SLAB DESIGN EXAMPLE

Design example is in accordance with the AASHTO LRFD Bridge Design Specifications, 5th Ed. (2010) as prescribed by TxDOT Bridge Design Manual - LRFD (May 2009).

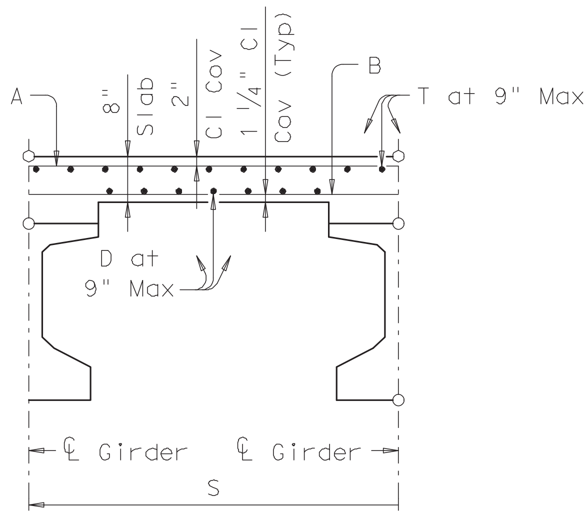
Design: 8" Slab with standard reinforcing
Type Tx40 Prestressed Precast Concrete Beams (36" top flange)
9'-0" Beam Spacing, 3'-0" Overhang with a T551 rail

"AASHTO LRFD" refers to the AASHTO LRFD Bridge Design Specification, 5th Ed. (2010)

"BDM-LRFD" refers to the TxDOT Bridge Design Manual - LRFD (May 2009)

"DM" refers to the TxDOT Bridge Detailing Manual (August 2001)

"TxSP" refers to TxDOT guidance, recommendations, and standard practice.



TYPICAL SECTION

(DM, Ch. 5, Sect. 9, Typical Transverse Section Reinforcing)

8 in Slab thickness,
2 in top clear cover,
1.25 in bottom clear cover

(BDM-LRFD, Ch. 3, Sect. 2, Geometric Constraints)

Bars A ~ # 5's @ 6"
Bars B ~ # 5's @ 6"

(BDM-LRFD, Ch. 3, Sect. 2, Design Criteria)

Bars T ~ # 4's @ 9"
Bars D ~ # 5's @ 9"

(DM, Ch. 5, Sect. 9, Typical Reinforcing)

Deck Design *(AASHTO LRFD 9.7.1)*

Use the Traditional Method in AASHTO LRFD 9.7.3 to design the slab. *(BDM-LRFD, Ch.3, Sect. 2, Design Criteria)*

Use approximate analysis method of AASHTO LRFD 4.6.2.1. *(AASHTO LRFD 9.6.1)* For interior bays, use the unfactored live load moments in AASHTO LRFD Table A4-1. *(AASHTO LRFD C4.6.2.1.6)* For overhangs place one wheel load one foot from the rail. *(AASHTO LRFD 3.6.1.3.1)*

Check the Service Limit State and the Strength Limit State. *(AASHTO LRFD 9.5.2 & AASHTO LRFD 9.5.4)* The Service Limit State is checked by the crack control limits. The live load deflection of the slab is satisfactory by inspection. The Strength Limit State is checked by checking the Ultimate Moment Capacity and the Minimum Steel Requirement *(AASHTO LRFD 5.7.3.3.2)*. Fatigue need not be checked for concrete decks. *(AASHTO LRFD 9.5.3)* The Extreme Event Limit State *(AASHTO LRFD 9.5.5)* is satisfied through rail crash testing. *(BDM-LRFD, C. 3, Sect. 2, Design Criteria)*

Check the distribution reinforcement in the secondary direction. *(AASHTO LRFD 9.7.3.2)*

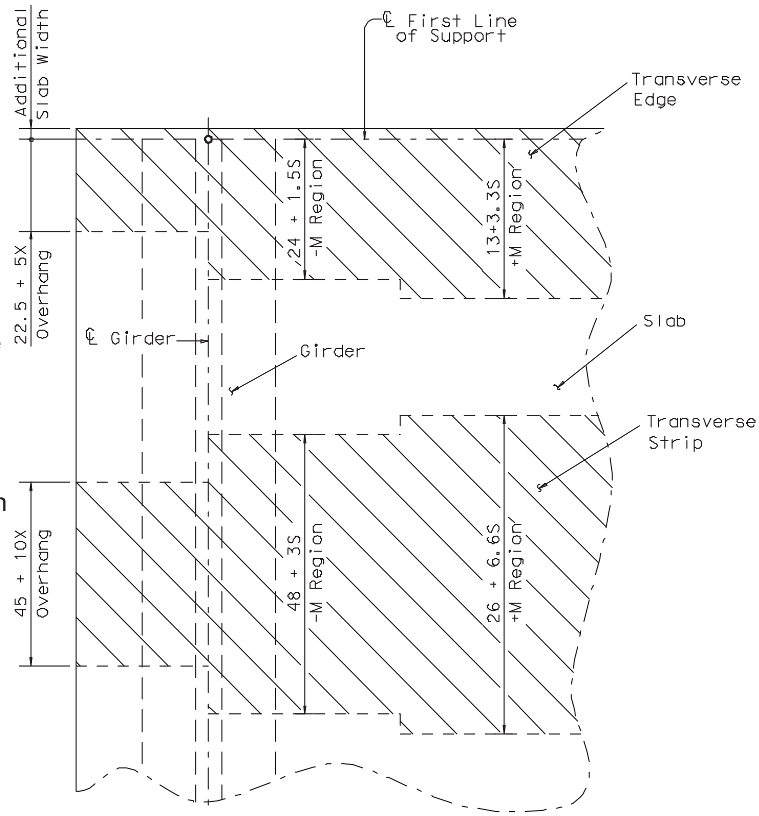
Effective Strip Widths:

The effective width of the strip is the width over which one axle of the design truck or tandem acts.

To get the load per unit width, divide the live load by the effective strip width. *(AASHTO LRFD C4.6.2.1.3)*

Alternately, the live load moments for the positive and negative regions in interior bays from AASHTO LRFD Table A4-1 can be used for design. *(AASHTO LRFD C4.6.2.1.6)*

Therefore, we will use the live load moments from AASHTO LRFD Table A4-1 for the positive and negative regions in interior bays, and place one axle on the effective strip for the overhang.



Depiction of Transverse Strip Widths

In the equations for strip widths, the values for "X" and "S" are in feet but the strip width resulting from the equations is in inches.

(AASHTO LRFD Table 4.6.2.1.3-1)

Overhang Region:

Primary Strip Width:

$$PrimaryStrip_{OH} = 45 + 10\,X$$

Edge Strip Width:

AASHTO LRFD 4.6.2.1.4c defines a transverse edge as a transverse strip along the beam that is located at the edge of the slab as shown above. The Article states, "The effective width of a strip, with or without an edge beam, may be taken as the sum of the distance between the transverse edge of the deck and the centerline of the first line of support for the deck, usually taken as a girder web, plus one-half of the width of strip as specified in Article 4.6.2.1.3."

The intention of this article is to take an effective strip width as half of the transverse strip width plus the additional slab width past the beam end. When the beams are parallel to traffic, the centerline of the first line of support for the deck is the line that intersects the ends of the beams at the center of the web. The additional slab width is the distance from the beam end to the center of the joint minus half of the joint width. This distance is negligible and therefore neglected in this design example.

$$EdgeStrip_{OH} = \frac{1}{2}\,PrimaryStrip_{OH}$$

Define Variables:

$f_y = 60\ \text{ksi}$ — *(BDM-LRFD, Ch. 3, Sect. 2, Materials)*

$E_s = 29000\ \text{ksi}$ — *(AASHTO LRFD 5.4.3.2)*

$f_c = 4\ \text{ksi}$ — *(BDM-LRFD, Ch. 3, Sect. 2, Materials)*

$K_1 = 1.0$ — *(AASHTO LRFD 5.4.2.4)*

$w_c = 0.145\ \text{kcf}$ — *Unit Weight of Concrete for E_c*

$E_c = 33{,}000\ K_1\ w_c^{1.5}\sqrt{f_c}$ $\qquad E_c = 3644\ \text{ksi}$ — *Modulus of Elasticity of Concrete, (AASHTO LRFD Eq. 5.4.2.4-1)*

$\beta_1 = 0.85 - 0.05\left(f_c - 4\ \text{ksi}\right)$ — *(AASHTO LRFD 5.7.2.2)*

Bounded by: $0.65 \le \beta_1 \le 0.85$ $\qquad \beta_1 = 0.85$

$n = \dfrac{E_s}{E_c}$ $\qquad n = 7.96$ — *(AASHTO LRFD 5.7.1)*

$b = 12\ \text{in}$ — *Width of a 1 ft strip*

$h = 8\ \text{in}$ — *Slab Thickness*

$S = 9\ \text{ft}$ — *Beam Spacing*

$OH = 3\ \text{ft}$ — *Length of Slab Overhang*

$RW = 1\ \text{ft}$ — *Nominal Width of Rail (T551)*

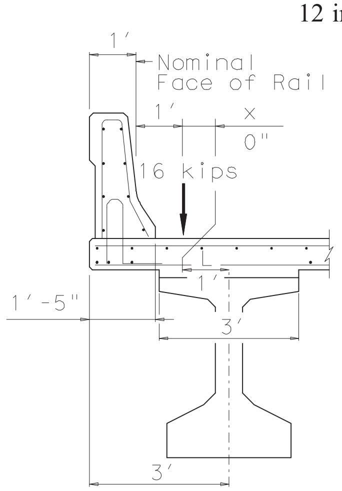
$d_{RailToe} = 1\ \text{ft} + 5\ \text{in}\cdot\dfrac{1\ \text{ft}}{12\ \text{in}}$ $\qquad d_{RailToe} = 1.417\ \text{ft}$ — *Distance from the Edge of the Overhang to the Toe of the Rail (T551)*

$b_{tf} = 36\ \text{in}$ — *Width of top flange of the Girder (IGD)*

Use approximate elastic methods in AASHTO LRFD 4.6.2.1

L = Minimum of: $\begin{cases} \dfrac{b_{tf}}{3} = 12\ \text{in} \\ 15\ \text{in} \end{cases}$ $\qquad L = 12\ \text{in}$ — *Distance from CL Girder to Design Section for Negative Moment (AASHTO LRFD 4.6.2.1.6)*

$X = OH - RW - 1\text{ft} - L\cdot\dfrac{1\ \text{ft}}{12\ \text{in}}$ $\qquad X = 0.00\ \text{ft}$ — *Distance from load to point of support for Overhangs (AASHTO LRFD 4.6.2.1.3)*



In the overhang, place a 16 kip wheel load 1ft from the toe of the rail. (AASHTO LRFD 3.6.1.3.1)

To make the slab design independent of the type of rail, we will place the wheel load 1ft from the nominal face of the rail (1ft from the edge of the slab).

$+M_r$ ~ Slab Positive Moment Capacity:

$A_s = 0.31\ in^2 \frac{b}{6\ in}$ — $A_s = 0.62\ in^2$ — *Area of steel in a 1 ft strip (Bars B)*

$c = \frac{A_s f_y}{0.85\beta_1 f_c b}$ — $c = 1.073\ in$ — *(AASHTO LRFD Eq. 5.7.3.1.2-4)*

$a = c\ \beta_1$ — $a = 0.912\ in$ — *(AASHTO LRFD 5.7.2.2)*

d_{posS} = slab thickness - bottom cover - 1/2 bar diameter

$d_{posS} = h - 1.25in - \frac{0.625in}{2}$ — $d_{posS} = 6.438\ in$

Calc. M_n: *(AASHTO LRFD 5.7.3.2.3)*

$M_n = A_s f_y \left(d_{posS} - \frac{a}{2}\right)\cdot\frac{1ft}{12in}$ — $M_n = 18.54\ kip\cdot ft$ — *(AASHTO LRFD Eq. 5.7.3.2.2-1)*

$\phi = 0.9$ — *(AASHTO LRFD 5.5.4.2.1)*

$M_r = \phi\ M_n$ — $\boxed{M_r = 16.69\ kip\cdot ft}$ — *(AASHTO LRFD Eq. 5.7.3.2.1-1)*

$M_{posR} = M_r$ — $M_{posR} = 16.69\ kip\cdot ft$ — *Positive Factored Flexural Resistance of a 1 ft strip*

$-M_r$ ~ Slab Negative Moment Capacity:

$A_s = 0.31\ in^2 \frac{b}{6\ in}$ — $A_s = 0.62\ in^2$ — *Area of steel in a 1 ft strip (Bars A)*

$c = \frac{A_s f_y}{0.85\beta_1 f_c b}$ — $c = 1.073\ in$ — *(AASHTO LRFD Eq. 5.7.3.1.2-4)*

$a = c\ \beta_1$ — $a = 0.912\ in$ — *(AASHTO LRFD 5.7.2.2)*

d_{negS} = slab thickness - top cover - 1/2 bar diameter

$d_{negS} = h - 2\ in - \frac{0.625\ in}{2}$ — $d_{negS} = 5.688\ in$

Calc. M_n: *(AASHTO LRFD 5.7.3.2.3)*

$M_n = A_s f_y \left(d_{negS} - \frac{a}{2}\right)\cdot\frac{1ft}{12in}$ — $M_n = 16.22\ kip\cdot ft$ — *(AASHTO LRFD Eq. 5.7.3.2.2-1)*

$\phi = 0.9$ — *(AASHTO LRFD 5.5.4.2.1)*

$M_r = \phi\ M_n$ — $\boxed{M_r = 14.60\ kip\cdot ft}$ — *(AASHTO LRFD Eq. 5.7.3.2.1-1)*

$M_{negR} = M_r$ — $M_{negR} = 14.60\ kip\cdot ft$ — *Negative Factored Flexural Resistance of a 1 ft strip*

Loads & Load Factors: *(AASHTO LRFD 3.4.1)*

2 Limit States apply:

Strength I = 1.25 DC + 1.5 DW + 1.75 (LL + IM)
Service I = DC + DW + LL + IM

Use $\eta_D * \eta_R * \eta_I = 1.0$ *(AASHTO LRFD 1.3.2)*

DC: (Slab Dead Load)

Rail DL is considered in overhang check.

DC Load on a 1 ft strip

$$\omega_{Slab} = 0.150\ \text{kcf}\left(h\cdot\frac{1\,\text{ft}}{12\text{in}}\right)\left(b\cdot\frac{1\,\text{ft}}{12\text{in}}\right)$$

$$\omega_{Slab} = 0.100\ \text{klf}$$

DW: (2.5" ACP overlay)

DW Load on a 1 ft strip

$$\omega_{DW} = 0.140\ \text{kcf}\left(2.5\ \text{in}\cdot\frac{1\ \text{ft}}{12\ \text{in}}\right)\left(b\cdot\frac{1\ \text{ft}}{12\ \text{in}}\right)$$

$$\omega_{DW} = 0.029\ \text{klf}$$

Design slabs for 2.5" of asphaltic overlay at 0.140 kcf (TxSP)

LL + IM: Use tabulated LL+IM moments in AASHTO LRFD Appendix A4-1 *(AASHTO LRFD C4.6.2.1.6)*

Check Ultimate Moments at Strength I:

Assume: $-M_{DL} = 0.107\omega\ell^2$

$+M_{DL} = 0.0772\omega\ell^2$

Equations for Moments: Four Continuous Equal Spans Uniformly Loaded

Recall: $S = 9\ \text{ft}$ *Beam Spacing (From Pg. 3)*

Positive Moment:

$$M_{posDC} = 0.0772\ \omega_{Slab}\ S^2$$

$$M_{posDC} = 0.63\ \text{kip}\cdot\text{ft}$$

Positive DC Moment on a 1 ft strip

$$M_{posDW} = 0.0772\ \omega_{DW}\ S^2$$

$$M_{posDW} = 0.18\ \text{kip}\cdot\text{ft}$$

Positive DW Moment on a 1 ft strip

$$M_{posLLIM} = 6.29\ \text{kip}\cdot\text{ft}$$

Positive LL + IM Moment on a 1 ft strip (AASHTO LRFD Table A4-1)

$$M_{posU} = 1.25\ M_{posDC} + 1.25\ M_{posDW} + 1.75\ M_{posLLIM}$$

$$M_{posU} = 12.02\ \text{kip}\cdot\text{ft}$$

Positive Factored Moment at Strength I Limit State on a 1 ft strip

$M_{posR} = 16.69\ \text{kip}\cdot\text{ft} \geq M_{posU}$ OK

Check Ultimate Moments at Strength I: (Con't)

Recall: $S = 9\ \text{ft}$ — *Beam Spacing (From Pg. 3)*

$L = 1\ \text{ft}$ — *Distance from CL Girder to Design Section for Negative Moment (From Pg. 3)*

Negative Moment:

$$M_{negDC} = 0.107\ \omega_{Slab}\ S^2$$
$$M_{negDC} = 0.87\ \text{kip}\cdot\text{ft}$$

Negative DC Moment on a 1 ft strip

$$M_{negDW} = 0.107\ \omega_{DW}\ S^2$$
$$M_{negDW} = 0.25\ \text{kip}\cdot\text{ft}$$

Negative DW Moment on a 1 ft strip

$$M_{negLLIM} = 3.71\ \text{kip}\cdot\text{ft}$$

Negative LL + IM Moment on a 1 ft strip (AASHTO LRFD Table A4-1 ~ Interpolated between values of L)

$$M_{negU} = 1.25\ M_{negDC} + 1.25\ M_{negDW} + 1.75\ M_{negLLIM}$$
$$M_{negU} = 7.89\ \text{kip}\cdot\text{ft}$$

Negative Factored Moment at Strength I Limit State on a 1 ft strip

$M_{negR} = 14.60\ \text{kip}\cdot\text{ft} \geq M_{negU}$ OK

Check Minimum Flexural Reinforcement: *(AASHTO LRFD 5.7.3.3.2)*

$S_x = \frac{b\ h^2}{6}$ — $S_x = 128.00\ \text{in}^3$ — *Section Modulus of a 1 ft section*

$f_r = 0.24\ \sqrt{f_c}$ — $f_r = 0.480\ \text{ksi}$ — *Modulus of Rupture (BDM-LRFD, Ch. 3, Sect. 2, Design Criteria)*

$M_{cr} = S_x\ f_r \cdot \frac{1\ \text{ft}}{12\ \text{in}}$ — $M_{cr} = 5.12\ \text{kip}\cdot\text{ft}$ — *Cracking Moment of a 1 ft section (AASHTO LRFD Eq. 5.7.3.3.2-1)*

Check Negative Moment Reinforcement:

$$M_{f\_neg} = \text{Minimum of: } \begin{cases} 1.2\ M_{cr} = 6.14\ \text{kip}\cdot\text{ft} \\ 1.33\ M_{negU} = 10.50\ \text{kip}\cdot\text{ft} \end{cases}$$

Design for the lesser of $1.2M_{cr}$ or 1.33Mu when determining minimum area of steel required.

$$M_{f\_neg} = 6.14\ \text{kip}\cdot\text{ft}$$

$M_{negR} = 14.60\ \text{kip}\cdot\text{ft} \geq M_{f\_neg}$ OK

Check Positive Moment Reinforcement:

$$M_{f\_pos} = \text{Minimum of: } \begin{cases} 1.2\ M_{cr} = 6.14\ \text{kip}\cdot\text{ft} \\ 1.33\ M_{posU} = 15.98\ \text{kip}\cdot\text{ft} \end{cases}$$

Design for the lesser of $1.2M_{cr}$ or 1.33Mu when determining minimum area of steel required.

$$M_{f\_pos} = 6.14\ \text{kip}\cdot\text{ft}$$

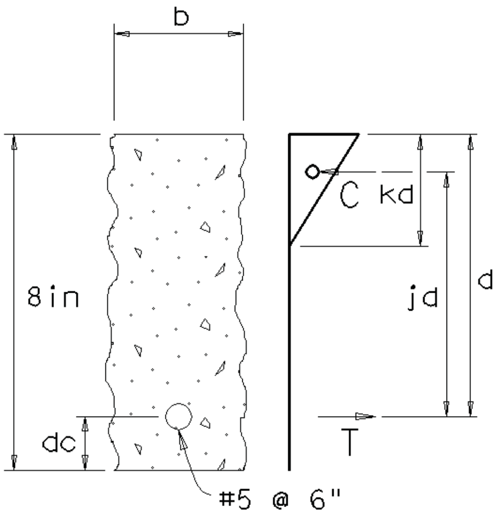
$M_{posR} = 16.69\ \text{kip}\cdot\text{ft} \geq M_{f\_pos}$ OK

Check Crack Control at Service I: *(AASHTO LRFD 5.7.3.4)*

Exposure Condition Factor:

$\gamma_e = 0.75$

For class 2 exposure conditions. (TxSP)



Positive Moment:

$M_{posS} = M_{posLLIM} + M_{posDC} + M_{posDW}$

$M_{posS} = 7.10 \text{ kip}\cdot\text{ft}$

Positive Moment at Service I Limit State on a 1 ft strip

$d_c = h - d_{posS}$ $\quad d_c = 1.562 \text{ in}$

$\rho = \dfrac{A_s}{b\ d_{posS}}$ $\quad \rho = 0.0080$ *Tension Reinforcement Ratio*

$k = -n\ \rho + \sqrt{(n\ \rho)^2 + 2\ n\ \rho}$ $\quad k = 0.299$ *Assuming steel does not yield.*

$j = 1 - \dfrac{1}{3} k$ $\quad j = 0.900$

$f_{ss} = \dfrac{M_{posS}}{j\ A_s\ d_{posS}} \cdot \dfrac{12 \text{ in}}{1 \text{ ft}}$ $\quad f_{ss} = 23.70 \text{ ksi}$ *Service Load Bending Stress in bottom layer of the reinforcing*

$\beta_s = 1 + \dfrac{d_c}{0.7\left(h - d_c\right)}$ $\quad \beta_s = 1.347$ *(AASHTO LRFD 5.7.3.4)*

s_{max} = minimum of: $\begin{cases} \dfrac{700\gamma_e}{\beta_s\ f_{ss}} - 2d_c = 13.32 \text{ in} & \textit{(AASHTO LRFD Eq. 5.7.3.4-1)} \\ 6 \text{ in} & \textit{(BDM-LRFD, Ch. 3, Sect. 2, Design Criteria)} \end{cases}$

$s_{max} = 6 \text{ in}$

$s_{actual} = 6 \text{ in} \quad \le \quad s_{max}$ OK

Check Crack Control at Service I: (Con't)

Negative Moment:

$$M_{negS} = M_{negLLIM} + M_{negDC} + M_{negDW}$$

Negative Moment at Service I Limit State on a 1 ft strip

$$M_{negS} = 4.83\ \text{kip}\cdot\text{ft}$$

$$d_c = h - d_{negS} \qquad d_c = 2.313\ \text{in}$$

$$\rho = \frac{A_s}{b\ d_{negS}} \qquad \rho = 0.0091$$

Tension Reinforcement Ratio

$$k = -n\ \rho + \sqrt{(n\ \rho)^2 + 2\ n\ \rho} \qquad k = 0.315$$

Assuming steel does not yield.

$$j = 1 - \frac{1}{3}k \qquad j = 0.895$$

$$f_{ss} = \frac{M_{negS}}{j\ A_s\ d_{negS}} \cdot \frac{12\ \text{in}}{1\ \text{ft}} \qquad f_{ss} = 18.36\ \text{ksi}$$

Service Load Bending Stress in top layer of the reinforcing

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)} \qquad \beta_s = 1.581$$

(AASHTO LRFD 5.7.3.4)

$$s_{max} = \text{minimum of: } \begin{cases} \dfrac{700\gamma_e}{\beta_s\ f_{ss}} - 2d_c = 13.46\ \text{in} \\ 6\ \text{in} \end{cases}$$

(AASHTO LRFD Eq. 5.7.3.4-1)

(BDM-LRFD, Ch. 3, Sect. 2, Design Criteria)

$$s_{max} = 6\ \text{in}$$

$$s_{actual} = 6\ \text{in} \quad \leq \quad s_{max}$$

OK

Check Overhang:

Slab strength for rail impacts *(AASHTO LRFD 9.5.5)* has been verified through full scale crash testing. *(BDM-LRFD, Ch. 3, Sect. 2, Design Criteria)*

To make the design independent of the rail type, don't design slabs using structurally continuous barriers. Therefore the provisions of AASHTO LRFD 3.6.1.3.4 cannot be used.

Live Load is composed of the axles of the design truck or tandem only; the lane load is not used to design the deck. *(AASHTO LRFD 3.6.1.3.3)*

Only one lane is loaded on the 3 ft overhang; lane spacing is 12 ft. *(AASHTO LRFD 3.6.1.1.1)* Only one wheel load can act on the 3 ft overhang; the first wheel is 1 ft from the face of the rail *(AASHTO LRFD 3.6.1.3.1)*, and the wheel spacing is 6 ft. *(AASHTO LRFD 3.6.1.2.3)*

The thickness of the slab in the primary region (8 in) is the same as the thickness of the slab in the edge strip, therefore Slab ends are critical ~ Check the Edge Strip Only

$m = 1.20$ — *Multiple Presence Factor (AASHTO LRFD Table 3.6.1.1.2-1 ~ For one lane loaded)*

$IM = 33$ — *Impact Load Allowance (AASHTO LRFD Table 3.6.2.1-1)*

Recall: $b = 12\ \text{in}$ — *Width of a 1 ft strip (From Pg. 3)*

$X = 0\ \text{ft}$ — *Distance from load to point of support for Overhangs (From Pg. 3)*

DC: (Slab Dead Load & Rail Dead Load)

$\omega_{Slab} = 0.100\ \text{klf}$ — *Slab Load on a 1 ft strip (From Pg. 5)*

$P_{Rail} = 0.382\ \text{klf}\left(b \cdot \frac{1\ \text{ft}}{12\ \text{in}}\right)$ — *Rail Load on a 1 ft strip (T551)*

$P_{Rail} = 0.382\ \text{kip}$

DW: (2.5" ACP overlay)

$\omega_{DW} = 0.029\ \text{klf}$ — *DW Load on a 1 ft strip (From Pg. 5)*

LL + IM:

$PrimaryStrip_{OH} = 45 + 10\,X$ — *(AASHTO LRFD Table 4.6.2.1.3-1)*

$PrimaryStrip_{OH} = 45\ \text{in}$ — *In the equations for strip width, the values for "X" is in feet but the strip width resulting from the equation is in inches.*

$EdgeStrip_{OH} = \frac{1}{2}\,PrimaryStrip_{OH}$ — *(AASHTO LRFD 4.6.2.1.4c) See discussion on Pg. 2 for clarification.*

$EdgeStrip_{OH} = 22.5\ \text{in}$

$P_{LLIM} = m \cdot 16\ \text{kip}\left(1 + \frac{IM}{100}\right)\frac{b}{EdgeStrip_{OH}}$ — *LL + IM Load on a 1 ft strip*

$P_{LLIM} = 13.62\ \text{kip}$

Check Overhang: (Con't)

Recall: $OH = 3\ ft$ — *Length of Slab Overhang (From Pg. 3)*

$RW = 1\ ft$ — *Nominal Width of Rail (From Pg. 3)*

$d_{RailToe} = 1.417\ ft$ — *Distance from the Edge of the Overhang to the Toe of the Rail (From Pg. 3)*

$L = 12\ in$ — *Distance from CL Girder to Design Section for Negative Moment (From Pg. 3)*

Negative Moment at the Design Section:

$$M_{slab} = \frac{1}{2}\,\omega_{Slab}\left(OH - L\cdot\frac{1\ ft}{12\ in}\right)^2$$

Positive Slab Moment on a 1 ft strip

$M_{slab} = 0.20\ kip\cdot ft$

$$M_{rail} = P_{Rail}\left(OH - \frac{1}{2}\,RW - L\cdot\frac{1\ ft}{12\ in}\right)$$

Positive Rail Moment on a 1 ft strip

$M_{rail} = 0.57\ kip\cdot ft$

$$M_{DW} = \frac{1}{2}\,\omega_{DW}\left(OH - d_{RailToe} - L\cdot\frac{1\ ft}{12\ in}\right)^2$$

Positive DW Moment on a 1 ft strip

$M_{DW} = 0.00\ kip\cdot ft$

$$M_{LLIM} = P_{LLIM}\left(OH - RW - 1\ ft - L\cdot\frac{1\ ft}{12\ in}\right)$$

Positive LL + IM Moment on a 1 ft strip

$M_{LLIM} = 0.00\ kip\cdot ft$

$$M_{negU} = 1.25M_{rail} + 1.25M_{slab} + 1.25\,M_{DW} + 1.75\,M_{LLIM}$$

Negative Factored Moment at Strength I Limit State on a 1 ft strip

$M_{negU} = 0.97\ kip\cdot ft$

$M_{negR} = 14.60\ kip\cdot ft \geq M_{negU}$ OK

Check Distribution Reinforcement:

$A_{sd} = 0.31\ in^2\,\frac{b}{9\ in}$ $\quad A_{sd} = 0.413\ in^2$ — *Area of distribution reinforcement in a 1 ft strip (Bars D)*

$A_s = 0.31\ in^2\,\frac{b}{6\ in}$ $\quad A_s = 0.62\ in^2$ — *Area of primary reinforcement in a 1 ft strip (Bars A)*

$$A_{sd\_min} = \text{minimum of: } \begin{cases} A_s\cdot\dfrac{220\%}{\sqrt{S}} = 0.455\ in^2 \\ A_s\cdot 67\% = 0.415\ in^2 \end{cases}$$

(AASHTO LRFD 9.7.3.2 ~ For primary reinforcement perpendicular to traffic)

$A_{sd\_min} = 0.415\ in^2$

$A_{sd} = 0.413\ in^2 \approx A_{sd\_min}$ OK

Summary: 8" slab with #5 @ 6" O.C. Top & Bottom
OK for prestressed beams spcaced ≤ 10'-6"
OK for steel beams spcaced ≤ 10-3"