9. DECKS AND DECK SYSTEMS

Reinforced concrete decks on girders are the predominant type of deck used on highway bridges in Minnesota. The deck is the structural element that transfers vehicle and pedestrian loads to the girders. It is analyzed as a continuous beam with the girders acting as supports. The top and bottom primary moment resisting steel runs transversely in the deck. The stool between the beam top flange and the deck bottom varies to allow placement of the deck to the proper elevation.

Timber decks may be used on secondary roads and temporary bridges as part of the superstructure. Guidance for the design of timber decks is provided in Section 8.

Specialized deck systems are used for railroad bridges. A common design is a thru-girder system with floor beams supporting a bent plate. This channel shaped bent plate holds the ballast on which the rails are supported. These specialized deck systems are not currently covered in this manual.

9.1 General

Deck Protection Policy

The following practices are used to extend the service life of new concrete bridge decks:

- All reinforcement bars shall be epoxy coated. Also, use epoxy coated reinforcement when widening a bridge or when adding a new railing.
- The top reinforcing bars shall have a total of 3 inches of cover.
- Primary bridges shall be constructed with a 2 inch low slump concrete wearing course.

Primary bridges are defined as:

- All bridges carrying interstate traffic.
- All interstate highway bridges at an interchange with access to the interstate route.
- All bridges carrying trunk highway traffic within major metropolitan areas and municipalities with populations of 5,000 or greater.
- All bridges on highways with a 20 year projected ADT greater than 2,000.

The State Bridge Engineer shall determine the appropriate action on any exceptions to this policy.

9.1.1 Deck Drainage

Deck Drainage Considerations

The design of a deck requires:

- Removing potential hydroplaning water from the driving surface using a crown cross-section.
- Channeling drainage water away from the bridge and features below the bridge using road grades and end slopes respectively.

Superstructure Drains

Drain outlets shall be avoided over roadways, shoulders, sidewalks, streams, railroad tracks, or end slopes. Drains placed over riprap will require the area to be grouted, or a grouted flume section provided. At down spouts or deck drains provide splash blocks.

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

Drainage systems shall avoid direct runoff to waters of the State. Bridges over lakes or streams, where bridge length is less than 500 feet, shall be designed such that deck drains are not necessary. Narrow bridges that are longer than 500 feet may have problems with deck flooding in severe rainstorms. Discuss this issue with the Hydraulics Unit prior to beginning final design.

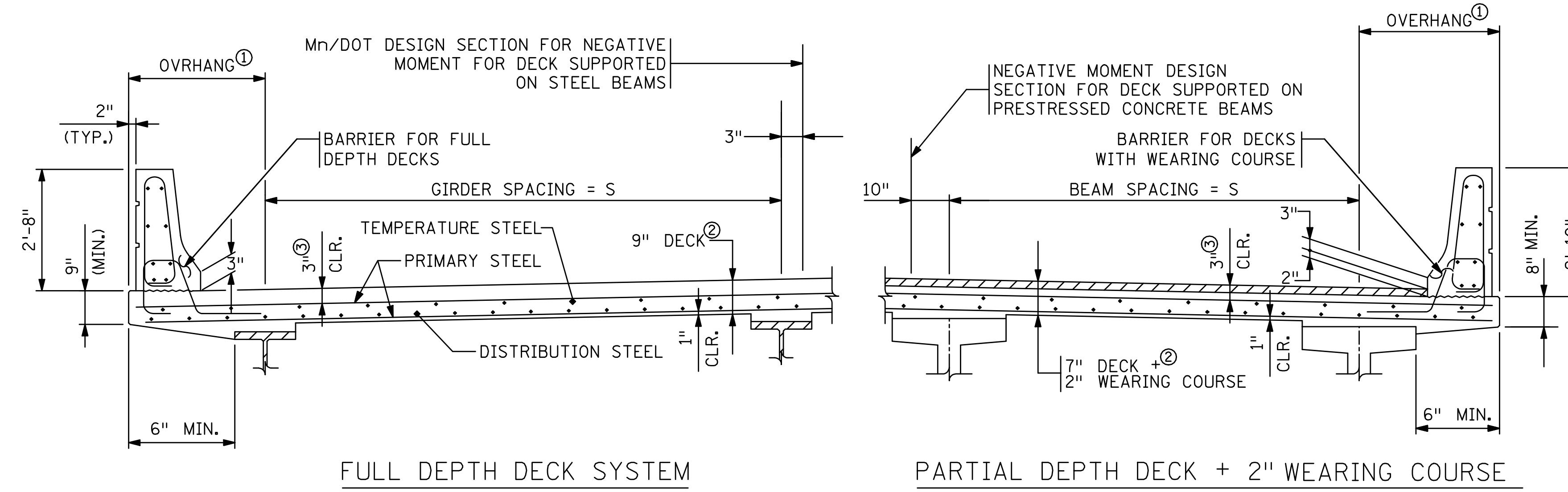
Also note that special drainage requirements are necessary for bridges where a Corps of Engineers "404 permit" is required. The Hydraulic's Unit may also require the addition of containment and treatment features to the project for bridges located in or near scenic waterways or near public water supply sources.

The materials and gages for corrugated metal (C.M.) drains, and semi-circle deck drains, such as those used on railroad bridges, are to be provided in the plan details. Use 16 gage metal for other C.M. drains.

Drains shall extend a minimum of 1 inch below the bottom of superstructure. See Standard Bridge Detail B701, B702, B705, or B706.

9.2 Concrete Deck on Beams

Figure 9.2.1 illustrates the two most common concrete deck systems used. The deck system selected is based on the protection policy. The left side of the figure shows a deck constructed with a single concrete pour. The right side illustrates a deck with a wearing course.



GEOMETRY FOR CONCRETE DECK DESIGN

NOTE: EITHER DECK SYSTEM CAN BE USED WITH EITHER SUPERSTRUCTURE

① OVERHANG LIMITATIONS:

- LESS THAN BEAM DEPTH
- LESS THAN 40% OF BEAM SPACING
- LESS THAN [2'-8" + 0.5 FLANGE WIDTH]

② REQD. DECK THICKNESS IS DEPENDANT ON BEAM SPACING (SEE TABLES 9.2.1.1 & 9.2.1.2). FOR PEDESTRIAN BRIDGES, USE 7" DECK THICKNESS WITH NO WEARING COURSE

③ FOR PEDESTRIAN BRIDGES, PROVIDE 2" CLEAR

Figure 9.2.1

The wearing course is less permeable and consequently reduces the rate at which chlorides penetrate into the deck.

9.2.1 Deck Design and Detailing

Design

The traditional approximate method of analysis shall be used in deck design. Do not use the empirical deck design method shown in LRFD 9.7.2. The deck shall be treated as a continuous beam. Moments as provided in LRFD Table A4.1-1 are to be applied at the design sections shown in Figure 9.2.1. The use of LRFD Table A4.1-1 must be within the assumptions and limitations listed at the beginning of the appendix. Tables 9.2.1.1 and 9.2.1.2 provide minimum reinforcement requirements based on the traditional deck design method for decks supported on prestressed concrete beams and steel beams, respectively. The tables may be used for all LRFD deck designs that fit the assumptions, as well as for decks of bridges designed by the AASHTO Standard Specifications Load Factor method (bridge widenings).

The transverse reinforcement given in Tables 9.2.1.1 and 9.2.1.2 is adequate for deck overhangs (measured from centerline of beam to edge of deck) up to 40% of the beam spacing when a Type F concrete barrier, which meets Test Level 4 (Standard Details Part II Figures 5-397.114 through 5-397.117) is used.

The amount of longitudinal steel placed in decks is increased in the negative moment regions over the piers. The amount of steel must be consistent with the superstructure modeling assumptions. If precast beams are made continuous over the piers an appropriate amount of reinforcement must be included in the deck to provide adequate negative moment capacity. Similarly for steel beams, the amount of longitudinal reinforcement must be consistent with the design section property assumptions. For steel beam or girder superstructures, the LRFD specifications require at least one percent reinforcing over the piers. See Figure 9.2.1.9 for additional information.

[6.10.1.7]

The design of the distribution steel for the entire bridge shall be based on the widest beam spacing found in any span.

The top longitudinal steel in non-pier areas shall satisfy the requirements for shrinkage and temperature reinforcement.

For skews less than or equal to 20°, detail deck reinforcement parallel to the skew. For design of the reinforcement, use the beam spacing measured along the skew for the deck span length.

For skews greater than 20°, provide reinforcing at right angles to the centerline of roadway. For this case, use the beam spacing measured normal to the roadway centerline for the deck span length.

Overhangs are to be designed to meet the strength requirements of Section 13. LRFD A13.4.2 specifies that the vehicle collision force to be used in deck overhang design is to be equal to the rail capacity R_w. This ensures that the deck will be stronger than the rail and that the yield line failure mechanism will occur in the parapet. For example, the interior panel of a TL-4 F-rail on a deck with no wearing course has a capacity $R_w = 124.1$ kips (see Table 13.2.4.1in this manual), which is well above the rail design collision force $F_t = 54$ kips for a Test Level 4 railing. Because of the large difference between rail capacity and collision force, Mn/DOT requires the deck overhang to carry the lesser of the rail capacity R_w or $^4/_3 \times F_t$.

Geometry

Figures 9.2.1.4 through 9.2.1.7 contain standard Mn/DOT deck details. Typical deck reinforcement layouts at deck edges and medians are illustrated in the figures.

Use a uniform deck thickness for all spans based on the minimum thickness required for the widest beam spacing.

The main transverse reinforcement will vary with the beam spacing. For skewed bridges, continue the reinforcement for the wider beam spacing until the reinforcement is completely outside of the span with the wider beam spacing.

The standard height of bridge sidewalks is 8 inches above the top of roadway. Bridge medians shall match approach roadway median shape and height.

Use a uniform thickness for the edge of deck in all spans. Use a 9 inch minimum thickness on structures without a wearing course. Use an 8 inch minimum thickness on bridges with a wearing course or sidewalk.

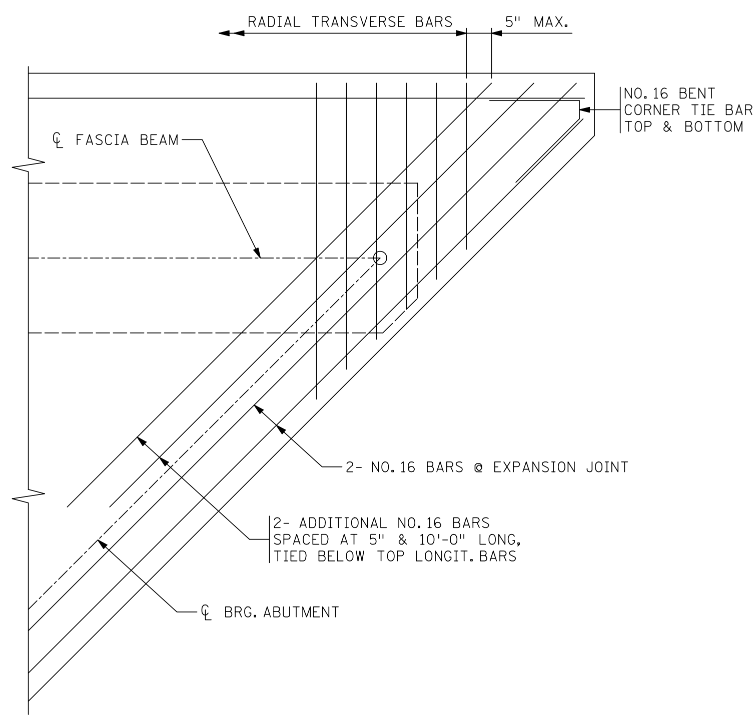
Dimension the bottom of deck on the outside of the fascia beam at 1 inch below the top of the beam for prestressed concrete beams. For steel beams, detail the bottom of deck on the outside of the fascia beam to meet the bottom of the top flange. See Figures 9.2.1.4 through 9.2.1.7.

Check the slope of the bottom of the deck on overhangs. The edge of the deck should be higher than the location next to the beam top flange.

Detailing

The main transverse deck reinforcement shall consist of straight bars located in both the top and the bottom reinforcing mats.

For the acute corners of highly skewed bridges, detail the deck reinforcement as follows: In addition to the 2-#16 bars that run parallel to the expansion joint at the end of the deck, place 2 top mat #16 bars that are 10 feet long and run parallel to the joint with a spacing of 5 inches. Also, run a series of radial transverse bars that shorten as they progress into the corner. Finally, place a bent bar in the corner that ties to the outside deck longitudinal bar and the end bar running parallel to the joint. See Figure 9.2.1.1.



TYPICAL DECK REINFORCEMENT PLAN FOR HIGHLY SKEWED CORNERS

Figure 9.2.1.1

Add a longitudinal tie at the end of the deck if the deck projects past the end of the diaphragm more than 1 foot.

Several detailing practices are to be used near piers:

- Detail longitudinal steel (temperature and distribution) as continuous over piers.
- Provide additional longitudinal steel to minimize transverse deck cracking. See Figures 9.2.1.8 and 9.2.1.9.
- For decks supported on non-continuous prestressed beams, detail a partial depth sawcut in the deck over the pier backfilled with a sealant. See Figure 9.2.1.10.
- Place polystyrene on the corners of prestressed concrete beam bridges with skews greater than 20° to reduce wandering of the transverse deck crack at the centerline of pier. See Figure 9.2.1.10.

For bridges with transverse deck reinforcement parallel to the skew, dimension transverse bar spacing along edge of deck.

Deck Placement Sequence

One contributor to through-deck transverse cracking is inadequate sequencing of deck pours. A deck placement sequence shall be provided for the following types of bridges:

- Bridges with decks wider than 90 feet.
- Continuous bridges with spans exceeding 150 feet.
- Bridges where the concrete placement rate is lower than 60% of the span length per hour. (Note that a single pump truck can be assumed to maintain a pour rate of 70 cubic yards per hour.)

Generally, for continuous superstructures containing span lengths between 150 and 200 feet, locate the transverse construction joint for the first pour at the 0.6 point of the first span. Start the following pour at the 0.6 point of the adjacent span and proceed toward and terminate at the end of the previous pour. Continue this pattern for all interior spans. The last placement will extend from the end of the bridge to the previous placement. A typical deck placement sequence for a 3 span bridge fitting the above criteria is shown in Figure 9.2.1.2.

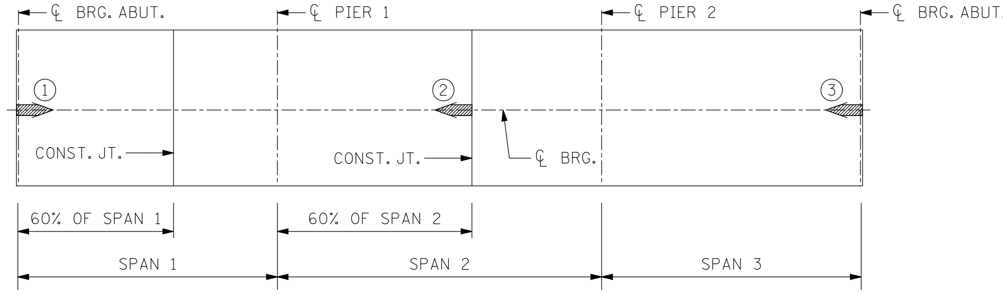


Figure 9.2.1.2

For continuous superstructures containing span lengths greater than 200 feet, locate construction joints at points of dead load contraflexure on the deck placement plan. Positive moment sections are to be placed prior to negative moment sections. Sequence pours so as to minimize upward deflections in previously placed spans (i.e. longer pour sections should be placed before shorter adjacent sections). An acceptable pour sequence for a multi-span bridge fitting the above criteria is shown in Figure 9.2.1.3. Since adjacent spans may not be poured within 72 hours of each other, the second pour is permitted to be the next most flexible section after the first pour. Note that the third and fourth pours require placement of both positive and negative moment sections. If the contractor will be unable to complete the placement of the entire section in one pour, the positive moment area is to be placed first followed by the negative sections.

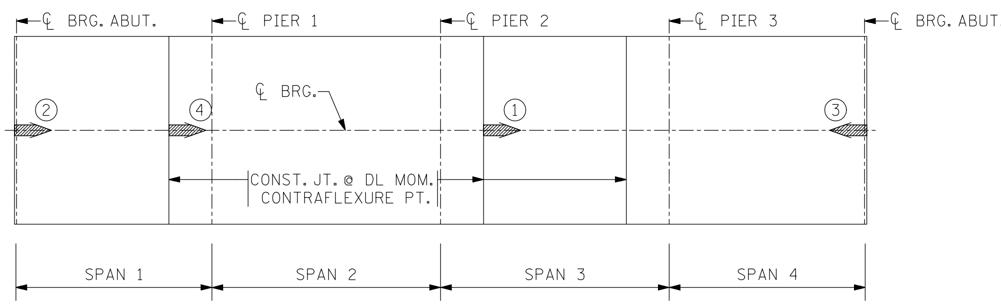


Figure 9.2.1.3

For superstructures which consist of a series of simply supported spans that require a deck placement sequence, transverse construction joints shall be located at the end of each span.

In all cases, a minimum of 72 hours must be provided between adjacent deck pours.

For unusual span length configuration, discuss the deck placement sequence with the Regional Construction Engineer.

REINFORCEMENT FOR DECK ON PRESTRESSED CONCRETE BEAMS

(Negative Moment @ 10 inches from CL I-Beam & 8.7 inches from CL Rectangular Beam)

Maximum Beam Spacing①	Transverse Reinforcement				Deck Thickness ②	Longitudinal Reinforcement Bottom③	Longitudinal Reinforcement Top③
	Bottom		Top				
	w/ Wearing Course	w/o Wearing Course	Deck on I-Beam	Deck on Rect. Beam			
5'-0"	13 @ 5"	13 @ 6.5"	13 @ 10"	13 @ 9.5"	9''	13 @ 7"	13 @ 1'-6"
5'-6"	13 @ 5"	13 @ 6"	13 @ 9"	13 @ 8.5"	9''	13 @ 7"	13 @ 1'-6"
6'-0"	16 @ 7"	13 @ 6"	13 @ 8.5"	13 @ 7.5"	9''	16 @ 10"	13 @ 1'-6"
6'-6"	16 @ 7"	13 @ 6"	13 @ 7.5"	13 @ 7"	9''	16 @ 10"	13 @ 1'-6"
7'-0"	16 @ 7"	13 @ 6"	13 @ 6.5"	13 @ 6"	9''	16 @ 10"	13 @ 1'-6"
7'-6"	16 @ 7"	13 @ 6"	13 @ 6"	13 @ 5.5"	9''	16 @ 10"	13 @ 1'-6"
8'-0"	16 @ 7"	13 @ 6"	13 @ 5.5"	13 @ 5"	9''	16 @ 10"	13 @ 1'-6"
8'-6"	16 @ 7"	13 @ 6"	13 @ 5"	13 @ 5"	9''	16 @ 10"	13 @ 1'-6"
9'-0"	16 @ 7"	13 @ 6"	13 @ 5"	16 @ 7"	9''	16 @ 10"	13 @ 1'-6"
9'-6"	16 @ 6.5"	13 @ 5.5"	16 @ 7"	16 @ 7"	9''	16 @ 9"	13 @ 1'-6"
10'-0"	16 @ 6"	13 @ 5"	16 @ 7"	16 @ 6.5"	9''	16 @ 8"	13 @ 1'-6"
10'-6"	16 @ 6"	13 @ 5"	16 @ 6.5"	16 @ 6"	9''	16 @ 8"	13 @ 1'-6"
11'-0"	16 @ 5.5"	16 @ 7.5"	16 @ 6"	16 @ 6"	9''	16 @ 8"	13 @ 1'-6"
11'-6"	16 @ 5.5"	16 @ 7"	16 @ 5.5"	16 @ 5.5"	9''	16 @ 8"	13 @ 1'-6"
12'-0"	19 @ 7"	16 @ 6.5"	16 @ 5.5"	16 @ 5"	9''	16 @ 7"	13 @ 1'-6"
12'-6"	19 @ 7"	16 @ 6.5"	16 @ 5"	16 @ 5"	9''	16 @ 7"	13 @ 1'-6"
13'-0"	19 @ 7"	16 @ 6.5"	16 @ 5"	16 @ 5"	9 1/2''	16 @ 7"	13 @ 1'-6"
13'-6"	19 @ 7"	16 @ 6.5"	16 @ 5"	16 @ 5"	9 3/4''	16 @ 7"	13 @ 1'-6"
14'-0"	19 @ 7.5"	16 @ 6.5"	16 @ 5"	19 @ 6"	10''	16 @ 7"	13 @ 1'-6"
14'-6"	19 @ 7.5"	16 @ 6.5"	16 @ 5"	19 @ 6"	10 1/4''	16 @ 7"	13 @ 1'-6"
15'-0"	19 @ 7.5"	16 @ 6.5"	16 @ 5"	19 @ 6"	10 1/2''	16 @ 7"	13 @ 1'-6"

① For skews ≤ 20°, beam spacing is measured along the skew.
For skews > 20°, beam spacing is measured normal to roadway centerline.

② Deck thickness includes wearing course.

③ Reinforcement shown is for bridges where beams are not continuous at piers. Note that additional reinforcement may be required when beams are continuous at piers. See Figure 9.2.1.6 for additional top reinforcement required at piers when only deck is continuous.

Design Assumptions:

- Live load moments are from LRFD Table A4.1-1.
- The 2" wearing course is sacrificial and is not used in determining a structural depth d for bottom steel.
- The control of cracking by distribution of flexural reinforcement requirements have been met using a clear cover of 1" for bottom steel and limiting clear cover for calculations of d_c to 2" for top steel with a γ_e=0.75.
- The LRFD code (under empirical design) states the ratio of the effective beam spacing to slab thickness should be less than 18 (Ontario uses 15); this slab thickness increase fits these requirements and is similar to what we have used successfully in the past.
- A future wearing course of 20 psf with a load factor of 1.25 has been used.
- Concrete strength of 4 ksi; reinforcing steel strength of 60 ksi.

Table 9.2.1.1

REINFORCEMENT FOR DECK ON STEEL BEAMS
(Negative Moment @ 3 inches from CL Beam)

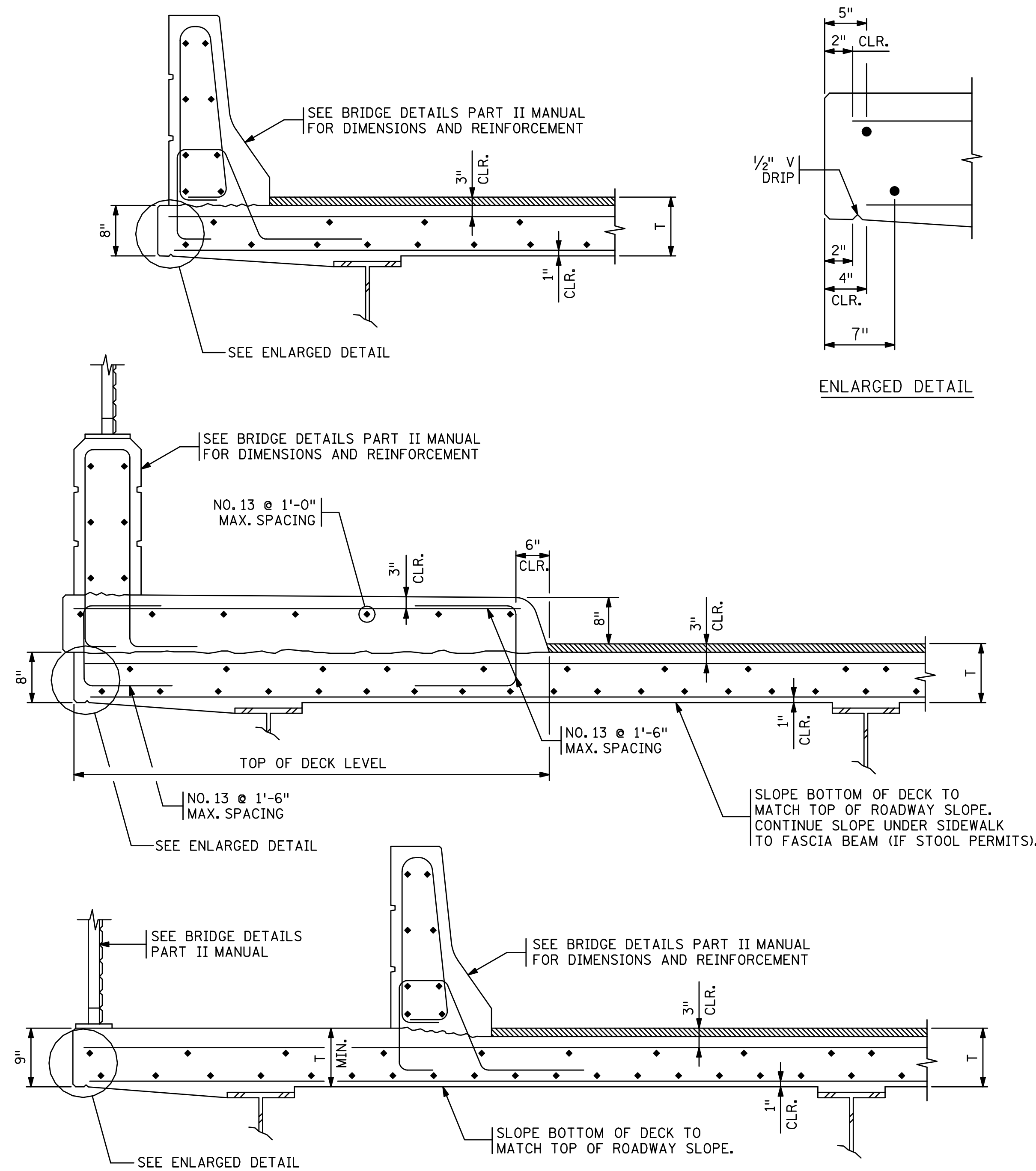
Maximum Beam Spacing①	Transverse Reinforcement			Deck Thickness ②	Longitudinal Reinforcement Bottom③	Longitudinal Reinforcement Top③
	Bottom		Top			
	with Wearing Course	w/o Wearing Course				
5'-0"	13 @ 5"	13 @ 6.5"	13 @ 6.5"	9''	13 @ 7"	13 @ 1'-6"
5'-6"	13 @ 5"	13 @ 6"	13 @ 5.5"	9''	13 @ 7"	13 @ 1'-6"
6'-0"	16 @ 7"	13 @ 6"	13 @ 5"	9''	13 @ 6"	13 @ 1'-6"
6'-6"	16 @ 7"	13 @ 6"	16 @ 7"	9''	13 @ 6"	13 @ 1'-6"
7'-0"	16 @ 7"	13 @ 6"	16 @ 7"	9''	13 @ 6"	13 @ 1'-6"
7'-6"	16 @ 7"	13 @ 6"	16 @ 7"	9''	13 @ 6"	13 @ 1'-6"
8'-0"	16 @ 7"	13 @ 6"	16 @ 6.5"	9''	13 @ 6"	13 @ 1'-6"
8'-6"	16 @ 7"	13 @ 6"	16 @ 6.5"	9''	13 @ 6"	13 @ 1'-6"
9'-0"	16 @ 7"	13 @ 6"	16 @ 6.5"	9''	13 @ 6"	13 @ 1'-6"
9'-6"	16 @ 6.5"	13 @ 5.5"	16 @ 6"	9''	13 @ 6"	13 @ 1'-6"
10'-0"	16 @ 6"	13 @ 5"	16 @ 5.5"	9''	13 @ 6"	13 @ 1'-6"
10'-6"	16 @ 6"	16 @ 7.5"	16 @ 5"	9''	13 @ 6"	13 @ 1'-6"
11'-0"	16 @ 6"	16 @ 7.5"	16 @ 5"	9 1/4''	13 @ 6"	13 @ 1'-6"
11'-6"	16 @ 6"	16 @ 7.5"	16 @ 5"	9 1/2''	13 @ 6"	13 @ 1'-6"
12'-0"	16 @ 6"	16 @ 7.5"	16 @ 5"	9 3/4''	13 @ 6"	13 @ 1'-6"
12'-6"	16 @ 6"	16 @ 7.5"	19 @ 6"	10''	13 @ 6"	13 @ 1'-6"
13'-0"	16 @ 6"	16 @ 7"	19 @ 6"	10 1/4''	13 @ 6"	13 @ 1'-6"
13'-6"	16 @ 5.5"	16 @ 7"	19 @ 6"	10 1/2''	13 @ 6"	13 @ 1'-6"
14'-0"	16 @ 5.5"	16 @ 7"	19 @ 6"	10 3/4''	13 @ 6"	13 @ 1'-6"
14'-6"	16 @ 5.5"	16 @ 7"	19 @ 6"	11''	13 @ 6"	13 @ 1'-6"
15'-0"	16 @ 5.5"	16 @ 7"	19 @ 6"	11 1/4''	13 @ 6"	13 @ 1'-6"

① For skews ≤ 20°, beam spacing is measured along the skew.
For skews > 20°, beam spacing is measured normal to roadway centerline.
② Deck thickness includes wearing course.
③ Requirements for positive moment area shown; See Figure 9.2.1.7 for reinforcing requirements over the pier.

Design Assumptions:

- Live load moments are from LRFD Table A4.1-1.
- The 2" wearing course is sacrificial and is not used in determining a structural depth d for bottom steel.
- The control of cracking by distribution of flexural reinforcement requirements have been met using a clear cover of 1" for bottom steel and limiting clear cover for calculation of d_c to 2" for top steel with a γ_e=0.75.
- The LRFD code (under empirical design) states the ratio of the effective beam spacing to slab thickness should be less than 18 (Ontario uses 15); this slab thickness increase fits these requirements and is similar to what we have used successfully in the past.
- A future wearing course of 20 psf with a load factor of 1.25 has been used.
- Concrete strength of 4 ksi; reinforcing steel strength of 60 ksi.

Table 9.2.1.2



CONCRETE DECK REINFORCEMENT SECTIONS

(WITH CONCRETE WEARING COURSE)

Figure 9.2.1.4

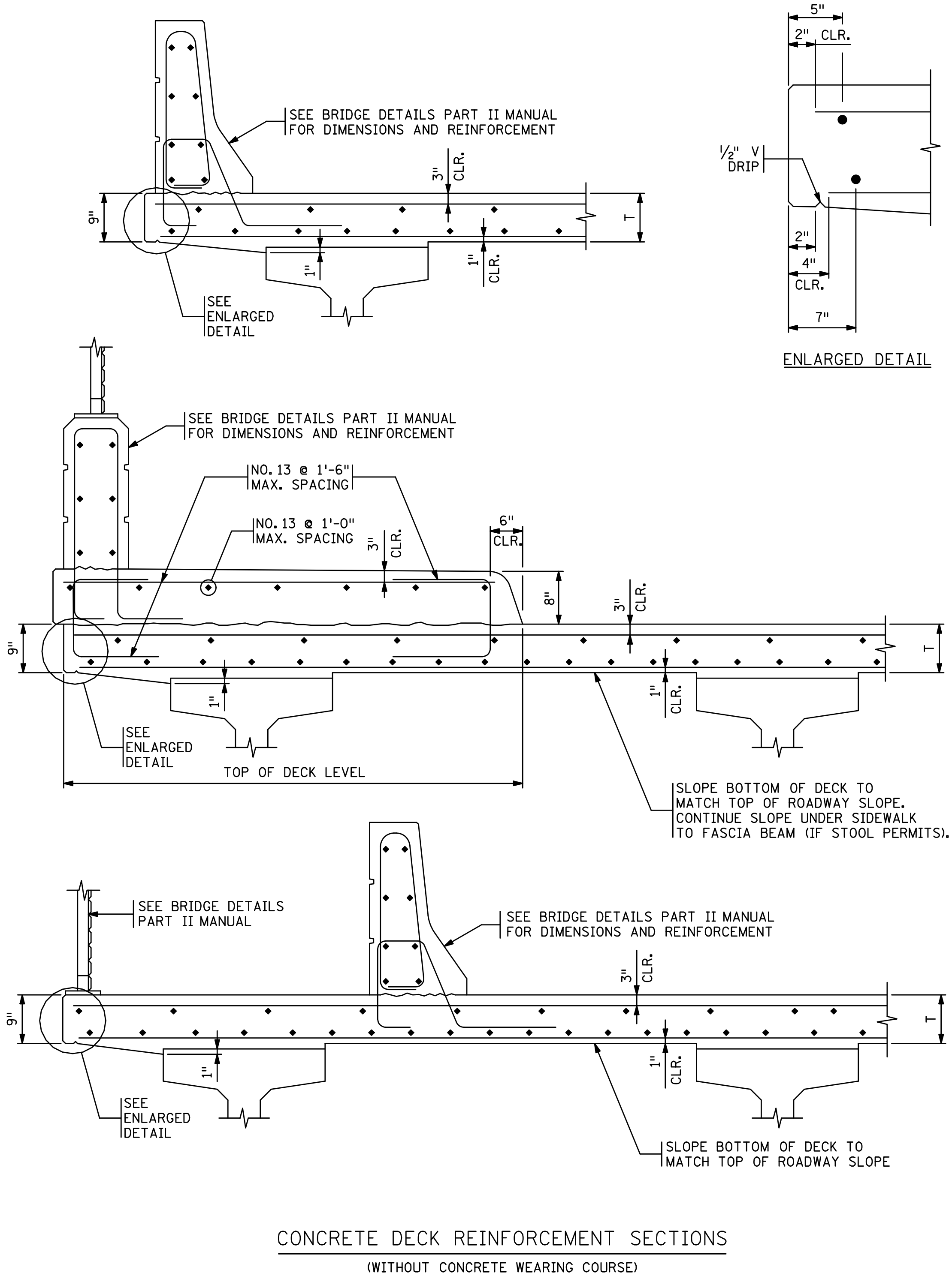
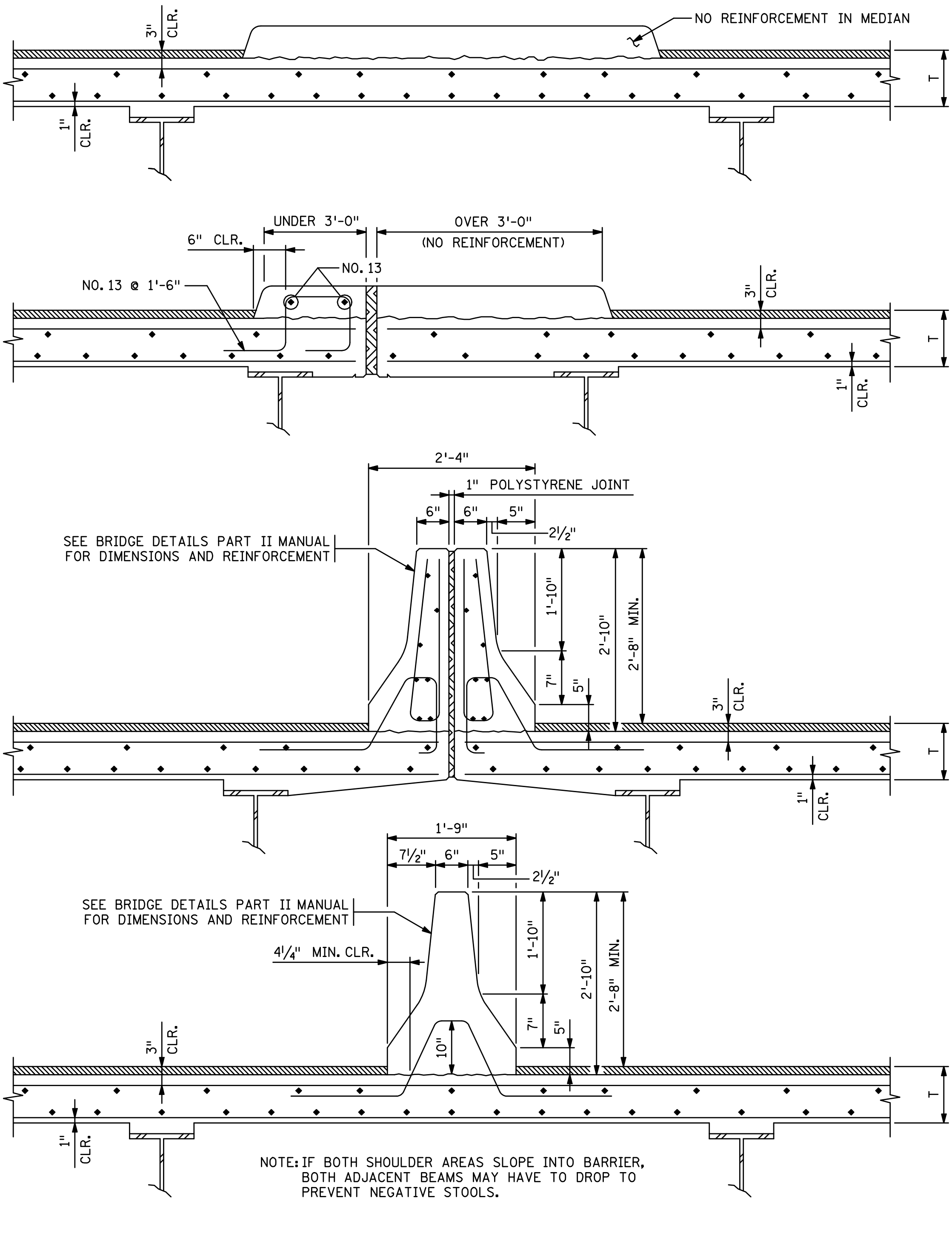

SEE BRIDGE DETAILS PART II MANUAL
FOR DIMENSIONS AND REINFORCEMENT
3" CLR.
9"
T
1"
1" CLR.
SEE
ENLARGED
DETAIL
5"
2" CLR.
½" V
DRIP
2"
4"
CLR.
7"
ENLARGED DETAIL
NO. 13 @ 1'-6"
MAX. SPACING
NO. 13 @ 1'-0"
MAX. SPACING
6"
CLR.
8"
TOP OF DECK LEVEL
SLOPE BOTTOM OF DECK TO
MATCH TOP OF ROADWAY SLOPE.
CONTINUE SLOPE UNDER SIDEWALK
TO FASCIA BEAM (IF STOOL PERMITS).
SEE BRIDGE DETAILS
PART II MANUAL
SLOPE BOTTOM OF DECK TO
MATCH TOP OF ROADWAY SLOPE
CONCRETE DECK REINFORCEMENT SECTIONS
(WITHOUT CONCRETE WEARING COURSE)

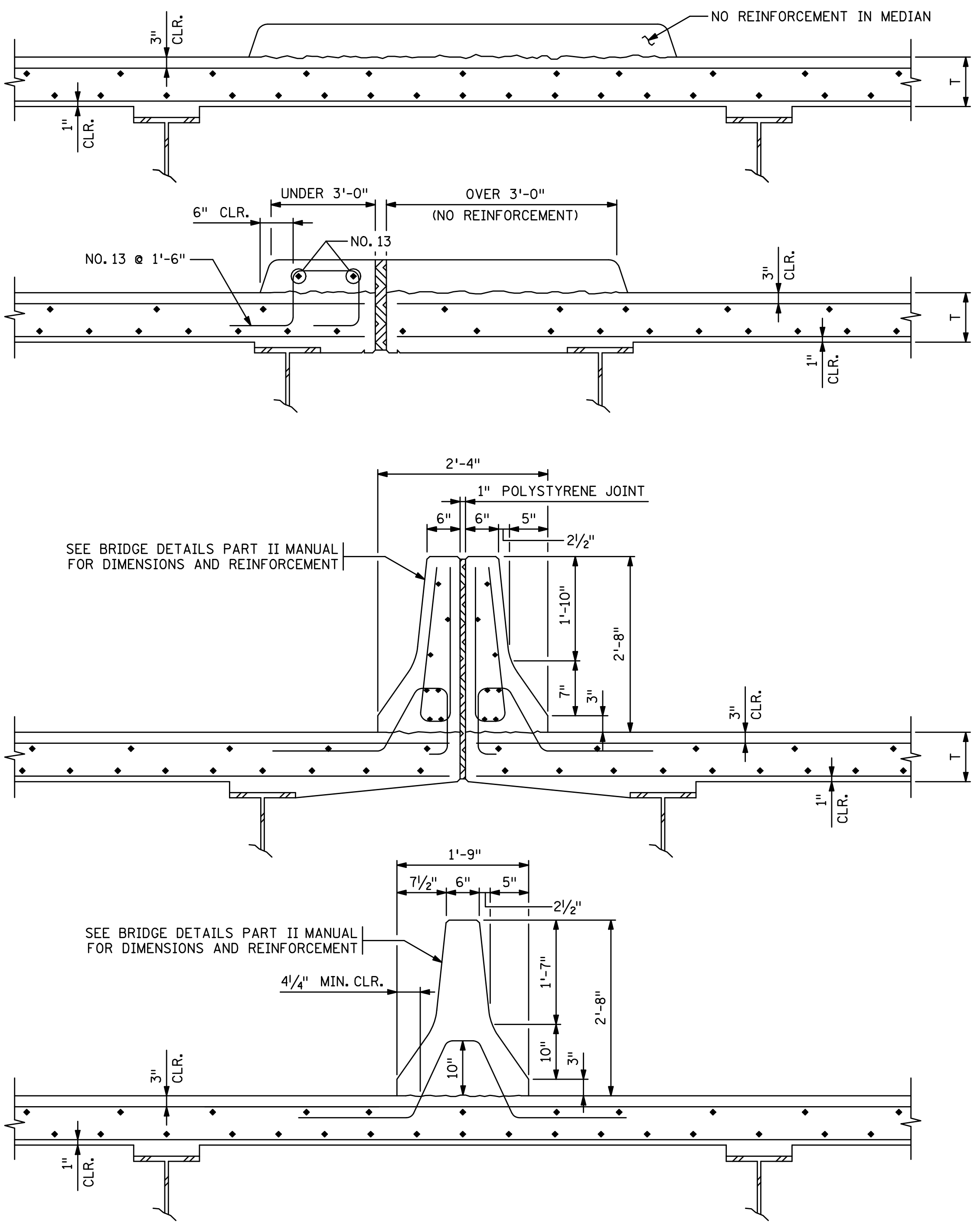
Figure 9.2.1.5



CONCRETE DECK REINFORCEMENT SECTIONS

(WITH CONCRETE WEARING COURSE)

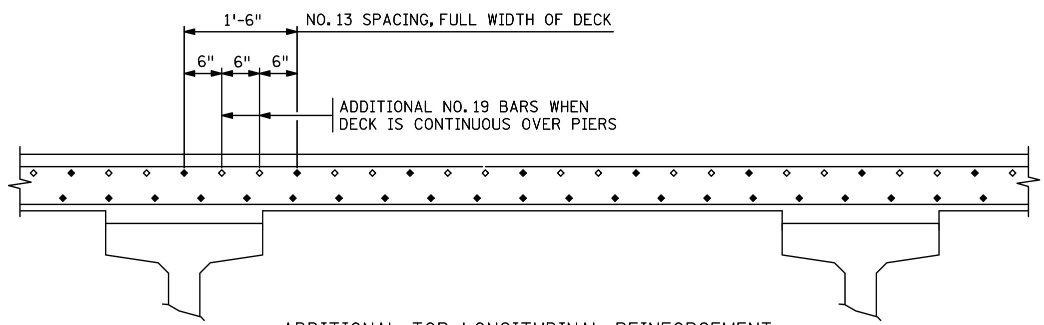
Figure 9.2.1.6



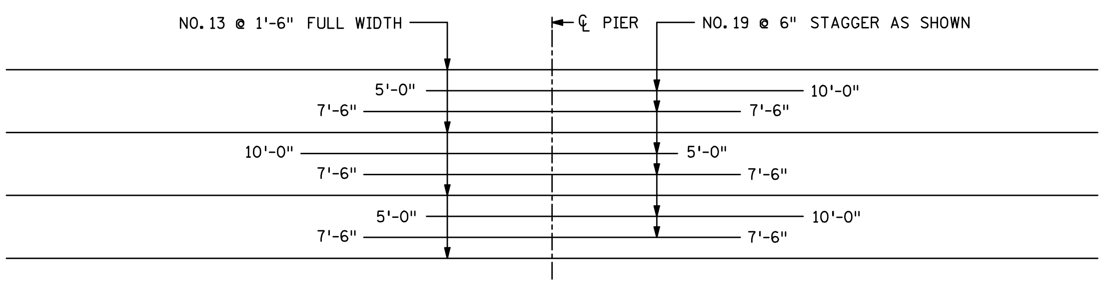
CONCRETE DECK REINFORCEMENT SECTIONS

(WITHOUT CONCRETE WEARING COURSE)

Figure 9.2.1.7



ADDITIONAL TOP LONGITUDINAL REINFORCEMENT
FOR DECK CONTINUOUS OVER PIERS

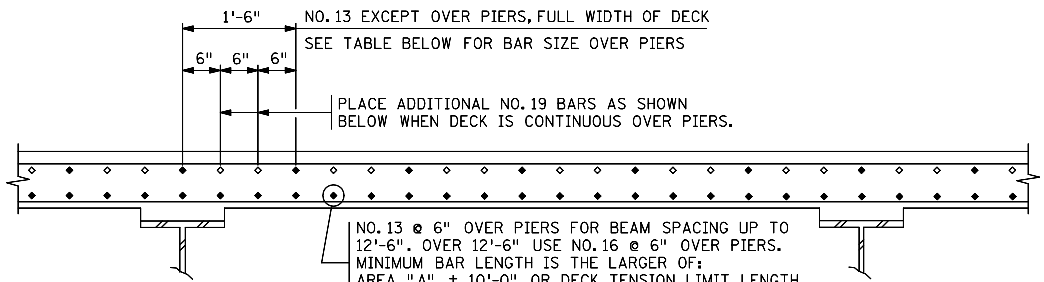


PLAN OF TOP LONGITUDINAL REINFORCEMENT AT PIER

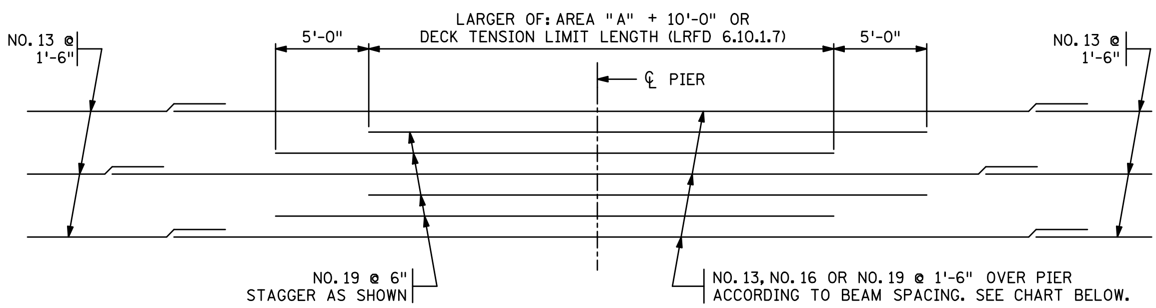
LONGITUDINAL REINFORCEMENT FOR CONCRETE DECK WITH MAIN REINFORCEMENT PERPENDICULAR TO TRAFFIC WITH ONLY DECK (NOT BEAMS) CONTINUOUS OVER PIER.

CONCRETE DECK REINFORCEMENT DETAILS
FOR PRESTRESSED CONCRETE BEAM SPANS

Figure 9.2.1.8



TRANSVERSE SECTION



PLAN OF TOP LONGITUDINAL REINFORCEMENT AT PIER

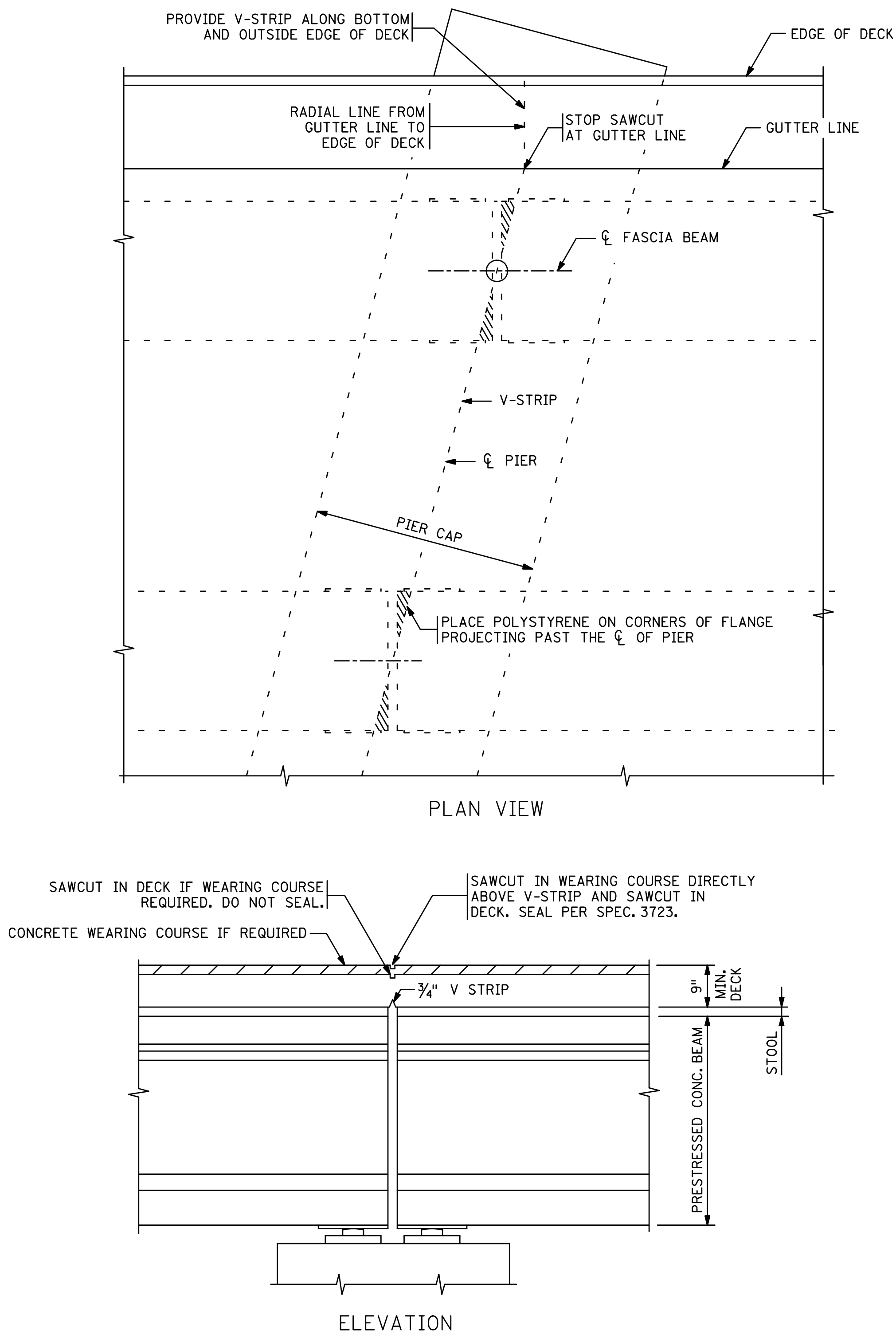
THROUGH REINFORCEMENT OVER PIERS		
BEAM SPACING	BOTTOM LONGITUDINAL	TOP LONGITUDINAL
UP TO 10'-6"	NO. 13 @ 6"	NO. 13 @ 1'-6"
OVER 10'-6" TO 12'-6"	NO. 13 @ 6"	NO. 16 @ 1'-6"
OVER 12'-6" TO 14'-6"	NO. 16 @ 6"	NO. 19 @ 1'-6"
OVER 14'-6"	SPECIAL DESIGN	

PERMISSIBLE SPLICES IN REINFORCEMENT BARS TO BE LOCATED AWAY FROM ℄ OF PIER AND ALTERNATED ON EACH SIDE OF PIER.

LONGITUDINAL REINFORCEMENT FOR CONCRETE DECK WITH MAIN REINFORCEMENT PERPENDICULAR TO TRAFFIC AND CONTINUOUS OVER 3 OR MORE BEAMS.

CONCRETE DECK REINFORCEMENT DETAILS FOR CONTINUOUS STEEL BEAM SPANS (AASHTO LRFD 6.10.1.7)

Figure 9.2.1.9



SAWCUT DETAIL AT PIERS

(CONTINUOUS DECK OVER NON-CONTINUOUS PRESTRESSED BEAMS)

Figure 9.2.1.10

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9.3 Reinforced Concrete Deck Design Example

This example demonstrates the design of a typical reinforced concrete deck. The first part describes the design of the interior region of a reinforced concrete deck supported on beam or stringer elements. The second part provides design procedures for the deck overhang region.

[4.6.2.2.4]

The deck is assumed to carry traffic loads to the supporting members (beams or girders) via one-way slab or beam action. The supporting members for the deck are parallel to the direction of traffic. The substructures are not skewed, so the primary reinforcement for the deck is placed perpendicular to the supporting members. Distribution steel is placed parallel to the beams.

The reinforced concrete deck section with wearing course is illustrated in Figure 9.3.1.

A. Material and Design Parameters

[9.7.1.1]
[9.7.1.3]

Deck

Unit weight of deck and wearing course (for loads), $w_c = 0.150\text{ kcf}$
Unit weight of deck and wearing course (for E_c), $w_c = 0.145\text{ kcf}$
Skew angle of bridge, $\varphi = 0$ degrees
Out-to-out bridge deck transverse width, $b_{deck} = 51.33\text{ ft} = 616\text{ in}$
Weight of future wearing course, $w_{ws} = 0.020\text{ kcf}$
Yield Strength of reinforcing bars, $f_y = 60\text{ ksi}$
Reinforcing bar modulus of elasticity, $E_s = 29{,}000\text{ ksi}$
28 day concrete strength, $f'_c = 4\text{ ksi}$
Center-to-center beam spacing, $L_s = 9\text{ ft}$
Railing weight, $w_{barrier} = 0.477\text{ klf}$ (see Std. Figure 5-397.117)
Beam flange width, $b_f = 30\text{ in}$ (63M Prestressed I-Beam)

B. Structural Analysis of Interior Region

[9.6.1]

The deck is modeled as a continuous beam on pinned supports provided at the centerline of the supporting beams. The beams are assumed to be rigid, not permitting vertical movement. Recognizing that beams have top flanges that provide support for the deck over a finite dimension, the specifications permit designing negative moment reinforcement for locations that are offset from the centerline of the beam.

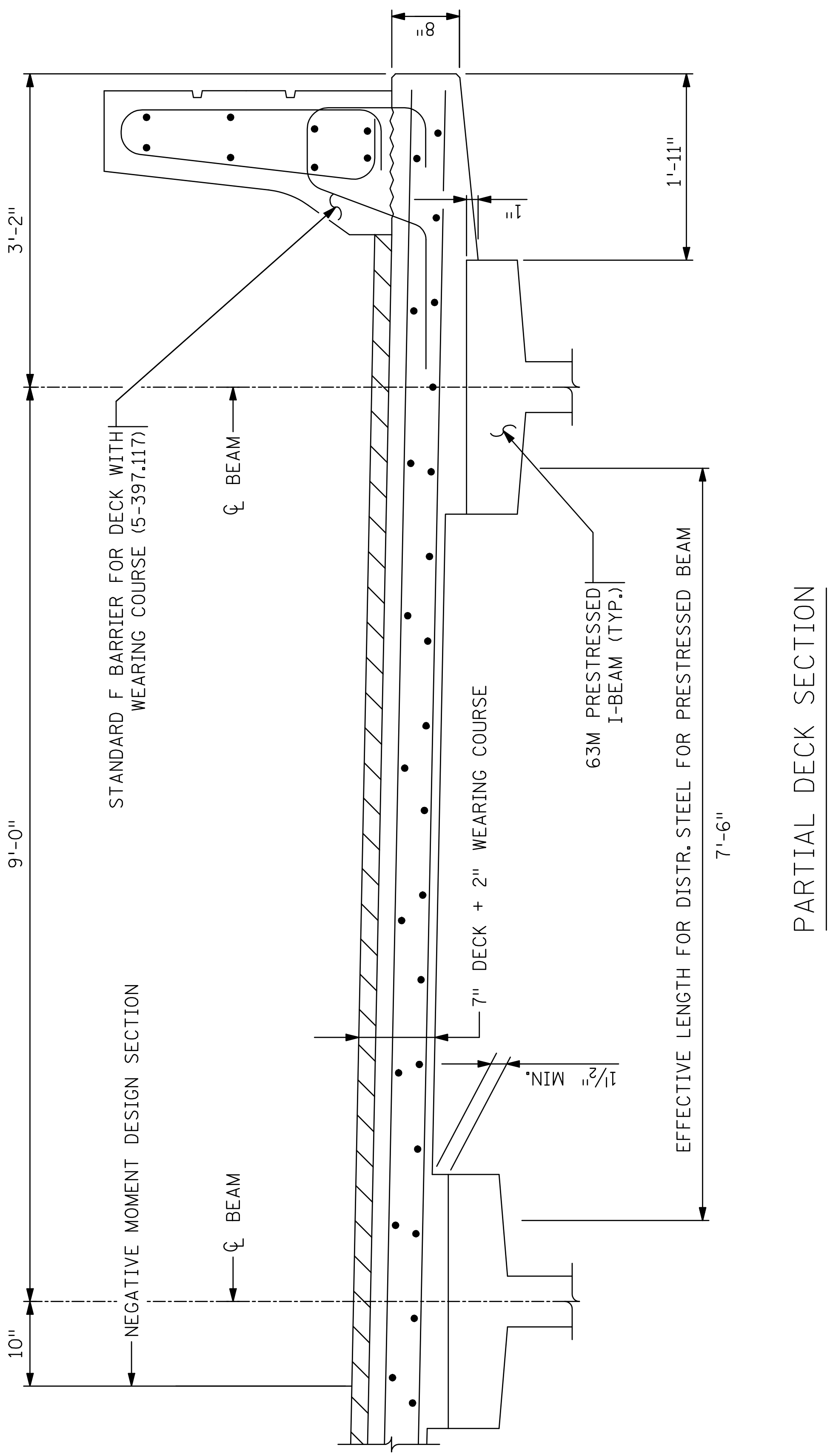

8"
3'-2"
1'-11"
1"
STANDARD F BARRIER FOR DECK WITH
WEARING COURSE (5-397.117)
℄ BEAM
9'-0"
63M PRESTRESSED
I-BEAM (TYP.)
7" DECK + 2" WEARING COURSE
EFFECTIVE LENGTH FOR DISTR. STEEL FOR PRESTRESSED BEAM
7'-6"
PARTIAL DECK SECTION
1½" MIN.
NEGATIVE MOMENT DESIGN SECTION
℄ BEAM
10"

Figure 9.3.1

[4.6.2.1.6]

For prestressed beams, negative moments should be checked at the design section located $^{1}/_{3}$ of the flange width away from the beam centerline. The offset can be no more than 15 inches. For the top flange width of 30 inches, check negative moments at a location 10 inches away from beam centerline. The design uses a unit strip one foot wide.

C. Live Loads
[Appendix A4]

The AASHTO LRFD Specifications contain tables listing the design live load moments (positive and negative) for decks supported on different beam spacings. The tabularized moments are for a one foot wide strip.

The limitations for use of the tables include a check on the overhang dimension. A minimum of 1.75 feet from the centerline of the fascia beam is permitted. The maximum overhang permitted is the lesser of [6.0 feet or (0.625 x beam spacing)].

$$0.625 \cdot 9.0 \text{ ft} = 5.63 \text{ ft} \leq 6.0 \text{ ft}$$

For this example the overhang is:

$$\frac{1}{2} \cdot (51.33 - 45.0) = 3.17 \text{ ft} \qquad 1.75 \text{ ft} < 3.17 \text{ ft} < 5.63 \text{ ft} \qquad \underline{\text{OK}}$$

The overhang dimension checks are satisfied, as are all other parameters specified for use of the design live load moment tables.

Interpolate Design Live Load Moments

LRFD Table A4.1-1 lists the following design live load moments for a beam spacing of 9.0 ft:

Positive moment	= 6.29 kip-ft
Negative moment (9 in)	= 4.28 kip-ft
Negative moment (12 in)	= 3.71 kip-ft

Interpolate to obtain a value at the design section (10 inches away from the center of the supporting beam):

$$4.28 - 1 \cdot \left(\frac{4.28 - 3.71}{3}\right) = 4.09 \text{ kip-ft/ft}$$

The values in LRFD Table A4.1-1 include the multiple presence and dynamic load allowance factors.

D. Dead Loads

The dead load moments are based on the self-weight of the 7 inch deck, a 2 inch wearing course, and a 0.020 ksf future wearing surface.

Depth of concrete deck, $d_{deck} = 7 \text{ in} + 2 \text{ in} = 9 \text{ in}$

Dead loads will be computed for a strip of deck 1 foot wide. Mn/DOT practice is to simplify the dead load bending moment calculations, by computing both the positive and negative dead load bending moments using:

$$M_{DC} = \frac{W_{DC} \cdot L_s^{2}}{10}$$

Deck and Wearing Course Load:

$$W_{deck} = w_c \cdot d_{deck} = (0.150) \cdot (9) \cdot \frac{1}{12} = 0.11 \text{ klf}$$

Future Wearing Surface Load:

$$W_{WS} = 0.02 \text{ klf}$$

Combined Dead Load:

$$W_{DC} = W_{deck} + W_{WS} = 0.11 + 0.02 = 0.13 \text{ klf}$$

Dead Load Bending Moment:

$$M_{DC} = \frac{0.13 \cdot (9)^2}{10} = 1.05 \text{ kip-ft}$$

E. Flexural Design Moments [1.3.3 – 1.3.5]

The load modifiers for the deck design are:

$$\eta_D = 1.00$$
$$\eta_R = 1.00$$
$$\eta_I = 1.00$$

Then $\eta_{cum} = \eta_D \cdot \eta_R \cdot \eta_I = 1.00$

[Table 3.4.1-1]

Use the load factors provided in LRFD Article 3.4.1 to generate the Strength I and Service I design moments.

Strength I Limit State Loads

$$U_1 = 1.00 \cdot [1.25 \cdot (DC) + 1.75 \cdot (LL)]$$

Negative Design Moment:

$$M_{u(neg)} = 1.00 \cdot [1.25 \cdot (M_{DC}) + 1.75 \cdot (M_{LL(neg)})]$$
$$= 1.00 \cdot [1.25 \cdot (1.05) + 1.75 \cdot (4.09)] = 8.47 \text{ kip-ft}$$

Positive Design Moment:

$$M_{u(pos)} = 1.00 \cdot \left[1.25 \cdot \left(M_{DC}\right) + 1.75 \cdot \left(M_{LL(pos)}\right)\right]$$

$$= 1.00 \cdot \left[1.25 \cdot (1.05) + 1.75 \cdot (6.29)\right] = 12.32 \text{ kip-ft}$$

Service I Limit State Loads

$$S_1 = 1.00 \cdot \left[1.0 \cdot (DC) + 1.0 \cdot (LL)\right]$$

Negative Design Moment:

$$M_{S(neg)} = 1.00 \cdot \left[1.0 \cdot \left(M_{DC}\right) + 1.0 \cdot \left(M_{LL(neg)}\right)\right]$$

$$= 1.00 \cdot \left[1.0 \cdot (1.05) + 1.0 \cdot (4.09)\right] = 5.14 \text{ kip-ft}$$

Positive Design Moment:

$$M_{S(pos)} = 1.00 \cdot \left[1.0 \cdot \left(M_{DC}\right) + 1.0 \cdot \left(M_{LL(pos)}\right)\right]$$

$$= 1.00 \cdot \left[1.0 \cdot (1.05) + 1.0 \cdot (6.29)\right] = 7.34 \text{ kip-ft}$$

F. Top Steel (Negative Moment)

[5.7.3]

Flexure Strength Check

The top reinforcement has a clear cover of 3 inches (which includes the 2 inch wearing course). Design the negative moment reinforcement assuming a singly reinforced cross section and that #13 bars are used ($d_b = 0.50$ inches).

[5.5.4.2]

Assume the section is controlled in tension and the flexural resistance factor, $\phi = 0.90$

Determine distance from extreme compression fiber to tension reinforcement.

$$d_s = d_{deck} - \text{cover} - \frac{1}{2} \cdot d_b = 9 - 3 - \frac{1}{2} \cdot 0.5 = 5.75 \text{ in}$$

Try #13 bars with a 5 inch center-to-center spacing.

Width of compression face of member, $b = 12$ in

Area of top steel provided per foot,

$$A_{s(top)} = A_b \cdot \left(\frac{12}{spacing}\right) = 0.20 \cdot \left(\frac{12}{5}\right) = 0.48\ in^2$$

$$a = c\beta_1 = \frac{A_{s(top)} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.48 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.71\ in$$

$$M_n = A_{s(top)} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 0.48 \cdot 60 \cdot \left(5.75 - \frac{0.71}{2}\right) \cdot \left(\frac{1}{12}\right)$$

$$= 12.95\ \text{kip-ft}$$

$$\varphi \cdot M_n = 0.9 \cdot 12.95 = 11.66\ \text{kip-ft} > 8.47\ \text{kip-ft} \qquad \underline{\text{OK}}$$

[5.5.4.2]

Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the section in compression:

$$d_t = d_s = 5.75\ in$$

$$c = \frac{a}{\beta_1} = \frac{0.71}{0.85} = 0.84\ in$$

$$\phi = 0.65 + 0.15 \cdot \left(\frac{d_t}{c} - 1\right) = 0.65 + 0.15 \cdot \left(\frac{5.75}{0.84} - 1\right) = 1.53 > 0.9$$

Therefore, $\phi = 0.9$

[5.7.3.4]

Crack Control Check

LRFD crack control check places a limit on the spacing of reinforcement to prevent severe and excessive flexural cracking. This is accomplished by limiting the spacing of reinforcing bars as follows:

[5.7.3.4-1]

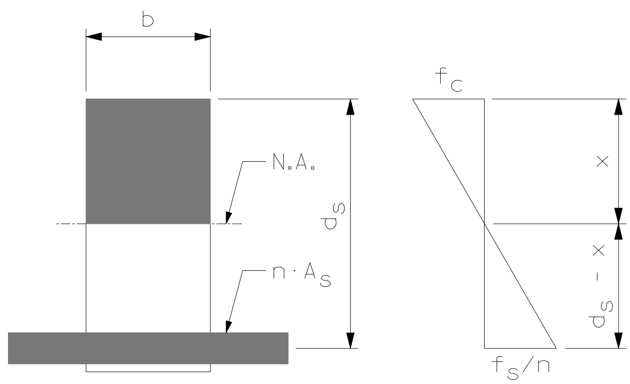
$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Per article 5.3.2 of this manual, use a maximum clear cover of 2.0 inches to compute d_c. Assuming #13 bars are used, $d_c = 2.0 + 0.5 \cdot d_b = 2.25\ in$. Also, the deck thickness will be limited to 8 inches

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

[5.4.2.4 & 5.7.1]

Locate the neutral axis:



$$n = \frac{E_s}{E_c} = \frac{29{,}000}{33{,}000 \cdot (0.145)^{1.5} \cdot \sqrt{4.0}} = 7.96 \quad \therefore \quad \text{Use } n = 8$$

$$n \cdot A_s = 8 \cdot 0.48 = 3.84 \, \text{in}^2$$

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

$$\frac{12 \cdot x^2}{2} = 3.84 \cdot (5.75 - x) \qquad \text{solving, } x = 1.62 \, \text{in}$$

Determine the lever arm between service load flexural force components:

$$j \cdot d_s = d_s - \frac{x}{3} = 5.75 - \frac{1.62}{3} = 5.21 \, \text{in}$$

The stress in the reinforcement when subjected to the Service I moment is:

$$f_{ss} = \frac{M_{s(neg)}}{A_s \cdot j \cdot d_s} = \frac{5.14 \cdot 12}{0.48 \cdot 5.21} = 24.7 \, \text{ksi}$$

Find β_s:

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.25}{0.7 \cdot (8 - 2.25)} = 1.56$$

For severe exposure, use γ_e=0.75

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.56 \cdot 24.7} - 2 \cdot 2.25 = 9.13 \geq 5 \text{ in} \quad \underline{\text{OK}}$$

[5.7.3.3.2]

Minimum Reinforcement

Reinforcement should be provided to carry the smaller of 1.2 times the cracking moment (M_{cr}) or 1.33 times the Strength I bending moment (M_u).

Conservatively assume a full 9 inch deep section for the minimum reinforcement check.

$$S_{deck} = \frac{b \cdot (d_{deck})^2}{6} = \frac{12 \cdot (9)^2}{6} = 162 \text{ in}^3$$

[5.4.2.6]

The rupture stress (f_r) of concrete is assumed to be:

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

Set the cracking moment (M_{cr}) equal to $f_r \cdot S$:

$$1.2 \cdot M_{cr} = 1.2 \cdot (f_r \cdot S) = 1.2 \cdot (0.74 \cdot 162) \cdot \left(\frac{1}{12}\right) = 11.99 \text{ kip-ft}$$

$$1.33 \cdot M_{u(neg)} = 1.33 \cdot 8.47 = 11.27 \text{ kip-ft}$$ GOVERNS

Use the $1.33 \cdot M_{u(neg)}$ value to check minimum reinforcement.

$$\varphi \cdot M_n = 11.66 \text{ kip-ft} > 11.27 \text{ kip-ft}$$ OK

G. Bottom Steel (Positive Moment) [5.7.3]

Flexure Strength Check

The bottom reinforcement has a clear cover of one inch. Because the wearing course may be removed in future milling operations, do not include it in structural capacity computations. Size the positive moment reinforcement assuming a singly reinforced cross section. Assume that #16 bars are used.

Determine distance from extreme compression fiber to tension reinforcement.

$$d_s = d_{deck} - \text{cover} - \text{wear course} - \frac{1}{2} \cdot d_b = 9 - 1 - 2 - \frac{1}{2} \cdot 0.63 = 5.69 \text{ in}$$

Try #16 bars with a 7 inch center-to-center spacing.
Area of steel per foot $= A_{s(bot)}$

$$A_{s(bot)} = A_b \cdot \left(\frac{12}{\text{spacing}}\right) = 0.31 \cdot \left(\frac{12}{7}\right) = 0.53 \text{ in}^2$$

$$a = c\beta_1 = \frac{A_{s(bot)} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.53 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.78 \text{ in}$$

$$M_n = A_{s(bot)} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 0.53 \cdot 60 \cdot \left(5.69 - \frac{0.78}{2}\right) \cdot \left(\frac{1}{12}\right)$$
$$= 14.05 \text{ kip-ft}$$

$$\varphi \cdot M_n = 0.9 \cdot 14.05 = 12.65 \text{ kip-ft} > 12.32 \text{ kip-ft} \qquad \underline{\text{OK}}$$

[5.5.4.2] Validate the assumption of 0.9 for resistance factor:

$$d_t = d_s = 5.69 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{0.78}{0.85} = 0.92 \text{ in}$$

$$\phi = 0.65 + 0.15 \cdot \left(\frac{d_t}{c} - 1\right) = 0.65 + 0.15 \cdot \left(\frac{5.69}{0.92}\right) = 1.58 > 0.9$$

Therefore, $\phi = 0.9$

[5.7.3.4] **Crack Control Check**

Depth of concrete measured from extreme tension fiber to center of bar located closest is d_c:

$$d_c = \text{cover} + \frac{d_b}{2} = 1.0 + \frac{0.63}{2} = 1.31 \text{ in}$$

[5.4.2.4 & 5.7.1] Find the modular ratio:

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{33{,}000 \cdot (0.145)^{1.5} \cdot \sqrt{4.0}} = 7.96 \quad \therefore \quad \text{Use } n = 8$$

[5.7.3.4-1] The spacing of reinforcing bars is limited to:

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

Compute the stress in the reinforcement using a cracked section analysis. Begin by locating the neutral axis.

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

$$\frac{12 \cdot x^2}{2} = 8 \cdot 0.53 \cdot (5.69 - x) \qquad \text{solving, } x = 1.68 \text{ in}$$

Determine the lever arm between service load flexural force components.

$$j \cdot d_s = d_s - \frac{x}{3} = 5.69 - \frac{1.68}{3} = 5.13 \text{ in}$$

The stress in the reinforcement when subjected to the Service I design moment is:

$$f_s = \frac{M_{s(pos)}}{A_s \cdot j \cdot d_s} = \frac{7.34 \cdot 12}{0.53 \cdot 5.13} = 32.4 \text{ ksi}$$

Find β_s:

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{1.31}{0.7 \cdot (7 - 1.31)} = 1.33$$

Use γ_e=0.75

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.33 \cdot 32.4} - 2 \cdot 1.31 = 9.56 \geq 7 \text{ in} \qquad \underline{\text{OK}}$$

[5.7.3.3.2]

Minimum Reinforcement Check

Reinforcement should be provided to carry the smaller of 1.2 times the cracking moment (M_{cr}) or 1.33 times the Strength I bending moment (M_u).

$$1.2 \cdot M_{cr} = 1.2 \cdot (f_r \cdot S) = 1.2 \cdot (0.74 \cdot 162) \cdot \left(\frac{1}{12}\right) = 11.99 \text{ kip-ft} \qquad \underline{\text{GOVERNS}}$$

$$1.33 \cdot M_{u(pos)} = 1.33 \cdot 12.32 = 16.39 \text{ kip-ft}$$

Use the $1.2 \cdot M_{cr}$ value to check minimum reinforcement.

$$\varphi \cdot M_n = 12.65 \text{ kip-ft} > 11.99 \text{ kip-ft} \qquad \underline{\text{OK}}$$

H. Bottom Longitudinal Reinforcement

[9.7.3.2]

As part of the Traditional Design Method an "equivalent width method" for reinforced bridge deck designs is utilized. The constraints for reinforced concrete decks, designed in accordance with "traditional" methods, are given in LRFD 9.7.3. To ensure proper load distribution, reinforcement placed perpendicular to the primary reinforcement must be placed in the bottom mat. This reinforcement is a fraction of the primary steel required for the bottom of the section (positive moment). For decks where the primary reinforcement is placed perpendicular to traffic, the longitudinal reinforcement requirement in the bottom mat is:

$$\left(\frac{220}{\sqrt{S_e}}\right) \leq 67\% \qquad \text{where } S_e = \text{effective span length in feet}$$

[9.7.2.3]

The effective span length is a function of the beam or stringer spacing and the type of beam or stringer. For prestressed concrete I-beam sections, the effective span length is the distance between flange tips plus the distance the flange overhangs the web on one side.

$$S_e = \text{beam spacing} - \text{top flange width} + \text{flange overhang}$$

$$= 9.0 \text{ ft} - 2.5 \text{ ft} + 1.0 \text{ ft} = 7.5 \text{ ft}$$

$$\left(\frac{220}{\sqrt{7.5}}\right) = 80.3\% \geq 67\% \qquad \underline{\text{Use 67\%}}$$

Use 67% of the primary steel in the bottom mat. The required area of steel is:

$$A_{s(req)} = 0.67 \cdot A_{s(bot)} = 0.67 \cdot 0.53 = 0.36 \text{ in}^2/\text{ft}$$

Try #16 bars on 10 inch centers. Area of steel provided equals:

$$A_{s(prov)} = A_b \cdot \left(\frac{12}{\text{spacing}}\right) = 0.31 \cdot \left(\frac{12}{10}\right) = 0.37 \text{ in}^2/\text{ft}$$

$$= 0.37 \text{ in}^2/\text{ft} \geq 0.36 \text{ in}^2/\text{ft} \qquad \underline{\text{OK}}$$

I. Top Longitudinal Reinforcement [5.10.8]

The top longitudinal bars must meet the shrinkage and temperature reinforcement requirements.

$$\text{Temperature } A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.30 \cdot 12 \cdot 9}{2 \cdot (12 + 9) \cdot 60} = 0.06 \text{ in}^2/\text{ft}$$

However, the area of steel has the limits, $0.11 \leq A_s \leq 0.60$, and a minimum spacing of 18 inches is required

Therefore, for each face, Min. temp. $A_s = 0.11 \text{ in}^2/\text{ft}$

Use #13 bars spaced at 18 inches ($A_s = 0.13 \text{ in}^2/\text{ft}$) for top longitudinal reinforcement.

Mn/DOT uses additional reinforcement over the piers for continuous decks over piers where the beams are not continuous. The additional reinforcing consists of two #19 bars placed on 6 inch centers between the top mat #13 bars. Refer to Figure 9.2.1.8 for typical reinforcement detailing.

Figure 9.3.2 illustrates the final reinforcement layout for the interior region of the deck.

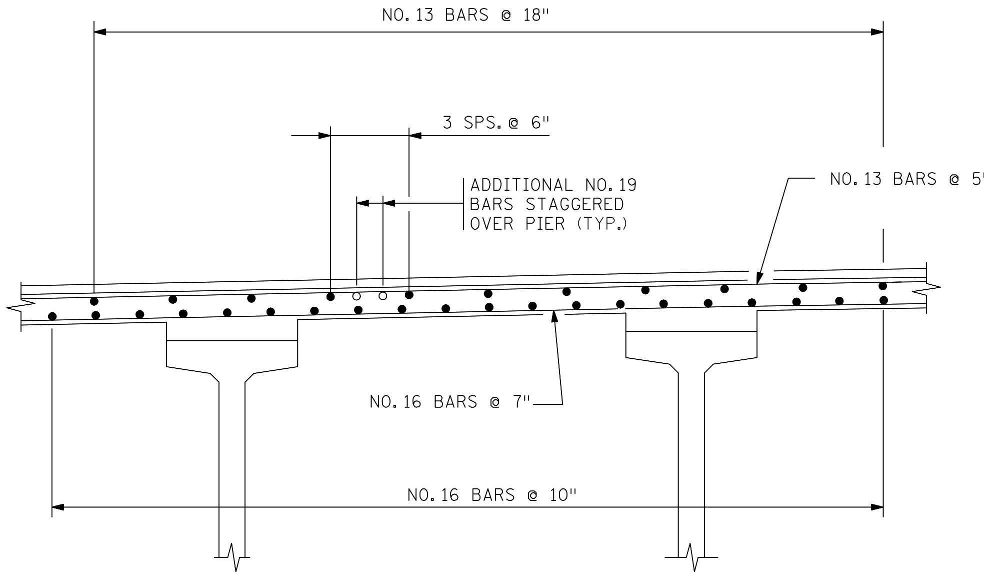


Figure 9.3.2

J. Structural Analysis of Overhang Region [A13.2-1]

Figure 9.3.3 illustrates the overhang region. Four cases must be considered for the overhang design:

- Case 1: Extreme Event II evaluated at the gutter line for the dead load plus horizontal collision force.
- Case 2: Extreme Event II evaluated at the edge of the beam flange for the dead load plus horizontal collision force plus live load.
- Case 3: Strength I evaluated at the edge of the beam flange for the dead load plus live load.
- Case 4: Extreme Event II evaluated at the edge of the beam flange for the dead load plus vertical collision force plus live load.

For this example, the distance from the edge of flange to the gutter line is small, so by inspection Case 2 and Case 3 will not govern. Case 4 will never govern when the Mn/DOT overhang limitations are followed and a Test Level 4 F-rail is used. Therefore, only Case 1 calculations are included in this example.

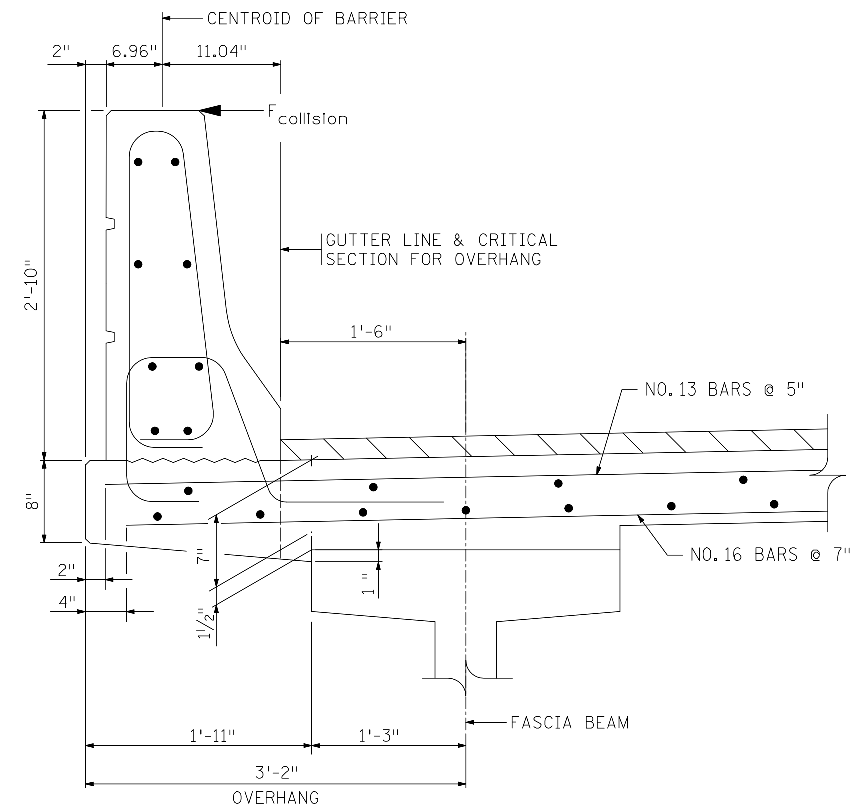


Figure 9.3.3

1. Geometry and Loads

Overhang = 3.17 ft (calculated earlier)

Edge of deck to negative moment section location = 20 in

Deck thickness at gutter line (ignoring wearing course):

$$8 + (9.5 - 8) \cdot \left(\frac{20}{23}\right) = 9.30 \text{ in}$$

Average deck thickness:

$$\left(\frac{8 + 9.30}{2}\right) = 8.65 \text{ in}$$

Deck Bending Moment (at Gutter Line)

$$M_{deck} \approx \left(\frac{8.65}{12}\right) \cdot 0.150 \cdot 1.67 \cdot \left(\frac{1.67}{2}\right) = 0.15 \text{ kip-ft}$$

Barrier Bending Moment

Barrier weight is 0.477 kips per foot. The centroid of the barrier is 11.04 inches outside of the gutter line.

$$M_{barrier} = w_{barrier} \cdot x_{cb} = 0.477 \cdot \left(\frac{11.04}{12}\right) = 0.44 \text{ kip-ft}$$

[A13.2]

Collision Force Tension and Bending Moment

The F-rail with wearing course has a maximum capacity $R_w = 122.9$ kips (see Table 13.2.4.1 in this manual). The factored design force $F_t = 54$ kips for a Test Level 4 railing.

Mn/DOT requires the deck to carry the lesser of the rail capacity R_w or $^4/_3$ x F_t:

$$F_{collision} = R_w = 122.9 \text{ kips}$$

or

$$F_{collision} = \frac{4}{3} \cdot F_t = \frac{4}{3} \cdot (54) = 72 \text{ kips} \qquad \underline{\text{GOVERNS}}$$

The collision force is applied at a height of 34 inches above the top of the structural deck. It generates a tension force and a bending moment in the overhang portion of the deck. The moment arm to the center of the deck cross-section at the gutter line is:

$$\text{Moment arm} = 34 + \frac{9.30}{2} = 38.65 \text{ in} = 3.22 \text{ ft}$$

(wearing course is ignored)

The collision force is applied to a length of barrier 3.5 feet long. Computations for the standard F-rail (see Table 13.2.4.1 in this manual) indicate that 10.2 feet of barrier length (L_c) is engaged in resisting the collision force in the "interior" regions. Assume that a deck width of 10.2 feet plus two barrier heights (using a 45 degree distribution) resists the tension force and overturning moment.

$$F_{c(linear)} = F_{collision} / \text{effective deck width}$$

$$= \frac{F_{collision}}{L_c + L_{45deg}} = \frac{72}{10.2 + 2.83 \cdot 2} = 4.54 \text{ kips/ft}$$

$$M_c = F_{c(linear)} \cdot (\text{moment arm}) = 4.54 \cdot 3.22 = 14.62 \text{ kip-ft/ft}$$

Extreme Event II Limit State Bending Moment

Dead Load Moment:

$$M = (M_{deck} + M_{barrier}) = (0.15 + 0.44) = 0.59 \text{ kip-ft/ft}$$

Total Factored Moment:

[A13.4.1]

$$M_u = 1.00 \cdot M_c + 1.00 \cdot M_{DL} = 1.00 \cdot 14.62 + 1.00 \cdot 0.59 = 15.21 \text{ kip-ft/ft}$$

Total Factored Axial Force:

$$P_u = F_{c(linear)} = 4.54 \text{ kips/ft}$$

The eccentricity of P_u is:

$$e_u = \frac{M_u}{P_u} = \frac{15.21}{4.54} = 3.35 \text{ ft} = 40.20 \text{ in}$$

2. Overhang Resistance

The overhang must resist both axial tension and bending moment. The capacity of the overhang will be determined by considering the tension side of the structural interaction diagram for a one foot wide portion of the overhang. See Figure 9.3.4.

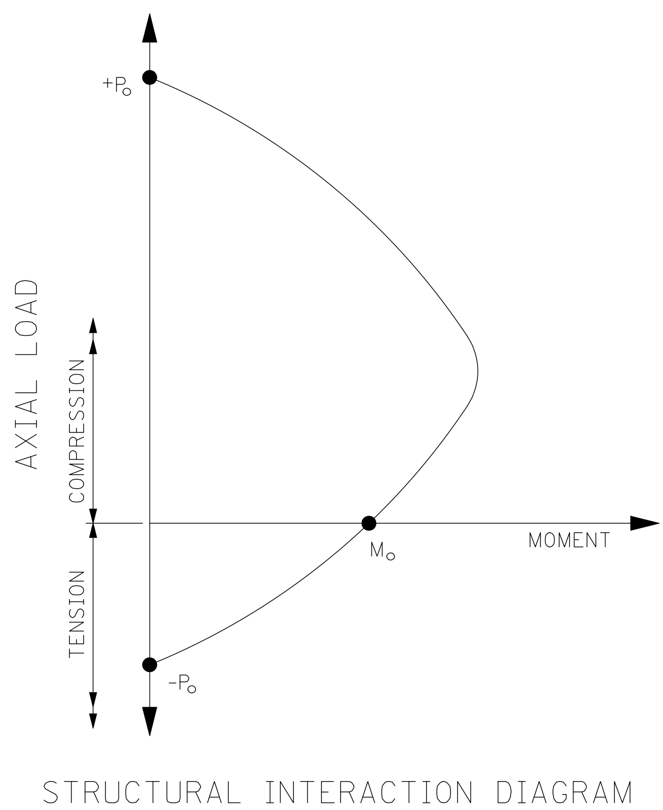


Figure 9.3.4

Check if the reinforcement chosen for the interior region will be adequate for the overhang region. The interior region reinforcement is:

Top reinforcement – #13 bars @ 5" ($A_s = 0.48 \text{ in}^2/\text{ft}$)

Bottom reinforcement – #16 bars @ 7" ($A_s = 0.53 \text{ in}^2/\text{ft}$)

Referring to Figure 9.3.5, determine the capacity of the overhang section for the eccentricity e_u equal to 40.20 inches.

Start by assuming that for both the top and bottom reinforcement, $\varepsilon_s > \varepsilon_y$.

Next, check development of the top and bottom bars from the edge of deck. From Figure 5.2.2.2 of this manual:

For #13 bars, $\ell_d = 12$ in

For #16 bars, $\ell_d = 15$ in

For #13 top bars, available $\ell_{davail(13)} = 18$ in 100% developed

For #16 bottom bars, available $\ell_{davail(16)} = 16$ in 100% developed

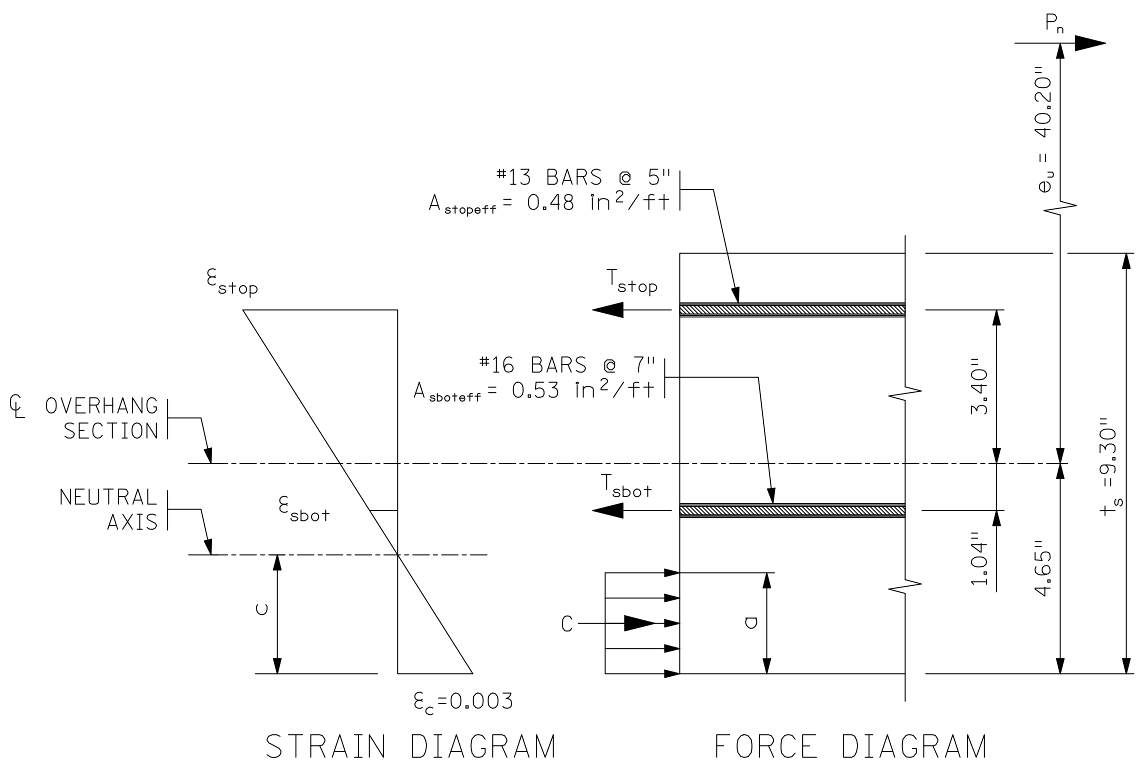


Figure 9.3.5

Then:

$$A_{stopeff} = 0.48 \ \text{in}^2/\text{ft}$$

$$A_{sboteff} = 0.53 \ \text{in}^2/\text{ft}$$

$$T_{stop} = A_{stopeff} \cdot f_y = 0.48 \cdot 60 = 28.80 \ \text{kips}$$

$$T_{sbot} = A_{sboteff} \cdot f_y = 0.53 \cdot 60 = 31.80 \ \text{kips}$$

$$T_{stot} = 28.80 + 31.80 = 60.6 \ \text{kips}$$

The total compression force C is:

$$C = 0.85 \cdot f'_c \cdot b \cdot a = 0.85 \cdot 4.0 \cdot 12.0 \cdot 0.85 \cdot c = 34.68 \cdot c$$

Find the distance from the bottom of the section to the neutral axis by taking moments about P_n:

$$28.80 \cdot (40.20 - 3.40) + 31.80 \cdot (40.20 + 1.04) - 34.68 \cdot c \cdot \left(40.20 + 4.65 - \frac{0.85 \cdot c}{2}\right) = 0$$

$$2371.27 - 1555.4 \cdot c + 14.74 \cdot c^2 = 0$$

$$c = 1.55 \ \text{in}$$

Check if original assumption was correct that $\varepsilon_s > \varepsilon_y$.

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

$$\varepsilon_{stop} = (4.65 + 3.40 - 1.55) \cdot \left(\frac{0.003}{1.55}\right) = 0.0126 > 0.00207$$

$$\varepsilon_{sbot} = (4.65 - 1.04 - 1.55) \cdot \left(\frac{0.003}{1.55}\right) = 0.0040 > 0.00207$$

Therefore the assumption was correct.

Then,

$$C = 34.68 \cdot c = 34.68 \cdot 1.55 = 53.75 \ \text{kips}$$

And,

$$P_n = T_{stop} + T_{sbot} - C$$
$$= 28.80 + 31.80 - 53.75 = 6.85 \ \text{kips}$$

[1.3.2.1]

The resistance factor ϕ for Extreme Event II limit state is 1.0. Therefore,

$$\phi \cdot P_n = P_n = 6.85 \text{ kips} > 4.54 \text{ kips} \qquad \underline{\text{OK}}$$

$$\phi \cdot M_n = P_n \cdot e_u = 6.85 \cdot 40.20 \cdot \frac{1}{12}$$

$$= 22.95 \text{ kip-ft} > 15.36 \text{ kip-ft} \qquad \underline{\text{OK}}$$

Therefore, the interior region reinforcement is adequate for the overhang region. Note that if the reinforcement was found inadequate, the barrier bar that extends into the deck could also have been included as tension reinforcement.