The following appendix is supplemental to *NCHRP Research Report 1109: Bridge Railing Design Requirements* (NCHRP Project 22-41).

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APPENDIX

Design Examples

This appendix presents three design examples for railings and two design examples for deck overhangs. The deck overhang design examples are based on the work of this project, which predated the research documented in NCHRP Research Report 1078: MASH Railing Load Requirements for Bridge Deck Overhang (Steelman et al. 2023). The recommendations from NCHRP Research Report 1078 are the basis of the recommended overhang design procedures in the proposed AASHTO LRFD Bridge Design Specifications (LRFD BDS) Section 13, and the overhang examples presented herein are considered out-of-date but are included as documentation of the work completed for this research project. Each rail and overhang has its resistance separately determined by the AASHTO LRFD BDS, 9th Edition (AASHTO 2020) and with the provisions of the proposed Section 13, as outlined in the proposed ballot item. The examples, in accordance with LRFD BDS 9th Edition, use the loads, load distribution, and design methodology listed in Appendix A13. These loads reflect the vehicles of NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features (Ross et al. 1993).

In all examples, the loads, load distribution, and design methodologies are specifically intended for the design of rail system crash test articles. The provisions of the current Appendix A13 and in the proposed Articles 13.7 and 13.10.2.4.2 through 13.10.2.4.4 are specifically meant to apply to the design of crash test articles.

For examples based on actual crash-tested rails and overhangs, their designs can be effectively used by states by adoption (using the provisions of the proposed Article 13.6.1.1.) or by providing a structural geometric equivalent with another rail or overhang (using the provisions of the proposed Article 13.6.1.1.2).

Any supposed lack of capacity in these examples is negated by successful crash testing or by demonstrated equivalency. As such, the value of these examples is very limited.

The railing examples are:
- Example 1: TL-5 Concrete Barrier
- Example 2: TL-4 Steel Post and Beam Railing (side-mount)
- Example 3: TL-3 Curb-Mounted Steel Post and Aluminum Rails

The deck overhang examples are:
- Example 4: Deck Overhang for TL-5 Concrete Barrier
- Example 5: Deck Overhang for TL-3 Post and Beam Railing
Not all relevant materials, reinforcing, and details are presented in the graphic depictions of each railing or overhang. Please refer to the source noted for example for any missing information.
Example 1, TL-5 Concrete Barrier

Railing Construction Details

Railing details taken from TxDOT standard drawing T80SS (issue date 2019).
DESIGN AND ANALYSIS OF BARRIER - PROPOSED METHODS
TxDOT TYPE T80SS

Design Loads and Load Distribution:
The transverse and longitudinal forces are as specified in proposed Table 13.7.2-2

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-5 Barrier</th>
<th>Design Force Equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_t$, Transverse (kips)</td>
<td>162</td>
<td>17.2H-560 for 42≤H≤48</td>
</tr>
<tr>
<td>$F_L$, Longitudinal (kips)</td>
<td>74</td>
<td>0.31H+60.6 for 42≤H≤54</td>
</tr>
<tr>
<td>$F_v$, Vertical (kips) Down</td>
<td>160</td>
<td>496-8H for 42≤H≤54</td>
</tr>
<tr>
<td>$L_t$ and $L_d$ (ft)</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>$H_v$ (min) (in.)</td>
<td>34.2</td>
<td>1.43H-25.9 for 42≤H≤54</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>42</td>
<td>42</td>
</tr>
</tbody>
</table>

Rail Dimensions, Reinforcing and Concrete:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrier Height (H)</td>
<td>42 in.</td>
</tr>
<tr>
<td>Base Width</td>
<td>15.5 in.</td>
</tr>
<tr>
<td>Top Width</td>
<td>7.5 in.</td>
</tr>
<tr>
<td></td>
<td>discounts 1.5&quot; back face projection</td>
</tr>
<tr>
<td>Front Face Concrete Cover</td>
<td>2 in.</td>
</tr>
<tr>
<td>Rear Face Concrete Cover</td>
<td>2 in.</td>
</tr>
<tr>
<td>Inset from Deck Edge</td>
<td>1.5 in.</td>
</tr>
<tr>
<td>Steel $f_y$</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Steel ASTM Grade</td>
<td>A615</td>
</tr>
<tr>
<td>Concrete $f_c'$</td>
<td>3,600 psi</td>
</tr>
<tr>
<td>Expected Material Strength Factor (ESF$_{steel}$)</td>
<td>1.13</td>
</tr>
<tr>
<td>Expected Material Strength Factor (ESF$_{concrete}$)</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Interior Region

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Bar No.</td>
<td>No. 5 bar</td>
</tr>
<tr>
<td>Longitudinal Bar Diameter</td>
<td>0.63 in.</td>
</tr>
<tr>
<td>Longitudinal Bar Spacing</td>
<td>8.75 in.</td>
</tr>
<tr>
<td>Top Transverse Bar No.</td>
<td>No. 5 bar</td>
</tr>
<tr>
<td>Top Transverse Bar Diameter</td>
<td>0.63 in.</td>
</tr>
<tr>
<td>Top Transverse Bar Spacing</td>
<td>6.00 in.</td>
</tr>
<tr>
<td>Bottom Transverse Bar No.</td>
<td>No. 5 bar</td>
</tr>
<tr>
<td>Bottom Transverse Bar Diameter</td>
<td>0.63 in.</td>
</tr>
<tr>
<td>Bottom Transverse Bar Spacing</td>
<td>6.00 in.</td>
</tr>
</tbody>
</table>
End Region

<table>
<thead>
<tr>
<th>Longitudinal Bar No.:</th>
<th>No. 5 bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Bar Diameter:</td>
<td>0.63 in.</td>
</tr>
<tr>
<td>Longitudinal Bar Spacing:</td>
<td>8.75 in.</td>
</tr>
<tr>
<td>Top Transverse Bar No.:</td>
<td>No. 5 bar</td>
</tr>
<tr>
<td>Top Transverse Bar Diameter:</td>
<td>0.63 in.</td>
</tr>
<tr>
<td>Top Transverse Bar Spacing:</td>
<td>6.00 in.</td>
</tr>
<tr>
<td>Bottom Transverse Bar No.:</td>
<td>No. 5 bar</td>
</tr>
<tr>
<td>Bottom Transverse Bar Diameter:</td>
<td>0.63 in.</td>
</tr>
<tr>
<td>Bottom Transverse Bar Spacing:</td>
<td>6.00 in.</td>
</tr>
</tbody>
</table>

Reinforcing Development:

Development of reinforcing steel is determined in accordance with LRFD Section 5.10.8.2. The reinforcing is not epoxy coated. Expected material strength factors are applied to the steel yield stress and concrete strength in the development length equations.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>d_b (in.)</th>
<th>Tension Development</th>
<th>Hook Development</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>l_d (in.)</td>
<td>l_d (in.)</td>
</tr>
<tr>
<td>No. 5</td>
<td>0.63</td>
<td>47.0</td>
<td>24.4</td>
</tr>
<tr>
<td>No. 6</td>
<td>0.75</td>
<td>56.4</td>
<td>29.3</td>
</tr>
</tbody>
</table>

A - Clear distance between reinforcing bars greater than 6d_b and cover is greater than 3d_b
B - Clear distance between reinforcing bars less than 6d_b or cover is less than 3d_b
C - Applicable for side cover normal to the plane of the hook equal or greater than 2.5 in.

M_w for Interior and End Region:

Moment capacity is determined in accordance with the provisions of Article 5.6 for reinforced concrete.

\[
\phi M_w = \phi A_{s,TOTAL} f_y (d-a/2)
\]

\[
c = A_{s,TOTAL} f_y / \alpha f'_c \beta_1 b
\]

\[
a = c \beta_1
\]

\[
\alpha_1 = 0.85
\]

\[
\phi = 1.00 \text{ (Extreme Limit State)}
\]

\[\text{LRFD 5.6.3.1.2-4}\]

\[\text{LRFD 5.6.2.2}\]

\[\text{LRFD 5.6.2.2}\]

\[\text{LRFD 13.6.2}\]

Traffic Face Reinf Contribution

\[
A_{s,TOTAL} = 1.53 \text{ in}^2
\]

\[
ESF_{steel} f_y = 68 \text{ ksi}
\]

\[
ESF_{concrete} f'_c = 4,680 \text{ psi}
\]

\[
a = c \beta_1 = 0.62 \text{ in.}
\]

\[
a/2 = 0.31 \text{ in.}
\]

\[
d_{agg} = 8.56 \text{ in.}
\]

\[
M_w = \phi M_n = 71.5 \text{ k-ft}
\]

Back Face Reinf Contribution

\[
A_{s,TOTAL} = 1.53 \text{ in}^2
\]

\[
ESF_{steel} f_y = 68 \text{ ksi}
\]

\[
ESF_{concrete} f'_c = 4,680 \text{ psi}
\]

\[
a = c \beta_1 = 0.62 \text{ in.}
\]

\[
a/2 = 0.31 \text{ in.}
\]

\[
d_{agg} = 2.94 \text{ in.}
\]

\[
M_w = \phi M_n = 22.8 \text{ k-ft}
\]

\[
M_w = 94.3 \text{ kip-ft}
\]
**MC,base for Interior and End Regions:**

<table>
<thead>
<tr>
<th>Region</th>
<th>Face</th>
<th>d (in)</th>
<th>As (in²)</th>
<th>Bar Spa (in)</th>
<th>b (in)</th>
<th>As (in²/ft)</th>
<th>a (in)</th>
<th>Mc,base (k-ft/ft)</th>
<th>Sum Mc,base (k-ft/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>Traffic Face</td>
<td>13.19</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>44.2</td>
<td>50.7</td>
</tr>
<tr>
<td></td>
<td>Back Face</td>
<td>2.31</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>End</td>
<td>Traffic Face</td>
<td>13.19</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>44.2</td>
<td>50.7</td>
</tr>
<tr>
<td></td>
<td>Back Face</td>
<td>2.31</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>6.5</td>
<td></td>
</tr>
</tbody>
</table>

**Mc,avg for Interior and End Regions:**

<table>
<thead>
<tr>
<th>Region</th>
<th>Face</th>
<th>d (in)</th>
<th>As (in²)</th>
<th>Bar Spa (in)</th>
<th>b (in)</th>
<th>As (in²/ft)</th>
<th>a (in)</th>
<th>Mc,avg (k-ft/ft)</th>
<th>Sum Mc,avg (k-ft/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>Traffic Face</td>
<td>9.19</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>30.3</td>
<td>36.8</td>
</tr>
<tr>
<td></td>
<td>Back Face</td>
<td>2.31</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>End</td>
<td>Traffic Face</td>
<td>9.19</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>30.3</td>
<td>36.8</td>
</tr>
<tr>
<td></td>
<td>Back Face</td>
<td>2.31</td>
<td>0.31</td>
<td>6.00</td>
<td>12.00</td>
<td>0.61</td>
<td>0.87</td>
<td>6.5</td>
<td></td>
</tr>
</tbody>
</table>

**Calculate Capacity of Interior Region:**

A trapezoidal modified yield-line mechanism is used to determine barrier strength.

\[
L_c = L_t + \sqrt{(8M_wH / M_{c,avg})}
\]

\[
R_w = [M_{c,base}(Lt/H) + M_{c,avg}(Lc-Lt / H) + M_{w,avg}(8 / Lc-Lt)](H/H_e)
\]

\[
H = 3.50 \text{ ft} > 3.50 \text{ ft \ OK}
\]

\[
H_e = 2.85 \text{ ft}
\]

\[
M_w = 94.3 \text{ k-ft}
\]

\[
M_{c,avg} = 36.85 \text{ k-ft/ft}
\]

\[
M_{c,base} = 50.71 \text{ k-ft/ft}
\]

\[
L_t = 10.00 \text{ ft}
\]

\[
L_c = 18.46 \text{ ft}
\]

\[
R_w = 397 \text{ kip} > 162.00 \text{ kip \ OK}
\]

**Calculate Capacity of End Region:**

\[
L_c = [5M_{c,avg}Lt + \sqrt{(M_{c,avg}M_{c,avg}Lt^2 + 4M_{c,base}Lt^2 + 128HM_w})] / 8M_{c,avg}
\]

\[
R_w = (H/H_e)[8M_w/(L_t-0.5L_c) + 4M_{c,avg}(L_c-0.5L_c)/H + 2M_{c,base}L_w/H][/(3+(L_c-L_t)/(L_c-0.5L_c))]
\]

\[
M_w = 94.3 \text{ k-ft}
\]

\[
M_{c,avg} = 36.85 \text{ k-ft/ft}
\]

\[
M_{c,base} = 50.71 \text{ k-ft/ft}
\]

\[
L_t = 11.55 \text{ ft}
\]

\[
R_w = 258 \text{ kip} > 162.00 \text{ kip \ OK}
\]
Calculate Punching Shear Capacity:

The contribution of steel to the punching shear capacity is ignored for this example.

**Interior Region Capacity**

\[ V_n = 0.125b_0d_f\sqrt{(ESF_{concrete}f'c)} \]

\[ b_0 = 12L_i + d_f + 2(H - H_e + d_f/2) \]

- \( L_i = 10 \) ft
- \( H - H_e = 7.80 \) in.
- \( d_f = 9.19 \) in. Used average depth
- \( b_0 = 154.0 \) in.

\( V_n = 383 \) kip > 162 kip OK

**End Region Capacity**

\[ V_n = 0.125b_0d_f\sqrt{(ESF_{concrete}f'c)} \]

\[ b_0 = 12L_i + d_f + H - H_e \]

- \( L_i = 10 \) ft
- \( H - H_e = 7.80 \) in.
- \( d_f = 9.19 \) in. Used average depth
- \( b_0 = 137.0 \) in.

\( V_n = 340 \) kip > 162 kip OK

**Summary of Rail Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below.

- Interior Region Barrier Capacity = 397 kip
- Interior Region Capacity/Demand = 2.45 C/D Ratio OK
- End Region Barrier Capacity = 258 kip
- End Region Barrier Capacity/Demand = 1.59 C/D Ratio OK
- Interior Region Punching Shear Capacity = 383 kip
- Interior Region Punching Shear Capacity/Demand = 2.36 C/D Ratio OK
- End Region Punching Shear Capacity = 340 kip
- End Region Punching Shear Capacity/Demand = 2.10 C/D Ratio OK
Design Methodology:

### Design Forces and Designations

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-4 Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_T$ Transverse (kips)</td>
<td>54</td>
</tr>
<tr>
<td>$F_L$ Longitudinal (kips)</td>
<td>18</td>
</tr>
<tr>
<td>$F_V$ Vertical (kips) Down</td>
<td>18</td>
</tr>
<tr>
<td>$L_a$ and $L_L$ (ft)</td>
<td>35</td>
</tr>
<tr>
<td>$L_a$ (ft)</td>
<td>18</td>
</tr>
<tr>
<td>$H_o$ (min) (in.)</td>
<td>32</td>
</tr>
</tbody>
</table>

Minimum $H$ Height of Rail (in.) 32

### Rail Dimensions, Reinforcing and Concrete

Barrier Height ($H$): 36 in.
Height of Top Rail ($h_{TOP}$): 34 in.
Height of Middle Rail ($h_{MID}$): 25 in.
Height of Bottom Rail ($h_{BTM}$): 13 in.
Post Shape: W6x15
Post ASTM Grade: A992
Post $f_y$: 50,000 psi
Post $f_u$: 65,000 psi
Unbraced Post Length ($L_b$): 48 in.
Post Spacing ($S$): 96 in.
$Z_{x,POST}$: 10.8 in$^3$
$r_{x,POST}$: 1.66 in.
$b_f/2t_{POST}$: 1.15
Tube ASTM Grade: A500 Grd C
Tube $f_y$: 50,000 psi
Tube $f_u$: 62,000 psi
Top Rail Shape: HSS 12x4x1/4

### Top Rail

- $A_{TOP}$: 7.1 in$^2$
- $S_{x,TOP}$: 19.8 in$^3$
- $r_{y,TOP}$: 1.72 in.
- $Z_{x,TOP}$: 25.5 in$^3$
- $Z_{y,TOP}$: 11.7 in$^3$
- $(h/t)_{TOP}$: 48.5
- $J_{TOP}$: 59.8 in$^4$

### Front Rail

- $A_{FACE}$: 6.17 in$^2$
- $S_{x,FACE}$: 9.9 in$^3$
- $r_{y,FACE}$: 2.43 in.
- $Z_{x,FACE}$: 12.5 in$^3$
- $(h/t)_{FACE}$: 22.8
- $J_{FACE}$: 70.3 in$^4$

**IL-OH TL-4 Rail**

- Post $Spa = 8$ Max
- Materials: Posts - A992
- Rails - A500 Grd C
- Plates - A572 Gr. 50

- Rail bolted to posts with A499 round-head bolts

- Beam or Slab

- HSS 12x4x1/4 Top Plate

- 2 - HSS 8x6x1/4 Gussets

- 2 - 1" F1554 Gr. 105 anchor rods

- 2 - 1" A499 anchor rods

Minimum $H_e$ Height of Rail (in.) 32

**Design and Analysis of Barrier - AASHTO LRFD 9th Ed**

**Illinois Side-Mounted Bridge Rail - TL4 Compliant**
Calculation of Top Rail Capacity:

\[ M_{rail} = \phi M_{p,top} = \phi F_y Z_{x,rail} \]
\[ M_{pe} = 1273.50 \text{ k-in} \]
\[ \phi M_n = 1273.50 \text{ k-in} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

Calculation of Front Rail Capacity:

\[ M_{rail} = \phi M_{p,front} = \phi F_y Z_{x,rail} \]
\[ M_{pe} = 623.00 \text{ k-in} \]
\[ \phi M_n = 623.00 \text{ k-in} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

Calculation of Post Capacity:

This example does not address the anchorage of the railing as the railing support type varies. For the purposes of this example, the anchorage is assumed adequate to develop the post plastic moment strength.

\[ M_{POST} = \phi M_p = \phi F_y Z_{x,POST} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]
\[ M_{POST} = 540.00 \text{ k-in} \]

Calculate Capacity at Interior Segments:

\[ \bar{Y} = \frac{\Sigma(M_i H_i)}{\Sigma M_i} \]

\[ M_{TOP} = 1273.50 \text{ k-in} \]
\[ M_{FACE} = 623.00 \text{ k-in} \]
\[ \Sigma(M_i H_i) = 97207 \text{ k-in}^2 \]
\[ \Sigma M_i = 2519.50 \text{ k-in} \]
\[ \bar{Y} = 38.58 \text{ in.} \]

\[ P_{post} = \frac{M_{POST}}{\bar{Y}} \]
\[ P_{post} = 14.00 \text{ kip} \]

\[ R = \frac{[16M_p + (N-1)(N+1)P_{post}L]}{[2NL - L_t]} \]

\[ R = \frac{[16M_p + N^2P_{post}L]}{[2NL - L_t]} \]

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>R_{ODD} (kip)</th>
<th>R_{EVEN} (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>268.75</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>133.59</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>288</td>
<td>95.62</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>85.14</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>79.04</td>
<td></td>
</tr>
</tbody>
</table>

\[ R_{CONTROL} = 79.04 \text{ kip} > 54.00 \text{ kip} \]

**OK**
Calculate Capacity at End Segments:

\[
R = \left[ 2M_p + 2P_{post}L(\sum N) \right] / [2NL - L_t]
\]  

[LRFD A13.3.2-3]

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>( R_{\text{END}} ) (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>51.51</td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>38.31</td>
</tr>
<tr>
<td>3</td>
<td>288</td>
<td>39.63</td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>43.96</td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>49.40</td>
</tr>
</tbody>
</table>

\( R_{\text{CONTROL}} = 38.31 \text{ kip} > 54.00 \text{ kip} \quad \text{NG} \)

Vertical Load on Segments:

Calculate Capacity of Rail

Assume the top rail acts as a simply supported beam between posts with a uniformly applied load.

\[
M_{pe} = F_yZ_y
\]

\[
M_{pe} = 585.00 \text{ k-in}
\]

\[
\phi M_p = \phi = 1.00 \quad \text{(Extreme Limit State)}
\]

\[
M = \frac{wS^2}{8}
\]

\[
w = \frac{Fv}{Lv} = 1.00 \text{ kip/ft}
\]

\[
M = 96.00 \text{ k-in} < 585.00 \text{ k-in} \quad \text{OK}
\]

Summary of Rail Capacity vs Demand

The capacity to demand ratios for all design checks are summarized below. Rail anchorage design not included within this example should be considered in a full evaluation.

- Interior Region Capacity = 79.04 kip
  Interior Region Capacity/Demand = 1.46 C/D Ratio \quad \text{OK}

- End Region Capacity = 38.31 kip
  End Region Capacity/Demand = 0.71 C/D Ratio \quad \text{NG}

- Vertical Capacity = 585.00 k-in
  Vertical Capacity/Demand = 6.09 C/D Ratio \quad \text{OK}
DESIGN AND ANALYSIS OF BARRIER - PROPOSED METHODS
ILLINOIS SIDE-MOUNTED BRIDGE RAIL - TL4 COMPLIANT

Design Methodology:

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<tr>
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<td>22</td>
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<td>F_v Vertical (kips) Down</td>
<td>38</td>
<td>101-1.75H for 36 ≤ H ≤ 42</td>
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<td>18</td>
</tr>
<tr>
<td>H_e (min) (in.)</td>
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</tr>
<tr>
<td>Minimum H Height of Rail (in.)</td>
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</tr>
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</table>

**Rail Dimensions, Reinforcing and Concrete:**

- Barrier Height (H): 36 in.
- Height of Top Rail (h_{TOP}): 34 in.
- Height of Middle Rail (h_{MID}): 25 in.
- Height of Bottom Rail (h_{BTM}): 13 in.
- Post Shape: W6x15
- Post ASTM Grade: A992
- Post f_y: 50,000 psi
- Post f_u: 65,000 psi
- Expected Material Strength Factor (ESF\_POST): 1.10
- Unbraced Post Length (L_o): 48 in.
- Post Spacing (S): 96 in.
- Z_x,POST: 10.8 in^3
- r_x,POST: 1.66 in.
- b/2t,POST: 11.5
- Tube ASTM Grade: A500
- Tube f_y: 50,000 psi
- Tube f_u: 62,000 psi
- Expected Material Strength Factor (ESF\_TUBE): 1.10
- Top Rail Shape: HSS 12x6x1/4
- A_{TOP}: 7.1 in^2
- S_x,TOP: 19.8 in^3 Reduced for holes

![IL-OH TL-4 Rail](image-url)
Calculation of Top Rail Capacity:

\[ M_{rail} = \phi M_{p,\text{top}} = \phi F_y Z_{x,\text{rail}} \]
\[ M_{p,\text{top}} = 1400.85 \text{ k-in} \]
\[ \phi M_n = 1400.85 \text{ k-in} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

Calculation of Front Rail Capacity:

\[ M_{rail} = \phi M_{p,\text{front}} = \phi F_y Z_{x,\text{rail}} \]
\[ M_{p,\text{front}} = 685.30 \text{ k-in} \]
\[ \phi M_n = 685.30 \text{ k-in} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

Calculation of Post Capacity:

This example does not address the anchorage of the railing as the railing support type varies. For the purposes of this example, the anchorage is assumed adequate to develop the post plastic moment strength.

\[ M_{POST} = \phi M_p = \phi F_y Z_{x,\text{POST}} \]
\[ M_p = 594 \text{ k-in} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]
\[ M_{POST} = 594.00 \text{ k-in} \]

Calculate Capacity at Interior Segments:

\[ \bar{Y} = \frac{\sum(M_i H_i)}{\sum M_i} \]
\[ M_{TOP} = 1400.85 \text{ k-in} \]
\[ M_{FACE} = 685.30 \text{ k-in} \]
∑(M_iH_i) = 106928 k-in^2 Where H is measured to the CL of the top anchor bolt

∑M_i = 2771.45 k-in

\( \bar{Y} = 38.58 \text{ in.} \)

\[ P_{\text{post}} = \frac{M_{\text{POST}}}{\bar{Y}} \]

\[ P_{\text{post}} = 15.40 \text{ kip} \]

\[ R = \frac{16M_p + (N-1)(N+1)P_{\text{post}}L}{2NL - L_t} \]

\[ R = \frac{16M_p + N^2P_{\text{post}}L}{2NL - L_t} \]

<table>
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<tr>
<th>N</th>
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<th>R_{EVEN} (kip)</th>
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</tr>
<tr>
<td>5</td>
<td>480</td>
<td>87.52</td>
<td></td>
</tr>
</tbody>
</table>

\[ R_w = \frac{\bar{Y}R_{\text{CONTROL}}}{(H_e + 12')} \]

\[ R_w = 119.64 \text{ kip} > 68.00 \text{ kip} \quad \text{OK} \]

**Calculate Capacity at End Segments:**

\[ R = \frac{2M_p + 2P_{\text{post}}L(\sum N)}{2NL - L_t} \]

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<td>48.75</td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>54.70</td>
</tr>
</tbody>
</table>

\[ R_w = 58.63 \text{ kip} > 68.00 \text{ kip} \quad \text{NG} \]
**Vertical Load on Segments:**

**Calculate Capacity of Rail**

Assume the top rail acts as a simply supported beam between posts with a uniformly applied load.

\[ M_{pe} = F_y Z_y \]
\[ M_{pe} = 643.50 \text{ k-in} \]
\[ \phi M_n = 643.50 \text{ k-in} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

\[ M = w \frac{S^2}{8} \]
\[ w = \frac{F_v}{L_v} \]
\[ w = 2.11 \text{ kip/ft} \]
\[ M = 202.67 \text{ k-in} \]
\[ M < 643.50 \text{ k-in} \]
\[ \text{OK} \]

**Summary of Rail Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below. Rail anchorage design not included within this example should be considered in a full evaluation.

**Interior Region Capacity** = 119.64 kip
**Interior Region Capacity/Demand** = 1.76 C/D Ratio
\[ \text{OK} \]

**End Region Capacity** = 58.63 kip
**End Region Capacity/Demand** = 0.86 C/D Ratio
\[ \text{NG} \]

**Vertical Load Capacity** = 643.50 k-in
**Vertical Load Capacity/Demand** = 3.18 C/D Ratio
\[ \text{OK} \]
Example 2, Details, TL-4 Steel Post-and-Beam Railing (Side-Mount)

Railing Construction Details

Railing details are taken from *Development of a MASH Test Level 4 Steel, Side-Mounted, Beam-and-Post, Bridge Rail* (Pena et al. 2020).

**Post Spa = 8' Max**

**Materials**
- Posts - A992
- Rails - A500 Gr. C
- Plates - A572 Gr. 50

**IL-0H TL-4 Rail**

- HSS 12 x 4 x 3/4
- 2 HSS 8 x 6 x 3/4
- W 6 x 15 Post
- P 3/8 x 8 x 8 Top Plate
- 4 P 3/4 x 6 1/6 x 5 1/6 Gusses
- 2 ~ 1" F1554 Gr. 105 anchor rods
- 2 ~ 1" A449 anchor rods

Rails bolted to posts with A449 round-head bolts

Beam or Slab
**Design Methodology:**

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-4 Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_t$ Transverse (kips)</td>
<td>54</td>
</tr>
<tr>
<td>$F_l$ Longitudinal (kips)</td>
<td>18</td>
</tr>
<tr>
<td>$F_v$ Vertical (kips) Down</td>
<td>18</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>3.5</td>
</tr>
<tr>
<td>$L_3$ (ft)</td>
<td>18</td>
</tr>
<tr>
<td>$H_e$ (min) (in.)</td>
<td>32</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>32</td>
</tr>
</tbody>
</table>

### Rail Dimensions, Reinforcing and Concrete:

- **Barrier Height ($H$):** 36 in.
- **Height of Top Rail ($h_{TOP}$):** 34 in.
- **Height of Middle Rail ($h_{MID}$):** 25 in.
- **Height of Bottom Rail ($h_{BTM}$):** 13 in.
- **Post Shape:** W6x15
- **Post ASTM Grade:** A992
  - Post $f_y$: 50,000 psi
  - Post $f_u$: 65,000 psi
- **Unbraced Post Length ($L_b$):** 48 in.
- **Post Spacing ($S$):** 96 in.
- **$Z_{x,POST}$:** 10.8 in$^3$
- **$r_{x,POST}$:** 1.66 in.
- **$b/2r_{x,POST}$:** 11.5
- **Tube ASTM Grade:** A500 Grd C
  - Tube $f_y$: 50,000 psi
  - Tube $f_u$: 62,000 psi
- **Top Rail Shape:** HSS 12x4x1/4
  - $A_{TOP}$: 7.1 in$^2$
  - $S_{x,TOP}$: 19.8 in$^3$ Reduced for holes
  - $r_{x,TOP}$: 1.72 in.
  - $Z_{x,TOP}$: 25.5 in$^3$ Reduced for holes
  - $Z_{y,TOP}$: 11.7 in$^3$
  - $(h/t)_{TOP}$: 48.5
  - $I_{TOP}$: 59.8 in$^4$
Front Rail Shape: HSS 8x6x1/4

\[ A_{\text{FACE}} = 6.17 \text{ in}^2 \]
\[ S_{\text{FACE}} = 9.9 \text{ in}^3 \quad \text{Reduced for holes} \]
\[ r_y,\text{FACE} = 2.43 \text{ in.} \]
\[ Z_{\text{FACE}} = 12.5 \text{ in}^3 \quad \text{Reduced for holes} \]
\[ (b/t),\text{FACE} = 22.8 \]
\[ J_{\text{FACE}} = 70.3 \text{ in}^4 \]

**Calculation of Top Rail Capacity:**

\[ M_{\text{rail}} = \phi M_{\text{p,top}} = \phi F_y Z_{x,\text{rail}} \]
\[ M_{pe} = 1273.50 \text{ k-in} \]
\[ \phi M_n = 1273.50 \text{ k-in} \quad \phi = 1.00 \text{ (Extreme Limit State)} \]

**Calculation of Front Rail Capacity:**

\[ M_{\text{rail}} = \phi M_{\text{p,front}} = \phi F_y Z_{x,\text{rail}} \]
\[ M_{pe} = 623.00 \text{ k-in} \]
\[ \phi M_n = 623.00 \text{ k-in} \quad \phi = 1.00 \text{ (Extreme Limit State)} \]

**Calculation of Post Capacity:**

This example does not address the anchorage of the railing as the railing support type varies. For the purposes of this example, the anchorage is assumed adequate to develop the post plastic moment strength.

\[ M_{\text{POST}} = \phi M_n = \phi F_n Z_{x,\text{POST}} \]
\[ \phi = 1.00 \text{ (Extreme Limit State)} \]
\[ M_{\text{POST}} = 540.00 \text{ k-in} \]

**Calculate Capacity at Interior Segments:**

\[ \bar{Y} = \sum(M_i H_i) / \Sigma M_i \quad \text{[LRFD A13.3.3]} \]
\[ M_{\text{TOP}} = 1273.50 \text{ k-in} \]
\[ M_{\text{FACE}} = 623.00 \text{ k-in} \]
\[ \Sigma(M_i H_i) = 97207 \text{ k-in}^2 \quad \text{Where } H \text{ is measured to the CL of the top anchor bolt} \]
\[ \Sigma M_i = 2519.50 \text{ k-in} \]
\[ \bar{Y} = 38.58 \text{ in.} \]

\[ P_{\text{post}} = M_{\text{POST}} / \bar{Y} \]
\[ P_{\text{post}} = 14.00 \text{ kip} \]
R = \[16M_p + (N-1)(N+1)P_{post}L\] / \[2NL - L_t\]  \hspace{1cm} [LRFD A13.3.2-1]

R = \[16M_p + N^2P_{post}L\] / \[2NL - L_t\]  \hspace{1cm} [LRFD A13.3.2-2]

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>R_{ODD} (kip)</th>
<th>R_{EVEN} (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>268.75</td>
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<tr>
<td>2</td>
<td>192</td>
<td>133.59</td>
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</tr>
<tr>
<td>3</td>
<td>288</td>
<td>95.62</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>85.14</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>79.04</td>
<td></td>
</tr>
</tbody>
</table>

R_{CONTROL} = 79.04 \text{ kip} > 54.00 \text{ kip} \hspace{1cm} \text{OK}

**Calculate Capacity at End Segments:**

R = \[2M_p + 2P_{post}L(\sum N)\] / \[2NL - L_t\]  \hspace{1cm} [LRFD A13.3.2-3]

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<tr>
<th>N</th>
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<tr>
<td>1</td>
<td>96</td>
<td>51.51</td>
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<td>2</td>
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<td>38.31</td>
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</tr>
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<td>5</td>
<td>480</td>
<td>49.40</td>
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R_{CONTROL} = 38.31 \text{ kip} > 54.00 \text{ kip} \hspace{1cm} \text{NG}

**Vertical Load on Segments:**

**Calculate Capacity of Rail**

Assume the top rail acts as a simply supported beam between posts with a uniformly applied load.

\[
M_{pe} = F_y Z_y \\
M_{pe} = 585.00 \text{ k-in} \\
\phi M_n = 585.00 \text{ k-in} \\
\phi = 1.00 \text{ (Extreme Limit State)} \hspace{1cm} [LRFD 13.6.2]
\]

\[
M = wS_y^2/8 \\
w = F_y/L_y \\
w = 1.00 \text{ kip/ft} \\
M = 96.00 \text{ k-in} < 585.00 \text{ k-in} \hspace{1cm} \text{OK}
\]
**Summary of Rail Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below. Rail anchorage design not included within this example should be considered in a full evaluation.

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<th>Status</th>
</tr>
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<tr>
<td>Interior Region</td>
<td>79.04 kip</td>
<td>1.46</td>
<td>OK</td>
</tr>
<tr>
<td>End Region</td>
<td>38.31 kip</td>
<td>0.71</td>
<td>NG</td>
</tr>
<tr>
<td>Vertical Capacity</td>
<td>585.00 k-in</td>
<td>6.09</td>
<td>OK</td>
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- Post Shape: W6x15
- Post ASTM Grade: A992
- Post f_y: 50,000 psi
- Post f_u: 65,000 psi
- Expected Material Strength Factor (ESF_{POST}): 1.10
- Unbraced Post Length (L_u): 48 in.
- Post Spacing (S): 96 in.
- Z_{x,POST}: 10.8 in^2
- r_{t,POST}: 1.66 in.
- b/2t_{POST}: 11.5
- Tube ASTM Grade: A500
- Tube f_y: 50,000 psi
- Tube f_u: 62,000 psi
- Expected Material Strength Factor (ESF_{TUBE}): 1.10
- Top Rail Shape: HSS 12x4x1/4

- A_{TOP}: 7.1 in^2
- S_{x,TOP}: 19.8 in^3 Reduced for holes
Calculation of Top Rail Capacity:

\[ M_{\text{rail}} = \phi M_{p,\text{top}} = \phi F_y Z_{x,\text{rail}} \]

\[ M_{p,\text{top}} = 1400.85 \text{ k-in} \]

\[ \phi M_p = 1400.85 \text{ k-in} \]

\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

Calculation of Front Rail Capacity:

\[ M_{\text{rail}} = \phi M_{p,\text{front}} = \phi F_y Z_{x,\text{rail}} \]

\[ M_{p,\text{front}} = 685.30 \text{ k-in} \]

\[ \phi M_p = 685.30 \text{ k-in} \]

\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

Calculation of Post Capacity:

This example does not address the anchorage of the railing as the railing support type varies. For the purposes of this example, the anchorage is assumed adequate to develop the post plastic moment strength.

\[ M_{\text{POST}} = \phi M_p = \phi F_y Z_{x,\text{POST}} \]

\[ M_p = 594 \text{ k-in} \]

\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

\[ M_{\text{POST}} = 594.00 \text{ k-in} \]

Calculate Capacity at Interior Segments:

\[ \bar{Y} = \frac{\sum(M_i H_i)}{\sum M_i} \]

\[ M_{\text{TOP}} = 1400.85 \text{ k-in} \]

\[ M_{\text{FACE}} = 685.30 \text{ k-in} \]
\[ \sum (M_i H_i) = 106928 \text{ k-in}^2 \quad \text{Where } H \text{ is measured to the CL of the top anchor bolt} \]
\[ \sum M_i = 2771.45 \text{ k-in} \]
\[ \bar{Y} = 38.58 \text{ in.} \]

\[ P_{post} = \frac{M_{POST}}{\bar{Y}} \]
\[ P_{post} = 15.40 \text{ kip} \]

\[ R = \frac{16M_p + (N-1)(N+1)P_{post}L}{2NL - L_t} \]
\[ R = \frac{16M_p + N^2P_{post}L}{2NL - L_t} \]

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<td></td>
</tr>
<tr>
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<td>480</td>
<td>87.52</td>
<td></td>
</tr>
</tbody>
</table>

\[ R_w = \frac{\bar{Y}R_{CONTROL} \times (H_e + 12)}{H} \]
\[ R_w = 119.64 \text{ kip} > 68.00 \text{ kip} \quad \text{OK} \]

**Calculate Capacity at End Segments:**
\[ R = \frac{2M_p + 2P_{post}L(\sum N)}{2NL - L_t} \]

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<td>480</td>
<td>54.70</td>
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\[ R_w = 58.63 \text{ kip} > 68.00 \text{ kip} \quad \text{NG} \]
**Vertical Load on Segments:**

**Calculate Capacity of Rail**

Assume the top rail acts as a simply supported beam between posts with a uniformly applied load.

\[
M_{pe} = F_y Z_y
\]

\[
M_{pe} = 643.50 \text{ k-in}
\]

\[
\phi M_n = 643.50 \text{ k-in} \quad \phi = 1.00 \text{ (Extreme Limit State)}
\]

\[
M = w S^2/8
\]

\[
w = 2.11 \text{ kip/ft}
\]

\[
M = 202.67 \text{ k-in} \quad < 643.50 \text{ k-in} \quad \text{OK}
\]

**Summary of Rail Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below. Rail anchorage design not included within this example should be considered in a full evaluation.

- **Interior Region Capacity** = 119.64 kip
  - Interior Region Capacity/Demand = 1.76 C/D Ratio \text{ OK}

- **End Region Capacity** = 58.63 kip
  - End Region Capacity/Demand = 0.86 C/D Ratio \text{ NG}

- **Vertical Load Capacity** = 643.50 k-in
  - Vertical Load Capacity/Demand = 3.18 C/D Ratio \text{ OK}
Example 3, TL-3 Curb-Mounted Steel Post and Aluminum Rails

Railing Construction Details

Railing details are taken from TxDOT standard drawing T1F (issue date 2019).
DESIGN AND ANALYSIS OF BARRIER - AASHTO LRFD 9TH ED

TxDOT TYPE T1F BRIDGE RAIL
MASH TL-3 COMPLIANT

Design Methodology:

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-3 Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_T$ Transverse (kips)</td>
<td>54</td>
</tr>
<tr>
<td>$F_L$ Longitudinal (kips)</td>
<td>18</td>
</tr>
<tr>
<td>$F_V$ Vertical (kips) Down</td>
<td>4.5</td>
</tr>
<tr>
<td>$L_a$ and $L_d$ (ft)</td>
<td>4</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>18</td>
</tr>
<tr>
<td>$H_e$ (min) (in.)</td>
<td>24</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>27</td>
</tr>
</tbody>
</table>

[LRFD Table A13.2-1]

Rail Dimensions, Reinforcing and Concrete:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrier Height ($H$):</td>
<td>33 in.</td>
</tr>
<tr>
<td>Height of Top Rail ($h_{TOP}$):</td>
<td>30.75 in.</td>
</tr>
<tr>
<td>Height of Bottom Rail ($h_{BTM}$):</td>
<td>18 in.</td>
</tr>
<tr>
<td>Height of Curb ($h_c$):</td>
<td>9 in.</td>
</tr>
<tr>
<td>Post Shape:</td>
<td>Custom</td>
</tr>
<tr>
<td>Post ASTM Grade:</td>
<td>A572</td>
</tr>
<tr>
<td>Post $f_y$:</td>
<td>50,000 psi</td>
</tr>
<tr>
<td>Post $f_u$:</td>
<td>65,000 psi</td>
</tr>
<tr>
<td>Post Thickness:</td>
<td>1.25 in.</td>
</tr>
<tr>
<td>Post Depth:</td>
<td>5.00 in.</td>
</tr>
<tr>
<td>Baseplate Thickness:</td>
<td>1.50 in.</td>
</tr>
<tr>
<td>Baseplate Width:</td>
<td>12.00 in.</td>
</tr>
<tr>
<td>Baseplate Length:</td>
<td>12.00 in.</td>
</tr>
<tr>
<td>Baseplate Face to Anchor:</td>
<td>2.63 in.</td>
</tr>
<tr>
<td>No. Anchor Bolts:</td>
<td>4</td>
</tr>
<tr>
<td>Anchor Bolt Diameter:</td>
<td>0.875 in.</td>
</tr>
<tr>
<td>Anchor ASTM Grade:</td>
<td>A325</td>
</tr>
<tr>
<td>Anchor Rod Steel $f_y$:</td>
<td>120,000 psi</td>
</tr>
<tr>
<td>Anchor to Front Concrete Edge:</td>
<td>3.63 in.</td>
</tr>
<tr>
<td>Anchor to Back Concrete Edge:</td>
<td>5.63 in.</td>
</tr>
<tr>
<td>Anchor Spacing ($S_a$):</td>
<td>8.00 in.</td>
</tr>
<tr>
<td>Unbraced Post Length ($L_a$):</td>
<td>12.75 in.</td>
</tr>
<tr>
<td>Post Spacing ($S$):</td>
<td>96 in.</td>
</tr>
<tr>
<td>Rail Shape:</td>
<td>Custom</td>
</tr>
<tr>
<td>Rail ASTM Grade:</td>
<td>B221 (6061-T6)</td>
</tr>
<tr>
<td>Rail $f_y$:</td>
<td>35,000 psi</td>
</tr>
</tbody>
</table>
Rail $f_u$: 38,000 psi
Rail $E_v$: 10,100 ksi
$I_{x,RAIL}$: 28.53 in$^4$
$I_{y,RAIL}$: 13.50 in$^4$
$S_{x,RAIL}$: 8.15 in$^3$
$S_{y,RAIL}$: 6.00 in$^3$
$Z_{x,RAIL}$: 11.10 in$^3$
$Z_{y,RAIL}$: 8.08 in$^3$
Curb Steel ASTM Grade: A706
Curb Steel $f_y$: 60,000 psi
Concrete $f'_c$: 4,000 psi
Base Width: 14 in.
Top Width: 14 in.
Concrete Cover: 1.4375 in.
Inset from Deck Edge: 1.5 in.
Embedment into Deck: 6.25 in.
Stirrup No.: 5 bar
Stirrup Diameter: 0.63 in.
Stirrup Spacing: 6 in.

Calculation of Rail Capacity:

Plastic moment capacity is used for the rails.

$$\phi = 1.00 \text{ (Extreme Limit State)}$$

$$M_{p,\text{rail}} = ZF_y$$

$$M_{p,X} = 388.52 \text{ k-in}$$
$$M_{p,Y} = 282.74 \text{ k-in}$$

Calculation of Post Capacity:

Post capacity is evaluated at the least width of the post plate. Post to base plate weld is a complete joint penetration weld which develops the post strength.

$$M_{p,\text{post}} = ZF_y$$

$$Z_a = 7.81 \text{ in}^3$$
$$M_p = 390.63 \text{ k-in}$$
$$\phi M_n = 390.63 \text{ k-in}$$
**Capacity of Anchor Rod**

Anchor rods are evaluated for pullout capacity. The failure of the two tension bolts is controlled by a 45 degree cone radiating from the head of the bolts upward. These cones intersect therefore one equivalent cone is used to determine the pullout capacity at each post. This pullout capacity should not exceed the sum of capacities of each anchor without consideration of group effects. Capacity reductions from side cover is assumed to be offset by capacity increases due to reinforcing steel within the cone failure zone. A more thorough investigation of capacity multipliers may be warranted for other barriers.

\[
\text{h}_{\text{eff}} = h_{p} - \text{Bolt Head Height} - \text{Plate Thickness}
\]

\[
A_{\text{CONE}} = \text{Base Width} \times [S_{b} + 2h_{\text{eff}} / \sin 45]\n\]

\[
N_{n} = A_{\text{CONE}} f_{t} = 0.126A_{\text{CONE}} f'_{c}
\]

\[
h_{\text{eff}} = 8.22 \text{ in.}
\]

\[
A_{\text{CONE}} = 437.45 \text{ in}^2
\]

\[
N_{n} = 110.24 \text{ kip}
\]

\[
T = C = 0.85f'_{c} l_{bp} d_{a}
\]

\[
M_{n} = T(d_{a} - a/2)
\]

\[
l_{bp} = 12.00 \text{ in.}
\]

\[
d_{a} = 9.38 \text{ in.}
\]

\[
a = 2.70 \text{ in.}
\]

\[
M_{n} = 884.54 \text{ k-in}
\]

\[
M_{\text{POST}} = \min[M_{n,\text{STEEL}}, M_{n,\text{PULLOUT}}]
\]

\[
M_{\text{POST}} = 390.63 \text{ k-in}
\]

Anchor rods are evaluated for a lateral block shear failure. The base plate is assumed sufficient to transfer all anchorage loading for this example. The failure cone is assumed to remain within the height of curb. The block shear cone is assumed to emanate from the back-most anchors for determination of surface area. The front anchors are conservatively assumed to have equal capacity as the back anchors. Any reductions for limited side cover is assumed to be offset by reinforcing steel within the block shear cone for this example. More thorough calculations may be warranted for other examples.

\[
V_{a} = A_{\text{BLOCK}} f_{t} = 0.126A_{\text{BLOCK}} f'_{c}
\]

\[
P = S_{b} + 2C_{a}/\sin 45
\]

\[
A_{\text{BLOCK}} = PC_{a}/\sin 45
\]

\[
C_{a} = 5.63 \text{ in.}
\]

\[
P = 23.91 \text{ in.}
\]

\[
A_{\text{BLOCK}} = 190.20 \text{ in}^2
\]

\[
V_{a} = 95.86 \text{ kip} \quad > \quad 54.00 \text{ kip} \quad \text{OK}
\]

Interaction equations between tension and shear are not considered for this example due to the capacity to demand ratios for each. More thorough calculation may be warranted for other examples.
Reinforcing Development

Development of reinforcing steel is determined in accordance with LRFD Section 5.10.8.2.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>d_b (in.)</th>
<th>Tension Development</th>
<th>Hook Development</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>l_d (in.)</td>
<td>l_d (in.) A</td>
</tr>
<tr>
<td>5</td>
<td>0.63</td>
<td>45.0</td>
<td>28.1</td>
</tr>
</tbody>
</table>

A - Clear distance between reinforcing bars greater than 6d_b and cover is greater than 3d_b
B - Clear distance between reinforcing bars less than 6d_b or cover is less than 3d_b
C - Applicable for side cover normal to the plane of the hook equal or greater than 2.5 in.

Capacity of Curb:
The curb's contribution to the railing resistance is conservatively ignored.

Calculation of Capacity at Interior Segments:

\[ Y = \frac{\sum(M,H_i)}{\Sigma M_i} \]  \[LRFD A13.3.3\]

\[ M_{RAIL} = 388.52 \text{ k-in} \]
\[ \Sigma(M,H_i) = 19815 \text{ k-in}^2 \]
\[ \Sigma M_i = 777.04 \text{ k-in} \]
\[ \bar{Y} = 25.50 \text{ in.} \]

\[ P_{post} = \frac{M_{POST}}{[\bar{Y} - h_{cont}]} \]
\[ P_{post} = 23.67 \text{ kip} \]

\[ R = \frac{[16M_p + (N-1)(N+1)P_{post}L]}{[2NL - L]} \]  \[LRFD A13.3.2-1\]
\[ R = \frac{[16M_p + NL_p^2]}{[2NL - L]} \]  \[LRFD A13.3.2-2\]

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>R_{ODD} (kip)</th>
<th>R_{EVEN} (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>86.34</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>64.06</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>288</td>
<td>57.98</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>67.77</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>73.44</td>
<td></td>
</tr>
</tbody>
</table>

\[ R_{CONTROL} = 57.98 \text{ kip} > 54.00 \text{ kip} \text{ OK} \]
Calculation of Capacity at End Segments:

\[
R = \frac{2M_p + 2P_{post}L(\sum N)}{2NL - L_t}
\]  

\[LRFD \ A13.3.2-3\]

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>R(\text{END} ) (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>42.36</td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>45.21</td>
</tr>
<tr>
<td>3</td>
<td>288</td>
<td>54.60</td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>65.29</td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>76.46</td>
</tr>
</tbody>
</table>

\(R_{\text{CONTROL}} = 42.36 \text{ kip} > 54.00 \text{ kip} \quad \text{NG}\)

**Vertical Load on Segments:**

**Calculate Capacity of Rail**

Assume the top rail acts as a simply supported beam between posts with a uniformly applied load.

\[
M_{pe} = F_yZ_y
\]

\[
M_{pe} = 282.74 \text{ k-in}
\]

\[
\phi M_n = 282.74 \text{ k-in} \quad \phi = 1.00 \text{ (Extreme Limit State)} \]

\[LRFD \ 13.6.2\]

\[
M = wS^2/8
\]

\[
w = 0.25 \text{ kip/ft}
\]

\[
M = 24.00 \text{ k-in} < 282.74 \text{ k-in} \quad \text{OK}
\]

**Summary of Rail Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below.

Interior Region Capacity = 57.98 kip

Interior Region Capacity/Demand = 1.07 C/D Ratio \quad \text{OK}

End Region Capacity = 42.36 kip

End Region Capacity/Demand = 0.78 C/D Ratio \quad \text{NG}

Capacity for Vertical Load = 282.74 k-in

Capacity/Demand for Vertical Load = 11.78 C/D Ratio \quad \text{OK}
DESIGN AND ANALYSIS OF BARRIER - PROPOSED METHOD

TxDOT TYPE T1F BRIDGE RAIL
MASH TL-3 COMPLIANT

Design Methodology:

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-3 Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_t$ Transverse (kips)</td>
<td>70</td>
</tr>
<tr>
<td>$F_L$ Longitudinal (kips)</td>
<td>18</td>
</tr>
<tr>
<td>$F_v$ Vertical (kips) Down</td>
<td>4.5</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>4</td>
</tr>
<tr>
<td>$L_e$ (ft)</td>
<td>18</td>
</tr>
<tr>
<td>$H_e$ (in.)</td>
<td>19</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>29</td>
</tr>
</tbody>
</table>

Rail Dimensions, Reinforcing and Concrete:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrier Height ($H$)</td>
<td>33 in.</td>
</tr>
<tr>
<td>Height of Top Rail ($h_{Top}$)</td>
<td>30.75 in.</td>
</tr>
<tr>
<td>Height of Bottom Rail ($h_{BTM}$)</td>
<td>18 in.</td>
</tr>
<tr>
<td>Height of curb ($h_P$)</td>
<td>9 in.</td>
</tr>
<tr>
<td>Height to minimum post section ($h_{cont}$)</td>
<td>14 in.</td>
</tr>
<tr>
<td>Post Shape</td>
<td>Custom</td>
</tr>
<tr>
<td>Post ASTM Grade</td>
<td>A572</td>
</tr>
<tr>
<td>Post $f_y$</td>
<td>50,000 psi</td>
</tr>
<tr>
<td>Post $f_u$</td>
<td>65,000 psi</td>
</tr>
<tr>
<td>Post Expected Matl Strength Factor (ESFpost)</td>
<td>1.10</td>
</tr>
<tr>
<td>Post Thickness</td>
<td>1.25 in.</td>
</tr>
<tr>
<td>Post Depth</td>
<td>5.00 in.</td>
</tr>
<tr>
<td>Baseplate Thickness</td>
<td>1.50 in.</td>
</tr>
<tr>
<td>Baseplate Width</td>
<td>12.00 in.</td>
</tr>
<tr>
<td>Baseplate Length</td>
<td>12.00 in.</td>
</tr>
<tr>
<td>Baseplate Face to Anchor</td>
<td>2.63 in.</td>
</tr>
<tr>
<td>No. Anchor Bolts</td>
<td>4</td>
</tr>
<tr>
<td>Anchor Bolt Diameter</td>
<td>0.875 in.</td>
</tr>
<tr>
<td>Anchor ASTM Grade</td>
<td>A325</td>
</tr>
<tr>
<td>Anchor Rod Steel $f_u$</td>
<td>120,000 psi</td>
</tr>
<tr>
<td>Anchor Expected Matl Strength Factor (ESFanchor)</td>
<td>1.50</td>
</tr>
<tr>
<td>Anchor to Concrete Edge</td>
<td>3.63 in.</td>
</tr>
<tr>
<td>Anchor to Back Concrete Edge</td>
<td>5.63 in.</td>
</tr>
<tr>
<td>Anchor Spacing ($S_b$)</td>
<td>8.00 in.</td>
</tr>
<tr>
<td>Unbraced Post Length ($L_{ab}$)</td>
<td>12.75 in.</td>
</tr>
<tr>
<td>Post Spacing ($S$)</td>
<td>96 in.</td>
</tr>
<tr>
<td>Rail Shape</td>
<td>Custom</td>
</tr>
</tbody>
</table>
Rail ASTM Grade: B221
Rail fy: 35,000 psi
Rail fu: 38,000 psi
Rail Expected Matl Strength Factor (ESFrail): 1.10

Sx,Rail: 8.15 in³
Sy,Rail: 6.00 in³
Zx,Rail: 11.10 in³
Zy,Rail: 8.08 in³

Curb Steel ASTM Grade: A706
Curb Steel fy: 60,000 psi
Rebar Expected Matl Strength Factor (ESFbar): 1.10
Concrete f’c: 4,000 psi
Concrete Expected Matl Strength Factor (ESFconcrete): 1.30

Base Width: 14 in.
Top Width: 14 in.
Concrete Cover: 1.4375 in.
Inset from Deck Edge: 1.5 in.
Embedment into Deck: 6.25 in.
Stirrup No.: 5 bar
Stirrup Diameter: 0.63 in.
Stirrup Spacing: 6 in.

**Calculation of Rail Capacity:**

Plastic moment capacity is used for the rails.

\[
\phi = 1.00 \text{ (Extreme Limit State)} \\
M_{p,rail} = ESF_{rail} ZF_y \\
M_{p,X} = 427.37 \text{ k-in} \\
M_{p,Y} = 311.02 \text{ k-in}
\]

**Calculation of Post Capacity:**

Post capacity is evaluated at the least width of the post plate. Post to base plate weld is a complete joint penetration weld which develops the post strength.

\[
M_{p,post} = ESF_{post} ZF_y \\
Z_x = 7.81 \text{ in}³ \\
M_p = 429.69 \text{ k-in} \\
\phi M_p = 429.69 \text{ k-in}
\]
Capacity of Anchor Rod

Anchor rods are evaluated for pullout capacity. The failure of the two tension bolts is controlled by a 45 degree cone radiating from the head of the bolts upward. These cones intersect therefore one equivalent cone is used to determine the pullout capacity at each post. This pullout capacity should not exceed the sum of capacities of each anchor without consideration of group effects. Capacity reductions from side cover is assumed to be offset by capacity increases due to reinforcing steel within the cone failure zone. A more thorough investigation of capacity multipliers may be warranted for other barriers.

\[
\begin{align*}
  h_{ef} &= h_b - \text{Bolt Head Height - Plate Thickness} \\
  A_{\text{CONE}} &= \text{Base Width} \times \left[ S_b + 2\frac{h_{ef}}{\sin45} \right] \\
  N_a &= A_{\text{CONE}} f_t = 0.126 A_{\text{CONE}} \sqrt{f_c} \\
  h_{ef} &= 8.22 \text{ in.} \\
  A_{\text{CONE}} &= 437.45 \text{ in}^2 \\
  N_a &= 125.69 \text{ kip} \\
  T &= C = 0.85 \text{ESF}_{\text{concrete}} f'_c L_{bp} a \\
  M_n &= T (d_s - a/2) \\
  L_{bp} &= 12.00 \text{ in.} \\
  d_s &= 9.38 \text{ in.} \\
  a &= 2.37 \text{ in.} \\
  M_{ns} &= 1029.41 \text{ k-in} \\
\end{align*}
\]

\[
M_{\text{POST}} = \min[M_n_{\text{STEEL}}, M_n_{\text{PULLOUT}}] \\
M_{\text{POST}} = 429.69 \text{ k-in}
\]

Anchor rods are evaluated for a lateral block shear failure. The base plate is assumed sufficient to transfer all anchorage loading for this example. The failure cone is assumed to remain within the height of curb. The block shear cone is assumed to emanate from the back-most anchors for determination of surface area. The front anchors are conservatively assumed to have equal capacity as the back anchors. Any reductions for limited side cover is assumed to be offset by reinforcing steel within the block shear cone for this example. More thorough calculations may be warranted for other examples.

\[
V_{\alpha} = A_{\text{BLOCK}} f_t = 0.126 A_{\text{BLOCK}} \sqrt{\text{ESF}_{\text{concrete}} f'_c} \\
P = S_b + 2C_s / \sin 45 \\
A_{\text{BLOCK}} = PC_s / \sin 45 \\
C_s &= 5.63 \text{ in.} \\
P &= 23.91 \text{ in.} \\
A_{\text{BLOCK}} &= 190.20 \text{ in}^2 \\
V_{\alpha} &= 109.30 \text{ kip} \quad > \quad 70.00 \text{ kip} \quad \text{OK}
\]
Interaction equations between tension and shear are not considered for this example due to the capacity to demand ratios for each. More thorough calculation may be warranted for other examples.

**Reinforcing Development**

Development of reinforcing steel is determined in accordance with LRFD Section 5.10.8.2. Expected material strength factors are applied.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>$d_b$ (in.)</th>
<th>Tension Development</th>
<th>Hook Development</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$l_d$ (in.)</td>
<td>$l_{db}$ (in.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A$</td>
<td>$B$</td>
</tr>
<tr>
<td>5</td>
<td>0.63</td>
<td>43.4 27.1 20.8 29.5 26.0</td>
<td>11.5 11.0</td>
</tr>
</tbody>
</table>

A - Clear distance between reinforcing bars greater than 6$d_b$ and cover is greater than 3$d_b$
B - Clear distance between reinforcing bars less than 6$d_b$ or cover is less than 3$d_b$
C - Applicable for side cover normal to the plane of the hook equal or greater than 2.5 in.

**Capacity of Curb:**

The curb’s contribution to the railing resistance is conservatively ignored.

**Calculation of Capacity at Interior Segments:**

$$
Y = \frac{\sum (M_i H_i)}{\sum M_i}
$$

$$M_{RAIL} = 427.37 \text{ k-in}
$$

$$\sum (M_i H_i) = 20834 \text{ k-in}^2
$$

$$\sum M_i = 854.75 \text{ k-in}
$$

$$Y = 24.38 \text{ in.}
$$

$$P_{post} = \frac{M_{POST}}{[Y - h_{cont}]}
$$

$$P_{post} = 41.42 \text{ kip}
$$

$$R = \frac{16M_p + (N-1)(N+1)P_{post}L}{2NL - L_t}
$$

$$R = \frac{16M_p + N^2P_{post}L}{2NL - L_t}
$$

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>$R_{ODD}$ (kip)</th>
<th>$R_{EVEN}$ (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>94.97</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>88.03</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>288</td>
<td>86.14</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>107.35</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>119.62</td>
<td></td>
</tr>
</tbody>
</table>

$$R_{CONTROL} = 86.14 \text{ kip}
$$

then, adjust for ratio of height of rail force and required effective height

$$R_w = 110.51 \text{ kip} > 70.00 \text{ kip} \quad \text{OK}$$
Calculation of Capacity at End Segments:

\[ R = \frac{2M_p + 2P_{\text{post}}L(\sum N)}{2NL - L_t} \]

<table>
<thead>
<tr>
<th>N</th>
<th>NL (in.)</th>
<th>( R_{\text{END}} ) (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96</td>
<td>67.09</td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>76.09</td>
</tr>
<tr>
<td>3</td>
<td>288</td>
<td>93.60</td>
</tr>
<tr>
<td>4</td>
<td>384</td>
<td>112.82</td>
</tr>
<tr>
<td>5</td>
<td>480</td>
<td>132.66</td>
</tr>
</tbody>
</table>

\( R_{\text{CONTROL}} = 67.09 \text{ kip} \) then, adjust for ratio of height of rail force and required effective height

\( R_w = 86.07 \text{ kip} \) > 70.00 kip \( \text{OK} \)

Vertical Load on Segments:

Calculate Capacity of Rail

The flexural capacity of the top rail subject to vertical load can be assumed equal to its plastic capacity. Assume the top rail acts as a simply supported beam between posts with a uniformly applied load.

\[ M_{pe} = ESF_{rail} F_y Z_y \]

\[ M_{pe} = 311.02 \text{ k-in} \]

\[ \phi M_k = 311.02 \text{ k-in} \]

\( \phi = 1.00 \) (Extreme Limit State)

\[ M = w S^2/8 \]

\[ w = 0.25 \text{ kip/ft} \]

\[ M = 24.00 \text{ k-in} \] < 311.02 k-in \( \text{OK} \)

Summary of Rail Capacity vs Demand

The capacity to demand ratios for all design checks are summarized below.

Interior Region Capacity = 110.51 kip

Interior Region Capacity/Demand = 1.58 C/D Ratio \( \text{OK} \)

End Region Capacity = 86.07 kip

End Region Capacity/Demand = 1.23 C/D Ratio \( \text{OK} \)

Capacity for Vertical Load = 311.02 k-in

Capacity/Demand for Vertical Load = 12.96 C/D Ratio \( \text{OK} \)
Example 4, Deck Overhang for TL-5 Concrete Barrier

Railing and Overhang Construction Details

Deck overhang details are taken from TxDOT standard drawing T80SS (issue date 2019) and TxDOT standard drawing IGTS (issue date 2017).
PARTIAL PLAN FOR
SLABS WITHOUT BREAKBACK
DESIGN AND ANALYSIS OF BRIDGE DECK OVERHANG - PROPOSED METHODS
TxDOT TYPE T80SS

**Design Methodology:**

Bridge deck overhangs shall be designed for the following design cases considered separately:

- **Case 1:** The transverse and longitudinal forces specified in proposed Table 13.7.2-2 - Extreme Event Load Combination II
- **Case 2:** The vertical forces specified in proposed Table 13.7.2-2 - Extreme Event Load Combination II

**Design Forces and Designations:** TL-5 Barrier

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-5 Barrier</th>
<th>Design Force Equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_t$ Transverse (kips)</td>
<td>162</td>
<td>$17.2H - 560$ for $42 \leq H \leq 48$</td>
</tr>
<tr>
<td>$F_L$ Longitudinal (kips)</td>
<td>74</td>
<td>$0.31H + 60.6$ for $42 \leq H \leq 54$</td>
</tr>
<tr>
<td>$F_v$ Vertical (kips) Down</td>
<td>160</td>
<td>$496 - 8H$ for $42 \leq H \leq 54$</td>
</tr>
<tr>
<td>$L_i$ and $L_L$ (ft)</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>$H_i$ (min) (in.)</td>
<td>34.2</td>
<td>$1.43H - 25.9$ for $42 \leq H \leq 54$</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>42</td>
<td>42</td>
</tr>
</tbody>
</table>

**Rail Dimensions, Reinforcing and Concrete:**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrier Height (H):</td>
<td>42 in.</td>
</tr>
<tr>
<td>Barrier Base Width:</td>
<td>15.5 in.</td>
</tr>
<tr>
<td>Barrier CG:</td>
<td>7.75 in.</td>
</tr>
<tr>
<td>Barrier Weight:</td>
<td>0.53 kip/ft</td>
</tr>
<tr>
<td>Interior Barrier Moment Capacity</td>
<td>50.71 kip-ft/ft</td>
</tr>
<tr>
<td>End Barrier Moment Capacity</td>
<td>50.71 kip-ft/ft</td>
</tr>
<tr>
<td>Interior Yield Line Length</td>
<td>18.46 ft</td>
</tr>
<tr>
<td>End Yield Line Length</td>
<td>11.55 ft</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>0.15 kcf</td>
</tr>
<tr>
<td>Overhang Width</td>
<td>40.0 in.</td>
</tr>
<tr>
<td>Overhang Depth</td>
<td>8.5 in.</td>
</tr>
<tr>
<td>Concrete Top Cover</td>
<td>2.5 in.</td>
</tr>
<tr>
<td>Concrete Bottom Cover</td>
<td>1.25 in.</td>
</tr>
<tr>
<td>Inset from Deck Edge</td>
<td>1.5 in.</td>
</tr>
<tr>
<td>Grade Beam Width</td>
<td>0.00 in.</td>
</tr>
<tr>
<td>Steel $f_y$:</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Steel ASTM Grade</td>
<td>A615</td>
</tr>
<tr>
<td>Concrete $f_c$:</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>Expected Material Strength Factor (ESF$_{steel}$):</td>
<td>1.13</td>
</tr>
<tr>
<td>Expected Steel $f_y$:</td>
<td>67.80 ksi</td>
</tr>
<tr>
<td>Expected Material Strength Factor (ESF$_{concr}$):</td>
<td>1.30</td>
</tr>
<tr>
<td>Expected Concrete $f_c$:</td>
<td>5.20 ksi</td>
</tr>
<tr>
<td>Overstrength Factor (I):</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Design Case 1 (Transverse Loads)

Overhang design for the full barrier cantilever moment is unnecessarily conservative by experiments performed by Alberson et al. Expected material strength factors are applied to all equations where capacities are calculated hereafter. Future wearing surface excluded from this example, but should be considered for overhangs that may later have a wearing surface.

Calculate Capacity at Interior Region

At Design Section 1-1

\[
M_{\text{DESIGN}} = M_{\text{DECK}} + M_{\text{BARRIER}} + M_{\text{s}}
\]

Distance to Design Section \((X_{12})\) = 1.42 ft
Support to mid-width of Rail \((D)\) = 0.65 ft
Deck Uniform Load \((w)\) = 0.11 k/ft

\[
M_{\text{DECK}} = 0.11 \text{ k-ft/ft}
\]

\[
M_{\text{BARRIER}} = 0.34 \text{ k-ft/ft}
\]

\[
M_{\text{s,int}} = \frac{F_t H_e}{(L_c + 2H)}
\]

\[
F_t = 162.00 \text{ k}
\]

\[
H = 42 \text{ in}
\]

\[
H_e = 34.2 \text{ in}
\]

\[
L_c = 18.46 \text{ ft}
\]

\[
M_{\text{INT}} = 18.13 \text{ k-ft/ft}
\]

\[
M_{\text{DESIGN}} = 18.58 \text{ k-ft/ft}
\]

\[
T_{\text{INT}} = \frac{F_t}{(L_c + 2H)}
\]

\[
T_{\text{INT}} = 6.36 \text{ kip/ft}
\]

\[
A_d \text{ for } T = 0.094 \text{ in}^2/\text{ft} < 0.13 \text{ in}^2/\text{ft}
\]

Bottom Steel is sufficient to take all the tensile load
\[ a = \frac{[A_s \cdot f_y]}{[\alpha \cdot f'c \cdot b]} \]

\[ \phi M_n = \phi A_s f_y (d-a/2) \]

\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

\[ A_s = 1.08 \text{ in}^2/\text{ft} \]
\[ a = 1.38 \text{ in.} \]
\[ d = 5.69 \text{ in.} \]
\[ \phi M_n = 30.56 \text{ k-ft/ft} > 18.58 \text{ k-ft/ft} \quad \text{OK} \]

**At Design Section 2-2**

Distance to Design Section \( (X_{12}) \) = 1.92 ft

Support to mid-width of Rail \( (D) \) = 2.56 ft

Deck Uniform Load \( (w) \) = 0.11 k/ft

\[ M_{DECK} = 0.20 \text{ k-ft/ft} \]
\[ M_{BARRIER} = 1.37 \text{ k-ft/ft} \]
\[ H_e = 34.2 \text{ in} \]

\[ M_{s,int} = F_t H_e / [L_c + H + 2X_{12} \tan 30] \]
\[ M_{s,int} = 16.68 \text{ k-ft/ft} \]

\[ M_{DESIGN} = M_{DECK} + M_{BARRIER} + M_{s,int} \]
\[ M_{DESIGN} = 18.24 \text{ k-ft/ft} \]

\[ \phi M_n = 30.56 \text{ k-ft/ft} > 18.24 \text{ k-ft/ft} \quad \text{OK} \]

**Calculate Capacity at End Region**

**At Design Section 1-1**

\[ M_{s,end} = F_t H_e / (L_c + H) \]

\[ F_t = 162.00 \text{ k/ft} \]
\[ M_{s,end} = 37.68 \text{ k-ft/ft} \]
\[ M_{DESIGN} = 38.13 \text{ k-ft/ft} \]

\[ T_{out} = F_t / (L_c + H) \]

\[ T_{out} = 10.77 \text{ kip/ft} \]

\[ A_{st} \text{ for } T_{out} = 0.16 \text{ in}^2/\text{ft} < 0.26 \text{ in}^2/\text{ft} \]

\[ A_s = 1.27 \text{ in}^2/\text{ft} \]
\[ a = 1.62 \text{ in.} \]
\[ d_s = 5.69 \text{ in.} \]
\[ \phi M_n = 34.97 \text{ k-ft/ft} > 38.13 \text{ k-ft/ft} \quad \text{NG} \]

Bottom Steel is sufficient to take all the tensile load
At Design Section 2-2

\[ M_{s,\text{end}} = \frac{F_{v}H_{e}}{[L_{c} + H + X_{12}\tan30]} \]

\[ M_{s,\text{end}} = 28.58 \text{ k-ft/ft} \]
\[ M_{\text{DESIGN}} = 30.14 \text{ k-ft/ft} \]

\[ \phi M_{u} = 34.97 \text{ k-ft/ft} > 30.14 \text{ k-ft/ft} \quad \text{OK} \]

Design Case 2 (Vertical Loads)

Calculate Capacity at Interior Region

\[ F_{V} = 160.00 \text{ kip} \]
\[ L_{V} = 40.00 \text{ ft} \]
\[ F_{V} = 4.00 \text{ k/ft} \]
\[ M_{\text{DESIGN}} = 5.17 \text{ k-ft/ft} \]
\[ \phi M_{u} = 30.56 \text{ k-ft/ft} > 5.17 \text{ k-ft/ft} \quad \text{OK} \]

\[ M_{s,\text{int}} = F_{V}D/(L_{V}+H/12+X_{12}\tan30) \]
\[ X_{12} = 1.92 \text{ ft} \]
\[ D = 2.56 \text{ ft} \]
\[ M_{s,\text{int}} = 8.72 \text{ k-ft/ft} \]
\[ M_{U} = 10.28 \text{ k-ft/ft} \]
\[ \phi M_{u} = 30.56 \text{ k-ft/ft} > 10.28 \text{ k-ft/ft} \quad \text{OK} \]

Calculate Capacity at End Region

\[ F_{V} = 160.00 \text{ kip} \]
\[ L_{V} = 40.00 \text{ ft} \]
\[ F_{V} = 4.00 \text{ k-ft} \]
\[ M_{U} = 5.17 \text{ k-ft/ft} \]
\[ \phi M_{u} = 34.97 \text{ k-ft/ft} > 5.17 \text{ k-ft/ft} \quad \text{OK} \]

\[ M_{s,\text{end}} = F_{V}D/(L_{V}+H/12+X_{12}\tan30) \]
\[ X_{12} = 1.92 \text{ ft} \]
\[ D = 2.56 \text{ ft} \]
\[ M_{s,\text{end}} = 9.19 \text{ k-ft/ft} \]
\[ M_{U} = 10.75 \text{ k-ft/ft} \]
\[ \phi M_{u} = 34.97 \text{ k-ft/ft} > 10.75 \text{ k-ft/ft} \quad \text{OK} \]
**Summary of Overhang Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below. These ratios are valid only for the design assumptions and barrier configuration herein. Other design assumptions and configurations may produce different outcome.

<table>
<thead>
<tr>
<th>Case</th>
<th>Capacity (k-ft/ft)</th>
<th>Demand (C/D Ratio)</th>
<th>Outcome</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Interior Region</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Case 1 =</td>
<td>30.56</td>
<td>1.64</td>
<td>OK</td>
</tr>
<tr>
<td>Design Case 2 =</td>
<td>30.56</td>
<td>2.97</td>
<td>OK</td>
</tr>
<tr>
<td><strong>End Region</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Case 1 =</td>
<td>34.97</td>
<td>0.92</td>
<td>NG</td>
</tr>
<tr>
<td>Design Case 2 =</td>
<td>34.97</td>
<td>3.25</td>
<td>OK</td>
</tr>
</tbody>
</table>
Example 5, Deck Overhang for TL-3 Post-and-Beam Rail

Railing and Overhang Construction Details

Railing and deck overhang details are taken from TxDOT standard drawing T223 (issue date 2019).
DESIGN AND ANALYSIS OF BRIDGE DECK OVERHANG - AASHTO LRFD 9TH ED
TxDOT TYPE T223 TRAFFIC RAIL
MASH TL-3 COMPLIANT

Design Methodology:

Bridge deck overhangs are designed for the following design cases considered separately:

Case 1: The transverse and longitudinal forces specified in Table A13.2-1 - Extreme Event Load Combination II
Case 2: The vertical forces specified in Table A13.2-1 - Extreme Event Load Combination II

Design Forces and Designations

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-3 Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_t$ Transverse (kips)</td>
<td>54</td>
</tr>
<tr>
<td>$F_L$ Longitudinal (kips)</td>
<td>18</td>
</tr>
<tr>
<td>$F_v$ Vertical (kips) Down</td>
<td>4.5</td>
</tr>
<tr>
<td>$L_t$ and $L_L$ (ft)</td>
<td>4</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>18</td>
</tr>
<tr>
<td>$H_e$ (min) (in.)</td>
<td>24</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>27</td>
</tr>
</tbody>
</table>

Rail Dimensions, Reinforcing and Concrete:

- Railing Top Width: 15.5 in.
- Railing Height: 32 in.
- Post Width: 9.5 in.
- Railing CG from Backface: 7.31 in.
- Railing Weight: 0.36 kip/ft
- Post to Edge of Deck: 1.50 in.
- Railing Front Cover: 1.75 in.
- Deck to Top Rail Distance (Y): 24.00 in. conservatively taken as He
- Interior Post Length ($L_{INT}$): 4.00 ft
- End Post Length ($L_{EXT}$): 4.00 ft
- Opening Length ($L_{OPEN}$): 6.00 ft
- Concrete Density: 0.15 kcf
- Overhang Width: 30.00 in.
- Overhang Depth: 8.00 in.
- Deck Bottom Cover: 1.25 in.
- Deck Top Cover: 2.00 in.
- Steel $f_y$: 60,000 psi
- Steel ASTM Grade: A706
- Concrete $f_c$: 3,600 psi
**Interior Region**

- Post Bar No.: #5 bar
- Post Bar Diameter: 0.63 in.
- Post Bar Spacing: 6.00 in.
- Top Deck Bar No.: #5 bar
- Top Deck Bar Diameter: 0.63 in.
- Top Deck Bar Spacing: 6.00 in.
- Bottom Bar No.: #4
- Bottom Deck Bar Diameter: 0.50 in.
- Bottom Deck Bar Spacing: 18 in.

**End Region**

- Post Bar No.: #5 bar
- Post Bar Diameter: 0.63 in.
- Post Bar Spacing: 3.50 in.
- Top Deck Bar No.: #5 bar
- Top Deck Bar Diameter: 0.63 in.
- Top Deck Bar Spacing: 3.50 in.
- Bottom Bar No.: #4
- Bottom Deck Bar Diameter: 0.50 in.
- Bottom Deck Bar Spacing: 18 in.

**Reinforcing Development:**

Development of reinforcing steel is determined in accordance with LRFD Section 5.10.8.2.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>$d_b$ (in.)</th>
<th>Tension Development</th>
<th>Hook Development</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$l_{ab}$ (in.)</td>
<td>$l_d$ (in.)</td>
</tr>
<tr>
<td>#5</td>
<td>0.63</td>
<td>47.4</td>
<td>29.6</td>
</tr>
</tbody>
</table>

- **A** - Clear distance between reinforcing bars greater than $6d_b$ and cover is greater than $3d_b$
- **B** - Clear distance between reinforcing bars less than $6d_b$ or cover is less than $3d_b$
- **C** - Applicable for side cover normal to the plane of the hook equal or greater than 2.5 in.
Design Case 1
(Only transverse loads are addressed in this example)

Calculate Capacity at Interior Region

The embedment of railing reinforcing steel shall be considered in determining the plastic moment resistance of the post.

\[
M_{\text{post}} = \phi (A_s f_s)(d_s/a/2)
\]

\[
c = [A_s f_s]/\alpha f'_b
\]

\[
a = c \beta_1
\]

\[
\text{As} = 0.61 \text{ in}^2/\text{ft}
\]

\[
a = 0.52 \text{ in.}
\]

\[
d_s = 7.44 \text{ in.}
\]

\[
M_{\text{post}} = 11.45 \text{ k-ft/ft} \quad \Rightarrow \text{entire post, } M_{\text{post}} = 45.81 \text{ k-ft}
\]

\[
M_d = 12M_{\text{post}} / (W_b + d_s)
\]

\[
M_d = 9.92 \text{ k-ft/ft}
\]

\[
T = 12P_p / (W_b + d_s)
\]

\[
P_{\text{post}} = M_{\text{post}}/\bar{Y} = 22.90 \text{ k/ft}
\]

\[
T = 4.96 \text{ k/ft}
\]

Future wearing surface loading is excluded from this example.

<table>
<thead>
<tr>
<th>Section</th>
<th>Embedment (in.)</th>
<th>(l_{db}) (in.)</th>
<th>Factor</th>
<th>Equivalent (f_s) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Embedment</td>
<td>6.25</td>
<td>12.0</td>
<td>0.52</td>
<td>31,207</td>
</tr>
<tr>
<td>Post Embedment</td>
<td>10.75</td>
<td>12.0</td>
<td>0.89</td>
<td>53,676</td>
</tr>
</tbody>
</table>

Controlling \(f_s = 31,207\)

<table>
<thead>
<tr>
<th>Section</th>
<th>Embedment (in.)</th>
<th>(l_{db}) (in.)</th>
<th>Factor</th>
<th>Equivalent (f_s) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Deck Bar</td>
<td>9.5</td>
<td>12.0</td>
<td>0.79</td>
<td>47,434</td>
</tr>
<tr>
<td>L(#5) Deck Bar</td>
<td>6.75</td>
<td>12.0</td>
<td>0.56</td>
<td>33,703</td>
</tr>
</tbody>
</table>

Controlling \(f_s = 33,703\)

Conservatively use the same stress in both the deck bars and the L bars

\[
M_{\text{DESIGN}} = M_{\text{DECK}} + M_{\text{BARRIER}} + M_d
\]

\[
M_{\text{DECK}} = w L^2/2
\]

\[
M_{\text{BARRIER}} = P x
\]
Distance to Front Face (L) = 0.92 ft
Support to CG of Barrier (x) = 0.31 ft
Deck Uniform Load (w) = 0.10 ksf
Barrier Load Along Post (P) = 0.90 k/ft

- \(M_{\text{DECK}} = 0.04 \text{ k-ft/ft}\)
- \(M_{\text{BARRIER}} = 0.28 \text{ k-ft/ft}\)
- \(M_d = 9.92 \text{ k-ft/ft}\)
- \(M_{\text{DESIGN}} = 10.23 \text{ k-ft/ft}\)

\[
c = \frac{[A_{A^s}f_y - T]}{\alpha f'}b \quad [\text{LRFD 5.6.3.1.2-4}]
\]
\[
a = c\beta_1 \quad [\text{LRFD 5.6.2.2}]
\]
\[
\phi M_n = \phi A_{A^s}f_y(d_s-a/2) + T(H/2 - a/2) \quad \phi = 1.00 \text{ (Extreme Limit State)} \quad [\text{LRFD 13.6.2}]
\]

\[
As = 1.23 \text{ in}^2/\text{ft}
\]
\[
a = 0.99 \text{ in.}
\]
\[
d_s = 5.69 \text{ in.}
\]

\[
A_n \text{ for T}= 0.08 \text{ in}^2/\text{ft} < 0.13 \text{ in}^2/\text{ft} \quad \text{Bottom Steel takes all the tension}
\]

\[
\phi M_n = 17.89 \text{ k-ft/ft} > 10.23 \text{ k-ft/ft} \quad \text{OK}
\]

**Calculate Capacity at End Region:**

The embedment of railing reinforcing steel shall be considered in determining the plastic moment resistance of the post.

<table>
<thead>
<tr>
<th>Section</th>
<th>Embedment (in.)</th>
<th>(l_{dh}) (in.)</th>
<th>Factor</th>
<th>Equivalent fs (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Embedment</td>
<td>6.25</td>
<td>12.0</td>
<td>0.52</td>
<td>31,207</td>
</tr>
<tr>
<td>Post Embedment</td>
<td>10.75</td>
<td>12.0</td>
<td>0.89</td>
<td>53,676</td>
</tr>
</tbody>
</table>

Controlling \(f_s = 31,207\)

\[
M_{\text{post}} = \phi (A_f s)(d_s-a/2) \quad \phi = 1.00 \text{ (Extreme Limit State)} \quad [\text{LRFD 13.6.2}]
\]
\[
c = \frac{[A_f s]}{\alpha f'}b \quad [\text{LRFD 5.6.3.1.2-4}]
\]
\[
a = c\beta_1 \quad [\text{LRFD 5.6.2.2}]
\]

\[
As = 1.05 \text{ in}^2/\text{ft}
\]
\[
a = 0.89 \text{ in.}
\]
\[
d_s = 7.44 \text{ in.}
\]

\[
M_{\text{post}} = 19.12 \text{ k-ft/ft} \quad \text{For the entire post, } M_{\text{post}} = 76.49 \text{ k-ft}
\]

\[
M_d = 12M_{\text{post}} / W_h + d_s \quad [\text{LRFD A13.4.3.1-1}]
\]
\[
M_d = 16.56 \text{ k-ft/ft}
\]

\[
T = 12P_{p} / W_h + d_s \quad [\text{LRFD A13.4.3.1-2}]
\]
\[
P_{p} = M_{\text{post}} / \bar{Y} = 9.56 \text{ k/ft}
\]
\[
T = 2.07 \text{ k/ft}
\]
Bottom Steel takes all the tension

\[
M_{\text{DESIGN}} = M_{\text{DECK}} + M_{\text{BARRIER}} + M_{\text{CT}}
\]
\[
M_{\text{DESIGN}} = 16.87 \text{ k-ft/ft}
\]
\[
A_s = 2.10 \text{ in}^2/\text{ft}
\]
\[
a = 1.73 \text{ in.}
\]
\[
d_s = 5.69 \text{ in.}
\]
\[
\phi M_n = 26.38 \text{ k-ft/ft} > 16.87 \text{ k-ft/ft} \quad \text{OK}
\]

**Calculation of Punching Shear Resistance:**

The factored punching shear for concrete posts shall be equal to the maximum developable tensile capacity of the front most reinforcing steel. The factored shear will be greatest at end regions therefore interior regions are not investigated in this example.

\[
V_u = A_f f_y
\]
\[
\phi = 1.00 \text{ (Extreme Limit State)}
\]
\[
V_u = 32.83 \text{ k/ft}
\]
\[
V_u = 131.30 \text{ kip}
\]

\[
V = \phi V_n
\]
\[
V_n = v_c [W_b + h + 2(E + B/2 + h/2)]h
\]
\[
v_c = (0.0633 + 0.1265/\beta_c) f'_c \leq 0.1265 f'_c
\]
\[
B/2 + h/2 \leq B
\]
\[
\beta_c = W_b/d_b
\]
\[
W_b = 48.00 \text{ in.}
\]
\[
d_b = 7.44 \text{ in.}
\]
\[
\beta_c = 6.45
\]
\[
h = 8.00 \text{ in.}
\]
\[
B = 6.99 \text{ in.}
\]
\[
B/2 + h/2 = 7.50 \text{ in.}
\]
\[
v_c = 0.16 \text{ ksi}
\]
\[
E = 1.95 \text{ in.}
\]
\[
V_n = 92.96 \text{ kip}
\]
\[
V_r = 92.96 \text{ kip} > 131.30 \text{ kip} \quad \text{NG}
\]

**Design Case 2**

**Calculate Capacity at Interior Region:**

\[
P_v = F_v L / L_v
\]
\[
P_v = 2.50 \text{ kip}
\]
\[
V_n = 92.96 \text{ kip} > 2.50 \text{ kip} \quad \text{OK}
\]
\[ M_d = \frac{P \cdot X}{b} \]  
\[ b = 2X + \frac{W_b}{12} \leq L \]  
\[ X = 0.92 \text{ ft} \]  
\[ b = 5.83 \text{ ft} \]  
\[ M_d = 0.39 \text{ k-ft/ft} \]  
\[ M_{\text{DESIGN}} = 0.71 \text{ k-ft/ft} \]  
\[ \phi M_n = 17.89 \text{ k-ft/ft} \quad > \quad 0.71 \text{ k-ft/ft} \quad \text{OK} \]

**Calculate Capacity at End Region:**

\[ V_n = 92.96 \text{ kip} \quad > \quad 2.50 \text{ kip} \quad \text{OK} \]

\[ M_d = \frac{P \cdot X}{b} \]  
\[ b = 2X + \frac{W_b}{12} \leq L \]  
\[ X = 0.92 \text{ ft} \]  
\[ b = 5.83 \text{ ft} \]  
\[ M_d = 0.39 \text{ k-ft/ft} \]  
\[ M_{\text{DESIGN}} = 0.71 \text{ k-ft/ft} \]  
\[ \phi M_n = 26.38 \text{ k-ft/ft} \quad > \quad 0.71 \text{ k-ft/ft} \quad \text{OK} \]

**Summary of Overhang Capacity vs Demand**

The capacity to demand ratios for all design checks are summarized below.

- **Interior Region Design Case 1 =** 17.89 kip-ft/ft  
  Capacity/Demand= 1.75 C/D Ratio \text{OK}

- **End Region Design Case 1 =** 26.38 kip-ft/ft  
  Capacity/Demand= 1.56 C/D Ratio \text{OK}

- **Punching Shear Design Case 1 =** 92.96 kip  
  Capacity/Demand= 0.71 C/D Ratio \text{NG}

- **Interior Region Design Case 2 =** 17.89 kip-ft/ft  
  Capacity/Demand= 25.19 C/D Ratio \text{OK}

- **End Region Design Case 2 =** 26.38 kip-ft/ft  
  Capacity/Demand= 37.14 C/D Ratio \text{OK}

- **Interior Region Punching Shear =** 92.96 kip  
  Capacity/Demand= 37.18 C/D Ratio \text{OK}

- **End Region Punching Shear =** 92.96 kip  
  Capacity/Demand= 37.18 C/D Ratio \text{OK}
Design Methodology:

Bridge deck overhangs shall be designed for the following design cases considered separately:

Case 1: The transverse and longitudinal forces specified in Table 13.7.2.1 Extreme Event Load Combination II

Case 2: The vertical forces specified in Table 13.7.2.1 Extreme Event Load Combination II

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>TL-3 Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_t$ Transverse (kips)</td>
<td>70</td>
</tr>
<tr>
<td>$F_L$ Longitudinal (kips)</td>
<td>18</td>
</tr>
<tr>
<td>$F_V$ Vertical (kips) Down</td>
<td>4.5</td>
</tr>
<tr>
<td>$L_i$ and $L_k$ (ft)</td>
<td>4</td>
</tr>
<tr>
<td>$L_v$ (ft)</td>
<td>18</td>
</tr>
<tr>
<td>$H_t$ (min) (in.)</td>
<td>19</td>
</tr>
<tr>
<td>Minimum $H$ Height of Rail (in.)</td>
<td>29</td>
</tr>
</tbody>
</table>

Rail Dimensions, Reinforcing and Concrete:

- Railing Top Width: 15.5 in.
- Post Width: 9.5 in.
- Railing CG from Backface: 7.31 in.
- Railing Weight: 0.36 kip/ft
- Railing Edge Cover: 1.50 in.
- Railing Front Cover: 1.75 in.
- Deck to Center of Top Rail ($\bar{Y}$): 22.50 in.
- Interior Post Length ($L_{INT}$): 4.00 ft
- End Post Length ($L_{EXT}$): 4.00 ft
- Opening Length ($L_{OPEN}$): 6.00 ft
- Concrete Density: 0.15 kcf
- Overhang Width: 30.00 in.
- Overhang Depth: 8.00 in.
- Deck Bottom Cover: 1.25 in.
- Deck Top Cover: 2.00 in.
- Steel $f_y$: 60,000 psi
- Expected Material Strength Factor (ESF$_{steel}$): 1.10
- Steel ASTM Grade: A706
- Concrete $f'_c$: 3,600 psi
- Expected Material Strength Factor (ESF$_{concrete}$): 1.30
- Overstrength Factor (b): 1.20
### Interior Region

- **Post Bar No.**: #5 bar
- **Post Bar Diameter**: 0.63 in.
- **Post Bar Spacing**: 6.00 in.
- **Top Deck Bar No.**: #5 bar
- **Top Deck Bar Diameter**: 0.63 in.
- **Top Deck Bar Spacing**: 6.00 in.
- **Bottom Bar No.**: #4
- **Bottom Deck Bar Diameter**: 0.50 in.
- **Bottom Deck Bar Spacing**: 18 in.

### End Region

- **Post Bar No.**: #5 bar
- **Post Bar Diameter**: 0.63 in.
- **Post Bar Spacing**: 3.50 in.
- **Top Deck Bar No.**: #5 bar
- **Top Deck Bar Diameter**: 0.63 in.
- **Top Deck Bar Spacing**: 3.50 in.
- **Bottom Bar No.**: #4
- **Bottom Deck Bar Diameter**: 0.50 in.
- **Bottom Deck Bar Spacing**: 18 in.

### Reinforcing Development:

Development of reinforcing steel is determined in accordance with LRFD Section 5.10.8.2.

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>$d_b$ (in.)</th>
<th>Tension Development</th>
<th>Hook Development</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$l_{dh}$ (in.)</td>
<td>$l_d$ (in.) $^A$</td>
</tr>
<tr>
<td>#5</td>
<td>0.63</td>
<td>45.8</td>
<td>28.6</td>
</tr>
</tbody>
</table>

- **A** - Clear distance between reinforcing bars greater than $6d_b$ and cover is greater than $3d_b$.
- **B** - Clear distance between reinforcing bars less than $6d_b$ or cover is less than $3d_b$.
- **C** - Applicable for side cover normal to the plane of the hook equal or greater than 2.5 in.
Design Case 1  
(Only transverse loads are considered in this example)

Calculate Capacity at Interior Region

The embedment of railing reinforcing steel is considered in determining the plastic moment resistance of the post.

\[
M_{\text{post}} = \phi(A_s \text{ESF}_{\text{steel}} f_s)(d_s - a/2)
\]

\[
c = \frac{[A_s \text{ESF}_{\text{steel}} f_s]}{\alpha \text{ESF}_{\text{concrete}} C_y c \beta b}
\]

\[
a = c \beta_1
\]

\[
A_s = 0.61 \text{ in}^2/\text{ft}
\]

\[
a = 0.42 \text{ in.}
\]

\[
d_s = 7.44 \text{ in.}
\]

\[
M_{\text{post}} = 11.96 \text{ k-ft/ft}
\]

\[
M_d = 12M_{\text{post}} / (W_b + d_s)
\]

\[
M_d = 12.42 \text{ k-ft/ft}
\]

\[
T = 12P_{\text{post}} / W_b + d_s
\]

\[
P_{\text{post}} = \frac{M_{\text{post}}}{Y} = 7.65 \text{ k/ft}
\]

\[
T = 1.66 \text{ k/ft}
\]

Future wearing surface loading is excluded from this example.

\[
M_{\text{DESIGN}} = M_{\text{DECK}} + M_{\text{BARRIER}} + M_d
\]

\[
M_{\text{DECK}} = \frac{w L^2}{2}
\]

\[
M_{\text{BARRIER}} = Px
\]

<table>
<thead>
<tr>
<th>Section</th>
<th>Embedment (in.)</th>
<th>(l_{db}) (in.)</th>
<th>Factor</th>
<th>Equivalent (f_s) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Embedment</td>
<td>6.25</td>
<td>11.6</td>
<td>0.54</td>
<td>32,346</td>
</tr>
<tr>
<td>Post Embedment</td>
<td>10.75</td>
<td>11.6</td>
<td>0.93</td>
<td>55,636</td>
</tr>
</tbody>
</table>

Controlling \(f_s = 32,346\)

<table>
<thead>
<tr>
<th>Section</th>
<th>Embedment (in.)</th>
<th>(l_{db}) (in.)</th>
<th>Factor</th>
<th>Equivalent (f_s) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Embedment</td>
<td>9.50</td>
<td>11.6</td>
<td>0.82</td>
<td>49,167</td>
</tr>
<tr>
<td>Post Embedment</td>
<td>6.75</td>
<td>11.6</td>
<td>0.58</td>
<td>34,934</td>
</tr>
</tbody>
</table>

Controlling \(f_s = 34,934\)

Distance to Design section (X) = 0.92 ft
Support to mid-width of rail (D) = 0.31 ft
Deck Uniform Load (w) = 0.10 ksf
Barrier Load Along Post (P) = 0.90 k/ft

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Calculate Capacity at End Region:

The embedment of railing reinforcing steel shall be considered in determining the plastic moment resistance of the post.

<table>
<thead>
<tr>
<th>Section</th>
<th>Embedment (in.)</th>
<th>$l_{dh}$ (in.)</th>
<th>Factor</th>
<th>Equivalent $f_s$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Embedment</td>
<td>6.25</td>
<td>11.6</td>
<td>0.54</td>
<td>32,346</td>
</tr>
<tr>
<td>Post Embedment</td>
<td>10.75</td>
<td>11.6</td>
<td>0.93</td>
<td>55,636</td>
</tr>
</tbody>
</table>

Controlling $f_s = 32,346$

$M_{post} = \phi(A_s f_s)(d_s-a/2)$

$\phi = 1.00$ (Extreme Limit State)

$c = [A_s f_s]/\alpha f', \beta b$  

$A_s = 1.05 \text{ in}^2/\text{ft}$

$a = 0.71 \text{ in.}$

$d_s = 7.44 \text{ in.}$

$M_{post} = 20.08 \text{ k-ft/ft}$

$M_d = 12M_{post} / W_b + d_s$

$M_d = 20.86 \text{ k-ft/ft}$
\[ T = \frac{12P_{\text{post}}}{W_b + d_b} \]

\[ P_p = \frac{M_{\text{post}}}{Y} = 12.85 \text{ k/ft} \]

\[ T = 2.78 \text{ k/ft} \]

Ast for \( T = 0.05 \text{ in}^2/\text{ft} < 0.13 \text{ in}^2/\text{ft} \) Bottom Steel takes all the tension

\[ M_{\text{DESIGN}} = M_{\text{DECK}} + M_{\text{BARRER}} + M_d \]

\[ M_{\text{DESIGN}} = 21.18 \text{ k-ft/ft} \]

\[ A_s = 2.10 \text{ in}^2/\text{ft} \]

\[ a = 1.51 \text{ in.} \]

\[ d_s = 5.69 \text{ in.} \]

\[ \phi M_{n} = 30.77 \text{ k-ft/ft} > 21.18 \text{ k-ft/ft} \] OK

**Calculation of Punching Shear Resistance:**

The factored punching shear for concrete posts is equal to the maximum developable tensile capacity of the front most reinforcing steel. The factored shear will be greatest at end regions therefore interior regions are not investigated in this example.

\[ V_u = A_f f_y \]

\[ \phi = 1.00 \text{ (Extreme Limit State)} \]

\[ V_u = 40.83 \text{ k/ft} \]

\[ V_u = 163.32 \text{ kip} \]

\[ V_t = \phi V_u \]

\[ V_u = v_c (W_b + h + 2(E + B/2 + h/2))h \]

\[ v_c = (0.063 + 0.126/\beta_c) \sqrt{(\text{ESF}_{\text{concrete}} f'_c)} \leq 0.126 \sqrt{(\text{ESF}_{\text{concrete}} f'_c)} \]

\[ B/2 + h/2 \leq B \]

\[ \beta_c = W_b/d_b \]

\[ W_b = 48.00 \text{ in.} \]

\[ d_b = 7.44 \text{ in.} \]

\[ \beta_c = 6.45 \]

\[ h = 8.00 \text{ in.} \]

\[ B = 7.08 \text{ in.} \]

\[ B/2 + h/2 = 7.54 \text{ in.} \]

\[ v_c = 0.27 \text{ ksi} \]

\[ E = 1.86 \text{ in.} \]

\[ V_u = 161.73 \text{ kip} \]

\[ \phi V_t = 161.73 \text{ kip} > 163.32 \text{ kip} \] NG

**Design Case 2**

**Calculate Capacity at Interior Region:**

\[ P_v = \frac{F_v L}{L_v} \]

\[ P_v = 2.50 \text{ kip} \]

\[ V_u = 161.73 \text{ kip} > 2.50 \text{ kip} \] OK
\[ M_d = \frac{P_x X}{b} \]
\[ b = 2X + \frac{W_b}{12} \leq L \]
\[ X = 0.92 \text{ ft} \]
\[ b = 5.83 \text{ ft} \]
\[ M_d = 0.39 \text{ k-ft/ft} \]
\[ M_{\text{DESIGN}} = 0.71 \text{ k-ft/ft} \]
\[ \phi M_n = 20.41 \text{ k-ft/ft} > 0.71 \text{ k-ft/ft} \quad \text{OK} \]

**Calculate Capacity at End Region:**
\[ V_n = 161.73 \text{ kip} > 2.50 \text{ kip} \quad \text{OK} \]
\[ M_d = \frac{P_x X}{b} \]
\[ b = 2X + \frac{W_b}{12} \leq L \]
\[ X = 0.92 \text{ ft} \]
\[ b = 5.83 \text{ ft} \]
\[ M_d = 0.39 \text{ k-ft/ft} \]
\[ M_{\text{DESIGN}} = 0.71 \text{ k-ft/ft} \]
\[ \phi M_n = 30.77 \text{ k-ft/ft} > 0.71 \text{ k-ft/ft} \quad \text{OK} \]

**Summary of Overhang Capacity vs Demand**

The capacity to demand ratios for all design checkes are summarized below.

Interior Region Design Case 1 = 20.41 kip-ft/ft
Interior Region Design Case 1 Capacity/Demand = 1.60 C/D Ratio OK
End Region Design Case 1 = 30.77 kip-ft/ft
End Region Design Case 1 Capacity/Demand = 1.45 C/D Ratio OK
Punching Shear Design Case 1 = 161.73 kip
Punching Shear Design Case 1 Capacity/Demand = 0.99 C/D Ratio NG
Interior Region Design Case 2 = 20.41 kip-ft/ft
Interior Region Design Case 2 Capacity/Demand = 28.73 C/D Ratio OK
End Region Design Case 2 = 43.32 C/D Ratio OK
End Region Punching Shear = 64.69 C/D Ratio OK

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References


