = \frac{\text{Span}}{800}$$

$$\text{Spans 1 and 3} = \frac{44 \cdot 12}{800} = 0.66 \text{ in}$$

$$\text{Span 2} = \frac{55 \cdot 12}{800} = 0.83 \text{ in}$$

[3.6.1.3.2]

Use the design truck alone or design lane load plus 25% of truck load.

Using the Table 5.7.1.1 live load deflection values, the following maximum live load deflections were obtained:

Midspans 1 and 3

Truck: 0.475 in < 0.66 in

Lane + 25% Truck: 0.141 + 0.25 (0.475) = 0.260 in < 0.66 in

Midspan 2

Truck: 0.653 in < 0.83 in

Lane + 25% Truck: 0.178 + 0.25 (0.653) = 0.341 in < 0.66 in

J. Shear in Slab [5.13.3.6]

$V_r = \phi \cdot V_n = 0.9\ V_n$

Check the one-way shear capacity of the slab.

[5.8.2.9]

Critical Section

Shear should be checked at all sections. In many cases the governing location is at the abutment, a pier, or at the start of the haunch. Calculations for the shear check at the start of the linear haunch for the side span (Span Point 1.81) follow.

The effective shear depth d_v is the distance between the internal tension and compression force components to resist flexure, which is unkown at this point in the design.

But the shear depth need not be less than

$$0.9 \cdot d_e = 0.9 \cdot (17.0) = 15.30 \text{ in}$$

or

$$0.72 \cdot h = 0.72 \cdot (19.0) = 13.68 \text{ in}$$

Use $d_v = 15.30$ in

The shear loads at adjacent span points are interpolated to determine the shear at Span Point 1.81:

$$V_U = 15.0 \text{ kips} + \left(\frac{1.81 - 1.8}{1.9 - 1.8}\right) \cdot (17.9 \text{ kips} - 15.0 \text{ kips}) = 15.3 \text{ kips}$$

[5.8.3.3]

Nominal Shear Resistance

The nominal shear resistance is the sum of the contributions from the concrete and steel.

$$V_n = V_c + V_s$$

It can be no more than:

$$V_n \leq 0.25 \cdot f'_c \cdot b_v \cdot d_v = 0.25 \cdot 4.0 \cdot 12 \cdot 15.30 = 183.6 \text{ kips}$$

To simplify the calculation for the concrete contribution, assume $\beta = 2.0$. If shear reinforcement is found necessary, the designer should first try thickening the slab to eliminate the need for shear reinforcement. If shear reinforcement must be used, the appropriate β and θ values should be used for the shear design.

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 2.0 \cdot \sqrt{4.0} \cdot 12 \cdot 15.30 = 23.2 \text{ kips}$$

Without shear reinforcement, $V_s = 0$

The nominal shear capacity of the slab is:

$$V_n = 23.2 + 0 = 23.2 \text{ kips} < 183.6 \text{ kips} \qquad \underline{\text{OK}}$$

Check if the shear resistance is greater than the shear demand:

$$V_r = \phi \cdot V_n = 0.90 \cdot (23.2) = 20.9 \text{ kips} > 15.3 \text{ kips} \qquad \underline{\text{OK}}$$

K. Design Positive Moment Reinforcement [5.7.2.2] [5.7.3.2]

Determine the required area of flexural reinforcement to satisfy the Strength I Load Combination.

Flexural Resistance

Assume a rectangular stress distribution and solve for the required area of reinforcing based on M_u and d. Also assume a resistance factor of 0.9.

For $f'_c = 4.0$ ksi , $\beta_1 = 0.85$

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

$$M_u = \phi \cdot A_s \cdot f_y \cdot \left(d - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b}\right)$$

$$M_u = 0.90 \cdot A_s \cdot (60) \cdot \left(d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12}\right) \cdot \left(\frac{1}{12}\right)$$

$$3.309 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u = 0$$

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618}$$

The "d" value used in positive moment regions does not include the 2 inch wearing course.

$$d_{int} = 21 - 2 - 1.5 - 0.5 \cdot (1.0) = 17.00 \text{ in}$$
$$d_{ext} = 21 - 2 - 1.5 - 0.5 \cdot (1.27) = 16.86 \text{ in}$$

Trial reinforcement information for Span Points 1.4 and 2.5 are provided in the following table. After evaluating the areas of steel required, a layout based on a 5 inch base dimension was selected for the interior strip.

Trial Bottom Longitudinal Reinforcement

Span Point	Interior Strip					Exterior Strip				
	M_u	d	A_s (req)	Trial Bars	A_s (prov)	M_u	d	A_s (req)	Trial Bars	A_s (prov)
1.4	133	17.00	1.89	#8 @ 5	1.90	167	16.86	2.47	#10 @ 6	2.54
2.5	119	17.00	1.68	#8 @ 5	1.90	149	16.86	2.17	#10 @ 6	2.54

[5.5.4.2.1]

Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the Whitney stress block.

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{1.90 \cdot 60}{0.85 \cdot 4 \cdot 12} = 2.79$$

The depth of the section in compression is:

$$c = \frac{a}{\beta_1} = \frac{2.79}{0.85} = 3.28$$

$$\phi = 0.65 + 0.15 \cdot \left(\frac{d_t}{c} - 1\right) = 0.65 + 0.15 \cdot \left(\frac{17.00}{3.28} - 1\right) = 1.28 > 0.9$$

Therefore, $\varphi = 0.9$

[5.7.3.4]

Crack Control

To ensure that cracking is limited to small cracks that are well distributed, a limit is placed on the spacing of the reinforcing steel. LRFD Equation 5.7.3.4-1 defines the maximum spacing permitted:

$$s \le s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

At Span Point 1.4 the Service I positive moment is 86 kip-ft.

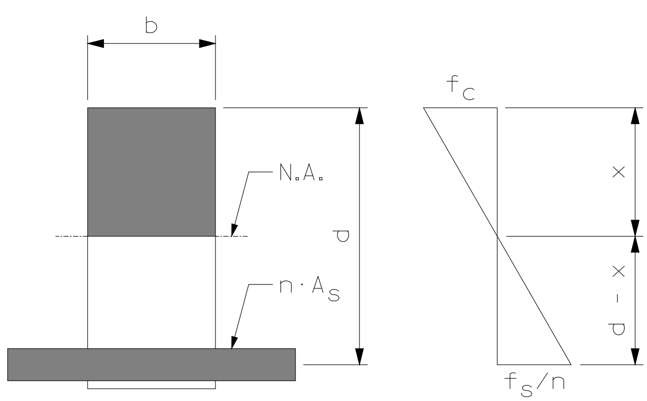
The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. To simplify the calculations, the section is assumed to be singly reinforced.

[5.4.2.4 & 5.7.1]

$$n = \frac{E_S}{E_C} = \frac{29{,}000}{33{,}000 \cdot (0.145)^{1.5} \cdot \sqrt{4.0}} = 7.96$$ <u>Use n = 8</u>

$$n \cdot A_S = 8 \cdot (1.90) = 15.2 \text{ in}^2$$

Determine the location of the neutral axis:



$$b \cdot x \cdot \frac{x}{2} = n \cdot A_S \cdot (d - x)$$

$$\frac{(12) \cdot x^2}{2} = 15.2 \cdot (17.0 - x)$$ solving, x = 5.42 in

Determine the lever arm between service load flexural force components.

$$j \cdot d = d - \frac{x}{3} = 17.0 - \frac{5.42}{3} = 15.2 \text{ in}$$

Compute the stress in the reinforcement.

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{86 \cdot 12}{1.90 \cdot (15.2)} = 35.7 \text{ ksi}$$

For $d_c = 2.00$ in (1.5 in + $^1/_2$ of #25 bar)

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2}{0.7 \cdot (21 - 2)} = 1.15$$

Use $\gamma_e = 0.75$

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.15 \cdot 35.7} - 2 \cdot 2 = 8.8 \text{ in} > 5 \text{ in}$$ OK

[5.5.3]

Fatigue

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

[Table 3.4.1-1] U = 1.50 (LL + IM)

[3.6.2.1] IM = 15%

At Span Point 1.4 the one lane fatigue moments are:
Maximum positive moment = 299 kip-ft
Maximum negative moment = -98 kip-ft

Multiplying the one lane moments by the appropriate load factor, dynamic load allowance, and distribution factor results in the following fatigue moments:

$$\text{Fatigue LL } M_{max} = 299 \cdot (1.50) \cdot 1.15 \cdot 0.052 = 26.8 \text{ kip-ft}$$
$$\text{Fatigue LL } M_{min} = -98 \cdot (1.50) \cdot 1.15 \cdot 0.052 = -8.8 \text{ kip-ft}$$

The unfactored dead load moment at Span Point 1.4 is 34.3 kip-ft.

The moments on the cross section when fatigue loading is applied are:
Maximum moment = 34.3 + 26.8 = 61.1 kip-ft
Minimum moment = 34.3 – 8.8 = 25.5 kip-ft

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{61.1 \cdot 12}{1.90 \cdot (15.2)} = 25.4 \text{ ksi}$$

For the minimum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{25.5 \cdot 12}{1.90 \cdot (15.2)} = 10.6 \text{ ksi}$$

The stress range in the reinforcement (f_f) is the difference between the two stresses

$$f_f = (25.4 - 10.6) = 14.8 \text{ ksi}$$

[5.5.3.2] The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

$$f_{f(max)} = 24 - 0.33 \cdot f_{min} = 24 - 033 \cdot (10.6)$$

$$= 20.5 > 14.8 \text{ ksi} \qquad \underline{\text{OK}}$$

[5.7.3.3.2] **Check Minimum Reinforcement**

To prevent a brittle failure, adequate flexural reinforcement needs to be placed in the cross section. For this check, the thickness of the slab including the wearing course is used to be conservative.

$$f_r = 0.37 \cdot \sqrt{f'_c} = 0.37 \cdot \sqrt{4} = 0.74 \text{ ksi}$$

$$t = 21.0 \text{ in}$$

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (21)^3 = 9261 \text{ in}^4$$

$$y_t = 10.5 \text{ in}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.74 \cdot 9261}{10.5 \cdot (12)} = 54.4 \text{ kip-ft}$$

$$1.2 \ M_{cr} = 65.3 \text{ kip-ft}$$

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

$$M_r = 0.9 \cdot (1.90) \cdot (60) \cdot \left(17.0 - \frac{2.79}{2}\right) \cdot \frac{1}{12}$$

$$M_r = 133.4 \text{ kip-ft} > 1.2 \ M_{cr} = 65.3 \text{ kip-ft} \qquad \underline{\text{OK}}$$

<u>Use #8 bars at 5 inches at Span Point 1.4</u>

[5.11.1.2.1] **Bar Cutoff Location**

[5.11.1.2.2] Determine the location where the 5 inch spacing can be increased to 10 inches. Assume that the bars will be dropped in non-haunched regions of the span. The moment capacity of #8 bars at 10 inches (A_s = 0.95 in^2) for positive flexure is:

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

$$M_r = 0.9 \cdot (0.95) \cdot (60) \cdot \left(17.0 - \frac{0.95 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)}\right) \cdot \frac{1}{12} = 69.7 \text{ kip-ft}$$

For the interior strip, the positive bending moments are:

Span Point	$M_{Strength\ I}$ (kip-ft)/ft	$M_{Service\ I}$ (kip-ft)/ft
1.6	99	62
1.7	58	32
2.2	28	8
2.3	74	44

Knowing that span points are 4.4 feet apart in Span 1 and 5.5 feet apart in Span 2, the drop point locations which meet the positive Strength I bending moment of 69.7 kip-ft can be found.

For Span 1, interpolate between Span Points 1.6 and 1.7:

$$1.6 + \left(\frac{99 - 69.7}{99 - 58}\right) \cdot 0.1 = 1.67 \text{ or 14.5 ft from Pier 1 centerline.}$$

For Span 2, interpolate between Span Points 2.2 and 2.3:

$$2.2 + \left(\frac{69.7 - 28}{74 - 28}\right) \cdot 0.1 = 2.29 \text{ or 16.0 ft from Pier 1 centerline.}$$

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

[5.7.3.4] For #8 bars @ 10", ($A_s = 0.95 \text{ in}^2$), and d_c=2 in :

$$s = 10 \text{ in} \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

where

$$f_{ss} = \frac{M_{drop}}{A_s \cdot j \cdot d}$$

Determine neutral axis:

$$n \cdot A_s = 8 \cdot 0.95 = 7.60 \ \text{in}^2$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{12 \cdot x^2}{2} = 7.60 \cdot (17.00 - x) \qquad \text{solving, } x = 4.05 \text{ in}$$

Then $j \cdot d = d - \frac{x}{3} = 17.00 - \frac{4.05}{3} = 15.65 \ \text{in}$

$$s = 10 \ \text{in} \le \frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{M_{drop}}{A_s \cdot j \cdot d}} - 2 \cdot d_c$$

Solving for M_{drop}:

$$M_{drop} = \frac{700 \cdot \gamma_e \cdot A_s \cdot j \cdot d}{\beta_s \cdot (s + 2 \cdot d_c)} = \frac{700 \cdot 0.75 \cdot 0.95 \cdot 15.65}{1.15 \cdot (10 + 2 \cdot 2)} \cdot \frac{1}{12} = 40.4 \ \text{kip} - \text{ft}$$

Interpolate to determine span point location of drop point:

For Span 1:

$$1.6 + \left(\frac{62 - 40.4}{62 - 32}\right) \cdot 0.1 = 1.67 \ \text{ or 14.5 ft from Pier 1 centerline.}$$

For Span 2:

$$2.2 + \left(\frac{40.4 - 8}{44 - 8}\right) \cdot 0.1 = 2.29 \ \text{ or 16.0 ft from Pier 1 centerline.}$$

Therefore, the drop point locations based on crack control match those based on strength.

By inspection, the fatigue stress range check and the minimum reinforcement check are satisfied.

[5.11.1.2.1]

Due to the uncertainty associated with the design moments, the reinforcement cannot be terminated at the theoretical drop point. It must be carried beyond the theoretical point by the greater of: the depth of the member, 15 times the nominal diameter of the bar, or $^{1}/_{20}$ of the clear span.

The required extension L_{ext1} for Span 1 is:

$$L_{ext1} = d = 17.0 \text{ in}$$

or

$$L_{ext1} = 15 \cdot d_b = 15 \cdot 1.00 = 15.0 \text{ in}$$

or

$$L_{ext1} = \frac{1}{20} \cdot (44 \cdot 12) = 26.4 \text{ in}$$ <u>GOVERNS</u>

The required extension L_{ext2} for Span 2 is:

$$L_{ext2} = \frac{1}{20} \cdot (55 \cdot 12) = 33.0 \text{ in}$$

Adding the extension length to the theoretical distance from the pier at which the bars can be dropped results in the following cutoff locations:

For Span 1: $14.5 - \frac{26.4}{12} = 12.3 \text{ ft}$ <u>Use 12'-0"</u>

For Span 2: $16.0 - \frac{33.0}{12} = 13.25 \text{ ft}$ <u>Use 13'-0"</u>

By continuing half of the reinforcement for the entire length of the bridge, LRFD Article 5.11.1.2.2 is satisfied.

[5.8.3.5]

Check Longitudinal Reinforcement

Check the minimum longitudinal reinforcement requirements at the abutments, assuming that a diagonal crack would start at the inside edge of the bearing area.

The slab sits on a 2'-10" wide integral abutment.

For $\theta = 45°$ determine the length from the end of the slab, L_{crack}, at which a diagonal crack will intersect the bottom longitudinal reinforcement (#8 bars @ 5"):

$$L_{crack} = 2.83 + \left(\frac{2.00}{12}\right)\cot(45°) = 3.00 \text{ ft} = 36.00 \text{ in}$$

From Figure 5.2.2.2 of this manual, the development length for #8 bars @ 5" with 1.5" cover is:

$$\ell_{d25} = 3'\text{-}9" = 45"$$

Then the tensile resistance of the longitudinal bars at the crack location

$$T_r = f_y \cdot A_s = \frac{L_c - (\text{end cover})}{\ell_{d25}}$$

$$= 60 \cdot 1.90 \cdot \left(\frac{36.0 - (\sim 3.5)}{45.0}\right) = 82.3 \text{ kips}$$

The force to be resisted is:

$$T = \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

$$= \left(\frac{13.6}{0.9} - 0.5 \cdot 0 - 0\right)\cot 45°$$

$$= 15.1 \text{ kips} < 82.3 \text{ kips}$$ <u>OK</u>

Note that LRFD C5.8.3.5 states that V_u may be taken at $0.5 d_v \cot\theta$ or d_v away from the face of support. For simplicity, the value for V_u at the abutment centerline of bearing was used in the equation above.

L. Design Negative Moment Reinforcement
[5.7.2.2]
[5.7.3.2]

Determine the required area of flexural reinforcement to satisfy the Strength I Load Combination.

Flexural Resistance

Assume a rectangular stress distribution and solve for the required area of reinforcing based on M_u and d.

Use the same general equation developed for the positive moment reinforcement.

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.23 \cdot M_u}}{6.618}$$

$$d_{int} = 36 - 3 - 0.5 \cdot (1.0) = 32.50 \text{ in}$$

$$d_{ext} = 36 - 3 - 0.5 \cdot (1.128) = 32.44 \text{ in}$$

The required area of steel and trial reinforcement is presented in the following table.

Trial Top Longitudinal Reinforcement

Span Point	Interior Strip					Exterior Strip				
	M_u	d	A_s (req)	Trial Bars	A_s (prov)	M_u	d	A_s (req)	Trial Bars	A_s (prov)
2.0	-215	32.50	1.52	#8 @ 5	1.90	-263	32.44	1.88	#8 @ 6	2.00

By inspection, the resistance factor is equal to 0.9

[5.7.3.4]

Crack Control
At Span Point 2.0 the Service I moment is −149 kip-ft

Similar to the positive moment sections, the stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. For this check, the section is assumed to be singly reinforced.

$$n \cdot A_s = 8 \cdot (1.90) = 15.2 \text{ in}^2$$

Determine the location of the neutral axis:

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{(12) \cdot x^2}{2} = 15.2 \cdot (32.5 - x) \qquad \text{solving, } x = 7.90 \text{ in}$$

Determine the lever arm between service load flexural force components.

$$j \cdot d = d - \frac{x}{3} = 32.5 - \frac{7.90}{3} = 29.9 \text{ in}$$

Compute the stress in the reinforcement.

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{149 \cdot 12}{1.90 \cdot (29.9)} = 31.5 \text{ ksi}$$

For $d_c = 2.50$ in (2.0 in max cover + 1/2 of #8 bar)

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.5}{0.7 \cdot (36 - 2.5)} = 1.11$$

Use $\gamma_e = 0.75$:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.11 \cdot 31.5} - 2 \cdot 2.5 = 10.0 \text{ in} > 5 \text{ in} \quad \underline{\text{OK}}$$

[5.5.3]

Fatigue

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

[Table 3.4.1-1]

U = 1.5 (LL + IM)
IM = 15%

[3.6.2.1]

At Span Point 2.0 the one lane fatigue moments are:
Maximum positive moment = 75 kip-ft
Maximum negative moment = -387 kip-ft

Multiplying the one lane moments by the appropriate load factor, dynamic load allowance, and distribution factor results in the following fatigue moments:

Fatigue LL $M_{max} = 75 \cdot (1.50) \cdot 1.15 \cdot 0.052 = 6.7$ kip-ft

Fatigue LL $M_{min} = -387 \cdot (1.50) \cdot 1.15 \cdot 0.052 = -34.7$ kip-ft

The unfactored dead load moment at Span Point 2.0 is –90.9 kip-ft.

The moments on the cross section when fatigue loading is applied are:
Maximum moment = -90.9 + 6.7 = -84.2 kip-ft
Minimum moment = -90.9 – 34.7 = -125.6 kip-ft

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{84.2 \cdot 12}{1.90 \cdot (29.9)} = 17.8 \text{ ksi}$$

For the minimum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{125.6 \cdot 12}{1.90 \cdot (29.9)} = 26.5 \text{ ksi}$$

The stress range in the reinforcement (f_f) is the difference between the two stresses.

$$f_f = (26.5 - 17.8) = 8.7 \text{ ksi}$$

[5.5.3.2]

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

$$f_{f(max)} = 24 - 0.33 \cdot f_{min} = 24 - 0.33 \cdot (17.8)$$

$$= 18.1 \text{ ksi} > 8.7 \text{ ksi} \qquad \underline{\text{OK}}$$

[5.7.3.3.2]

Check Minimum Reinforcement

To prevent a brittle failure, adequate flexural reinforcement needs to be placed in the cross section.

$$f_r = 0.74 \text{ ksi}$$

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (36)^3 = 46{,}656 \text{ in}^4$$

$$y_t = 18.0 \text{ in}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.74 \cdot 46{,}656}{18.0 \cdot (12)} = 159.8 \text{ kip-ft}$$

$$1.2 \ M_{cr} = 191.8 \text{ kip-ft}$$

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

$$M_r = 0.9 \cdot (1.90) \cdot (60) \cdot \left(32.5 - \frac{2.79}{2}\right) \cdot \frac{1}{12}$$

$$= 265.9 \text{ kip-ft} > 1.2 \ M_{cr} = 191.8 \text{ kip-ft} \qquad \underline{\text{OK}}$$

<u>Use #8 bars at 5 inches at Span Point 2.0</u>

[5.11.1.2.1]
[5.11.1.2.3]

Bar Cutoff Location

Determine the location where the 5 inch spacing can be increased to 10 inches. Assume that the bars will be dropped in non-haunched regions of the span. The moment capacity of #8 bars at 10 inches $(A_s = 0.95 \text{ in}^2)$ for negative flexure is:

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

$$= 0.9 \cdot (0.95) \cdot (60) \cdot \left[17.5 - \frac{0.95 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)}\right] \cdot \frac{1}{12} = 71.8 \qquad \text{kip-ft}$$

For the interior strip, the negative bending moments are:

Span Point	$M_{Strength\ I}$ (kip-ft)/ft	$M_{Service\ I}$ (kip-ft)/ft
1.6	-35	-12
1.7	-61	-35
1.8	-98	-63
1.9	-148	-100
2.0	-215	-149
2.1	-128	-87
2.2	-73	-46
2.3	-37	-18

Knowing that span points are 4.4 feet apart in Span 1 and 5.5 feet apart in Span 2, the drop point locations which meet the Strength I negative bending moment of 71.8 kip-ft can be found.

For Span 1, interpolate between Span Points 1.7 and 1.8:

$$1.7 + \left(\frac{71.8 - 61}{98 - 61}\right) \cdot 0.1 = 1.73 \text{ or 11.9 ft from Pier 1 centerline.}$$

For Span 2, interpolate between Span Points 2.2 and 2.3:

$$2.2 + \left(\frac{73 - 71.8}{73 - 37}\right) \cdot 0.1 = 2.20 \text{ or 11.0 ft from Pier 1 centerline.}$$

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

[5.7.3.4]

For #8 bars @ 10", ($A_S = 0.95\ in^2$), $d_c = 2.50\ in$:

Determine neutral axis:

$$n \cdot A_S = 8 \cdot 0.95 = 7.60\ in^2$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_S \cdot (d - x)$$

$$\frac{12 \cdot x^2}{2} = 7.60 \cdot (17.5 - x)$$ solving, x=4.12 in

Then $j \cdot d = d - \frac{x}{3} = 17.5 - \frac{4.12}{3} = 16.13\ in$

$$\beta_s = 1 + \frac{2.50}{0.7\ (21 - 2.50)} = 1.19$$

Solve for the moment at the drop point:

$$M_{drop} = \frac{700 \cdot \gamma_c \cdot A_s \cdot j \cdot d}{\beta_s \cdot (s + 2 \cdot d_c)} = \frac{700 \cdot 0.75 \cdot 0.95 \cdot 16.13}{1.19 \cdot (10 + 2 \cdot 2.5)} \cdot \frac{1}{12} = 37.6\ \ kip - ft$$

Interpolate to determine span point location of drop point:

For Span 1:

$$1.7 + \left(\frac{37.6 - 35}{63 - 35} \right) \cdot 0.1 = 1.71$$ or 12.8 ft from Pier 1 centerline.

For Span 2:

$$2.2 + \left(\frac{46 - 37.6}{46 - 18} \right) \cdot 0.1 = 2.23$$ or 12.7 ft from Pier 1 centerline

Therefore, the drop point locations based on crack control govern the bar cutoff locations.

By inspection, the fatigue stress range check and the minimum reinforcement check are satisfied.

[5.11.1.2.1]

Due to the uncertainty associated with the design moments, the reinforcement cannot be terminated at the theoretical drop point. It must be carried beyond the theoretical point by the greater of: the depth of the member, 15 times the nominal diameter of the bar, or $^1/_{20}$ of the clear span.

The required extension L_{ext1} for Span 1 is:

$$L_{ext1} = d = 21 - 3 - 0.5 \cdot (1.0) = 17.5 \text{ in}$$

or

$$L_{ext1} = 15 \cdot d_b = 15 \cdot 1.00 = 15.0 \text{ in}$$

or

$$L_{ext1} = \frac{1}{20} \cdot (44 \cdot 12) = 26.4 \text{ in} \qquad \underline{\text{GOVERNS}}$$

The required extension L_{ext2} for Span 2 is:

$$L_{ext2} = \frac{1}{20} \cdot (55 \cdot 12) = 33.0 \text{ in}$$

Adding the extension length to the theoretical distance from the pier at which the bars can be dropped results in the following cutoff locations from the pier:

For Span 1: $12.8 + \frac{26.4}{12} = 15.0 \text{ ft}$ <u>Use 15'-0"</u>

For Span 2: $12.7 + \frac{33.0}{12} = 15.5 \text{ ft}$ <u>Use 15'-6"</u>

By continuing half of the reinforcement for the entire length of the bridge, LRFD Article 5.11.1.2.3 is satisfied.

M. Distribution Reinforcement [5.14.4.1]

The amount of transverse reinforcement may be taken as a percentage of the main reinforcement required:

$$\frac{100}{\sqrt{L}} \leq 50\%$$

$$\frac{100}{\sqrt{44}} = 15.1\%$$

For the interior strip, the maximum reinforcement is #8 bars at 5 inches (1.90 in^2/ft). The required transverse reinforcement for load distribution is:

$$0.151 \cdot (1.90) = 0.29 \ in^2/ft$$

Use #5 @ 12", $A_s = 0.31 \ in^2$/ft for bottom transverse reinforcement.

N. Shrinkage and Temperature Reinforcement [5.10.8]

Adequate reinforcement needs to be provided in the slab to ensure that cracks from shrinkage and temperature changes are small and well distributed.

$$\text{Temperature } A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.3 \cdot 568 \cdot 21}{2 \cdot (568 + 21) \cdot 60} = 0.22 \ in^2/ft$$

(total in each direction, on each face)

$0.11 \leq A_s \leq .60$ use A_s= 0.22 in^2/ft

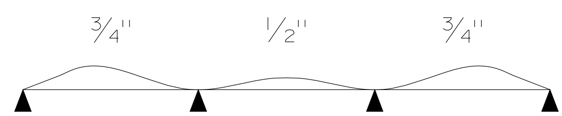
Use #5 @ 12", $A_s = 0.31 \ in^2$/ft for transverse top reinforcement.

O. Dead Load Camber

[5.7.3.6.2]

The total weight of the superstructure is used for dead load deflections. The gross moment of inertia is used and a computer analysis is used to obtain instantaneous deflections. A longtime deflection multiplier of 4.0 is used in conjunction with the gross moment of inertia. The slab is cambered upward an amount equal to the immediate deflection + $^1/_2$ of the long-term deflection. A camber diagram for the interior strip is shown below:

Instantaneous +
$^1/_2$ Long-term Camber



P. Final Reinforcement Layout

Figure 5.7.1.3 contains a plan view and Figure 5.7.1.4 contains a cross section that illustrates the reinforcement for the slab. As one would expect, the figures show that the exterior strips contain more reinforcing steel than the interior of the slab.

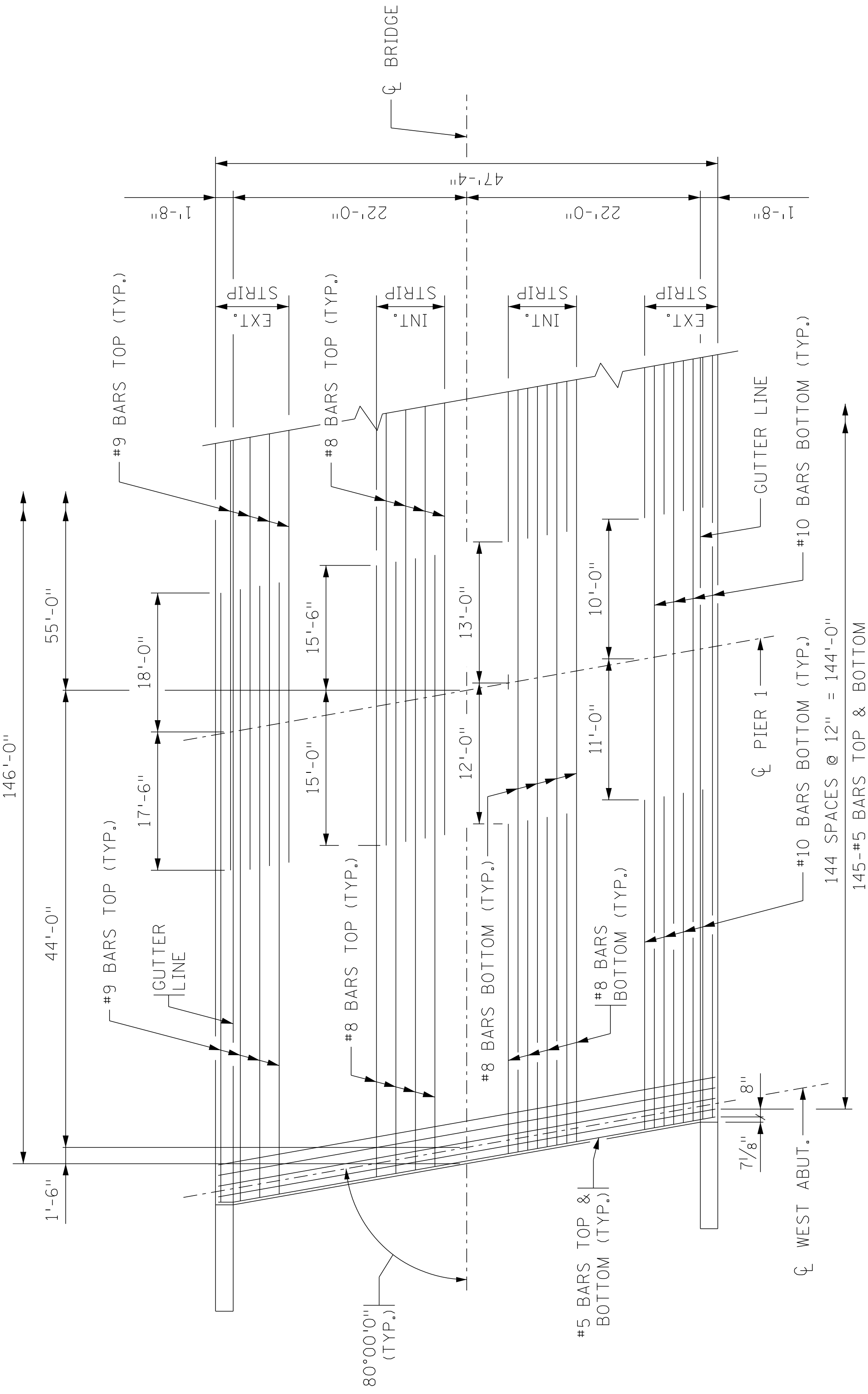
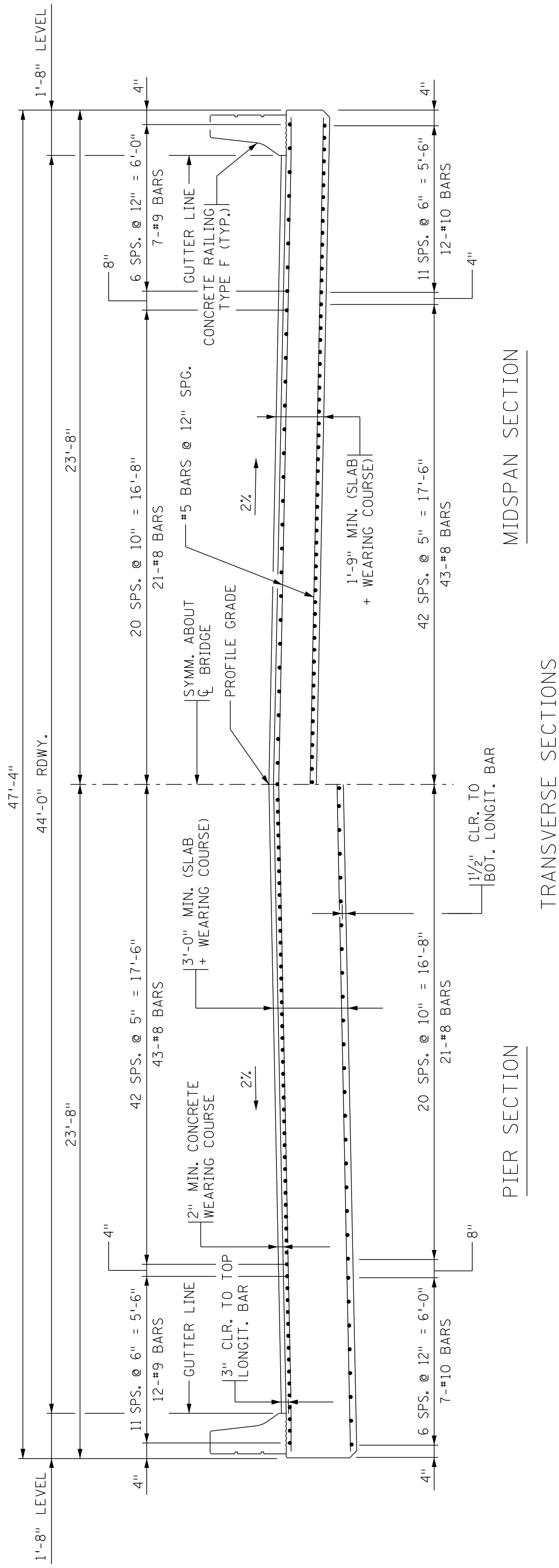

℄ BRIDGE
47'-4"
1'-8"
22'-0"
22'-0"
1'-8"
EXT. STRIP
INT. STRIP
INT. STRIP
EXT. STRIP
#9 BARS TOP (TYP.)
#8 BARS TOP (TYP.)
GUTTER LINE
#10 BARS BOTTOM (TYP.)
146'-0"
55'-0"
44'-0"
1'-6"
18'-0"
17'-6"
15'-6"
15'-0"
13'-0"
12'-0"
10'-0"
11'-0"
℄ PIER 1
#10 BARS BOTTOM (TYP.)
144 SPACES @ 12" = 144'-0"
145-#5 BARS TOP & BOTTOM
#9 BARS TOP (TYP.)
GUTTER LINE
#8 BARS TOP (TYP.)
#8 BARS BOTTOM (TYP.)
#8 BARS BOTTOM (TYP.)
8"
7 1/8"
℄ WEST ABUT.
#5 BARS TOP & BOTTOM (TYP.)
80°00'0" (TYP.)

Figure 5.7.1.3



1'-8" LEVEL
4"
47'-4"
44'-0" RDWY.
23'-8"
23'-8"
8"
6 SPS. @ 12" = 6'-0"
7-#9 BARS
GUTTER LINE
CONCRETE RAILING TYPE F (TYP.)
20 SPS. @ 10" = 16'-8"
21-#8 BARS
#5 BARS @ 12" SPG.
2%
SYMM. ABOUT ℄ BRIDGE
PROFILE GRADE
1'-9" MIN. (SLAB + WEARING COURSE)
11 SPS. @ 6" = 5'-6"
12-#10 BARS
42 SPS. @ 5" = 17'-6"
43-#8 BARS
MIDSPAN SECTION
TRANSVERSE SECTIONS
3'-0" MIN. (SLAB + WEARING COURSE)
1½" CLR. TO BOT. LONGIT. BAR
2" MIN. CONCRETE WEARING COURSE
3" CLR. TO TOP LONGIT. BAR
PIER SECTION

Figure 5.7.1.4

5.7.2 Prestressed I-Beam Design Example

This example illustrates the design of a pretensioned I-Beam for a two span bridge without skew. The 137'-0" spans are supported with MnDOT "MN63" beams. MnDOT standard details and drawings for diaphragms (B403, B814), railings (Fig. 5-397.117), and beams (Fig. 5-397.509) are to be used with this example. This example contains the design of a typical interior beam at the critical sections in positive flexure, shear, and deflection. The superstructure consists of six beams spaced at 9'-0" centers. A typical transverse superstructure section is provided in Figure 5.7.2.1. A framing plan is provided in Figure 5.7.2.2. The roadway section is composed of two 12' traffic lanes and two 12' shoulders. A Type F railing is provided on each side of the bridge and a 9" composite concrete deck is used. End diaphragms (B814) are used at each end of the bridge and interior diaphragms (B403) are used at the interior third points and at the pier.

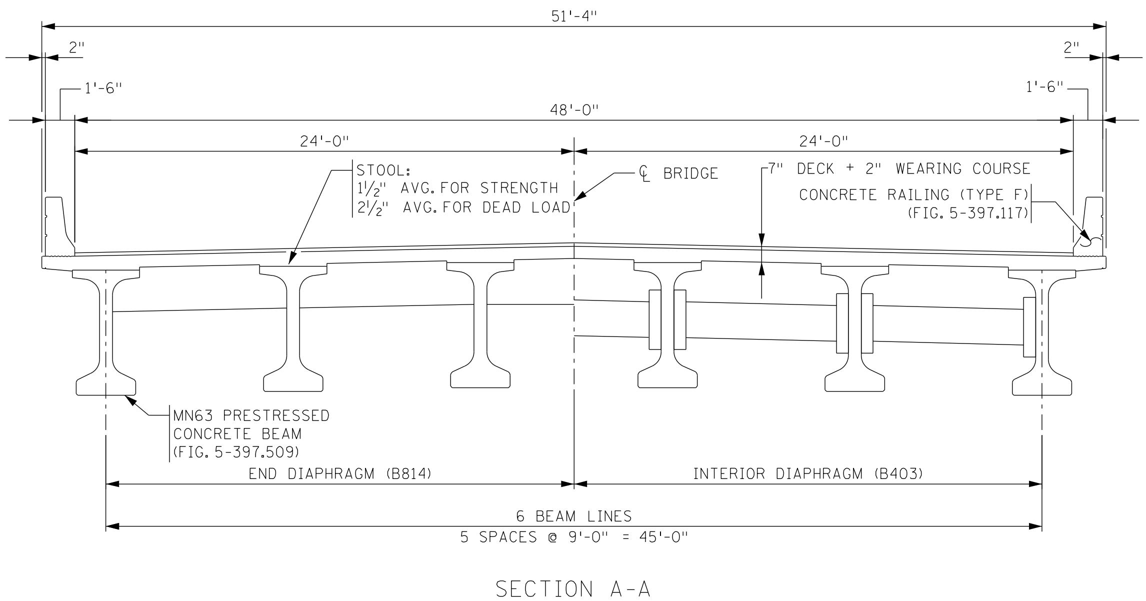


Figure 5.7.2.1

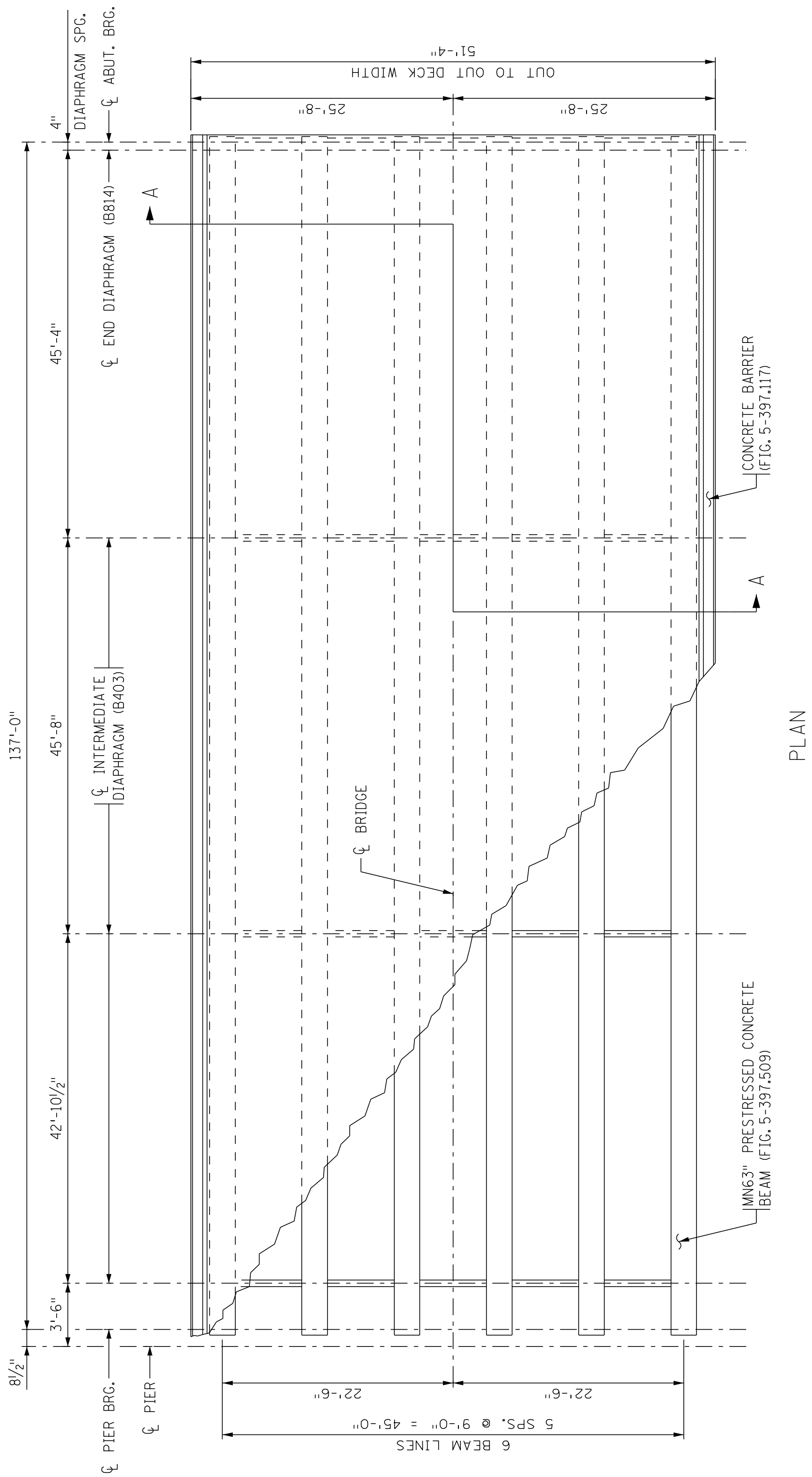


Figure 5.7.2.2

A. Materials

The modulus of elasticity for high strength concrete suggested by ACI Committee 363 is used for the beam concrete. The composite deck is assumed to have a unit weight of 0.150 kcf for dead load computations and 0.145 kcf for elastic modulus computations. The beam concrete is assumed to have a unit weight of 0.155 kcf for dead load computations.

The material and geometric parameters used in the example are shown in Table 5.7.2.1:

Table 5.7.2.1
Material Properties

Material Parameter		Prestressed Beam	Deck
Concrete	f'_{ci} at transfer	7.5 ksi	---
	f_c at 28 days	9.0 ksi	4 ksi
	E_{ci} at transfer	$\left(1265 \cdot \sqrt{f'_{ci}}\right) + 1000$ $= 4464$ ksi	---
	E_c at 28 days	$\left(1265 \cdot \sqrt{f'_c}\right) + 1000$ $= 4795$ ksi	$33{,}000 \cdot (0.145)^{1.5} \cdot \sqrt{f'_c}$ $= 3644$ ksi
Steel	f_y for rebar	60 ksi	60 ksi
	f_{pu} for strand	270 ksi	---
	E_s for rebar	29,000 ksi	29,000 ksi
	E_p for strand	28,500 ksi	---
	Strand type	0.6 inch diameter 270 ksi, low relaxation	---

B. Determine Cross-Section Properties for a Typical Interior Beam

The beams are designed to act compositely with the deck on simple spans. The deck consists of a 7 inch thick concrete slab with a 2 inch wearing course. For simplicity and in order to be conservative, the beams are designed assuming the full 9 inches of thickness is placed in a single pour. A $^1/_2$ inch of wear is assumed. A thickness of $8^1/_2$ inches is used for composite section properties. The haunch or stool is assumed to have an average thickness of $2^1/_2$ inches for dead load computations and $1^1/_2$ inches for section property computations.

[4.6.2.6.1]

The effective flange width, b_e, is equal to the average beam spacing:

$b_e = 108.0$ in

The modular ratio of the deck concrete to the beam concrete is:

$$n = \frac{E_{cdeck}}{E_{cbeam}} = \frac{3644}{4795} = 0.76$$

This results in a transformed effective flange width of:

$$b_{etrans} = 0.76 \cdot (108.0) = 82.1 \text{ in}$$

Properties for an interior beam are given in Table 5.7.2.2.

Table 5.7.2.2
Cross-Section Properties

Parameter	Non-composite Section	Composite Section
Height of section, h	63 in	73.0 in
Deck thickness	---	8.5 in
Average stool thickness	---	1.5 in (section properties) 2.5 in (dead load)
Effective flange width, b_e	---	108.0 in (deck concrete) 82.1 in (beam concrete)
Area, A	807 in^2	1543 in^2
Moment of inertia, I	422,570 in^4	1,034,168 in^4
Centroidal axis height, y	28.80 in	47.74 in
Bottom section modulus, S_b	14,673 in^3	21,664 in^3
Top section modulus, S_t	12,356 in^3	53,862 in^3 (beam concrete)
Top of prestressed beam, S_{tbm}	12,356 in^3	67,753 in^3

C. Shear Forces and Bending Moments

Three load combinations will be considered; Strength I, Service I, and Service III. As a result of the simple span configuration, only maximum γ_p values need to be considered.

Load effects related to settlement, thermal effects, water load, or stream pressure will not be considered.

Assume that traffic can be positioned anywhere between the barriers.

$$\text{Number of design lanes} = \frac{\text{distance between barriers}}{\text{design lane width}} = \frac{48}{12} = 4$$

[3.6.2] Dynamic load allowance $IM = 33\%$

[4.6.2.2] **1. Determine Live Load Distribution Factors**

Designers should note that the approximate distribution factor equations include the multiple presence factors.

[4.6.2.2.2] **Distribution Factor for Moment – Interior Beams**

LRFD Table 4.6.2.2.1-1 lists the common deck superstructure types for which approximate live load distribution equations have been assembled. The cross section for this design example is Type (k). To ensure that the approximate distribution equations can be used, several parameters need to be checked.

1) $3.5\ \text{ft} \leq \text{beam spacing} = 9.0\ \text{ft} \leq 16.0\ \text{ft}$ <u>OK</u>
2) $4.5\ \text{in} \leq \text{slab thickness} = 8.5\ \text{in} \leq 12.0\ \text{in}$ <u>OK</u>
3) $20\ \text{ft} \leq \text{span length} = 130\ \text{ft} \leq 240\ \text{ft}$ <u>OK</u>
4) $4 \leq \text{number of beams} = 6$ <u>OK</u>

The distribution factor equations use a K_g factor that is defined in LRFD Article 4.6.2.2.1.

$$\eta = \frac{E_{c\ beam}}{E_{c\ deck}} = \frac{4795}{3644} = 1.316$$

$$e_g = (\text{deck centroid}) - (\text{beam centroid}) = 68.75 - 28.80 = 39.95\ \text{in}$$

$$K_g = \eta \cdot \left[I + A \cdot (e_g)^2\right] = 1.316 \cdot \left[422{,}570 + 807 \cdot (39.95)^2\right] = 2.25\ \times 10^6$$

One design lane loaded:

$$gM = 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K}{12 \cdot L \cdot t_s^{\,3}}\right)^{0.1}$$

$$gM = 0.06 + \left(\frac{9.0}{14}\right)^{0.4} \cdot \left(\frac{9.0}{137}\right)^{0.3} \cdot \left(\frac{2.25\ \times 10^6}{12 \cdot 137 \cdot 8.5^3}\right)^{0.1}$$

$$gM = 0.461\ \text{lanes/beam}$$

Two or more design lanes loaded:

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

$$gM = 0.075 + \left(\frac{9.0}{9.5}\right)^{0.6} \cdot \left(\frac{9.0}{137}\right)^{0.2} \cdot \left(\frac{2.25 \ \text{x} \ 10^6}{12 \cdot 137 \cdot 8.5^3}\right)^{0.1}$$

$$gM = 0.683 \ \text{lanes/beam}$$

[4.6.2.2.2d]

Distribution Factor for Moment - Exterior Beams

LRFD Table 4.6.2.2.2d-1 contains the approximate distribution factor equations for exterior beams. Type (k) cross-sections have a deck dimension check to ensure that the approximate equations are valid.

The distance from the inside face of barrier to the centerline of the fascia beam is defined as d_e. For the example this distance is:

$$d_e = 24 - (2.5 \cdot 9.0) = 1.50 \ \text{ft}$$

The check to use the approximate equations is:

$$-1.0 \ \text{ft} \le d_e = 1.50 \ \text{ft} \le 5.5 \ \text{ft} \qquad \underline{\text{OK}}$$

One design lane loaded:

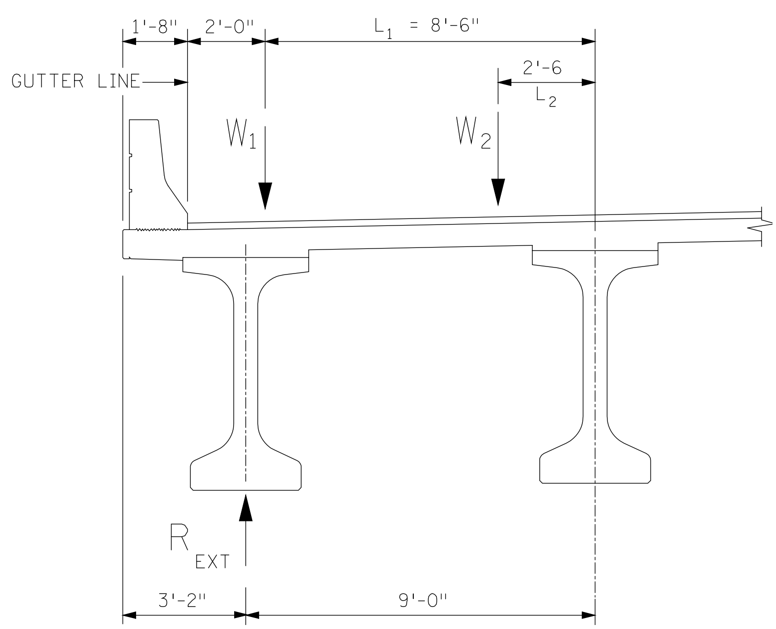


Figure 5.7.2.3

Use the lever rule to determine the live load distribution factor for one lane.

The exterior beam live load distribution factor is found by determining the exterior beam reaction:

$$W_1 = W_2 = 0.5 \text{ lanes}$$

$$gM = 1.2 \cdot \left(\frac{W_1 \cdot L_1 + W_2 \cdot L_2}{S} \right) = 1.2 \cdot \left(\frac{0.5 \cdot 8.5 + 0.5 \cdot 2.5}{9.0} \right)$$

$$gM = 0.733 \text{ lanes/beam}$$

Two or more design lanes loaded:
The distribution factor is equal to the factor "e" multiplied by the interior girder distribution factor for two or more lanes

$$e = 0.77 + \left(\frac{d_e}{9.1} \right) = 0.77 + \left(\frac{1.5}{9.1} \right) = 0.935$$

$$gM = e \cdot g_{int} = 0.935 \cdot 0.683 = 0.639 \text{ lanes/beam}$$

[4.6.2.2.2e] **Skew Factor**

No correction is necessary for a skew angle of zero.

[4.6.2.2.3] **Distribution Factor for Shear – Interior Beams**

[4.6.2.2.3a] LRFD Table 4.6.2.2.3a-1 can be used.

One design lane loaded:

$$gV = 0.36 + \left(\frac{S}{25.0} \right) = 0.36 + \left(\frac{9.0}{25.0} \right) = 0.720 \text{ lanes/beam}$$

Two or more design lanes loaded:

$$gV = 0.2 + \left(\frac{S}{12} \right) - \left(\frac{S}{35} \right)^2 = 0.2 + \left(\frac{9.0}{12} \right) - \left(\frac{9.0}{35} \right)^2 = 0.884 \text{ lanes/beam}$$

[4.6.2.2.3b] **Distribution Factor for Shear – Exterior Beams**

One Design Lane Loaded:
Use the lever rule, which results in the same factor that was computed for flexure and is equal to 0.733 lanes/beam

Two or more design lanes loaded:

$$e = 0.6 + \left(\frac{d_e}{10} \right) = 0.6 + \left(\frac{1.5}{10} \right) = 0.750$$

The exterior beam shear distribution factor for two or more design lanes is determined by modifying the interior distribution factor:

$$gV = e \cdot g_{int} = 0.750 \cdot 0.884 = 0.663 \text{ lanes/beam}$$

[4.6.2.2.3c]

Skew Factor

No correction is necessary for a skew angle of zero.

[2.5.2.6.2]
[Table 3.6.1.1.2-1]

Distribution Factor for Deflection

The distribution factor for checking live load deflections assumes that the entire cross section participates in resisting the live load. The minimum Multiple Presence Factor (MPF) used by MnDOT when checking live load deflection is 0.85. The deflection distribution factor is:

$$gD = \frac{(\text{\# of lanes}) \cdot (\text{MPF})}{(\text{\# of beam lines})} = \frac{4 \cdot 0.85}{6} = 0.567 \text{ lanes/beam}$$

Table 5.7.2.3 contains a summary of the live load distribution factors.

Table 5.7.2.3
Distribution Factor Summary (lanes per beam)

Loading		Flexure	Shear
Interior Beam	One Design Lane	0.461	0.720
	Two or More Design Lanes	0.683	0.884
	Deflection	0.567	-
Exterior Beam	One Design Lane	0.733	0.733
	Two or More Design Lanes	0.639	0.663
	Deflection	0.567	-

[1.3.3 – 1.3.5]

2. Load Modifiers

The following load modifiers will be used for this example:

		Strength	Service	Fatigue
Ductility	η_D	1.0	1.0	1.0
Redundancy	η_R	1.0	1.0	1.0
Importance	η_I	1.0	n/a	n/a
	$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0	1.0

3. Dead and Live Load Summary

$$\text{Beam Selfweight} = (807/144)\cdot\left(0.155\ \text{kip/ft}^3\right) = 0.869\ \text{kip/ft}$$

$$\text{Stool Weight} = (2.83\ \text{ft})\cdot(0.208\ \text{ft})\cdot\left(0.150\ \text{kip/ft}^3\right) = 0.088\ \text{kip/ft}$$

$$\text{Deck Weight} = (9.0\ \text{ft})\cdot(0.75\ \text{ft})\cdot\left(0.150\ \text{kip/ft}^3\right) = 1.013\ \text{kip/ft}$$

$$\text{Future Wearing Surface} = \left(0.020\ \text{kip/ft}^2\right)\cdot(48\ \text{ft})\cdot(1/6) = 0.160\ \text{kip/ft}$$

$$\text{Barrier Weight} = 2\cdot(0.439\ \text{kip/ft})\cdot(1/6) = 0.146\ \text{kip/ft}$$

$$\text{Diaphragm Weight} \cong (9.0)\cdot(0.0427)$$

$$+\,2\cdot(1.75)\cdot(1.0)\cdot\left(\frac{0.375}{12}\right)\cdot(0.490) = 0.44\ \text{kip}$$

The bending moments and shears for the dead and live loads were obtained with a line girder model of the bridge. They are summarized in Tables 5.7.2.4 and 5.7.2.5.

Table 5.7.2.4
Shear Force Summary (kips/beam)

Load Type/Combination		Brg CL (0.0')	Brg Face (0.63')	Trans Point (2.38')	Critical Shear Point (5.03')	0.1 Span Point (13.7')	0.2 Span Point (27.4')	0.3 Span Point (41.1')	0.4 Span Point (54.8')	0.5 Span Point (68.5')
Dead Loads	Selfweight	60	59	57	55	48	36	24	12	0
	Stool	6	6	6	6	5	4	2	1	0
	Deck	69	69	67	64	56	42	28	14	0
	FWS	11	11	11	10	9	7	4	2	0
	Barrier	10	10	10	9	8	6	4	2	0
	Diaphragms	0	0	0	0	0	0	0	0	0
	Total	156	155	151	144	126	95	62	31	0
Live Loads	Uniform Lane	39	39	37	36	31	25	19	14	10
	Truck with DLA	78	78	77	75	70	62	53	45	36
	Total	117	117	114	111	101	87	72	59	46
Strength I Load Comb $(1.25\cdot DL + 1.75\cdot LL)$		401	399	388	376	335	269	205	142	81
Service I Load Comb $(1.00\cdot DL + 1.00\cdot LL)$		273	272	265	255	227	182	134	90	46
Service III Load Comb $(1.00\cdot DL + 0.80\cdot LL)$		250	249	242	233	207	165	120	78	37

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

Load Type/Combination			Brg CL (0.0')	Brg Face (0.63')	Trans Point (2.38')	Critical Shear Point (5.03')	0.1 Span Point (13.7')	0.2 Span Point (27.4')	0.3 Span Point (41.1')	0.4 Span Point* (54.8')	0.5 Span Point (68.5')
Dead Loads	DC1	Selfweight	0	37	139**	288	734	1305	1713	1957	2039
		Stool	0	4	14	29	74	132	173	198	206
		Deck	0	43	162	336	856	1521	1996	2282	2377
		Diaphragms	0	0	1	2	6	12	18	20	20
		Total DC1	0	84	316	655	1670	2970	3900	4457	4642
	DC2	Barrier	0	6	23	48	123	219	288	329	343
		FWS	0	7	26	53	135	240	315	360	375
		Total DC2	0	13	49	101	258	459	603	689	718
	Total (DC1+DC2)		0	97	365	756	1928	3429	4503	5146	5360
Live Loads	Uniform Lane		0	19	70	145	369	656	861	985	1026
	Truck with DLA		0	38	142	294	746	1312	1699	1927	1986
	Total		0	57	212	439	1115	1968	2560	2912	3012
Strength I - Load Comb $(1.25 \cdot DL + 1.75 \cdot LL)$			0	221	827	1713	4361	7730	10109	11529	11971
Service I - Load Comb $(1.00 \cdot DL + 1.00 \cdot LL)$			0	154	577	1195	3042	5397	7063	8058	8372
Service III – Load Comb $(1.00 \cdot DL + 0.80 \cdot LL)$			0	143	535	1107	2820	5003	6551	7476	7770

* Drape point for strands.
** Beam selfweight at strand release = 176 k-ft
Beam selfweight at erection on bearings = 139 k-ft

D. Design Prestressing

Typically the tension at the bottom of the beam at midspan dictates the required level of prestressing.

1. Estimate Required Prestress

Use the Service III load combination

Bottom of beam stress:

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

$$= \left(\frac{4642 \cdot 12}{14{,}673}\right) + \left(\frac{718 \cdot 12}{21{,}664}\right) + \left(\frac{3012 \cdot 12 \cdot 0.8}{21{,}664}\right) = 5.53 \text{ ksi}$$

As a starting point, the total prestress losses will be assumed to be 25%. This results in an effective prestress of

$$f_{pe} = 0.75 \cdot f_{pu} \cdot (1 - 0.25) = 0.75 \cdot 270 \cdot 0.75 = 151.9 \ \text{ksi}$$

Strands are typically placed on a 2" grid. The bottom flange of an "MN63" beam can hold a maximum of 54 strands. The centroid of a 54 strand pattern would be

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

$$= \left[\frac{12 \cdot (2 + 4 + 6) + (6 \cdot 8) + 2 \cdot (3 + 5 + 7 + 9 + 11 + 13)}{54}\right] = 5.33 \ \text{in}$$

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

$$e_{54} = y_g - 5.33 = 28.80 - 5.33 = 23.47 \ \text{in}$$

The area of a 0.6" diameter 7-wire strand is $0.217 \ \text{in}^2$

The axial compression produced by the prestressing strands is

$$P = A_s \cdot f_{pe} = (\#\text{ of strands}) \cdot (0.217) \cdot (151.9)$$

The internal moment produced by the prestressing strands is

$$M_{p/s} = A_s \cdot f_{pe} \cdot e_{54} = (\#\text{ of strands}) \cdot 0.217 \cdot 151.9 \cdot 23.47$$

The allowable tension after losses $= 0.19 \cdot \sqrt{f'_c} = 0.19 \cdot \sqrt{9} = 0.57 \ \text{ksi}$

This moment and the axial compression from the prestress must reduce the bottom flange tension from 5.53 ksi tension to a tension of 0.57 ksi or

Required $f_{pe} = 5.53 - 0.57 = 4.96 \ \text{ksi}$

Using the fact that $f_{pe} = \dfrac{P}{A} + \dfrac{M}{S}$

One can estimate the required number of strands:

$$\frac{\left[\dfrac{4.96}{\left(\dfrac{1}{A_g}+\dfrac{23.47}{S_{gb}}\right)}\right]}{(0.217\cdot 151.9)}=\frac{\left[\dfrac{4.96}{\left(\dfrac{1}{807}+\dfrac{23.47}{14{,}673}\right)}\right]}{(0.217\cdot 151.9)}=53.0\ \text{strands}$$

Try a strand pattern with 54 strands.

After reviewing Bridge Details Part II Figure 5-397.509, a 54 strand draped strand pattern was selected. Also, the drape points were chosen to be at 0.40L = 54.8 ft from the centerline of bearing locations. The trial strand pattern is shown in Figure 5.7.2.4.

The properties of this strand pattern at midspan are:

$$y_{strand}=\left[\frac{12\cdot(2+4+6)+(4\cdot 8)+2\cdot(3+5+7+9+11+13+15)}{54}\right]=5.59\ \text{in}$$

$$e_{strand}=y_b-y_{strand}=28.80-5.59=23.21\ \text{in}$$

Section Modulus at the strand pattern centroid is

$$S_{gps}=\frac{I_g}{e_{strand}}=\frac{422{,}570}{23.21}=18{,}206\ \text{in}^3$$

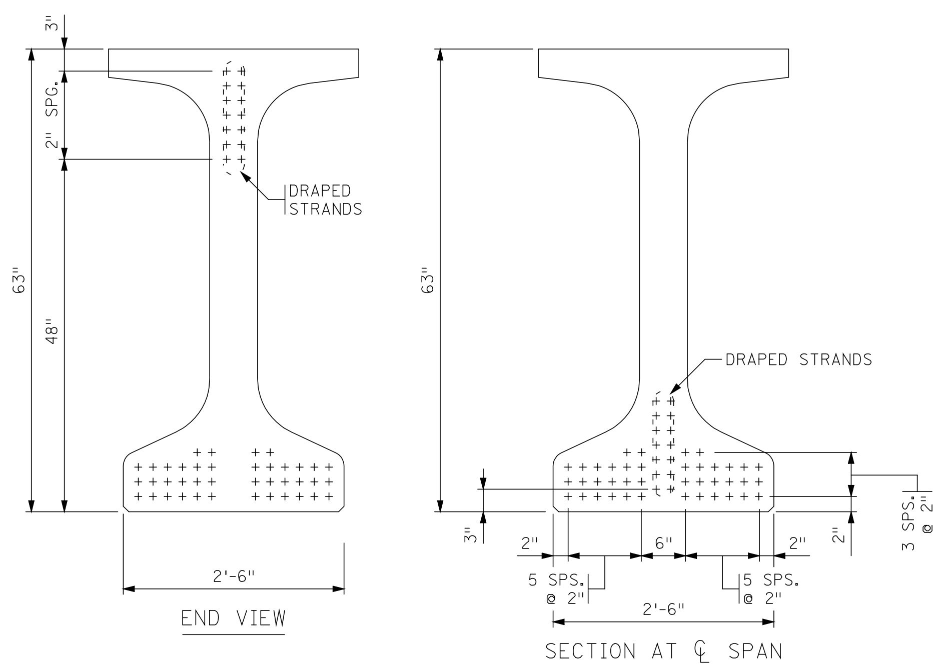


Figure 5.7.2.4

[5.9.5]

2. Prestress Losses

Prestress losses are computed using the approximate method.

[5.9.5.2.3]

Elastic Shortening Loss

Use the alternative equation presented in the LRFD C5.9.5.2.3a.

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

$$A_{ps} = (\#\text{ of strands}) \cdot (\text{strand area}) = 54 \cdot 0.217 = 11.72 \ \text{in}^2$$

$$f_{pbt} = f_{pj} = 202.50 \ \text{ksi}$$

$$e_m = e_{strand} = 23.21 \ \text{in}$$

$$\frac{A_g\, I_g\, E_{ci}}{E_p} = \frac{807\,(422{,}570)(4464)}{28{,}500} = 53{,}413{,}560 \ \text{in}^6$$

$$A_{ps}\left(I_g + e_m{}^2\, A_g\right) = 11.72\left[422{,}570 + (23.2)^2\,(807)\right] = 10{,}047{,}605 \ \text{in}^6$$

$$\Delta f_{pES} = \frac{202.5 \cdot (10{,}047{,}605) - 23.21\,(2039)\,(12)\,(807)}{10{,}047{,}605 + 53{,}413{,}560} = 24.8 \ \text{ksi}$$

[5.9.5.3]

Long Term Losses

Use the approximate equation in the LRFD 5.9.5.3

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$$

For an average humidity in Minnesota of 73%

$$\gamma_h = 1.7 - 0.01 \cdot H = 1.7 - 0.01 \cdot 73 = 0.97$$

$$\gamma_{st} = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 7.5} = 0.59$$

$$\Delta f_{pLT} = 10.0 \cdot \frac{202.5 \cdot (11.72)}{807} \cdot 0.97(0.59) + 12.0(0.97)(0.59) + 2.4 = 26.1 \ \text{ksi}$$

[5.9.5.1]

Total Losses

$$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pLT} = 24.8 + 26.1 = 50.9 \ \text{ksi}$$

$$f_{pe} = f_{pj} - \Delta f_{pt} = 202.5 - 50.9 = 151.6 \ \text{ksi}$$

prestress loss percentage $= \dfrac{\Delta f_{pt}}{f_{pj}} \cdot 100 = \dfrac{50.9}{202.50} \cdot 100 = 25.1\%$

initial prestress force

$$P_i = A_{ps} \cdot (f_{pj} - \Delta f_{pES}) = 11.72 \cdot (202.5 - 24.8) = 2083 \quad \text{kips}$$

prestress force after all losses

$$P_e = A_{ps} \cdot f_{pe} = 11.72 \cdot 151.6 = 1777 \quad \text{kips}$$

[5.9.4.1]

3. Stresses at Transfer (compression +, tension -)

Stress Limits for P/S Concrete at Release

Compression in the concrete is limited to:

$$0.60 \cdot f'_{ci} = 0.60 \cdot 7.5 = 4.50 \ \text{ksi}$$

Tension in the concrete is limited to:

The minimum of $-0.0948 \cdot \sqrt{f'_{ci}} = -0.0948 \cdot \sqrt{7.5} = -0.26 \ \text{ksi}$
or $-0.20 \ \text{ksi}$

Tension limit = -0.20 ksi

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

The selfweight moment is calculated using the design length to simplify calculations. This is conservative for calculation of release stresses.

$$P_i \cdot e_{strand} = 2083 \cdot 23.21 = 48{,}346 \ \text{kip-in}$$

Top stress due to P/S $= \left(\dfrac{P_i}{A_g}\right) - \left(\dfrac{P_i \cdot e_{strand}}{S_{gt}}\right) = \left(\dfrac{2083}{807}\right) - \left(\dfrac{48{,}346}{12{,}356}\right)$

$= -1.33 \ \text{ksi}$

Bottom Stress due to P/S $= \left(\dfrac{P_i}{A_g} + \dfrac{P_i \cdot e_{strand}}{S_{gb}}\right) = \left(\dfrac{2083}{807}\right) + \left(\dfrac{48{,}346}{14{,}673}\right)$

$= 5.88 \ \text{ksi}$

Selfweight moment at drape point $= M_{sw0.40} = 1957 \ \text{kip-ft}$

Top stress due to selfweight $= \left(\dfrac{M_{sw0.40}}{S_{gt}}\right) = \left(\dfrac{1957 \cdot 12}{12{,}356}\right) = 1.90 \ \text{ksi}$

Bottom stress due to selfweight $= \left(\dfrac{M_{sw0.40}}{S_{gb}}\right) = \left(\dfrac{1957 \cdot 12}{14{,}673}\right) = -1.60 \ \text{ksi}$

Top stress at drape point $= -1.33 + 1.90 = 0.57 \ \text{ksi} < -0.20 \ \text{ksi}$ <u>OK</u>

Bottom stress at drape point $= 5.88 - 1.60 = 4.28 \ \text{ksi} < 4.50 \ \text{ksi}$ <u>OK</u>

Check Release Stresses at End of Beam

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

Centroid of strand pattern at the end of the beams:

$$y_{strand} = \left[\frac{12 \cdot (2 + 4 + 6) + (4 \cdot 8) + 2 \cdot (60 + 58 + 56 + 54 + 52 + 50 + 48)}{54}\right]$$

$$= 17.26 \text{ in}$$

Centroid of strand at the transfer point:

$$y_{strand} = 17.26 - \frac{60 \cdot 0.6}{137 \cdot 0.4 \cdot 12 + \dfrac{15}{2}} \cdot (17.26 - 5.59) = 16.63 \text{ in}$$

The eccentricity of the strand pattern is:

$$e_{strand} = y_b - y_{strand} = 28.80 - 16.63 = 12.17 \text{ in}$$

The internal prestress moment is:

$$P_i \cdot e_{strand} = 2083 \cdot 12.17 = 25{,}350 \text{ kip-in}$$

Top stress due to P/S $= \left(\dfrac{P_i}{A_g}\right) - \left(\dfrac{P_i \cdot e_{strand}}{S_{gt}}\right) = \left(\dfrac{2083}{807}\right) - \left(\dfrac{25{,}350}{12{,}356}\right)$

$= 0.53$ ksi

Bottom stress due to P/S $= \left(\dfrac{P_i}{A_g}\right) + \left(\dfrac{P_i \cdot e_{strand}}{S_{gb}}\right) = \left(\dfrac{2083}{807}\right) + \left(\dfrac{25{,}350}{14{,}673}\right)$

$= 4.31$ ksi

Top stress due to selfweight $= \left(\dfrac{M_{swtr}}{S_{gt}}\right) = \left(\dfrac{176 \cdot 12}{12{,}356}\right) = 0.17$ ksi

Bottom stress due to selfweight $= -\left(\dfrac{M_{swtr}}{S_{gb}}\right) = -\left(\dfrac{176 \cdot 12}{14{,}673}\right) = -0.14$ ksi

Top stress at transfer point $= 0.53 + 0.17 = 0.70 \text{ ksi} < -0.20 \text{ ksi}$ <u>OK</u>

Bottom stress at transfer point $= 4.31 - 0.14 = 4.17 \text{ ksi} < 4.50 \text{ ksi}$ OK

By back calculating the highest compressive stress at release, located at the drape point, it is found that the concrete compressive strength at release can be reduced:

$$f'_{ci} = \frac{4.28}{0.60} = 7.13 \text{ ksi} \qquad \text{try } f'_{ci} = 7.2 \text{ ksi}$$

When modifying the initial concrete strength, prestress losses must be recalculated. The new losses are summarized below.

$\Delta f_{pES} = 25.0 \text{ ksi}$ p/s loss % at release = 12.3%

$\Delta f_{pLT} = 26.9 \text{ ksi}$

$\Delta f_{pt} = 51.9 \text{ ksi}$ p/s loss % after all losses = 25.6%

initial prestress force

$$P_i = A_{ps} \cdot (f_{pj} - \Delta f_{pES}) = 11.72 \cdot (202.5 - 25.0) = 2080 \text{ kips}$$

prestress force after all losses

$$f_{pe} = f_{pj} - \Delta f_{pt} = 202.5 - 51.9 = 150.6 \text{ ksi}$$

$$P_e = A_{ps} \cdot f_{pe} = 11.72 \cdot 150.6 = 1765 \text{ kips}$$

The new bottom stress at the drape point = 4.27 < $0.60 \cdot f'_{ci} = 4.32 \text{ ksi}$ OK

[5.9.4.2]

4. Stresses at Service Loads (compression +, tension -)

Stress Limits for P/S Concrete after All Losses

Compression in the concrete is limited to (Service I Load Combination):

$0.45 \cdot f'_c = 0.45 \cdot 9.0 = 4.05 \text{ ksi}$

(for prestress and permanent loads)

Check the bottom stress at end of beam and the top stress at midspan against this limit.

$0.40 \cdot f'_c = 0.40 \cdot 9.0 = 3.60 \text{ ksi}$

(for live load and $^1/_2$ of prestress and permanent loads)

Check the top stress at midspan against this limit.

$0.60 \cdot Q_w \cdot f'_c = 0.60 \cdot 1.0 \cdot 9.0 = 5.40 \text{ ksi}$

(for live load, prestress, permanent loads, and transient loads)

Check the top stress at midspan against this limit.

Tension in the concrete is limited to (Service III Load Combination):

$$-0.19 \cdot \sqrt{f'_c} = -0.19 \cdot \sqrt{9.0} = -0.570 \text{ ksi}$$

Check the bottom stress at midspan against this limit.

Check Stresses at Midspan After Losses:

Bottom stress

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

$$= -\left(\frac{4642 \cdot 12}{14{,}673}\right) - \left(\frac{718 \cdot 12}{21{,}664}\right) - \left(\frac{3012 \cdot 12 \cdot 0.8}{21{,}664}\right) + \left(\frac{1765}{807}\right) + \left(\frac{1765 \cdot 23.21}{14{,}673}\right)$$

$$= -0.550 \text{ ksi} < -0.570 \text{ ksi}$$ OK

Top stress due to all loads

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

$$= \left(\frac{1765}{807}\right) - \left(\frac{1765 \cdot 23.21}{12{,}356}\right) + \left(\frac{4642 \cdot 12}{12{,}356}\right) + \left[\frac{(718 + 3012) \cdot 12}{67{,}753}\right]$$

$$= 4.04 \text{ ksi} < 5.40 \text{ ksi}$$ OK

Top stress due to permanent loads

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

$$= \left(\frac{1765}{807}\right) - \left(\frac{1765 \cdot 23.21}{12{,}356}\right) + \left(\frac{4642 \cdot 12}{12{,}356}\right) + \left(\frac{718 \cdot 12}{67{,}753}\right)$$

$$= 3.51 \text{ ksi} < 4.05 \text{ ksi}$$ OK

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

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

$$= \frac{1}{2}\left(\left(\frac{1765}{807}\right) - \left(\frac{1765 \cdot 23.21}{12{,}356}\right) + \left(\frac{4642 \cdot 12}{12{,}356}\right) + \left(\frac{718}{67{,}753}\right)\right) + \left(\frac{3012 \cdot 12}{67{,}753}\right)$$

$$= 2.23 \text{ ksi} < 3.60 \text{ ksi}$$ OK

Check the Compression Stresses at End of Beam After Losses

Bottom flange stress due to prestress and permanent loads. For simplicity, all loads are ignored for this calculation, which is conservative.

$$\frac{P_e}{A_g}+\frac{P_e \cdot e_{strand}}{S_{gb}}=\frac{1765}{807}+\frac{1765 \cdot 12.17}{14{,}673}=3.65 \text{ ksi} < 4.05 \text{ ksi} \qquad \underline{\text{OK}}$$

The final concrete stress may also be reduced by back calculating the bottom tensile stress under the Service III load condition. In this instance:

$$\text{Min } f'_c=\left(\frac{0.550}{0.19}\right)^2=8.38 \text{ ksi} \qquad \text{Try } f'_c=8.50 \text{ ksi}$$

This change will not effect the computed losses, but the actual and allowable stresses must be recomputed and compared.
At midspan:

Bottom stress due to all loads = -0.545 ksi < -0.554 ksi (Service III) <u>OK</u>

Top stress due to all loads = 4.02 ksi < 5.10 ksi <u>OK</u>

Top stress due to permanent loads = 3.51 ksi < 3.83 ksi <u>OK</u>

Top stress due to live load plus ½ (prestress + permanent loads)
= 2.27 ksi < 3.40 ksi <u>OK</u>

Therefore, use f'_{ci} = 7.2 ksi and f'_c = 8.5 ksi

[5.5.4]

5. Flexure – Strength Limit State

Resistance factors at the strength limit state are:

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

$\phi = 0.90$ for shear and torsion

$\phi = 1.00$ for tension in steel in anchorage zones

Strength I design moment is 11,971 kip-ft at midspan.

From previous calculations, distance to strand centroid from bottom of the beam is:

$$y_{strand}=5.59 \text{ in}$$

[5.7.3.1.1]

$$k=2\cdot\left(1.04-\frac{f_{py}}{f_{pu}}\right)=2\cdot\left(1.04-\frac{243}{270}\right)=0.280$$

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

$$=63+1.5+8.5-5.59=67.41 \text{ in}$$

[5.7.3.1.1-4]

$$c = \left[\frac{(A_{ps} \cdot f_{pu})}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)}\right]$$

$$= \left(\frac{11.72 \cdot 270}{0.85 \cdot 4.0 \cdot 0.85 \cdot 108 + 0.28 \cdot 11.72 \cdot \frac{270}{67.41}}\right) = 9.73 \text{ in}$$

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 270 \cdot \left(1 - 0.28 \cdot \frac{9.73}{67.41}\right) = 259.1 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 9.73 = 8.27 \text{ in}$$

Compression block depth is less than 8.5", the thickness of the slab, so T-section behavior is not considered.

Internal lever arm between compression and tension flexural force components:

$$d_p - \frac{a}{2} = 67.41 - \frac{8.27}{2} = 63.28 \text{ in}$$

$$M_n = A_{ps} \cdot f_{ps} \cdot 63.28 = 11.72 \cdot 259.1 \cdot 63.28 = 192{,}159 \text{ kip-in}$$

$$= 16{,}013 \text{ kip-ft}$$

$$\phi\ M_n = 1.0 \cdot 16{,}013 = 16{,}013 \text{ kip-ft} > M_u = 11{,}971 \text{ kip-ft}$$ <u>OK</u>

Validate the assumption of 1.0 for the resistance factor:

[5.5.4.2.1]

$$\phi = 0.583 + 0.25 \cdot \left(\frac{d_t}{c} - 1\right) = 0.583 + 0.25 \cdot \left(\frac{67.41}{9.73} - 1\right) = 2.07 > 1$$

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

[5.7.3.3.2]

6. Minimum Reinforcement

$$f_r = 0.37 \cdot \sqrt{f'_c} = 0.37 \cdot \sqrt{8.5} = 1.08 \text{ ksi}$$

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

$$= \frac{1765}{807} + \frac{1765 \cdot 23.21}{14{,}673} = 4.98 \text{ ksi}$$

$$M_{cr} = (f_r + f_{peb}) \cdot S_{cgb} - M_{DC1} \cdot \left(\frac{S_{cgb}}{S_{gb}} - 1\right)$$

$$= (1.08 + 4.98) \cdot 21{,}664 - (4642 \cdot 12) \cdot \left(\frac{21{,}664}{14{,}673} - 1\right) = 104{,}743 \text{ kip-in}$$

$$= 8729 \text{ kip-ft}$$

$$1.2 \cdot M_{cr} = 1.2 \cdot 8729 = 10{,}475 < 16{,}013 \text{ kip-ft provided} \qquad \underline{\text{OK}}$$

E. Design Reinforcement for Shear

[5.8]

1. Vertical Shear Design

Determine d_v and Critical Section for Shear

Begin by determining the effective shear depth d_v at the critical section for shear.

$$d_v = d_p - \frac{a}{2}$$

The effective shear depth is no less than:

$$d_v \geq 0.72 \cdot h = 0.72 \cdot (63 + 1.5 + 8.5) = 52.6 \text{ in}$$

Assume d_v = 52.6 inches at critical section location of d_v from face of support. The internal face is assumed to be at the inside edge of the 15 inch sole plate. Then the critical section will be at least 67.6 inches $(52.6 + 15)$ or 5.63 feet away from the end of the beam. Find the centroid of the prestressing strands at this location:

The centroid of the prestressing strands is at:

$$y_{str@d_v} = y_{end} - \left(\frac{5.63}{0.40 \cdot \text{span} + \frac{L_{soleplate}}{2}} \right) \cdot (y_{end} - y_{drape})$$

$$= 17.26 - \left(\frac{5.63}{0.40 \cdot 137 + \frac{15}{2 \cdot 12}} \right) \cdot (17.26 - 5.59) = 16.1 \text{ in}$$

With this approximation to the strand centroid, d_p can be computed:

$$d_p = h - y_{str@d_v} = (63 + 1.5 + 8.5) - 16.1 = 56.9 \text{ in}$$

From the flexural strength computations, a = 8.27 inches at midspan. The value of "a" varies slightly along the beam length, but the value at midspan is close enough for design purposes.

$$d_v = d_p - \frac{a}{2} = 56.9 - \frac{8.27}{2} = 52.8 \text{ in}$$

But the effective shear depth d_v need not be less than

$$d_v \geq 0.72 \cdot h = 52.6 \text{ in}$$

or

$$d_v \geq 0.9\,d_e = 0.9\,d_p = 0.9\,(56.9) = 51.2 \text{ in}$$

Therefore take $d_v = 52.8$ inches, which is sufficiently close to the original assumption of $d_v = 52.6$ in

Then $x_{critv} = 7.5 + d_v = 7.5 + 52.8 = 60.3 \text{ in} = 5.03 \text{ ft}$ from centerline of bearing

Check Maximum Factored Shear Limit

From Table 5.7.2.4 the Strength I design shear at 5.03 ft is

$$V_u = 376 \text{ kips}$$

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

$$P_{14d} = 14 \cdot 0.217 \cdot 150.6 = 457.5 \text{ kips}$$

The inclination of the draped strands is:

$$\varphi = \arctan\left[\frac{(54-9)/12}{55.43}\right] = 3.87 \text{ degrees}$$

The vertical prestress component is:

$$V_p = P_{14d} \cdot \sin(\varphi) = 457.5 \cdot \sin(3.87) = 30.9 \text{ kips}$$

[5.8.3.2] The superstructure is supported by a parapet type abutment. Therefore, the nominal shear capacity of the section is limited to:

$$V_n = 0.18 \cdot f'_c \cdot d_v \cdot b_v + V_p = 0.18 \cdot 8.5 \cdot 52.8 \cdot 6.5 + 30.9 = 556 \text{ kips}$$

The maximum design shear the section can have is:

$$\phi_v \cdot V_n = 0.90 \cdot 556 = 500 \text{ kips} > 376 \text{ kips} \qquad \underline{\text{OK}}$$

Determine Longitudinal Strain ε_x

Assume that minimum transverse reinforcement will be provided in the cross section.

First, determine A_{ps}. Note that A_{ps} computed here is different than the A_{ps} computed earlier. This A_{ps} includes only the area of prestressing steel found on the flexural tension side of the member. Near the end of the beam, A_{ps} must also be reduced for development.

Development length ℓ_d is:

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

$$= 1.6\left[259.1 - \frac{2}{3}(150.6)\right](0.6) = 152.4 \text{ in}$$

Transfer length ℓ_{tr} is:

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

At the critical section $x_{critve} = (60.3 + 15/2) = 67.8 \text{ in}$ from the beam end, the strand development fraction is:

$$F_{dev} = \frac{f_{pe}}{f_{pu}} + \frac{d_{critv} - \ell_{tr}}{\ell_d - \ell_{tr}}\left(1 - \frac{f_{pe}}{f_{pu}}\right)$$

$$= \frac{150.6}{270} + \frac{67.8 - 36.0}{152.4 - 36.0}\left(1 - \frac{150.6}{270}\right) = 0.68$$

The flexural tension side of the member is defined as:

$$\frac{h_{comp}}{2} = \frac{63}{2} = 31.5 \text{ in}$$

At x_{critve} none of the draped strands fall on the flexural tension side. Therefore, $A_{ps} = (\#\text{ straight str.})(\text{strand area})(F_{dev})$

$$= (40)(0.217)(0.68) = 5.90 \text{ in}^2$$

[5.8.3.4.2] Use equation 5.8.3.4.2-4 to compute the strain:

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

$$= \left[\frac{\frac{1713 \cdot 12}{52.8} + |376 - 30.9| - (5.90 \cdot 0.70 \cdot 270)}{28{,}500 \cdot 5.90} \right] = -0.002$$

Because the value is negative, the strain will be recalculated using an additional concrete term:
From Figure 5.4.6.1 of this manual, $A_c = 486 \text{ in}^2$
For $f'_c = 8.5$ ksi, $E_c = 1265 \cdot \sqrt{8.5} + 1000 = 4688$ ksi

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

$$= \left[\frac{\frac{1713 \cdot 12}{52.8} + |376 - 30.9| - (5.90 \cdot 0.70 \cdot 270)}{28{,}500 \cdot 5.90 + 4688 \cdot 486} \right] = -0.00016$$

Computed strain limits:

-0.0004 < -0.00016 < 0.006 <u>OK</u>

Compute the tensile stress factor β using equation 5.8.3.4.2-1

$$\beta = \frac{4.8}{1 + 750 \cdot \varepsilon_s} = \frac{4.8}{1 + 750 \cdot (-0.00016)} = 5.45$$

Compute the angle θ using equation 5.8.3.4.2-3

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500 \cdot (-0.00016) = 28.44 \text{ degrees}$$

Compute the concrete contribution:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 5.45 \cdot \sqrt{8.5} \cdot 6.5 \cdot 52.8 = 172.3 \text{ kips}$$

The required steel contribution is:

$$V_s = V_n - V_c - V_p = \frac{V_u}{\phi_v} - V_c - V_p = \frac{376}{0.90} - 172.3 - 30.9 = 214.6 \text{ kips}$$

Find the required spacing of double leg #4 stirrups:

$$s = \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{V_s} = \frac{2 \cdot 0.20 \cdot 60 \cdot 52.8 \cdot \cot(28.44)}{214.6} = 10.90 \text{ in}$$

Try double leg stirrups at a 10 inch spacing at the end of the beam.

$$A_v = \frac{0.4 \cdot 12}{10} = 0.48 \text{ in}^2/\text{ft} \quad V_s = 234.0 \text{ kips}$$

[5.8.2.5]

Check that the minimum transverse reinforcement requirement is satisfied:

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

$$= 0.0316\sqrt{8.5}\,\frac{6.5}{60} \cdot 12 = 0.12 \frac{\text{in}^2}{\text{ft}} < 0.48 \frac{\text{in}^2}{\text{ft}} \qquad \underline{\text{OK}}$$

[5.8.2.7]

Check maximum stirrup spacing:

$$V_{spac} = 0.125 \cdot f'_c \cdot b_v \cdot d_v$$

$$= 0.125 \cdot 8.5 \cdot 6.5 \cdot 52.8 = 364.7 \text{ kips} < 376 \text{ kips}$$

Then the maximum spacing is the smaller of:

$$s_{max} = 0.4 \cdot d_v = 0.4 \cdot 52.8 = 21.1 \text{ in}$$

or $s_{max} = 12$ in <u>GOVERNS</u>

$$s_{max} = 12 \text{ in} > 10 \text{ in} \qquad \underline{\text{OK}}$$

Therefore use double leg #13 stirrups at 10 inch spacing. Other sections are investigated similarly.

[5.8.4]

2. Interface Shear Transfer

Top flange width $b_v = 34$ in

The Strength I vertical shear at the critical shear section due to all loads is:

$$V_u = 376$$

Interface shear force is:

$$V_h = \frac{V_u}{d_e} = \frac{V_u}{d_v} = \frac{376}{52.8} \cdot \frac{12 \text{ in}}{\text{ft}} = 85.5 \qquad \text{kip/ft}$$

Required nominal interface design shear is:

$$V_{nreq} = \frac{V_h}{\varphi_v} = \frac{85.5}{0.90} = 95.0 \text{ kip/ft}$$

The interface area per 1 foot length of beam is:

$$A_{cv} = 34 \cdot 12 = 408.0 \text{ in}^2/\text{ft}$$

[5.8.4.3] A note on Bridge Details II Fig. 5-397.509 requires the top flanges of the beam to be roughened. Then:

c= 0.28 ksi μ= 1.0 K_1= 0.3 K_2= 1.8 ksi

The upper limits on nominal interface shear are:

$$K_1 \cdot f'_c \cdot A_{cv} = 0.3 \cdot 4 \cdot 408.0 = 489.6 \text{ kip/ft} > 95.0 \text{ kip/ft} \qquad \underline{\text{OK}}$$

and

$$K_2 \cdot A_{cv} = 1.8 \cdot 408.0 = 734.4 \text{ kip/ft} > 95.0 \text{ kip/ft} \qquad \underline{\text{OK}}$$

The nominal interface shear resistance is:

$$V_n = c\,A_{cv} + \mu\left(A_{vf} \cdot f_y + P_c\right)$$

$$P_c = 0.0 \text{ kip}$$

Substitute and solve for required interface shear steel:

$$A_{vf} = \frac{V_{nreq} - c\,A_{cv}}{\mu \cdot f_y} = \frac{95.0 - 0.28\,(408.0)}{1.0 \cdot 60} = -0.32 \text{ in}^2/\text{ft}$$

Calculated value is negative, so A_{vf} is taken as zero

[5.8.4.4] Check minimum interface shear requirements:

$$A_{vf\,min} = \frac{0.05 \cdot b_v}{f_y} = \frac{0.05 \cdot 34}{60} = 0.028 \text{ in}^2/\text{in} = 0.34 \text{ in}^2/\text{ft}$$

The minimum requirement may be waived for girder-slab interfaces with the surface roughened to an amplitude of 0.25 in if the factored interface shear stress is less than 0.210 ksi.

$$v_{ui} = \frac{V_{ui}}{b_v d_v} = \frac{376.0}{34 \cdot 52.8} = 0.209 \text{ ksi} < 0.210 \text{ ksi}$$

No additional reinforcement is required for interface shear. Other sections are investigated similarly.

[5.8.3.5]

3. Minimum Longitudinal Reinforcement Requirement

The longitudinal reinforcement must be checked to ensure it is adequate to carry the tension caused by shear. The amount of strand development must be considered near the end of the beam. There are 2 cases to be checked:

Case 1: From the inside edge of bearing at the end supports out to a distance d_v, the following must be satisfied:

$$A_{ps} \cdot f_{ps} \geq \left(\frac{V_u}{\varphi_v} - 0.5 \cdot V_s - V_p \right) \cdot \cot\theta$$

A crack starting at the inside edge of the bearing sole plate will cross the center of gravity of the straight strands at:

$$x_{crack} = L_{soleplate} + y_{str} \cdot \cot(\theta) = 15 + 4.40 \cdot \cot(28.44) = 23.1 \text{ in}$$

The transfer length for 0.6" strands is: $\ell_{tr} = 36.0$ in

Interpolate to find the tensile capacity of the straight strands at the crack:

$$T_r = f_{pe} \cdot A_{ps} \cdot \frac{x_{crack}}{\ell_{tr}} = 150.6 \cdot 40 \cdot 0.217 \cdot \frac{23.1}{36} = 838.8 \text{ kips}$$

The force to carry is:

$$T = \left(\frac{V_u}{\varphi_v} - 0.5 \cdot V_s - V_p \right) \cdot \cot(\theta)$$

$$= \left(\frac{376}{0.90} - 0.5 \cdot 234.0 - 30.9 \right) \cdot \cot(28.44)$$

$$= 498.3 \text{ kips} < 838.8 \text{ kips}$$

<u>OK</u>

Case 2: At d_v, the following must be satisfied:

$$A_{ps} \cdot f_{ps} \geq \frac{M_u}{\varphi d_v} + \left(\frac{V_u}{\varphi} - 0.5 \cdot V_s - V_p \right) \cdot \cot\theta$$

Based on the fraction of strands developed as calculated earlier:

$$A_{ps} = 40 \cdot 0.217 \cdot 0.68 = 5.90 \text{ in}^2$$

Following steps shown earlier, f_{ps} at d_v from the inside edge of bearing is:

$$f_{ps} = 257.1 \text{ ksi}$$

Then $T_r = A_{ps} \cdot f_{ps} = 5.90 \cdot 257.1 = 1516.9$ kips

$$T = \frac{M_u}{\varphi d_v} + \left(\frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p\right) \cdot \cot(\theta) = \frac{1713 \cdot 12}{1.0 \cdot 52.8} + 498.3$$

$$T = 887.6 \text{ kips} < 1516.9 \text{ kips}$$ OK

F. Design Pretensioned Anchorage Zone Reinforcement [5.10.10.1]

Splitting Reinforcement

To prevent cracking in the beam end due to the transfer of the prestressing force from the strands to the concrete, splitting steel needs to be provided in the anchorage zone.

Use a load factor of 1.0 and lateral force component of 4% to determine the required amount of steel.

The factored design bursting force is:

$$P_b = 1.0 \cdot 0.04 \cdot P_i = 1.0 \cdot 0.04 \cdot 2080 = 83.2 \text{ kips}$$

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

$$A_s = \frac{P_b}{f_s} = \frac{83.2}{20} = 4.16 \text{ in}^2$$

This steel should be located at the end of the beam within a distance of:

$$\frac{h}{4} = \frac{63}{4} = 15.75 \text{ in}$$

The number of #5 double legged stirrups necessary to provide this area is:

$$\frac{A_s}{2 \cdot A_b} = \frac{4.16}{2 \cdot 0.31} = 6.7$$

The first set of stirrups is located 2 inches from the end of the beam. Provide seven sets of #5 stirrups spaced at 2 1/2 inch centers.

$$x_{splitting} = 2 + 6 \cdot 2.5 = 17 \text{ in} > 15.75 \text{ in}$$

Although the splitting reinforcement does not fit within h/4, #5 bars are the largest allowed and 2.5 inches is the tightest spacing allowed. This is OK per MnDOT practice.

[5.10.10.2]

Confinement Reinforcement

Reinforcement is required at the ends of the beam to confine the prestressing steel in the bottom flange. G403E and G507E bars (see Figure 5.7.2.5) will be placed at a maximum spacing of 6 inches out to 1.5d from the ends of the beam. For simplicity in detailing and ease of tying the reinforcement, space the vertical shear reinforcement with the confinement reinforcement in this area.

$$1.5\,d = 1.5(63) = 94.5 \text{ in}$$

G. Determine Camber and Deflection
[2.5.2.6.2]
[3.6.1.3.2]
[5.7.3.6.2]

Camber Due to Prestressing and Dead Load Deflection

Using the PCI handbook (Figure 4.10.13 of the 3rd Edition), the camber due to prestress can be found. The centroid of the prestressing has an eccentricity e_{mid} of 23.21 inches at midspan. At the end of the beams the eccentricity e_e is 12.17 inches. E is the initial concrete modulus (4394 ksi), P_o equals the prestress force just after transfer (2080 kips). The drape points are at 0.4 of the span. The span length is 137.0 feet. Using the equation for the two point depressed strand pattern:

$$e' = e_{mid} - e_e = 23.21 - 12.17 = 11.04 \text{ in}$$

$$\Delta_{ps} = \frac{P_o\, e_e\, L^2}{8EI} + \frac{P_o\, e'}{EI}\left(\frac{L^2}{8} - \frac{a^2}{6}\right)$$

$$= \frac{2080(12.17)(137 \cdot 12)^2}{8(4394)(422{,}570)} + \frac{2080(11.04)}{4394(422{,}570)}\left[\frac{(137 \cdot 12)^2}{8} - \frac{(0.4 \cdot 137 \cdot 12)^2}{6}\right]$$

$$= 7.89 \text{ in}$$

Downward deflection due to selfweight

$$\Delta_{sw} = \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I} = \frac{5 \cdot \frac{0.869}{12}(137 \cdot 12)^4}{384 \cdot 4394 \cdot 422{,}570} = -3.71 \text{ in}$$

Camber at release $\Delta_{rel} = \Delta_{ps} - \Delta_{sw} = 7.89 - 3.71 = 4.18$ in

To estimate camber at the time of erection the deflection components are multiplied by standard MnDOT multipliers. They are:

Release to Erection Multipliers:
Prestress = 1.4
Selfweight = 1.4

Camber and selfweight deflection values at erection are:

Prestress: $1.4 \cdot 7.89 = 11.05$ in

Selfweight: $1.4 \cdot (-3.71) = -5.19$ in

Diaphragm DL: -0.04 in

Deck and stool DL: -4.41 in

Parapet: -0.24 in

The values to be placed in the camber diagram on the beam plan sheet are arrived at by combining the values above.

"Erection Camber" = $11.05 - 5.19 - 0.04 = 5.82$ in say 5 7/8 in

"Est. Dead Load Deflection" $= 4.41 + 0.24 = 4.65$ in say 4 5/8 in

"Est. Residual Camber" = 5 7/8 – 4 5/8 = 1 1/4 in

Live Load Deflection

The deflection of the bridge is checked when subjected to live load and compared against the limiting values of $^{L}/_{800}$ for vehicle only bridges and $^{L}/_{1000}$ for bridges with bicycle or pedestrian traffic.

Deflection due to lane load is:

$$\Delta_{lane} = \left(\frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I}\right) = \left[\frac{5 \cdot \frac{0.64}{12} \cdot (137 \cdot 12)^4}{384 \cdot 4688 \cdot 1{,}034{,}168}\right] = 1.05 \text{ in}$$

Deflection due to a truck with dynamic load allowance is found using hand computations or computer tools to be:

$$\Delta_{truck} = 1.74 \text{ in}$$

Two deflections are computed and compared to the limiting values; that of the truck alone and that of the lane load plus 25% of the truck. Both deflections need to be adjusted with the distribution factor for deflection.

$$\Delta_1 = DF_{\Delta} \cdot \Delta_{truck} = 0.567 \cdot 1.74 = 0.99 \text{ in}$$

$$\Delta_2 = DF_{\Delta} \cdot (\Delta_{lane} + 0.25 \cdot \Delta_{truck}) = 0.567 \cdot (1.05 + 0.25 \cdot 1.74) = 0.84 \text{ in}$$

There is no bicycle or pedestrian traffic on the bridge, so the deflection limit is:

$$\frac{L}{800} = \frac{137 \cdot 12}{800} = 2.1 \text{ in} >> \text{ than } \Delta_1 \text{ or } \Delta_2 \quad \underline{\text{OK}}$$

H. Detailing Items

Approximate weight of each beam is:

$$L \cdot A \cdot \gamma = 138.25 \cdot \frac{807}{144} \cdot 0.155 \cdot \frac{1}{2} = 60.0 \text{ tons}$$

Initial prestress force at jacking is:

$$54 \cdot 0.217 \cdot 0.75 \cdot 270 = 2373 \text{ kips}$$

Figure 5.7.2.5 shows the detailed beam sheet for the bridge.

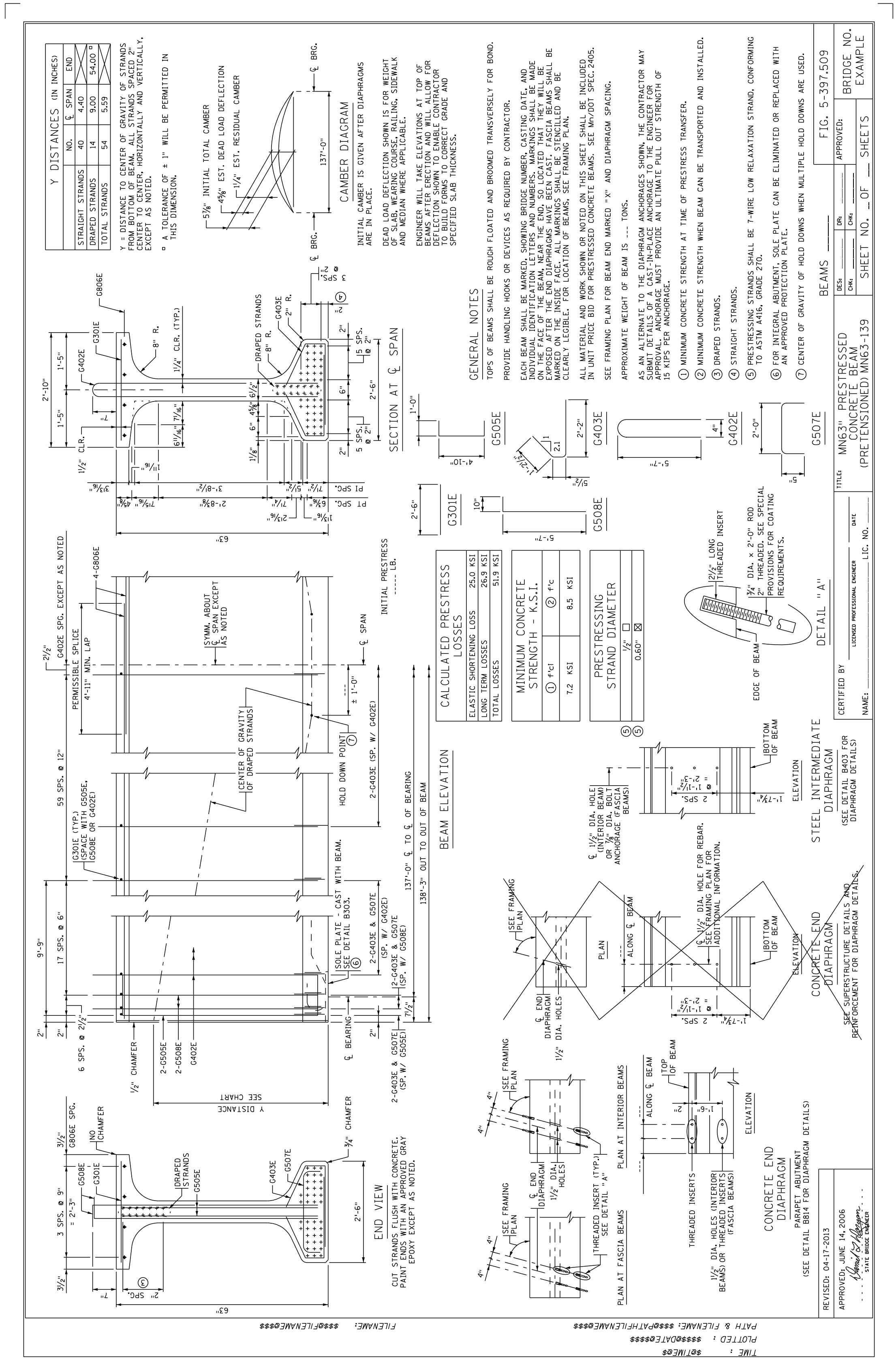

Y DISTANCES (IN INCHES)
NO. | ℄ SPAN | END
STRAIGHT STRANDS 40 4.40
DRAPED STRANDS 14 9.00 54.00
TOTAL STRANDS 54 5.59
Y = DISTANCE TO CENTER OF GRAVITY OF STRANDS FROM BOTTOM OF BEAM. ALL STRANDS SPACED 2" CENTER TO CENTER, HORIZONTALLY AND VERTICALLY, EXCEPT AS NOTED.
A TOLERANCE OF ± 1" WILL BE PERMITTED IN THIS DIMENSION.
CAMBER DIAGRAM
INITIAL CAMBER IS GIVEN AFTER DIAPHRAGMS ARE IN PLACE.
DEAD LOAD DEFLECTION SHOWN IS FOR WEIGHT OF SLAB, WEARING COURSE, RAILING, SIDEWALK AND MEDIAN WHERE APPLICABLE.
ENGINEER WILL TAKE ELEVATIONS AT TOP OF BEAMS AFTER ERECTION AND WILL ALLOW FOR DEFLECTION SHOWN TO ENABLE CONTRACTOR TO BUILD FORMS TO CORRECT GRADE AND SPECIFIED SLAB THICKNESS.
GENERAL NOTES
TOPS OF BEAMS SHALL BE ROUGH FLOATED AND BROOMED TRANSVERSELY FOR BOND.
PROVIDE HANDLING HOOKS OR DEVICES AS REQUIRED BY CONTRACTOR.
EACH BEAM SHALL BE MARKED, SHOWING BRIDGE NUMBER, CASTING DATE, AND INDIVIDUAL IDENTIFICATION LETTERS AND NUMBERS. MARKINGS SHALL BE MADE ON THE FACE OF THE BEAM, NEAR THE END, SO LOCATED THAT THEY WILL BE EXPOSED AFTER THE END DIAPHRAGMS HAVE BEEN CAST. FASCIA BEAMS SHALL BE MARKED ON THE INSIDE FACE. ALL MARKINGS SHALL BE STENCILLED AND BE CLEARLY LEGIBLE. FOR LOCATION OF BEAMS, SEE FRAMING PLAN.
ALL MATERIAL AND WORK SHOWN OR NOTED ON THIS SHEET SHALL BE INCLUDED IN UNIT PRICE BID FOR PRESTRESSED CONCRETE BEAMS. SEE Mn/DOT SPEC. 2405.
SEE FRAMING PLAN FOR BEAM END MARKED "X" AND DIAPHRAGM SPACING.
APPROXIMATE WEIGHT OF BEAM IS --- TONS.
AS AN ALTERNATE TO THE DIAPHRAGM ANCHORAGES SHOWN, THE CONTRACTOR MAY SUBMIT DETAILS OF A CAST-IN-PLACE ANCHORAGE TO THE ENGINEER FOR APPROVAL. ANCHORAGE MUST PROVIDE AN ULTIMATE PULL OUT STRENGTH OF 15 KIPS PER ANCHORAGE.
1 MINIMUM CONCRETE STRENGTH AT TIME OF PRESTRESS TRANSFER.
2 MINIMUM CONCRETE STRENGTH WHEN BEAM CAN BE TRANSPORTED AND INSTALLED.
3 DRAPED STRANDS.
4 STRAIGHT STRANDS.
5 PRESTRESSING STRANDS SHALL BE 7-WIRE LOW RELAXATION STRAND, CONFORMING TO ASTM A416, GRADE 270.
6 FOR INTEGRAL ABUTMENT, SOLE PLATE CAN BE ELIMINATED OR REPLACED WITH AN APPROVED PROTECTION PLATE.
7 CENTER OF GRAVITY OF HOLD DOWNS WHEN MULTIPLE HOLD DOWNS ARE USED.
CALCULATED PRESTRESS LOSSES
ELASTIC SHORTENING LOSS 25.0 KSI
LONG TERM LOSSES 26.9 KSI
TOTAL LOSSES 51.9 KSI
MINIMUM CONCRETE STRENGTH - K.S.I.
1 f'ci 7.2 KSI
2 f'c 8.5 KSI
PRESTRESSING STRAND DIAMETER
1/2"
0.60"
SECTION AT ℄ SPAN
BEAM ELEVATION
END VIEW
CUT STRANDS FLUSH WITH CONCRETE. PAINT ENDS WITH AN APPROVED GRAY EPOXY EXCEPT AS NOTED.
DETAIL "A"
STEEL INTERMEDIATE DIAPHRAGM
(SEE DETAIL B403 FOR DIAPHRAGM DETAILS)
CONCRETE END DIAPHRAGM
PARAPET ABUTMENT
(SEE DETAIL B814 FOR DIAPHRAGM DETAILS)
SEE SUPERSTRUCTURE DETAILS AND REINFORCEMENT FOR DIAPHRAGM DETAILS
G301E G505E G508E G403E G402E G507E
137'-0" ℄ TO ℄ OF BEARING
138'-3" OUT TO OUT OF BEAM
TITLE: MN63" PRESTRESSED CONCRETE BEAM (PRETENSIONED) MN63-139
BEAMS
FIG. 5-397.509
BRIDGE NO. EXAMPLE
SHEET NO. OF SHEETS
REVISED: 04-17-2013
APPROVED: JUNE 14, 2006
STATE BRIDGE ENGINEER

Figure 7.2.5

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

This example illustrates the design of a haunched post-tensioned concrete slab bridge. The three continuous spans are 55'-0", 70'-0", and 55'-0" in length. The roadway width is 44'-0" with MnDOT Type F barrier railings for a total out-to-out width of 47'-4". A plan view and typical sections of the bridge are shown in Figures 5.7.3.1 and 5.7.3.2.

After computing the dead and live loads, a preliminary tendon profile is developed. Prestress losses for the preliminary layout are computed for anchor set, friction, elastic shortening, creep, shrinkage, and relaxation. Subsequently, the load combinations are assembled (with the secondary post-tensioning force effects included). Flexural and shear strength checks are performed, after which deflection and camber calculations are assembled. Lastly, the design of the anchorage zone is performed.

Single ended jacking is assumed for the design. The construction documents will require that the jacked end and the dead ends alternate. With the tendons stressed at alternating ends, the results for the friction losses and anchor set losses for tendons stressed at opposite ends will be averaged to obtain losses for a "typical" tendon.

The following material and design parameters are used in this example:

A. Material and Design Parameters

[5.4.2.4]

[Table 5.4.4.1-1]

[5.4.4.2]

[5.4.3.2]

Table 5.7.3.1
Design Parameters

Material		Parameter	Value
Concrete		Compressive Strength at Transfer, f'_{ci}	4.5 ksi
Concrete		Compressive Strength at 28 days, f'_c	5.0 ksi
Concrete		Modulus of Elasticity at Transfer, E_{ci}	3865 ksi
Concrete		Modulus of Elasticity at 28 days, E_c	4074 ksi
Steel Reinforcement	Prestressing	7-wire Strand	0.60 in dia., low-lax
Steel Reinforcement	Prestressing	Area of one Strand	0.217 in^2
Steel Reinforcement	Prestressing	Tensile Strength, f_{pu}	270.0 ksi
Steel Reinforcement	Prestressing	Yield Strength, f_{py}	243.0 ksi
Steel Reinforcement	Prestressing	Modulus of Elasticity, E_p	28,500 ksi
Steel Reinforcement	Rebar	Yield Strength, F_y	60 ksi
Steel Reinforcement	Rebar	Modulus of Elasticity, E_s	29,000 ksi

Additional Dead Loads

Future Wearing Surface $= 0.020$ ksf

Type F Barriers, $w = 0.439$ kip/ft/barrier

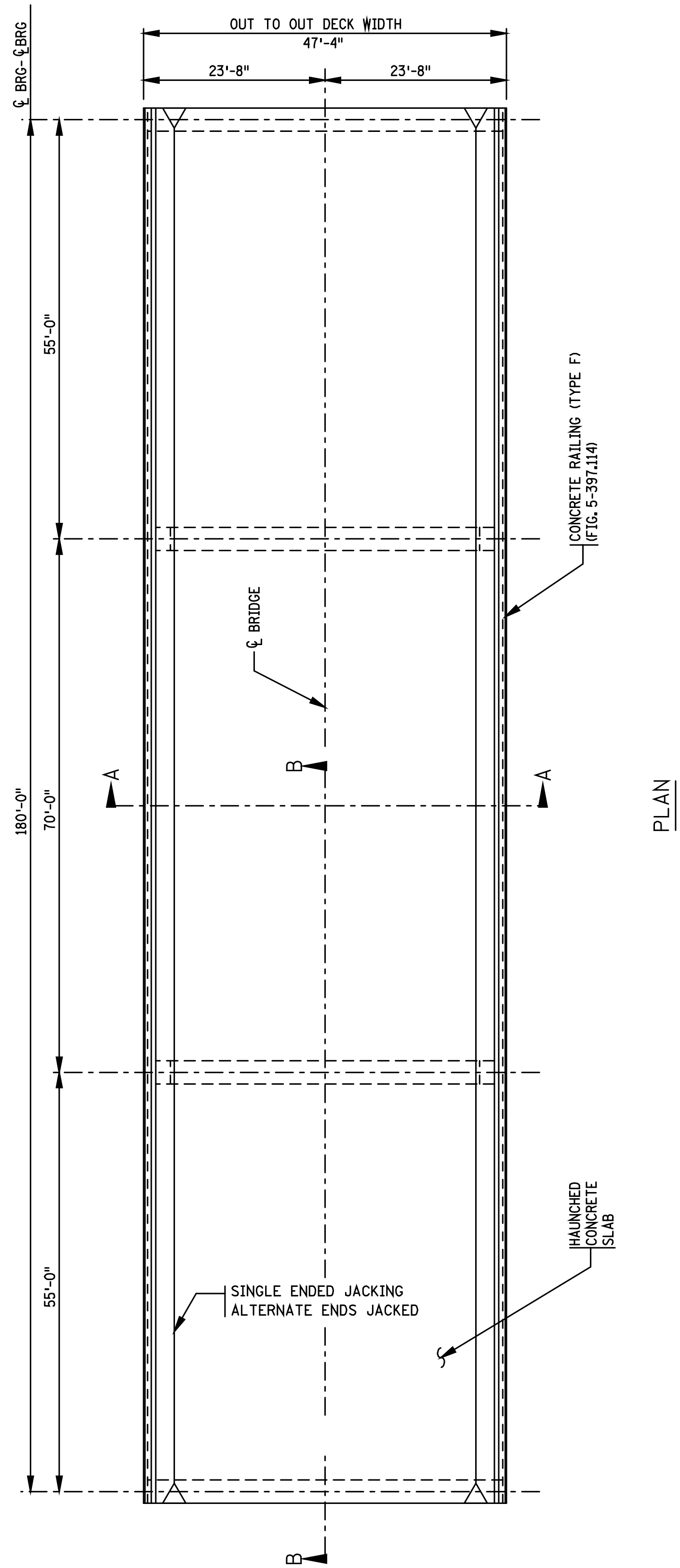


Figure 5.7.3.1
Bridge Layout

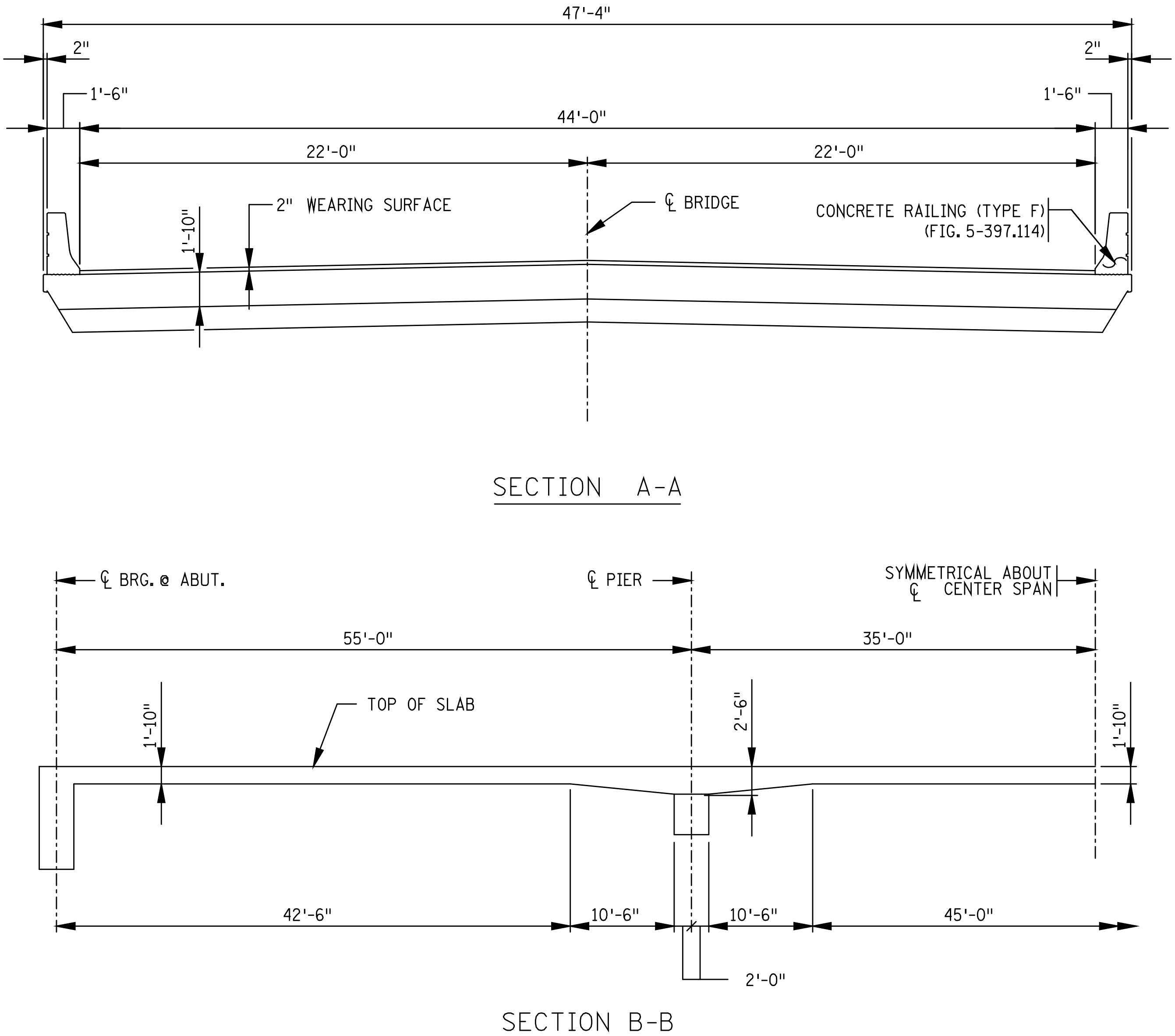


Figure 5.7.3.2
Transverse and Longitudinal Sections

B. Haunch Length and Minimum Recommended Slab Depth

MnDOT's standard design practice is to use linear haunches, with a haunch length of 15% of the length of the longest span in the continuous system.

Haunch Length:

$0.15 \cdot 70 = 10.5$ ft <u>Use 10.5 ft</u>

[2.5.2.6.3]

The depth of the slab at midspan can be estimated with:

$0.95 \cdot (0.027 \cdot L) = 0.95 \cdot (0.027 \cdot 70) = 1.80$ ft <u>Use 1.83 ft</u>

A trial depth of the slab at the piers can be found with:

$\frac{4}{3} \cdot (\text{midspan slab depth}) = \frac{4}{3} \cdot (1.80) = 2.39$ ft <u>Use 2.50 ft</u>

C. Live Load Strip Widths and Distribution Factors [4.6.2.3] [3.6.1.1.1]

The equations in the LRFD Specifications are arranged to determine the width of slab that resists a particular live load. To simplify the design process (which is based on a 1 foot wide design strip) the resultant widths are rearranged to determine the fraction of lane load carried by a 1 foot wide strip of slab.

Distribution Factor for Flexure – One Lane Loaded

The equation used to find the width of slab resisting one lane of live loading is:

$$E = 10.0 + 5.0 \cdot \sqrt{L_1 \cdot W_1}$$

Where:

L_1 is the modified span length (the smaller of the actual span length and 60 feet)

W_1 is the modified bridge width (the smaller of the actual width and 30 feet)

Substituting in values for the side and main spans produces:

55 ft Spans:

$$E_s = 10.0 + 5.0 \cdot \sqrt{55 \cdot 30} = 213.1 \text{ in/lane}$$

$$\frac{1}{E_s} = \left(\frac{1}{213.1}\right) \cdot \left(\frac{12}{1}\right) = 0.056 \text{ lanes/ft}$$

70 ft Span:

$$E_s = 10.0 + 5.0 \cdot \sqrt{60 \cdot 30} = 222.1 \text{ in/lane}$$

$$\frac{1}{E_s} = \left(\frac{1}{222.1}\right) \cdot \left(\frac{12}{1}\right) = 0.054 \text{ lanes/ft}$$

Distribution Factor for Flexure – Multiple Lanes Loaded

A similar procedure is used to determine the width of slab that carries multiple lanes of live load. The general equation is:

$$E = 84.0 + 1.44 \cdot \sqrt{L_1 \cdot W_1} \le \frac{12.0 \cdot W}{N_L}$$

Where:

L_1 is the modified span length (smaller of the span length and 60 ft)

W_1 is the modified bridge width (smaller of the bridge width and 60 ft)

W is the physical edge-to-edge width of the bridge (47.33 ft)

N_L is the number of design lanes:

$$N_L = \frac{44}{12} = 3.7 \qquad \underline{\text{Use } 3}$$

Substituting values into the equations for the side and main spans produces:

55 ft Spans:

$$E_m = 84.0 + 1.44 \cdot \sqrt{55 \cdot 47.33} = 157.5 \le \frac{12.0 \cdot 47.33}{3} = 189.3 \text{ in/lane}$$

$$\frac{1}{E_m} = \left(\frac{1}{157.5}\right) \cdot \left(\frac{12}{1}\right) = 0.076 \text{ lanes/ft}$$

70 ft Span:

$$E_m = 84.0 + 1.44 \cdot \sqrt{60 \cdot 47.33} = 160.7 \le \frac{12.0 \cdot 47.33}{3} = 189.3 \text{ in/lane}$$

$$\frac{1}{E_m} = \left(\frac{1}{160.7}\right) \cdot \left(\frac{12}{1}\right) = 0.075 \text{ lanes/ft}$$

Distribution Factor for Shear

The shear check is performed with a single distribution factor where all design lanes are loaded and the entire slab is assumed to participate in carrying the load.

$$E_v = (\text{\# of lanes}) \cdot \left(\frac{\text{MPF}}{\text{deck width}}\right) = 3 \cdot \left(\frac{0.85}{47.33}\right) = 0.054 \text{ lanes/ft}$$

[2.5.2.6.2]

Distribution Factor for Deflection

Deflection computations should be based on the same distribution factor calculated for shear forces:

$$E_{\Delta} = E_{v} = 0.054 \ \text{lanes/ft}$$

D. Edge Beam Width and Distribution Factor [4.6.2.1.4]

The exterior strip is assumed to carry one wheel line and a tributary portion of lane live load.

Check if the equivalent strip is less than the maximum width of 72 inches.

$$E = (\text{edge of slab to inside face of barrier}) + 12.0 + \frac{(\text{strip width})}{2}$$

$$E = 20 + 12 + \frac{157.4}{2} = 110.7 \ \text{in} > 72.0 \ \text{in}$$ <u>Use 72.0 in</u>

Compute the distribution factor associated with one truck wheel line:

$$LLDF_{EXTT} = \left[\frac{1 \ \text{wheel line} \cdot (\text{MPF})}{(2 \ \text{wheel lines/lane}) \cdot (E/12)}\right]$$

$$= \left[\frac{1 \cdot (1.2)}{2 \cdot (72/12)}\right] = 0.100 \ \text{lanes/ft}$$

Compute the distribution factor associated with lane load on a 72 inch wide exterior strip. Subtract the gutter line to edge of deck distance to obtain the deck width loaded:

$$LLDF_{EXTL} = \left[\frac{\left(\frac{\text{deck width loaded}}{10 \ \text{ft. load width}}\right) \cdot \text{MPF}}{(\text{exterior strip width})}\right]$$

$$LLDF_{EXTL} = \left[\frac{\left(\frac{72/12 - 20/12}{10}\right) \cdot 1.2}{72/12}\right] \ \text{lanes/ft}$$

For simplicity, the larger value (0.100 lanes/ft) is used for both load types when assembling design forces for the exterior strip.

Table 5.7.3.2 summarizes the distribution factors for the different force components.

Table 5.7.3.2
Distribution Factor Summary

Force Component	Width	Type of Loading	Span (ft)	Distribution Factor (lanes/ft)
Flexure	Interior Strip	One Lane	55	0.056
			70	0.054
		Multiple Lanes	55	0.076
			70	0.075
	Exterior Strip	One Lane	55 & 70	0.100
Shear	Slab Width	Multiple Lanes	55 & 70	0.054
Deflections	Slab Width	Multiple Lanes	55 & 70	0.054

The following load modifiers will be used for this example:

		Strength	Service	Fatigue
Ductility	η_D	1.0	1.0	1.0
Redundancy	η_R	1.0	1.0	1.0
Importance	η_I	1.0	n/a	n/a
	$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0	1.0

E. Load Combinations, Load Factors, and Load Modifiers
[3.4.1]
[1.3.3-1.3.5]

The load combinations considered for the design example are identified below:

STRENGTH I – Used to ensure adequate strength under normal vehicular use.

$$U = 1.0 \cdot [1.25 \cdot DC + 1.25 \cdot DW + 1.75 \cdot (LL + IM)]$$

SERVICE I – Used for compression checks in prestressed concrete.

$$U = 1.0 \cdot (DC + DW) + 1.0 \cdot (LL + IM)$$

SERVICE III – Used for tension checks in prestressed concrete for crack control purposes.

$$U = 1.0 \cdot (DC + DW) + 0.8 \cdot (LL + IM)$$

[5.5.3.1]

FATIGUE – No fatigue check is necessary for fully prestressed sections.

F. Live Loads
[3.6.1]

The HL-93 live load components used for this example are:

Design Truck
Design Lane
Design Tandem
Truck Train

The live load components are combined in the following manner:

Design Truck + Design Lane
Design Tandem + Design Lane
0.90 (Truck Train + Design Lane) (Neg. Moment Regions)

[3.6.2]

Dynamic Load Allowance

The dynamic load allowance, (IM) for truck and tandem live loads is 33% for all applicable limit states and load combinations.

G. Dead Loads

Interior Strip (1'-0" Width)

The 2 inch wearing course is included in the slab depth (h) used to determine the dead loads (w_{DC}). It is not considered part of the structural section resisting loads.

$$w_{DC} = (\text{width}) \cdot w_c \cdot h + \left(\frac{2 \cdot w_{barrier}}{\text{deck width}}\right)$$

$$= (1.0) \cdot 0.150 \cdot h + \left(\frac{2 \cdot 0.439}{47.33}\right) = 0.150 \cdot h + 0.019 \ \text{kip/ft}$$

For design simplicity the dead load associated with the future wearing surface (0.020 ksf) is combined with the other DC loads.

$$w_{DC} = 0.150 \cdot h + 0.019 + 0.020 = 0.150 \cdot h + 0.039 \ \text{kip/ft}$$

Edge Strip (1'-0" Width)

For the design of the edge strip, it is conservatively assumed that the dead load of one barrier is carried by each edge strip.

$$w_{DC} = 0.150 \cdot h + \left(\frac{0.439}{6.0}\right) = 0.150 \cdot h + 0.073 \ \text{kip/ft}$$

The future wearing surface load is:

$$w_{DW} = 0.120 \cdot \left(\frac{6.0 - 1.67}{6.0}\right) = 0.014 \ \text{kip/ft}$$

The combined dead load is:

$$w_{DC} = 0.150 \cdot h + 0.073 + w_{DW} = 0.150 \cdot h + 0.087 \ \text{kip/ft}$$

H. Structural Analysis Model and Resultant Loads

The dead and live loads were applied to a continuous beam model with gross section properties. Nonprismatic properties were used to account for the presence of the linear haunches near the piers. The results of the analysis are presented in Tables 5.7.3.3 and 5.7.3.4.

Table 5.7.3.3
Moment Load Components (kip-ft)

Span Point	Dead Load (per ft)		Live Load (per lane)*						
	Interior Strip M_{DC}	Exterior Strip M_{DC}	Lane		Truck		Tandem		Truck Train
			Max.	Min.	Max.	Min.	Max.	Min.	Min.
1.0	0	0	0	0	0	0	0	0	-
1.1	31	35	78	-20	263	-47	227	-36	-
1.2	51	59	136	-39	433	-94	384	-71	-
1.3	62	71	175	-59	519	-140	475	-107	-
1.4	62	71	195	-78	552	-187	507	-142	-
1.5	52	59	196	-98	534	-234	492	-178	-
1.6	31	36	177	-118	476	-281	437	-213	-
1.7	1	1	138	-137	359	-327	345	-249	-286
1.8	-40	-46	83	-159	204	-374	228	-284	-380
1.9	-92	-105	52	-225	116	-421	97	-320	-516
2.0	-155	-177	46	-335	129	-468	102	-355	-698
2.1	-78	-89	42	-190	77	-335	118	-265	-483
2.2	-20	-23	71	-109	246	-284	268	-224	-303
2.3	22	25	133	-93	416	-232	392	-183	-
2.4	47	53	181	-93	520	-180	472	-143	-
2.5	55	63	196	-93	545	-129	496	-102	-

* Values do not include dynamic load allowance.

Table 5.7.3.4
Shear Load Components (kips)

Span Point	Dead Load (per ft)	Live Load (per lane)*					
		Lane		Truck		Tandem	
		Max.	Min.	Max.	Min.	Max.	Min.
1.0	6.5	15.9	-3.6	56.7	-8.5	47.7	-6.5
1.1	4.7	12.6	-3.8	47.8	-8.5	41.2	-6.5
1.2	2.8	9.8	-4.5	39.4	-8.5	34.9	-10.5
1.3	0.9	7.4	-5.6	31.4	-14.1	28.8	-16.8
1.4	-0.9	5.4	-7.1	24.1	-22.0	23.0	-22.8
1.5	-2.8	3.8	-9.1	17.4	-29.6	17.7	-28.5
1.6	-4.6	2.6	-11.4	11.6	-37.6	12.8	-33.7
1.7	-6.5	1.8	-14.1	6.5	-45.2	8.5	-38.5
1.8	-8.3	1.2	-17.0	2.3	-52.1	4.9	-42.6
1.9	-10.4	0.9	-20.3	2.3	-58.2	1.9	-46.0
2.0	12.4	25.1	-2.7	64	-63.6	49	-49
2.1	9.6	20.7	-2.8	57.4	-7.4	45.2	-5.8
2.2	7.1	16.8	-3.3	49.9	-7.4	40.7	-6.6
2.3	4.7	13.2	-4.3	41.7	-10.2	35.3	-11.6
2.4	2.4	10.2	-5.7	33.3	-17.1	29.4	-17.2
2.5	0.0	7.7	-7.7	24.9	-24.9	23.3	-23.3

* Values do not include dynamic load allowance.

I. Develop Preliminary Tendon Profile

Begin by determining the eccentricity of the tendon at primary locations and calculating the dead load and live load moments. Preliminary runs with assumed prestress losses are used to determine an appropriate tendon area or tendon force per foot. The prestress needs to provide zero tension stress in the slab at the service limit state. For this example, a 12 strand (0.6 inch diameter) tendon is found to be appropriate when spaced at 2'-5".

A handful of suppliers provide post-tensioning products in the U.S. Catalogs from the suppliers should be reviewed to ensure that standard tendons and ducts are used. LRFD Article 5.4.6.2 places maximum and minimum limits on the size of ducts based on the size of the tendon and the least concrete dimension of the member.

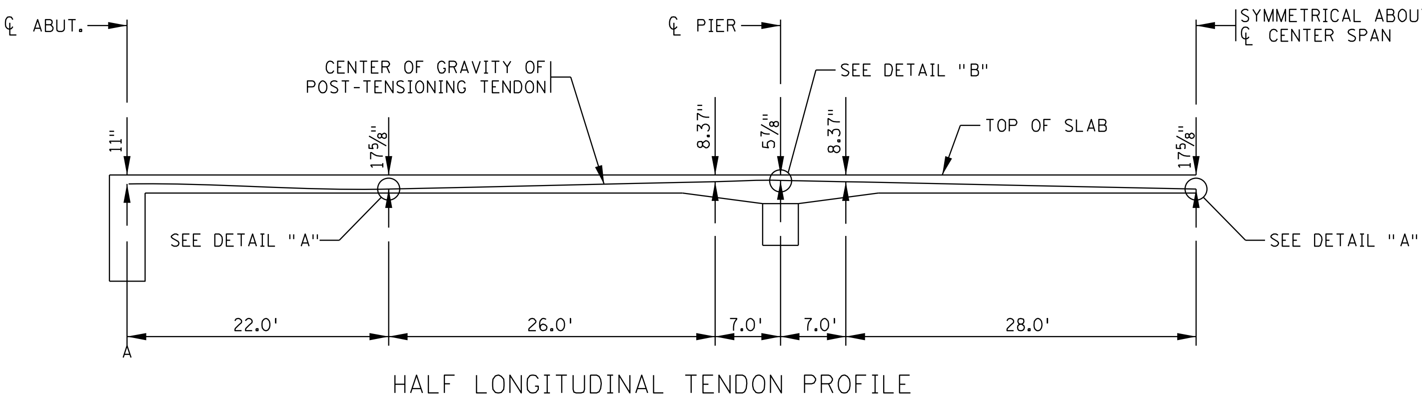
The tendon low points for the side spans will be placed at Span Points 1.4 and 3.6 (22 ft away from the abutment end of the span). The tendon low point for the center span will be placed at midspan (Span Point 2.5). The tendon high points will be located over the piers at Span Points 2.0 and 3.0. The tendon will be at the centroid of the gross cross section at each end of the structure (Span Points 1.0 and 4.0). See Figure 5.7.3.3 for a sketch of the proposed tendon profile and tendon centroid locations at high and low points of the tendon profile.

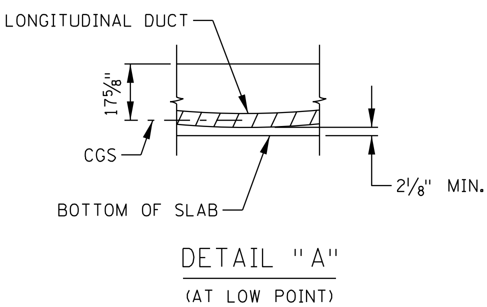
Critical points of the tendon geometry are calculated as:

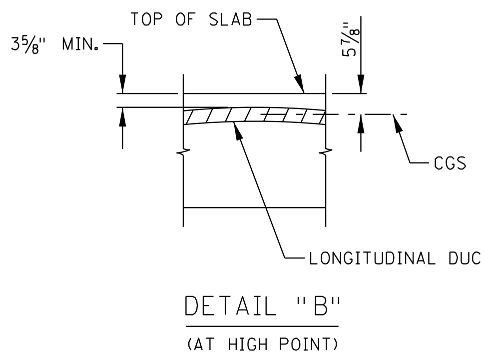
d_{top} at Span Point 1.0 = 11.00 in

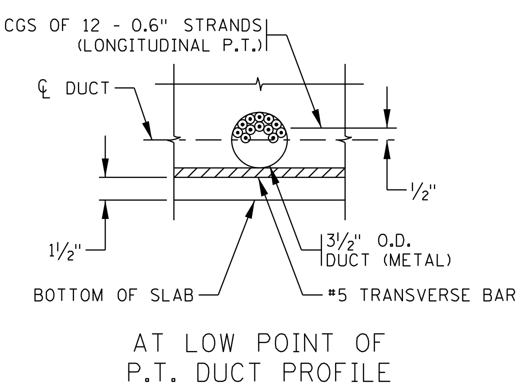
d_{top} at Span Points 1.4 and 2.5 = 22 - 1.5 - 0.625 - 2.25 = 17.63 in

d_{top} at Span Points 2.0 = 3 + 0.625 + 2.25 = 5.88 in









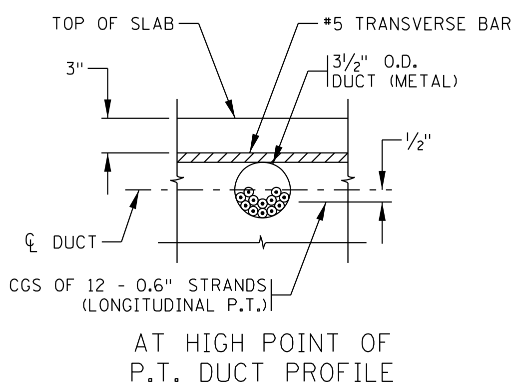


Figure 5.7.3.3
Tendon Profile and Centroid Locations

Tendon Equations

The tendon profile can be defined with a series of parabolas where for each parabola:

$$y = a \cdot x^2 + b \cdot x + c$$

With the section depth varying along the slab, use the top of the slab as the datum for defining the parabolic curves. The tendon profile is described with three parabolas; one describing the positive moment region of the side spans, a second describing the negative moment regions over the piers, and lastly a third parabola describing the positive moment region of the center span. Using the constraints:

y= 11.00 inches at x = 0 feet
y = 17.63 inches at x = 22 feet
Slope = 0 at x = 22 feet

The equation for the parabola for the positive moment regions of the side spans is found to be:

$$y = (-0.01369) \cdot x^2 + (0.6023) \cdot x + 11.00 \qquad \text{(x in feet, y in inches)}$$

Knowing that the y-coordinate and the slope for the tendon profile needs to be consistent at the location where parabolas meet, the second and third parabolas can be found.

Set the origin for the second parabola to be at Span Point 2.0. The following constraints can be used to determine the constants for the parabola:

y = 5.875 inches at x = 0 feet
Slope = 0 at x = 0 feet
y at the end of the curve matches that of the 1st parabola
Slope at the end of the curve matches that of the 1st parabola

The location where the 1st and 2nd parabolas meet was found by changing the length of the 2nd parabola until the y value and slope matched that of the 1st parabola. The parabolas satisfy the criteria if they meet at a point 7.00 feet away from the pier (Span Point 1.873). The equation for the 2nd parabola is:

$$y = (0.05092) \cdot x^2 + (0) \cdot x + 5.875 \qquad \text{(x in feet, y in inches)}$$

With the 2nd parabola defined, the same procedure can be used to determine the constants for the 3rd parabola. With x = 0 at Span Point 2.5, the constants are:

y = 17.625 inches at x = 0 feet
Slope = 0 at x = 0 feet
y at the end of the curve matches that of the 2nd parabola
Slope at the end of the curve matches that of the 2nd parabola

After iterating the length of the 3rd parabola, the location where the y values and slopes match for the 2nd and 3rd parabolas is at a location 7.00 feet away from the pier (Span Point 2.1). The equation for the 2nd parabola is:

$$y = (-0.0118) \cdot x^2 + (0) \cdot x + 17.625 \qquad \text{(x in feet, y in inches)}$$

Tendon Geometry

The tendon profile information for different points along the bridge are presented in Table 5.7.3.5. The equations presented above are in mixed units with the y values in inches and the x values in feet. To arrive at the tendon slopes in radians, the equation constants were divided by 12.

Table 5.7.3.5
Tendon Geometry

Span Point	Depth of Section (in)	Section Centroid (in)	* Tendon Centroid (in)	Tendon Eccentricity (in)	Tendon Slope (radians)	Cumulative Length of Tendon (ft)	Cumulative Angle Change (radians)
1.0/4.0	22.00	11.00	11.00	0.00	+/- 0.050	0.000/180.094	0.000/0.559
1.1/3.9	22.00	11.00	13.90	-2.90	+/- 0.038	5.505/174.588	0.013/0.546
1.2/3.8	22.00	11.00	15.97	-4.97	+/- 0.025	11.008/169.086	0.025/0.534
1.3/3.7	22.00	11.00	17.21	-6.21	+/- 0.013	16.509/163.585	0.038/0.521
1.4/3.6	22.00	11.00	17.63	-6.63	0.000	22.009/158.084	0.050/0.508
1.5/3.5	22.00	11.00	17.21	-6.21	-/+ 0.013	27.509/152.584	0.063/0.496
1.6/3.4	22.00	11.00	15.97	-4.97	-/+ 0.025	33.010/147.083	0.075/0.483
1.7/3.3	22.00	11.00	13.90	-2.90	-/+ 0.038	38.513/141.581	0.088/0.471
1.8/3.2	22.96	11.48	11.00	0.48	-/+ 0.050	44.018/136.075	0.100/0.458
1.873/3.127	25.52	12.76	8.37	4.39	-/+ 0.059	48.024/132.069	0.110/0.449
1.9/3.1	26.48	13.24	7.42	5.82	-/+ 0.047	49.527/130.567	0.122/0.436
2.0/3.0	30.00	15.00	5.88	9.13	0.000	55.029/125.065	0.169/0.390
2.1/2.9	25.52	12.76	8.37	4.39	+/- 0.059	62.033/118.061	0.224/0.334
2.2/2.8	22.00	11.00	12.42	-1.42	+/- 0.041	69.041/111.053	0.238/0.321
2.3/2.7	22.00	11.00	15.31	-4.31	+/- 0.028	76.045/104.049	0.252/0.307
2.4/2.6	22.00	11.00	17.05	-6.05	+/- 0.014	83.047/97.047	0.266/0.293
2.5	22.00	11.00	17.63	-6.63	0.000	90.047	0.279

* Measured from top of structural slab.

J. Initial Prestress Losses

Calculate the prestress losses due to friction, anchor set, and elastic shortening.

[5.9.5.2.2]

Friction Losses

An exponential equation is used to determine the friction losses at different tendon locations.

$$\Delta f_{pF} = f_{pj} \cdot \left[1 - e^{-(K \cdot x + \mu \cdot \alpha)}\right]$$

where:

f_{pj} = stress in prestressing steel at jacking (ksi)

x = length of prestressing tendon between any two points (ft)

K = wobble coefficient from LRFD Table 5.9.5.2.2b-1 <u>Use 0.0002</u>

μ = coefficient of friction from LRFD Table 5.9.5.2.2b-1 <u>Use 0.25</u>

α = absolute value of angular change of prestressing path between two points (radians)

The friction coefficients assume that the strands are installed in rigid galvanized ducts.

The ratio of the force in the tendon to the force at any location after friction losses (Friction Factor) is summarized in Table 5.7.3.6.

[5.9.5.2.1]

Anchor Set Losses

The release of the tensioning jack from the PT tendon is accomplished by engaging strand wedges in the permanent anchor plate. A small shortening displacement in the tendon is necessary to seat the wedges. During construction, the tendon displacement is dependent on the jacking equipment used (some jacks can power seat wedges, others cannot). For design, a typical seating displacement is assumed (a standard value is 0.375 inches). The effective tension in the post-tensioning tendons at the jacking end is reduced due to the shortening of the tendon. This localized loss in tendon stress is called anchor set. The effect of anchor set is represented in Figure 5.7.3.4.

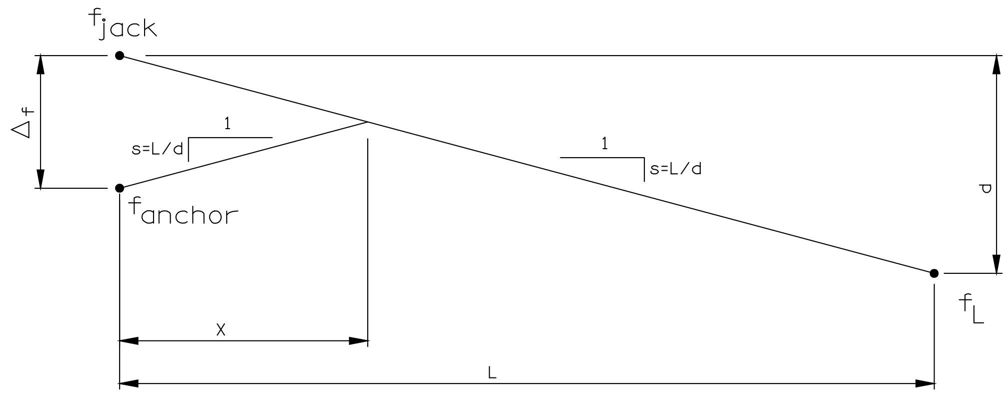


Figure 5.7.3.4

Assume the anchor set is 0.375 inches and use the friction losses at the dead end of the tendon to compute "d". Assume the tendons are stressed to 80% of GUTS (Guaranteed Ultimate Tensile Strength):

$$f_{jack} = 0.80 \cdot f_{pu} = 0.80 \cdot 270.0 = 216.0 \text{ ksi}$$

The jacking stress at the dead end after friction losses is:

$$f_L = f_{jack} \cdot (\text{friction factor}) = 216.0 \cdot 0.839 = 181.2 \text{ ksi}$$

"d" represents the friction loss between the two end points:

$$d = f_{jack} - f_L = 216.0 - 181.2 = 34.8 \text{ ksi}$$

With "L" and "d" determined, the slope of the friction loss line is known. The increase in stress in the tendon as one moves away from the jacking end is assumed to have the same slope. With that assumption, the relationship between stress loss at the anchor and the location where the anchor loss dissipates can be found:

$$X = \left(\frac{\Delta_f}{2}\right) \cdot \left(\frac{L}{d}\right)$$

The change in stress due to anchor set is found with Hooke's law:

$$\sigma = \frac{\Delta_f}{2} = E \cdot \varepsilon = E \cdot \left(\frac{\Delta_L}{12 \cdot X}\right)$$

Which can be substituted into the earlier equation:

$$X = \left(E \cdot \frac{\Delta_L}{12 \cdot X}\right) \cdot \left(\frac{L}{d}\right)$$

Which leads to:

$$X^2 = \frac{E \cdot \Delta_L \cdot L}{12 \cdot d} = \frac{28{,}500 \cdot 0.375 \cdot 180.094}{12 \cdot 34.8} = 4609$$

and

$$X = 67.9 \text{ ft}$$

Which, when put into the Hooke's law, determines the change in stress due to anchor set:

$$\Delta_f = 2 \cdot \left(\frac{E \cdot \Delta_L}{12 \cdot X}\right) = \left(\frac{E \cdot \Delta_L}{6 \cdot X}\right) = \frac{28{,}500 \cdot 0.375}{6 \cdot 67.9} = 26.2 \text{ ksi}$$

The stress at the anchor is:

$$f_{anchor} = f_{jack} - \Delta_f = 216.0 - 26.2 = 189.8 \text{ ksi}$$

The stress in the tendon between the anchor and point "X" can be found with interpolation.

[5.9.5.2.3]

Elastic Shortening Losses

Elastic shortening losses for post-tensioned structures vary with the number of tendons used and the jacking processes followed. The LRFD Specifications provide a straightforward equation to estimate the prestress losses associated with elastic shortening for design.

$$\Delta f_{pES} = 0.25 \cdot \frac{E_p}{E_{ci}} \cdot f_{cgp}$$

The concrete stress at the height of the tendon when the slab is subjected to only dead load and prestress forces is computed for Span Point 2.5 in Section N and found to be:

$$f_{cgp} = \frac{P_T}{A} + \frac{P_T \cdot e^2}{I} - \frac{M_{DL} \cdot e}{I}$$

$$= \frac{211.1}{264} + \frac{211.1 \cdot 6.63^2}{10{,}650} - \frac{(52 + 19.8) \cdot 12 \cdot 6.63}{10{,}650} = 1.1 \text{ ksi}$$

$$\Delta f_{pES} = 0.25 \cdot \frac{28{,}500}{3865} \cdot 1.1 = 2.0 \text{ ksi}$$

Table 5.7.3.6 summarizes the stresses in the tendon at tenth point span point locations. Losses associated with friction, anchor set, elastic shortening, shrinkage, creep, and relaxation (see Part N of this example for calculation of Shrinkage, Creep, and Relaxation losses) are presented. Initial and final tendon stresses are also presented.

Table 5.7.3.6
Tendon Stresses (ksi)

Span Point	Friction Factor	Jacking Stress	Anchor Set Loss	Net Stress	Reversed Net Stress	Average Tendon Stress	Elastic Shortening Loss	Initial Tendon Stress	Shrinkage Loss	Creep Loss	Relaxation Loss	Final Tendon Stress
1.0	1.000	216.0	26.2	189.8	181.2	185.5	2.0	183.5	4.5	13.0	4.7	161.3
1.1	0.996	215.1	24.1	191.0	182.0	186.5	2.0	184.5	4.5	13.0	4.7	162.3
1.2	0.992	214.2	22.0	192.2	182.7	187.5	2.0	185.5	4.5	13.0	4.7	163.3
1.3	0.987	213.3	19.8	193.4	183.5	188.5	2.0	186.5	4.5	13.0	4.7	164.3
1.4	0.983	212.4	17.7	194.7	184.3	189.5	2.0	187.5	4.5	13.0	4.7	165.3
1.5	0.979	211.5	15.6	195.9	185.1	190.5	2.0	188.5	4.5	13.0	4.7	166.3
1.6	0.975	210.6	13.5	197.1	185.9	191.5	2.0	189.5	4.5	13.0	4.7	167.3
1.7	0.971	209.7	11.3	198.3	186.7	192.5	2.0	190.5	4.5	13.0	4.7	168.3
1.8	0.967	208.8	9.2	199.6	187.5	193.5	2.0	191.5	4.5	13.0	4.7	169.3
1.9	0.960	207.4	7.1	200.3	188.7	194.5	2.0	192.5	4.5	13.0	4.7	170.3
2.0	0.948	204.8	5.0	199.8	191.1	195.5	2.0	193.5	4.5	13.0	4.7	171.3
2.1	0.934	201.7	2.3	199.4	194.0	196.7	2.0	194.7	4.5	13.0	4.7	172.5
2.2	0.929	200.7	0.0	200.7	195.0	197.9	2.0	195.9	4.5	13.0	4.7	173.7
2.3	0.925	199.8	0.0	199.8	195.9	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.4	0.920	198.8	0.0	198.8	196.9	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.5	0.916	197.8	0.0	197.8	197.8	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.6	0.911	196.9	0.0	196.9	198.8	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.7	0.907	195.9	0.0	195.9	199.8	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.8	0.903	195.0	0.0	195.0	200.7	197.9	2.0	195.9	4.5	13.0	4.7	173.7
2.9	0.898	194.0	0.0	194.0	199.4	196.7	2.0	194.7	4.5	13.0	4.7	172.5
3.0	0.885	191.1	0.0	191.1	199.8	195.5	2.0	193.5	4.5	13.0	4.7	171.3
3.1	0.874	188.7	0.0	188.7	200.3	194.5	2.0	192.5	4.5	13.0	4.7	170.3
3.2	0.868	187.4	0.0	187.4	199.6	193.5	2.0	191.5	4.5	13.0	4.7	169.3
3.3	0.864	186.7	0.0	186.7	198.3	192.5	2.0	190.5	4.5	13.0	4.7	168.3
3.4	0.860	185.9	0.0	185.9	197.1	191.5	2.0	189.5	4.5	13.0	4.7	167.3
3.5	0.857	185.1	0.0	185.1	195.9	190.5	2.0	188.5	4.5	13.0	4.7	166.3
3.6	0.853	184.3	0.0	184.3	194.7	189.5	2.0	187.5	4.5	13.0	4.7	165.3
3.7	0.850	183.5	0.0	183.5	193.4	188.5	2.0	186.5	4.5	13.0	4.7	164.3
3.8	0.846	182.7	0.0	182.7	192.2	187.5	2.0	185.5	4.5	13.0	4.7	163.3
3.9	0.842	182.0	0.0	182.0	191.0	186.5	2.0	184.5	4.5	13.0	4.7	162.3
4.0	0.839	181.2	0.0	181.2	189.8	185.5	2.0	183.5	4.5	13.0	4.7	161.3

K. Check Stress Limits on Prestressing Strands [Table 5.9.3-1]

Stress Limits for Prestressing Strands:

Prior to seating	$f_s \le 0.90 \cdot f_{py} = 218.7$ ksi
At anchorages after anchor set	$f_s \le 0.70 \cdot f_{pu} = 189.0$ ksi
End of seating zone after anchor set	$f_s \le 0.74 \cdot f_{pu} = 199.8$ ksi
At service limit after losses	$f_s \le 0.80 \cdot f_{py} = 194.4$ ksi

A review of the values in Table 5.7.3.6 indicates that none of the stress limits are exceeded.

L. Summary of Analysis Results

From this point forward, the design of an interior strip at points of maximum positive and negative moment subject to dead and live loads will be presented. The design procedure for the edge strip is similar. A summary of bending moments obtained at different locations along the superstructure for a 1 foot wide design strip is presented in Table 5.7.3.7. The analysis results are symmetric about midspan of the center span.

Secondary Post-Tensioning Forces

The linear haunches complicate the analysis of the slab superstructure. The nonprismatic section, combined with the parabolic tendon profiles prevent an easy hand method from being used to determine the secondary moments associated with post-tensioning a continuous superstructure. Therefore, a beam analysis program was used to determine secondary post-tensioning forces.

The tensioning of the tendon redistributes the dead load reactions of the superstructure. For the design example the redistribution was an increase in the abutment reaction of 0.87 kips and a corresponding reduction in the pier reactions of 0.87 kips. This implies that the positive moment regions of the tendon profile introduced slightly more curvature into the superstructure than the negative moment regions. The secondary moments associated with the redistribution amount to a linearly increasing positive moment in the side spans (0.0 kip-ft at the abutments and 47.9 kip-ft at the piers). The secondary moment in the center span is a constant positive value of 47.9 kip-ft.

Table 5.7.3.7

Interior Strip Moment Summary (per foot)

Span Point	M_{DC} (kip-ft)	PT Secondary Moments (kip-ft)	* Truck + Lane (kip-ft)		* Tandem + Lane (kip-ft)		* 0.9 (Truck Tr + Lane) (kip-ft)
			Max.	Min.	Max.	Min.	
1.0	0	0.0	0	0	0	0	-
1.1	31	4.8	32.5	-6.3	28.9	-5.2	-
1.2	51	9.6	54.1	-12.5	49.2	-10.1	-
1.3	62	14.4	65.8	-18.6	61.3	-15.3	-
1.4	62	19.1	70.6	-24.8	66.1	-20.3	-
1.5	52	23.9	68.9	-31.1	64.6	-25.4	-
1.6	31	28.7	61.6	-37.4	57.6	-30.5	-
1.7	1	33.5	46.8	-43.5	45.4	-35.6	-35.4
1.8	-40	38.3	26.9	-49.9	29.4	-40.8	-45.4
1.9	-92	43.1	15.7	-59.7	13.8	-49.5	-62.3
2.0	-155	47.9	16.5	-72.8	13.8	-61.3	-86.4
2.1	-78	47.9	11.0	-48.3	15.1	-41.2	-56.9
2.2	-20	47.9	30.3	-37.0	32.5	-30.9	-35.0
2.3	22	47.9	52.2	-30.5	49.7	-25.6	-
2.4	47	47.9	66.3	-25.3	61.5	-21.5	-
2.5	55	47.9	70.0	-20.1	65.0	-17.4	-

* Includes dynamic load allowance.

M. Check Stress Limits on Concrete

The service limit state stresses at each of the critical locations are evaluated using the general equation (compression +, tension -):

$$f = \frac{P}{A} + \frac{M_P}{S} + \frac{M_S}{S}$$

where M_P is the total prestress moment and M_S is the service moment. The stress limits are:

[5.9.4]

At Transfer

Tension $f_t = 0$ ksi

Compression $f_c \le 0.60\ f'_{ci} = 2.7$ ksi

At Final

Tension $f_t = 0$ ksi

Compression

$DC + PT + LL + IM$ $f_c \le 0.60\ f'_c = 3.0$ ksi

$DC + PT$ $f_c \le 0.45\ f'_c = 2.25$ ksi

$^1/_2\ (DC + PT) + LL + IM$ $f_c \le 0.40\ f'_c = 2.0$ ksi

Check Location 1.0 (Interior Strip)

Unfactored DC and PT Secondary Moment $= 0$ kip-ft

Tendon stress at transfer $= 183.5$ ksi

Tendon stress at final $= 183.5 - 0.11 \cdot (0.80 \cdot 270) = 159.7$ ksi

(assumes 11% long term losses)

Area of strand per foot $= \dfrac{12(0.217)}{2.42} = 1.078$ in^2/ft

Prestress force at transfer: $P_i = 183.5 \cdot 1.078 = 197.8$ kips

Prestress force at final: $P_f = 159.7 \cdot 1.078 = 172.2$ kips

Prestress eccentricity: $e = 0$ in

Concrete area: $A = 12 \cdot 22 = 264$ in^2

Concrete section modulus: $S = \dfrac{12 \cdot 22^2}{6} = 968$ in^3

Check the stress in the concrete. Because the dead and live load moment and the prestress eccentricity are all equal to zero, the top and bottom fiber concrete stress is the same:

At transfer: $f_b = f_t = \dfrac{P_i}{A} = \dfrac{197.8}{264} = 0.75$ ksi < 2.7 ksi <u>OK</u>

At final: $f_b = f_t = \dfrac{P_f}{A} = \dfrac{172.2}{264} = 0.65$ ksi < 2.0 ksi <u>OK</u>

Check Location 1.4 (Interior Strip)

Unfactored DC and PT Secondary Moment:

$M_{DC+PT} = 62 + 19.1 = 81.1 \text{ kip-ft}$

Service I Moment: $M_{SI} = 62 + 19.1 + 70.6 = 151.7 \text{ kip-ft}$

Service III Moment: $M_{SIII} = 62 + 19.1 + 0.8 \cdot (70.6) = 137.6 \text{ kip-ft}$

Tendon stress at transfer $= 187.5 \text{ ksi}$

Tendon stress at final $= 187.5 - 0.11 \cdot (0.80 \cdot 270) = 163.7 \text{ ksi}$

(assumes 11% long term losses)

Area of strand per foot $= \dfrac{12(0.217)}{2.42} = 1.078 \text{ in}^2/\text{ft}$

Prestress force at transfer: $P_i = 187.5 \cdot 1.078 = 202.1 \text{ kips}$

Prestress force at final: $P_f = 163.7 \cdot 1.078 = 176.5 \text{ kips}$

Prestress eccentricity: $e = 6.63 \text{ in}$

Concrete area: $A = 12 \cdot 22 = 264 \text{ in}^2$

Concrete section modulus: $S = \dfrac{12 \cdot 22^2}{6} = 968 \text{ in}^3$

Check the bottom fiber stress at transfer:

$$f_b = \frac{P_i}{A} + \frac{P_i \cdot e}{S} - \frac{M_{DC+PT}}{S} = \frac{202.1}{264} + \frac{202.1 \cdot 6.63}{968} - \frac{81.1 \cdot 12}{968}$$

$= 1.14 \text{ ksi} < 2.7 \text{ ksi}$ <u>OK</u>

Check the bottom fiber stress at final:

$$f_b = \frac{P_f}{A} + \frac{P_f \cdot e}{S} - \frac{M_{SIII}}{S} = \frac{176.5}{264} + \frac{176.5 \cdot 6.63}{968} - \frac{137.6 \cdot 12}{968}$$

$= 0.17 \text{ ksi} > 0 \text{ ksi}$ <u>OK</u>

Check the top fiber stress at transfer:

$$f_t = \frac{P_i}{A} - \frac{P_i \cdot e}{S} + \frac{M_{DC+PT}}{S} = \frac{202.1}{264} - \frac{202.1 \cdot 6.63}{968} + \frac{81.1 \cdot 12}{968}$$

$= 0.39 \text{ ksi} > 0 \text{ ksi}$ <u>OK</u>

Check the top fiber compressive stress at final:

For DC + PT + LL + IM,

$$f_t = \frac{P_f}{A} + \frac{P_f \cdot e}{S} + \frac{M_{SI}}{S} = \frac{176.5}{264} - \frac{176.5 \cdot 6.63}{968} + \frac{151.7 \cdot 12}{968}$$

$= 1.34 \text{ ksi} < 3.0 \text{ ksi}$ <u>OK</u>

By inspection, the compressive stresses due to DC + PT and $^1/_2$ (DC + PT) + LL + IM are less than the allowables.

Check Location 2.0 (Interior Strip)

Unfactored DC and PT Secondary Moment:

$$M_{DC+PT} = -155 + 47.9 = -107.1 \text{ kip-ft}$$

Service I Moment: $M_{SI} = -155 + 47.9 - 86.4 = -193.5 \text{ kip-ft}$

Service III Moment: $M_{SIII} = -155 + 47.9 + 0.8 \cdot (-86.4) = -176.2 \text{ kip-ft}$

Tendon stress at transfer $= 193.5 \text{ ksi}$

Tendon stress at final $= 193.5 - 0.11 \cdot (0.80 \cdot 270) = 169.7 \text{ ksi}$

(assumes 11% long term losses)

Area of strand per foot $= \dfrac{12(0.217)}{2.42} = 1.078 \text{ in}^2/\text{ft}$

Prestress force at transfer: $P_i = 193.5 \cdot 1.078 = 208.6 \text{ kips}$

Prestress force at final: $P_f = 169.7 \cdot 1.078 = 182.9 \text{ kips}$

Prestress eccentricity: $e = 9.13 \text{ in}$

Concrete area: $A = 12 \cdot 30 = 360 \text{ in}^2$

Concrete section modulus: $S = \dfrac{12 \cdot 30^2}{6} = 1800 \text{ in}^3$

Check the top fiber stress at transfer:

$$f_t = \frac{P_i}{A} + \frac{P_i \cdot e}{S} - \frac{M_{DC+PT}}{S} = \frac{208.6}{360} + \frac{208.6 \cdot 9.13}{1800} - \frac{107.1 \cdot 12}{1800}$$

$= 0.92 \text{ ksi} < 2.7 \text{ ksi}$ <u>OK</u>

Check the top fiber stress at final:

$$f_t = \frac{P_f}{A} + \frac{P_f \cdot e}{S} - \frac{M_{SIII}}{S} = \frac{182.9}{360} + \frac{182.9 \cdot 9.13}{1800} - \frac{176.2 \cdot 12}{1800}$$

$= 0.26 \text{ ksi} > 0 \text{ ksi}$ <u>OK</u>

Check the bottom fiber stress at transfer:

$$f_b = \frac{P_i}{A} - \frac{P_i \cdot e}{S} + \frac{M_{DC+PT}}{S} = \frac{208.6}{360} - \frac{208.6 \cdot 8.13}{1800} + \frac{107.1 \cdot 12}{1800}$$

$= 0.23 \text{ ksi} > 0 \text{ ksi}$ <u>OK</u>

Check the bottom fiber compressive stress at final:

For DC + PT + LL + IM,

$$f_b = \frac{P_f}{A} + \frac{P_f \cdot e}{S} + \frac{M_{SI}}{S} = \frac{182.9}{360} - \frac{182.9 \cdot 9.13}{1800} + \frac{193.5 \cdot 12}{1800}$$

$= 0.87 \text{ ksi} < 3.0 \text{ ksi}$ <u>OK</u>

By inspection, the compressive stresses due to DC + PT and $^1/_2$ (DC + PT) + LL + IM are less than the allowables.

Check Location 2.5 (Interior Strip)

Unfactored DC and PT Secondary Moment:

$M_{DC+PT} = 55 + 47.9 = 102.9$ kip-ft

Service I Moment: $M_{SI} = 55 + 47.9 + 70.0 = 172.9$ kip-ft

Service III Moment: $M_{SIII} = 55 + 47.9 + 0.8 \cdot (70.0) = 158.9$ kip-ft

Tendon stress at transfer $= 195.8$ ksi

Tendon stress at final $= 195.8 - 0.11 \cdot (0.80 \cdot 270) = 172.0$ ksi

(assumes 11% long term losses)

Area of strand per foot $= \dfrac{12(0.217)}{2.42} = 1.078\ \text{in}^2/\text{ft}$

Prestress force at transfer: $P_i = 195.8 \cdot 1.078 = 211.1$ kips

Prestress force at final: $P_f = 172.0 \cdot 1.078 = 185.4$ kips

Prestress eccentricity: $e = 6.63$ in

Concrete area: $A = 12 \cdot 22 = 264\ \text{in}^2$

Concrete section modulus: $S = \dfrac{12 \cdot 22^2}{6} = 968\ \text{in}^3$

Check the bottom fiber stress at transfer:

$$f_b = \frac{P_i}{A} + \frac{P_i \cdot e}{S} - \frac{M_{DC+PT}}{S} = \frac{211.1}{264} + \frac{211.1 \cdot 6.63}{968} - \frac{102.9 \cdot 12}{968}$$

$= 0.97$ ksi < 2.7 ksi <u>OK</u>

Check the bottom fiber stress at final:

$$f_b = \frac{P_f}{A} + \frac{P_f \cdot e}{S} - \frac{M_{SIII}}{S} = \frac{185.4}{264} + \frac{185.4 \cdot 6.63}{968} - \frac{158.9 \cdot 12}{968}$$

$= 0.002$ ksi > 0 ksi <u>OK</u>

Check the top fiber stress at transfer:

$$f_t = \frac{P_i}{A} - \frac{P_i \cdot e}{S} + \frac{M_{DC+PT}}{S} = \frac{211.1}{264} - \frac{211.1 \cdot 6.63}{968} + \frac{102.9 \cdot 12}{968}$$

$= 0.63$ ksi > 0 ksi <u>OK</u>

Check the top fiber compressive stress at final:

For DC + PT + LL + IM,

$$f_t = \frac{P_f}{A} + \frac{P_f \cdot e}{S} + \frac{M_{SI}}{S} = \frac{185.4}{264} - \frac{185.4 \cdot 6.63}{968} + \frac{172.9 \cdot 12}{968}$$

$= 1.58$ ksi < 3.0 ksi <u>OK</u>

By inspection, the compressive stresses due to DC + PT and $^1/_2$ (DC + PT) + LL + IM are less than the allowables.

N. Time-Dependent Losses (Refined Method) [5.9.5.4]

Use Location 2.5 to calculate losses due to shrinkage, creep and relaxation because the highest effective prestressing force occurs at this location. This will result in conservative values for creep and relaxation losses.

Shrinkage

[5.9.5.4.2]

$$f_{pSR} = (13.5 - 0.125 \cdot H)$$

$H =$ relative humidity (use 73%)

$$f_{pSR} = [13.5 - 0.125 \cdot (73)] = 4.5 \text{ ksi}$$

Creep

[5.9.5.4.3]

The moment associated with the wear course and barriers for a 1 foot wide section of slab is 4 kip-ft.

$$\Delta f_{pCR} = 12.0 \cdot f_{cgp} - 7.0 \cdot \Delta f_{cdp}$$

$f_{cgp} = 1.1$ ksi (calculated earlier in Part J)

$$\Delta f_{cdp} = \frac{M_{DW} \cdot e}{I} = \frac{4 \cdot (12) \cdot 6.63}{10{,}650} = 0.030 \text{ ksi}$$

$$\Delta f_{pCR} = 12.0 \cdot (1.1) - 7.0 \cdot (0.030) = 13.0 \text{ ksi}$$

Relaxation

[5.9.5.4.4]

For low-relaxation strands:

$$0.30 \cdot \Delta f_{pR2} = 0.30 \cdot [20.0 - 0.3 \cdot \Delta f_{pF} - 0.4 \cdot \Delta f_{pES} - 0.2 \cdot (\Delta f_{pSR} + \Delta f_{pCR})]$$

If the friction losses are such that the tendon stresses after jacking are above $0.70 \cdot f_{pu}$, then Δf_{pF} is assumed equal to zero.

$$= 0.30 \cdot [20.0 - 0.3 \cdot (0) - 0.4 \cdot (2.0) - 0.2 \cdot (4.5 + 13.0)]$$

$$= 4.7 \text{ ksi}$$

Total Losses

Total time-dependent losses $= 4.5 + 13.0 + 4.7 = 22.2$ ksi
(10.3% of $0.80 \cdot f_{pu}$)

The computed losses of 10.3% are approximately the same as those assumed in the stress checks (11%). If the computed losses are significantly different from the assumed, designers will need to recalculate the stresses based on a new assumed loss and iterate until the computed and assumed losses converge.

O. Investigate Strength Limit State – Flexure [5.7.3.3.2]

The flexural strength of the slab needs to be sufficient to carry factored loads associated with the strength limit state and also satisfy the maximum and minimum reinforcement checks.

Check Location 1.4 (Interior Strip)

Compute the Strength I design moment for a 1 foot wide strip of slab:

$$M_u = 1.0 \cdot [1.25 \cdot (62) + 1.00 \cdot (19.1) + 1.75 \cdot (70.6)] = 220 \text{ kip-ft}$$

Determine the theoretical cracking moment for the cross section (M_{cr}).

To compute the maximum cracking moment, use the prestress force at transfer (202.1 kips).

Solve for the moment that produces f_r at the bottom of the section:

$$M_{cr} = (f_r + f_{PTS}) \cdot S$$

[5.4.2.6]

The assumed rupture or cracking stress for concrete is:

$$f_r = 0.24 \cdot \sqrt{f'_c} = 0.24 \cdot \sqrt{5.0} = 0.537 \text{ ksi}$$

The stress due to prestressing (including secondary moments) is:

$$f_{PTS} = \frac{P}{A} + \frac{P \cdot e}{S} - \frac{M_{secondary}}{S}$$

$$= \frac{202.1}{264} + \frac{202.1 \cdot 6.63}{968} - \frac{19.1 \cdot 12}{968} = 1.913 \text{ ksi}$$

The cracking moment is:

$$M_{cr} = 2372 \text{ kip-in} = 198 \text{ kip-ft}$$

$$1.2 M_{cr} = 238 \text{ kip-ft}$$

[5.7.3.2]
[5.7.3.1.1]

Compute the capacity neglecting any benefit from mild steel. Use the equations for bonded tendons:

$k = 0.28$ (LRFD Table C5.7.3.1.1-1)

$$d_p = 11.00 + 6.625 = 17.625 \text{ in}$$

$$c = \left(\frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \right)$$

$$= \left(\frac{10.78 \cdot 270}{0.85 \cdot 5.0 \cdot 0.80 \cdot 12 + 0.28 \cdot 1.078 \cdot \frac{270}{17.625}} \right) = 6.41 \text{ in}$$

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 270 \cdot \left(1 - 0.28 \cdot \frac{6.41}{17.625}\right) = 242.5 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.80 \cdot 6.41 = 5.13 \text{ in}$$

The flexural resistance can be computed as:

$$\phi \cdot M_n = \phi \cdot A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) = 1.0 \cdot 1.078 \cdot 242.5 \cdot \left(17.625 - \frac{5.13}{2}\right)$$

$$= 3937 \text{ kip-in} = 328 \text{ kip-ft}$$

which is greater than $1.2 \cdot M_{cr}$ (238 kip-ft) and M_u (220 kip-ft)

Check Location 2.0 (Interior Strip)

Using the moments given in Table 5.7.3.7:

$$M_u = 1.0 \cdot [1.25 \cdot (155) + 1.00 \cdot (47.9) + 1.75 \cdot (86.4)] = 393 \text{ kip-ft}$$

Solve for the moment that produces f_r at the bottom of the section:

$$f_b = f_r = \frac{P}{A} + \frac{P \cdot e}{S} + \frac{M_{secondary}}{S} - \frac{M_{cr}}{S}$$

$$-0.537 = \frac{208.6}{360} + \frac{208.6 \cdot 9.13}{1800} + \frac{47.9 \cdot 12}{1800} - \frac{M_{cr}}{1800}$$

$$M_{cr} = 4489 \text{ kip-in} = 374 \text{ kip-ft}$$

$$1.2 \cdot M_{cr} = 449 \text{ kip-ft}$$

[5.7.3.2] Compute the capacity neglecting any benefit from mild steel.

$k = 0.28$ (LRFD Table C5.7.3.1-1)

$$d_p = 15.00 + 9.13 = 24.13 \text{ in}$$

$$c = \left(\frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \right)$$

$$= \left(\frac{1.078 \cdot 270}{0.85 \cdot 5.0 \cdot 0.80 \cdot 12 + 0.28 \cdot 1.078 \cdot \frac{270}{24.13}} \right) = 6.59 \text{ in}$$

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 270 \cdot \left(1 - 0.28 \cdot \frac{6.59}{24.13}\right) = 249.4 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.80 \cdot 6.59 = 5.27 \text{ in}$$

The flexural resistance can be computed as:

$$\phi \cdot M_n = \phi \cdot A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) = 1.0 \cdot 1.078 \cdot 249.4 \cdot \left(24.13 - \frac{5.27}{2}\right)$$

$$= 5779 \text{ kip-in} = 482 \text{ kip-ft}$$

which is greater than $1.2 \cdot M_{cr}$ (449 kip-ft) and M_u (393 kip-ft).

Check Location 2.5 (Interior Strip)

Using the moments given in Table 5.7.3.7:

$$M_u = 1.0 \cdot [1.25 \cdot (55) + 1.00 \cdot (47.9) + 1.75 \cdot (70)] = 239 \text{ kip-ft}$$

Solve for the moment that produces f_r at the bottom of the section:

$$f_b = f_r = \frac{P}{A} + \frac{P \cdot e}{S} + \frac{M_{secondary}}{S} - \frac{M_{cr}}{S}$$

$$-0.537 = \frac{211.1}{264} + \frac{211.1 \cdot 6.63}{968} + \frac{47.9 \cdot 12}{968} - \frac{M_{cr}}{968}$$

$$M_{cr} = 2119 \text{ kip-in} = 177 \text{ kip-ft}$$

$$1.2 \cdot M_{cr} = 212 \text{ kip-ft}$$

[5.7.3.2] Compute the capacity neglecting any benefit from mild steel.

$k = 0.28$ (LRFD Table C5.7.3.1-1)

$$d_p = 11.00 + 6.625 = 17.625 \text{ in}$$

$$c = \left(\frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}\right)$$

$$= \left(\frac{1.078 \cdot 270}{0.85 \cdot 5.0 \cdot 0.80 \cdot 12 + 0.28 \cdot 1.078 \cdot \frac{270}{17.625}}\right) = 6.41 \text{ in}$$

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 270 \cdot \left(1 - 0.28 \cdot \frac{6.41}{17.625}\right) = 242.5 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.80 \cdot 6.41 = 5.13 \text{ in}$$

The flexural resistance can be computed as:

$$\phi \cdot M_n = \phi \cdot A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) = 1.0 \cdot 1.078 \cdot 242.5 \cdot \left(17.625 - \frac{5.13}{2}\right)$$

$$= 3937 \text{ kip-in} = 328 \text{ kip-ft}$$

which is greater than M_u (239 kip-ft) and $1.2 \cdot M_{cr}$ (212 kip-ft).

Check Principal Stresses

[Future manual content]

[5.7.3.3.1]

Check Maximum Reinforcement

The maximum amount of reinforcement permitted in a section is based on the ratio of the depth of the section in compression compared to the depth of the distance to the tension reinforcement from the compression side of the section. The ratio can be no more than 0.42. When calculating "d", do not include the wearing course.

For Span Point 1.4:

$$\frac{c}{d} = \frac{6.41}{17.625} = 0.36 \qquad \underline{\text{OK}}$$

For Span Point 2.0:

$$\frac{c}{d} = \frac{6.59}{24.13} = 0.27 \qquad \underline{\text{OK}}$$

For Span Point 2.5:

$$\frac{c}{d} = \frac{6.41}{17.625} = 0.36 \qquad \underline{\text{OK}}$$

P. Shear
[5.13.3.6]

The shear force components for a typical 1 foot wide strip of slab are summarized in Table 5.7.3.8.

Table 5.7.3.8
Shear Summary (per foot)

Span Point	V_{DC} (kips)	PT Secondary Shear (kips)	* Truck + Lane (kips)		* Tandem + Lane (kips)	
			Max	Min	Max	Min
1.0	6.5	0.87	4.9	-0.8	4.3	-0.7
1.1	4.7	0.87	4.1	-0.8	3.6	-0.7
1.2	2.8	0.87	3.4	-0.8	3.0	-1.0
1.3	0.9	0.87	2.7	-1.3	2.5	-1.5
1.4	-0.9	0.87	2.0	-2.0	1.9	-2.0
1.5	-2.8	0.87	1.5	-2.6	1.5	-2.5
1.6	-4.6	0.87	1.0	-3.3	1.1	-3.0
1.7	-6.5	0.87	0.6	-4.0	0.7	-3.5
1.8	-8.3	0.87	0.2	-4.7	0.4	-4.0
1.9	-10.4	0.87	0.2	-5.3	0.2	-4.4
2.0	12.4	0.0	6.0	-4.7	4.9	-3.7
2.1	9.6	0.0	5.2	-0.7	4.4	-0.6
2.2	7.1	0.0	4.5	-0.7	3.8	-0.7
2.3	4.7	0.0	3.7	-1.0	3.3	-1.1
2.4	2.4	0.0	2.9	-1.5	2.7	-1.5
2.5	0.0	0.0	2.2	-2.2	2.1	-2.1

* Includes dynamic load allowance.

The LRFD Specifications do not require that a shear check be performed, however MnDOT design practice is to do so.

To minimize the effort associated with the shear check, conservatively check the largest design shear force on a non-haunch portion of the slab. If the check is satisfied, all sections of the slab can be considered adequate. If the check is not satisfied additional investigation is necessary.

The Strength I design shear at Span Point 2.0 is:

$$V_u = 1.25 \cdot (12.4) + 1.00 \cdot (0.0) + 1.75 \cdot (6.0) = 26.0 \text{ kips}$$

Investigate LRFD Equation 5.8.3.4.2-2. No axial load is applied. Neglect any mild flexural reinforcement and any beneficial vertical prestressing effect. As a starting point, assume θ is equal to 30 degrees.

$$\varepsilon_x = \left[\frac{\dfrac{M_u}{d_v} + 0.5 \cdot N_u + 0.5 \cdot \left(V_u - V_p\right) \cdot \cot\theta - A_{ps} \cdot f_{po}}{E_s \cdot A_s + E_p \cdot A_{ps}} \right]$$

Use the M_u and d_v from Span Point 1.4

$$\varepsilon_x = \frac{\dfrac{220 \cdot 12}{15.84} + 0.5 \cdot 0 + 0.5 \cdot 26 \cdot \left(\cot 30°\right) - 1.078 \cdot 0.7 \cdot 270}{29{,}000 \cdot 0 + 28{,}500 \cdot 1.078}$$

$$= -0.000244$$

Determine the crack spacing parameter next. Use d_v for s_x

[Eqn. 5.8.3.4.2-4]

$$s_{xe} = s_x \cdot \frac{1.38}{a_g + 0.63} = 15.84 \cdot \frac{1.38}{1.5 + 0.63} = 10.3$$

With the strain and crack parameters determined, refer to Table 5.8.3.4.2-2 to determine the appropriate β and θ values for use in computing the shear capacity of the concrete. Use the values in the cell for $s_{xe} < 15$ and $\varepsilon_x < -0.20$ ($\beta = 5.34$ and $\theta = 29.5°$).

The required nominal shear capacity is:

$$V_n = \frac{V_u}{\phi_v} = \frac{26.0}{0.9} = 28.9 \text{ kips}$$

The shear capacity of the concrete is:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 5.34 \cdot \sqrt{5.0} \cdot 12 \cdot 15.84$$

$$= 0.0316 \cdot 5.34 \cdot \sqrt{5.0} \cdot 12 \cdot 15.84 = 71.7 >> 28.9 \text{ kips}$$

Q. Minimum Longitudinal Reinforcement [5.8.3.5]

[Future manual content]

R. Distribution Reinforcement [5.14.4.1]

The minimum amount of transverse reinforcement in a horizontal plane shall be taken as a percentage of the main reinforcement:

$$\frac{100}{\sqrt{L}} \cdot \frac{f_{pe}}{60} \le 50\%$$

For Spans 1 and 3

$$\frac{100}{\sqrt{55}} \cdot \frac{170.3}{60} = 38\%$$

Interior Strip: Maximum positive moment

Positive moment prestressing $= 1.078 \text{ in}^2/\text{ft}$

Transverse reinforcement $= 0.38 \cdot (1.078) = 0.41 \text{ in}^2/\text{ft}$

Use #6 @ 12", $A_s = 0.44 \text{ in}^2/\text{ft}$

S. Shrinkage and Temperature Reinforcement [5.10.8.2]

Using an average thickness of 26 inches, the required temperature steel is:

$$A_s \ge 0.11 \cdot \frac{A_g}{f_y} = 0.11 \cdot \left(\frac{12 \cdot 26}{60}\right) = 0.57 \text{ in}^2/\text{ft each direction, both faces}$$

Half should be placed in each face:

$$A_s = \frac{1}{2} \cdot (0.57) = 0.29 \text{ in}^2/\text{ft each direction}$$

Use #5 @ 12", $A_s = 0.31 \text{ in}^2/\text{ft}$

T. Deformations [2.5.2.6]

Dead Load Deflection Plus Prestress Camber

The total weight of the superstructure is used for dead load deflections. The gross moment of inertias are used and a computer analysis is run to obtain instantaneous deflections. The results of the computer analysis, (dead load deflections and camber due to prestress) are presented below. Using the long-term multipliers (from Section 4.6 of the PCI Handbook with composite topping), the long-term deflections are calculated as:

	Δ at release (Spans 1 and 3) (in)	Δ at release (Span 2) (in)	Multiplier	Δ final (Spans 1 and 3) (in)	Δ final (Span 2) (in)
Prestress	+0.92	0.73	2.20	2.02	1.61
w_{DC}	-0.46	-0.50	2.40	-1.10	-1.20
Total	0.46	0.23		0.92	0.41

With a net upward deflection in all spans, the slab is cambered downward. The camber is equal to the camber at release plus $^1/_2$ of the long-term camber.

Spans 1 and 3:

$$0.46 + \frac{1}{2} \cdot (0.92) = 0.92 \text{ in}$$

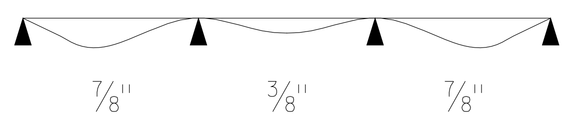
Round down and use $^7/_8$ in

Span 2:

$$0.23 + \frac{1}{2} \cdot (0.41) = 0.43 \text{ in}$$

Round down and use $^3/_8$ in

Total Camber



[2.5.2.6.2] **Live Load Deflections**

$$\text{Allowable } \Delta_{LL+I} = \frac{\text{Span}}{800}$$

$$= \frac{55 \cdot 12}{800} = 0.83 \text{ in (Span 1 and 3)}$$

$$= \frac{70 \cdot 12}{800} = 1.05 \text{ in (Span 2)}$$

[3.6.1.3.2] Two live load cases are evaluated as part of the live load deflection check. One is the design truck alone. The other is lane load combined with 25% of the truck load deflection.

A computer analysis (based on gross nonprismatic section properties) had the following deflections for a full lane of live load:

Spans 1 and 3:

Truck deflection: 6.24 in/lane
Lane deflection: 2.59 in/lane

Truck check:

$$= (1 + IM) \cdot (\Delta_{truck}) \cdot (\text{distribution factor})$$

$$= 1.33 \cdot 6.24 \cdot 0.054 = 0.45 < 0.83 \text{ in}$$ OK

Lane/truck check:

$$= 0.25 \cdot (1 + IM) \cdot (\Delta_{truck} + \Delta_{lane}) \cdot (\text{distribution factor})$$

$$= 0.25 \cdot 1.33 \cdot (6.24 + 2.59) \cdot (0.054) = 0.25 < 0.83 \text{ in}$$ OK

Span 2:

Truck deflection: 8.83 in/lane
Lane deflection: 3.60 in/lane

Truck check:

$$= (1 + IM) \cdot (\Delta_{truck}) \cdot (\text{distribution factor})$$

$$= 1.33 \cdot 8.83 \cdot 0.054 = 0.63 < 1.05 \text{ in}$$ OK

Lane/truck check:

$$= 0.25 \cdot (1 + IM) \cdot (\Delta_{truck} + \Delta_{lane}) \cdot (\text{distribution factor})$$

$$= 0.25 \cdot 1.33 \cdot (8.83 + 3.60) \cdot (0.054) = 0.35 < 1.05 \text{ in}$$ OK

U. Anchorage Zone [5.10.9]

Anchorages are designed at the strength limit state for the factored jacking force.

Due to the simplicity of the geometry of the anchorage and the lack of substantial deviation in the force flow path, the approximate procedure described in LRFD Article 5.10.9.6 is used.

For a 12- 0.6" diameter strand tendon, use a square anchorage plate with a side dimension of 12.875 inches (a and b). Assume a duct outer diameter of 6.25 inches.

[5.10.9.6.2]

General Zone Compressive Stresses

Determine the allowable concrete compressive stress from:

$$f_{ca} = \frac{0.6 \cdot P_u \cdot K}{A_b \cdot \left[1 + \ell_c \cdot \left(\frac{1}{b_{eff}} - \frac{1}{t}\right)\right]}$$

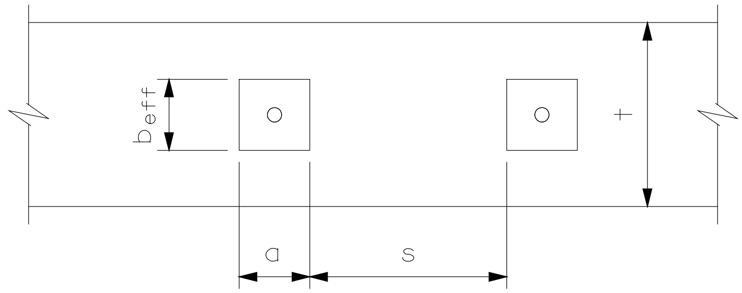


Figure 5.7.3.5
Anchorage Dimensions

Determine the value for K based on the spacing of the tendons and the size of the anchorage plate.

$s = 29 > 2 \cdot a = 25.75$ <u>Use K = 1</u>

The jacking force is:

$$P_{jack} = (\text{jacking stress}) \cdot (\text{\# of strands}) \cdot A_{strand}$$
$$= 216 \cdot 12 \cdot 0.217 = 562 \text{ kips}$$

[3.4.3.2] The factored tendon force for anchorage design is:

$$P_u = 1.2 \cdot (562) = 674.4 \text{ kips}$$

$$b_{eff} = 12.875 \text{ in}$$

$$\ell_c = 1.15 \cdot b_{eff} = 1.15 \cdot (12.875) = 14.81 \text{ in}$$

$$t = 22 \text{ in}$$

$$A_b = (a \cdot b_{eff}) - \frac{\pi \cdot D^2}{4} = (12.875)^2 - \frac{\pi \cdot 6.25^2}{4} = 135.1 \text{ in}^2$$

$$f_{ca} = \frac{0.6 \cdot 674.4 \cdot 1.0}{135.1 \cdot \left[1 + 14.81 \cdot \left(\frac{1}{12.875} - \frac{1}{22}\right)\right]} = 2.03 \text{ ksi}$$

[5.10.9.3.1] The factored concrete compressive strength for the general zone shall not exceed $0.7 \cdot \phi \cdot f'_{ci}$.

$\phi = 0.80$ for compression in anchorage zones

$f_{ca} \le 0.7 \cdot \phi \cdot f'_{ci} = 0.7 \cdot (0.8) \cdot 4.5 = 2.52 \text{ ksi}$

Therefore, use $f_{ca} = 2.03 \text{ ksi}$

Determine the compressive stress at a distance equal to the plate's smaller dimension. Assume the load distributes at an angle of 30°.

$$A_e = (a + 2 \cdot \tan 30^\circ \cdot a)^2 - \frac{\pi \cdot D^2}{4}$$

$$= (12.875 + 2 \cdot 0.577 \cdot 12.875)^2 - \frac{\pi \cdot 6.25^2}{4} = 739 \text{ in}^2$$

$$f_e = \frac{P_u}{A_e} = \frac{674.4}{739} = 0.90 \text{ ksi} < f_{ca} = 2.03 \text{ ksi}$$ OK

[5.10.9.6.3]

General Zone Bursting Force

The tendon slope at the ends of the superstructure from Table 5.7.3.5 is 0.050 radians (3 degrees).

The bursting forces in the anchorage is calculated as:

$$T_{burst} = 0.25 \cdot P_u \cdot \left(1 - \frac{a}{h}\right) + 0.5 \cdot \left| P_u \cdot \sin\alpha \right|$$

$$= 0.25 \cdot (674.4) \cdot \left(1 - \frac{12.875}{22}\right) + 0.5 \cdot (674.4) \cdot 0.052 = 87.5 \text{ kips}$$

$$d_{burst} = 0.5 \cdot (h - 2 \cdot e) + 5 \cdot e \cdot (\sin\alpha)$$ (for this example, e = 0)

$$= 0.5 \cdot (22) = 11 \text{ in}$$

Using $\phi = 1.00$ for tension in steel in anchorage zones, then $\phi \cdot f_y = 1.0 \cdot (60) = 60$ ksi:

$$A_s \text{ req'd} = \frac{87.5}{60} = 1.46 \text{ in}^2 \text{ (spaced within } 2.5 \cdot d_{burst} = 27.5 \text{ in)}$$

Use 4 - #5 closed stirrups spaced at 6 inches (refer to Figure 5.7.3.6).

$$A_s = 4 \cdot (0.31) \cdot 2 \text{ legs/stirrup} = 2.48 > 1.46 \text{ in}^2$$ OK

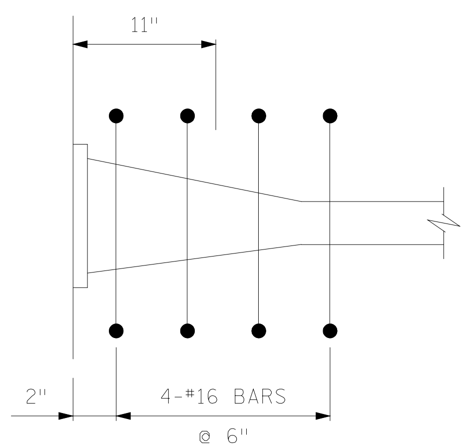


Figure 5.7.3.6
Bursting Force Reinforcing

[5.10.9.3.2]

General Zone Edge Tension Forces

Edge tension forces are the tensile forces in the anchorage zone acting close to the transverse edge (spalling forces) and longitudinal edges (longitudinal edge tension forces). For the case of a concentrically loaded anchorage zone, the longitudinal edge tension forces are insignificant, and the magnitude of the design spalling force may be taken as 2% of the total post-tensioning force.

$$\text{Spalling Force} = 0.02 \cdot (674.4) = 13.5 \text{ kips}$$

Using $\phi \cdot f_y = 1.0 \cdot (60) = 60 \text{ ksi}$:

$$A_s \text{ req'd} = \frac{13.5}{60} = 0.22 \text{ in}^2$$

Use 2- #5 bars, $A_s = 0.62 \text{ in}^2$

V. Summary of Final Design

A summary of the primary reinforcement for the slab is provided in Figure 5.7.3.7. A typical transverse half section is illustrated for the midspan section and for the section over the piers.

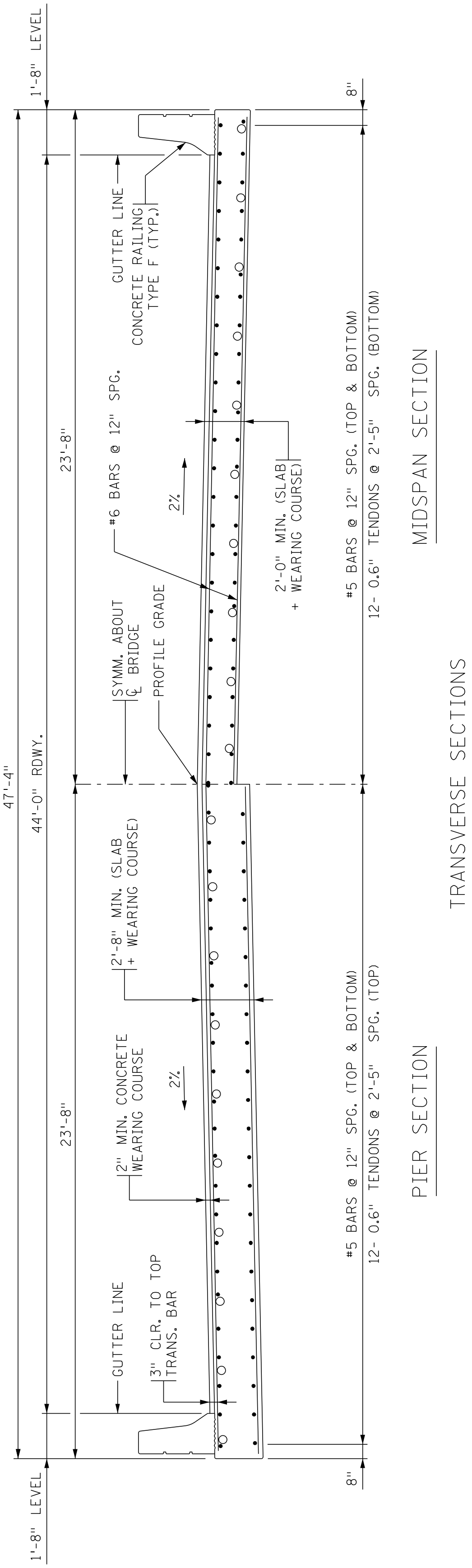

1'-8" LEVEL
47'-4"
44'-0" RDWY.
23'-8"
GUTTER LINE
CONCRETE RAILING TYPE F (TYP.)
#6 BARS @ 12" SPG.
2%
SYMM. ABOUT ℄ BRIDGE
PROFILE GRADE
2'-0" MIN. (SLAB + WEARING COURSE)
8"
#5 BARS @ 12" SPG. (TOP & BOTTOM)
12- 0.6" TENDONS @ 2'-5" SPG. (BOTTOM)
MIDSPAN SECTION
TRANSVERSE SECTIONS
2'-8" MIN. (SLAB + WEARING COURSE)
2" MIN. CONCRETE WEARING COURSE
2%
23'-8"
GUTTER LINE
3" CLR. TO TOP TRANS. BAR
#5 BARS @ 12" SPG. (TOP & BOTTOM)
12- 0.6" TENDONS @ 2'-5" SPG. (TOP)
PIER SECTION
8"
1'-8" LEVEL

Figure 5.7.3.7

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APPENDIX 5-A

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

DECKS:

Top Transverse Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 for bar size and spacing. A Class A splice is provided where all top transverse bar splices occur between beams, with 50% of the bars spliced at a given location. A Class B splice is provided where 100% of the bars are spliced at a given location between beams or where 50% of the bars are spliced at a given location over beams. Avoid splicing 100% of bars over beams.

Top Transverse Deck Bar Lap Splice Lengths

Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	All Splices Between Beams and 50% are at Same Location *(preferred)*	100% of Splices at Same Location Between Beams or 50% of Splices Over Beams at Same Location
3"	> 5"	#4	1'-6"	1'-11"
		#5	1'-10"	2'-5"
		#6	2'-2"	2'-10"
	5"	#4	1'-6"	1'-11"
		#5	1'-10"	2'-5"
		#6	2'-9"	3'-7"

Top Longitudinal Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 & Figure 9.2.1.6 or Table 9.2.1.2 & Figure 9.2.1.7 for bar size and spacing. Detail reinforcement such that no more than 50% of top longitudinal bars are spliced at any cross-section through the deck (Class A splice).

Top Longitudinal Deck Bar Lap Splice Lengths

Concrete Cover to Bar Being Considered	Bar Size	Lap Splice Length
≥ 3 1/2"	#4	1'-6"
	#5	1'-10"
	#6	2'-2"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

DECKS: (cont'd)

Bottom Transverse Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 for bar size and spacing. A Class A splice is provided where all bottom transverse bars are spliced over beams, with 50% of the bars spliced at a given location. A Class B splice is provided where 100% of the bars are spliced at a given location over beams or where 50% of the bars are spliced at a given location between beams. Avoid splicing 100% of bars between beams.

Bottom Transverse Deck Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	All Splices Over Beams and 50% are at Same Location *(preferred)*	100% of Splices at Same Location Over Beams or 50% of Splices Between Beams at Same Location
1"	≥ 4"	#4	1'-10"	2'-5"
		#5	2'-9"	3'-6"
		#6	3'-9"	4'-10"

Bottom Longitudinal Deck Bars

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 & Figure 9.2.1.7 for bar size and spacing. A Class B splice is provided. Where possible, detail such that no more than 50% of the bottom longitudinal deck bars are spliced at a given cross-section through the deck.

Bottom Longitudinal Deck Bar Lap Splice Lengths			
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	50% of Splices at Same Location *(Preferred)* or 100% of Splices at Same Location
≥ 1 1/2"	≥ 4"	#4	1'-11"
		#5	3'-0"
		#6	3'-7"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

ABUTMENTS:

Abutment and Wingwall Vertical Bars

Back face vertical bars are all spliced at the same location, so a Class B splice is used. See LRFD 5.11.3.1. Front face bars are conservatively assumed to act as tension reinforcement, so compressive development lengths are not used in splice length computations. Although all front face bars are spliced at the same location, excess reinforcement is provided, so a Class A splice is used.

Abutment and Wingwall Vertical Bar Lap Splice Lengths

Concrete Cover to Bar Being Considered	Bar Size	Back Face Bar Spacing			Front Face Bar Spacing
		4"	5"	≥6"	≥6"
≥ 2"	#4	--	--	--	1'-6"
	#5	3'-0"	2'-5"	2'-5"	1'-10"
	#6	3'-7"	3'-7"	3'-7"	2'-9"
	#7	4'-6"	4'-2"	4'-2"	3'-2"
	#8	5'-11"	4'-9"	4'-9"	3'-8"
	#9	7'-6"	6'-0"	5'-10"	--
	#10	9'-6"	7'-7"	7'-2"	--
	#11	11'-8"	9'-4"	8'-8"	--
	#14	--	13'-5"	11'-10"	--

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

ABUTMENTS: (cont'd)

Abutment and Wingwall Horizontal Bars

All horizontal bars are assumed to have more than 12" of concrete cast below. For abutments, horizontal bars are assumed to provide excess reinforcement, so a Class A splice is used. For long wingwalls on separate footings, horizontal bars become primary reinforcement, so a Class B splice is used.

Abutment and Wingwall Horizontal Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Size	Abutment Horizontal Bar Spacing	Wingwall Horizontal Bar Spacing	
		≥ 6"	4"	≥ 5"
≥ 2"	#4	1'-11"	2'-6"	2'-6"
	#5	2'-5"	3'-4"	3'-1"
	#6	3'-1"	4'-0"	4'-0"
	#7	3'-7"	5'-1"	4'-8"
	#8	4'-1"	6'-8"	5'-4"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

PIERS:

Pier Cap Top Longitudinal Bars

All horizontal bars are assumed to have more than 12" of concrete cast below. For splices between columns where no more than 50% of the bars are spliced at the same location, a Class A splice is used. For all other cases, use a Class B splice.

Pier Cap Top Longitudinal Bar Lap Splice Lengths

Concrete Cover to Bar Being Considered	Bar Size	All Splices Located Between Columns and ≤ 50% of Bars Are Spliced at Same Location				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
≥ 2 1/2"	#5	2'-7"	2'-5"	2'-5"	2'-5"	2'-5"
	#6	3'-1"	3'-1"	2'-10"	2'-10"	2'-10"
	#7	3'-11"	3'-7"	3'-7"	3'-7"	3'-7"
	#8	5'-2"	4'-1"	4'-1"	4'-1"	4'-1"
	#9	6'-6"	5'-3"	4'-9"	4'-8"	4'-8"
	#10	8'-3"	6'-7"	6'-0"	5'-6"	5'-6"
	#11	10'-2"	8'-2"	7'-5"	6'-10"	6'-8"
	#14	--	11'-9"	10'-8"	9'-9"	9'-1"
	Bar Size	All Splices Located Between Columns and > 50% of Bars Are Spliced at Same Location				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
	#5	3'-4"	3'-1"	3'-1"	3'-1"	3'-1"
	#6	4'-0"	4'-0"	3'-8"	3'-8"	3'-8"
	#7	5'-1"	4'-8"	4'-8"	4'-8"	4'-8"
	#8	6'-8"	5'-4"	5'-4"	5'-4"	5'-4"
	#9	8'-6"	6'-9"	6'-2"	6'-0"	6'-0"
	#10	10'-9"	8'-7"	7'-10"	7'-2"	7'-2"
	#11	13'-3"	10'-7"	9'-8"	8'-10"	8'-7"
	#14	--	15'-3"	13'-10"	12'-9"	11'-10"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

PIERS: (cont'd)

Pier Cap Bottom Longitudinal Bars

For splices over columns where no more than 50% of the bars are spliced at the same location, a Class A splice is used. For all other cases, use a Class B splice.

Pier Cap Bottom Longitudinal Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	All Splices Located Over Columns and ≤ 50% of Bars Are Spliced at Same Location				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
≥ 2 1/2"	#5	2'-3"	1'-10"	1'-10"	1'-10"	1'-10"
	#6	2'-9"	2'-9"	2'-2"	2'-2"	2'-2"
	#7	3'-6"	3'-2"	3'-2"	3'-2"	3'-2"
	#8	4'-6"	3'-8"	3'-8"	3'-8"	3'-8"
	#9	5'-9"	4'-7"	4'-2"	4'-1"	4'-1"
	#10	7'-4"	5'-10"	5'-4"	4'-11"	4'-10"
	#11	9'-0"	7'-2"	6'-7"	6'-0"	5'-10"
	#14	--	10'-4"	9'-5"	8'-8"	8'-1"
	Bar Size	All Splices Located Over Columns and > 50% of Bars Are Spliced at Same Location				
		Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
	#5	3'-0"	2'-5"	2'-5"	2'-5"	2'-5"
	#6	3'-7"	3'-7"	2'-10"	2'-10"	2'-10"
	#7	4'-6"	4'-2"	4'-2"	4'-2"	4'-2"
	#8	5'-11"	4'-9"	4'-9"	4'-9"	4'-9"
	#9	7'-6"	6'-0"	5'-5"	5'-4"	5'-4"
	#10	9'-6"	7'-7"	6'-11"	6'-4"	6'-4"
	#11	11'-8"	9'-4"	8'-6"	7'-10"	7'-7"
	#14	--	13'-5"	12'-3"	11'-3"	10'-5"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

PIERS: (cont'd)

Other Pier Cap Longitudinal Bars Located on Side Faces of Pier Cap

Longitudinal bars located on the side faces of pier caps (typically skin or shrinkage and temperature reinforcement) are assumed to have more than 12" of concrete cast below. For these bars, a Class B splice is used.

Lap Splice Lengths for Longitudinal Bars Located on Side Faces of Pier Cap		
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing ≥ 4"
≥ 2 1/2"	#4	2'-6"
	#5	3'-4"
	#6	4'-0"
	#7	5'-1"

Pier Column Vertical Bars

For pier columns, all splices occur at the same location, so a Class B splice is used.

Pier Column Vertical Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing				
		4"	5"	5 1/2"	6"	≥ 6 1/2"
≥ 2 3/8"	#6	3'-7"	3'-7"	2'-10"	2'-10"	2'-10"
	#7	4'-6"	4'-2"	4'-2"	4'-2"	4'-2"
	#8	5'-11"	4'-9"	4'-9"	4'-9"	4'-9"
	#9	7'-6"	6'-0"	5'-5"	5'-4"	5'-4"
	#10	9'-6"	7'-7"	6'-11"	6'-4"	6'-4"
	#11	11'-8"	9'-4"	8'-6"	7'-10"	7'-7"
	#14	--	13'-5"	12'-3"	11'-3"	10'-5"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

SLAB SPANS:

Top Bars

This table applies to both top longitudinal and transverse bars. All bars are assumed to have more than 12" of concrete cast below. A Class B splice is used.

Top Longitudinal and Transverse Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing				
		4"	5"	6"	7"	≥ 8"
≥ 3"	#4	2'-6"	2'-6"	2'-6"	2'-6"	2'-6"
	#5	3'-4"	3'-1"	3'-1"	3'-1"	3'-1"
	#6	4'-0"	4'-0"	3'-8"	3'-8"	3'-8"
	#7	5'-1"	4'-8"	4'-8"	4'-4"	4'-4"
	#8	6'-8"	5'-4"	5'-4"	4'-11"	4'-11"
	#9	8'-6"	6'-9"	6'-0"	6'-0"	6'-0"
	#10	10'-9"	8'-7"	7'-2"	6'-9"	6'-9"
	#11	13'-3"	10'-7"	8'-10"	7'-7"	7'-6"
	#14	--	15'-3"	12'-9"	10'-11"	9'-11"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

> Based on LRFD 5.11.2 and 5.11.5
> Use of epoxy coated bars is assumed
> Excess reinforcement factor λ_{er} is taken equal to 1.0

SLAB SPANS: (cont'd)

Bottom Bars

The table applies to both bottom longitudinal and transverse bars. A Class B splice is used.

Bottom Longitudinal and Transverse Bar Lap Splice Lengths			
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing	
		4"	≥ 5"
≥ 1 1/2"	#4	1'-11"	1'-11"
	#5	3'-0"	3'-0"
	#6	3'-7"	3'-7"
	#7	4'-8"	4'-8"
	#8	5'-11"	5'-11"
	#9	7'-6"	7'-3"
	#10	9'-6"	8'-11"
	#11	11'-8"	10'-7"
	#14	--	14'-4"

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