Since 1986, the Federal Highway Administration has required all new bridge railings installed on the National Highway System to be crash tested or to be essentially the same as a railing that was tested. Since many States and municipalities in particular often desire not only architectural or aesthetic enhancements to existing acceptable bridge rails but often request acceptance of untested designs, strict compliance with this requirement could result in full scale testing of scores of essentially similar designs, increased project costs, and significant delays in construction. The AASHTO LRFD Bridge Specifications contain a procedure for analyzing certain types of bridge railings for structural adequacy and provide guidelines for desirable post and beam geometry based on the dimensions of railings that have been successfully crash tested in the past. However, a static analysis of untested designs has not been acceptable as an alternative to crash test verification of railing performance.

The Colorado Department of Transportation (CDOT) essentially combined both approaches by analyzing the capacity of a fully crash-tested railing and comparing the results to a similar Colorado design. The original Colorado design was then modified and re-analyzed to show that it equaled or exceeded the capacity of the tested rail. The FHWA accepted the modified Colorado design for use on the National Highway System based on the State’s analysis, a copy of which has been added, along with this memorandum, to FHWA’s Report 350 Hardware web site under “Bridge Railings.” Specific questions on the Colorado analysis procedure may be addressed to Mr. Michael McMullen, CDOT, at (303) 757-9587 or via e-mail at michael.mcmullen@dot.state.co.us.

The FHWA bridge engineers may use this type of analysis as a basis for acceptance of bridge railings that are similar to a design that has been tested under the National Cooperative Highway Research Program (NCHRP) Report 350 guidelines. It is critical to note that this is not a “cookbook” approach, but rather one that requires careful analysis of all possible failure modes and assumed behavior of all rail elements and connection details. The failure modes may differ from those identified in the Colorado analysis if the bridge railing designs are significantly different. In addition to the structural analysis, bridge railings must also meet the height requirements, size of openings between rails for combination traffic/pedestrian rails, and the recommended rail height-to-traffic face ratio and rail-to-post offsets noted in the LRFD Bridge Specifications.
Our goal is to give highway agencies a greater choice of railing designs without requiring unnecessary testing and without compromising motorist safety. As more rails are tested to comply with NCHRP Report 350, the choice of tested designs will increase and there should be less need to seek acceptance for any design that has not been tested. Please call Mr. Richard Powers of my staff at (202) 366-1320 if you have any questions.
COMPARISON OF THE COLORADO TYPE 10 BRIDGE RAIL TO THE WYOMING TL-4 RAIL

This is a comparison of the geometry, strength, and potential crashworthiness of these two similar bridge rails. The Colorado Type 10 (Attachment 2) is derivative of the Oregon two-tube rail on a curb with stronger anchorage and tube splices changed to generate tension field action under large deformations of the tubes from heavy loads. Recently we decided to raise the curb slightly and close the space between the tubes slightly. The Wyoming TL-4 rail (Attachment 3) is a two tube railing derivative from previous Wyoming two tube rails, with the principal change being enlarging and strengthening the tubes and crash testing the new NCHRP 350 standard.

GEOMETRY

The Oregon rail was successfully crash tested to the NCHRP 230 standard. Consequently geometry and not strength is the primary issue with the Type 10 rail. Geometry is of particular interest with regard to the NCHRP 350 2000P vehicle; i.e., pick-up truck.

Attachment 1, Figure A13.1.1-2 from the AASHTO LRFD specifications shows the post impact potential versus post setback and vertical clear opening. The Wyoming rail has a small (3.5") setback and substantially larger (10.39") openings. This places the Wyoming rail near the boundary of the preferred zone. The Colorado Type 10 Bridge Rail has a larger setback (5") and smaller openings (6.25") which places it in the middle of the preferred zone.

Attachment 3, Figure A14.1.1-3 shows the snagging potential versus the post setback and ratio of rail contact width to rail height. The Wyoming rail has a small ratio (.394) which places it in the questionable area near the boundary of not recommended. The Colorado rail has a higher ratio (.636) which places it centrally in the preferred area well away from the questionable area.

Note that the Verindreel truss post of the Wyoming rail presents the flat unstiffened edge of a plate to vehicle parts that may protrude between the rails during a collision. This plate edge may bend away from impacts by more rigid vehicle parts, thereby decreasing its snagging potential.

LOAD CAPACITY

Using the 3.5' spread of load for PL-2 loads in the LRFD Bridge Design Code, the tubes of the Wyoming rail will resist a single span load of 76.5 KIPS at a 25.4" height using plastic bending analysis. The Colorado Type 10 rail resists a load of 38 KIPS at a similar height. If partial plastic and tensile action is considered in a large deformation mode, a load of 76.5 KIPS can be resisted with a deformation of 9.3". The Wyoming rail will not generate significant tensile action at moderate deformations due to the high longitudinal flexibility of the posts, and the greater play and lower strength in the splices, compared to the tube strength and to the Colorado rail. This tensile action will not be present in any significant degree in the rail bays near expansion joints, but in Colorado we have been minimizing the number of expansion joints used on our new bridges.

Extending this analysis to a two span failure mode (point of impact at post location), the Colorado Type 10 and the Wyoming rails have similar post strengths (50.5 KIPS Wyoming, 61.8 KIPS Colorado) with the difference mostly due to the higher Colorado curb. This results in a rail strength of 83.5
KIPS for the Wyoming rail and 78.9 KIPS for the Colorado rail. By way of comparison, the LRFD code recommends a strength to resist a load of 54 KIPS for the PL-2 load (assumed to be similar to the NCHRP 350 TL-4 load). Tensile effects will not significantly improve either of these strengths, because the deformation needed to generate substantial forces for this longer length failure mode is large.

The ability to resist large tensile loads in the rail tubes may nonetheless provide containment in collisions well beyond the intended load capacity and deformation of the rail system if the vehicle either becomes entangled with the rail, or if the posts break (not bend over). Our experience in Colorado seems to verify this, as we do not see penetration of our Type 10 rail by large heavy vehicles except for only one known instance.

**IMPROVED COLORADO TYPE 10**

If the load capacity of the Colorado Type 10 rail is deemed to be insufficient or the analysis with tensile field action is unacceptable, the rail can be upgraded (Attachment 4). The principal changes would be to reduce the post spacing to 10' maximum, and increase the wall thickness of the tube from 0.1875" to 0.3125". Simplifications to the posts and anchorages and upgrading the splice capacity to follow the tube capacity would also accompany such a change. Costs would increase about $8 per linear foot of rail. The load capacity would be 78 KIPS single span plastic analysis, 158 KIPS at 9" deflection for single span plastic with tensile analysis, and 93.5 KIPS with a two span analysis.

<table>
<thead>
<tr>
<th></th>
<th>TYPE 10</th>
<th>WY TL-4</th>
<th>IMPROVED TYPE 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>SINGLE-SPAN</td>
<td>38 KIPS</td>
<td>77 KIPS</td>
<td>78 KIPS</td>
</tr>
<tr>
<td>SINGLE-SPAN WITH TENSILE ACTION</td>
<td>77 KIPS @ 9.3&quot;</td>
<td>77 KIPS</td>
<td>158 KIPS @ 9&quot;</td>
</tr>
<tr>
<td>POST ONLY</td>
<td>62 KIPS</td>
<td>51 KIPS</td>
<td>55 KIPS</td>
</tr>
<tr>
<td>TWO-SPAN</td>
<td>79 KIPS</td>
<td>84 KIPS</td>
<td>94 KIPS</td>
</tr>
</tbody>
</table>
RAIL PANEL ON WING TERMINAL SECTION
(See roadway plans for ends not attached to Guard Rail.)

- 2-1/4" x 2" threaded anchor studs with hex nuts, hardened washers, and lock washers automatically welded to tube.
- 1-1/4" bolt with hex nut and lock washers.

RAIL PANEL AT EXPANSION DEVICE

ELEVATION - BRIDGE RAIL

- Reflector tube at post. See M-060-1 for details.
- 1/2" horizontal slots in post at tube. See M-060-1 for details.
- Keep end of anchor bolt holes clear.

PLAN - POST DETAIL

- 1" dia. holes at 3 in. centers for anchor bolts.
- Slab reinforcing as shown.

SECTION
- Submitt 2" when bituminous pavement is not used.

ANCHOR DETAIL

- Tube splice, and 1" x 4-1/4" slide at bridge exp'n device. Slot both inner and outer tubes. Stopper top and bottom splices into different post spicings except at expansion joint, place at opposite ends of same post space. (Range of motion = 1" - 0" at bridge expansion device.)
- Plan - tube splice.
MAXIMUM POST SPACING IS 3000 MM

SECTION A-A

SECTION B-B

SECTION C-C

RAIL BOLT DETAIL

POST DETAIL

EXPANSION SLEEVE DETAILS

STANDARD SLEEVE DETAILS

Note: 1) Vertical and diag.-as shown in rail and stanchion shall be shown on fabricator's shop plans.
2) Anchor bolts may be field welded to anchorage, (Shop or Field) at
3) Post bases shall be ground smooth.
4) Posts shall be flush with top after fabrication.
5) Rails shall be shop fabricated.
6) Rolling posts shall be in place and in proper alignment prior to placing of concrete.
7) Posts shall be shop or field drilled 39# to receive rail bolts.
8) After installation of rail, the exposed rail bolt threads shall be painted with two coats of zinc rich paint conforming to the requirements of Section 5.4.3.4-1/galvanizing.
9) Stabs may be utilized in standard sleeves where bolts are required to be field drilled.
10) For details of several types and location of Sections A-A, B-B and C-C see Sheet No. 2.
11) In the area indicated on the PLANS requiring an expansion sleeve, the expansion sleeve shall be located in the rolling panel which passes over the bridge expansion joint.

Attachment 3
Section 13 - Railings

**HIGH POTENTIAL**

![Diagram showing post setback criteria for high potential railings.]

**LOW POTENTIAL**

- **POST SETBACK DISTANCE (in)**
- **S = POST SETBACK DISTANCE (in)**

**COLORADO TYPE 10**

- **Wyoming TL-4:**
  \[
  \frac{A}{H} = \frac{6''\text{curb} + 3''\text{tube} + 4''\text{tube}}{33''} = 0.394
  \]
  \[
  S = 3.54''
  \]
  \[
  \text{OPENINGS} = 9.37'' + 10.39''
  \]

**COLORADO TYPE 10**

- **Wyoming TL-4:**
  \[
  \frac{A}{H} = \frac{11''\text{curb} + 5''\text{tube} + 5''\text{tube}}{33''} = 0.636
  \]
  \[
  S = 5''
  \]
  \[
  \text{OPENINGS} = 5\frac{3}{4}'' + 6\frac{3}{4}''
  \]

Figure A13.1-2 - Potential for wheel, bumper or hood impact with post

For combination and pedestrian railings, the maximum clear vertical opening between succeeding rails or post shall be as specified in Sections 13.8, 13.9, and 13.10.

**ATTACHMENT 1**
Rail Analysis

Given:

Rail Height: \( H := 33, \text{in} \) (Before Future Overlay)

Curb:

Height \( H_c := 11, \text{in} \) (At Post Center Line)

Concrete \( f_c := 4.35, \text{ksi} \)

Post

W200x36 AASHTO M-183 (W8x24 ASTM A-36)

Spacing: \( s := 12.5, \text{ft} \)

Yield Strength \( F_{YP} := 36, \text{ksi} \)

Width \( W_p := 6.495, \text{in} \)

Base Plate

Thickness: \( T_p := 1, \text{in} \)

Width \( W_{pl} := 12, \text{in} \)

Width \( W_{ply} := 10, \text{in} \)

Transverse Y.P.

Plastic Modulus \( z_{PX} := 23.2 \% \text{in}^3 \)

Depth to CL Bolts

\( d_{px} := 8.5, \text{in} \)

\( d_{ply} := 10.5, \text{in} \)

Anchor Bolts

7/8" H.S.

Ultimate Strength \( F_{uA} := 120, \text{ksi} \)

Number

\( N_{ax} := 2 \)

\( N_{ay} := 2 \)

Diameter \( \frac{7}{8}, \text{in} \)

Tubes

Top

\[ 127 \times 127 \times 4.8 \] (5x5x3/1 6)

Height from Roadway \( H_{tt} := 30.5, \text{in} \)

Depth (Horizontal) \( D_{tt} := 5, \text{in} \)

Width (Vertical) \( W_{tt} := 5, \text{in} \)

Thickness (Wall) \( T_{tt} := \frac{3}{16}, \text{in} \)

Area \( A_{tt} := 3.52, \text{in}^2 \)

Plastic Modulus \( Z_{tt} := 6.29, \text{in}^3 \)

Bottom

\[ 127 \times 127 \times 4.8 \] (5x5x3/1 6)

Height from Roadway \( H_{bt} := 19.25, \text{in} \)

Depth (Horizontal) \( D_{bt} := 5, \text{in} \)

Width (Vertical) \( W_{bt} := 54, \text{in} \)

Thickness (Wall) \( T_{bt} := \frac{3}{16}, \text{in} \)

Area \( A_{bt} := 3.52, \text{in}^2 \)

Plastic Modulus \( Z_{bt} := 6.29, \text{in}^3 \)

Yield Strength \( F_{yt} := 46, \text{ksi} \)

Cold Formed ASTM A-500 Grade B

Minimum Tensile Strength \( F_{ut} := 58, \text{ksi} \)

Tube Splice

Number of Bolts \( N_b := 2 \)

Single Shear Planes per Bolt \( N_s := 2 \)

Bolt Diameter \( D_b := 0.875, \text{in} \)

Slotted Hole Size \( S := 1.25, \text{in} \)

Slot Width \( S := 1.0, \text{in} \)

Slot End Distance \( E_n := 4, \text{in} \)

Number of Slips Before Splice Bolts are in Bearing \( N_{sb} := 4 \)

Slot Spacing \( S := 7, \text{in} \)

Post I Tube Connection

Slotted Hole Size

\[ 1.5, \text{in} \]

Anchor Diameter

\[ 0.75, \text{in} \]

Shoulder of end welded

\( s := 0.875, \text{in} \)
Calculations:

Check Plastic Bending Between Posts:
(aka - Single Span Failure Mode)

\[ P_{\text{cl}} := \text{READBMP( "one bump" )} \]

Transverse Load:
\[ F_t := 54 \text{kip} \]
Distributed Length:
\[ L_t := 3.5 \text{ ft} \]
Longitudinal Load:
\[ F_l := 16.5 \text{kip} \]
Flexure Resistance Factor
\[ S f := 1.0 \]
Clear Spacing Between Posts:
\[ CL := S-W \]
Top Tube Plastic Moment:
\[ M_{\text{ptt}} := Z_{tt}F_yt \]
Bottom Tube Plastic Moment:
\[ M_{\text{pbt}} := Z_{bt}F_yt \]
Total Tube Plastic Moment:
\[ M_p := M_{\text{ptt}} + M_{\text{pbt}} \]

Total Ultimate Resistance (i.e. nominal resistance of the railing):
Derived from Eq. A13.3.2-1 for a single span failure mode with plastic hinges at edge of posts.

\[ R_1 := S f \frac{16.M_p}{2CL-L_t} \]
\[ R_1 = 38 \text{kip} \]

Resultant Location:
\[ Y_{\text{bar}} := \frac{M_{\text{ptt}}H_{tt} + M_{\text{pbt}}H_{bt}}{M_P} \]
\[ Y_{\text{bar}} = 24.875411 \]
Check Post:

**Bending Capacity at the base**

<table>
<thead>
<tr>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_f = 1$ Sec. 6.5.5</td>
<td>$\phi_f = 1$ Sec. 6.5.5</td>
</tr>
</tbody>
</table>

**Flexure Resistance Factor**

- Plastic Moment Capacity
  - $M_{ppx} := F_{yp,z} \cdot px$
  - $M_{ppy} := F_{yp,z} \cdot py$
  - $M_{ppx} = 70\%$ ft
  - $M_{ppy} = 260$ kip-ft

**Moment Arm**

- $Arm := Ybar - H, Tp/l$
  - $Arm = 13.1\%$

**Point Load due to Post Bending Capacity:**

- $P_{bend x} := \frac{M_{ppx}}{Arm}$
  - $P_{bend x} = 64\%$
- $P_{bend y} := \frac{M_{ppy}}{Arm}$
  - $P_{bend y} = 24$ kip

**Anchor Capacity**

- Concrete Bearing Resistance Factor
  - $b := 1.0$ Sec. 6.5.5 and 5.4.2
  - $\phi t := 1.0$ Sec. 6.5.5 and 6.4.2

**Bolt Tension Resistance Factor**

- Bolt Area
  - $A_b := \pi \cdot \frac{D^2}{4}$
  - $A_b = 0.601 \text{ in}^2$

- Bolt Tension
  - $T_{ux} := No_{ux} \cdot \phi t \cdot 0.76 \cdot A_b \cdot F_{wa}$
  - $T_{uy} := No_{uy} \cdot \phi t \cdot 0.76 \cdot A_b \cdot F_{ua}$
  - $T_{ux} = 110$ kip
  - $T_{uy} = 110$ kip

**Concrete Compression Block**

- Derived Eq. 5.7.5-2
- Assumes:
  - $\sqrt{\text{concrete area/steel plate area}} \geq 2$
  - Effect of base plate bending is neglected.

- $ax := \frac{T_{ux}}{\phi b \cdot 0.85 f \cdot a_x \cdot W_{plx}}$
  - $ax = 1.236\%$
- $ay := \frac{T_{uy}}{\phi b \cdot 0.85 f \cdot c \cdot 2 \cdot wp \cdot y}$
  - $ay = 1.483$ in

**Point Load due to Anchor Capacity**

- Anchor $x := \frac{T_{ux} \cdot \left( d \cdot plx - \frac{a_x}{2} \right)}{Ybar - H}$
  - Anchor $x = 62$ kip
- Anchor $y := \frac{T_{uy} \cdot \left( d \cdot ply - \frac{a_y}{2} \right)}{Ybar - H}$
  - Anchor $y = 77$ kip

**Ultimate Load Resistance of a Single Post**

- Controlling Post Capacity
  - $Post x := \{ P_{bend x} \text{ Anchor } x \}$
  - $Post y := \{ P_{bend y} \text{ Anchor } y \}$
  - $P_{px} := \min (Post x)$
  - $P_{py} := \min (Post y)$
  - $P_{px} = 62$ kip
  - $P_{py} = 24$ kip
Check Load Capacity @ Post using Combined Post and Tube Strength:  
(aka - Two Span Failure Mode)

Pict2 := READBMP("Two.bmp")

Total Tube Plastic Moment Capacity: \[ M_p = 48.223 \text{kip} \cdot \text{ft} \]

Clear Distance for Two Post Spacings: \[ \text{CL2} := 2 \cdot S - W_p \quad \text{CL2} = 293.505 \text{in} \]

Combined Capacity

Derived from Eq. A13.3.2-2 for a two span failure mode with plastic hinges at edge of posts

\[ R_2 := \frac{P_{px} + \frac{16 \cdot M_p}{2 \cdot \text{CL2} - L_t}}{2} \]

\[ R_2 = 79 \text{kip} \]

if \( R_2 > P_{t, \text{Ft}} "OK", "LOW" \) = "OK"

LOW Two Span Failure Mode Capacity means a three of more span failure mode would have to be used to control the transverse capacity.
Check Splice: Tube splice is assumed to have greater area and thickness than the tube so that the tube controls the splice strength.

Splice Bolt Area

\[ A_{sh} = \frac{D_b^2}{4} \]

\[ A_{sb} = 0.60\text{i}'' \]

Bolt Factored Shear Capacity Assumes: Anchor and Splice Bolts have the same Ultimate Strength

\[ \phi_s := 1.0 \]

Eq. 6.13.2.7

\[ R_{rs} = 2\cdot N_{ob}\cdot 0.6\cdot A_{sh}\cdot F_{ub} \cdot N \]

\[ R_{rs} = 346 \text{ kip} \]

Tube Bolt Factored Bearing Capacity

Sec. 6.5.5 and Tbl. 6.5.4.2 (Set at 1.0 for rail comparison)

\[ \phi_{bb} := 1.0 \]

Eq. 6.13.2.9 (as modified by 6.13.2.7)

Also compared to AISC LRFD 1993 Eq. J3-lb which is applicable when deformation around the bolt holes is not a design consideration

\[ R_{rb} = 228 \text{ kip} \]

Tube Tensile Resistance

Sec. 6.13.5.2

Eq. 6.8.2.1-f Grass Section Yield

\[ P_{rg} := \phi_y Q(\text{Att} + "bt") \]

\[ P_{rg} = 324 \text{ kip} \]

Eq. 6.8.2.1-f Met Section Fracture

\[ A_{ncal} := \text{Att} - 2(\text{Slot Width}, + 0.0625\text{in}, T_{tt}, A_{bt} - 2(\text{Slot Width}, + 0.0625\text{in}), T_{bt} \]

\[ A_{ncal} = 6.243\text{in}^2 \]

Eq. 6.13.5.2 Tension Net Area for Splices

\[ A_{a} := 0.85(A_{n} + A_{bt}) \]

\[ A_{nmax} = 5.984\text{in}^2 \]

\[ A_n = 5.984\text{in}^2 \]

\[ P_m := \text{\$"Fut.AI.U} \]

\[ P_m = 347\text{ kip} \]

Splice Capacity

\[ \text{Splice} := [R_{rs} \quad R_{rb}] \]

\[ R_f := \min(\text{Splice}) \]

\[ R_f = 228 \text{ kip} \]

Splice strength greater than or equal to half the tube gross tension is a recommendation from the 1989 AASHTO Guide Specification for Bridge Railings with 1992 revisions.

\[ \text{if} \left( \frac{P_{rg}}{2}, "Yields", "Fractures" \right) = "Yields" \]

\[ \text{if} \left( R_f > \frac{P_{rg}}{2}, "OK", "LOW" \right) = "OK" \]
Check Mixed Plastic and Tension Field Between Posts:

\[ \text{Post} = \text{Longitudinal Post Resistance} \quad \text{Ppy} \]
\[ \text{WT} = \text{Web Tension} \]
\[ \text{MP} = \text{Plastic Moment} \]

- Additional capacity is available if the rail goes into mixed plastic and tension field action.
- These calculations are intended to show the range of that predicted behavior.
- Arbitrarily use the webs in tension and the flanges with a plastic couple to predict behavior.

**Web Tension**

The effect of the corner radii is neglected.

\[ \text{WT} := 2 \times T_{tt} \times (D_{tt} - 2 \times T_{tt} \times F) + 2 \times T_{bt} \times (D_{bt} - 2 \times T_{bt} \times F) \]

**Flange Plastic Couple**

\[ M_{pf} := \begin{cases} W_{tt} \times T_{tt} \times F \times (D_{tt} - T_{tt}) \\ W_{bt} \times T_{bt} \times F \times (D_{bt} - T_{bt}) \end{cases} \]

**Equivalent Load**

\[ \text{Pf} := \frac{M_{pf} \times 8}{(C_L - L) \times 0.5} \]

**Minimum** number of posts required on each side of load to support the web in tension.

In order to achieve the level of tension shown by the web in tension it is expected that adjacent posts will have to share the tension load.

\[ N_{post} := \text{ceil} \left( \frac{\text{WT}}{\text{Ppy}} \right) \]

**Connection Slip**

Assuming the connection bolts are centered in slotted holes.

This is shown to give a magnitude of slip required to achieve bearing on adjacent posts.

\[ \text{Post/Tube} \quad \text{Slip}_t := \frac{\text{SlotLength} - \text{Anchor}}{2} \]

\[ \text{Splice} \quad \text{Slip}_s := N_{sb} \times \frac{\text{SlotLength} - \frac{D}{2}}{2} \]

**Predicted Total Slip to Achieve Web in Tension**

Assuming 40 ft between Splices and an Impact Midway Between Two Splices.

\[ \text{Slip} := \text{Slip}_t + \text{Slip}_s \left[ 1 + \text{floor} \left( \frac{N_{post} - 1}{40} \times 0.5 \times 5 - 20 \times \text{ft} \right) \right] \]

\[ \text{Slip} = 1.813\% \]
Check Mixed Plastic and Tension Field Between Posts (Continued):

**Delta** at a load equal to Twice the Post Transverse Capacity

Twice the transverse post capacity was chosen as the upper limit of tension field between two posts because once the post transverse capacity is exceeded the first adjacent posts are assumed to be gone and the calculated delta value would be invalid.

\[ 2P_{px} = 125 \text{kip} \]

**Tube with Web in Tension**

Splice Resistance

\[ A := \frac{2PX - Pf}{2 \cdot wT} \]

\[ \Delta t := \sqrt{\frac{\Delta^2 + \left[ (CL - L \cdot 0.5) \cdot 0.5 \right]^2 - (CL - L \cdot 0.5) \cdot 0.5}{2}} \]

\[ \text{Length change of tube} \]

\[ A = 18.715+ \]

Constants:

- psi = \( \frac{\text{lb}}{\text{in}^2} \)
- ksi = 1000 psi
- kip = 1000 lb
- klf = 1000 lb/ft

Arrow = Readbmp( "Amw.bmp" )
Given:

**Rail Height:**

- **H**: 830 mm (Before Future Overlay)

**Curb:**

- **H**, := 150 mm (At Post Center Line)

**Concrete**

- **f_c**: 4.35 ksi (Assumed)

**Post**

- 2 - 16mm x 250mm Plates

**Spacing:**

- **s**: 3000 mm

**Yield Strength**

- **F_{yp}**: 36 ksi (assumed)

**Width**

- **wp**: 200 mm

**Base Plate**

- **T_{pl}**: 16 mm

**Anchor Bolts**

- **M22 H.S.**

**Ultimate Strength**

- **F_{ut}**: 110 ksi

**Diameter**

- **D_{a}**: \( \frac{\text{in}}{8} \)

**Tubes**

- **TOP**
  - 152x102x7.9 (6x4x5/1 6)
- **Bottom**
  - 152x76x6.4 (6x3x1/4)

**Height from Roadway**

- **H_{tt}**: 779 mm

**Depth (Horizontal)**

- **D_{n}**: 6 in

**Width (Vertical)**

- **W_{tt}**: 4 in

**Thickness (Wall)**

- **T_{tt}**: \( \frac{5}{16} \) in

**Area**

- **A_{tt}**: 5.61 in^2

**Plastic Modulus**

- **Z_{tt}**: 10.90 in^3

**Yield Strength**

- **F_{yt}**: 46 ksi

**Minimum Tensile Strength**

- **F_{Ut}**: 58 ksi

**Cold Formed ASTM A-500 Grade B (Assumed)**

**Plates at Base**

- PL:\( t=16 \) mm
- PLn:\( t=25 \) mm

**Plates at 1st Rail**

- PLtr:\( t=16 \) mm
- PLlr:\( t=16 \) X mm

**Width**

- W:\( \text{pl}_{x}=300 \) mm
- W:\( \text{ply}=290 \) mm

**Depth to CL Bolts**

- d:\( \text{pl}_{x}=240 \) mm
- d:\( \text{ply}=330 \) mm

**Values taken from AISC 9th Edition ASD**

- **Z_{bt}**: 7.62 in^3

- **FY_t**: 46 ksi

- **F_{Ut}**: 58 ksi
Given:

Double Bolted Tube Splice

Number of Bolts \( N_b = 2 \)  
Single Shear Planes per Bolt \( n_s = 2 \)  
Bolt Diameter \( D_b = 0.75 \text{ in} \)  
Slotted Hole Size \( s = (90 \text{ mm}, 25 \text{ mm}) \)  
Slot End Distance \( E_n = 100 \text{ mm} \)  
Slot Spacing \( S p = 150 \text{ mm} \)

Splice Tubes

<table>
<thead>
<tr>
<th>TOP</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>5mm Bent Plate</td>
<td>5mm Bent Plate</td>
</tr>
</tbody>
</table>

Depth (Horizontal)

\( D_{stt} = 133 \text{ mm} \)  
\( D_{sbt} = 136 \text{ mm} \)

Width (Vertical)

\( W_{tt} = 82 \text{ mm} \)  
\( W_{sbt} = 60 \text{ mm} \)

Thickness (Wall)

\( T_{stt} = 5 \text{ mm} \)  
\( T_{sbt} = 5 \text{ mm} \)

Area

\( A_{stt} = (2D_{tt} + 2W_{tt} - 4T_{tt})T_{tt} \)  
\( A_{sbt} = (2D_{sbt} + 2W_{sbt} - 4T_{sbt})T_{sbt} \)

\( A_{stt} = 3.17 \text{ in}^2 \)  
\( A_{sbt} = 2.834 \text{ in}^2 \)
Calculations: All references are from AASHTO LRFD 2nd Edition 1998 unless otherwise noted

Check Plastic Bending Between Posts:
(aka - Single Span Failure Mode)

Transverse Load:
\[ F_t := 54 \text{kip} \]
Distributed Length:
\[ L_t := 3.5 \text{ft} \]
Longitudinal Load:
\[ F_r := 18 \text{kip} \]

Flexure Resistance Factor
\[ i_f := 1.0 \text{ sec. s/s/s} \]

Clear Spacing Between Posts:
\[ CLZS - w_p \quad CL = 110.2 \text{el} \]

Top Tube Plastic Moment:
\[ M_{ptt} := Z_{tt} \cdot F_y t \quad M_{ptt} = 42 \text{kip-ft} \]

Bottom Tube Plastic Moment:
\[ M_{pbt} := Z_{bt} \cdot F_y t \quad M_{pbt} = 29 \text{kip-ft} \]

Total Tube Plastic Moment:
\[ M_p := M_{ptt} + M_{pbt} \quad M_p = 71 \text{kip-ft} \]

Total Ultimate Resistance (i.e. nominal resistance of the railing):
Derived from Eq. A13.3.2-1 for a single span failure mode with plastic hinges at edge of posts

\[ R_s := e F_y \& \quad R_s = 76\% \]

Resultant Location:
\[ Y_{bar} := M_{ptt} H_t \quad Y_{bar} = 25.4\% \]
Check Post:

**Sending Capacity at the base**

- **Flexure Resistance Factor**
  
  - *f* = sec. 6.5.5

- **Plastic Moment Capacity**
  
  \[ M_{ppx} := F_{yp} z_{px} \]
  \[ M_{px} = 92k_{ip} ft \]

- **Moment Arm**
  
  \[ \text{Ann} := Y_{bar} - H_{pl} \]

**Transverse**

**Longitudinal**

- \[ M_{ppx} = 92k_{ip} ft \]
- \[ M_{ppy} = 60k_{ip} ft \]

Modelled as frame sideways with rail remaining horizontal

- \[ \text{arm} := h \bar{y} - \text{tc-t pl-w bt 0.5} \]

- \[ \text{Arm} = 18.837\% \]

- \[ \text{Arm} = 5.837\% \]

**Point Load due to Post Bending Capacity:**

- \[ P_{bend x} = \frac{M_{ppx}}{\text{Arm}} \]
- \[ P_{bend y} = 12\% \]

**Anchor Capacity**

- **Concrete Bearing Resistance Factor**
  
  \[ \$ b := 1.00 \text{ sec. 5.5.5} \text{ and } 5.5.4.2 \]
  \[ \phi b := 1.0 \text{ sec. 5.5.5 and Tbl. 6.5.4.6 for rail comparison) ,} \]

- **Bolt Tension Resistance Factor**
  
  \[ A b := \frac{A b}{4} = 0.601 + I^* \]

- **Bolt Area**
  
  \[ T \bar{y} := 110\text{eip} \]
  \[ T \bar{x} = 55\text{eip} \]

- **Concrete Compression Block**
  
  Derived Eq. 5.7.5

**Assumes:**

- \[ a_x = \frac{T_{ux}}{\phi b 0.85 - f_{c}^* - 2W \text{ plx}} \]
- \[ a_y = \frac{T_{uy}}{\phi b 0.85 - f_{c}^* - 2W \text{ ply}} \]
- \[ ax = 0.991 \text{in} \]
- \[ ay = 0.65 \text{in} \]

Point Load due to **Anchor Capacity**

- \[ \text{Anchor x} := \frac{T_{ux} (d \text{ plx} - \frac{a_x}{2})}{Y_{bar} - H_{c}} \]
- \[ \text{Anchor y} := \frac{T_{uy} (d \text{ ply} - \frac{a_y}{2})}{Y_{bar} - H_{c}} \]

- \[ \text{Anchor x} = 50\text{kip} \]
- \[ \text{Anchor y} = 36\text{kip} \]
Check Post (Continued):

**Ultimate Load Resistance of a Single Post**

Controlling Post Capacity

\[
\text{Post}_x := \left[ \text{Phend}_x \times \text{Anchor}_x \right] \quad \text{Post}_y := \left[ \text{Phend}_y \times \text{Anchor}_y \right]
\]

\[P_{px} = \min \left( \text{Post}_x \right)\quad P_{py} = \min \left( \text{Post}_y \right)\]

\[P_{px} = 50 \text{kip}\quad P_{py} = 12 \text{kip}\]

**Check Load Capacity @ Post using Combined Post and Tube Strength:**

(aka - Two Span Failure Mode)

\[\text{Pic2} := \text{READBMP} (\text{"Two.bmp"})\]

Total Tube Plastic Moment Capacity:

\[M_p = 71 \text{kip-ft}\]

Clear Distance for Two Post Spacings:

\[CL2 := 2.8 - W_p \quad CL2 = 228.346 \text{ in}\]

Combined Capacity

Derived from Eq. A13.3.2-2 for a two span failure mode with plastic hinges at edge of posts

\[R_2 := P_{px} + \frac{16 \cdot M_p}{2 \cdot CL2 - L_t}\]

\[R_2 = 83 \text{kip}\]

\[\text{if} \left( R_2 > F_t \text{, "OK" }, \text{"LOW"} \right) = \text{"OK"}\]

LOW two span span failure mode capacity would mean that a three or more span failure mode would have to be used to achieve the required transverse capacity.
Check Double Bolted Splice:

Splice Bolt Area

\[ A_{sb} := \frac{D_b Z}{4} \]

\[ A_{sb} = 0.442 \text{in}^2 \]

Bolt Factored Shear Capacity assumes: Anchor and Splice Bolts have the same Uniaxle Strength

Sec. 6.5.5 and Tbl. 6.5.42 (set at 1.0 for rail comparison)

\[ \beta := 1.0 \]

Eq. 6.13.2.7-1 as modified by C6.3.E,

\[ R_{rs} := 2.2 \cdot b \cdot s \cdot (0.6 \cdot \beta \cdot b) \]

\[ R_{rs} = 254\% \]

Tube Bolt Factored Bearing Capacity

Sec. 6.5.5 and Tbl. 6.5.4.2 (set at 1.0 for rail comparison)

\[ \beta := 1.0 \]

Eq. 6.13.2.9-1 as modified by C6.13.2.7

Also compared to AISC LRFD 1993 Eq. J3-1b which is applicable when deformation around the bolt holes is not a design consideration

Tube \[ R_{trb} := Q_{bb} \cdot N_{Ob} \cdot N_{stt} + 3.0 \cdot D_b \cdot (T_{tt} + T_{bt}) \cdot F_{ut} \]

\[ R_{trb} = 2940 \text{kip} \]

Splice \[ R_{srb} := \beta \cdot 80 \cdot b \cdot N \cdot [3.0. D_b \cdot (T_{stt} + T_{bt}) \cdot F_{ut}] \]

\[ R_{srb} = 206\% \]

\[ R_{rb} := \min (R_{trb}, R_{srb}) \]

\[ R_{rb} = 206 \text{kip} \]

Tube Tensile Resistance

sec. 6.5.5

sec. 6.13.5.2

Eq. 6.8.2.1-1 Gross Section Yield

Tube \[ P_{ug} := +y \cdot F_{yt}(A_a + \beta t) \]

\[ P_{ug} = 446 \text{kip} \]

Splice \[ P_{srg} := y' \cdot F_{yt}(A_{stt} + A_{stt}) \]

\[ P_{srg} = 279 \text{kip} \]

\[ P_{rg} := m_4(p_{trg} p_{srg}) \]

\[ P_{rg} = 279 \text{kip} \]
Check Double Bolted Splice (Continued):

Tube Tensile Resistance

\[ A_{mc} = 0.85(A_{t} + A_{bt}) \]
\[ A_{tn} = q[A_{mc} bmx] \]

Splice

\[ A_{sn} = 0.85(A_{st} + A_{bt}) \]
\[ A_{sn} = mqp sncalc '4 S'"'mx 1) \]
\[ A_{s} = min([A_{t}, A_{tn}]) \]
\[ P = 299eip \]

Splice Capacity

\[ R_r = \text{in}([\% Rrb prg pm]) \]
\[ R_r = 206 \text{kip} \]

Splice strength greater than or equal to half the tube gross tension is a recommendation from the 1989 AADHTO Guide Specification for Bridge Railings with 1992 revisions.
constants:

\[ \text{psi} = 1 \, \text{in}^2 \]
\[ \text{ksi} = 1000 \, \text{psi} \]
\[ \text{kip} = 1000 \, \text{lb} \]
\[ \text{klfs} = 1000 : \]

\text{AITOW-READBMP}( \text{“Amw.bmp”} )
Given:

**Rail Height:** H := 33 in (Before Future Overlay)

**Curb:**
- Height: Hc := 11.3 in (At Post Center Line)
- Concrete: fc := 4.35 ksi

**Post:** W200x27 (W8x18 ASTM A572)
- Spacing: S := 10 ft
- Yield Strength: EYP := 50 ksi
- Plastic Modulus: zPX := 17.0 in
- Width: Wp := 5.25 in
- Base Plate Thickness: TPi := 0.75 in
- Width: Wn := 5 in
- Thickness (Wall): t := 0.16
- Area: Aa := 5.61 in²
- Plastic Modulus: Zn := 9.70 in³

**Anchor Bolts I" H.S.:**
- Ultimate Strength: Fua := 120 k.G
- Number: No := 2
- Diameter: Ds := 1 in

**Tubes:**
- Top: 127x127x7.9 (5x5x5/16)
- Bottom: 127x127x7.9 (5x5x5/16)
- Height from Roadway: Ht := 30.5 in
- Depth (Horizontal): Dtt := 5 in
- Width (Vertical): Wn := 5 in
- Thickness (Wall): t := 0.16
- Area: Aa := 5.61 in²
- Plastic Modulus: Zn := 9.70 in³

**Tube Splice:**
- Number of Bolts: No := 2
- Single Shear Planes per Bolt: N := 2
- Bolt Diameter: Db := 1 in
- Slotted Hole Size: SlotLength := 1.375 in
- SlotWidth := 1.125 in
- Slot End Distance: End := 4 in
- Number of Slips Before Splice Bolts are in Bearing: Nsh := 4
- Slot Spacing: Spacing := 7 in

**Post I Tube Connection:**
- Slotted Hole Size: SlotLength := 1.5 in
- SlotWidth := 1 in
- Anchor Diameter: Anchor := 0.75 in
- Anchor Sin Diameter: Anhrn := an in

Values taken from AISC 9th Edition ASD

Cold Formed ASTM A-500 Grade B

Yield Strength: Fyt := 46 ksi
Minimum Tensile Strength: Fnt := 5% ksi
Calculations: All references are from AASHTO LRFD 2nd Edition 1998 unless otherwise noted.

Check Plastic Bending Between Posts:
(aka - Single Span Failure Mode)

Pictl := READBMP("One.bmp")

Transverse Load:  \( F_t := 54 \text{kip} \)
Distributed Length:  \( L_t := 3.5 \text{ft} \)
Longitudinal Load:  \( F_1 := 18 \&p \)
Flexure Resistance Factor:  \( \& f := 1.0 \)
Clear Spacing Between Posts:  \( cL := s - wp \)
Top Tube Plastic Moment:  \( M_{ptt} := Z_{tt} \cdot F_{yt} \)
Bottom Tube Plastic Moment:  \( M_{pbt} := Z_{bt} \cdot F_{yt} \)
Total Tube Plastic Moment:  \( M_p := M_{ptt} + M_{pbt} \)

Total Ultimate Resistance (i.e. nominal resistance of the railing):

Derived from Eq. A13.3.2-1 for a single span failure mode with plastic hinges at edge of posts.

\[
R_I = \frac{16 \cdot M_p}{2CL - L_t}
\]

\( R_I = 76 \text{kip} \)

\[
\text{if} \left( R_I > F_t, "OK", "LOW" \right) = "OK"
\]

LOW Single Span Failure Mode Capacity means a two or more span failure mode would have to be used to achieve the required transverse capacity.

Resultant Location:

\[
Y_{bar} := \frac{M_{ptt} \cdot H_{tt}}{M_{pbt} \cdot H_{bt}}
\]

\( Y_{bar} = 24.875 \text{in} \)
Check Post:

**Sending Capacity at the base**

Transverse

Longitudinal

\[ Q_f = 1 \]

\[ \text{sec. 6.55} \]

\[ M_{PPX} = F_y p_z x \]

\[ M_{PPY} = F_{YP}.2 \text{PY} \]

\[ M_{ppx} = 71 \text{ kip.ft} \]

\[ M_{ppy} = 19 \text{ kip.ft} \]

\[ \text{Arm} = Y_{bar} - H - T_p, \text{ Arm} = 12.825\text{el} \]

Moment Arm

Point Load due to Post Bending Capacity:

\[ \text{Phend } x = \frac{M_{PPX}}{\text{Arm}} \]

\[ \text{Phend } y = \frac{M_{PPY}}{\text{Arm}} \]

\[ \text{Phend } x = 66\% \]

\[ \text{Phend } y = 18\text{ kip} \]

**Anchor Capacity**

Concrete Bearing Resistance Factor

\[ \phi_b := 1 \text{.0} \]

\[ \text{sec. 5.5.5 and 5.5.4.2 (Set at } 1.0 \text{ for all comparison)} \]

Bolt Tension Resistance Factor

\[ \phi_t := 1 \text{.0} \]

\[ \text{sec. 6.5.5 and Tab. 6.5.4.2 (Set at } 1.0 \text{ for all comparison)} \]

Bolt Area

\[ A_b := \frac{D_a}{4} \]

\[ A_b = 0.7\times 5\%? \]

Bolt Tension

\[ T_{ux} := N_{o}, \times 0.76 \times A_b \times F_{u}. \]

\[ T_{ux} = 143 \times \text{kip} \]

\[ T_{uy} = 72\% \]

Concrete Compression Block

Derived Eq. 5.752

\[ \text{Assumes: } \sqrt{(\text{concrete area/steel plate area}) \times 2} \]

\[ \text{Effect of base plate bending is neglected.} \]

Point Load due to Anchor Capacity

\[ T_{ux} := N_{o}, \times 0.76 \times A_b \times F_{u}. \]

\[ T_{ux} = 143 \times \text{kip} \]

\[ T_{uy} = 72\% \]

\[ a_x := 2.422 \times \text{kip} \]

\[ a_y = 0.969\% \]

\[ a_y = 0.969\% \]

\[ \text{Anchor } x := \frac{T_{ux}}{Y_{bar} - H,} \]

\[ \text{Anchor } y := \frac{T_{uy}}{Y_{bar} - H,} \]

\[ \text{Anchor } x = 60\% \]

\[ \text{Anchor } y = 29\% \]

Ultimate Load Resistance of a Single Post with the load located at \( Y_{bar} \) above the deck:

Controlling Post Capacity

\[ \text{Post } x := [\text{Phend } x \times \text{Anchor}] \]

\[ \text{Post } y := [\text{Phend } y \times \text{Anchor}] \]

\[ P_{px} := \min(\text{Post } x) \]

\[ P_{py} := \min(\text{Post } y) \]

\[ P_{px} = 60\% \]

\[ P_{py} = 18\text{ kip} \]
Check Load Capacity @ Post using Combined Post and Tube Strength:
(aka -Two Span Failure Mode)

Total Tube Plastic Moment Capacity: \( M_p = 74.367 \text{ kip-ft} \)

Clear Distance for Two Post Spacings: \( CL_2 := 2.5 \times WP \quad CL_2 = 234.75\% \)

Combined Capacity

Derived from Eq. A13.3.2.2 for a two span failure mode with plastic hinges at edge of posts

\[
16M_p \leq R_2 := PPx + 2(C - 2) - L
\]

\( R_2 = 93 \text{ kip} \)

\( \text{if} \left( R_2 > F, "OK","LOW" \right) = "OK" \)

LOW Two Span Failure Mode Capacity means a three or more span failure mode would have to be used to control the transverse capacity.
Check Splice:

Splice Bolt Area

\[ A_{sb} := \pi \frac{D_b^2}{4} \]

\[ A_{sb} = 0.785\text{in}^2 \]

Bolt Factored Shear Capacity assumes: Anchor and Splice Bolt have the same Ultimate Strength

Sec. 6.55 and Tbl. 654.2 (Set at \( \theta \) for rail comparison) \( \gamma := 1.0 \)

Eq. 6.13.2.7-1 as modified by C6.1-2.7

\[ R_{rs} := \frac{2}{3} \cdot \text{Nob. N.} \cdot (0.6 \cdot F_t) \cdot A, \cdot b \]

\[ R_{rs} = 452\% \]

Tube Bolt Factored Bearing Capacity

Sec. 6.63 and Tbl. 654.2 (Set at \( \theta \) for rail comparison) \( \gamma := 1.0 \)

Eq. 6.13.2.9-1 as modified by C6.13.2.7

Also compared to AISC LRFD 1993 Eq. 6.13b.5 which is applicable when deformation around the bolt holes is not a design consideration

\[ R_{rb} := \gamma \cdot \text{bb.2. Nob. N.} \cdot (3.1 \cdot \text{Db. Tt. Fut}) \]

\[ R_{rb} = 435 \text{kip} \]

Tube Tensile Resistance

sec. 6.55

sec. 6.13.6.2

\( \gamma := 1.0 \)

\( \gamma := 1.0 \)

Eq. 6.8.2.1-1 Gross Section Yield

\[ P_{rg} := \gamma \cdot \text{Fyt} \cdot (\text{Att} + \text{Abt}) \]

\[ P_{rg} = 516\% \]

Eq. 6.8.2.1-1 Net section Fracture

\[ A_{calc} := \text{Att} \cdot 2 \cdot \text{SlotWidth} + 0.0625 \cdot h \cdot T \cdot r \]

\[ A_{calc} = 9.736\% \]

\[ A_{calc} := \text{Att} \cdot 2 \cdot \text{SlotWidth} + 0.0625 \cdot \text{in} \cdot T \cdot b \]

\[ A_{calc} = 9.537\text{in}^2 \]

Eq. 6.13.52 Tension Net Area for Splices

\[ A_{,} := 0.85 \cdot (A_a + A_b) \]

\[ A_{calc} = 9.537\text{in}^2 \]

\[ A_{,} = 9.537\text{in}^2 \]

\[ P_m := \gamma \cdot \text{Fut} \cdot A, \cdot U \]

\[ P_m = 553\text{kip} \]

\[ P_m := \gamma \cdot \text{Fut} \cdot A, \cdot U \]

\[ P_m = 553\text{kip} \]

if\( P_m \& P_r \& \text{"Yields"}, \"Fractures\" \) = \"Yields\"

Splice Capacity

\[ \text{Splice} = \left[ R \left\lfloor \frac{R_{rb}}{R} \right\rfloor \right] \]

\[ R_{,} := \min \text{(Splice)} \]

\[ R = 435 \text{kip} \]

Splice strength greater than or equal to Half the tube gross tension is a recommendation from the 1989 AASHTO Guide Specification for Bridge Railings with 1992 revisions.

\[ \left\lfloor \frac{R_{rg}}{2} \right\rfloor, \text{"OK"}, \text{"LOW"} \] = \"OK\"
Constants:

\[
\text{psi} = \frac{1 \text{ lb}}{\text{in}^2}
\]

ksi = 1000·psi

kip = 1000·lb

klf = 1000 \frac{\text{lb}}{\text{ft}}

Arrow = READBMP("Arrow.bmp")