U.S.Department
of Transportation

Federal Highway Administration

Memorandum

Subject: INFORMATION: Bridge Rail Analysis

Date: May 16, 2000

From: Frederick G. Wright, Jr.
Program Manager, Safety

Reply to
Attn. of: HSA-1

To: Resource Center Directors
Division Administrators

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.

Enclosure

July 21, 1998

COMPARISON OF THE COLORADO TYPE 10 BRIDGE RAIL TO THE WYOMING TL-4 RAIL

This is a comparison of the geometry, strength, and potential crash worthiness 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

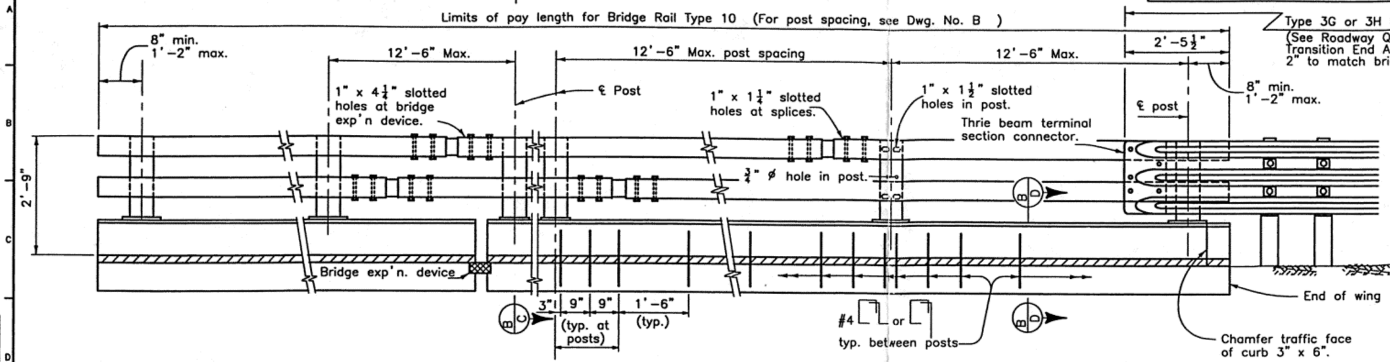
KIPS for the Wyoming rail and 78.9 KIPS for the Colorado rail. By way of comparison, the LRFD code recommends a strength to resist a load of 54 KIPS for the PL-2 load (assumed to be similar to the NCHRP 350 TL-4 load). Tensile effects will not significantly improve either of these strengths, because 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.

	TYPE 10	WY TL-4	IMPROVED TYPE 10
SINGLE-SPAN	38 KIPS	77 KIPS	78 KIPS
SINGLE-SPAN WITH TENSILE ACTION	77 KIPS @ 9.3″	77 KIPS	158 KIPS @ 9″
POST ONLY	62 KIPS	51 KIPS	55 KIPS
TWO-SPAN	79 KIPS	84 KIPS	94 KIPS



RAIL PANEL ON WING TERMINAL SECTION

(See Roadway plans for ends not attached to Guard Rail.)

RAIL PANEL AT EXPANSION DEVICE

ELEVATION – BRIDGE RAIL

RAIL PANEL ON WING TRANSITION SECTION

(See roadway plans for ends requiring attachment to guard rail.)

B-606-10

NOTES

All tubes shall be fabricated from ASTM A-500 Grade B. All posts and backing plates shall be fabricated from ASTM A-36 steel.

The above material and all anchor bolts and miscellaneous bolts, nuts, and washers shall be galvanized after fabrication in accordance with Section 509. Concrete, reinforcing steel, and structural steel elements shall conform to the requirements of Sections 601, 602 and 509, respectively.

Post anchor, encased in concrete, shall be ASTM A-36 or AASHTO M-169 steel and need not be galvanized.

The tubes shall be shop bent or fabricated to fit horizontal curve when radius is less than 1,500 feet.

Tubes shall be continuous over not less than two posts. No welded butt splices will be allowed in the tube sections.

The centerline of the posts shall be 2'-6" minimum and 3'-6" maximum from the centerline of the tube expansion splice, measured along the centerline of posts.

All bolts that have lock washers shall be tightened to snug only.

Posts shall be perpendicular to the longitudinal roadway grade.

One or more 12'-6" post spacings may be reduced (8'-4" min.) in order to maintain dimensions from the end of the wings and expansion joints.

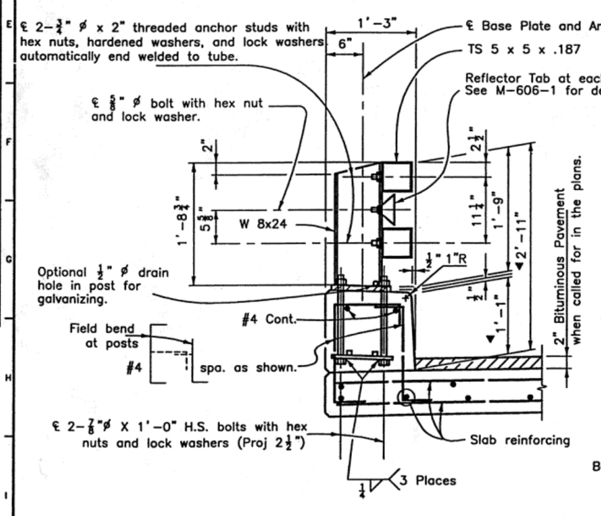
Optional drain hole for galvanizing may be drilled, punched, or clipped leaving smooth surfaces and transitions. No flame cutting or air carbon arc gouging is allowed.

Payment will be made under Item 606, Bridge Rail Type 10 for all posts, post anchors, base plates, backing plates, anchor bolts, miscellaneous bolts, nuts, washers, tubes, tube expansion devices, tube splices, end plates, curb concrete (Class D), curb reinforcing steel, and reflector tabs.

Prior to fabrication of this item, three sets of working drawings which comply with the requirements of Section 105, shall be submitted to the Department for information only. One set shall be sent to the Colorado Department of Transportation, Staff Materials Inspection Unit, 4340 E. Louisiana Avenue, Denver, Colorado 80222. Two sets shall be sent to the Engineer.

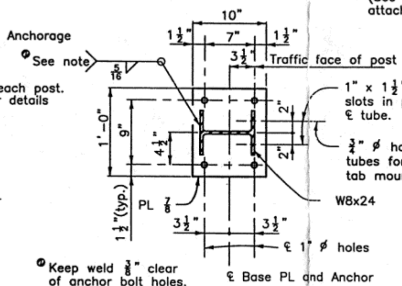
Structural Steel:
AASHTO M-183 (ASTM A-36) f_y=36,000 psi
Cold formed ASTM A-500 Grade B f_y=46,000 psi

For additional details see next rail sheets.



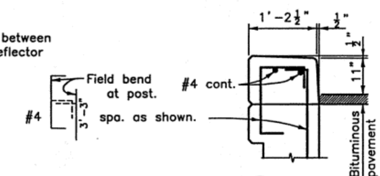
SECTION B/C

▼Subtract 2" when hot bituminous pavement is not used.



Keep weld ⅜" clear of anchor bolt holes.

PLAN – POST DETAIL

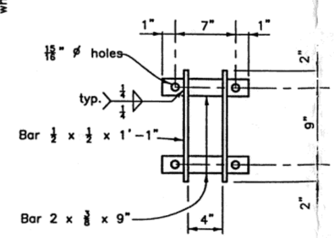


SECTION B/D

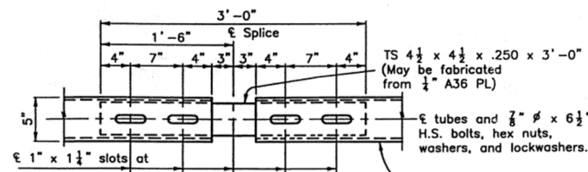
(Use when curb is to be placed on top of wingwall. See approach slab sheets for when the wingwall is outside the curb.)

INFORMATION ONLY

Description	Unit	Per Lin. Ft.
Structural Steel (Galvanized)	Lb.	33.1
Concrete Class D (Bridge)	Cu.Yd.	.05
Reinforcing Steel (Epoxy Coated)	Lb.	5.1



ANCHOR DETAIL



℄ 1" x 1½" slots at tube splice, and 1" x 4½" slots at bridge exp'n device. Slot both inner and outer tubes. Stagger top and bottom splices into different post spacings except at expansion joint, place at opposite ends of same post space. (Range of motion = 1'-0" at bridge. expansion device.)

PLAN – TUBE SPLICE

COLORADO DEPARTMENT OF TRANSPORTATION

BRIDGE RAIL TYPE 10
Attachment 2

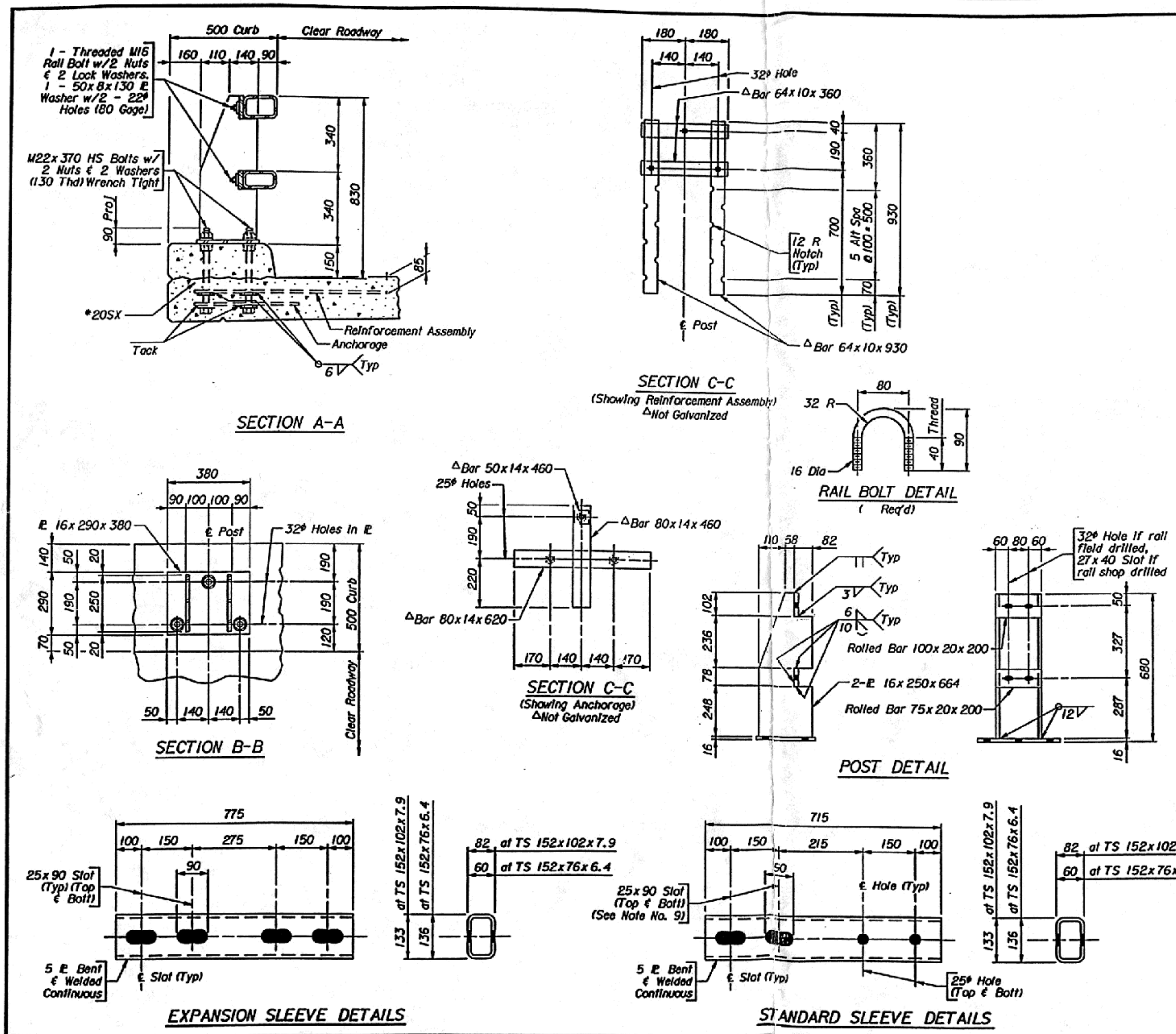
Designer		Structure	
Detailer		Numbers	
Drawing Number B	of	Drawings	

Revision Dates (Preliminary Stage Only)

11-90	11-91	3-92	8-94	8-95	2-96	1-97	6-98

Design	Designed By	Checked By	Detail	Detailed By	Checked By	Quantities	Quantities By	Checked By
INITIAL			INITIAL			INITIAL		
DATE			DATE			DATE		

06/25/1998 S:\B60610



MAXIMUM POST SPACING IS 3000 mm

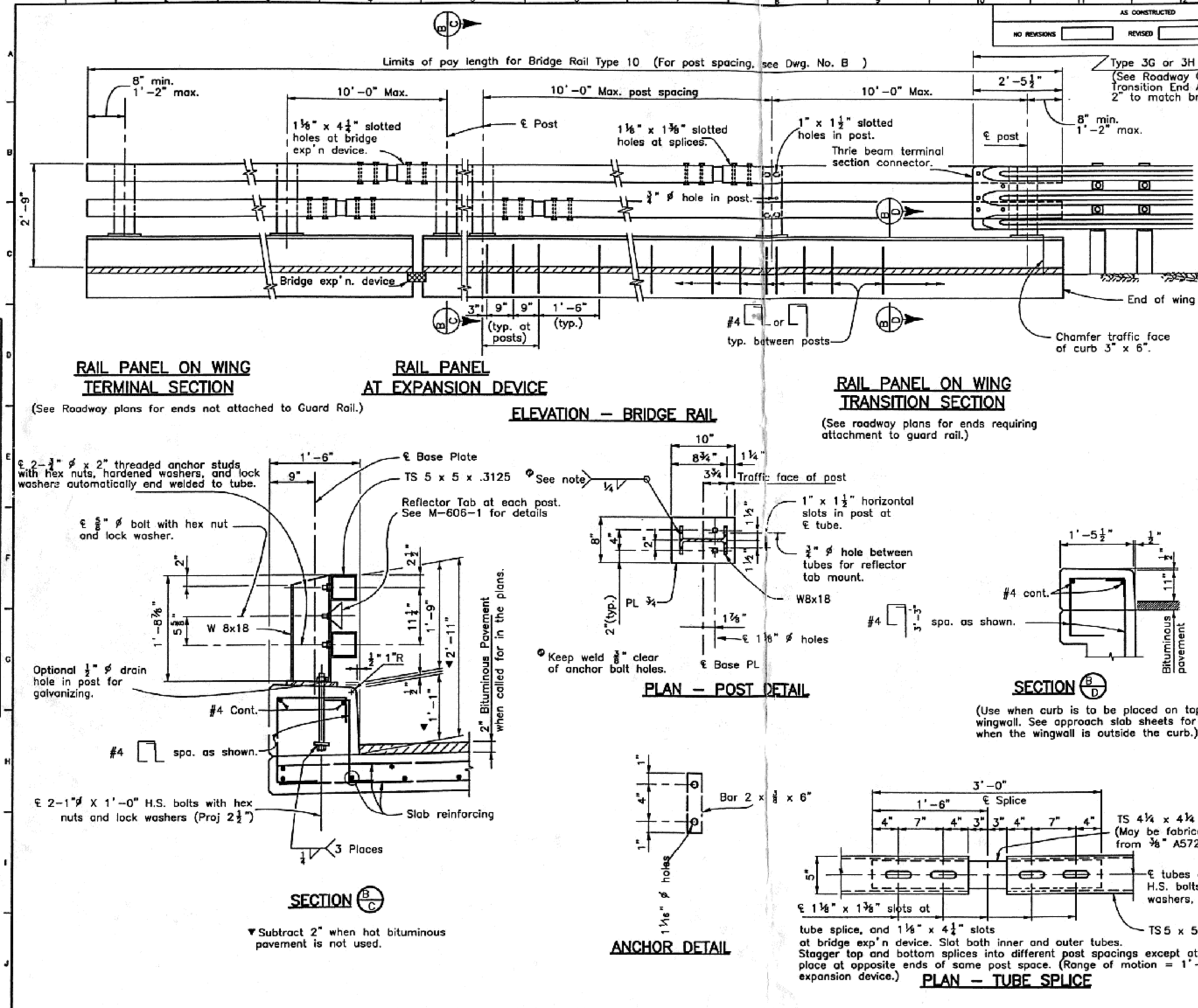
Note:
1) Venting and pick-up holes in rails and sleeves shall be shown on Fabricator's shop plans.
2) Anchor bolts may be tack welded to anchorage. (Shop or Field)
3) All rough edges on posts and rails shall be ground smooth.
4) Post base ℄'s shall be flat after fabrication.
5) Rails shall not be shop spliced.
6) Railing posts shall be in place and in proper alignment prior to placement of curb.
7) Rails shall be shop or field drilled 32ϕ 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 Subsection 501.43-Galvanizing.
9) Slots may be omitted in standard sleeves where bolts are required on one side of splice only.
10) For details of terminal types and location of Sections A-A, B-B and C-C, see Sheet No. .
11) In the areas indicated on the PLAN requiring an expansion splice, the expansion splice shall be located in the railing panel which passes over the bridge expansion joint.

WYOMING DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM

TL-4 BRIDGE RAILING DETAILS

Attachment 3

propirfl.dgn



B-606-10Z

NOTES

All tubes shall be fabricated from ASTM A-500 Grade B. All posts and backing plates shall be fabricated from ASTM A-572 steel.

The above material and all anchor bolts and miscellaneous bolts, nuts, and washers shall be galvanized after fabrication in accordance with Section 509. Concrete, reinforcing steel, and structural steel elements shall conform to the requirements of Sections 601, 602 and 509, respectively.

Post anchor, encased in concrete, shall be ASTM A-36 or AASHTO M-169 steel and need not be galvanized.

The tubes shall be shop bent or fabricated to fit horizontal curve when radius is less than 1,500 feet.

Tubes shall be continuous over not less than two posts. No welded butt splices will be allowed in the tube sections.

The centerline of the posts shall be 2'-0" minimum and 3'-0" maximum from the centerline of the tube expansion splice, measured along the centerline of posts.

All bolts that have lock washers shall be tightened to snug only.

Posts shall be perpendicular to the longitudinal roadway grade.

One or more 10'-0" post spacings may be reduced (6'-8" min.) in order to maintain dimensions from the end of the wings and expansion joints.

Optional drain hole for galvanizing may be drilled, punched, or clipped leaving smooth surfaces and transitions. No flame cutting or air carbon arc gouging is allowed.

Payment will be made under item 606, Bridge Rail Type 10 for all posts, post anchors, base plates, backing plates, anchor bolts, miscellaneous bolts, nuts, washers, tubes, tube expansion devices, tube splices, end plates, curb concrete (Class D), curb reinforcing steel, and reflector tabs.

Prior to fabrication of this item, three sets of working drawings which comply with the requirements of Section 105, shall be submitted to the Department for information only. One set shall be sent to the Colorado Department of Transportation, Staff Materials Inspection Unit, 4340 E. Louisiana Avenue, Denver, Colorado 80222. Two sets shall be sent to the Engineer.

Structural Steel:
AASHTO M-223 (ASTM A-572) f_y=50,000 psi
Cold formed ASTM A-500 Grade B f_y=46,000 psi

For additional details see next rail sheets.

INFORMATION ONLY

Description	Unit	Per Lin. Ft.
Structural Steel (Galvanized)	Lb.	44.2
Concrete Class D (Bridge)	Cu.Yd.	.06
Reinforcing Steel (Epoxy Coated)	Lb.	5.6

COLORADO DEPARTMENT OF TRANSPORTATION

IMPROVED BRIDGE RAIL TYPE 10

Attachment 4

Designer	M. McMullen	Structure Numbers	
Detailer	M. McMullen		
Drawing Number B	¼ of		Drawings

Revision Dates (Preliminary Stage Only)

11-90	11-91	3-92	8-94	8-95	2-96	1-97	6-98

AS CONSTRUCTED — NO REVISIONS / REVISED / VOID

FEDERAL ROAD REGION NO. VIII — DIVISION COLORADO — PROJECT NUMBER — SHEET NUMBER

REVISIONS

Design / Detail / Quantities — Designed By / Checked By / Initial / Date

07/20/1998 T:\MJM\STDS\B606102

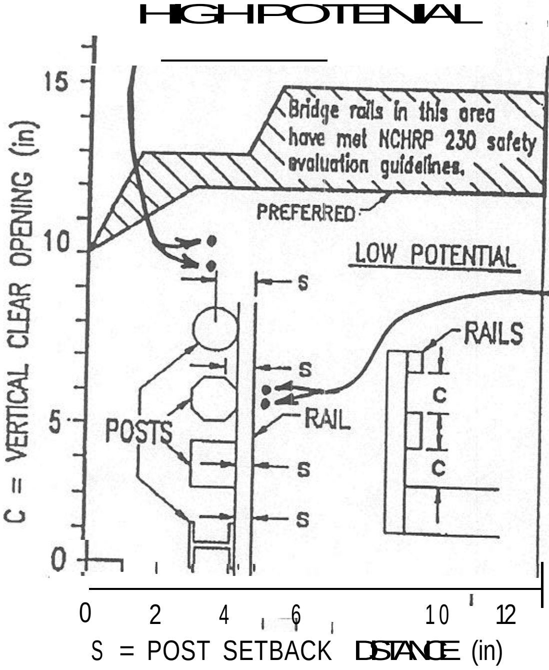


Figure A13.1 .l -2 - Potential for wheel, bumper or hood impact with post

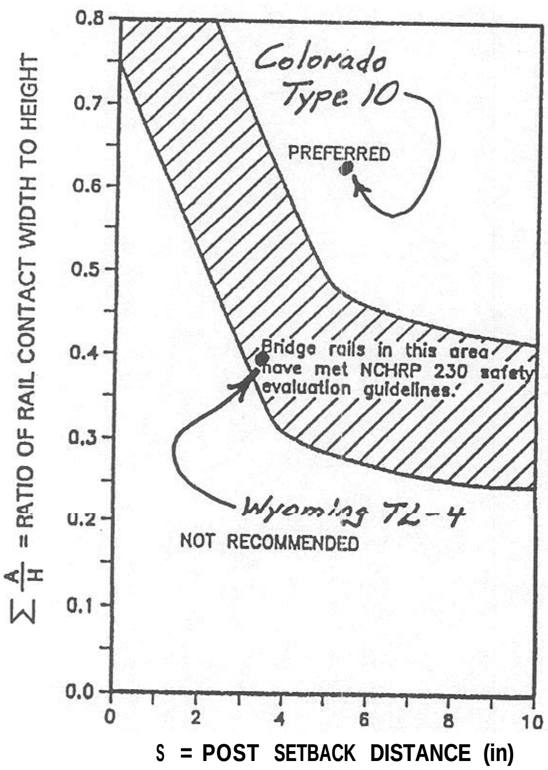


Figure A13.1.1-3 - Post Setback Criteria

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.

Colorado Type 10

WYOMING TL-4:

$$\frac{A}{H} = \frac{6'' \text{curb} + 3'' \text{tube} + 4'' \text{tube}}{33''}$$

$= 0.394$

$S = 3.54''$

OPENINGS = 9.37" & 10.39"

COLORADO TYPE 10:

$$\frac{A}{H} = \frac{11'' \text{curb} + 5'' \text{tube} + 5'' \text{tube}}{33''}$$

$= 0.636$

$S = 5''$

OPENINGS = 5¾" & 6¼"

ATTACHMENT 1

Given:

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

Curb:

Height H,:=ll.in (At Post Center Line)

Concrete fc =4.35-ksi

Post W2OOx36 AASHTO M-183 (W8x24 ASTM A-36)

			Transverse	Longitudinal
Spacing:	s := 12.5 .ft			
Yield Strength	FYP =36-ksi	Plastic Modulus	zPX :=23.2%?	zPY = 8.57 in3
Width	Wp ~6.495.i"			

Base Plate

Thickness:	$TPl := \frac{l}{8}.in$	Width	WPIX = 12 in	W ply = 10 in
		Depth to CL Bolts	d plx :=8.5.in	d ply = 10.5 in

Anchor Bolts 7/8" **H.S.**

Ultimate Strength	F ua = 120-ksi	Number	NO ax :=2	NO ay :=2
Diameter	$D_a := \frac{7}{8} \cdot in$			

Tubes

	Top	Bottom
	127x127x4.8 (5x5x3/1 6)	127x127x4.8 (5x5x3/1 6)
Height from Roadway	Htt:=30.5.in	H bt := 19.25.i"
Depth (Horizontal)	Dtt:=5,in	D bt:=5.in
Width (Vertical)	Wtt:=5.in	W bt:=54n
Thickness (Wall)	$T_{tt} := \frac{3}{16} \cdot in$	$T_{bt} := \frac{3}{16} \cdot in$
Area	A tt ~3.52.i"	A bt:=3.52.i"2
Plastic Modulus	**Z tt** := 6.29 .in3	Z bt = 6.29 .in3

Values Taken From AISC9th Edition ASD

Yield Strength FYt :=46,ksi Cold Formed ASTM A-500 Grade B

Minimum Tensile Strength F ut :=58ksi

Tube Splice

Number of Bolts No b :=2 Single Shear Planes per Bolt N, :=2

Bolt Diameter Db :=0.875.in Slotted Hole Size SlotLength S = 1.25.i" SlotWidth S := 1.0 in

Slot End Distance E n d :=4.in Number of Slips Before Splice Bolts are in Bearing N sb :=4

Slot Spacing Spacing :=7.in

Post I Tube Connection

Slotted Hole Size SlotLength := 1.5.i" SlotWidth := 1 .in

Anchor Diameter Anchor :=n 75.i" Anchor Slip Diameter Anrhnr s := 0.875·in

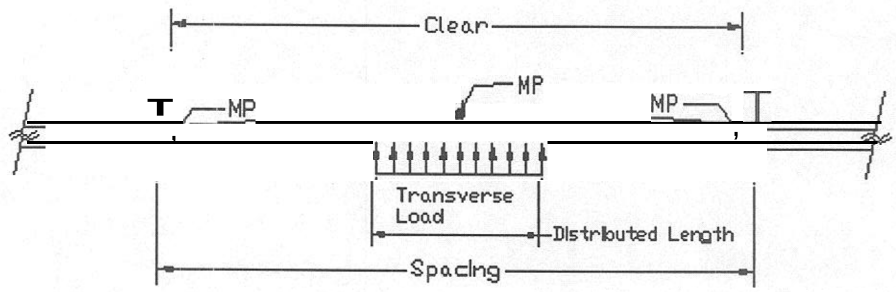
Shoulder of end welded Stud.

Calculations:

All references are from AASHTO LRFD 2nd Edition 1998 with 1999 Interims unless otherwise noted.

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

Pictl :=READBMP("one bump")



Transverse Load:	Ft :=54kip	Tbl. Al 3.2-i TL-4 (Test Level 4)
Distributed Length:	L_t :=3.5.ft	
Longitudinal Load:	Fl:=lS.kip	
Flexure Resistance Factor	$ f:=l.o	sec. 6.5.5
Clear Spacing Between Posts:	$CL := S - W_p$	CL = 143.505 om
Top Tube Plastic Moment:	$M_{ptt} := Z_{tt} \cdot F_{yt}$	M_{ptt} = 24•kip ft
Bottom Tube Plastic Moment:	$M_{pbt} := Z_{bt} \cdot F_{yt}$	M_{pbt} = 24eip.ft
Total Tube Plastic Moment:	Mp:="ptt+Mpbt	M_p =48*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.

$$R1 := \$f \frac{16.M_p}{2CL - L_t}$$

R_1 = 38•kip

if(R_1>F_t, "OK" , "LOW") = "LOW"

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:

$$Ybar := \frac{Mptt'Htt + Mpbt'Hbt}{MP}$$

Ybar = 24.875411

Check Post:

Bending Capacity at the base

	Transverse	Longitudinal
Flexure Resistance Factor	$\phi_f = 1$	Sec. 6.5.5
Plastic Moment Capacity	$M_{ppx} := F_{yp} \cdot z_{px}$	MPPY:=FYP.ZPY
	$M_{ppx} = 70\%\sim$ ft	$M_{ppy} = 26$okip.ft
Moment Arm	Arm:=Ybar-H,-Tpl	Arm=13%1
Point Load due to Post Bending Capacity:	$Pbend_x := \frac{M_{ppx}}{Arm}$	$Pbend_y$:= $\frac{M_{PPY}}{Arm}$
	$Pbend_x = 64\%\sim$	$Pbend_y = 24$okip

Anchor Capacity

Concrete Bearing Resistance Factor $\phi_b := 1.0$ Sec. 5.5.5and5.5.4.2 (Set at 1.0 for rail comparison)

Bolt Tension Resistance Factor $\phi_t := 1.0$ Sec. 6.5.5andTbl. 6.5.4.2 (Set at 1.0 for rail comparison)

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

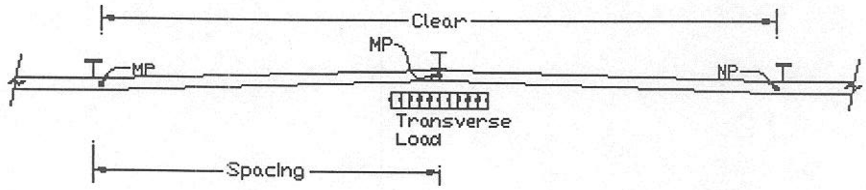
	Transverse	Longitudinal
Bolt Tension Eq. 6.13.2.10.2-1	$T_{ux} := No_{ax} \cdot \phi_t \cdot 0.76 \cdot A_b \cdot F_{ua}$	$T_{uy} := No_{ay} \cdot \phi_t \cdot 0.76 \cdot A_b \cdot F_{ua}$
	$T_{ux} = 110$ kip	$T_{uy} = 110$ kip
Concrete Compression Block Derived Eq. 5.7.5-2 Assumes: sqrt(concrete area/steel plate area) >= 2 Effect of base plate bending is neglected.	$a_x := \frac{T_{ux}}{\phi_b \cdot 0.85 \cdot f'_c \cdot 2 \cdot W_{plx}}$	$a_y := \frac{T_{uy}}{\phi_b \cdot 0.85 \cdot f'_c \cdot 2 \cdot W_{ply}}$
	$a_x = 1.236\%$	$a_y = 1.483$in
Point Load due to Anchor Capacity	$Anchor_x := \frac{T_{ux} \cdot \left(d_{plx} - \frac{a_x}{2}\right)}{Ybar - H,}$	$Anchor_y := \frac{T_{uy} \cdot \left(d_{ply} - \frac{a_y}{2}\right)}{Ybar\ H,}$
	$Anchor_x = 62$ kip	$Anchor_y = 77$ kip

Ultimate Load Resistance of a Single *Post* with the load located at Ybar above the deck:

	Transverse	Longitudinal
Controlling Post Capacity	$Post_x := [Pbend_x\ Anchor_x]$	$Post_y := [Pbend_y\ Anchor_y]$
	$P_{px} := \min(Post_x)$	$P_{py} := \min(Post_y)$
	$P_{PX} = 62$ 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: $M_p = 48.223 \cdot kip \cdot ft$

Clear Distance for Two Post Spacings: $CL2 := 2 \cdot S - W_p$ $CL2 = 293.505 \cdot in$

Combined Capacity

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

$$R_2 := P_{px} + \frac{16 \cdot M_p}{2 \cdot CL2 - L_t}$$ $R_2 = 79 \cdot kip$

$if(R_2 > F_t, "OK", "LOW") = "OK"$

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 thatn the tube so that the tube controls the splice strength.**

Splice Bolt Area Ash := Db2 x / 4 A sb = 0.601.i"

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

sec. 6.5.5 and Tb,. 6.5.4.2 (Set at 1.0 fpr rail comparison, $\phi_s := 1.0$

Eq.6.13.2.7.fasmodifiedbyC6.13.2.7

R rs:=2.Nob.~ s.0.6.Asb.Fua.N, R,, = 346 kip

Tube Bolt Factored Bearing Capacity

Sec. 65.5 and Tbl. 6.5.4.2 (Set at 1.0 for rail comparison) $\phi_{bb} := 1.0$

Eq. 6.13.2.9-f as modified byC6.13.2.7
Also compared to AISC LRFD 1993 Eq. J3-Ib which is applicable when deformation around the bolt holes is not a design consideration

R rb:=Nob.N,.c$ bb.3.0.D,,.(Ttt+Tbt).Fut R rb = 228 kip

Tube Tensile Resistance

Sec. 6.5.5 $\phi_y := 1.0$ $\phi_u := 1.0$

sec. 6.13.5.2 U := 1.0

Eq. 6.8.2.1-f Grass Section Yield

$P_{rg} := \phi$ yQ(Att+"bt) $P_{rg} = 324 \cdot kip$

Eq. 6.8.2.1-1 Met Section Fracture

A ncalc :=Att- 2(SlotWidth ,+ 0.0625in) T tt ,,,
+ A bt- 2(SlotWidth ,+ 0.0625.in) .T bt $A_{ncalc} = 6.243 \cdot in^2$

Eq. 6.13.5.2 Tension Net Area for Splices

A ,,:=0.85(An+Abt) $A_{nmax} = 5.984 \cdot in^2$

A " :=if(A*calc<Anmax.Anca*c,Anmax) $A_n = 5.984 \cdot in^2$

P m:=$ ".Fut.AI,.U P,=347eip

if$(P_{rn} > P_{rg}$, "Yields" , "Fractures"$)$ = "Yields"

Splice Capacity

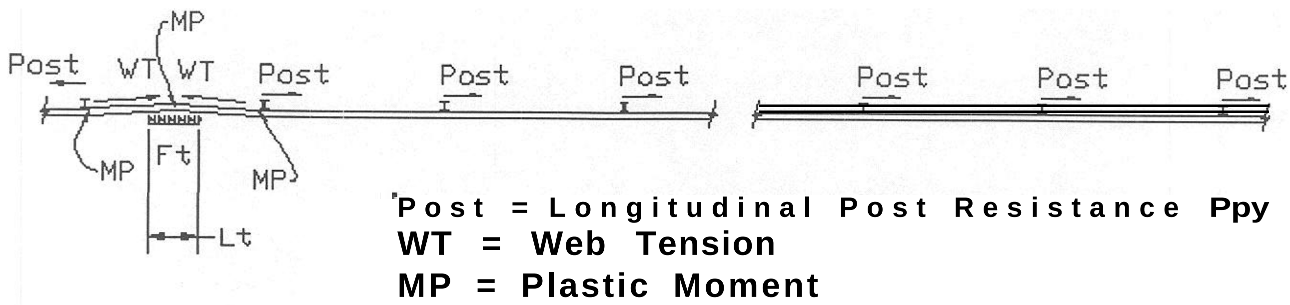
$Splice := [R_{rs} \; R_{rb}]$ $R_r := \min(Splice)$ $R_r = 228 \cdot kip$

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.

if$\left(R_r > \frac{P_{rg}}{2}, \text{"OK"}, \text{"LOW"}\right)$ = "OK"

Check Mixed Plastic and Tension Field Between Posts:

Pict3 :=READBMP("Tension.bmp")



- **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 comer radii is neglected.*

WT:=[2,Ttt.(Dtt-2.Ttt F
+[2.T $_{bt}$ ·(D $_{bt}$ -2.&j;- $_{Yt}$ | WT = 160%~

Flange Plastic Couple

$$M_{pf} := \left[W_{tt} \cdot T_{tt} \cdot F_{yt} \cdot \left(D_{tt} - T_{tt}\right)\right] \dots + W_{bt} \cdot T_{bt} \cdot F_{yt} \cdot \left(D_{bt} - T_{bt}\right)$$

M pf = 35%~ .ft

Equivalent Load

$$Pf := \frac{Mpf.8}{CL- L ,.0.5}$$

Pf=27%p

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.

$$N_{post} := \mathrm{ceil}\left(\frac{WT}{P_{py}}\right)$$

Npost =7

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.

Post/Tube

$$Slip_t := \frac{SlotLength}{2} - \frac{Anchor}{2}$$

Slip t =0.32 in

Splice

$$Slip_s := Nsb. \left(\frac{SlotLength\ S}{2} - \frac{D\ b}{2}\right)$$

Slip s =0.75*1x

Predicted Total Slip to Achieve Web in Tension
Assuming 40 ft Between Splices and an Impact Midway Between Two Splices.

$$Slip := Slip_t + Slip_s \cdot \left[1 + \mathrm{floor}\left[\frac{\left[\left(N_{post} - 1\right) \cdot S + 0.5 \cdot S\right] - 20 \cdot ft}{40 \cdot ft}\right]\right]$$

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.**

$2 \cdot P_{px} = 125 \cdot kip$

$if(2 \cdot P_{px} > F_t, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

Tube with Web in Tension

WI = 160 kip

Splice Resistance

R r = 228.375 kip

$if(R_r > WT, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

A :=2PX -Pf [C l - Lt/2]

2.wT

A = 18.715+

Length change of tube

$\Delta_t := \sqrt{\Delta^2 + [(CL - L_t \cdot 0.5) \cdot 0.5]^2} - (CL - L_t \cdot 0.5) \cdot 0.5$

A t = 2.7956n

$if(\Delta_t > Slip, \text{"OK"}, if(Slip - \Delta_t < Slip_t, \text{"Maybe OK"}, \text{"Still Slipping"})) = \text{"OK"}$

Constants:

$psi \equiv 1 \cdot \frac{lb}{in^2}$

$ksi \equiv 1000 \cdot psi$

kip=lOOO4b

klf=1000-lb / ft

Arrow=Readbmp(“Amw.bmp”)

Given:

Rail Height:	H:=830 mm	(Before Future Overlay)			
Curb:					
Height	H, := 15Omm	(At Post Center Line)			
Concrete	fc :=4.35ksi	(Assumed)			
Post	2 - 16mm x 250mm Plates			Transverse	Longitudinal
Spacing:	s :=3000 mm				
Yield Strength	F yp := 36,ksi (assumed)		Plastic Modulus	zPX :=30.51.in3	Zpy:=1.95+?
Width	wp :=2oo.mm		Plates at Base	PLt:=16mm	PL, :=25omm
			Plates at 1 st Rail	PLtr:=16mm	PLir:=16Xmm
Base Plate					
Thickness:	Tpl := 16.m.m		Width	Wplx :=3XOmm	W ply :=29Omm
			Depth to CL Bolts	d plx :=24Omm	dply:=330mm
Anchor Bolts	M22 H.S.				
Ultimate Strength	F,, := lZO.ksi		Number	NO=:=2	NO ay:=l
Diameter	Da :=?.in 8				

Tubes	TOP	Bottom	
	152x102x7.9 (6x4x5/1 6)	152x76x6.4 (6x3x1/4)	
Height from Roadway	Htt:=779mm	H bt :=452mm	
Depth (Horizontal)	Dn:=6.in	D bt:=6h	
Width (Vertical)	Wtt:=4.in	W bt:=3~in	
Thickness (Wall)	$T_{tt} := \frac{5}{16} \cdot in$	T bt :=$-in	
Area	A tt := 5.61 .in*	A bt :=4.09.inz	Values taken from AISC 9th Edition ASD
Plastic Modulus	Z tt := 10.9O.iJ	Z bt :=7.62.in3	

Yield Strength FYt ::46ksi Cold Formed ASTM A-500 Grade B (Assumed)

Minimum Tensile Strength F Ut :=58ksi

Given:

Double Bolted Tube Splice

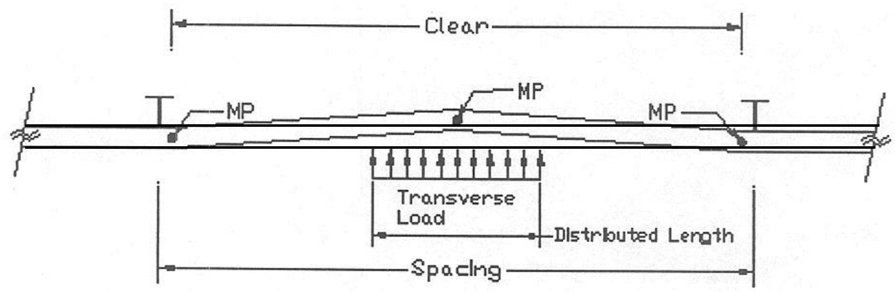
Number of Bolts	Nob :=2	Single Shear Planes per Bolt	N,:=Z
Bolt Diameter	Db :=0.75.in	Slotted Hole Size	SlotLength s :=9o.mm SlotWidth s := 25 .mm
Slot End Distance	End := 100.m		
Slot Spacing	Spacing := 15Omm		

	TOP	Bottom
Splice Tubes	5mm Bent Plate	5mm Bent Plate
Depth (Horizontal)	D stt := 133'mm	D sbt := 136mm
Width (Vertical)	W ,tt:=82mm	W sbt :=6Omm
Thickness (Wall)	T stt := 5 'mm	T s~t:=5mm
Area	Astt:=(2.D,tt+2.W,tt-4.T,tt).T,tt	$A_{sbt} := \left(2 \cdot D_{sbt} + 2 \cdot W_{sbt} - 4 \cdot T_{sbt}\right) \cdot T_{sbt}$
	A Stt = 3.17@inz	A sbt = 2.SS34n2

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:	Ft :=54kip	Tbl. A13.2-1 TL-4 (Test Level 4)
Distributed Length:	L t :=3.5.ft	
Longitudinal Load:	F, :=18kip	
Flexure Resistance Factor	i$ f:= 1.0	sec. s.s.s
Clear Spacing Between Posts:	CLZS- wp	CL = 110.2el
Top Tube Plastic Moment:	M ptt :=Ztt.Fyt	M ptt = 42*kip,ft
Bottom Tube Plastic Moment:	M pbt :=Z bt.Fyt	M pbt = 29 okip .ft
Total Tube Plastic Moment:	Mp:=“ptt+Mpbt	M p = 71 ekip .ft

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

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

16.M
R,:=efy&
t

R , = 76%~

if(R 1>F t, "OK" , "LOW") = "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:

Ybar := M ptt’H t; r pbCH bt

Ybar = 25.4%

Check Post:

Sending Capacity at the base Transverse Longitudinal

Flexure Resistance Factor +f=' sec. 6.5.5

Plastic Moment Capacity M ppx :=Fyp.zpx M PPY :=PYP.ZPY

M ppx = 92Qip.ft Mppy==60kip.ft

Moment Arm Ann:=Ybar-H,-Tpl

Modeled as frame sideway with rail remaining horizontal

arm :=h by- tc-t pl-w bt 0.5

$$\text{Arm}_l := \text{Arm}_{lr} \cdot \frac{PL_l}{PL_l + PL_{lr}}$$

Arm = 18.837% Arm = 5.837%l

Point Load due to Post Bending Capacity:

$$\text{Pbend}_x := \frac{M_{PPX}}{\text{Arm}}$$

Pbend y := z

Pbend x = 58%~ Pbend y = 12%~

Anchor Capacity

Concrete Bearing Resistance Factor '$ b := 1 0.0 Sec. 5.5.5and5.5.4.2 (set at 1.0 for rail comparison)

Bolt Tension Resistance Factor $\phi_t := 1.0$ Sec. 6.5.5and Tbl. 6.5.4.2 (Set at 1.0 for rail comparision)

Bolt Area A b :=R:D.'- 4 A b = 0.601 +I'

Bolt Tension T ".:=Noax.0t.0.76.Ab.F". T,Y:=No,~.~$~.O.~~.A~.F~,

Eq. 6.13.2.10.2-1 T "x = 1lOeip T "y =55eip

Concrete Compression Block

Derived Eq. 5.7.5-2

Assumes:
sqrt(concrete area/steel plate area) >= 2
Effect of base plate bending is neglected.

$$a_x := \frac{T_{ux}}{\phi_b \cdot 0.85 \cdot f'_c \cdot 2 \cdot W_{plx}} \qquad a_y := \frac{T_{uy}}{\phi_b \cdot 0.85 \cdot f'_c \cdot 2 \cdot W_{ply}}$$

$a_x = 0.991 \cdot in$ $a_y = 0.65 \cdot in$

Point Load due to Anchor Capacity

$$\text{Anchor}_x := \frac{T_{ux} \cdot \left(d_{plx} - \frac{a_x}{2}\right)}{Ybar - H_c} \qquad \text{Anchor}_y := \frac{T_{uy} \cdot \left(d_{ply} - \frac{a_y}{2}\right)}{Ybar - H_c}$$

$\text{Anchor}_x = 50 \cdot kip$ $\text{Anchor}_y = 36 \cdot kip$

Check Post (Continued):

Ultimate Load Resistance of a Single Post

Controlling Post Capacity

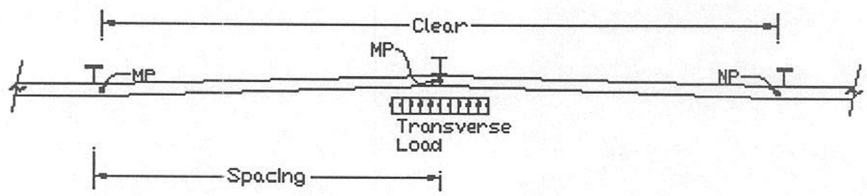
$Post_x := [Pbend_x \;\; Anchor_x]$ $\quad Post_y := [Pbend_y \;\; Anchor_y]$

$P_{px} := \min(Post_x)$ $\quad P_{py} := \min(Post_y)$

$P_{px} = 50 \cdot kip$ $\quad P_{py} = 12 \cdot kip$

Check Load Capacity @ Post using Combined Post and Tube Strength:
(aka - Two Span Failure Mode)

Pict2 := READBMP("Two.bmp")



Total Tube Plastic Moment Capacity: $M_p = 71 \cdot kip \cdot ft$

Clear Distance for Two Post Spacings: $CL2 := 2 \cdot S - W_p$ $\quad CL2 = 228.346 \cdot in$

Combined Capacity

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

$$R_2 := P_{px} + \frac{16 \cdot M_p}{2 \cdot CL2 - L_t} \qquad R_2 = 83 \cdot kip$$

$\text{if}(R_2 > F_t, \text{"OK"}, \text{"LOW"}) = \text{"OK"}$

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 $A_{sb} := \pi \cdot \frac{Db^2}{4}$ $A_{sb} = 0.442 \cdot in^2$

Bolt Factored Shear Capacity assumes: Anchor and Splice Bolts have the same Unima,e Strength

Sec. 6.5.5 and Tbl. 6.5.4.2 (Set at 1.0 for rail comparison) $\phi_s := 1.0$

Eq. 6.13.2.7-1 as modified by C6.13.2.7

R rs:=+ s.2.~~b.~s.(0.6.~“,.~,b) R Ts = 254%~

Tube Bolt Factored Bearing Capacity

Sec. 65.5 and Tbl 6.5.4.2 (Set at 1.0 for rail comparison) $\phi_{bb} := 1.0$

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 Rtrb:=Qbb.NOb.N,~[3.0.Db.(Ttt+Tbt).Fut] Rtrb=2940kip

Splice R srb := $ bb80 b.N ,.[3.O.D b(T stt+T &Fut] R srb =206%p

R rb:=min([Rtrb Rsrb]) R rb = 206okip

Tube Tensile Resistance

sec. 6.5.5 $\phi_y := 1.0$ $\phi_u := 1.0$

sec. 6.13.5.2 u := 1.0

Eq. 6.8.2.1-1 Gross Section Yield

Tube Pug:=+y.Fyt.(Aa+~bt) P trg = 446 Qip

Splice P srg:=$ y’Fyt’(Astt+Asbt) P srg = 279 -kip

P rg:=mq[ptrg psrg]) Prg =279eip

Check Double Bolted Splice (Continued):

Tube Tensile Resistance

Eq. 6.8.2.1-1 Net Section Fracture

Tube A mcalc :-A tt- 2. (SlotWidth ,f 0.0625,in) .T tt .. A tncalc = 8.522%?
+ Abt- 2. (S&Width s+ 0.0625.in) .T bt

Eq. 6.1352 Tension Net Area for Splices

A,,,:=0.85(A,+Abt) A mmax = 8.245tin*

A tn :=q[Amcalc bmx]) Am = 8.24X?

Splice

A sncalc :=A Stt- 2(SlotWidth $+ 0.0625k) .T SR A sncalc = S.236+1*
+Asbt- 2.(SlotWidth s + 0.0625%) .T sbt

Eq. 6.13.5.2 Tension Net Area for Splices

A snmax:=0~85(Astt+Asbt) A ,,,,=5.151*n*

A sn :=mqp sncalc ‘4 S”mx1) A Sn =5.151e12

A , :=min([At,, A~“]) A , =5.151&

P ,,,,:=I$,,.Fut.A,,.U P, =299eip

$\text{if}\left(P_{rn}>P_{rg}, \text{"Yields"}, \text{"Fractures"}\right) = \text{"Yields"}$

Splice Capacity

Rr:=“‘in([% Rrb prg pm]) $R_r = 206 \cdot \text{kip}$

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.

Half tube gross tension $\frac{P_{trg}}{2} = 223 \cdot \text{kip}$

$\text{if}\left(R_r > \frac{P_{trg}}{2}, \text{"OK"}, \text{"LOW"}\right) = \text{"LOW"}$

constants:

psi-1 .&
in2

ksi=lOOO.psi

kip=lOOO.lb

klfs 1000:

AITOW-READBMP(“Amw.bmp”)

Given:

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

Curb:

Height	H c := 11.3.i”	(At Post Center Line)
concrete	f c :=4.35.ksi	

Post W2OOx27 (W8x18 ASTM A572)

			Transverse	Longitudinal
Spacing:	S := lo.ft			
Yield Strength	EYP :=5Oksi	Plastic Modulus	zPX := 17.0.iJ	zPY :=4.66.in3
Width	Wp :=5.25.in			

Base Plate

Thickness:	TPl := 0.75 .in	Width	w plx := 8 .in	wPIY := lO.in
		Depth to CL Bolts	d plx :=6.875.in	dply::6.in

Anchor Bolts I”i$ H.S.

Ultimate Strength	F ua := 12Ok.G	Number	N o =:=2	N o ay:=l
Diameter	D,:=l.in			

Tubes	Top	Bottom
	127x127x7.9 (5x5x5/1 6)	127x127x7.9 (5x5x5/1 6)
Height from Roadway	H tt := 30.5 .in	H bt := 19.25.i”
Depth (Horizontal)	Dtt :=5.i”	D ht :=5.in
Width (Vertical)	Wn:=5.in	W bt:=5.in
Thickness (Wall)	T tt :=$\frac{5}{16}$.,n	T bt:=;.i”
Area	A a := 5.61 ,in’	A bt :=5.61$
Plastic Modulus	Z n := 9.704n’	Z bt:=9.70.i”3

Values taken from AISC 9th Edition ASD

Yield Strength FYt :=46ksi Cold Formed ASTM A-500 Grade B

Minimum Tensile Strength F nt :=5%ksi

Tube Splice

Number of Bolts	No b :=2	Single Shear Planes per Bolt	N,:=2
Bolt Diameter	Db:=lin	Slotted Hole Size	SlotLength s := 1.375.i” SlotWidth s := 1.125.i”
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

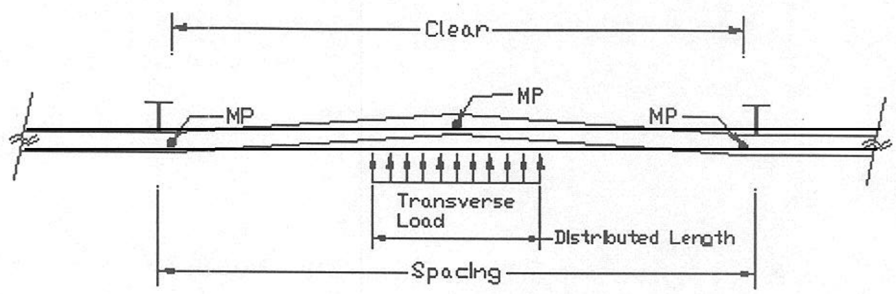
Slotted Hole Size SlotLength := 1.5.i” SlotWidth := 1 .in

Anchor Diameter Anchor := ”.75.in Anchor Sin Diameter Anrhnr s ‘=n *-/<.in

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.kip	Tbl. A13.2-1 TL-4 (Test Level 4)
Distributed Length:	L_t :=3.5.ft	
Longitudinal Load:	Fl :=18&p	
Flexure Resistance Factor	@ f:=l.o	sec. 6.55
Clear Spacing Between Posts:	cL:=s- wp	CL= 114.75el
Top Tube Plastic Moment:	M_{ptt} :=Ztt.Fyt	M_{ptt} = 37okip .ft
Bottom Tube Plastic Moment:	M_{pbt} :=Z bt.Fyt	M_{pbt} = 37eip.ft t
Total Tube Plastic Moment:	Mp:="ptt+Mpbt	M_p = 74akip.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.

$$R1:=ef \frac{16.M_p}{2CL-L_t}$$

RI =76tip

$\mathrm{if}\left(R_1>F_t, \text{"OK"}, \text{"LOW"}\right) = \text{"OK"}$

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:

Ybar := M ptt.H tt; r pbt." bt

Ybar = 24.875∘in

Check Post:

Sending Capacity at the base — Transverse — Longitudinal

Flexure Resistance Factor: Qf=l sec. 6.55

Plastic Moment Capacity: M PPX :=Fyp.Zpx — M PPY :=F YP.2 PY

M ppx = 71 okip.ft — M ppy = 19ekip.ft

Moment Arm: Arm:=Ybar-H,-Tp, — Arm = 12.825el

Point Load due to Post Bending Capacity: Pbend x := M PPX / Arm — Pbend y := M PPY / Arm

Pbend x = 66%~ — Pbend y = 18okip

Anchor Capacity

Concrete Bearing Resistance Factor: @ b := 1.0 sec. 5.5.5 and 5.5.4.2 (Set at 1.0 for rail coparison)

Bolt Tension Resistance Factor: ϕ_t := 1.0 Sec. 6.5.5 and Tbl. 6.5.4.2 (Set at 1.0 for rail comparison)

Bolt Area: Ab :=x.-.-s Da / 4 — A b = 0.7X5%?

Bolt Tension: T ux:=No,.$ t.0.76.Ab.Fu, — T “,,:=Noay.@ t.0.76.Ab.F,,

Eq. 6.13.2.10.2-1: T ux = 143 *kip — T uy = 72%~

Concrete Compression Block
Derived Eq. 5.752

Assumes:
sqlt(concrete area/steel plate area) × 2
Effect of base plate bending is neglected.

.- Tux / +-I$ b~0.85.f,.2.Wp,, — ay:= T UY / $ b.0.85.f ,.2.W ply

a,=2.422& — a y = 0.969%

Point Load due to Anchor Capacity: Anchor x := T “X · (d plx − a x / 2) / YbX-H, — Anchor y := T UY · (d ply − a y / 2) / Ybar- H,

Anchor x = 60%~ — Anchor y = 29%~

Ultimate Load Resistance *of* a *Sing/e Post* with the load located at Ybar above the deck:

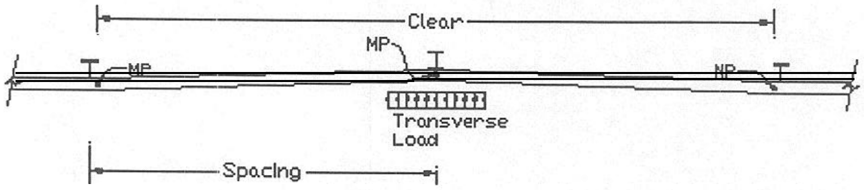
Controlling Post Capacity: Post x :=[Pbend x Anchor X] — Post y :=[Pbend y Anchor Y]

Ppx :=min(Post .$ — Ppy :=min(Post y)

P px = 60%~ — P py = 18mkip

Check Load Capacity @ **Post using Combined Post and Tube Strength:**
(aka -Two Span Failure Mode)

Pict2 := READBMP("Two.bmp")



Total Tube Plastic Moment Capacity: $M_p = 74.367\text{okip.ft}$

Clear Distance for Two Post Spacings: CL2:=2.S-Wp CL2=234.75%1

Combined Capacity

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

$$R2 := PPx + \frac{16.M_p}{2,C-2-L_t}$$

R_2 = 93okip

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

Splice Bolt Area $A_{sb} := \pi \cdot \frac{D_b^2}{4}$ $A_{sb} = 0.785 \cdot in^2$

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

Sec. 6.55 and Tbl. 654.2 (Set at 1 .0 for rail comparison) $,:=1.0

Eq.S.13.2.7-1 a.smodifiedbyC6.1~2.7

R rs:=$,.2.Nob.N,.(0.6.F",.A,b) R Ts = 452%~

Tube Bolt Factored Bearing Capacity

Sec. 6.63 and Tbl. 6.5.4.2 (Set at 1 .0 for rail comparison) **$ bb := 1.0**

Eq. 6.13.2.9-I 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

R rb:=$bb.2.Nob.N,(3.0.Db.Ttt.Fut) R rb = 435 &ip

Tube Tensile Resistance

sec. 6.5.5 $ y := 1.0 $":=1.o

sec. 6.13.6.2 u := 1.0

Eq. 6.8.2.1-1 Gross Section Yield

P rg:=@ y,Fyt(Att+Abt) Prg = 516%~

Eq. 6.8.2.1-I Nat section Fracture

A ncalc :=Att- 2~(SlotWidtb S+ 0.0625.h) .T R ,,, A ,,calc = 9.736%'
+Abt-2.(SlotWidths+0.0625.in) .Tbt

Eq. 6.1352 Tension Net Area for Splices

A ,,:=0.85.(Aa+Abt) A Nnax = 9.537.in2

An :=if(A.calc<Anmax,A"calc.Anmax) A, = 9.537.i"'

P m :=$ ".Fut.A,,-U P, =553skip

if(P ,.&P.&, 'Yields" , "Fractures") = "Yields"

Splice Capacity

Splice := [R rs R rb] R,:=min(Splice) R r = 435 okip

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

$\text{if}\left(R_r > \frac{P_{rg}}{2}, \text{"OK"}, \text{"LOW"}\right) = \text{"OK"}$

Constants:

$$psi \equiv 1 \cdot \frac{lb}{in^2}$$

$$ksi \equiv 1000 \cdot psi$$

$$kip \equiv 1000 \cdot lb$$

$$klf \equiv 1000 \cdot \frac{lb}{ft}$$

Arrow≡READBMP("Arrow.bmp")