July 17, 2002

HSA-10/B104

Gary L. Hoffman, P.E.
Chief Engineer for Highway Administration
Pennsylvania Department of transportation
400 North Street
Harrisburg, PA 17103

Dear Mr. Hoffman:

Your June 4 letter to Mr. Richard Powers of my staff requested Federal Highway Administration acceptance of a bridge rail design called the Pennsylvania Bridge Barrier. This design is similar to the currently accepted BR27C Test Level 4 (TL-4) bridge railing, but is 50 inches high and consists of two TS 5 x 4 x 5/16 rails supported by W8 x 28 posts on 7.5-foot centers. The support posts are bolted to a 24-inch tall reinforced concrete parapet that is 18 inches wide. The centers of the two rails are 35 inches and 48 inches above the bridge deck and in the same vertical plane as the concrete parapet.

To support your request, you also sent copies of a report entitled "Pennsylvania Bridge Rail – TL-5 Barrier" that included an analytical comparison of your proposed design with the Texas HT barrier and with a 42-inch tall F-shape barrier. Both of the latter are considered to be TL-5 designs based on full-scale crash testing. The analysis procedure was reviewed by our bridge engineers and found to be appropriate. One minor suggestion offered was that you consider using the same size anchor bolts on the field side of the post base plates as on the traffic side to minimize the potential for construction errors.

Based on staff review, I agree that the Pennsylvania Bridge Rail, as described above, is equivalent to an NCHRP Report 350 TL-5 design and it may be used on the National Highway System where such use is deemed appropriate by a highway agency. When you have finalized your drawing, please send an electronic copy in pdf format to Mr. Powers so it can be added to our safety hardware website.

Sincerely yours,

(original signed by Carol H. Jacoby)

Carol H. Jacoby, P.E.
Director, Office of Safety Design

See FHWA endorsement page 2.

OS-2 (4-01)

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
www.dot.state.pa.us

PENNDOT

May 4, 2004

Federal Highway Administration
228 Walnut Street, Room 558
Harrisburg, PA 17101-1720
Attention: Mr. James Cheatham
Division Administrator

RE: Redesign of PA Bridge Rail, Transition Details, Special Provisions, and Changes to BC and BD Drawings

Dear Mr. Cheatham:

This letter is to request final approval for the TL-5, PA Barrier including our Bridge and Roadway standard drawings and special provisions which incorporates comments from the previous clearance transmittal dated February 11, 2004. We also request final approval to fully implement the PA Bridge Barrier. The following items are included in this submission:

- Bridge Design and Construction Standards, BD-610M, BC-712M, and BC-713M

- Roadway Construction Standard, RC-50M

- Special Provisions associated with the PA Bridge Rail and transitions which eventually will become part of the Department's Publication 408 Specification

- Transition details for PA Bridge Rail to PENNDOT 42" F-Shape Barrier, for approval as an acceptable alternate standard detail. These details were used on the Fort Pitt Bridge in Pittsburgh and were previously approved for the subject structure by FHWA.

Some key highlights of the changes to the PA Bridge Barrier are depicted below:

- Weld type for the bends at the ends of barriers have been revised to a miter weld detail since the previous detail was very difficult to construct.

- As per our discussion with William Williams, a second mid-span bracket was added to the Mid-span Tube Assembly for changing the test level of the Thrie-Beam to PA Bridge Barrier Transition from TL-3 to a TL-4. We request the proposed transition be granted TL-4.

- Toggle bolts have been replaced with threaded anchor studs for the handrail attachment.

- For the rail splice detail, the stainless steel drive-fit pin was replaced with a welded stud to resolve a constructability issue associated with looseness at the splice.

- The base plate dimensions were increased to allow clearance between the anchor bolt nut and the fillet weld.

Also, we request your concurrence to publish Change #2 to Publication 219M (BC Drawings) and Change #1 to Publication 218M (BD Drawings). These changes include the implementation of the new Bulb Tee P/S girders developed by the Prestressed Committee for Economical Fabrication, the initial release of Steel Girder Bridges Lateral Bracing Criteria and Details, BD-620M, and multiple minor corrections (copies attached). These changes were previously sent via our February 10, 2004, Clearance Transmittal.

We appreciate your concurrence and any comments on the attached submission.

If you have any questions, please call Bryan Spangler, P.E. at 717-783-5347.

Attachments

Sincerely,

Dean A. Schreiber

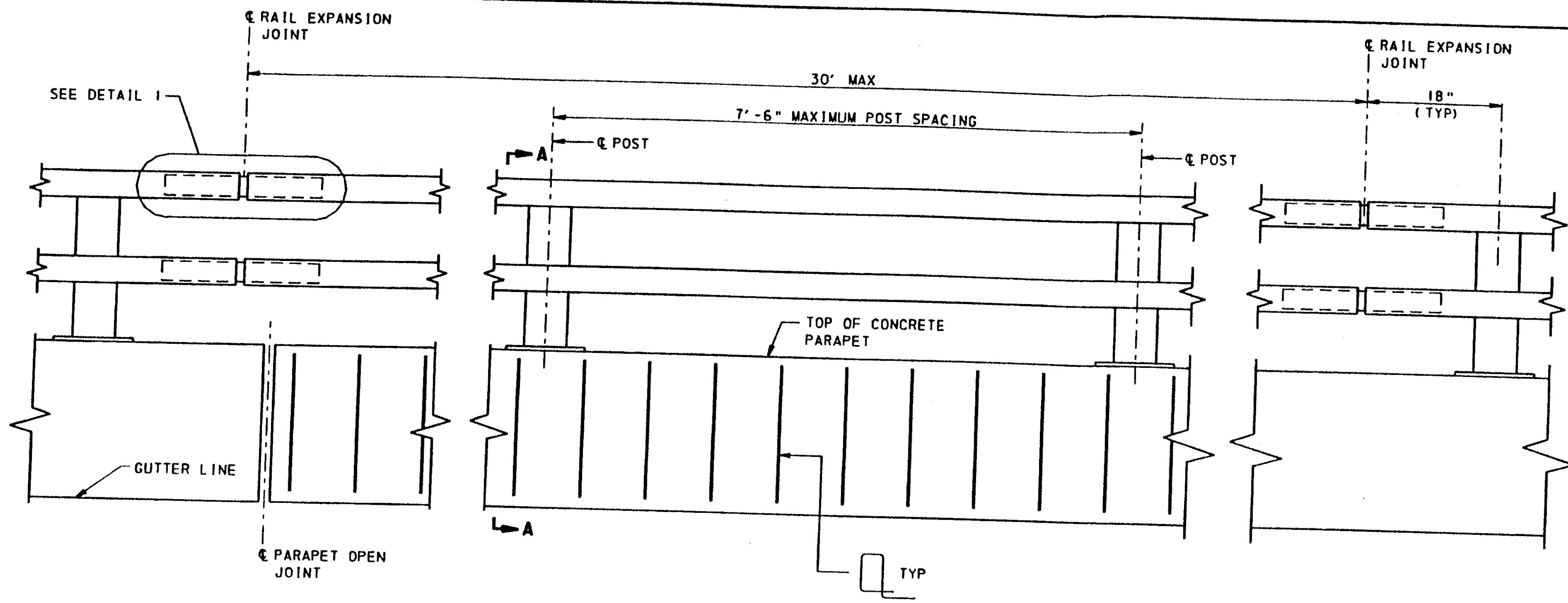
M. G. Patel, P.E.
Chief Engineer for
Highway Administration

Approved: William E Williams MAY 0 5 2004
(Date)

for Division Administrator
Federal Highway Administration

4310/BJS/pvd

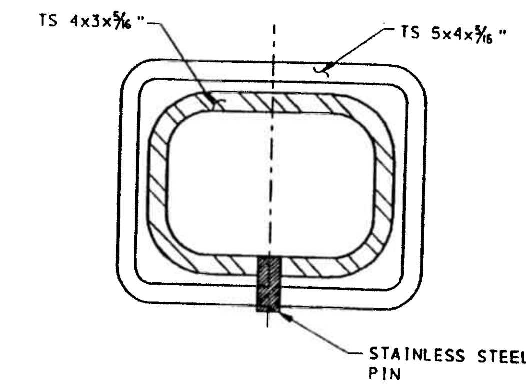
cc: G. L. Hoffman, P.E., 8th Floor, CKB
D. A. Schreiber, P.E., 7th Floor, CKB
R. Scott Christie, P.E., 7th Floor, CKB
Bryan J. Spangler, P.E., 7th Floor, CKB
Anthony J. McCloskey, P.E., 7th Floor, CKB



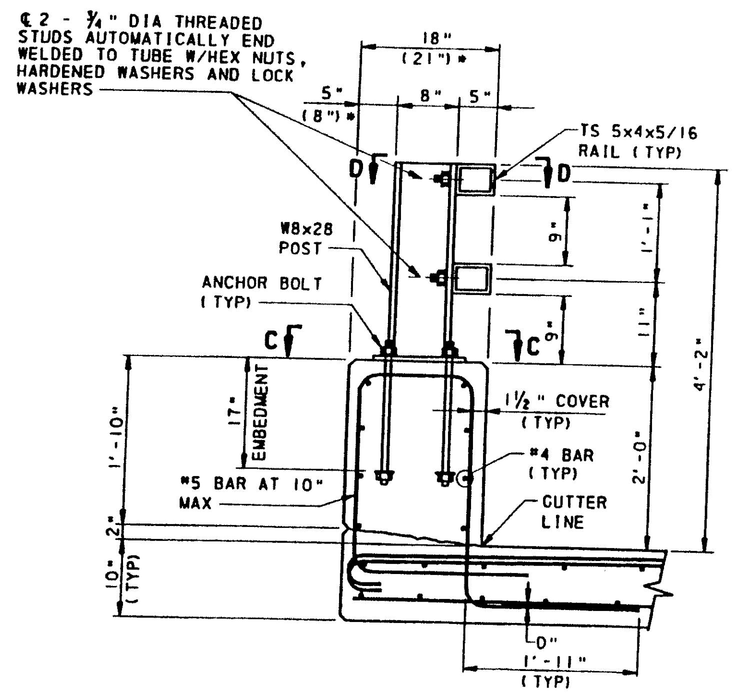
BRIDGE RAIL ELEVATION

12" 6" 0 1'

1" = 1'-0"

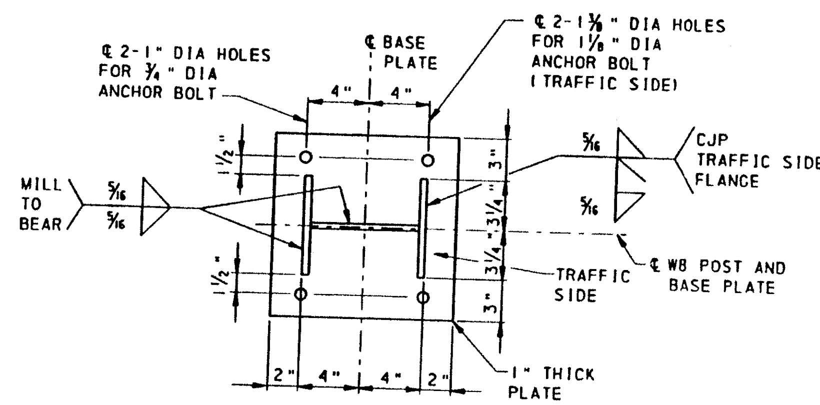


SECTION B-B



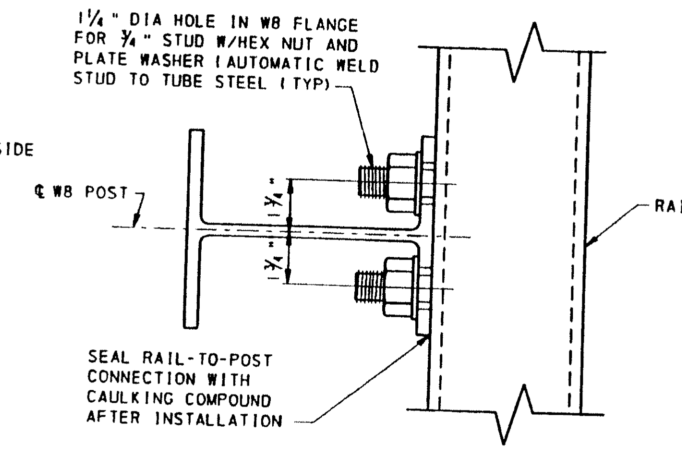
SECTION A-A

* - FOR THE FORT PITT BRIDGE PROJECT ONLY



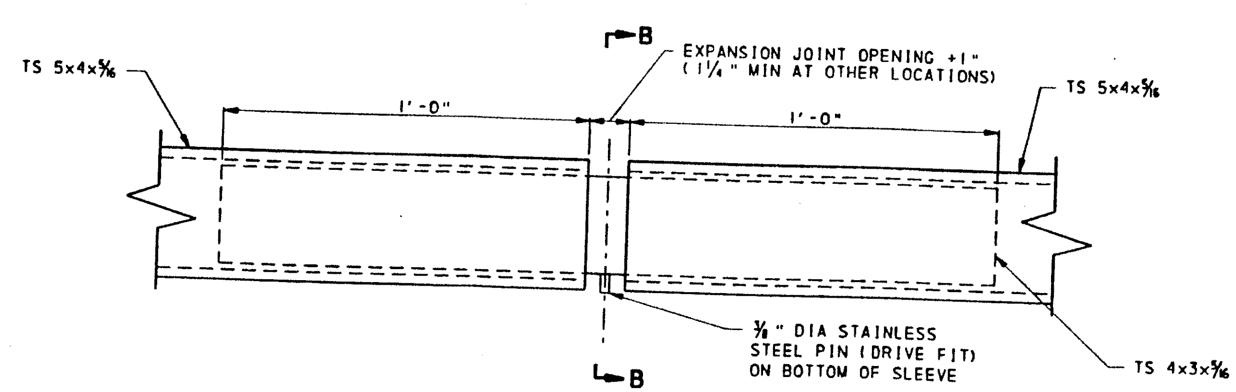
SECTION C-C

BASE PLATE



SECTION D-D

RAIL TO COLUMN DETAIL



DETAIL 1

TYPICAL RAIL EXPANSION JOINT DETAIL

NOTES:

1. ALL TUBE STEEL MATERIAL TO BE ASTM A500 GRADE B. ALL OTHER STRUCTURAL STEEL COMPONENTS SHALL BE AASHTO M270 (ASTM A709) GRADE 50.

2. FOR RAIL TO POST CONNECTION AUTOMATIC WELDED STUDS SHALL MEET THE REQUIREMENTS OF ASTM A307. HEX NUTS SHALL MEET THE REQUIREMENTS OF ASTM A563.

3. ALL WELDING SHALL CONFORM TO AWS D1.5. FOR WELDING OF TUBULAR MATERIAL, THE AWS D1.1 REQUIREMENTS MAY BE USED IN ADDITION TO D1.5 REQUIREMENTS. ALL ELECTRODES SHALL BE 70 ksi LOW HYDROGEN TYPE.

4. ANCHOR BOLTS SHALL BE ASTM F1554 GRADE 105 BOLTS WITH ONE ASTM A563 GRADE DH HEAVY HEX NUT AND ONE ASTM F436, 2 1/4" O.D. WASHER AT EACH END. OPTIONALLY USE RECTANGULAR 3/8" x 2" x 3" A36 PLATE WASHER WITH 5/8" DIA HOLE FOR 3/4" DIA BOLT AND 1 3/16" DIA HOLE FOR 1 1/8" DIA BOLT.

5. PROVIDE MATERIALS AND WORKMANSHIP IN ACCORDANCE WITH PENNSYLVANIA DEPARTMENT OF TRANSPORTATION SPECIFICATIONS. (PUBLICATION 408M).

6. GALVANIZE ALL STEEL COMPONENTS IN ACCORDANCE WITH AASHTO M111 AFTER WELDING.

7. PLACE POST AND POST ANCHOR BOLTS NORMAL TO GRADE. PLACE RAILS PARALLEL TO GRADE.

8. LOCATE RAIL JOINTS AT STRUCTURE EXPANSION JOINTS AND AT OTHER LOCATIONS WHERE NECESSARY. PROVIDE A MINIMUM OF THREE POSTS BETWEEN RAIL JOINTS. LOCATE ℄ RAIL JOINT 18" FROM ℄ POSTS.

9. COAT ALL SURFACES OF THE BASE PLATE IN CONTACT WITH CONCRETE WITH CAULKING COMPOUND PRIOR TO ERECTION. AFTER ERECTION AND ALIGNMENT, SEAL OPENINGS BETWEEN THE METAL SURFACES AND THE CONCRETE WITH CAULKING COMPOUND.

10. TIGHTEN ANCHOR NUTS 1/3 TURN PAST SNUG TIGHT POSITION.

MAY 29, 2002

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF DESIGN

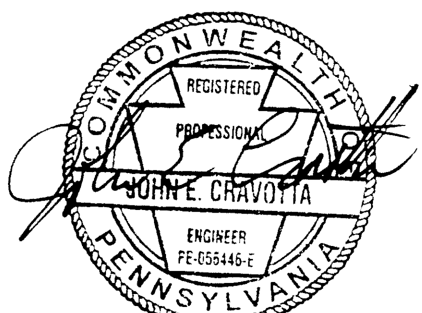
PENNSYLVANIA BRIDGE RAIL

RECOMMENDED ____	RECOMMENDED ____	RECOMMENDED ____	SHT. 1 OF 1
CHIEF BRIDGE ENGINEER	DIR., BUREAU OF DESIGN	CHIEF ENGINEER	001

DGN	MFH
DWN	KAS
CKD	JEC

Comparison of the Proposed PA Bridge Rail and the BR27C Bridge Rail

TABLE OF CONTENTS



6-3-02

Comparison of the Proposed PA Bridge Rail and the BR27C Bridge Rail

1.0 Introduction

This submission provides documentation for the acceptance of the PA Bridge Rail as a viable TL-5 traffic barrier. The information in this document demonstrates the similarity in geometry to the crash tested BR27C rail and provides calculations that indicate the increased capacity of the PA Bridge Rail is able to resist the TL-5 rated loads.

The development of this rail was initiated for use by the Pennsylvania Department of Transportation District 11-0 on the Fort Pitt Bridge, spanning the Monongahela River at the Point in Pittsburgh, Pennsylvania.

The River Life Task Force, a local public interest group concerned with development of the riverfronts in Pittsburgh, determined that there was public support for a bridge railing that would afford better views of the city and rivers than the 42-inch concrete F-Shaped barrier originally specified. Development of the PA Bridge Rail was a joint effort by the River Life Task Force, Dr. Sunil Saigal, HDR Engineering, and the Pennsylvania Department of Transportation.

The intent of this design is to provide a more appealing barrier for the Fort Pitt Bridge and to establish this traffic barrier as a standard TL-5 barrier for use on bridges throughout the Commonwealth of Pennsylvania.

2.0 Geometry

The PA Bridge Rail was modeled as a strengthened BR27C. The crash tested BR27C bridge rail could not be used directly because it has a rated capacity of TL-4. A stronger TL-5 barrier is required for the Fort Pitt Bridge Project, as well as for numerous other bridges owned by the Pennsylvania Department of Transportation.

The BR27C detailing was maintained wherever possible, but revisions have been made to enhance detailing or performance. Welded studs have been used for the rail to post connections to eliminate the snag points of the bolt heads on the face of the rails. The use of bolts at the rail expansion joints has also been eliminated for this reason.

Details of the proposed PA Bridge Rail are presented in Appendix A and can be compared with the sketches of the BR27C contained in Appendix B. The concrete parapet height of 24 inches remains the same. The total height of the rail has been increased from 42 inches to 50 inches with a second rail added, and other components have been strengthened. The post is a W section (W8x28 in lieu of a TS4x4 square tube) that provides greater bending capacity and the rails were increased to TS5x4x5/16 rectangular tubes from the TS4x3x1/4 used for the BR27C. The anchor bolts have also been increased to 1 1/8 inch diameter ASTM F1554 Grade 105 anchor bolts on the tension side of the rail.

The width of the concrete wall portion of the barrier has been increased to 18 inches from 10 inches and the reinforcement steel has also been increased to #5 bars spaced at 10 inches nominal (vs. #4 @ 8") in the transverse direction and eight (8) #4 bars (vs. 6 - #4 bars) longitudinally.

The separation of rail elements for the PA Bridge Rail have been calculated and plotted on the AASHTO charts. The calculations and charts are contained in Appendix C. All of the checks from AASHTO A13.1.1 are shown to be acceptable for the PA Bridge Rail. Rail-to-post contact is 31% and the ratio of rail contact width to height is 0.64. These parameters, coupled with a setback of 5 inches and a vertical clear opening between rails of 9 inches, indicate that the PA Bridge Rail is a geometrically acceptable rail.

It should be noted that for the Fort Pitt Bridge Project only, the width of the concrete portion of the PA Bridge Rail will be 21 inches. This widened base matches the base width of the barrier originally detailed for the project. Thus, the overall bridge width and gutter-to-gutter dimensions will not change. This will require fewer detailing changes to be made in incorporating the PA Bridge Rail details into the already-completed Fort Pitt Bridge plans. The same type, spacing and size reinforcement bars will be used in the widened concrete section, modified only for the increased barrier width.

3.0 Strength

The PA Bridge Rail has been designed as a TL-5 traffic barrier. Calculations supporting this capacity are contained in Appendix D. Since the BR27C is a TL-4 barrier, a direct comparison of strength is not appropriate; however, a comparison is made with the 42-inch F-shape barrier and the Texas HT Barrier.

Table 1

Barrier (Combined Rail and Concrete)	Resultant Strength at Midspan	Resultant Strength at / near Joint
PA Traffic Rail (Rail and Concrete)	459 kips	331 kips
Texas HT	276 kips	245 kips
42-inch F-shape	136 kips	152 kips
PA Traffic Rail (Minimum – Steel Rail only)	126 kips	127 kips
AASHTO TL-5 Requirement	124 kips	124 kips

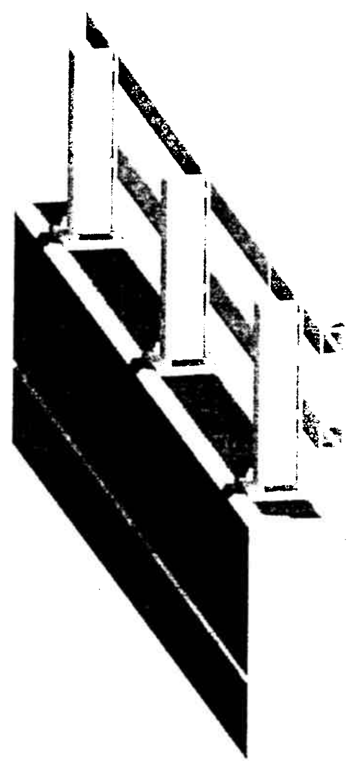
Note that the steel rails and post portion of the PA Bridge Rail have been designed to resist the entire 124 kip design load. Once coupled with the strength of the concrete barrier portion, the capacity of the whole system is well in excess of the AASHTO requirements.

Typical bridge deck overhangs are designed using PENNDOT's Bridge Design Standards BD-601M, which are based on the AASHTO LRFD code requirements. The overhang designs in this standard provide a capacity adequate for the standard 42-inch F- shape, a TL-5 barrier. The deck overhangs designed with BD-601M are appropriate for used with the PA Bridge Rail because the base width is similar to the F-shape (1'-6" vs. 1'-5 1/4") uses the same nominal transverse reinforcement.

All connections and details have been designed in accordance with the AASHTO LRFD code and these calculations are included in Appendix E. Designed items include rail-to-post connection, post-to-base plate connection, anchor bolts, anchor bolt embedment, and rebar development. All components have been found to have adequate capacity for the AASHTO TL-5 loadings.

4.0 Summary

Based on comparison to the BR27C and other TL-5 rated barriers, the PA Bridge Rail described in this submission has effective geometry, good detailing and adequate strength to be classified as a TL-5 traffic barrier.



Appendix A

Proposed PA Bridge Rail (TL-5)

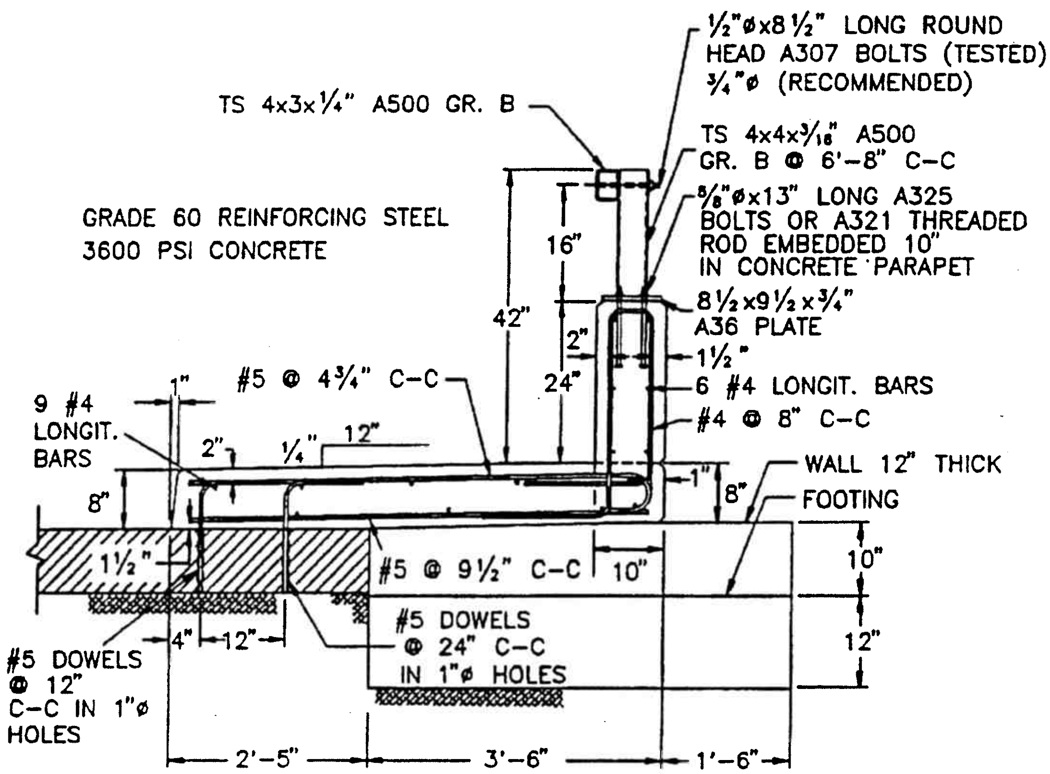
Appendix B

Crash Tested Comparison Barrier:

BR27C

FHWA BRIDGE RAIL MEMORANDUM, MAY 30, 1997: PART 1, 2 & 3 COMBINED AND SORTED BY TYPE

FIGURE NUMBER	BRIDGE RAILING	RAILING HEIGHT (IN.)	TEST VEHICLE	IMPACT SPEED (MPH)	IMPACT ANGLE DEGREES	Meets NCHRP 230	PERFORMANCE LEVEL	NCHRP 350 EQUIVALENT TEST LEVEL FROM FHWA MEMO 3	REFERENCES	FHWA Bridge Rail Memo #1	FHWA Bridge Rail Memo #2	FHWA Bridge Rail Memo #3
W-BEAM BRIDGE RAIL												
1	Texas Type T6 (Tubular W-beam)	27	2,280 lb Car 4,500 lb. Car 2,000P Pickup	58 61.6 100 kph	14 27.5 25	Yes		TL-2	1,2 1,2 3	3		1-3
2	West Virginia W-beam Retrofit Railing for Concrete Baluster designs, curb mounted	28.5	817 kg Car 2452 kg Pickup	80 kph 72 kph	20 20		Originally tested to PL-1	TL-2	4			3-11
THRIE-BEAM BRIDGE RAIL												
3	NCHRP SL1 Thrie Beam, Wood Posts	32	2,250 lb. Car 2,250 lb. Car 4,500 lb. Car	63 60.1 61.9	18.7 15.9 14.5	Yes		TL-2	2,5	1		1-1
4	NCHRP SL1 Thrie Beam, Steel Posts	32	1,987 lb. Car 2,250 lb. Car 2,250 lb. Car 20,000 lb. Bus	61.4 58.6 60 44.7	14.1 16 18 7.7	Yes		TL-2	2,5	2		1-2
5	Nebraska Tubular Thrie Beam	32	1,970 lb. Car 4,700 lb. Car	61.4 58.4	20 24.3	Yes		TL-3	6,7	11		1-12
6	Oregon Side-Mounted Thrie Beam	27	1,970 lb Car 5,737 lb Pickup	52.2 48.1	19.7 20.8	Yes	PL-1	TL-2	8,9,10		1-1	2-1
7	Washington 10 gage Thrie Beam Retrofit for Balusters Curb/Sidewalk	30	1,840 lb Car 4,725 lb Car 5,400 lb Pickup	58.8 60.7 68.3	19.5 15.6 19.4	Yes	PL-1	TL-2	11		1-8	2-7
8	California Thrie Beam	32	1,935 lb Car 5,565 lb Pickup	48.7 44.9	18.3 21	Yes	PL-1	TL-2	12,13		1-9	2-8
9	Missouri Thrie Beam and Channel (top mounted)	30-5/8", 30.625"	1,984 lb Car 4,495 lb Car	59.6 60.9	15 24	Yes		TL-3	14,15		1-10	2-13
10	Michigan 10 gage retrofit on Curb/Sidewalk (Michigan R4 Retrofit Bridge Rail)	34	1,972 lb Car 5,724 lb Pickup 18,000 lb Truck	61.5 60.6 49	20 20 15	Yes	PL-2	TL-4	16		1-15	2-15
11	Delaware Thrie-beam Retrofit Railing (curb-mounted)	32	8,000 kg Single-Unit Truck	80 kph	15		Tested to NCHRP 350	TL-4	4			3-24
METAL TUBE BRIDGE RAIL												
Aluminum Tube Bridge Rail												
12	Aluminum Tru-Beam (Modified AASHTO BR5)	32	2,150 lb. Car 4,500 lb. Car	61.3 58.9	21.5 27.2	Yes		TL-2	17,18	4		1-4
13	Foothills Parkway Aluminum Bridge Rail	33	2,069 lb Car 5,465 lb Pickup 5,565 lb Pickup	52 48.6 45.7	22 20.7 22.7		passed PL-1, NCHRP 230 failed, anchorage not long enough passed, corrected anchorage, PL-1	TL-2	19,20		2-22	3-12
Steel Tube Bridge Rail, Attached to Bottom of Deck												
14	Texas Energy Absorbing Bridge Rail	27	1,972 lb. Car 4,500 lb. Car	62.6 61	16 25	Yes		NCHRP 230	21,22,23	6		
Steel Tube Bridge Rail, Attached to Side of Deck												
15	California Type 18 (See-Through, Collapsing Ring)	36	1,850 lb. Car 4,530 lb. Car	59.7 60.7	12 23	Yes		NCHRP 230	24,13	17		
16	Collapsing Ring Bridge Railing	59	2,090 lb. Car 4,400 lb. Car 40,000 lb. Bus 40,000 lb. Tractor-Trailer 70,000 lb. Tractor-Trailer	55.7 62 53.9 57 44.4	23.5 22.7 15.1 15.6 10	Yes		NCHRP 230	25,26,27,28	21		
17	Ohio Box Beam Rail (W-beam backed up with box beam)	27	1,980 lb. Car 4,790 lb. Car	60.6 60	19.6 25	Yes		TL-2	8	8		1-8
18	California Type 115	30	1,965 lb Car 5,635 lb Pickup	59 64.2	18 21	Yes	PL-1	TL-2	29,13		1-7	2-6
19	Illinois Side Mounted Bridge Rail	32	1,970 lb Car 5,565 lb Pickup 18,000 lb Truck	59.9 60.4 51.4	20.1 20.4 14.7		tested to PL-2	TL-4	8,30		2-6	3-22
Steel Tube Bridge Rail, Attached to Top of Deck												
20	Texas T101 Bridge Rail	27	2780 lb. Car, 4,660 lb. Car 4,630 lb. Car 6,900 lb. Bus 19,940 lb. Bus 20,010 lb. Bus 31,880 lb. Bus	57.3 60.2 59.8 53.4 55.3 52 58.4	15 15 25.8 15 15.2 13.2 16	Yes		TL-3	18,31,32,33,2	7		1-11
21	Texas 421 Aesthetic Steel Pipe Bridge Rail	32	1,800 lb Car 4,500 lb Car	59.7 62.4	21.4 28.6	Yes		TL-2	34,35		1-12	2-11
22	Washington, D.C. Historic Bridgerail (curb-mounted retrofit)	27	1,965 lb Car 5,585 lb Pickup	49.7 47.7	21.5 20.8		PL-1	TL-2	36		2-23	3-15
Steel Tube Bridge Rail, Attached to Parapet												
23	AASHTO BR2 (California Type 9)	27	1,929 lb. Car 4,540 lb. Car	60.9 57	13.1 26	Yes		TL-2	2, unknown 37	5		1-5
24	North Carolina - Standard 1 Bar Metal Rail	32	1,990 lb. Car 4,680 lb. Car 19,920 lb. Bus	59.7 60 57.3	18.8 25 14.8	Yes		TL-2	6,8 6,8 18,31,32,33	13		1-10
25	Modified Texas C202 Bridge Rail	54	1,918 lb Car 4,400 lb Car 79,770 lb Van Type Tractor Trailer	61.3 59.4 49.1	21 25.9 15	Yes	Special PL-4 (originally PL-3)	TL-5	38 38 39		1-25	2-23
26	BR27D-two steel Rails on 18" concrete parapet w/ 8-in curb and 5-ft sidewalk	42	1,967 lb Car 5,565 lb Pickup	51.7 45.3	20.8 20.2		PL-1	TL-2	8,40,41		2-2	3-16
27	BR27D-flush-mounted	42	1,970 lb Car 5,566 lb Pickup	51.2 45.6	20.5 18.8		PL-1	TL-2	8,40,41		2-3	3-17
28	BR27C-single steel rail on 24" concrete parapet w/ 8-in. curb and 5-ft sidewalk	42	1,965 lb Car 5,568 lb Pickup 18,000 lb Truck	61.7 62.6 51	18.7 19.4 13.7		PL-2	TL-4	8,40,42		2-4	3-18
29	BR27C-flush-mounted	42	1,970 lb Car 5,570 lb Pickup 18,000 lb Truck	60.3 55.3 52.5	19.8 19.6 12.8		PL-2	TL-4	8,40,42		2-5	3-19
30	Minnesota Combination Bridge Rail, Design #3	36	18,000 lb Truck 4,420 lb Pickup 4,442 lb Pickup 1,960 lb Car	50.8 60.8 62.5 61	16.2 25.5 25.9 20.6		Tested to NCHRP 350	TL-4	43			3-26



1 in = 25.4 mm
1 ft = 0.305 m

Figure B7.28. BR27C Bridge Railing with Sidewalk (8,40,42).

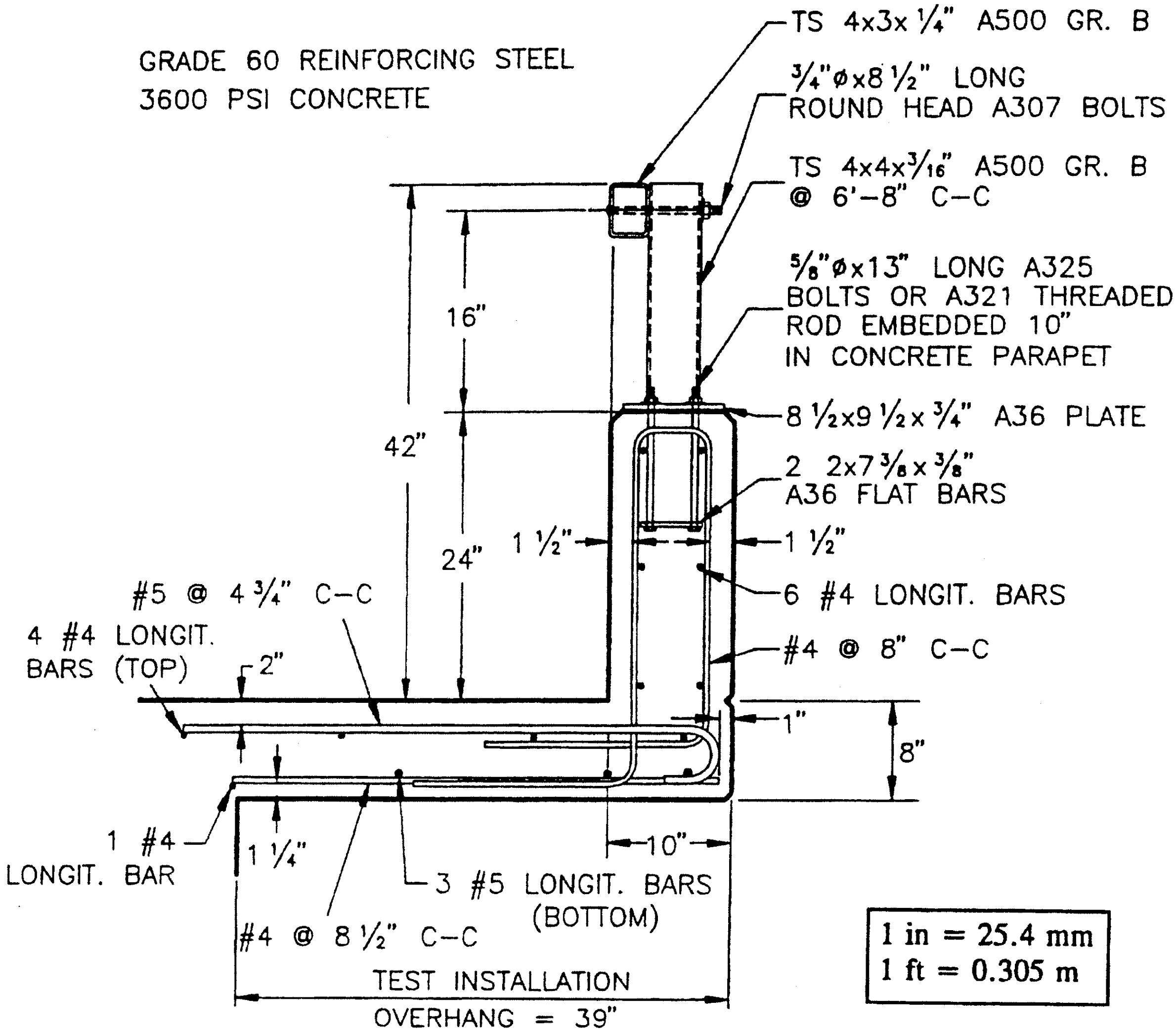


Figure B7.29. BR27C Flush Mounted Bridge Railing (8,40,42).

Appendix C

Separation of Rail Elements:

AASHTO A13.1.1

SPECIFICATIONS

COMMENTARY

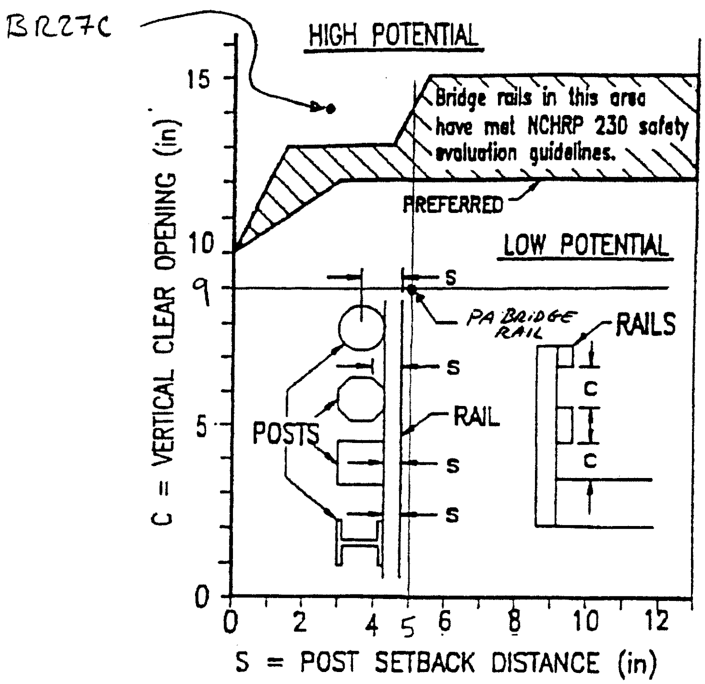


Figure A13.1.1-2 - Potential for Wheel, Bumper, or Hood Impact with Post

(A13.1)

S = 5"

Height of Rail = 26"

Contact width = 2·4 = 8"

$\frac{8}{26} = 0.308 > .25$ ✓

C = 9"

PA Bridge Rail O.K. ✓

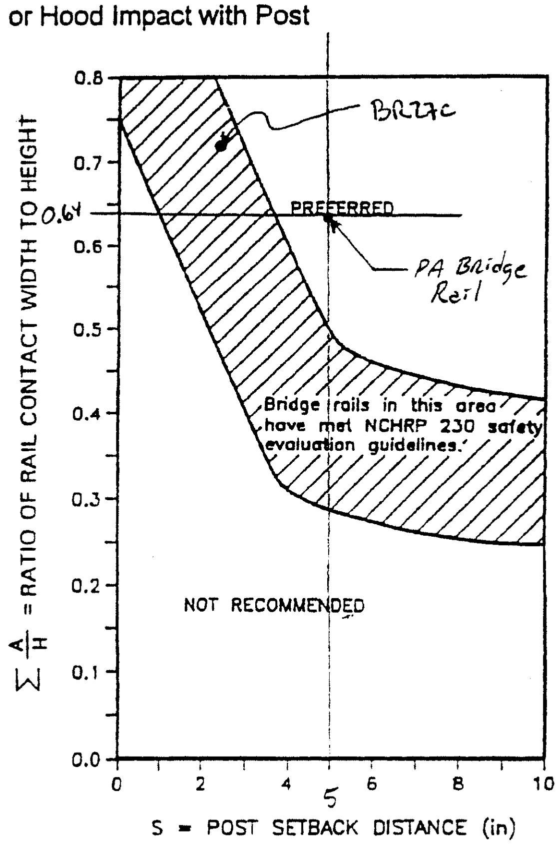


Figure A13.1.1-3 - Post Setback Criteria

concrete Rails

$\Sigma A = 24" + 2(4")$

$= 32"$

$H = 50"$

$\frac{\Sigma A}{H} = \frac{32}{50} = 0.64$

PA Bridge Rail O.K. ✓

Appendix D

AASHTO Railing Calculations

(a) Conversion Factors

1 kip = 4.45 kN

1 ft. = 304.8 mm

(b) PLASTIC BENDING BETWEEN POSTS: Single Span Failure Mode

Design Forces for Traffic Railings (from Table A 13.2-1)

Railing Test Level – 5 (TL-5)

Transverse Load: F_t = 550,000 N = 124 kips

Distributed Length: L_t = 2440 mm = 8.0 ft. = 96 in.

Longitudinal Load: F_L = 183,000 N = 41 kips

Barrier Post

Post = W8x28

Center to Center Spacing Between Posts: S = 7ft. 6in. = 90.0 in.

Width of Post: b_f = 6.5 in. (from AISC manual)

Clear Spacing Between Posts: $C_L = S - b_f = 90.0 - 6.5 = 83.5$ in.

Barrier Rails

Top Tube: 5 x 4 x 5/16"

Plastic Modulus: $Z_x = 8.24\ in^3$ $Z_y = 7.05\ in^3$ (from AISC manual)

Z_x applicable

Yield Strength: F_{yt} = 46 ksi

Plastic Moment: $M_{ptop} = Z_x * F_{yt} = 46 * 8.24 = 379$ kip.-in.

Bottom Tube: 5 x 4 x 5/16"

Plastic Modulus: $Z_x = 8.24\ in^3$ $Z_y = 7.05\ in^3$ (from AISC manual)

Z_x applicable)

Yield Strength: $F_{yt} = 46$ ksi

Plastic Moment: $M_{pbot} = Z_x * F_{yt} = 46 * 8.24 = 379$ kip.-in.

Total Tube Plastic Moment = $M_p = M_{ptop} + M_{pbot}$

$= 379 + 379 = 758$ kip.-in.

Flexure Resistance Factor: $\phi_f = 1.0$ (Section 6.5.5)

Total Ultimate Resistance (nominal resistance of the railing): R_l

Derived from Eqn. A13.3.2-1 for a single span failure mode with plastic hinges at end of post

$$R_l = \phi_f * 16. * M_p / (2. * S - L_t)$$

$$= 1.0 * 16. * (758) / (2. * 90 - 96)$$

$$= 144 \text{ kips.}$$

$$F_t = 124 \text{ kips}$$

$R_l > F_t$, OK

Location of center line of top railing: $H_{top} = 48.0$ in (from Figure 1)

Location of center line of bottom railing: $H_{bot} = 35.0$ in (from Figure 1)

Location of Resultant R_l: Y_l

$$Y_l = (M_{ptop} * H_{top} + M_{pbot} * H_{bot}) / M_p$$

$$= (379 * 48 + 379 * 35) / 758$$

$$= 41.5 \text{ in.}$$

(c) Check Post - Bending Capacity at the base

Post = W8x28

Flexure Resistance Factor: $\phi_f = 1.0$ (Section 6.5.5)

Plastic Modulus: $Z_{px} = 27.2\ in^3$, $Z_{py} = 10.1\ in^3$

Yield Strength: $F_{yp} = 50.0$ ksi

Transverse Plastic Moment Capacity: $M_{px} = Z_{px} * F_{yp}$

$= 27.2 * 50.0 = 1360$ kip.-in.

Longitudinal Plastic Moment Capacity: $M_{py} = Z_{py} * F_{yp}$

$= 10.1 * 50.0 = 505$ kip.-in.

Moment Arm: $Arm = Y_1 - t_{plate} - H_{concrete}$

$= 41.5 - 1.0 - 24.0 = 16.5$ in.

Point Load in x-direction Due to Post Bending Capacity: P_{bendx}

$P_{bendx} = M_{px} / Arm$

$= 1360 / 16.5 = 82$ kips

Point Load in y-direction Due to Post Bending Capacity: P_{bendy}

$P_{bendy} = M_{py} / Arm$

$= 505 / 16.5 = 31$ kips

(d) Check Post – Anchor Capacity

Concrete Bearing Resistance Factor: $\phi_b = 1.0$ (Sec. 6.5.5)

Bolt Tension Resistance Factor: $\phi_t = 0.8$ (Sec 6.5.5 and 6.5.4.2)

Anchor Bolt: 1-1/8" ASTM F 1554 Grade 105 anchor bolt (traffic side of barrier)

Ultimate Bolt Strength: $F_{ua} = 125$ ksi

Bolt Diameter: $D_a = 1.125$ in.

Bolt Area: $A_b = \pi * D_a^2 / 4$

$= \pi * (1.125)^2 / 4 = 0.9940 \text{ in}^2$

Anchor Bolt: 3/4" ASTM F 1554 Grade 105 anchor bolt (outside face of barrier)

Ultimate Bolt Strength: $F_{ua} = 125$ ksi

Bolt Diameter: $D_a = 0.75$ in.

Bolt Area: $A_b = \pi * D_a^2 / 4$

$= \pi * (1.125)^2 / 4 = 0.4418 \text{ in}^2$

Number of bolts in x-direction: $n_{ax} = 2$ 1-1/8 " diameter bolts

Bolt Tension in x-direction: $T_{ux} = n_{ax} * \phi_t * 0.76 * A_b * F_{ua}$

(Eq. 6.13.2.10.2-1)

$= 2 * 0.8 * 0.76 * 0.994 * 125$

$= 151$ kips

Number of bolts in y-direction: $n_{ay} = 1$ 1-1/8 " diameter bolt, 1 3/4 " diameter bolt

Bolt Tension in y-direction: $T_{uy} = n_{ay} * \phi_t * 0.76 * A_b * F_{ua}$

(Eq. 6.13.2.10.2-1)

$= 0.8 * 0.76 * (0.994 + 0.4418) * 125$

$= 109$ kips

Point Load Due to Anchor Capacity in the x-direction: $P_{anchorx}$

Compression is taken by the post flange

Post flange thickness = 7/16 in.

Flange to flange distance $d_f = 8$ in.

$P_{anchorx} = T_{ux}\,(d_p - t_f) / (Y_1 - H_{concrete})$

$= 151 * (8 - 7/16) / (41.5 - 24.0)$

$= 65$ kips

Point Load Due to Anchor Capacity in the y-direction: $P_{anchory}$

Compression is taken by the post section

Width of flange b_f=6.5"

Flange to flange distance $d_f = 8$ in.

$P_{anchory} = T_{uy}\,(b_f/2+1.5) / (Y_1 - H_{concrete})$

$= 109 * (6.5/2+ 1.5) / (41.5 - 24.0)$

$= 29.6$ kips

Controlling Post Capacity :

$P_{px} = \min(P_{bendx}, P_{anchorx}) = \min(82 \text{ kips}, 65 \text{ kips}) = 65$ kips

$P_{py} = \min(P_{bendy}, P_{anchory}) = \min(31 \text{ kips}, 30 \text{ kips}) = 30$ kips

(e) Critical Wall Nominal Resistance: R

Failure does not involve the end post of a segment

For failure involving an odd number of railing spans, N, Eqn. A13.3.2-1

$$R = [16 * M_p + (N-1) * (N+1) * P_p * S] / [2 * N * S - L_t]$$

For failure modes involving an even number of railing spans, N, Eqn. A13.3.2-2

$$R = [16 * M_p + N^2 * P_p * S] / [2 * N * S - L_t]$$

The results for various spans, N, are listed in the table below.

Number of Spans, N	Resistance R, kips
1	144
2	135
3	133
4	170
5	190
6	227

Impact at the End of Rail of Segents that Causes the End Post to Fail

For any number of railing spans, N, Eqn. A.13.3.2-3

$$R = 2. * M_p + 2 * P_p * S * \sum_{i=1}^{N} i \ / \ [2 * N * S - L_t]$$

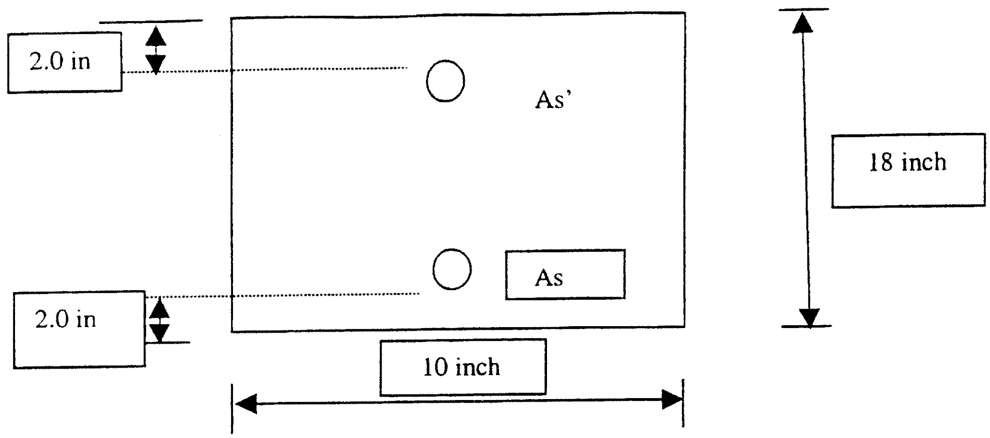
The results for various spans, N, are listed in the table below

Number of Spans	**Resistance R, kips**
1	158
2	139
3	162
4	191
5	221
6	252

Required Resistance $F_t = 124$ kips.

For failure spans N=1 to 6, the resistance requirements are met or exceeded by the capacity of the post and rail alone.

(f) Calculation of M_c



Area of reinforcing steel: $A_s = A_s' = 0.31$ sq. in. (1 #5 bars)

Diameter of reinforcing bars: $d_{bar} = 0.625$ in. for #5 bars

Concrete compressive strength: $f_c' = 3.5$ ksi

Steel yield stress: $F_y = 60$ ksi

Concrete Depth to Steel A_s: $d = 18 - (2.0 + 0.625/2) = 15.6875$ in

Concrete Width: $b = 10$ in

Neglecting the contribution of steel A_s'

Tensile force = Compressive force for concrete section

$$A_s * F_y = a * b * 0.85 * f_c'$$

$$0.31 * 60 = a * 10 * 0.85 * 3.5 \qquad \text{gives } a = 0.625 \text{ in}$$

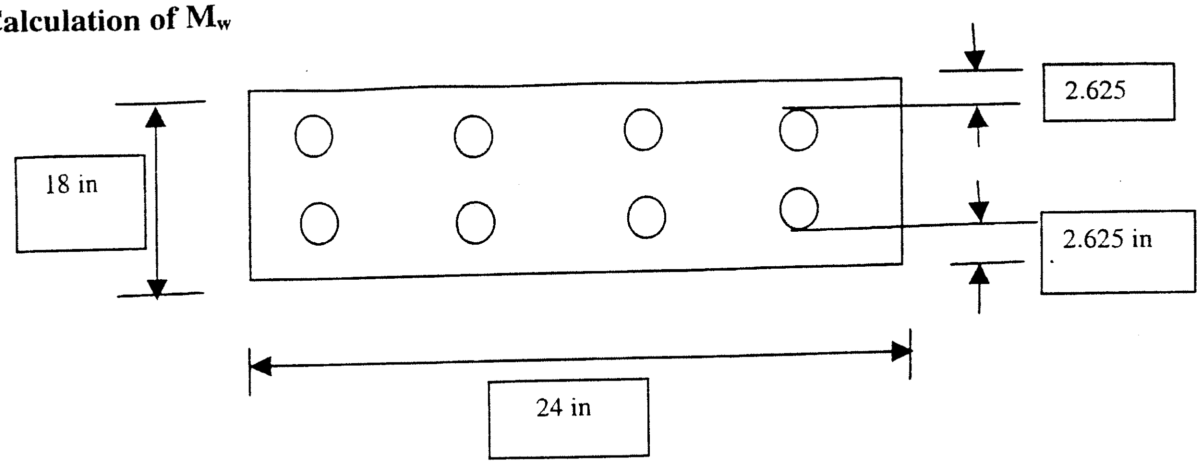
Moment resisted by the section: M_c

$$M_c = A_s * F_y * (d - a/2)$$

$$= 0.31 * 60 * (15.6875 - 0.625/2) = 286 \text{ kip-in per 10 inch width}$$

$$M_c = 286/12 * (12/10) = 28.6 \text{ kip-ft/ft}$$

(f) Calculation of M_w



Area of reinforcing steel: $A_s = A_s' = 0.8$ sq. in. (4 #4 bars)

Diameter of reinforcing bars: $d_{bar} = 0.5$ in for #5 bars

Concrete compressive strength: $f_c' = 3.5$ ksi

Steel yield stress: $F_y = 60$ ksi

Concrete Depth to Steel A_s: $d = 18 - (2.625 + 0.5/2) = 15.125$ in

Concrete Width: $b = 24$ in

Neglecting the contribution of steel A_s'

Tensile force = Compressive force for concrete section

$$A_s * F_y = a * b * 0.85 * f_c'$$

$$0.8 * 60 = a * 24 * 0.85 * 3.5 \qquad \text{gives } a = 0.67 \text{ in}$$

Moment resisted by the section: M_w

$$M_w = A_s * F_y * (d - a/2)$$

$$= 0.8 * 60 * (15.125 - 0.67/2) = 710 \text{ kip-in per 24 inch width}$$

$$M_w = 710/12 * (12/24) = 29.6 \text{ kip-ft/ft}$$

(g) Concrete Railing Resistance to Transverse load: R_w

For impacts within a wall segment

$L_c = L_t /2. + \text{sqrt}[(L_t/2)^2 + 8.* H * (M_b + M_w * H) / M_c]$ Eqn. A13.3.1-2

$R_w = [2./(2. * L_c - L_t)] * [8. * M_b + 8. * M_w * H + M_c * L_c^2 / H]$ Eqn. A13.3.1-1

Substitute M_c = 28.6 kip.-ft./ft.

M_w = 29.6 kip.-ft./ft.

M_b = 0. (no stiffening beam)

L_t = 8.0 ft.

H = 2.0 ft.

Then, L_c = 11.0 ft.

R_w = 315 kips.

For impacts at end of wall or a joint

$L_c = L_t /2. + \text{sqrt}[(L_t/2)^2 + H * (M_b + M_w * H) / M_c]$ Eqn. A13.3.1-4

$R_w = [2./(2. * L_c - L_t)] * [M_b + M_w * H + M_c * L_c^2 / H]$ Eqn. A13.3.1-3

Substitute M_c = 28.6 kip.-ft./ft.

M_w = 29.6 kip.-ft./ft.

M_b = 0. (no stiffening beam)

L_t = 8.0 ft.

H = 2.0 ft.

Then, L_c = 8.5 ft.

R_w = 243 kips.

(h) Combination Concrete Parapet and Metal Rail

Flexural Strength of Metal Rail Over One Span: $R_R = 144$ kips

Height of Rail Resultant: $H_R = 41.5$ inch

Flexural Strength of Metal Rail Over Two Spans: $R_R' = 135$ kips

Resistance of the post on top of the wall: $P_p = 65$ kips

Height of wall: $H_w = 24$ inch

Vehicle Impact at Midspan

Ultimate Capacity of Wall: $R_w = 315$ kips

Resultant Strength: $R_{combo} = R_R + R_w$, (A.13.3.3-1)

$= 144 + 315$

$= 459$ kips

Effective Height: $Y_{combo} = (R_R * H_R + R_w * H_w) / R_{combo}$ (A.3.3.3-2)

$= (144 * 41.5 + 315 * 24) / 459$

$= 29.5$ inch

Vehicle Impact at Post

Ultimate Capacity of Wall: $R_w = 243$ kips

Reduced Wall Strength: $R_w' = (R_w * H_w - P_P * H_R) / H_w$ (A.13.3.3-5)

$= (243 * 24 - 65 * 41.5) / 24 = 131$ kips

Resultant Strength: $R_{combo} = P_P + R_R' + R_w'$ (A.13.3.3-3)

$= 65 + 135 + 131$

$= 331$ kips

Effective Height: $Y_{combo} = (P_P * H_R + R_R' * H_R + R_w' * H_w) / R_{combo}$ (A.13.3.3-4)

$= (65 * 41.5 + 135 * 41.5 + 131 * 24) / 331$

$= 34.5$ inch

Condition	Resultant Strength, kips
Vehicle Impact at Midspan	459
Vehicle Impact at Post	331

Dimensions and properties for W8x28 (AISC LRFD Manual)

$d = 8.0$ in.

$t_w = 5/16$ in.

$b_f = 6.5$ in.

$t_f = 7/16$ in.

$b_f/2t_f = 7.0$

Also, properties for steel

$E = 29{,}000$ ksi

$F_y = 50$ ksi (345 Mpa)

Compact Section Check for W8x28

Compact section check is performed per Figure C6.10.4.-1, AASHTO code

D_{cp} = depth of the web in compression at the plastic moment

= $d/2 = 8/2 = 4.0$ in.

$2\,D_{cp}/t_w = 2 \times 4 / (5/16) = 25.6$

$\mathrm{sqrt}(E/F_y) = \mathrm{sqrt}(29{,}000/50) = 24.1$

Art. 6.10.4.1.2

$[2\,D_{cp}/t_w = 25.6\,]$ < $[3.76\ \mathrm{sqrt}(E/F_y) = 3.76 \times 24.1 = 90.6\,]$ O.K.

Art. 6.10.4.1.3.

$[b_f/2t_f = 7.0\,]$ < $[0.382\ \mathrm{sqrt}(E/F_y) = 0.382 \times 24.1 = 9.2\,]$ O.K.

Art. 6.10.4.1.6a

$[2\,D_{cp}/t_w = 25.6\,]$ < $[\,(0.75)\ 3.76\ \mathrm{sqrt}(E/F_y) = 0.75 \times 3.76 \times 24.1 = 68\,]$ O.K.

$[b_f/2t_f = 7.0\,]$ > $[\,(0.75)\ 0.382\ \mathrm{sqrt}(E/F_y) = 0.75 \times 0.382 \times 24.1 = 6.9\,]$

Go to Art. 6.10.4.1.6b

Art. 6.10.4.1.6b

[$2 D_{cp}/t_w + 9.35\ b_f/2t_f = 25.6 + 9.35 \times 7 = 91.1$] <

[$6.25\ sqrt(E/F_y) = 6.25 \times 24.1 = 151$] O.K.

Art. 6.10.4.1.7

L_b = height of post = 25 in = 25 x 25.4 mm = 635 mm.

r_y = 1.62 in. = 1.62 x 25.4 mm = 41.1 mm

[L_b = 635 mm] < [[$0.124 - 0.0759\ (M_l/M_p)$] [$r_y\ E/F_y$]

= [0.124 – 0] [41.1 x 29,000/50] = 2956 mm] O.K.

Use $M_n = M_p$.

Shear Check for Post W8x28

V_u = Factored Shear Load = 65 kips

Check performed per Figure C6.10.7.1-1, AASHTO

$D/\ t_w = 8\ /\ (5/16) = 25.6$

[$D/t_w = 25.6$] < [$2.46\ sqrt(E/F_y) = 2.46 \times 24.1 = 59.2$] O.K.

Use $V_n = V_p = 0.58\ F_y\ Dt_w$

Factored Shear Resistance $V_r = \phi_v\ V_n$

$\phi_v = 1.0$ (Art. 6.5.4.2)

$\phi_v\ V_n = \phi_v\ 0.58\ F_y\ t_w = 1.0 \times 0.58 \times 50 \times 8 \times 5/16 = 72.5$ kips

[$\phi_v\ V_n = 72.5$ kips] > [$V_u = 65$ kips] O.K.

Compact Section Check for W8x28

Refer Salmon and Johnson, Table 7.4.2, page 377

F_y = 50 ksi

$b_f/(2t_f)$ = 7.0

h_c/t_w = 22.2

$(\lambda = b_f/(2t_f) = 7.0) < (\lambda_p = 65/\mathrm{sqrt}(F_y) = 65/\mathrm{sqrt}(50) = 9.2)$ O.K

$(\lambda = h_c/t_w = 22.2) < (\lambda_p = 640/\mathrm{sqrt}(F_y) = 640/\mathrm{sqrt}(50) = 90.5)$ O.K.

Shear Check for Post W8x28

Depth of web = d = 8 in.

Thickness of Web = t_w = 5/16 in.

For rolled beam, T = 6.125 in.

Refer Salmon and Johnson, Eq. 7.7.10, page 393

Ratio h/tw = $T/t_w = 6.125/(5/16) = 19.6$

Ratio $418/\mathrm{sqrt}(F_y) = 418/\mathrm{sqrt}(50) = 59$

$(h/t_w = 19.6) < (418/\mathrm{sqrt}(F_y) = 59)$ O.K.

Refer Salmon and Johnson, Eq. 7.7.11, page 394

A_w = Area of web = $d\ t_w = 8 \times 5/16 = 2.5$ in.^{2}

V_n = Nominal Shear Strength = $0.6\ F_{yw}\ A_w$

= 0.6 x 50 x 2.5 = 75 kips

φ_v = 0.9

V_u = Factored Shear Load = 65 kips

$(\varphi_v\ V_n = 0.9 \times 75 = 67.5$ kips$) > (V_u = 65$ kips$)$ O.K.

Appendix E

Design Calculations for Details

JOB NO.

HDR Computation

HDR

Project: PA Traffic Barrier	Computed: MAH	Date: 5/22/02
Subject: Detail Design	Checked: JLG	Date: 5/24/02
Task: Post to Rail Connection	Sheet: 1	Of

Vertical Force; $F_v = 80^k$ distributed over 40 ft.
Ref: A13.2 [TL-5 Railing]
[98 LRFD AASHTO]

$80^k \div 40 ft = 2^{k}/ft$ @ 1 post $2^k/ft \cdot 7.5' = 15$ kips

℄ Post Spacing

$15 kips \div 2 studs = 7.5^k/stud.$

Studs are automatic welded, A307 grade studs.

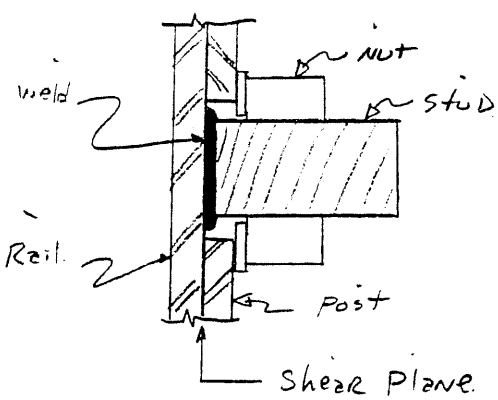
$F_u = 60 ksi$, (6.4.3.1)

AWS D1.1 7.2.2

Tube wall thickness $= 5/16"$ $5/16" \times 3 = 15/16"$ ∴ Max size $7/8"\,\phi$ stud.

Stud capacity: $\phi_s = 0.65$ (6.5.4.2)

6.13.2.7 Since the studs will have a weld at the base, the threads will not be in the shear plane.



try $3/4"\,\phi$

$R_N = 0.48\, A_b\, F_{ub}\, N_s$

F_u of bolt

℄ Area of bolt

℄ # of shear planes

$R_N = 0.48\,(0.442)(60 ksi)(1)$
$= 12.7^k$

$R_r = \phi_s R_n = 0.65\,(12.7^k)$
$= 8.3^k$

Req'd $= 7.5^k/stud$ < $R_r = 8.3^k/stud$ ✓ OK.

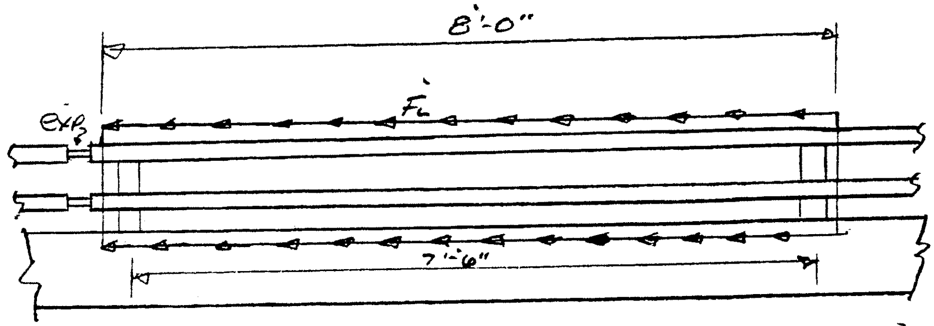
∴ Use 2 - 3/4" A307 studs per Rail to Post Connection.

HDR Computation

HDR

Project: PA Traffic Barrier	Computed: [illegible]	Date: 5/22/02
Subject: Detail Design	Checked: JLG	Date: 5/24/02
Task: Post to Rail Connection	Sheet: 2	Of

Longitudinal Force, $F_L = 41$ kips A13.2 [TL-5 railing]
distributed over 8'



- Assume the continuous rail will distribute this load to two posts. This assumption is conservative even if load is located immediately adjacent to a rail exp jt.
[Rails are continuous over 3 posts. ∴ If load is centered over a post, it would be dist. over 3 posts - not 2.]

∴ 4 rail to post connections
2 studs per connection
∴ $41^k/(2 \cdot 4) = 5.125\ ^k/\text{stud}$

see Vertical Force Design for stud capacity.

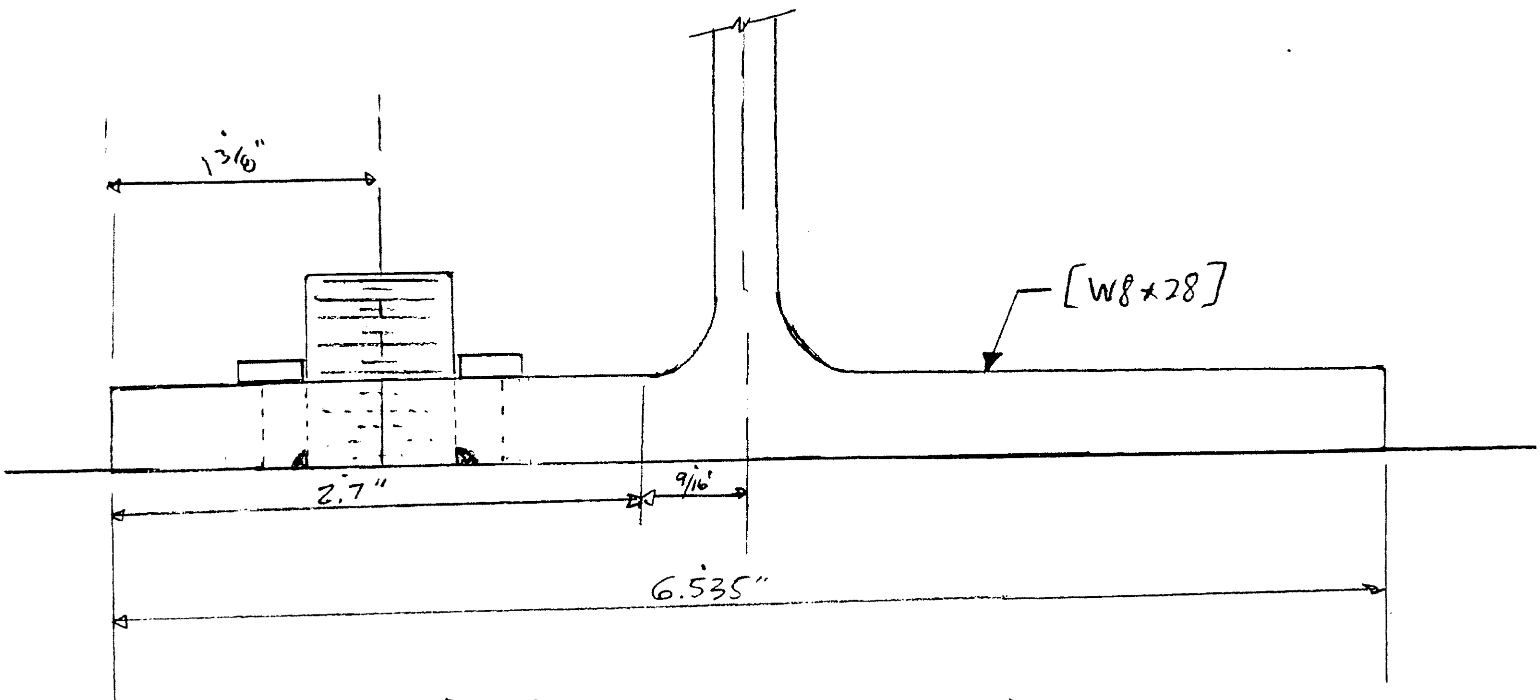
$\text{Req'd} = 5.2\ ^k/\text{stud} < R_r = 8.3\ ^k/\text{stud}$ ✓OK.

∴ 2 - 3/4" ⌀ A307 studs per rail to post connection is acceptable.

- Note: Since the studs are grade A307, they have been designed as a bearing connection. However, the post holes must be oversized to clear the stud weld. The oversized holes with the bearing connection will be accepted in this case because the maximum design load represents an extreme event (vehicle collision) and large displacements and deformations are expected. ∴ slippage of the bolts to engage will be accepted.
Bolt hole size: $3/4" + 2(1/8") + 1/4" =$ say $1\ 1/4"$ ⌀
- Provide locking mechanism on studs (lock washers, locking compound

HDR Computation HDR

Project: PA Traffic Barrier	Computed: MJH	Date: 5/22/02
Subject: Detail Design	Checked: JLG	Date: 5/25/02
Task: Post to Rail Connection	Sheet: 3	Of



$d = 3/4''$ bolt diameter
$t = 7/16''$ Flange thickness
$F_u = 58$ ksi [65 ksi]
$L_c = 1\,3/8'' - 3/4'' = 0.625$ in. [0.625''] [0.75'']

(6.13.2.9-2)

$R_n = 1.2 L_c t F_u$
$= 1.2 (0.625'')[0.75] (7/16'') 58$ ksi [65]
$R_n = 19.03$ k [25.6 k]

$R_r = \phi_{bb} R_n$ $\phi_{bb} = 0.8$ (6.5.4.2)

$R_r = 0.8 (19^k)[25.6] = 15.2^k$ [20.5 k]
← Conserv., OK

R_r for the connected material is greater that the stud capacity. ✓ O.K.

HDR Computation

HDR

Project: PA Traffic Barrier	Computed: MBH	Date: 5/23/02
Subject: Detail Design	Checked: JLG	Date: 5/25/02
Task: Anchor Bolts.	Sheet	Of

- Prying on the anchor bolts can be neglected based on the following rationale:

  1. Geometry of base plate: The anchor bolts are loacated off the tips of the flange of the post. The post flange acts to stiffen the base plate to resist the deformations that induce prying. I.e. the baseplate will behave as a thicker plate because of the stiffening effect of the flange.

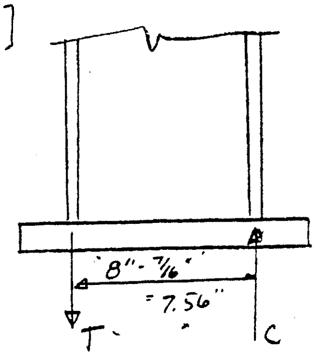
  2. The anchor bolts have very long grip lengths (15±) when compared to short grip lengths of standard bolted connections. The long grip lengths will add axial flexibility to the anchor bolts, thus reducing the prying induced stress.

HDR Computation HDR

Project: PA Traffic Barrier	Computed: MJd	Date: 5/23/02
Subject: Detail Design	Checked: JLG	Date: 5/28/02
Task: Post to Base PL Weld.	Sheet: 1	Of

Design weld for Limiting Capacity of Post
-check for F_T and F_L

[$< M_p = 1360$ k·in]



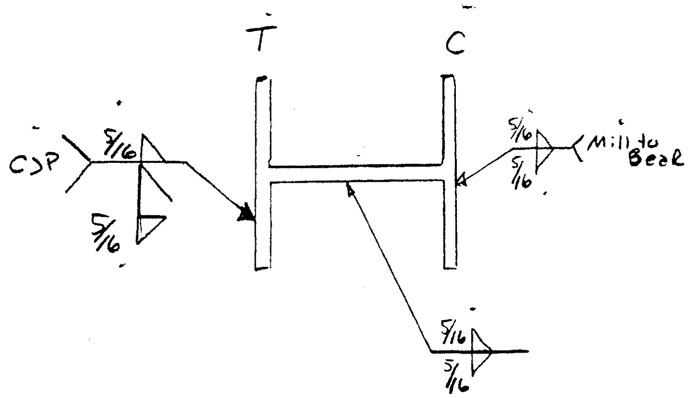
Post $M = 66^k \cdot 16.5_{in} = 1089$ kin

$1089^{\ k\cdot in} \div 7.56''$
$= 144^{\ k}$ Tension

$\frac{144^k}{6.5} = 22.2$ k/in ← too large for a reasonable size double fillet. Use CJP.

6.5 ← FL width

- To ensure full M_p of Post use CJP of Tension flange.
  Use double fillets on web and compression flange.



Weld Metal:
$R_n = 0.6\ \phi_{e2}\ F_{exx}$ (6.13.3.2)
$= 0.6(0.8)(70\text{ksi})$
$= 33.6$ ksi

Base Metal (6.13.5.3)
$R_r = \phi_v\ 0.58\ A_g\ F_y$ ← web DIMS ← Post F_y
$= 1.0(0.58)(0.285'' \cdot 8.06)$ 50ks
$= 66.6$ kips ✓OK.

- Check fillets for F_T and F_L
  $F_T = 66$ kips.

2 - 5/16" fillets $= 2 \cdot 5/16''(0.707) \cdot 33.6\text{ ksi} = 14.8$ k/in
66 kips / 14.8 k/in = 4.5 in of double 5/16" fillet on web. ✓OK.

HDR Computation

HDR

Project: PA Traffic Barrier	Computed: WJH	Date: 5/24/02
Subject: Detail Design	Checked: JLG	Date: 5/28/02
Task: Post to Base PL Weld	Sheet: 2	Of:

F_L = 41 kips ÷ 2 posts = 20.5 kips

20.5 kips ÷ 2 flanges = 10.25 kips.

Rear (compression flange) = double fillet.

10.25 ÷ 14.8 k/in = 0.69" of weld. ∴ 5/16" double fillet on entire flange is Adequate.

capacity of 5/16" double fillet

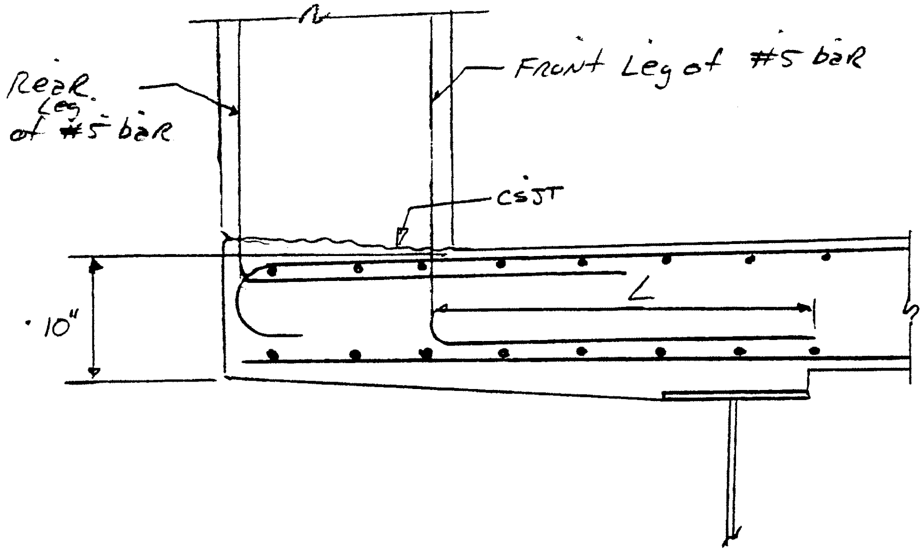
front (tension flange) = CJP.

∴ OK By inspection since CJP has more capacity than double fillets.

HDR Computation

HDR

Project: PA TRAFFIC RAIL	Computed: WJH	Date: 5/22/02
Subject: BARRIER Details	Checked: JLG	Date: 5/23/02
Task: Rebar Development.	Sheet:	Of:



#5 standard hook. Ref. PENNDOT BRIDGE Construction STANDARDS BC-736M 1999

Deck Concrete 4000psi

#5 bar $\quad L_{dh} = 12''$

for front Leg: All sides have at least 2½" clr cover: use 0.7
epoxy coated: use 1.2

$$L_{dh} = 12'' \cdot 0.7 \cdot 1.2 = 10.08''$$

since deck is only 10" thick, this #5 bar cannot be fully developed with a standard hook.

∴ Set hook length (L) as a development length: 1'-11"
∴ L = 1'-11"

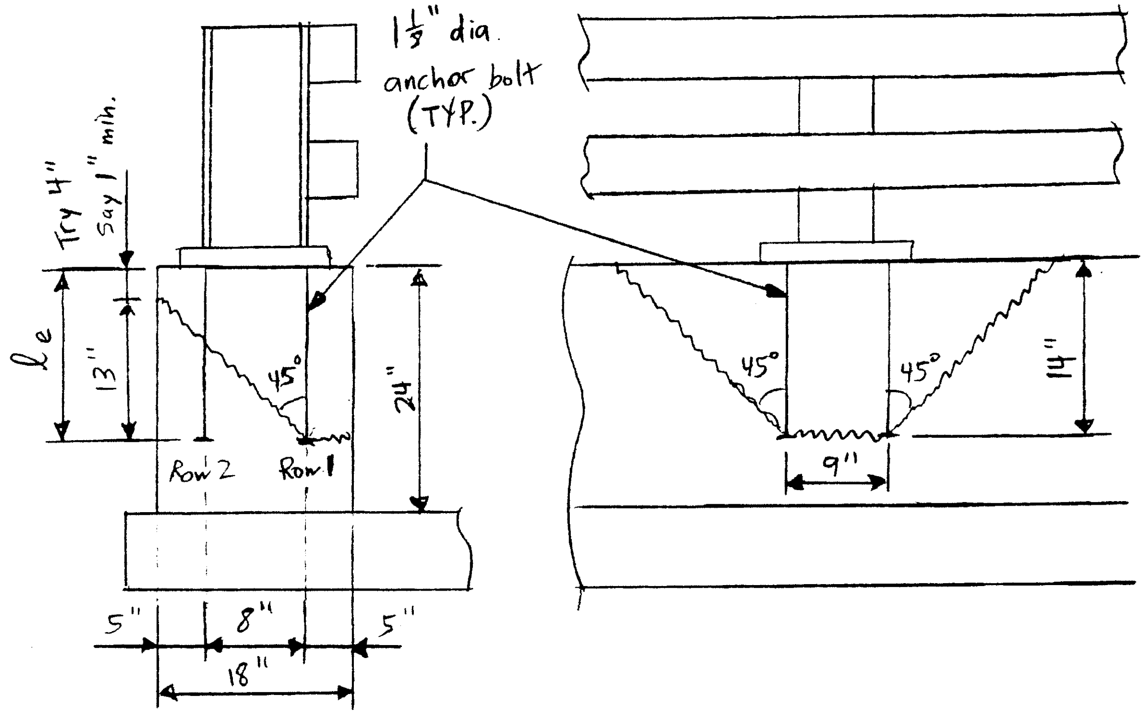
Rear Leg bar: Since bar is in compression under max load 100% development @ const. joint is not Required. Use same length Leg (1'-11") as Front Leg and hook at top mat to match BR27C detailing.

HDR Computation HDR

Project	Fort Pitt Bridge	Computed JLG	Date 5/23/02
Subject	Pennsylvania Barrier	Checked JEC	Date 6/2/02
Task	Anchor Bolt Embeddment	Sheet 1	Of

Determine Minimum Anchor Bolt Length

- Assume 45° slope failure from the tip of the bolt.
- Want crack to reach back face of parapet before it reaches the top face (see sketch below).



Transverse Elev. View

Longit. Elev. View

- Anchor bolts only in Row 1 see tension, ∴ Row 2 anchor bolts are ignored.

- Assume anchor bolt embeddment, $\underline{\underline{\ell_e = 17.0"}}$

(Does not include head of bolt.)

HDR Computation HDR

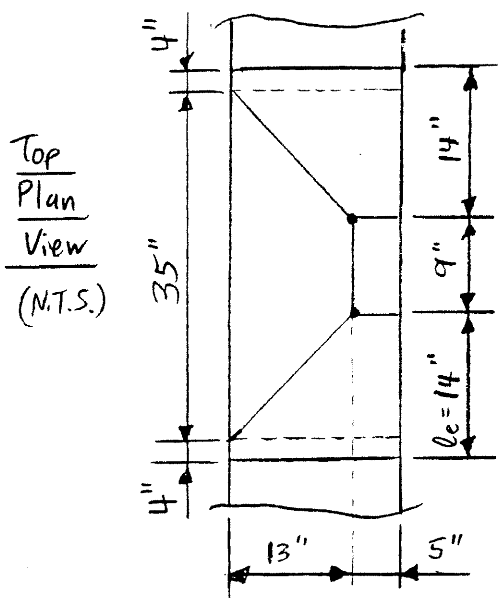
Project: Fort Pitt Bridge	Computed: JLG	Date: 5/23/02
Subject: Pennsylvania Barrier	Checked: JEC	Date: 6/2/02
Task: Anchor Bolt Embeddment	Sheet: 2	Of

Pull out strength of Concrete

- Reference PCI Manual, Design and Typical Details of Connections for Precast and Prestressed Concrete (1988)

Case I: Truncated Pyramid Failure

- Failure planes are shown in sketches on previous sheet



$$A_{flat} = (5")(9") = 45 \text{ in}^2$$

$$A_{slope} = [9" + 2(5")]\sqrt{2}(13") + 2\sqrt{2}(13")(13") + 2\sqrt{2}(4")(18") = 1031.0 \text{ in}^2$$

(PCI Eq. 4.11.5)

Design tensile strength $= \phi P_c = \phi \lambda \sqrt{f'_c}\,(2.8 A_{slope} + 4 A_{flat})$

$= (0.85)(1.0)\sqrt{3500}\,[2.8(1031.0 \text{ in}^2) + 4(45 \text{ in}^2)]$

$= 154\,219$ lb

$= \underline{\underline{154 \text{ k}}}$ for two bolt group

$\phi = 0.85$

$\lambda = 1.0$ for normal weight concrete (A 5.8.4.2)

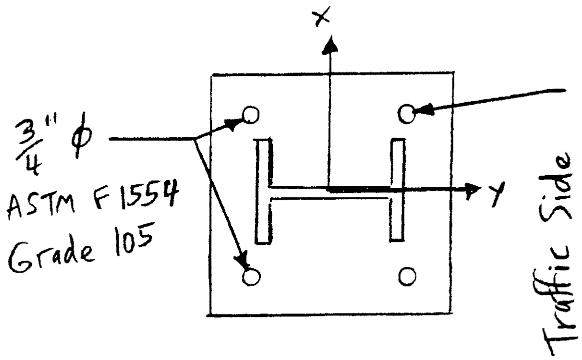
$f'_c = 3500$ psi

HDR Computation

HDR

Project: Fort Pitt Bridge	Computed: JLG	Date: 5/22/02	
Subject: Pennsylvania Barrier	Checked: JEC	Date: 6/2/02	
Task: Anchor Bolt Tensile Capacity	Sheet: 1	Of: 2	

Pennsylvania Barrier Plan View



$1\frac{1}{8}$" DIA ANCHOR BOLT ~~(TYP.)~~ [Traffic side]
ASTM F 1554 Grade 105.

Bolt Specs.

$F_{ub} = 125$ ksi.

$$A_b = \frac{\pi}{4}(1.125")^2 = 0.994 \text{ in}^2.$$

Factored Tensile Resistance for One Bolt ($1\frac{1}{8}$" φ)

$$T_r = \phi T_n \qquad (A\ 6.13.2.2)$$
$$= \phi_t (0.76) A_b F_{ub} \qquad (A\ 6.13.2.10.2)$$
$$= (0.8)(0.76)(0.994 \text{ in}^2)(125 \text{ ksi}) \qquad (A\ 6.5.4.2 \ \& \ 6.5.5) \qquad \phi_t = 0.8$$
$$= \underline{\underline{75.5 \text{ k/bolt}}}$$

Factored Tensile Resistance for Two Bolts (bending about x axis)

- Two $1\frac{1}{8}$" φ bolts in tension if bending occurs about x axis

$$T_{r\,(total)} = 2(75.5 \text{ k}) = \underline{\underline{151 \text{ k}}} \quad (\text{x-direction})$$

HDR Computation

HDR

Project: Fort Pitt Bridge	Computed: JLG	Date: 6/3/02	
Subject: Pennsylvania Barrier	Checked: JEC	Date: 6/3/02	
Task: Anchor Bolt Tensile Capacity	Sheet: 2	Of: 2	

Factored Tensile Resistance for One Bolt ($\frac{3}{4}$" ϕ)

$$A_b = \frac{\pi}{4}(0.75")^2 = 0.442 \text{ in}^2$$

$T_r = \phi T_n$ (A 6.13.2.2)

$= \phi_t (0.76) A_b F_{ub}$ (A 6.13.2.10.2)

$= (0.8)(0.76)(0.442 \text{ in}^2)(125 \text{ ksi})$ (A 6.5.4.2) $\phi_t = 0.8$

$= \underline{\underline{33.6 \text{ k/bolt}}}$

Factored Tensile Resistance for Two Bolts (bending about y axis)

- One $1\frac{1}{8}$" bolt and one $\frac{3}{4}$" ϕ bolt in tension

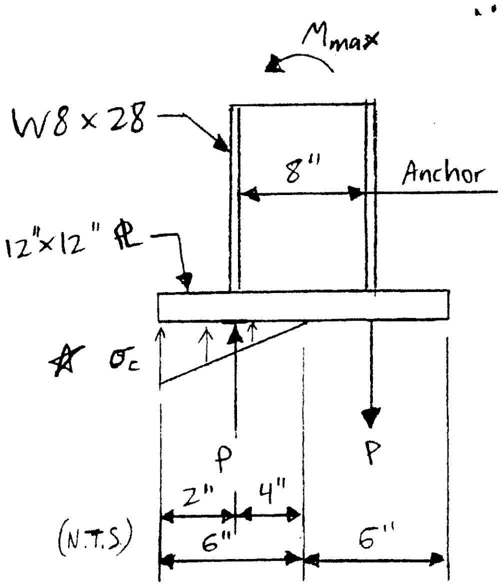
$T_{r\,(total)} = 75.5 \text{ k} + 33.6 \text{ k} = \underline{\underline{109.1 \text{ k}}}$ (y-direction)

HDR Computation

Project: Fort Pitt Bridge	Computed: JLG	Date: 5/24/02	
Subject: Pennsylvania Barrier	Checked: JEC	Date: 6/2/02	
Task: Plate Capacity	Sheet: 1	Of: 2	

Check Bending in the Plate

- Max tranverse design force on post = 66 k. This corresponds to the case where the $1\frac{1}{8}''\ \phi$ bolts are at full capacity in tension. ∴ Use max 2-bolt tension, P = 151 k, to determine the stress in the plate.



(Traffic Side)

- To determine σ_c, assume that the centroid of the bearing pressure acts at the same location as P.

$$\tfrac{1}{2}\sigma_c(6'')(12'') = P = 151\ k$$

$$\sigma_c = \frac{2(151\ k)}{(6'')(12'')} = 4.2\ ksi$$

★ Note: Assume bearing pressure varies linearly from center of plate to edge and is distributed across the entire width of the plate.

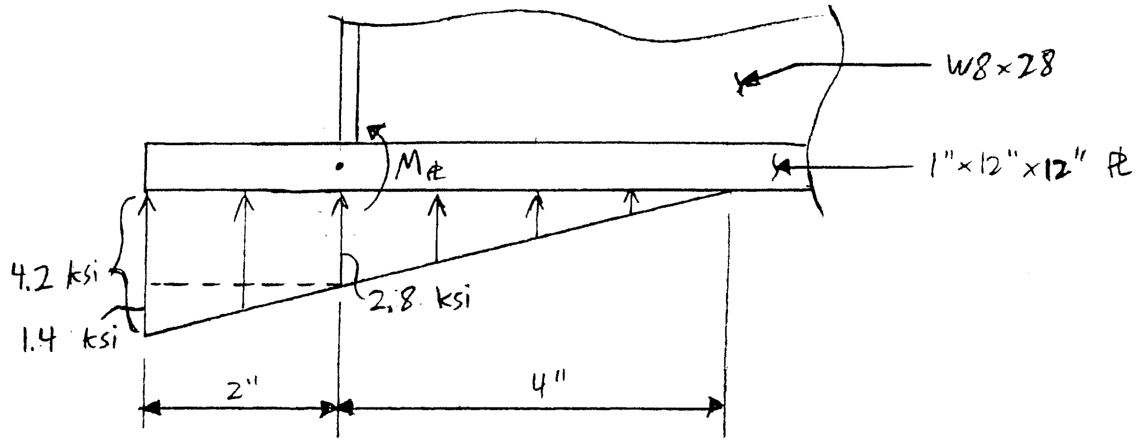
Computation

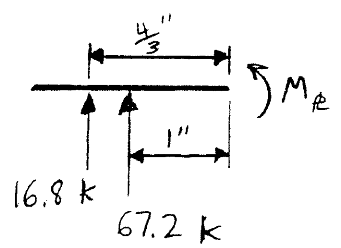
HDR

Project	Fort Pitt Bridge	Computed	JLG	Date	5/24/02
Subject	Pennsylvania Barrier	Checked	JEC	Date	6/2/02
Task	Plate Capacity	Sheet	2	Of	2

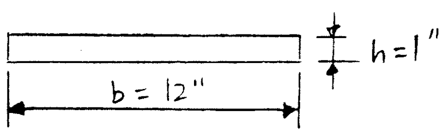
Check Bending in the Plate (cont.)

- Treat ℘ as cantilever. Check stress at section at edge of flange.





$$M_{℘} = (67.2\text{ k})(1") + (16.8\text{ k})(\tfrac{4}{3}") = 89.6\ \text{k·in}$$



$$S_{℘} = \tfrac{1}{6}bh^2 = \tfrac{1}{6}(12")(1")^2 = 2\ \text{in}^3$$

$$\sigma_{℘} = \frac{M_{℘}}{S_{℘}} = \frac{89.6\ \text{k·in}}{2\ \text{in}^3} = \underline{\underline{44.8\ \text{ksi}}} < F_y = 50\ \text{ksi} \quad \therefore \text{OK}$$

(for ℘ in Flexure, $\phi_f = 1.0$)