DEPARTMENT OF TRANSPORTATION

Appendices for Review and Assessment of Past MnDOT Bridge Barrier Types

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Research Project
Final Report 2020-24A

APPENDIX A: SITE VISIT PHOTOS AND DETAILS OF MNDOT BARRIERS

APPENDIX A1: J BARRIER - PHOTOS TAKEN FROM THE SITE VISIT AND DETAILS RECEIVED FROM MNDOT

Bridge No. 27169

Figure A1-1 Bridge No. 27169 Photos.

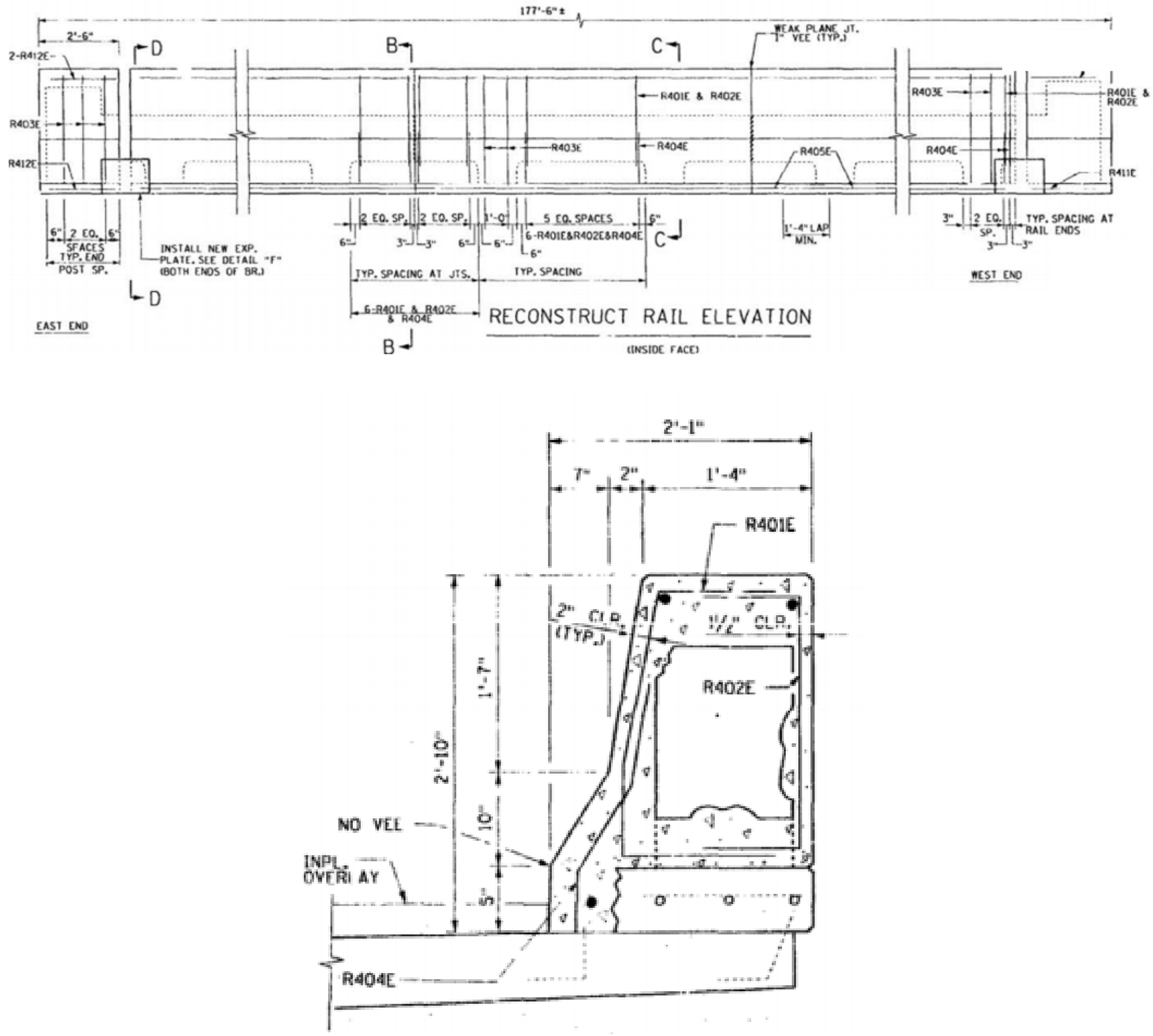
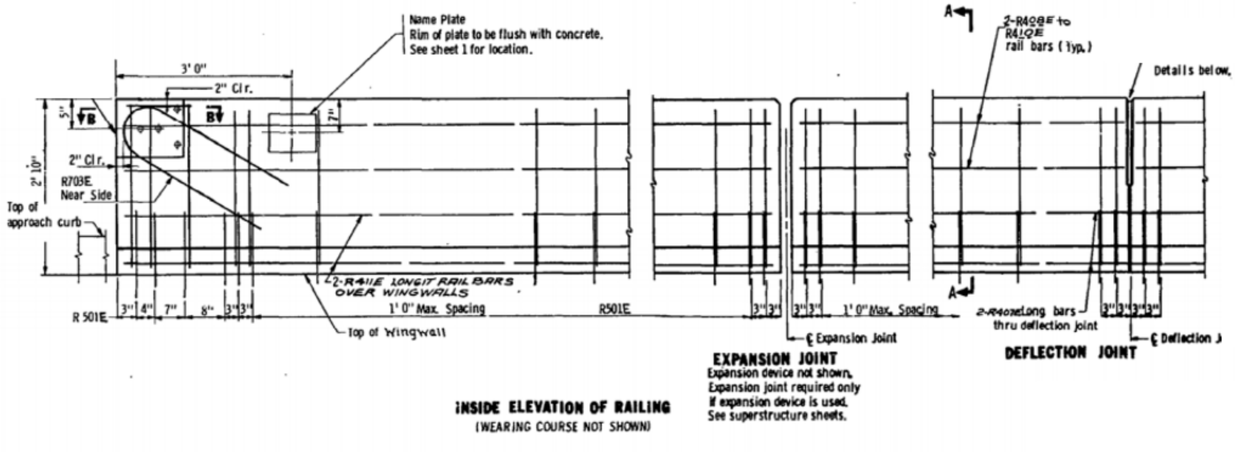


Figure A1-2 Bridge No. 27169 Details.

Bridge No. 19056

Figure A1-3 Bridge No. 19056 Photos.



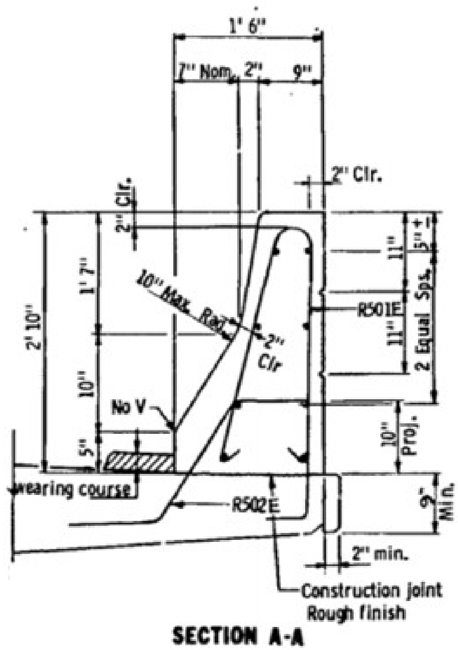


Figure A1-4 Bridge No. 19056 Details.

Bridge No. 82502

Figure A1-5 Bridge No. 82502 Photos.

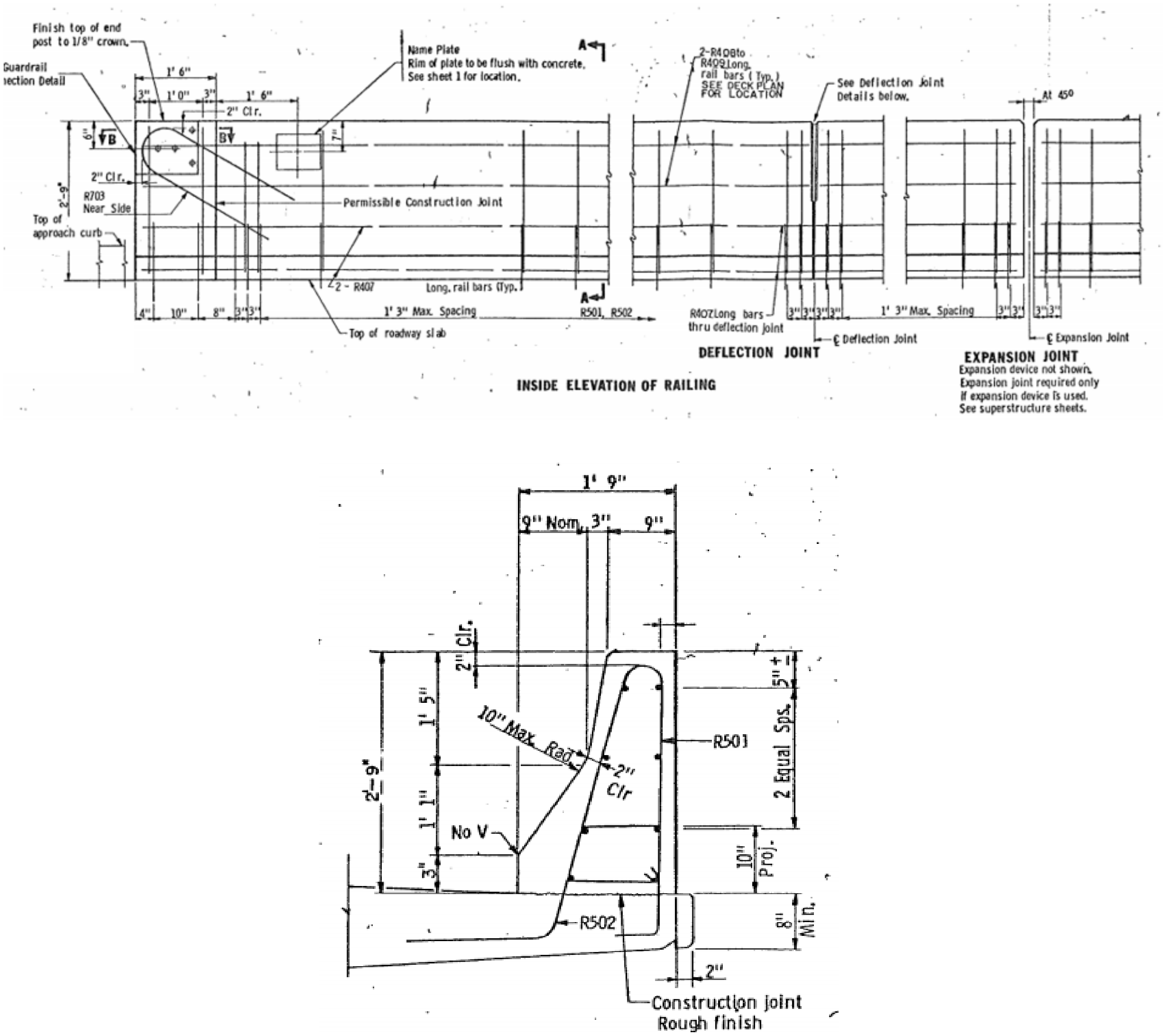


Figure A1-6 Bridge No. 82502 Details.

Bridge No. 19042

Figure A1-7 Bridge No. 19042 Photos.

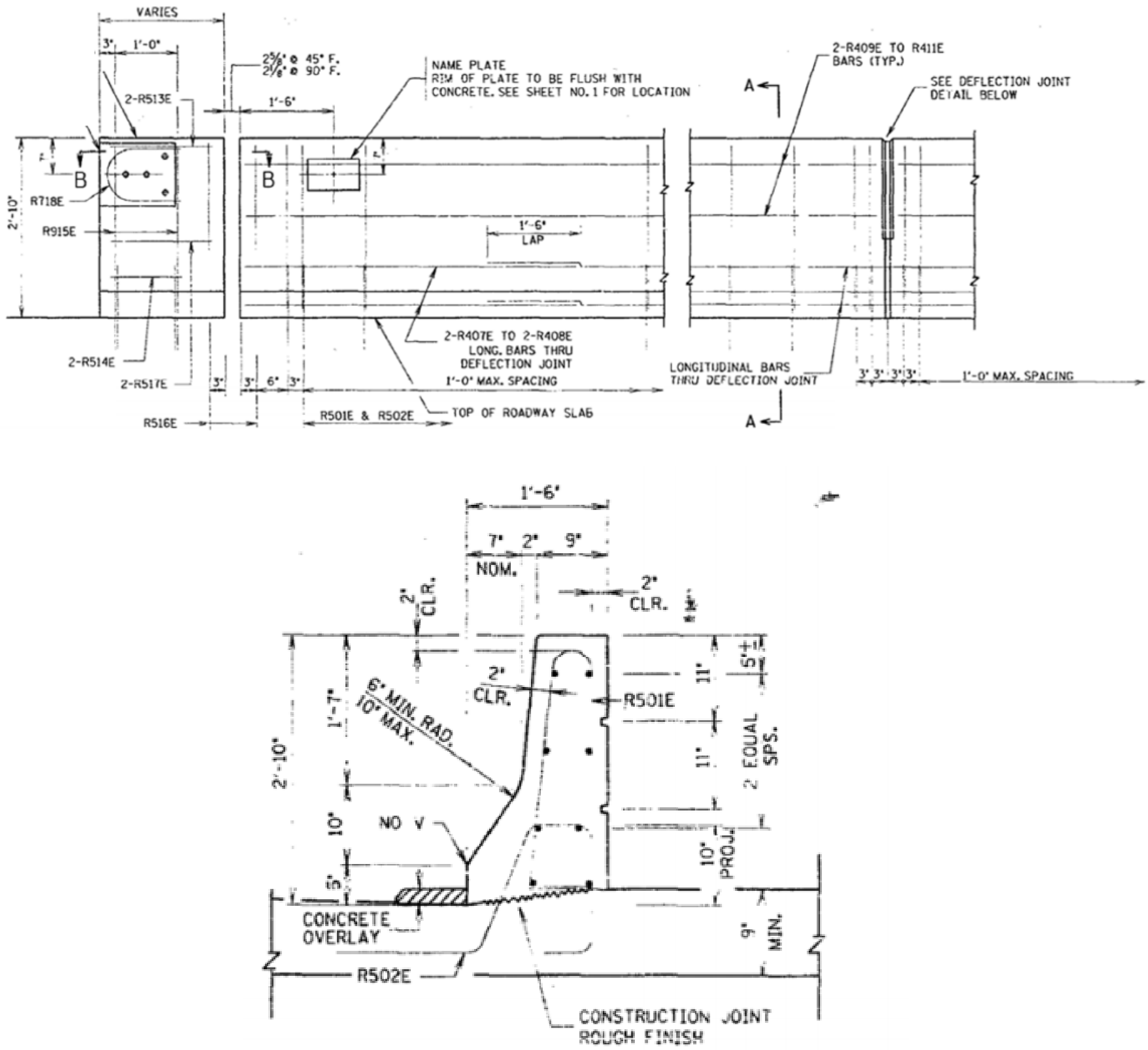


Figure A1-8 Bridge No. 19042 Details.

Bridge No. 62828

Figure A1-9 Bridge No. 62828 Photos.

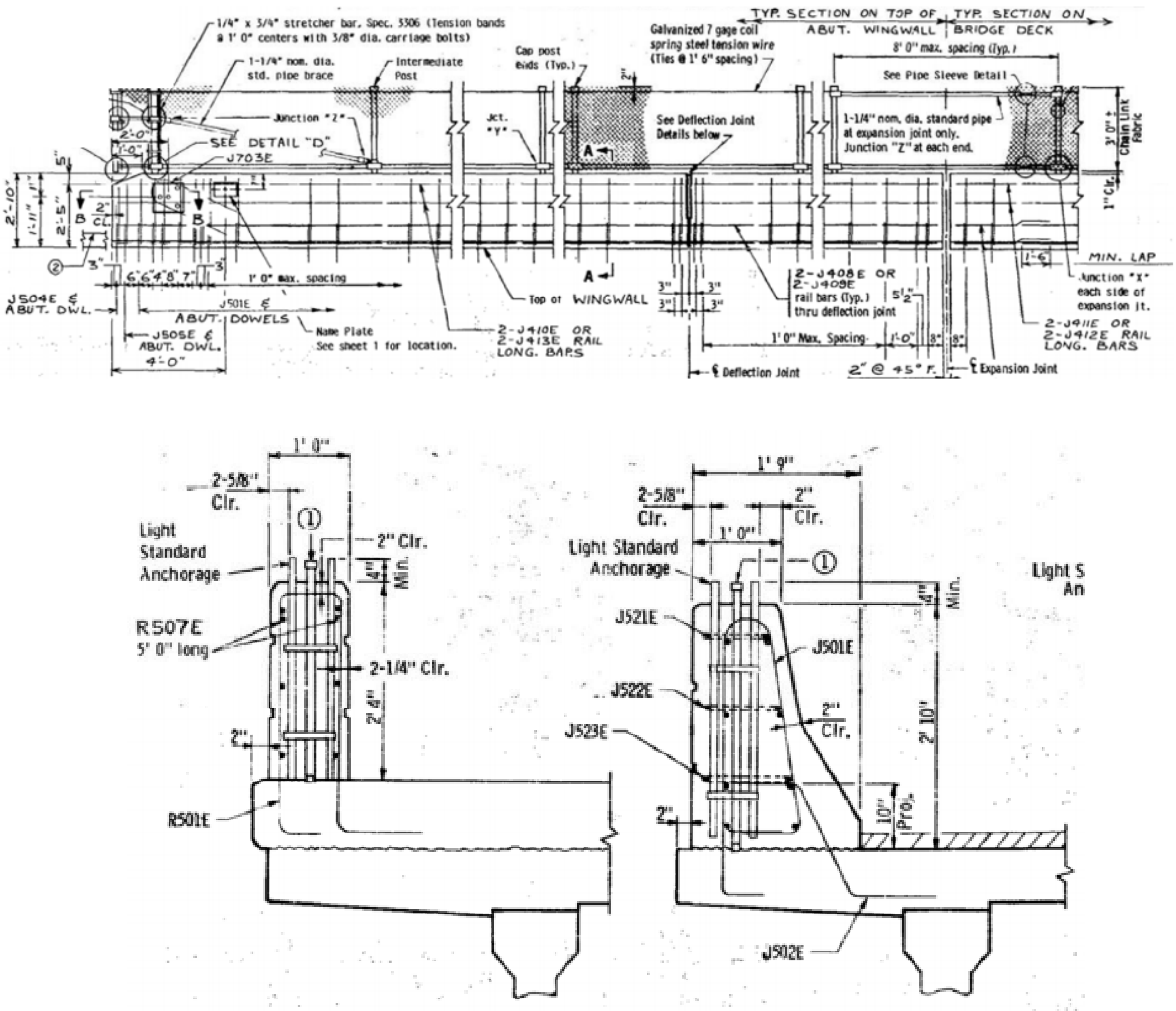
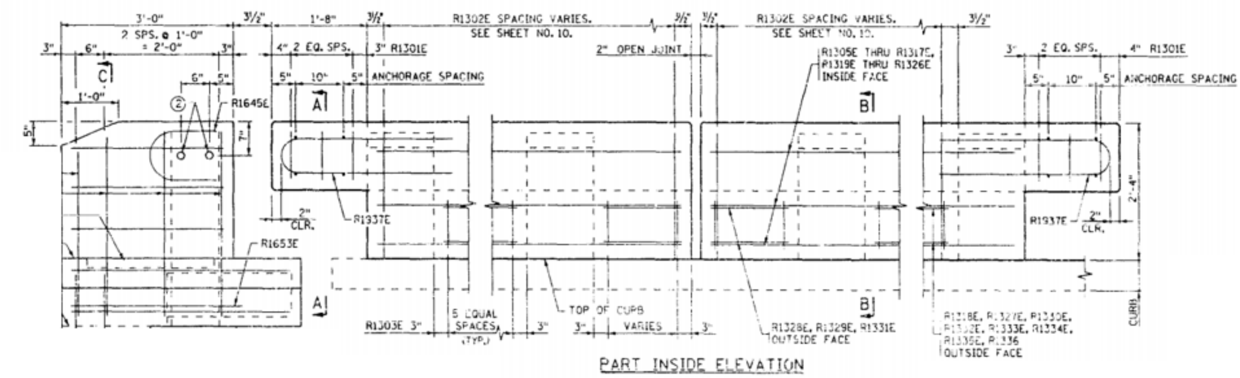


Figure A1-10 Bridge No. 62828 Details.

APPENDIX A2: ONE LINE BRIDGE RAIL - PHOTOS TAKEN FROM THE FIELD SITE VISIT AND DETAILS RECEIVED FROM MNDOT

Bridge No. 27944

Figure A2-1 Bridge No. 27944 Photos.



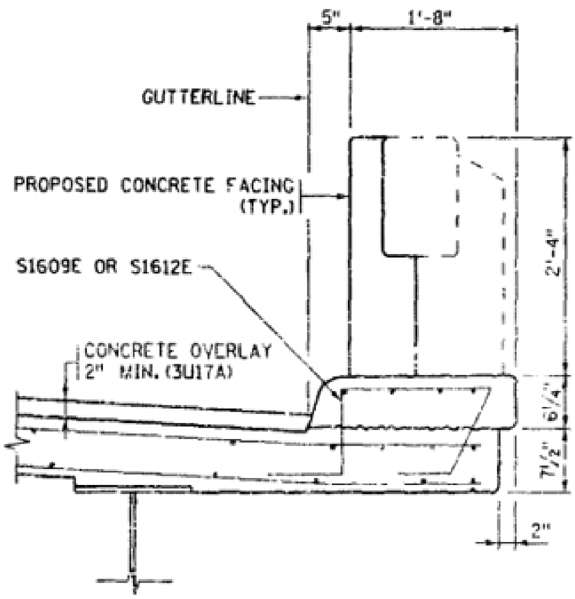


Figure A2-2 Bridge No. 27944 Details.

Bridge No. 30505

Figure A2-3 Bridge No. 30505 Photos.

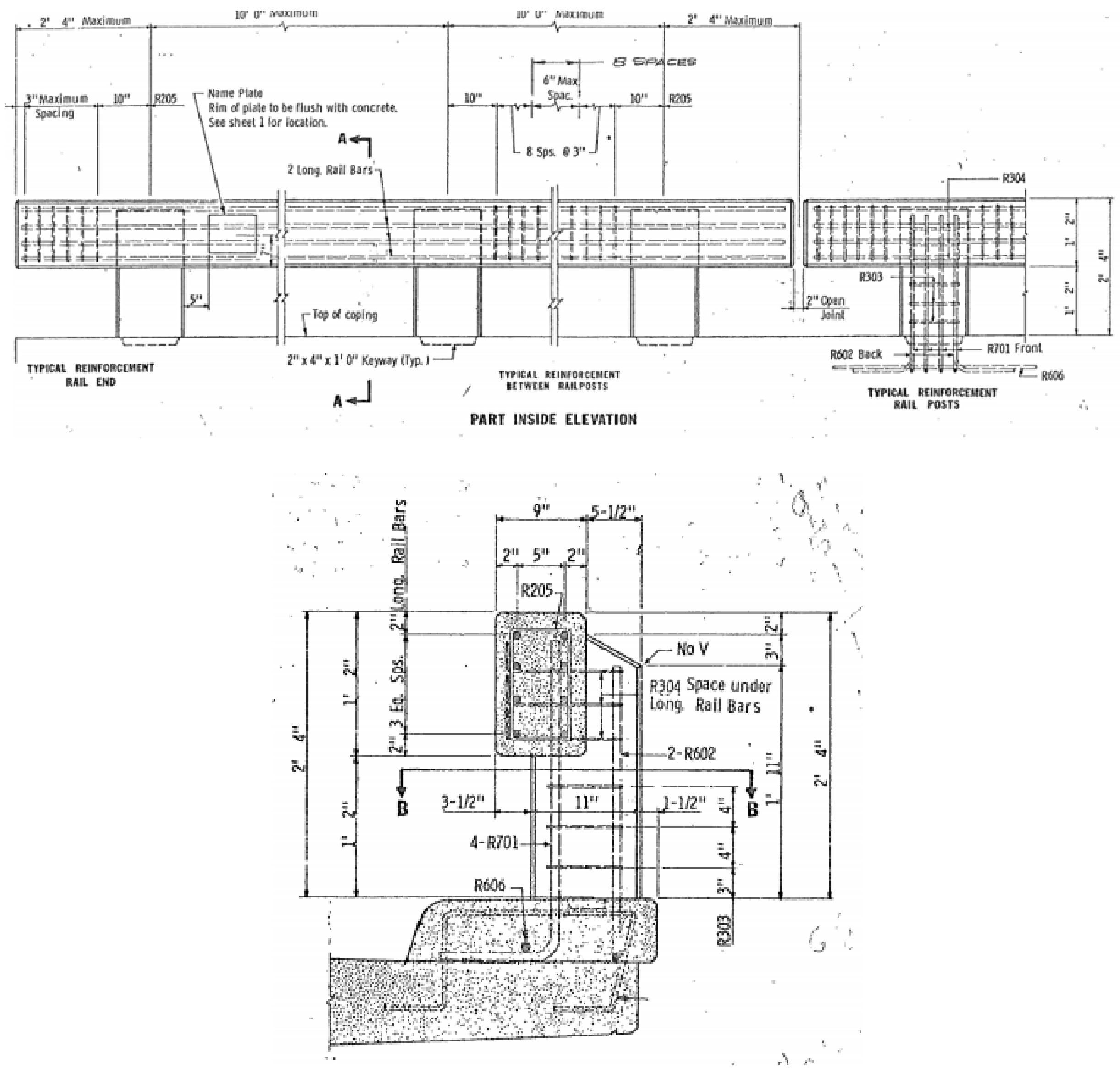


Figure A2-4 Bridge No. 30505 Details.

Bridge No. 69834

Figure A2-5 Bridge No. 69834 Photo.

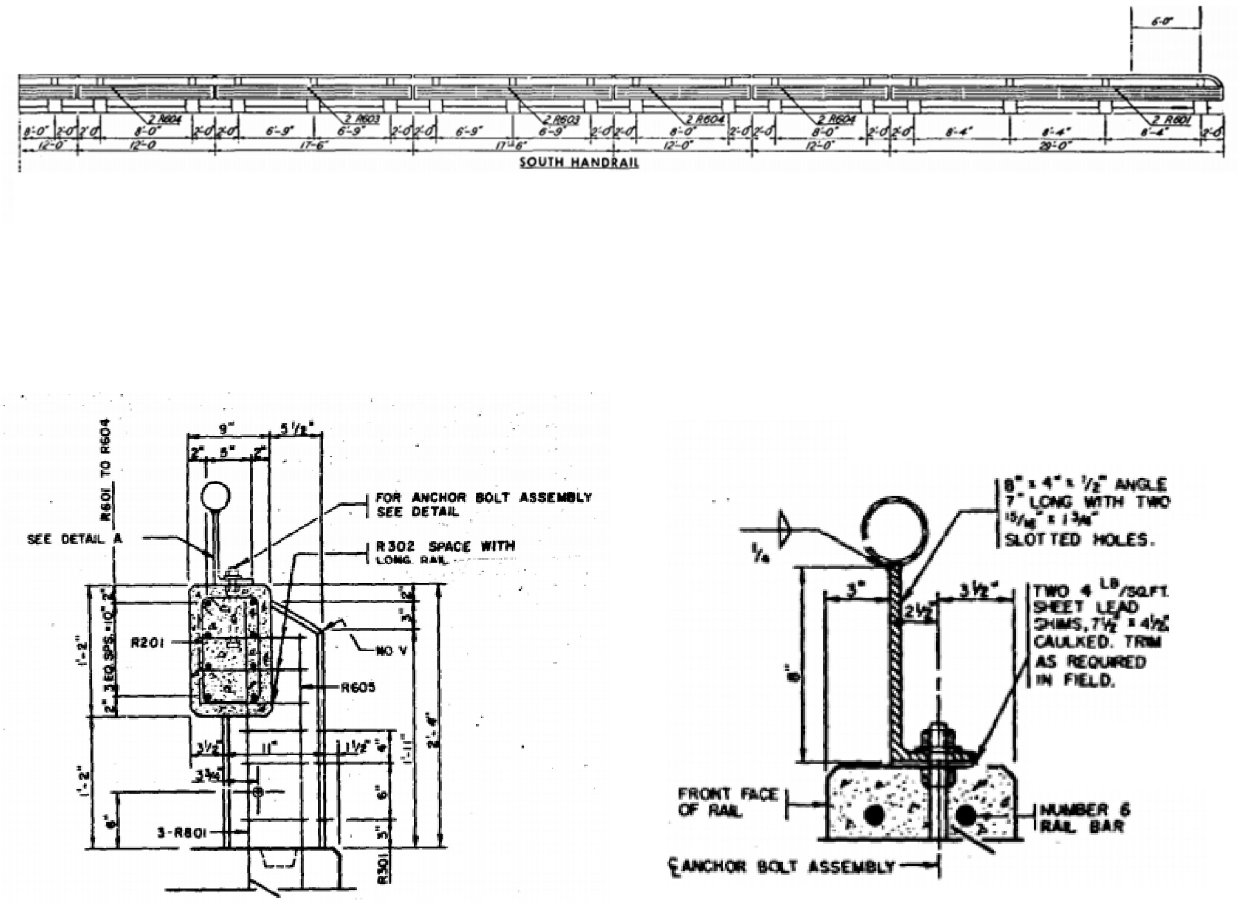


Figure A2-6 Bridge No. 69834 Details.

Bridge No. 70802

Figure A2-7 Bridge No. 70802 Photos.

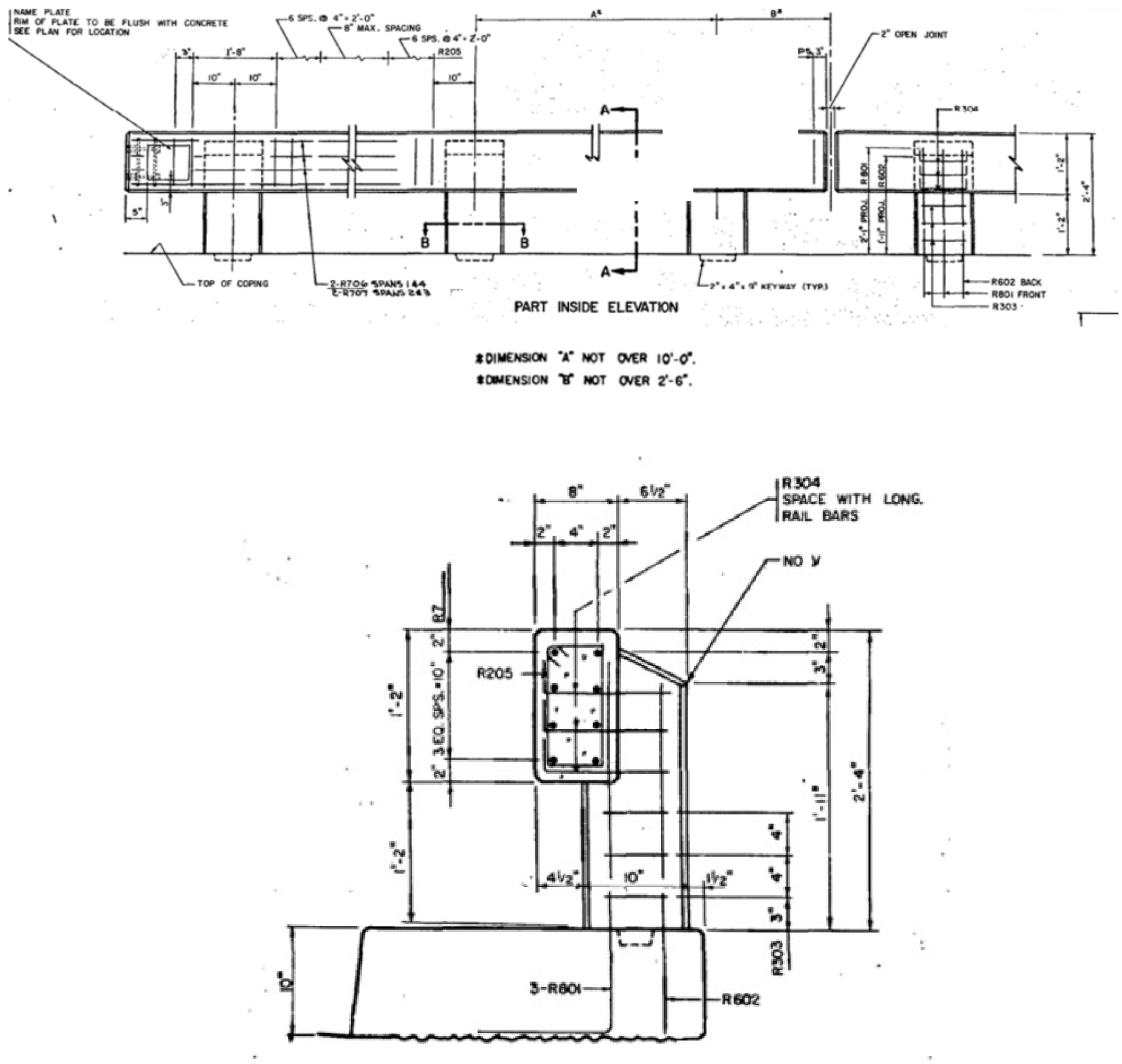


Figure A2-8 Bridge No. 70802 Details.

Figure A2-9 Bridge No. 25505 Photos.

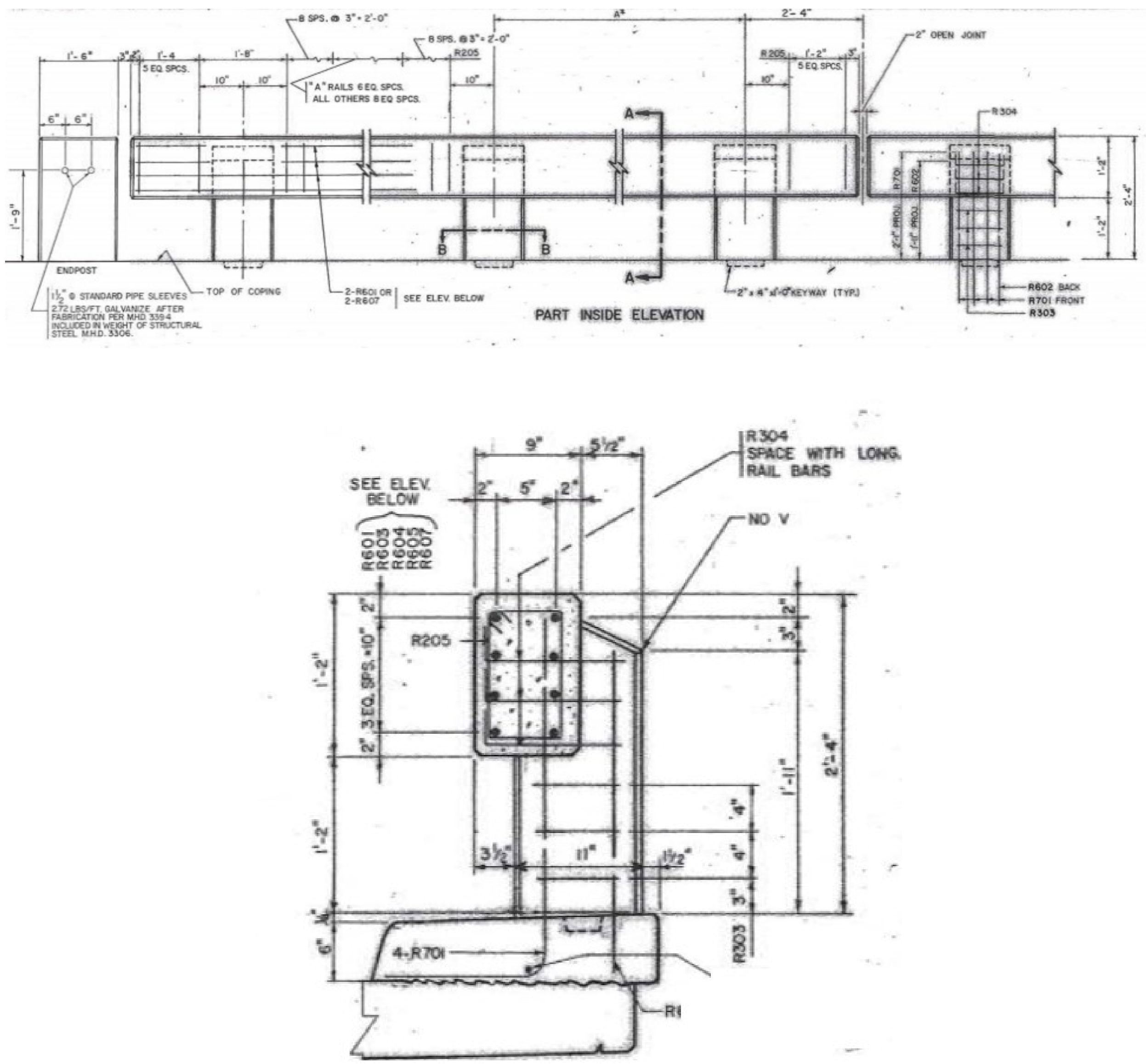
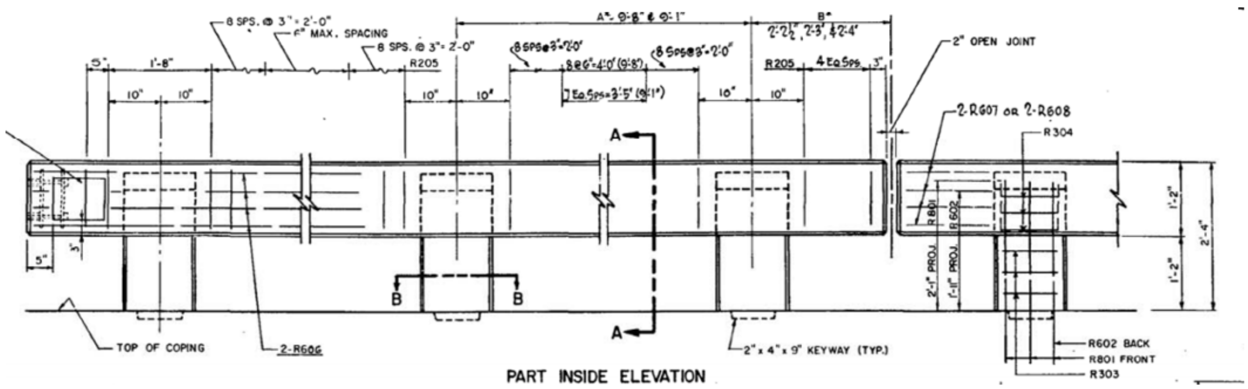


Figure A2-10 Bridge No. 25505 Details.

Bridge No. 82804

Figure A2-11 Bridge No. 82804 Photo.



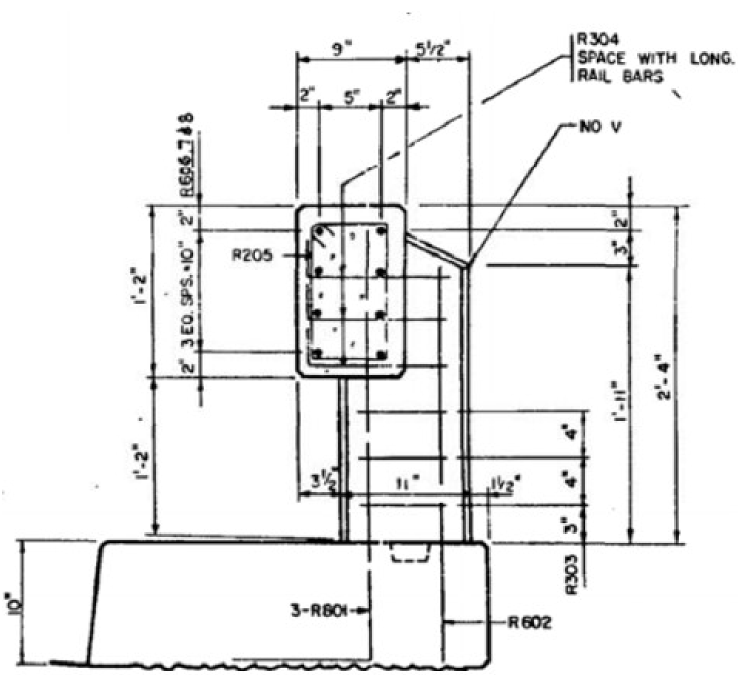


Figure A2-12 Bridge No. 82804 Details.

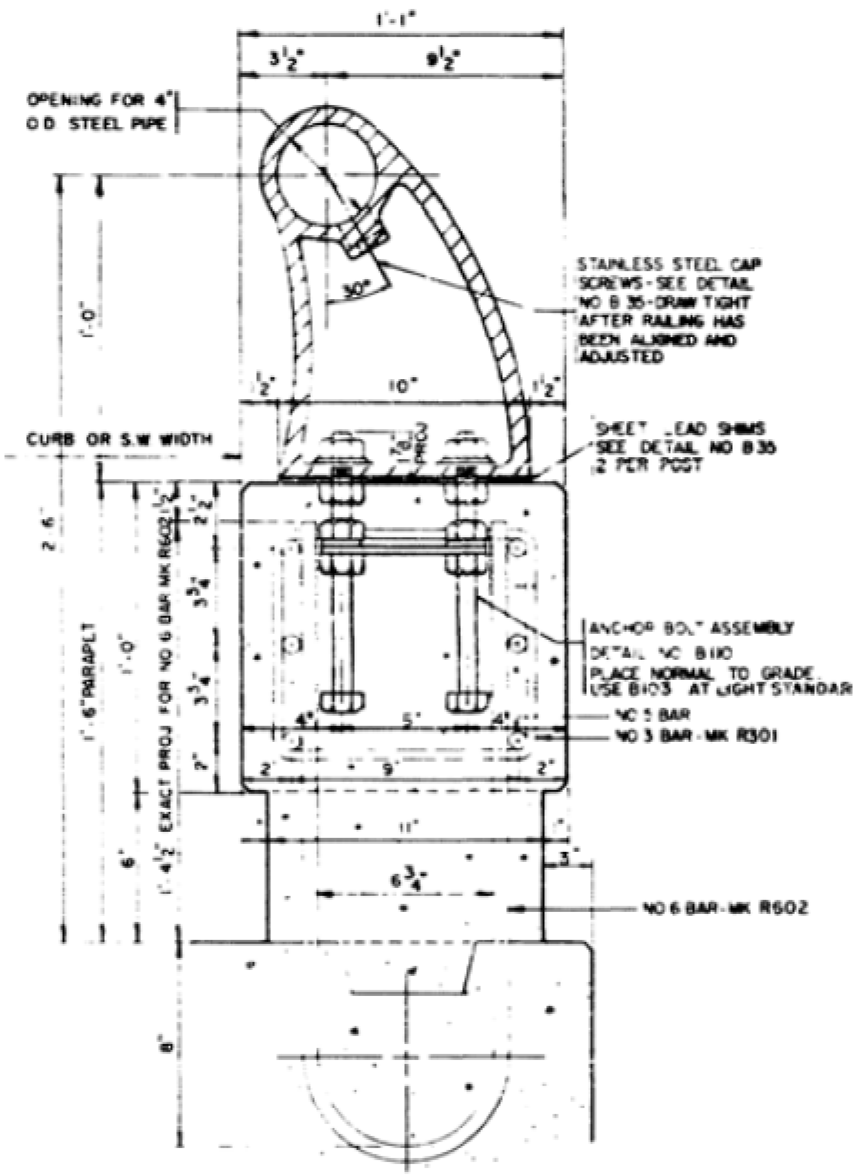
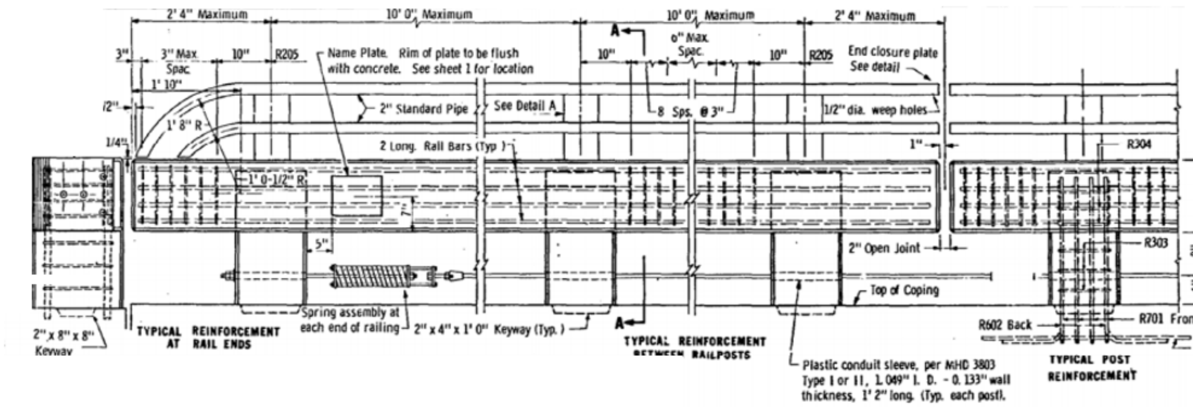


Figure A2-13 Bridge No. 9805 Details.

Bridge No. 62069

Figure A2-14 Bridge No. 62069 Photo.



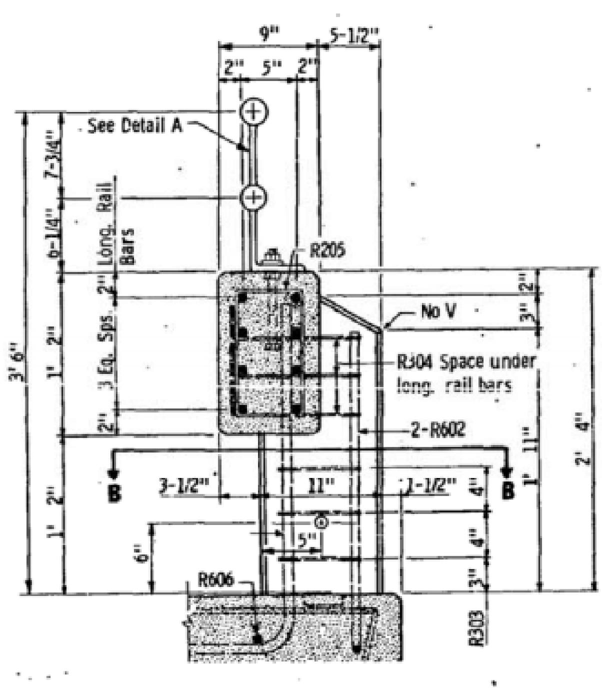
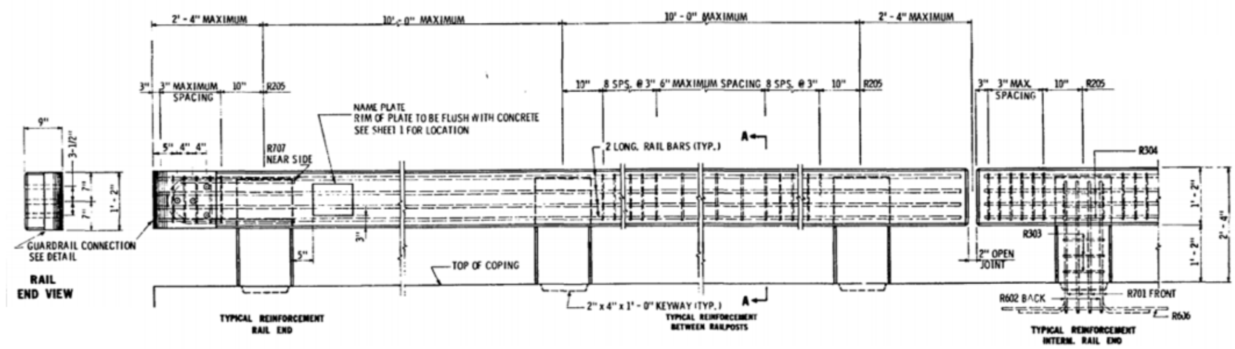


Figure A2-15 Bridge No. 62069 Details.

Bridge No. 27042

Figure A2-16 Bridge No. 27042 Photo.



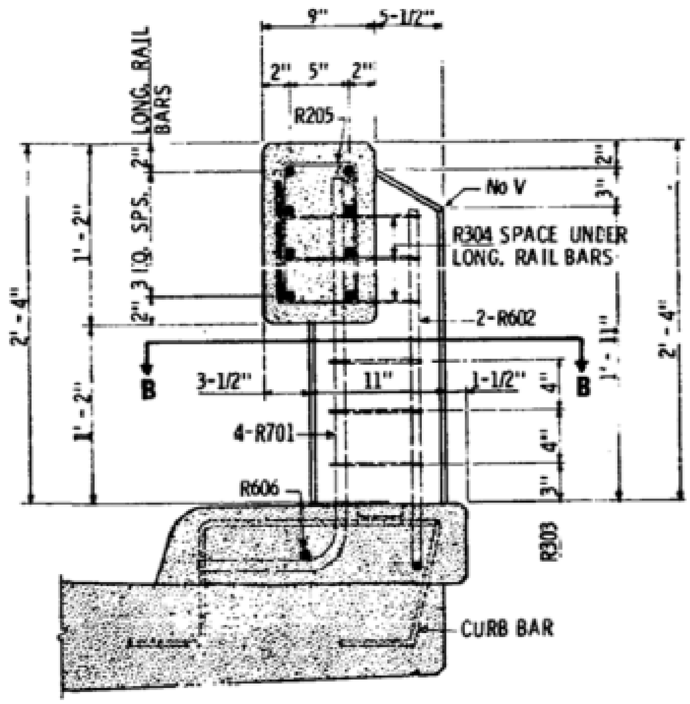
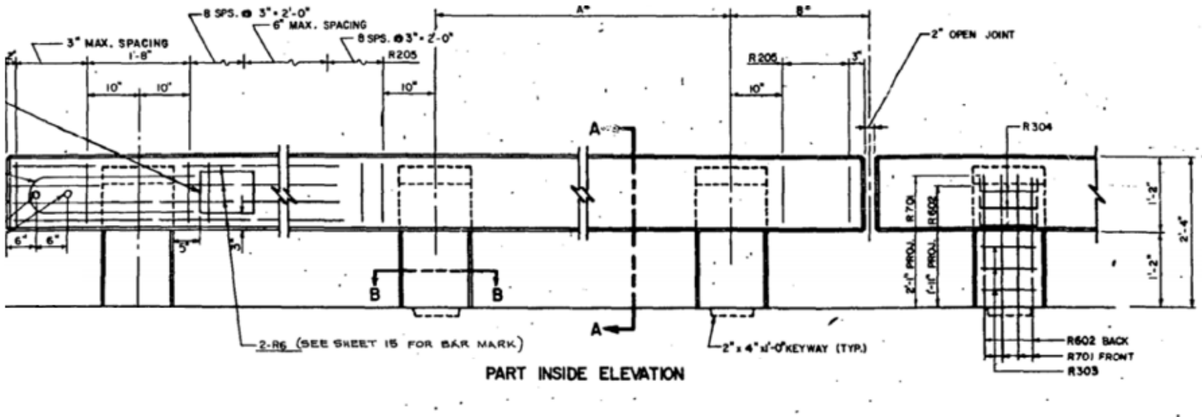


Figure A2-17 Bridge No. 27042 Details.



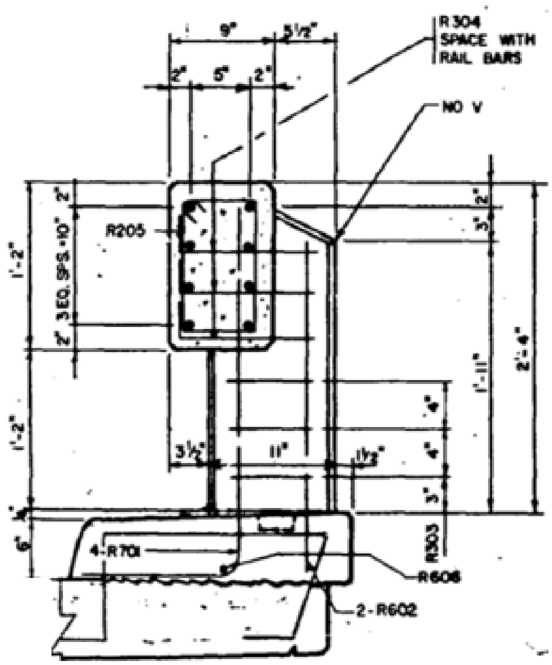


Figure A2-18 Bridge No. 62040 Details

APPENDIX A3: G BARRIER - PHOTOS TAKEN FROM THE FIELD SITE VISIT AND DETAILS RECEIVED FROM MNDOT

Bridge No. 09830

Figure A3-1 Bridge No. 09830 Photos.

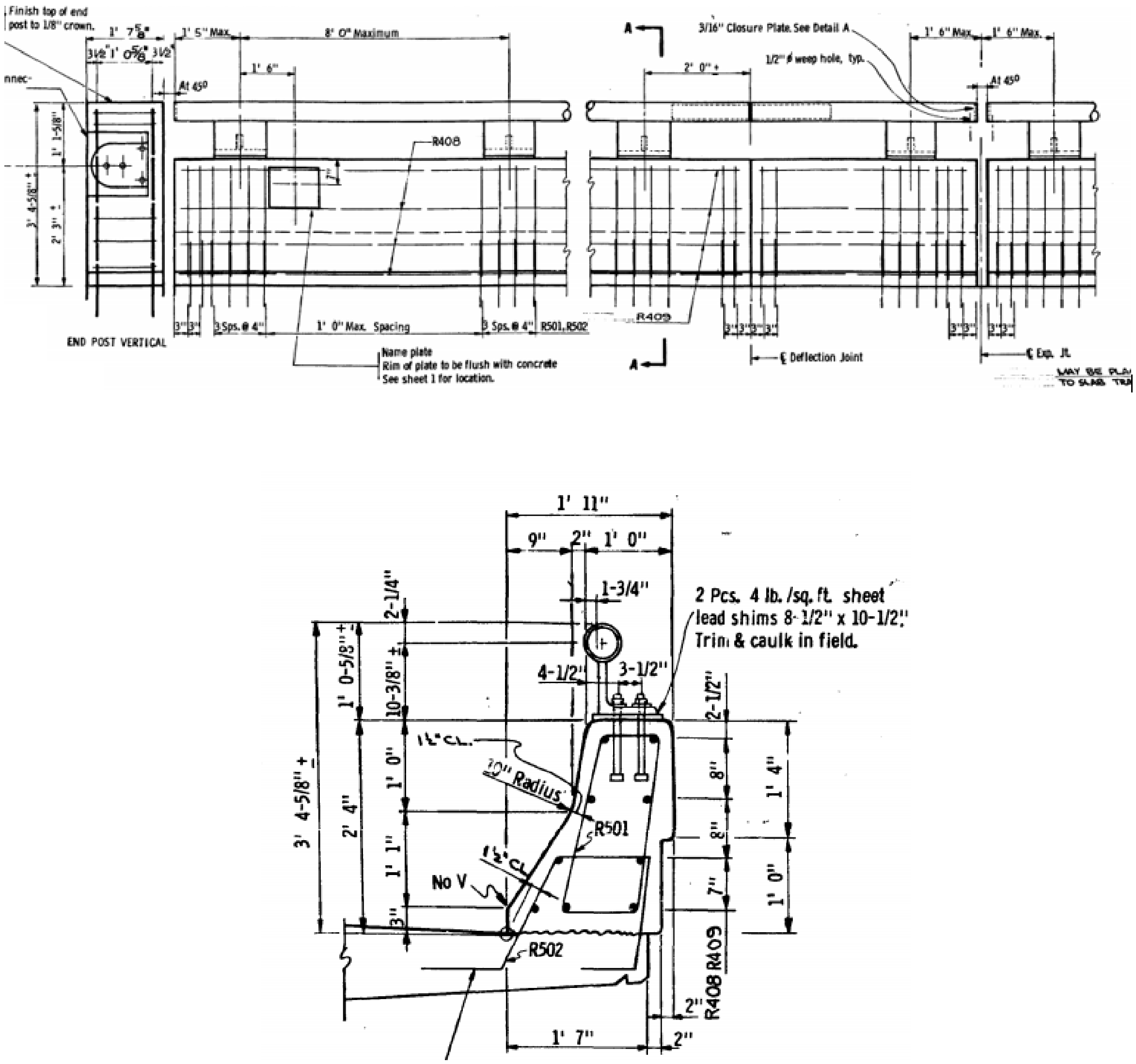
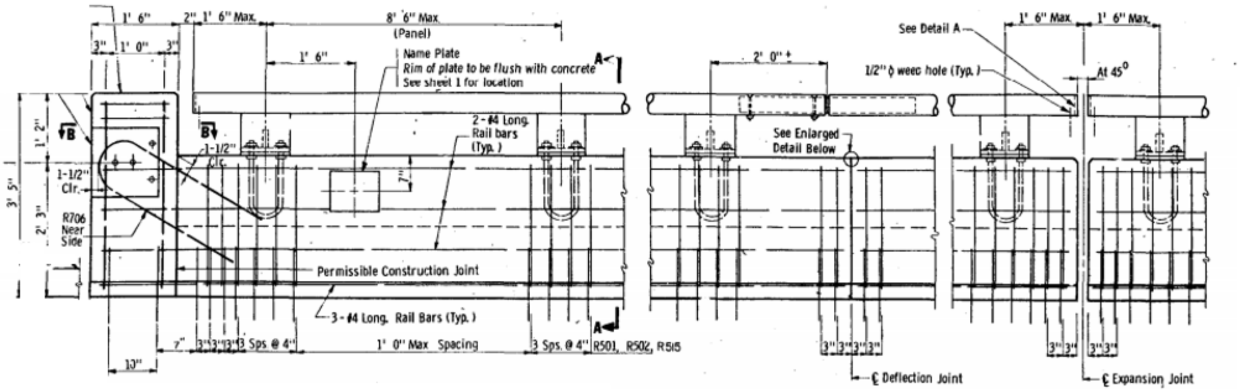


Figure A3-2 Bridge No. 09830 Details.

Bridge No. 19021

Figure A3-3 Bridge No. 19021 Photos.



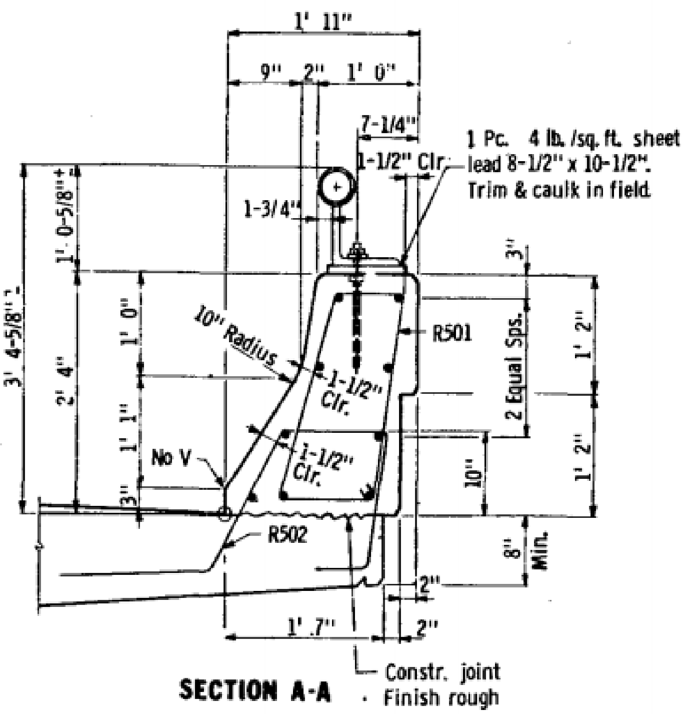


Figure A3-4 Bridge No. 19021 Details.

Bridge No. 86812

Figure A3-5 Bridge No. 86812 Photos.

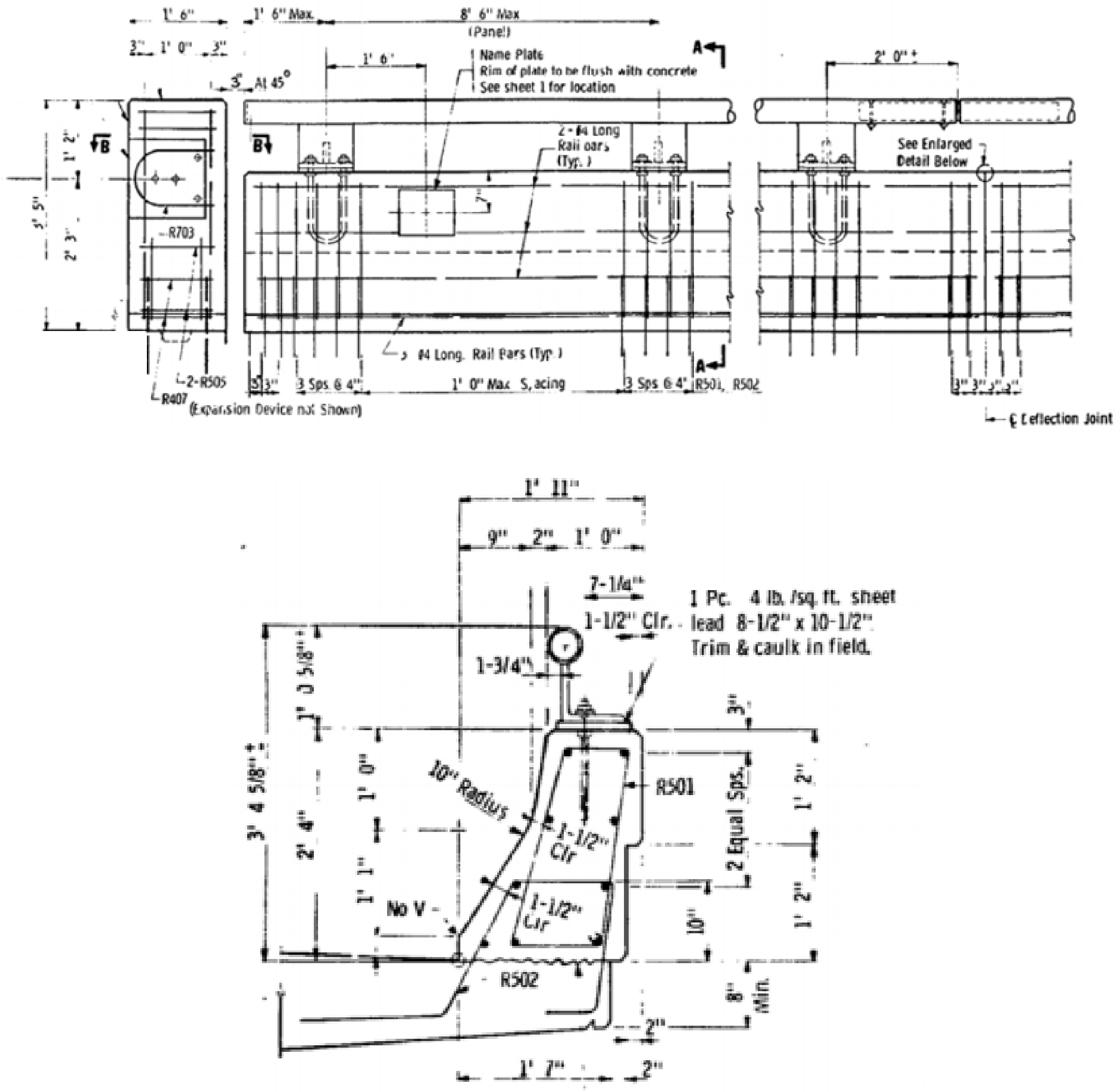
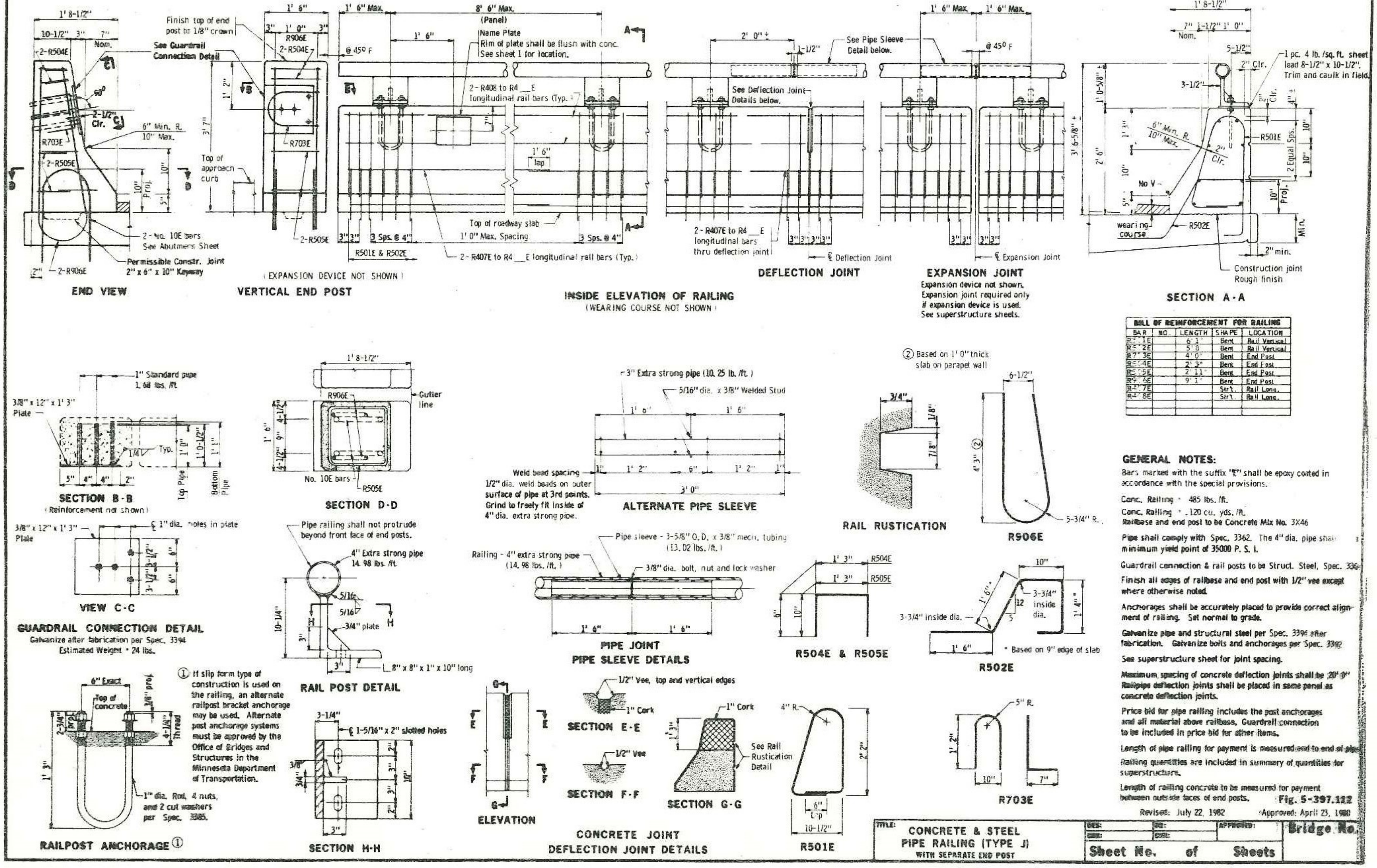


Figure A3-6 Bridge No. 86812 Details.

APPENDIX B: J AND F BARRIER ANALYSES

APPENDIX B1: J BARRIER ON FIGURE 5-397.112



BILL OF REINFORCEMENT FOR RAILING				
BAR	NO.	LENGTH	SHAPE	LOCATION
[illegible]1E		6' 1"	Bent	Rail Vertical
[illegible]2E		5' 8"	Bent	Rail Vertical
R7[illegible]3E		4' 0"	Bent	End Post
[illegible]4E		2' 3"	Bent	End Post
[illegible]5E		2' 11"	Bent	End Post
R9[illegible]6E		9' 1"	Bent	End Post
R[illegible]7E			Str't.	Rail Long.
R4[illegible]8E			Str't.	Rail Long.

GENERAL NOTES:

Bars marked with the suffix "E" shall be epoxy coated in accordance with the special provisions.

Conc. Railing = 485 lbs. /ft.

Conc. Railing = .120 cu. yds. /ft.
Railbase and end post to be Concrete Mix No. 3X46

Pipe shall comply with Spec. 3362. The 4" dia. pipe shall minimum yield point of 35000 P. S. I.

Guardrail connection & rail posts to be Struct. Steel, Spec. 336

Finish all edges of railbase and end post with 1/2" vee except where otherwise noted.

Anchorages shall be accurately placed to provide correct alignment of railing. Set normal to grade.

Galvanize pipe and structural steel per Spec. 3394 after fabrication. Galvanize bolts and anchorages per Spec. 3392

See superstructure sheet for joint spacing.

Maximum spacing of concrete deflection joints shall be 20' 0" Railpipe deflection joints shall be placed in same panel as concrete deflection joints.

Price bid for pipe railing includes the post anchorages and all material above railbase. Guardrail connection to be included in price bid for other items.

Length of pipe railing for payment is measured end to end of pipe

Railing quantities are included in summary of quantities for superstructure.

Length of railing concrete to be measured for payment between outside faces of end posts.

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.112
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$H_r := 42.625in$	Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)
$H_R := 40.375in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$t_o := 2in$	Thickness of overlay (inches)

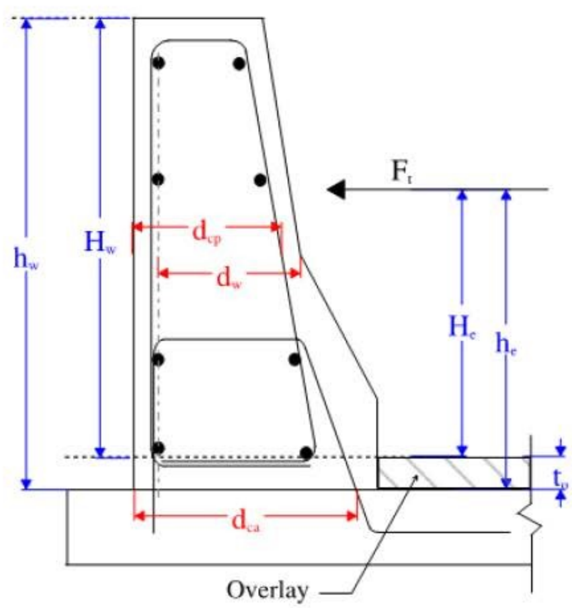


Figure 1. Sketch Showing Critical Input Variables

(1b-conti.) Concrete Parapet Inputs:

$H_w := 28in$ — Height of the concrete parapet/wall measured from the top of the roadway surface/overlay (in.)

$h_w := H_w + t_o$ $h_w = 30 \cdot in$ — Total height of the concrete parapet/wall (in.)

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{vp.mid} := 12in$ — Average Spacing of parapet vertical reinforcement at midspan (in.)

$d_{cp.mid} := 11.18in$ — Extreme distance of parapet vertical reinforcement in tension at midspan (in.)

$A_{vp1.end} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 4in$ — Average Spacing of parapet vertical reinforcement at joints/ends (in.)

$d_{cp.end} := 11.18in$ — Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$ — Area of longitudinal reinforcement bars in tension (in^2)

$d_w := 10.63in$ — Extreme distance of tension longitudinal reinforcement of wall (in.)

SUBJECT: MnDOT J-Barrier
Figure 5-397.112
MASH TL-3 Compliance Assessment

(1b-conti.) **Concrete Parapet Inputs:**

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502E} := 10in$ — Projected length of R502E reinforcement over the slab (in.)

$L_{wid\_R502E} := 10in$ — Outer width of R502E reinforcement (in.)

$Cover := 2in$ — Cover clear distance (in.)

$Ratio_{R502E} := \frac{5}{12}$ — Inclined angle of R502E reinforcement

$d_{b\_R502E} := 0.625in$ — Nominal diameter of R502E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_R502E} + L_{proj\_R502E} \cdot Ratio_{R502E} + Cover - \frac{1}{2} d_{b\_R502E} = 15.854 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in²)

$s_{va.mid} := 12in$ — Average Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 15.854 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

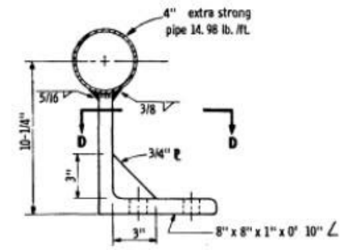
$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in²)

$s_{va.end} := 4in$ — Average Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := d_{ca} = 15.854 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

(1b) Steel Rail, Post, and Anchor Rod Inputs:

Steel Rail Inputs:
a) Steel Tube Rail is M.H.D. 3362 material, Fy=35ksi
b) Steel Tube Rail is a 4" extra strong pipe



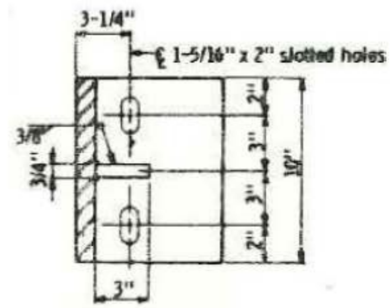
POST DETAIL

$F_{yR} := 35ksi$ Yield Strength of Steel Tube Rail (ksi)

$d_{oR} := 4.5in$ Outside diameter of Steel Tube Rail (in.)

$d_{iR} := 3.83in$ Inside diameter of Steel Tube Rail (in.)

Steel Post Inputs:
a) Steel Post is M.H.D. 3306 material, Fy= 36ksi
b) Steel Post is a 8"x8"x1"x10" Angle Member



SECTION H-H

$F_{yp} := 36ksi$ Yield Strength of Steel Post (ksi)

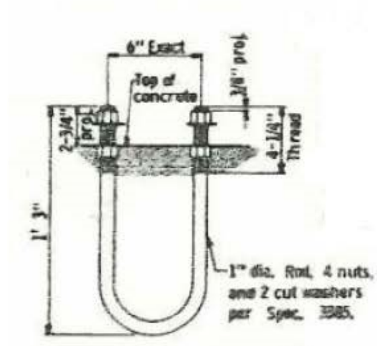
$w_p := 10in$ Width of Steel Post about the bending axis (in.)

$t_p := 1in$ Thickness of Steel Post (in.)

$h_p := 9.25in$ Height from the bottom of the post to the centroid of the steel tube rail (in.)

$L_p := 8.5ft$ Steel Post Spacing (ft.)

Anchor Rod Inputs:
a) Anchor Rods are Gr. 36 material, Fu=58 ksi
b) Anchor Rods are 1"ϕ x 15" U Bolt



RAILPOST ANCHORAGE ①

$F_{u.rod} := 58ksi$	Tensile Strength of Anchor Rods (ksi)
$N_{rod.shear} := 2$	Number of Anchor Rod cross-sections acting in Shear
$N_{rod.tension} := 2$	Number of Anchor Rod cross-sections acting in Tension
$d_{rod} := 4.75in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$\phi_{rod} := 1in$	Diameter of Anchor Rods (in)

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.112
MASH TL-3 Compliance Assessment

(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$	Test Level
$F_t := 71kip$	Transverse Impact Force
$L_t := 4ft$	Longitudinal Length of Distribution of Impact Force

$\mathbf{H_e := 19in}$ Height of Equivalent Transverse Load from top of overlay

$\mathbf{H_{min} := 29in}$ Minimum height of a MASH TL-3 barrier (in.)

$\mathbf{h_e := H_e + t_o = 21 \cdot in}$ Total Equivalent Trans. Impact Height

$\mathbf{H_r = 42.625 \cdot in}$ Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.112
MASH TL-3 Compliance Assessment

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_r = 42.625 \cdot in$ Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_r \geq H_{min} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

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SUBJECT: MnDOT J-Barrier
Figure 5-397.112
MASH TL-3 Compliance Assessment

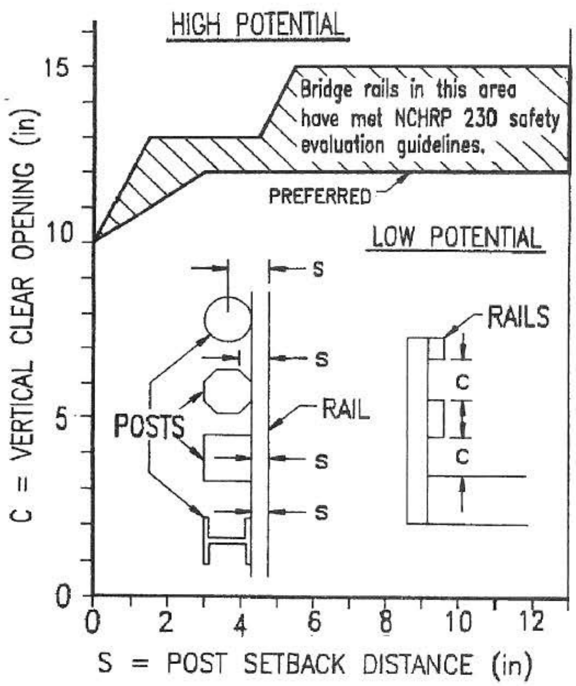
(4) <u>Geometric Criteria:</u>

$S_{post} := 1.75in$ — Post Setback (in.)
Note: Denoted as "S" in figure below.

$C_b := 10.375in$ — Vertical Clear Opening (in.)

$\Sigma A := H_w + \frac{d_{oR}}{2} = 30.25 \cdot in$ — Total Rail Contact Width (in.) — $H_w = 28 \cdot in$

$H_r = 42.625 \cdot in$ — Total height of the bridge rail measured from the top of the roadway surface/overlay (in.)
Note: Denoted as "H" in figure below.



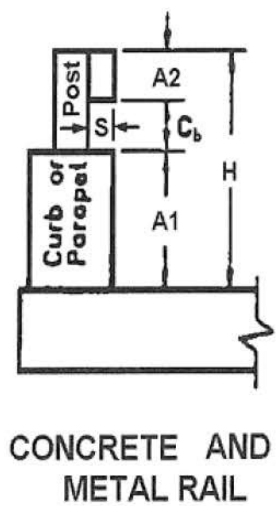


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

SUBJECT: MnDOT J-Barrier
Figure 5-397.112
MASH TL-3 Compliance Assessment

(3-conti.) Geometric Criteria:

$S_{post} = 1.75 \cdot in$ $\Sigma A = 30.25 \cdot in$ $H_r = 42.625 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_r} = 0.71$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

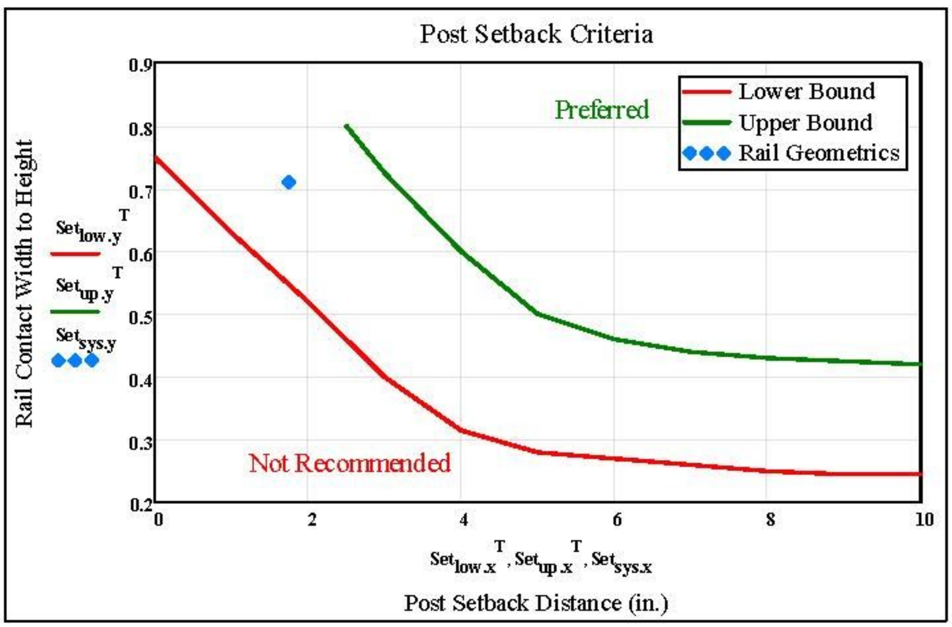
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 1.75$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.71$ Ratio of Contact Width to Total Height rail geometric point



NotRecommended := 1 Marginal := 2 Preferred := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Post_Setback_Criteria_Rail_Geometric_Point := Marginal

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Figure 5-397.112
MASH TL-3 Compliance Assessment

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0 \quad 3 \quad 13)$

$snag_{low.y} := (10 \quad 12 \quad 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0 \quad 1.25 \quad 4.25 \quad 5.25 \quad 13)$

$snag_{up.y} := (10 \quad 13 \quad 13 \quad 15 \quad 15)$

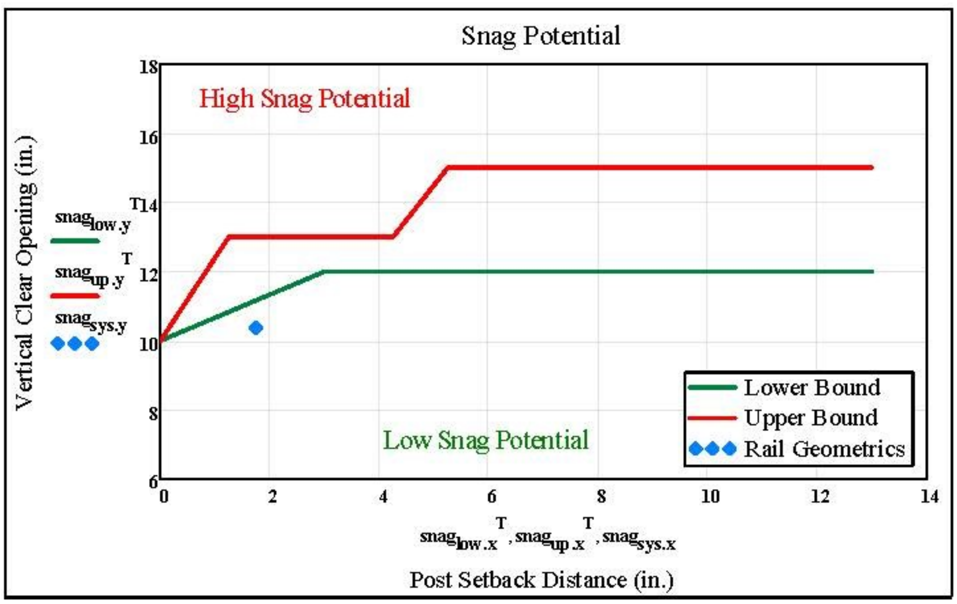
Upper Boundary for Snag Potential Criteria
x and y coordinates

$$snag_{sys.x} := \frac{S_{post}}{in} = 1.75$$

Post Setback rail geometric point

$$snag_{sys.y} := \frac{C_b}{in} = 10.375$$

Vertical Clear Opening rail geometric point



HighSnagPotential := **1** **Marginal** = **2** **LowSnagPotential** := **3**

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := **LowSnagPotential**

(3) Geometric Criteria - Summary of Results:

Post_Setback_Criteria_Check := | "OK" if Post_Setback_Criteria_Rail_Geometric_Point = Preferred
| "NOT OK" otherwise

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check := | "OK" if Snag_Potential_Criteria_Rail_Geometric_Point = LowSnagPotential
| "NOT OK" otherwise

Snag_Potential_Criteria_Check = "OK"

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(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$

Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 11.18 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 16.976 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

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(4a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 15.854 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$

Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 24.221 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 16.976 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(4b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 4 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.93 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.368 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 11.18 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 48.807 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 4 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.93 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.368 \cdot in$ — Depth of Whitney Stress Block (in.)

(4b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 15.854 \cdot in$ Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 70.542 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 48.807 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(4c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 10.63 \cdot in$ Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ Total Area of longitudinal reinforcement bars acting in tension (in²)

$h_w = 30 \cdot in$ Total height of the barrier (in.)

$h_w = 30 \cdot in$

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.471 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 41.579 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

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(4d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

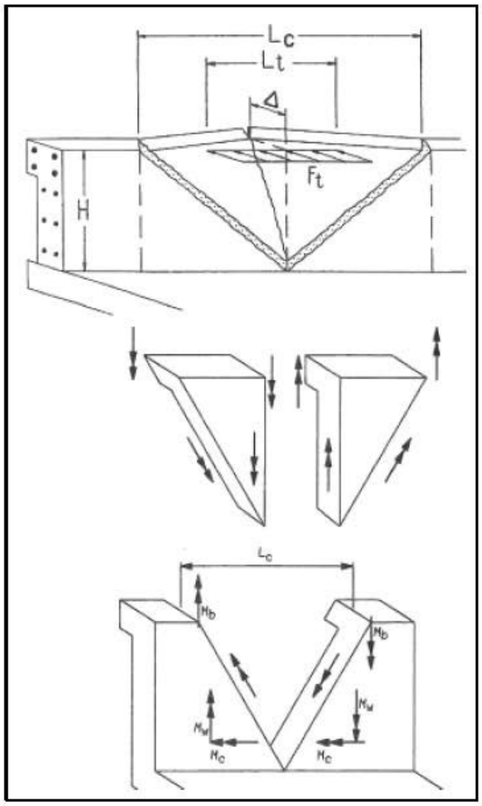

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 16.976 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 41.579 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$h_w = 30 \cdot in$ — Total Height of Barrier from Deck surface

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.279 \cdot ft$$

(Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 126.016 \cdot kip$$

(Equation A13.3.1-1)

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(4e) **Determine the Ultimate Resistance of the Wall at Joints/Ends:** R_{wend}

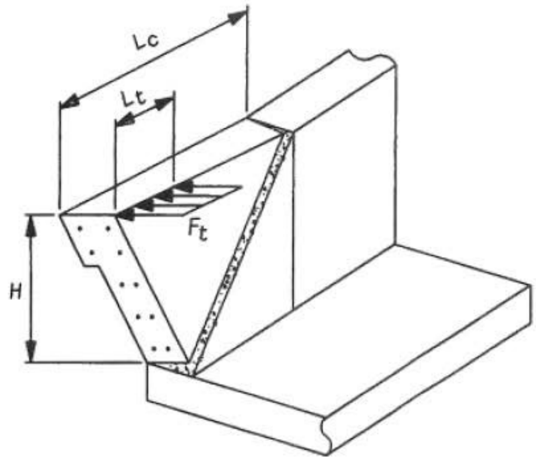

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)
Note: H_w = H in Figure 3e

$M_B = 0$ — No additional beam strength

$M_w = 41.579 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 48.807 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 4.476 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^2\right)}{h_w}\right] = 174.762 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

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(4f) Steel Rail & Post Strength Analysis:

$F_{yR} = 35 \cdot ksi$ — Yield Strength of Steel Tube Rail (ksi)

$d_{oR} = 4.5 \cdot in$ — Outside diameter of Steel Tube Rail (in.)

$d_{iR} = 3.83 \cdot in$ — Inside diameter of Steel Tube Rail (in.)

$$Z_R := \frac{\left(d_{oR}^3 - d_{iR}^3\right)}{6} = 5.824 \cdot in^3$$

Plastic Sectional Modulus of the Steel Tube Rail (in^3)

$$M_p := F_{yR} \cdot Z_R = 16.986 \cdot kip \cdot ft$$

Plastic Moment Capacity of the Steel Tube Rail (kip-ft)

Calculate the Plastic Strength of the Post: P_{P1}

$w_p = 10 \cdot in$ — Width of Steel Post about the bending axis (in.)

$t_p = 1 \cdot in$ — Thickness of Steel Post (in.)

$$Z_P := \frac{w_p \cdot t_p^2}{4} = 2.5 \cdot in^3$$

Plastic Sectional Modulus of the Steel Post about the bending axis (in.)

$F_{yp} = 36 \cdot ksi$ — Yield Strength of Steel Post (ksi)

$$M_{post} := F_{yp} \cdot Z_P = 7.5 \cdot kip \cdot ft$$

Plastic Moment Capacity of the Steel Post (kip-ft)

$h_p = 9.25 \cdot in$ — Height from the bottom of the post to the centroid of the steel tube rail (in.)

$$P_{P1} := \frac{M_{post}}{h_p} = 9.73 \cdot kip$$

Post Strength based on the Plastic Failure of a Steel Post (kip)

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Calculate the Post Strength based on the Ultimate Strength of the Anchor Rods: P_{P2}

$F_{u.rod} = 58 \cdot ksi$	Tensile Strength of the Anchor Rods (ksi)
$\phi_{rod} = 1 \cdot in$	Diameter of Anchor Rods (in)
$A_{rod} := \frac{\pi}{4} \cdot \phi_{rod}^{\ 2} = 0.785 \cdot in^2$	Area of a Anchor Rod (in^2)
$R_{nt} := F_{u.rod} \cdot (0.75 \cdot A_{rod}) = 34.165 \cdot kip$	Nominal strength of one Anchor Rod in Tension (kip)
$N_{rod.tension} = 2$	Number of Anchor Rods acting in tension
$d_{rod} = 4.75 \cdot in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$d_b := 1.5in$	Length of the steel plate bearing pressure acting on the concrete parapet (in.)
$w_{rod} := d_{rod} - \frac{d_b}{3} = 4.25 \cdot in$	Distance from anchor rods acting in tension to the centroid of the bearing pressure acting on the concrete parapet (in.)
$M_{t.rod} := w_{rod} \cdot R_{nt} \cdot N_{rod.tension} = 24.2 \cdot kip \cdot ft$	Moment strength of Post based on tensile capacity of Anchor Rods (k-ft)
$h_p = 9.25 \cdot in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$P_{t.rod} := \frac{M_{t.rod}}{h_p} = 31.395 \cdot kip$	Post Strength based on the tensile capacity of Anchor Rods (kip)
$R_{nv} := F_{u.rod} \cdot (0.45 \cdot A_{rod}) = 20.499 \cdot kip$	Nominal strength of one anchor rod in Shear w/h threads in shear plane (kip)
$P_{v.rod} := N_{rod.shear} \cdot R_{nv} = 40.998 \cdot kip$	Post Strength based on the shear capacity of Anchor Rods (kip)
$P_{P2} := \min(P_{t.rod}, P_{v.rod}) = 31.395 \cdot kip$	Post Strength based on the Ultimate Strength of the Anchor Rods (kip)

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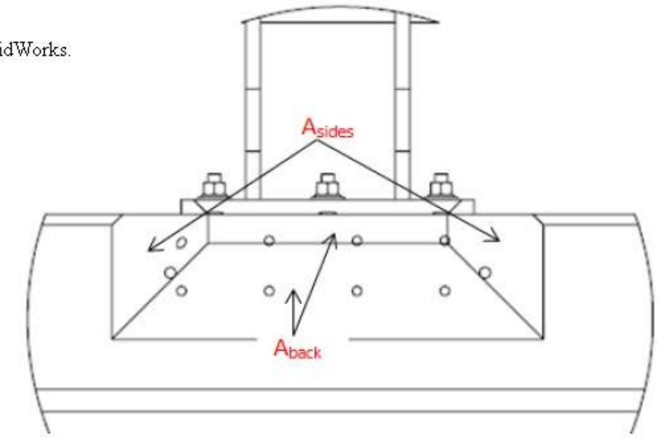
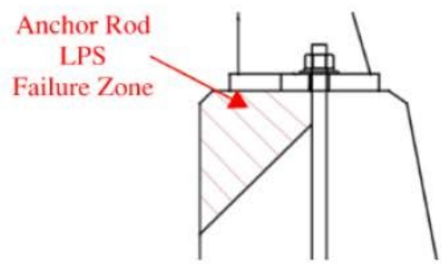
Calculate the Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods: P_{P3}

Note: This failure mechanism was modeled in SolidWorks.

$A_{sides} := 41in^2$

$A_{back} := 87in^2$





$\phi_v := 0.75$	Shear Strength Reduction Factor
$A_{LPS} := A_{sides} + A_{back} = 128 \cdot in^2$	Total Area of Failure Planes due to Lateral Punching Shear Failure (in^2)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength (ksi)
$V_{c.lat} := \phi_v \cdot 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi = 94.87 \cdot psi$	Concrete Stress from Block Shear of Anchor Rods (psi) -ACI 318-14 Eqn. 22.5.5.1
$P_{P3} := V_{c.lat} \cdot A_{LPS} = 12.143 \cdot kip$	Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)

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Determine the Limiting ("Worst Case") Post Strength (kips): P_P

$P_{P1} = 9.73 \cdot kip$	Post Strength based on the Plastic Failure of a Steel Post (kip)
$P_{P2} = 31.395 \cdot kip$	Post Strength based on the Ultimate Strength of the Anchor Rods (kip)
$P_{P3} = 12.143 \cdot kip$	Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)
$P_P := \min(P_{P1}, P_{P2}, P_{P3}) = 9.73 \cdot kip$	Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

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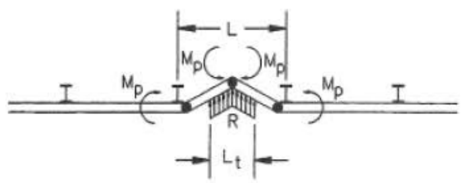
Determine Ultimate Capacity of Steel Rail for a Single and Double Span Failure Mode:

$L_t = 4 \cdot ft$ — Length of the Distribution of the Transverse Impact Force (ft.)

$L_p = 8.5 \cdot ft$ — Steel Post Spacing (ft.)

$M_p = 16.986 \cdot kip \cdot ft$ — Flexural Capacity of the Steel Tube Rail (kip-ft)

$P_p = 9.73 \cdot kip$ — Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

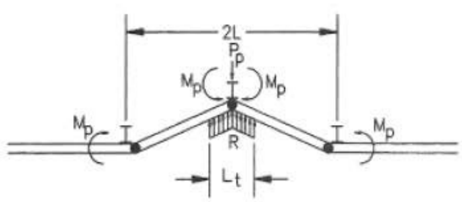


Single–Span Failure Mode

$N_1 := 1$ — Number of Spans

$$R_R := \frac{(16 \cdot M_p) + \left[(N_1 - 1)(N_1 + 1) \cdot P_p \cdot L_p\right]}{(2 \cdot N_1 \cdot L_p) - L_t} = 20.906 \cdot kip$$

Ultimate Capacity of rail over one span (kip)



Two–Span Failure Mode

$N_2 := 2$ — Number of Spans

$$R_{R'} := \frac{16 \cdot M_p + \left(N_2^{\,2} \cdot P_p \cdot L_p\right)}{(2N_2 \cdot L_p) - L_t} = 20.086 \cdot kip$$

Ultimate Capacity of rail over two spans (kip)

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(4g) Determine the Combined Resultant Strength of the Bridge Rail System:

Determine the Resultant Strength of the Bridge Rail System at Midspan: (R_1)

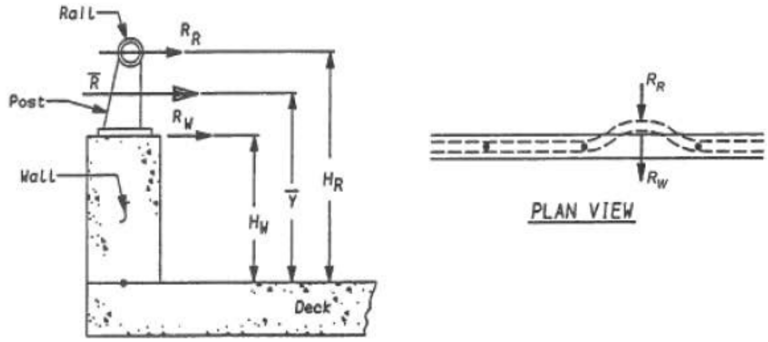

Figure A13.3.3-1—Concrete Wall and Metal Rail Evaluation—Impact at Midspan of Rail

$R_R = 20.906 \cdot kip$	Ultimate Capacity of rail over one span (kip)
$H_R = 40.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 126.016 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$h_w = 30 \cdot in$	
$R_{bar1} := R_R + R_{wmid} = 146.922 \cdot kip$	Resultant Strength of the Bridge Rail System Located at y_{bar1} AASHTO Eqn. A13.3.3-1
$y_{bar1} := \dfrac{R_R \cdot H_R + R_{wmid} \cdot H_w}{R_{bar1}} = 29.761 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-2
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load above road surface
$R_1 := R_{bar1} \cdot \left(\dfrac{y_{bar1}}{H_e}\right) = 230.134 \cdot kip$	Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

Determine the Resultant Strength of the Bridge Rail System at a Post: (R_2)

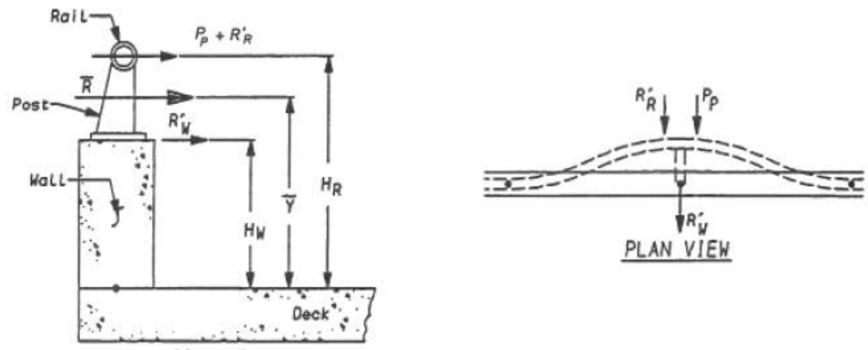

Figure A13.3.3-2—Concrete Wall and Metal Rail Evaluation—Impact at Post

$R_{R'} = 20.086 \cdot kip$	Ultimate Capacity of rail over two spans (kip)
$H_R = 40.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 126.016 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$R_{w'} := \dfrac{R_{wmid} \cdot H_w - P_P \cdot H_R}{H_w} = 111.986 \cdot kip$	Reduced Wall Strength (kip) AASHTO Eqn. A13.3.3-5
$R_{bar2} := P_P + R_{R'} + R_{w'} = 141.802 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at y_{bar2} AASHTO Eqn. A13.3.3-3
$y_{bar2} := \dfrac{H_R \cdot (P_P + R_{R'}) + R_{w'} \cdot H_w}{R_{bar2}} = 30.602 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-4
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load (in.)
$R_2 := R_{bar2} \cdot \left(\dfrac{y_{bar2}}{H_e}\right) = 228.392 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$R_1 = 230.134 \cdot kip$ — Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_1 \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$R_2 = 228.392 \cdot kip$ — Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_a\_Post\_Check} := \begin{cases} \text{"OK"} & \text{if } R_2 \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

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(5) Strength Analysis of the Seperate End Post:

(5a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

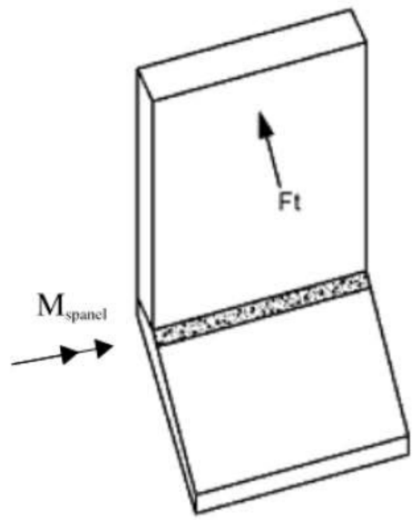

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 43in$ Height of the end post measured from the top of the roadway (in.)

$b_{spost} := 18in$ Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 1in^2$ Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
#9 Bars

$n_{p.spost} := 2$ Number of parapet vertical reinforcement in the end post (in.)

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(5a-conti.) <u>Bending Capacity of the End Post about the Longitudinal Axis</u>: M_{spost}

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 2 \cdot in^2$

Total Area of parapet vertical reinforment in the tension zone of the end post (in^2)

$a_{p.spost} := \dfrac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.961 \cdot in$

Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 9.75in$

Average extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{p.spost}}{2}\right) = 87.696 \cdot kip \cdot ft$

Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

<u>Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:</u>

<u>Note:</u> See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1.27in^2$

Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2)
#10 Bars

$n_{a.spost} := 2$

Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 2.54 \cdot in^2$

Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 2.49 \cdot in$

Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 16in$

Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 187.387 \cdot kip \cdot ft$

Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 87.696 \cdot kip \cdot ft$

Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

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SUBJECT: MnDOT J-Barrier
Figure 5-397.112
MASH TL-3 Compliance Assessment

(5a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ $h_e = 21 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 87.696 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \frac{M_{s.post}}{h_e} = 50.112 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \left(\begin{array}{ll} \text{"OK"} & \text{if } R_{s.post} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"NOT OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check = "OK"

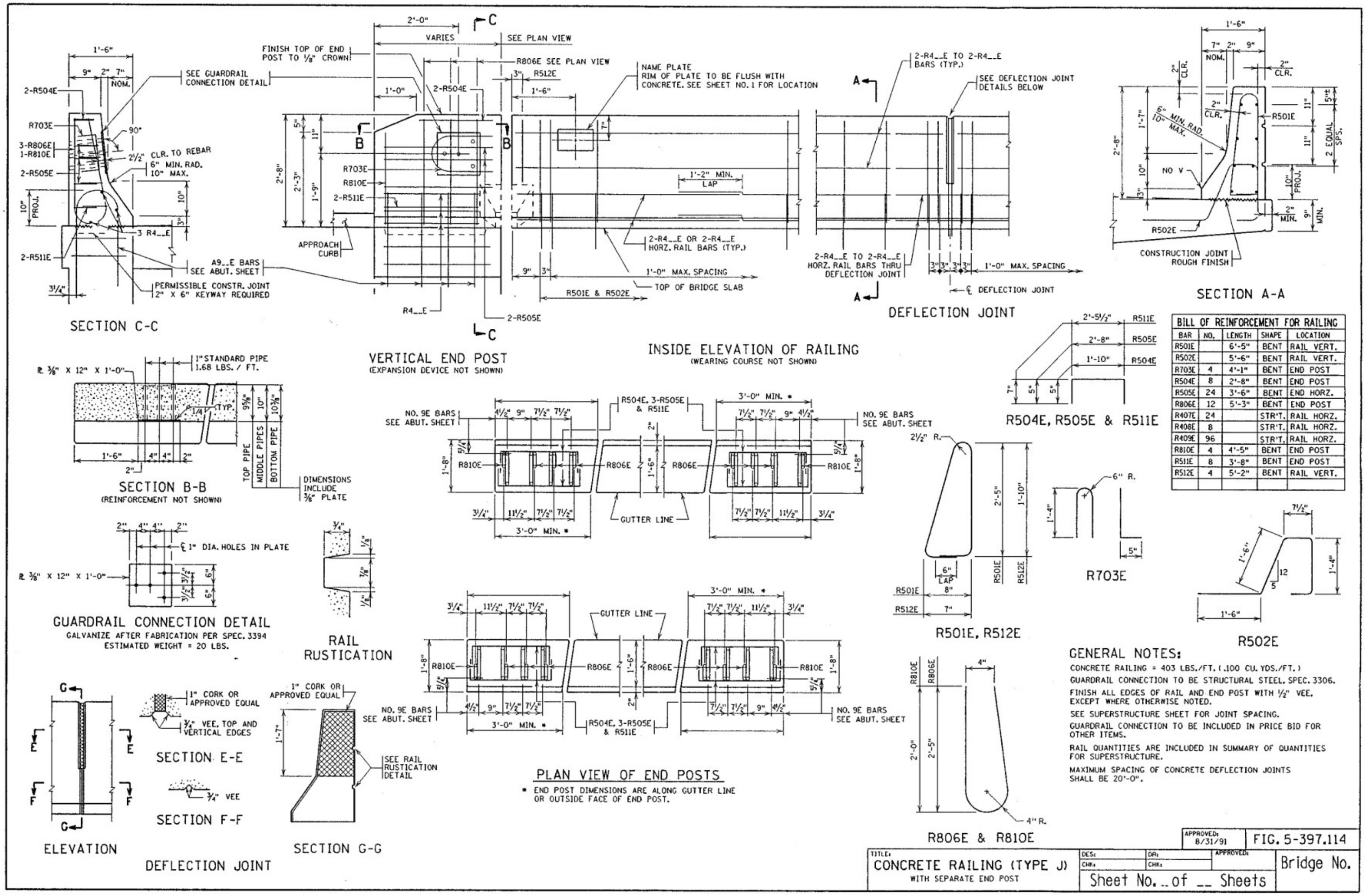
Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

Structural_Capacity_of_End_Post_Check = "NOT OK"

The J-Barrier from Figure 5-397.112 does not satisfy all MASH TL-3 Criteria

APPENDIX B2: J BARRIER ON FIGURE 5-397.114



BILL OF REINFORCEMENT FOR RAILING

BAR	NO.	LENGTH	SHAPE	LOCATION
R501E		6'-5"	BENT	RAIL VERT.
R502E		5'-6"	BENT	RAIL VERT.
R703E	4	4'-1"	BENT	END POST
R504E	8	2'-8"	BENT	END POST
R505E	24	3'-6"	BENT	END HORZ.
R806E	12	5'-3"	BENT	END POST
R407E	24		STR'T.	RAIL HORZ.
R408E	8		STR'T.	RAIL HORZ.
R409E	96		STR'T.	RAIL HORZ.
R810E	4	4'-5"	BENT	END POST
R511E	8	3'-8"	BENT	END POST
R512E	4	5'-2"	BENT	RAIL VERT.

GENERAL NOTES:

CONCRETE RAILING = 403 LBS./FT. (.100 CU. YDS./FT.)

GUARDRAIL CONNECTION TO BE STRUCTURAL STEEL, SPEC. 3306.

FINISH ALL EDGES OF RAIL AND END POST WITH 1/2" VEE. EXCEPT WHERE OTHERWISE NOTED.

SEE SUPERSTRUCTURE SHEET FOR JOINT SPACING.

GUARDRAIL CONNECTION TO BE INCLUDED IN PRICE BID FOR OTHER ITEMS.

RAIL QUANTITIES ARE INCLUDED IN SUMMARY OF QUANTITIES FOR SUPERSTRUCTURE.

MAXIMUM SPACING OF CONCRETE DEFLECTION JOINTS SHALL BE 20'-0".

APPROVED: 8/31/91 | FIG. 5-397.114

TITLE: CONCRETE RAILING (TYPE J) WITH SEPARATE END POST

DES: | DR: | CHK: | CHK: | APPROVED:

Sheet No. .. of __ Sheets | Bridge No.

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 32in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 0in$	Thickness of overlay (in.)
$h_w := H_w + t_o$	Total height of the barrier (in.)

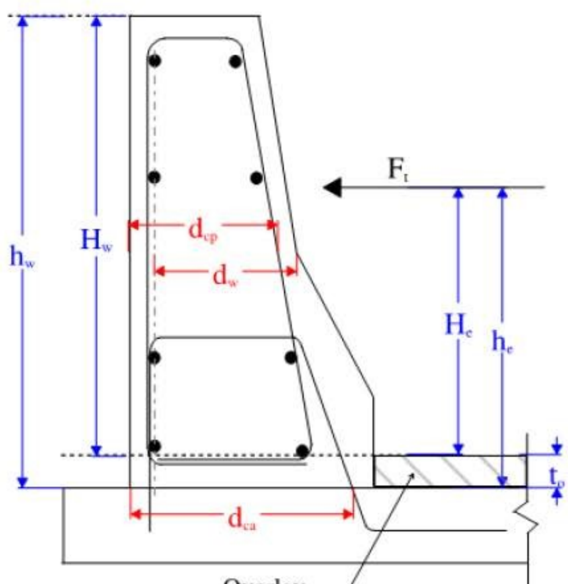

Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b) Concrete Parapet Inputs:

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{vp.mid} := 12in$	Spacing of parapet vertical reinforcement at midspan (in.)
$d_{cp.mid} := 8.19in$	Extreme distance of parapet vertical reinforcement in tension at midspan (in.)
$A_{vp1.end} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{vp.end} := 8in$	Spacing of parapet vertical reinforcement at joints/ends (in.). Averaged over 4 feet
$d_{cp.end} := 8.19in$	Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$	Area of longitudinal reinforcement bars in tension (in^2)
$d_w := 8.5in$	Extreme distance of tension longitudinal reinforcement of wall (in.)

(1b-conti.) Concrete Parapet Inputs:

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502E} := 10in$ — Projected length of R502E reinforcement over the slab (in.)

$L_{wid\_R502E} := 7.5in$ — Outer width of R502E reinforcement (in.)

$Cover := 2in$ — Cover clear distance (in.)

$Ratio_{R502E} := \frac{5}{12}$ — Inclined angle of R502E reinforcement

$d_{b\_R502E} := 0.625in$ — Nominal diameter of R502E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_R502E} + L_{proj\_R502E} \cdot Ratio_{R502E} + Cover - \frac{1}{2} d_{b\_R502E} = 13.354 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ — Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 8in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.). Averaged over 4 feet

$d_{ca.end} := d_{ca} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

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(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$h_w = 32 \cdot in$ Total height of barrier (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{array}{|l} \text{"OK"} \quad \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

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(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$ — Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$ — Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 8.19 \cdot in$ — Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 12.341 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(3a-conti.) **Bending Capacity of the Wall about the Longitudinal Axis at Midspan:** M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 13.354 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$

Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 20.346 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 12.341 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in²)

$s_{vp.end} = 8 \cdot in$ — Average Spacing of parapet vertical reinforcement at joints/ends (in.) over 4.0 feet

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.465 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in²)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.684 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 8.19 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 18.247 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in²)

$s_{va.end} = 8 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.465 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in²)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.684 \cdot in$ — Depth of Whitney Stress Block (in.)

(3b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 13.354 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 30.253 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 18.247 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(3c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 8.5 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 32 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.441 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 33.118 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

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(3d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

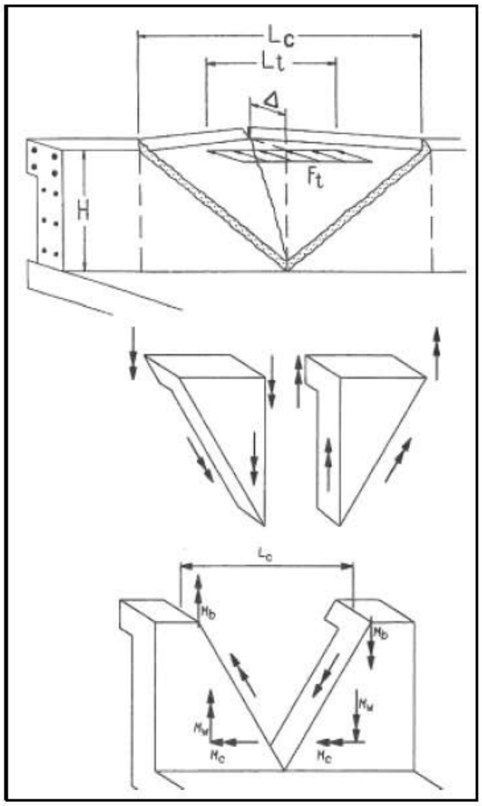

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 12.341 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 33.118 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot H_w \cdot (M_B + M_w)\right]}{M_{cmid}}} = 9.826 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot (L_{cmid})^2}{H_w}\right]\right] = 90.949 \cdot kip$$ (Equation A13.3.1-1)

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(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

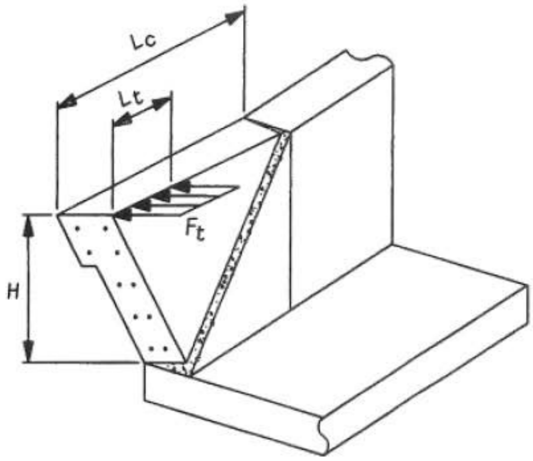

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3e

$M_B = 0$ — No additional beam strength

$M_w = 33.118 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 18.247 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 4.973 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\;2}\right)}{h_w}\right] = 68.059 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$H_W = 32 \cdot in$ — Height of the Concrete Barrier measured from the top of the roadway surface (in.)

$h_W = 32 \cdot in$

$R_{wmid} = 90.949 \cdot kip$ — Ultimate Resistance of the wall at midspan (kip)

$R_{wend} = 68.059 \cdot kip$ — Ultimate Resistance of the wall at the end of the wall or at a joint (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$$R_{R.mid} := R_{wmid} \cdot \left(\frac{h_W}{H_e + t_o}\right) = 153.178 \cdot kip$$

Structural Capacity of the Barrier at midspan located at H_e (kip)

$$R_{R.end} := R_{wend} \cdot \left(\frac{h_W}{H_e + t_o}\right) = 114.625 \cdot kip$$

Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.mid} \ge F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.end} \ge F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

(4) Strength Analysis of the Seperate End Post:

(4a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

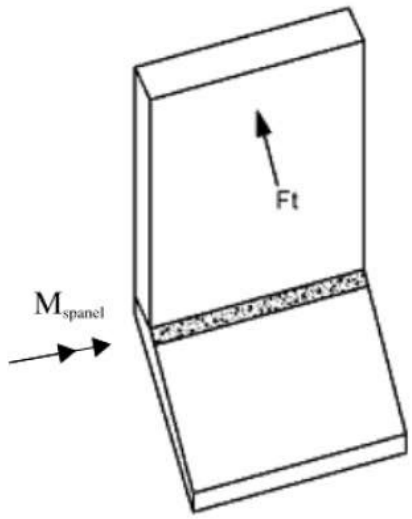

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 32in$ Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 36in$ Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 0.79in^2$ Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
#8 Bars

$n_{p.spost} := 4$ Number of parapet vertical reinforcement in the end post (in.)

(4a-conti.) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 3.16 \cdot in^2$ — Total Area of parapet vertical reinforment in the tension zone of the end post (in^2)

$a_{p.spost} := \dfrac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.549 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 8.5in$ — Average extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{p.spost}}{2}\right) = 122.063 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1in^2$ — Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2) #9 Bars

$n_{a.spost} := 4$ — Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 4 \cdot in^2$ — Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.961 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 14in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 260.392 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 122.063 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

(4a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 122.063 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$$R_{s.post} := \frac{M_{s.post}}{H_e} = 77.092 \cdot kip$$

Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \begin{pmatrix} \text{"OK"} & \text{if } R_{s.post} \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{pmatrix}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"OK"}$$

(4b) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

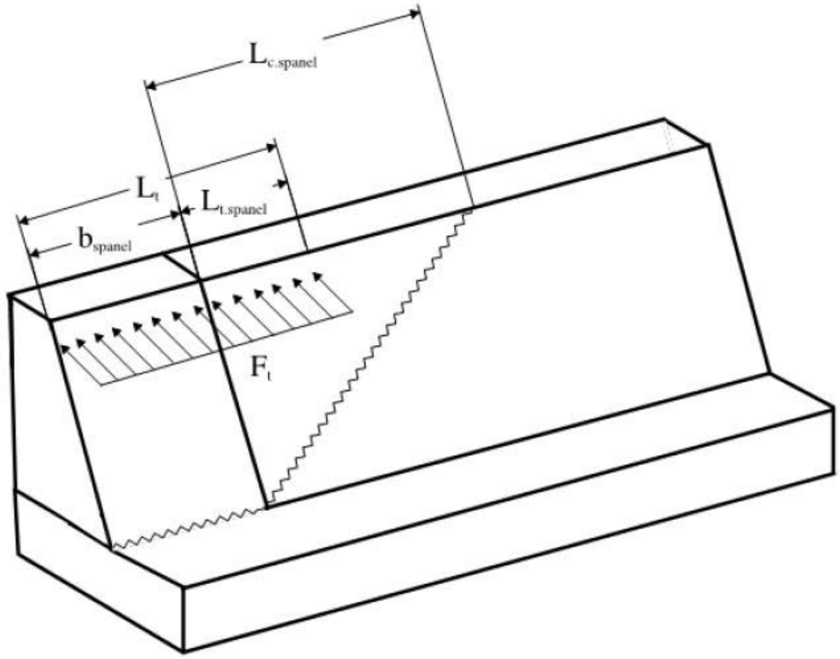

Figure 4b. Flexural Strength and Yield Line Analysis of the End Post and the Contributing Barrier Segment.

Note: $R_{R.spost}$ is equal to the structural capacity of the end post plus the structural capacity of the end of the barrier considering a reduced L_t (L_{tspost}).

Structural Capacity at the End of the Barrier: ($R_{w.spost}$)

Note: $R_{w.spost}$ considers a reduced L_t called L_{tspost}

$b_{spost} = 3 \cdot ft$	Width of the End Post (ft.)
$L_t = 4 \cdot ft$	Length of the Distribution of the Impact Force (ft.)
$L_{t.spost} := L_t - b_{spost} = 1 \cdot ft$	Distribution Length of the Impact Force acting at the End of the Barrier (ft.)

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SUBJECT: **MnDOT J-Barrier**
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(4b-conti.) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

$H_w = 32 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$M_{cend} = 18.247 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$M_B = 0$ — No beam addition to the barrier

$M_w = 33.118 \cdot kip \cdot ft$ — Flexural Resistance of the barrier about the vertical axis (kip-ft)

$$L_{c.spost} := \frac{L_{t.spost}}{2} + \sqrt{\left(\frac{L_{t.spost}}{2}\right)^2 + H_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 2.756 \cdot ft$$

Length of the ultimate resistance at the end of the barrier segment (ft.)
-Modified Equation A13.3.1-4

$$R_{end} := \left(\frac{2}{2 \cdot L_{c.spost} - L_{t.spost}}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{c.spost}^{\ 2}\right)}{H_w}\right] = 37.717 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_w (kip)
-Modified Equation A13.3.1-3

$$R_{w.spost} := R_{end} \cdot \left(\frac{H_w}{H_e}\right) = 63.524 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_e (kip)

$R_{s.post} = 77.092 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$$R_{R.spost} := R_{w.spost} + R_{s.post} = 140.616 \cdot kip$$

Structural Capacity of the end post and contributing segment of barrier (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} := \left(\begin{array}{l} \text{"OK"} \quad \text{if} \quad R_{R.spost} \geq F_t \\ \text{"NOT OK"} \quad \text{otherwise} \end{array} \right)$$

Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

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(5) Shear Capacity of the Barrier:

$\lambda := 1.0$	Concrete Modification Factor
$T_w := 9in$	Top Width of the parapet (in.)
$h_c := 15in$	Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)
$d_c := 7.5in$	Distance from compression face to the tension reinforcement (in.)
$L_t = 4 \cdot ft$	Length of the distribution of the impact force (ft.)
$f'_c = 4 \cdot ksi$	Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right) \cdot T_w\right] + 2 \cdot \left[\left(h_c + \frac{d_c}{2}\right) \cdot T_w\right] = 837 \cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in^2)

$$V_{c.int} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.int} = 105.873 \cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right) \cdot T_w + \left(h_c + \frac{d_c}{2}\right) \cdot T_w = 634.5 \cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in^2)

$$V_{c.end} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.end} = 80.259 \cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 80.259 \cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$$F_t = 71 \cdot kip$$

Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } V_c \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

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(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

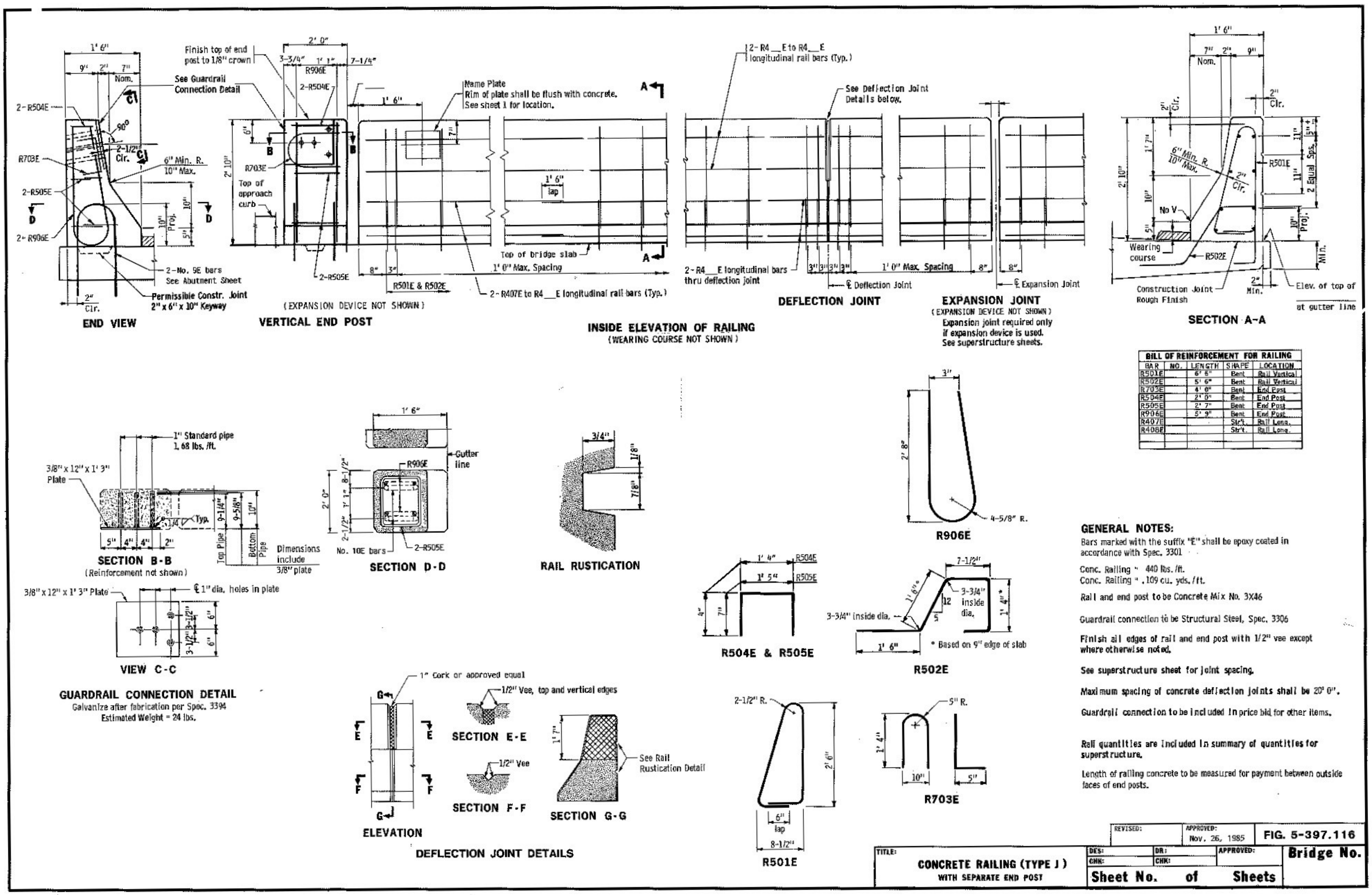
Structural_Capacity_of_End_Post_Check = "OK"

Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier from Figure 5-397.114 does satisfy all MASH TL-3 Criteria

APPENDIX B3: J BARRIER ON FIGURE 5-397.116



BILL OF REINFORCEMENT FOR RAILING

BAR	NO.	LENGTH	SHAPE	LOCATION
R501E		6' 6"	Bent	Rail Vertical
R502E		5' 6"	Bent	Rail Vertical
R703E		4' 0"	Bent	End Post
R504E		2' 0"	Bent	End Post
R505E		2' 7"	Bent	End Post
R906E		5' 9"	Bent	End Post
R407E			Str't.	Rail Long.
R408E			Str't.	Rail Long.

GENERAL NOTES:

Bars marked with the suffix "E" shall be epoxy coated in accordance with Spec. 3301

Conc. Railing = 440 lbs./ft.
Conc. Railing = .109 cu. yds./ft.

Rail and end post to be Concrete Mix No. 3X46

Guardrail connection to be Structural Steel, Spec. 3306

Finish all edges of rail and end post with 1/2" vee except where otherwise noted.

See superstructure sheet for joint spacing.

Maximum spacing of concrete deflection joints shall be 20' 0".

Guardrail connection to be included in price bid for other items.

Rail quantities are included in summary of quantities for superstructure.

Length of railing concrete to be measured for payment between outside faces of end posts.

REVISED:	APPROVED: Nov. 26, 1985	FIG. 5-397.116

TITLE: **CONCRETE RAILING (TYPE J)**
WITH SEPARATE END POST

DES: DR: APPROVED:
CHK: CHK:

Sheet No. of Sheets

Bridge No.

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.116
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 32in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 2in$	Thickness of overlay (in)
$h_w := H_w + t_o$	Total height of the barrier (in.)

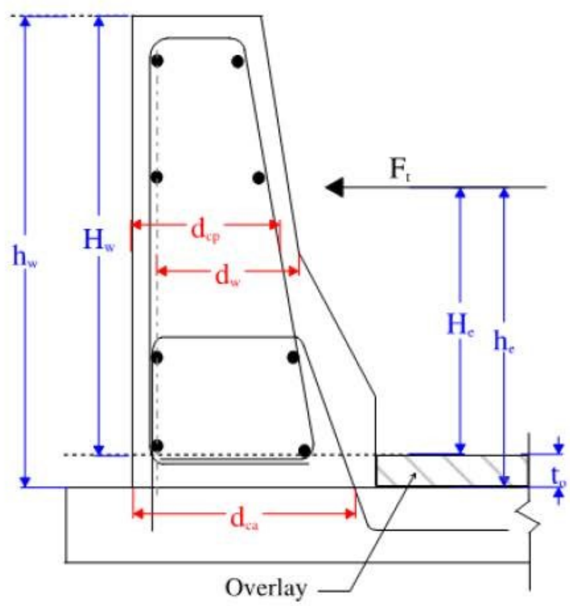

Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b) Concrete Parapet Inputs:

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{vp.mid} := 12in$	Spacing of parapet vertical reinforcement at midspan (in.)
$d_{cp.mid} := 9in$	Extreme distance of parapet vertical reinforcement in tension at midspan (in.)
$A_{vp1.end} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{vp.end} := 12in$	Spacing of parapet vertical reinforcement at joints/ends (in.)
$d_{cp.end} := 9in$	Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$	Area of longitudinal reinforcement bars in tension (in^2)
$d_w := 8.125in$	Extreme distance of tension longitudinal reinforcement of wall (in.)

(1b-conti.) <u>Concrete Parapet Inputs:</u>

<u>Deck Anchorage Vertical Reinforcement Inputs:</u>

$L_{proj\_R502E} := 10in$ — Projected length of R502E reinforcement over the slab (in.)

$L_{wid\_R502E} := 7.5in$ — Outer width of R502E reinforcement (in.)

$Cover := 2in$ — Cover clear distance (in.)

$Ratio_{R502E} := \frac{5}{12}$ — Inclined angle of R502E reinforcement

$d_{b\_R502E} := 0.625in$ — Nominal diameter of R502E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_R502E} + L_{proj\_R502E} \cdot Ratio_{R502E} + Cover - \frac{1}{2} d_{b\_R502E} = 13.354 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ — Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 12in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := d_{ca} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

SUBJECT: **MnDOT J-Barrier**
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(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load from top of overlay

$h_e := H_e + t_o$ Total equivalent transverse impact height (in.)

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

SUBJECT: MnDOT J-Barrier
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MASH TL-3 Compliance Assessment

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$

Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 9 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 13.597 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(3a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$ — Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$ — Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 20.346 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 13.597 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 12 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.31 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 9 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 13.597 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 12 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.31 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$ — Depth of Whitney Stress Block (in.)

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(3b-conti.) **Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends:** M_{cend}

$d_{ca.end} = 13.354 \cdot in$

Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 20.346 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 13.597 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(3c) **Bending Capacity of the Wall about the Vertical Axis:** M_w

$d_w = 8.125 \cdot in$

Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$

Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 34 \cdot in$

Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.415 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 31.67 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

(3d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

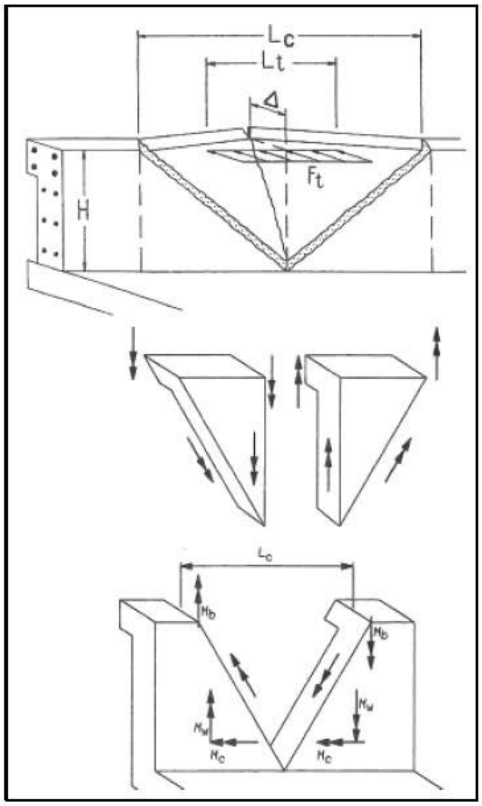

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 13.597 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 31.67 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.536 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 91.526 \cdot kip$$ (Equation A13.3.1-1)

SUBJECT: **MnDOT J-Barrier**
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(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

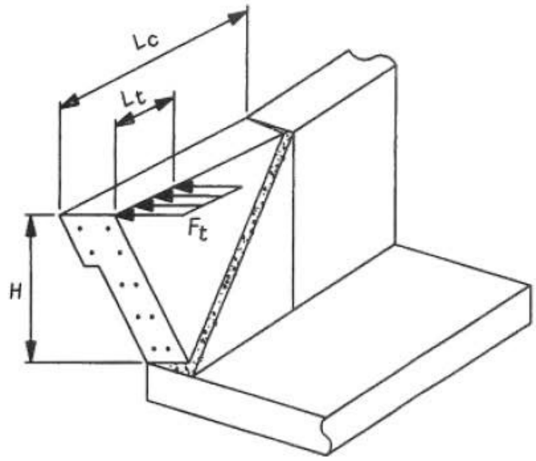

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)

$M_B = 0$ — No additional beam strength

$M_w = 31.67 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 13.597 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 5.256 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^2\right)}{h_w}\right] = 50.442 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$h_w = 34 \cdot in$ — Total Height of the Concrete Barrier measured from the top of the roadway surface (in.)

$R_{wmid} = 91.526 \cdot kip$ — Ultimate Resistance of the wall at midspan (kip)

$R_{wend} = 50.442 \cdot kip$ — Ultimate Resistance of the wall at the end of the wall or at a joint (kip)

$h_e = 21 \cdot in$ — Total Height of the Transverse Impact Force, F_t (in.)

$R_{R.mid} := R_{wmid} \cdot \left(\frac{h_w}{H_e + t_o} \right) = 148.185 \cdot kip$ — Structural Capacity of the Barrier at midspan located at H_e (kip)

$R_{R.end} := R_{wend} \cdot \left(\frac{h_w}{H_e + t_o} \right) = 81.668 \cdot kip$ — Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.mid} \ge F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.end} \ge F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

(4) Strength Analysis of the Seperate End Post:

(4a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

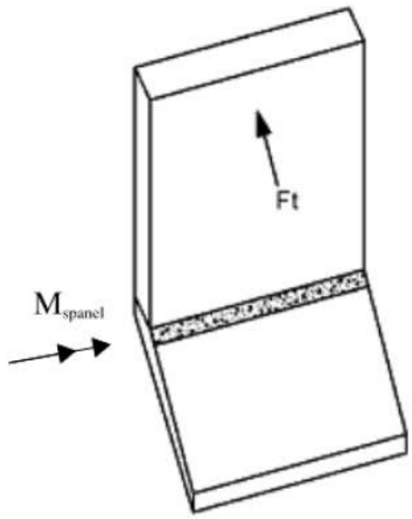

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 32in$ — Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 24in$ — Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 1in^2$ — Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
#9 Bars

$n_{p.spost} := 2$ — Number of parapet vertical reinforcement in the end post (in.)

(4a-conti.) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 2 \cdot in^2$ — Total Area of parapet vertical reinforment in the tension zone of the end post (in2)

$a_{p.spost} := \frac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.471 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 9in$ — Average extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \frac{a_{p.spost}}{2}\right) = 82.647 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1in^2$ — Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in2)
#9 Bars

$n_{a.spost} := 2$ — Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 2 \cdot in^2$ — Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in2)

$a_{a.spost} := \frac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.471 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 14in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \frac{a_{a.spost}}{2}\right) = 132.647 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 82.647 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

(4a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 82.647 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \dfrac{M_{s.post}}{H_e} = 52.198 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \begin{pmatrix} \text{"OK"} & \text{if } R_{s.post} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{pmatrix}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"NOT OK"}$$

(4b) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

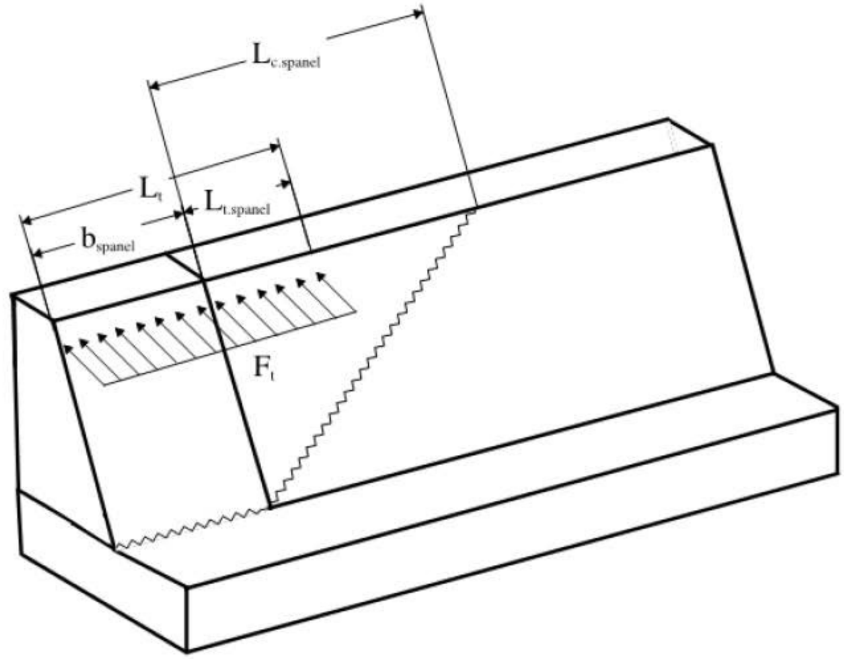

Figure 4b. Flexural Strength and Yield Line Analysis of the End Post and the Contributing Barrier Segment.

Note: $R_{R.spost}$ is equal to the structural capacity of the end post plus the structural capacity of the end of the barrier considering a reduced L_t ($L_{t.spost}$).

Structural Capacity at the End of the Barrier: ($R_{w.spost}$)

Note: $R_{w.spost}$ considers a reduced L_t called $L_{t.spost}$

$b_{spost} = 2 \cdot ft$	Width of the End Post (ft.)
$L_t = 4 \cdot ft$	Length of the Distribution of the Impact Force (ft.)
$L_{t.spost} := L_t - b_{spost} = 2 \cdot ft$	Distribution Length of the Impact Force acting at the End of the Barrier (ft.)

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(4b-conti.) <u>Structural Capacity of the End Post and the End of the Barrier</u>: $R_{R.spost}$

$(H_w) = 32 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$M_{cend} = 13.597 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$M_B = 0$ — No beam addition to the barrier

$M_w = 31.67 \cdot kip \cdot ft$ — Flexural Resistance of the barrier about the vertical axis (kip-ft)

$$L_{c.spost} := \frac{L_{t.spost}}{2} + \sqrt{\left(\frac{L_{t.spost}}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 3.757 \cdot ft$$

Length of the ultimate resistance at the end of the barrier segment (ft.)
-Modified Equation A13.3.1-4

$$R_{end} := \left(\frac{2}{2 \cdot L_{c.spost} - L_{t.spost}}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{c.spost}{}^2\right)}{h_w}\right] = 36.056 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_w (kip)
-Modified Equation A13.3.1-3

$$R_{w.spost} := R_{end} \cdot \left(\frac{h_w}{H_e + t_o}\right) = 58.376 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_e (kip)

$R_{s.post} = 52.198 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$$R_{R.spost} := R_{w.spost} + R_{s.post} = 110.574 \cdot kip$$

Structural Capacity of the end post and contributing segment of barrier (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.spost} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

(5) Shear Capacity of the Barrier:

$\lambda := 1.0$	Concrete Modification Factor
$T_w := 9in$	Top Width of the parapet (in.)
$h_c := 15in$	Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)
$d_c := 8.25in$	Distance from compression face to the tension reinforcement (in.)
$L_t = 4 \cdot ft$	Length of the distribution of the impact force (ft.)
$f'_c = 4 \cdot ksi$	Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right) \cdot T_w\right] + 2 \cdot \left[\left(h_c + \frac{d_c}{2}\right) \cdot T_w\right] = 850.5 \cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in2)

$$V_{c.int} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.int} = 107.581 \cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right) \cdot T_w + \left(h_c + \frac{d_c}{2}\right) \cdot T_w = 641.25 \cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in2)

$$V_{c.end} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.end} = 81.112 \cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 81.112 \cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$$F_t = 71 \cdot kip$$

Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \left(\begin{array}{|l} \text{"OK"} \quad \text{if } V_c \geq F_t \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array} \right) = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

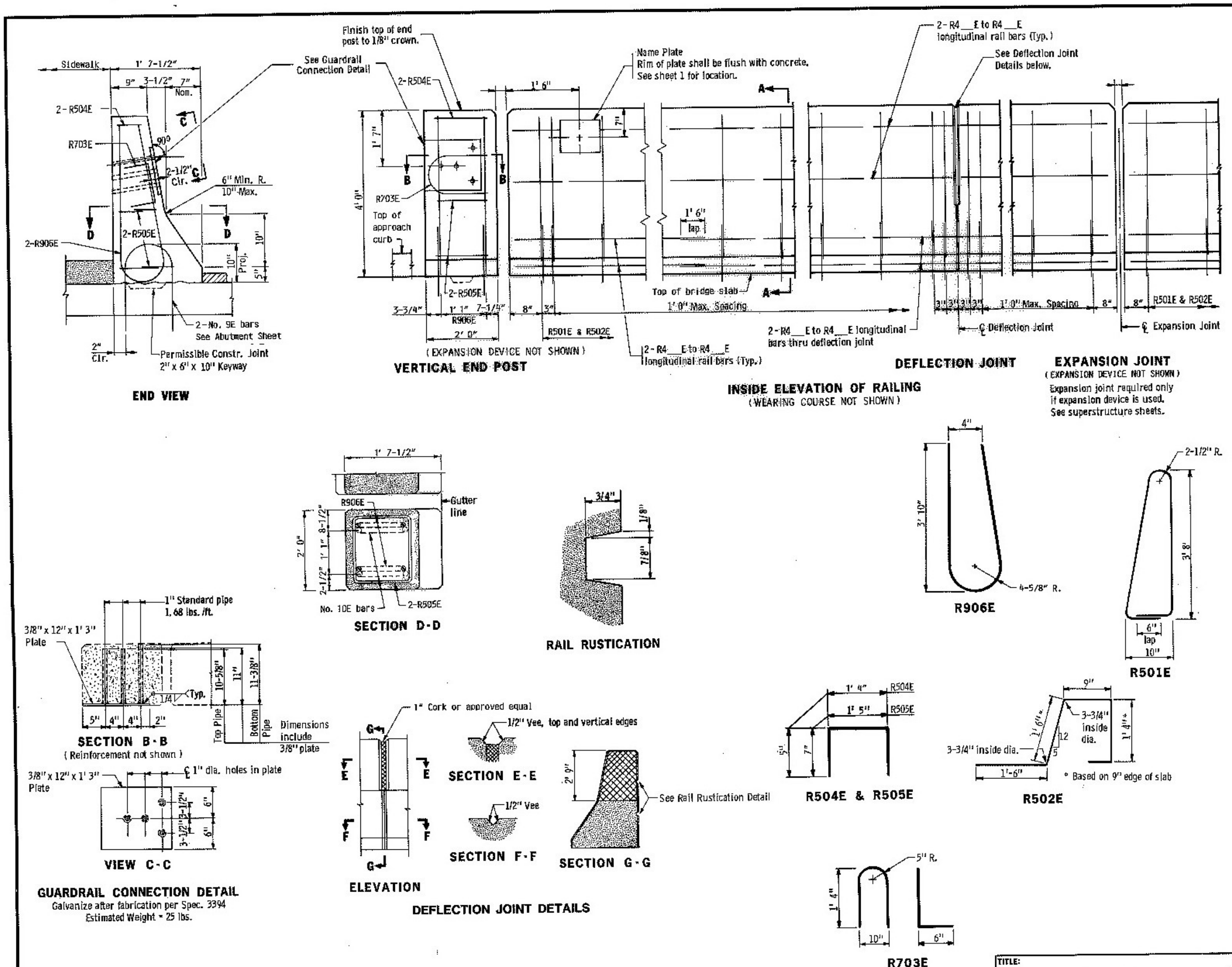
Structural_Capacity_of_End_Post_Check = "NOT OK"

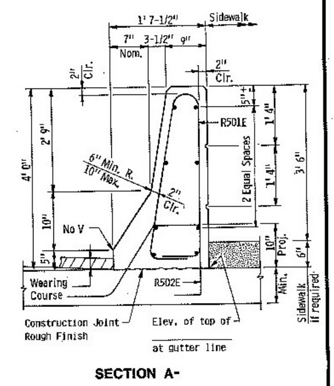
Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier from Figure 5-397.116 does not satisfy all MASH TL-3 Criteria

APPENDIX B4: J BARRIER ON FIGURE 5-397.118





BILL OF REINFORCEMENT FOR RAILING				
BAR	NO.	LENGTH	SHAPE	LOCATION
R501E		8' 11"	Bent	Rail Vertical
R502E		5' 7"	Bent	Rail Vertical
R703E		4' 2"	Bent	End Post
R504E		2' 2"	Bent	End Post
R505E		2' 7"	Bent	End Post
R906E		8' 1"	Bent	End Post
R407E			Str't.	Rail Long.
R408E			Str't.	Rail Long.

GENERAL NOTES:

Conc. Railing = 640 lbs./ft.
Conc. Railing = .158 cu. yds./ft.

Rail and end post to be Concrete Mix No. 3X46

Guradrail connection to be Structural Steel, Spec. 3306

Finish all edges of rail and end post with 1/2" vee except where otherwise noted.

See superstructure sheet for joint spacing.

Maximum spacing of concrete deflection joints shall be 20' 0"

Guardrail connection to be included in price bid for other items.

Rail quantities are included in summary of quantities for superstructure.

Length of railing concrete to be measured for payment between outside faces of end posts.

Bars marked with the suffix "E" shall be epoxy coated in accordance with Spec. 3301

REVISED:	APPROVED: Nov. 26, 1985	FIG. 5-397.118

TITLE: CONCRETE RAILING (TYPE J-SW) WITH SIDEWALK AND SEPARATE END POST

DES: | DR: | APPROVED: | CHK: | CHK:

Bridge No.

Sheet No. of Sheets

November 26, 1985 5-397.118

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.118
MASH Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 46in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 2.0in$	Thickness of overlay (in)
$h_w := H_w + t_o$	Total height of the barrier (in.)

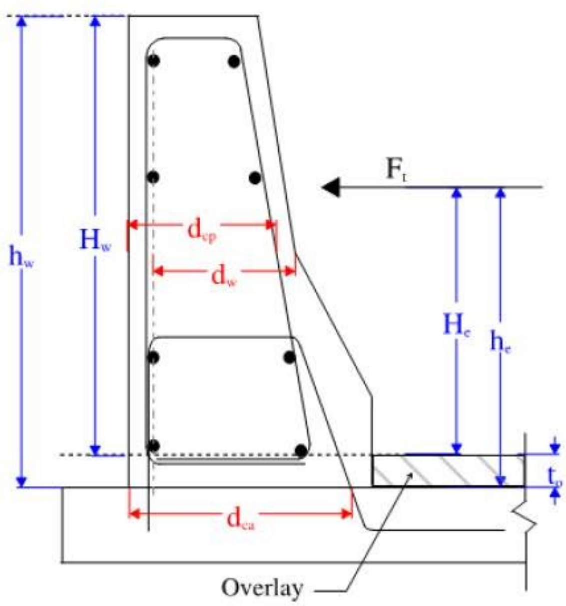


Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b) Concrete Parapet Inputs:

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{vp.mid} := 12in$ — Spacing of parapet vertical reinforcement at midspan (in.)

$d_{cp.mid} := 10.18in$ — Extreme distance of parapet vertical reinforcement in tension at midspan (in.)

$A_{vp1.end} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 9.6in$ — Spacing of parapet vertical reinforcement at joints/ends (in.) (5 bars over 48 inches)

$d_{cp.end} := 10.18in$ — Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$ — Area of longitudinal reinforcement bars in tension (in^2)

$d_w := 9.625in$ — Extreme distance of tension longitudinal reinforcement of wall (in.)

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SUBJECT: MnDOT J-Barrier
Figure 5-397.118
MASH TL-3 Compliance Assessment

(1b-conti.) Concrete Parapet Inputs:

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502E} := 10in$	Projected length of R502E reinforcement over the slab (in.)
$L_{wid\_R502E} := 9in$	Outer width of R502E reinforcement (in.)
$Cover := 2in$	Cover clear distance (in.)
$Ratio_{R502E} := \frac{5}{12}$	Inclined angle of R502E reinforcement
$d_{b\_R502E} := 0.625in$	Nominal diameter of R502E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_R502E} + L_{proj\_R502E} \cdot Ratio_{R502E} + Cover - \frac{1}{2} d_{b\_R502E} = 14.854 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

Deck Anchorage Vertical Reinforcement Inputs:

$A_{va1.mid} := 0.31in^2$	Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{va.mid} := 12in$	Spacing of deck anchorage vertical reinforcement at midspan (in.)
$d_{ca.mid} := d_{ca} = 14.854 \cdot in$	Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)
$A_{va1.end} := 0.31in^2$	Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{va.end} := 9.6in$	Spacing of deck anchorage vertical reinforcement at joints/ends (in.) (5 bars over 4 feet average)
$d_{ca.end} := d_{ca} = 14.854 \cdot in$	Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.118
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(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 46 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 46 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

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SUBJECT: MnDOT J-Barrier
Figure 5-397.118
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(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$

Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 10.18 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 15.426 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(3a-conti.) **Bending Capacity of the Wall about the Longitudinal Axis at Midspan:** M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 14.854 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 22.671 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 15.426 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

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(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 9.6 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.388 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.57 \cdot in$$

Depth of Whitney Stress Block (in.)

$d_{cp.end} = 10.18 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 19.172 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 9.6 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.388 \cdot in^2$$

Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.57 \cdot in$$

Depth of Whitney Stress Block (in.)

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(3b-conti.) **Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends:** M_{cend}

$d_{ca.end} = 14.854 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 28.228 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 19.172 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(3c) **Bending Capacity of the Wall about the Vertical Axis:** M_w

$d_w = 9.625 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 48 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.294 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 37.912 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.118
MASH TL-3 Compliance Assessment

(3d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

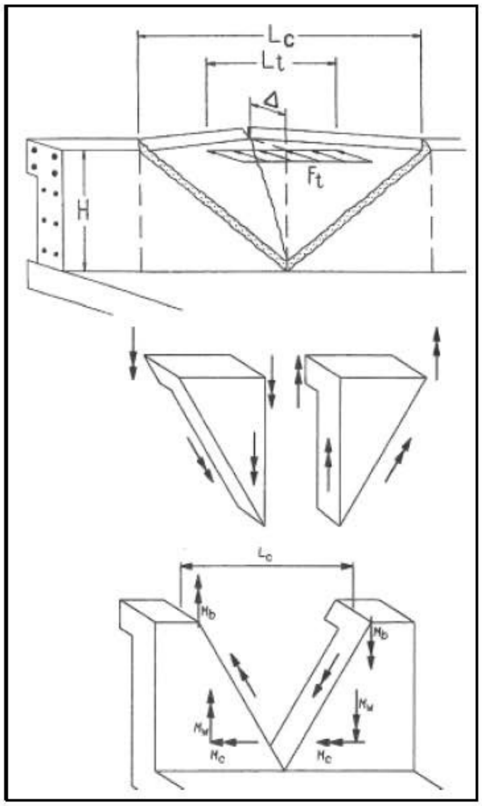

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 46 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 15.426 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 37.912 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 11.091 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 85.543 \cdot kip$$ (Equation A13.3.1-1)

(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

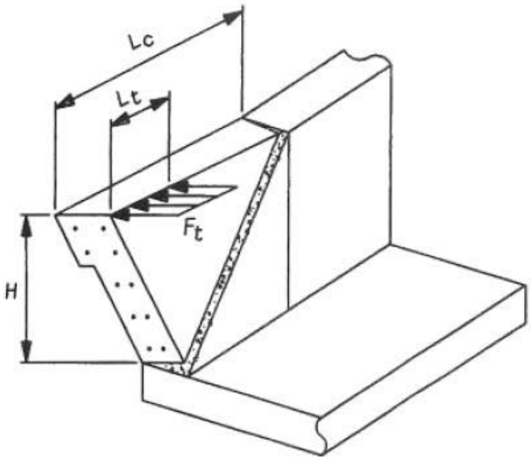

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 46 \cdot in$	Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3e
$M_B = 0$	No additional beam strength
$M_w = 37.912 \cdot kip \cdot ft$	Flex. Resistance of the Wall about the Vert. Axis (k-ft)
$L_t = 4 \cdot ft$	Longitudinal length of distribution of impact force (ft.)
$M_{cend} = 19.172 \cdot \frac{kip \cdot ft}{ft}$	Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 5.451 \cdot ft$$ (Equation A13.3.1-4)

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\ 2}\right)}{h_w}\right] = 52.253 \cdot kip$$ (Equation A13.3.1-3)

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SUBJECT: MnDOT J-Barrier
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(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$H_w = 46 \cdot in$ — Height of the Concrete Barrier measured from the top of the roadway surface (in.)

$R_{wmid} = 85.543 \cdot kip$ — Ultimate Resistance of the wall at midspan (kip)

$R_{wend} = 52.253 \cdot kip$ — Ultimate Resistance of the wall at the end of the wall or at a joint (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$R_{R.mid} := R_{wmid} \cdot \left(\frac{h_w}{H_e + t_o} \right) = 195.527 \cdot kip$ — Structural Capacity of the Barrier at midspan located at H_e (kip)

$R_{R.end} := R_{wend} \cdot \left(\frac{H_w}{H_e + t_o} \right) = 114.459 \cdot kip$ — Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

Structural_Capacity_of_Barrier_at_Midspan_Check := | "OK" if $R_{R.mid} \ge F_t$
| "NOT OKAY" otherwise

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_Ends_Check := | "OK" if $R_{R.end} \ge F_t$
| "NOT OKAY" otherwise

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

(4) Strength Analysis of the Seperate End Post:

(4a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

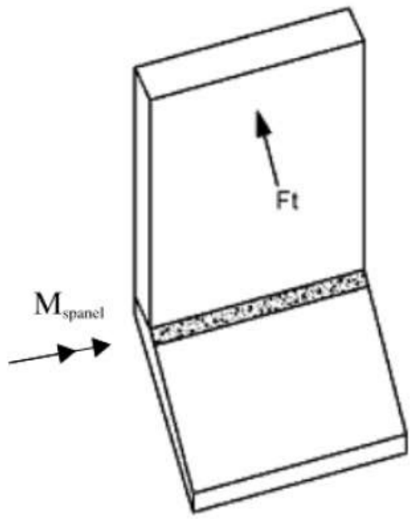

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 46in$ — Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 24in$ — Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 1in^2$ — Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
#9 Bars

$n_{p.spost} := 2$ — Number of parapet vertical reinforcement in the end post (in.)

(4a-conti.) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 2 \cdot in^2$

Total Area of parapet vertical reinforment in the tension zone of the end post (in^2)

$a_{p.spost} := \dfrac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.471 \cdot in$

Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 9.875in$

Average extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{p.spost}}{2}\right) = 91.397 \cdot kip \cdot ft$

Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1in^2$

Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2)
#9 Bars

$n_{a.spost} := 2$

Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 2 \cdot in^2$

Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.471 \cdot in$

Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 14in$

Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 132.647 \cdot kip \cdot ft$

Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 91.397 \cdot kip \cdot ft$

Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

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SUBJECT: MnDOT J-Barrier
Figure 5-397.118
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(4a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t(in.)

$M_{s.post} = 91.397 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \dfrac{M_{s.post}}{H_e + t_o} = 52.227 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \left(\begin{array}{|l} \text{"OK"} \quad \text{if} \quad R_{s.post} \geq F_t \\ \text{"NOT OK"} \quad \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"NOT OK"}$$

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(4b) **Structural Capacity of the End Post and the End of the Barrier**: $R_{R.spost}$

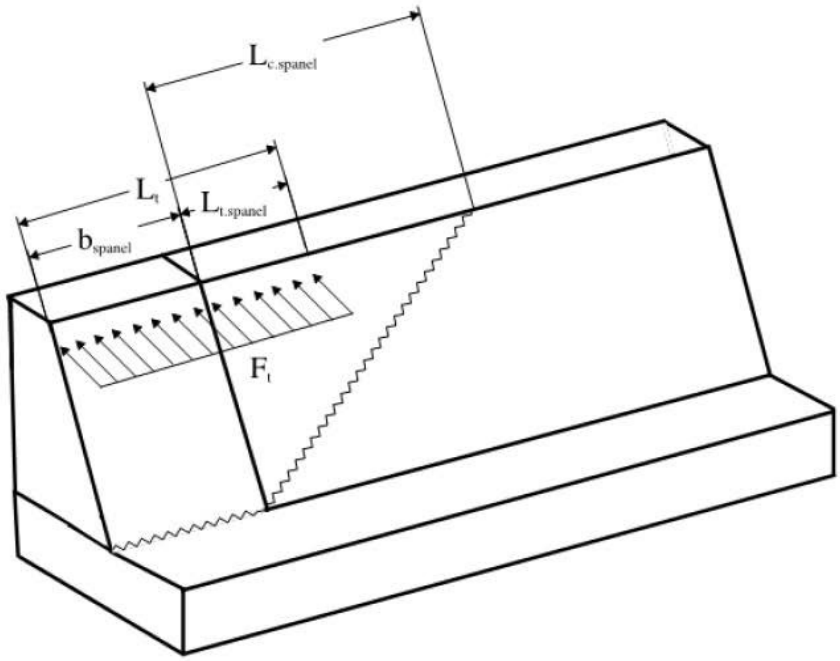

Figure 4b. Flexural Strength and Yield Line Analysis of the End Post and the Contributing Barrier Segment.

Note: $R_{R.spost}$ is equal to the structural capacity of the end post plus the structural capacity of the end of the barrier considering a reduced L_t ($L_{t.spost}$).

Structural Capacity at the End of the Barrier: ($R_{w.spost}$)
Note: $R_{w.spost}$ considers a reduced L_t called $L_{t.spost}$

$b_{spost} = 2 \cdot ft$ — Width of the End Post (ft.)

$L_t = 4 \cdot ft$ — Length of the Distribution of the Impact Force (ft.)

$L_{t.spost} := L_t - b_{spost} = 2 \cdot ft$ — Distribution Length of the Impact Force acting at the End of the Barrier (ft.)

SUBJECT: MnDOT J-Barrier
Figure 5-397.118
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(4b-conti.) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

$H_w = 46 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$M_{cend} = 19.172 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$M_B = 0$ — No beam addition to the barrier

$M_w = 37.912 \cdot kip \cdot ft$ — Flexural Resistance of the barrier about the vertical axis (kip-ft)

$$L_{c.spost} := \frac{L_{t.spost}}{2} + \sqrt{\left(\frac{L_{t.spost}}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 3.985 \cdot ft$$

Length of the ultimate resistance at the end of the barrier segment (ft.)
-Modified Equation A13.3.1-4

$$R_{end} := \left(\frac{2}{2 \cdot L_{c.spost} - L_{t.spost}}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{c.spost}^{\,2}\right)}{h_w}\right] = 38.199 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_w (kip)
-Modified Equation A13.3.1-3

$$R_{w.spost} := R_{end} \cdot \left(\frac{h_w}{H_e + t_o}\right) = 87.312 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_e (kip)

$R_{s.post} = 52.227 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$$R_{R.spost} := R_{w.spost} + R_{s.post} = 139.539 \cdot kip$$

Structural Capacity of the end post and contributing segment of barrier (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.spost} \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} = \text{"OK"}$$

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Figure 5-397.118
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(5) Shear Capacity of the Barrier:

$\lambda := 1.0$ — Concrete Modification Factor

$T_W := 9in$ — Top Width of the parapet (in.)

$h_c := 15in$ — Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)

$d_c := 8in$ — Distance from compression face to the tension reinforcement (in.)

$L_t = 4 \cdot ft$ — Length of the distribution of the impact force (ft.)

$f'_c = 4 \cdot ksi$ — Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right)\cdot T_W\right] + 2\cdot\left[\left(h_c + \frac{d_c}{2}\right)\cdot T_W\right] = 846\cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in^2)

$$V_{c.int} := 2\cdot\lambda\cdot\left[\left(\sqrt{\frac{f'_c}{psi}}\right)\cdot psi\right]\cdot A_{c.int} = 107.011\cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right)\cdot T_W + \left(h_c + \frac{d_c}{2}\right)\cdot T_W = 639\cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in^2)

$$V_{c.end} := 2\cdot\lambda\cdot\left[\left(\sqrt{\frac{f'_c}{psi}}\right)\cdot psi\right]\cdot A_{c.end} = 80.828\cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 80.828\cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$F_t = 71\cdot kip$ — Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } V_c \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

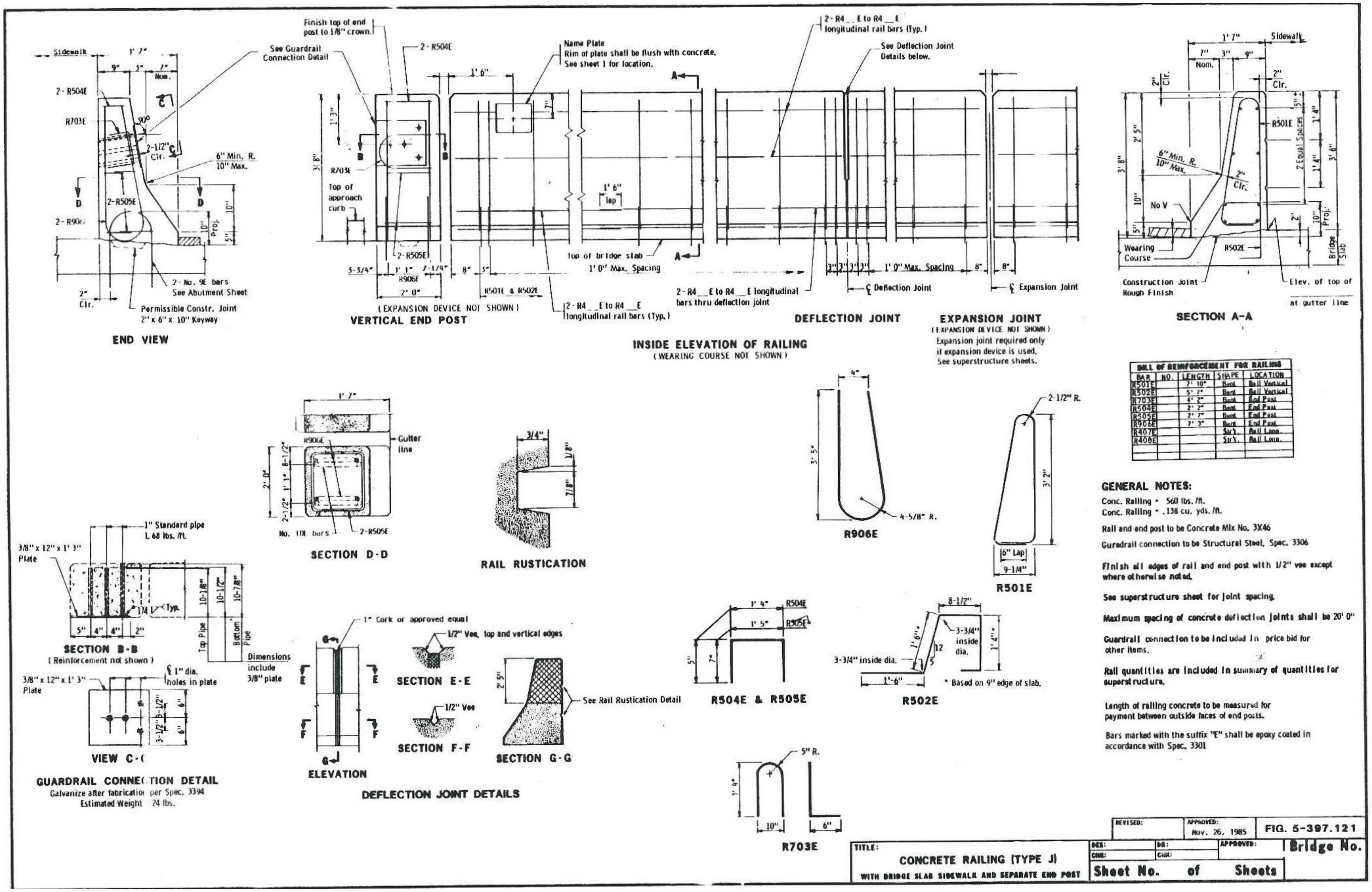
Structural_Capacity_of_End_Post_Check = "NOT OK"

Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier from Figure 5-397.118 does not satisfy all MASH TL-3 Criteria

APPENDIX B5: J BARRIER ON FIGURE 5-397.121



BILL OF REINFORCEMENT FOR RAILING				
BAR	NO.	LENGTH	SHAPE	LOCATION
R501E		7' 10"	Bent	Rail Vertical
R502E		5' 7"	Bent	Rail Vertical
R703E		4' 2"	Bent	End Post
R504E		2' 2"	Bent	End Post
R505E		2' 7"	Bent	End Post
R906E		7' 3"	Bent	End Post
R407E			Str't.	Rail Long.
R408E			Str't.	Rail Long.

GENERAL NOTES:

Conc. Railing - 560 lbs./ft.
Conc. Railing - .138 cu. yds./ft.

Rail and end post to be Concrete Mix No. 3X46

Guradrail connection to be Structural Steel, Spec. 3306

Finish all edges of rail and end post with 1/2" vee except where otherwise noted.

See superstructure sheet for joint spacing.

Maximum spacing of concrete deflection joints shall be 20' 0"

Guardrail connection to be included in price bid for other items.

Rail quantities are included in summary of quantities for superstructure.

Length of railing concrete to be measured for payment between outside faces of end posts.

Bars marked with the suffix "E" shall be epoxy coated in accordance with Spec. 3301

REVISED:	APPROVED: Nov. 26, 1985	FIG. 5-397.121

TITLE: CONCRETE RAILING (TYPE J)
WITH BRIDGE SLAB SIDEWALK AND SEPARATE END POST

DES: | DR: | APPROVED: | Bridge No.
CHK: | CHK:

Sheet No. of Sheets

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.121
MASH Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 42in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 2in$	Thickness of overlay (in.)
$h_w := H_w + t_o$	Total height of the barrier (in.)

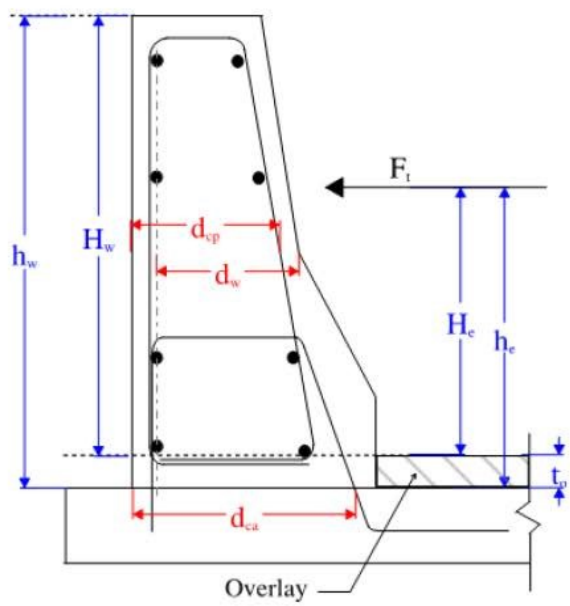

Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

SUBJECT: MnDOT J-Barrier
Figure 5-397.121
MASH Compliance Assessment

(1b) Concrete Parapet Inputs:

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$ Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{vp.mid} := 12in$ Spacing of parapet vertical reinforcement at midspan (in.)

$d_{cp.mid} := 9.875in$ Extreme distance of parapet vertical reinforcement in tension at midspan (in.)

$A_{vp1.end} := 0.31in^2$ Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 12in$ Spacing of parapet vertical reinforcement at joints/ends (in.)

$d_{cp.end} := 9.875in$ Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$ Area of longitudinal reinforcement bars in tension (in^2)

$d_w := 9.125in$ Extreme distance of tension longitudinal reinforcement of wall (in.)

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SUBJECT: MnDOT J-Barrier
Figure 5-397.121
MASH Compliance Assessment

(1b-conti.) Concrete Parapet Inputs:

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502E} := 10in$	Projected length of R502E reinforcement over the slab (in.)
$L_{wid\_R502E} := 8.5in$	Outer width of R502E reinforcement (in.)
$Cover := 2in$	Cover clear distance (in.)
$Ratio_{R502E} := \frac{5}{12}$	Inclined angle of R502E reinforcement
$d_{b\_R502E} := 0.625in$	Nominal diameter of R502E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_R502E} + L_{proj\_R502E} \cdot Ratio_{R502E} + Cover - \frac{1}{2} d_{b\_R502E} = 14.354 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$	Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{va.mid} := 12in$	Spacing of deck anchorage vertical reinforcement at midspan (in.)
$d_{ca.mid} := d_{ca} = 14.354 \cdot in$	Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)
$A_{va1.end} := 0.31in^2$	Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{va.end} := 12in$	Spacing of deck anchorage vertical reinforcement at joints/ends (in.)
$d_{ca.end} := d_{ca} = 14.354 \cdot in$	Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

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SUBJECT: MnDOT J-Barrier
Figure 5-397.121
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(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$h_e := H_e + t_o$ Total Height of Equivalent IMpact Force (in.)

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 42 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.121
MASH TL-3 Compliance Assessment

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 42 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$

Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 9.875 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 14.953 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(3a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in2)

$d_{ca.mid} = 14.354 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 21.896 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 14.953 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

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(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 12 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.31 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 9.875 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 14.953 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 12 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.31 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$ — Depth of Whitney Stress Block (in.)

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(3b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 14.354 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 21.896 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 14.953 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(3c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 9.125 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 44 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.321 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 35.858 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

(3d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

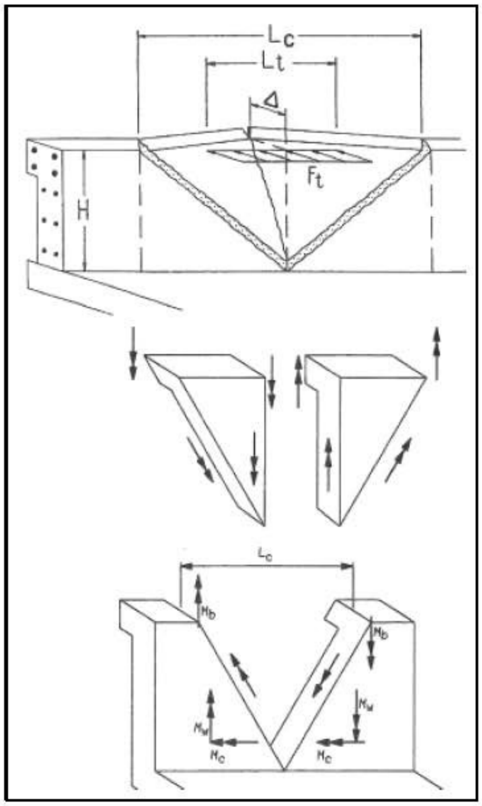

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 42 \cdot in$	Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: $H_w = H$ in Figure 3d
$M_B := 0 kip \cdot ft$	No additional beam strength
$M_{cmid} = 14.953 \cdot \frac{kip \cdot ft}{ft}$	Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)
$M_w = 35.858 \cdot kip \cdot ft$	Flex. Resistance of the Wall about the Vert. Axis (k-ft)
$L_t = 4 \cdot ft$	Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 10.622 \cdot ft$$

(Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 86.637 \cdot kip$$

(Equation A13.3.1-1)

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(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

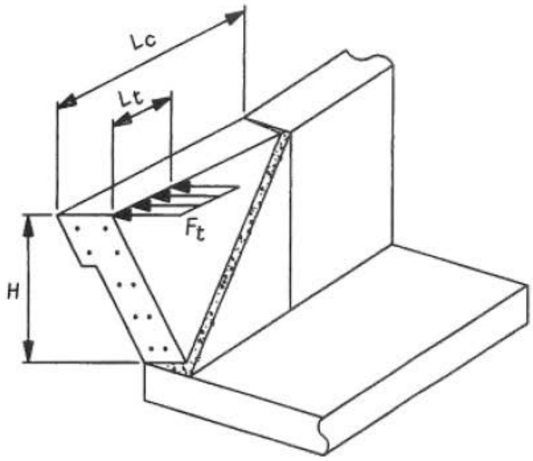

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 42 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)

$h_w = 44 \cdot in$

$M_B = 0$ — No additional beam strength

$M_w = 35.858 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 14.953 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 5.577 \cdot ft$$ (Equation A13.3.1-4)

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\ 2}\right)}{h_w}\right] = 45.485 \cdot kip$$ (Equation A13.3.1-3)

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(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$H_w = 42 \cdot in$	Height of the Concrete Barrier measured from the top of the roadway surface (in.)
$R_{wmid} = 86.637 \cdot kip$	Ultimate Resistance of the wall at midspan (kip)
$R_{wend} = 45.485 \cdot kip$	Ultimate Resistance of the wall at the end of the wall or at a joint (kip)
$H_e = 19 \cdot in$	Height of the Transverse Impact Force, F_t (in.)
$R_{R.mid} := R_{wmid} \cdot \left(\frac{h_w}{h_e}\right) = 181.525 \cdot kip$	Structural Capacity of the Barrier at midspan located at H_e (kip)
$R_{R.end} := R_{wend} \cdot \left(\frac{h_w}{h_e}\right) = 95.301 \cdot kip$	Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)
$F_t = 71 \cdot kip$	Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.mid} \geq F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} = \text{"OK"}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.end} \geq F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} = \text{"OK"}$$

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(4) Strength Analysis of the Seperate End Post:

(4a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

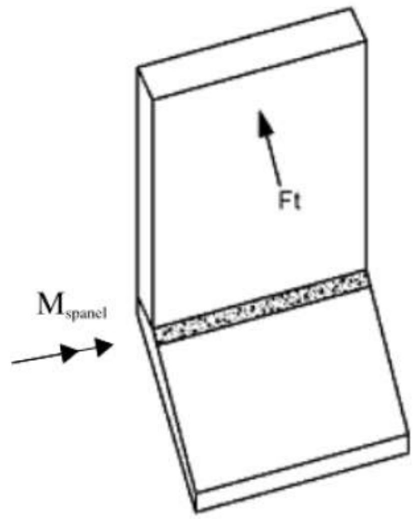

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 42in$ — Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 24in$ — Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 1in^2$ — Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
#9 Bars

$n_{p.spost} := 2$ — Number of parapet vertical reinforcement in the end post (in.)

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(4a-conti.) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 2 \cdot in^2$ — Total Area of parapet vertical reinforment in the tension zone of the end post (in^2)

$a_{p.spost} := \dfrac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.471 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 8.938in$ — Average extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{p.spost}}{2}\right) = 82.027 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1.27in^2$ — Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2) #10 Bars

$n_{a.spost} := 2$ — Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 2.54 \cdot in^2$ — Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.868 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 15in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 178.64 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 82.027 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

(4a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 82.027 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \frac{M_{s.post}}{h_e} = 46.873 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$Structural\_Capacity\_of\_End\_Post\_Check := \left(\begin{array}{|ll} \text{"OK"} & \text{if } R_{s.post} \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right)$$

$$Structural\_Capacity\_of\_End\_Post\_Check = \text{"NOT OK"}$$

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(4b) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

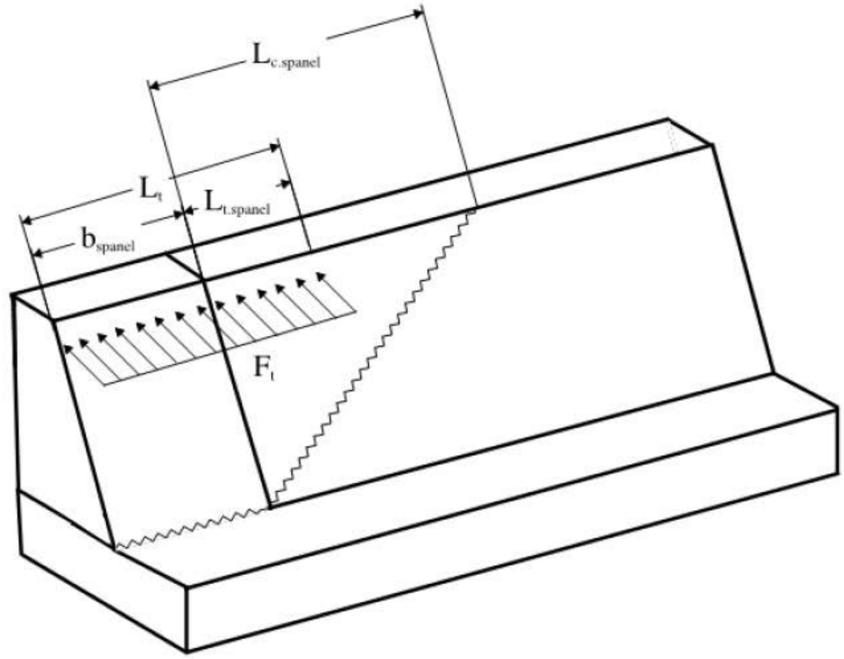

Figure 4b. Flexural Strength and Yield Line Analysis of the End Post and the Contributing Barrier Segment.

Note: $R_{R.spost}$ is equal to the structural capacity of the end post plus the structural capacity of the end of the barrier considering a reduced L_t ($L_{t.spost}$).

Structural Capacity at the End of the Barrier: ($R_{w.spost}$)

Note: $R_{w.spost}$ considers a reduced L_t called $L_{t.spost}$

$b_{spost} = 2 \cdot ft$ — Width of the End Post (ft.)

$L_t = 4 \cdot ft$ — Length of the Distribution of the Impact Force (ft.)

$L_{t.spost} := L_t - b_{spost} = 2 \cdot ft$ — Distribution Length of the Impact Force acting at the End of the Barrier (ft.)

(4b-conti.) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

$H_w = 42 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$M_{cend} = 14.953 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$M_B = 0$ — No beam addition to the barrier

$M_w = 35.858 \cdot kip \cdot ft$ — Flexural Resistance of the barrier about the vertical axis (kip-ft)

$$L_{c.spost} := \frac{L_{t.spost}}{2} + \sqrt{\left(\frac{L_{t.spost}}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 4.129 \cdot ft$$

Length of the ultimate resistance at the end of the barrier segment (ft.)
-Modified Equation A13.3.1-4

$$R_{end} := \left(\frac{2}{2 \cdot L_{c.spost} - L_{t.spost}}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{c.spost}^{\ 2}\right)}{h_w}\right] = 33.68 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_W (kip)
-Modified Equation A13.3.1-3

$$R_{w.spost} := R_{end} \cdot \left(\frac{h_w}{h_e}\right) = 70.567 \cdot kip$$

Structural Capacity at the end of the barrier segment using $L_{t.spost}$ located at H_e (kip) above road surface

$R_{s.post} = 46.873 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$$R_{R.spost} := R_{w.spost} + R_{s.post} = 117.44 \cdot kip$$

Structural Capacity of the end post and contributing segment of barrier (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.spost} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} = \text{"OK"}$$

(5) Shear Capacity of the Barrier:

$\lambda := 1.0$ Concrete Modification Factor

$T_w := 9in$ Top Width of the parapet (in.)

$h_c := 15in$ Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)

$d_c := 8in$ Distance from compression face to the tension reinforcement (in.)

$L_t = 4 \cdot ft$ Length of the distribution of the impact force (ft.)

$f'_c = 4 \cdot ksi$ Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right) \cdot T_w\right] + 2 \cdot \left[\left(h_c + \frac{d_c}{2}\right) \cdot T_w\right] = 846 \cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in^2)

$$V_{c.int} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.int} = 107.011 \cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right) \cdot T_w + \left(h_c + \frac{d_c}{2}\right) \cdot T_w = 639 \cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in^2)

$$V_{c.end} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.end} = 80.828 \cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 80.828 \cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$F_t = 71 \cdot kip$ Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \begin{vmatrix} \text{"OK"} & \text{if } V_c \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{vmatrix} = \text{"OK"}$$

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(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

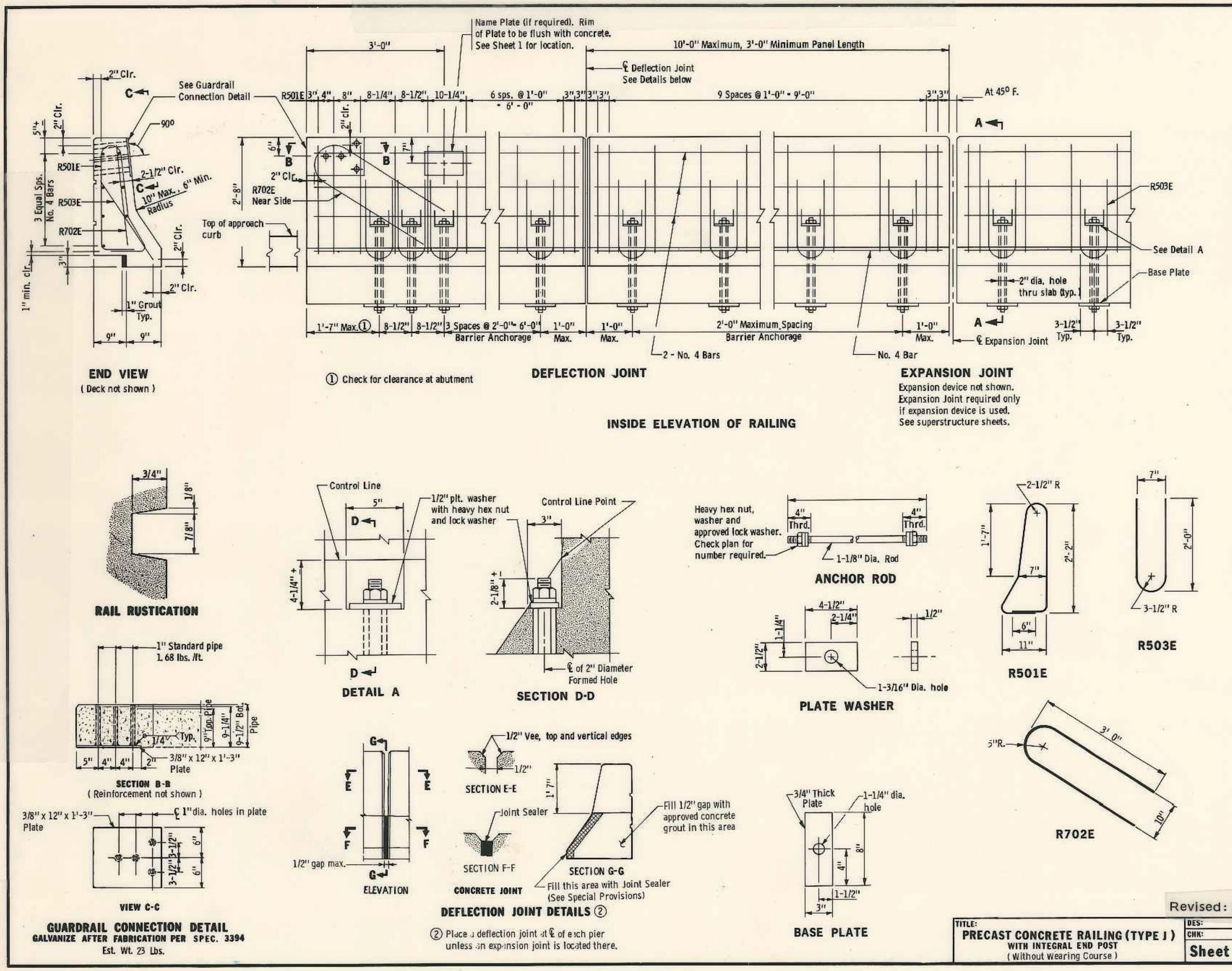
Structural_Capacity_of_End_Post_Check = "NOT OK"

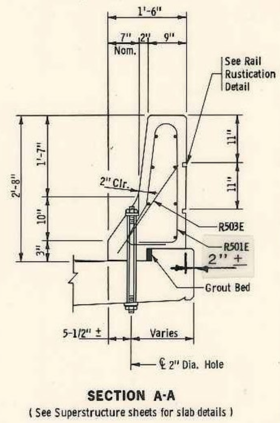
Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier from Figure 5-397.121 does not satisfy all MASH TL-3 Criteria

APPENDIX B6: J BARRIER ON FIGURE 5-397.141





GENERAL NOTES

Concrete Railing = 377 lbs. / ft.

Concrete Railing = .093 cu. yds. / ft.

Railing to be Concrete Mix No. 3X46.

Rods, plates and guardrail connections to be structural steel as per Spec. 3306 and shall be included in price bid for other items.

Rail quantities are included in summary of quantities for superstructure.

Finish all edges of rail with 1/2" vee except where otherwise noted.

Length of precast concrete railing to be measured for payment between outside faces of end of railing.

End of rail sections shall be marked to indicate location in field.

Short rail lengths shall have a minimum of two barrier anchorages.

All railing reinforcement, and bars marked with Suffix " E ", shall be Epoxy Coated in accordance with Special Provisions.

All precast rail sections to have an approved handling device.

All hardware is to be galvanized as per Spec. 3392 or 3394.

Revised: Sept. 12, 1980

FIG. 5-397.141
Approved: June 4, 1980

TITLE: PRECAST CONCRETE RAILING (TYPE J) WITH INTEGRAL END POST (Without Wearing Course)	DES:	DR:	APPROVED:	Bridge No.
	CHK:	CHK:		
	Sheet No.	of	Sheets	

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SUBJECT: **MnDOT J-Barrier**
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MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 32in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 0in$	Thickness of overlay (in.)
$h_w := H_w + t_o$	Total height of the barrier (in.)

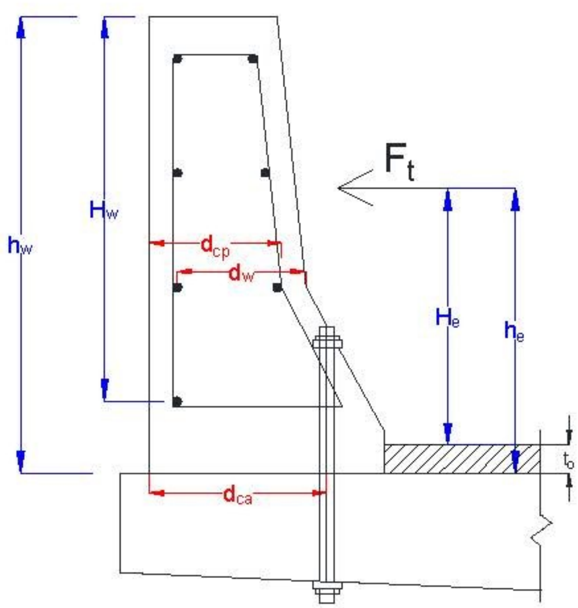

Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b) Concrete Parapet Inputs:

Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$	Area of one vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{vp.mid} := 12in$	Spacing of vertical reinforcement at midspan (in.)
$d_{cp.mid} := 8.6875in$	Extreme distance of vertical reinforcement in tension at midspan (in.)
$A_{vp1.end} := 0.31in^2$	Area of one vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{vp.end} := 12in$	Spacing of vertical reinforcement at joints/ends (in.)
$d_{cp.end} := 8.6875in$	Extreme distance of tension vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$	Area of longitudinal reinforcement bars in tension (in^2)
$d_w := 8.125in$	Extreme distance of tension longitudinal reinforcement of wall (in.)

Deck Anchorage Vertical Reinforcement Inputs:

$A_{va1.mid} := 1.0in^2$	Area of one deck anchorage vertical reinforcement leg in tension zone at midspan (in^2) (Bolts)
$s_{va.mid} := 24in$	Spacing of deck anchorage vertical reinforcement at midspan (in.) (Bolt spacing)
$d_{ca.mid} := 12.5in$	Extreme distance of tension deck anchorage vertical reinforcement (in.) (Bolts)
$A_{va1.end} := 1.0in^2$	Area of one deck anchorage vertical reinforcement leg in tension zone at end (in^2)
$s_{va.end} := 24in$	Spacing of deck anchorage vertical reinforcement at end (in.)
$d_{ca.end} := 12.5in$	Extreme distance of tension deck anchorage vertical reinforcement (in.)

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(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$h_e := H_e + t_o$ $h_e = 19 \cdot in$ Total Height of Equivalent Transverse Load (in.)

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.141
MASH TL-3 Compliance Assessment

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$

Spacing of vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 8.688 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 13.112 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment specified in Article A13.3.1 (k-ft/ft)

(3a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 1 \cdot in^2$ — Area of one vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 24 \cdot in$ — Spacing of vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.5 \cdot in^2$ — Total Area of vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 12.5 \cdot in$ — Average extreme distance of parapet vertical reinforcement in tension (in.)

$f_{y.bolt} := 36ksi$ — Yield stress of bolt (ASTM A307 is assumed)

$f_{ya} := \min\left(f_y, f_{y.bolt}\right) = 36 \cdot ksi$

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_{ya}}{0.85 \cdot f'_c \cdot b_c} = 0.441 \cdot in$ — Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_{ya} \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 18.419 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 13.112 \cdot kip \cdot \frac{ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

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SUBJECT: MnDOT J-Barrier
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MASH Compliance Assessment

(3b) **Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends:** M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 12 \cdot in$ — Spacing of vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.31 \cdot in^2$ — Total Area of vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 8.688 \cdot in$ — Average extreme distance of tension vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 13.112 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 1 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in tension zone at midspan (in^2)

$s_{va.end} = 24 \cdot in$ — Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.5 \cdot in^2$ — Total area of deck anchorage vertical reinforcement per unit length of the wall at midspan(in^2)

$d_{ca.end} = 12.5 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement (in.)

(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$$a_{ca.end} := \frac{(A_{va.end} \cdot f_{ya})}{0.85 \cdot f'_c \cdot b_c} = 0.441 \cdot in$$

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_{ya} \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 18.419 \cdot kip \cdot \frac{ft}{ft}$$

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 13.112 \cdot kip \cdot \frac{ft}{ft}$$

(3c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 8.125 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 32 \cdot in$ — Total height of the barrier (in.)

$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.441 \cdot in$ — Depth of the Whitney Stress Block (in.)

$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 31.618 \cdot kip \cdot ft$ — Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.141
MASH Compliance Assessment

(3d) **Determine the Ultimate Resistance of the Wall at Midspan:** R_{wmid}

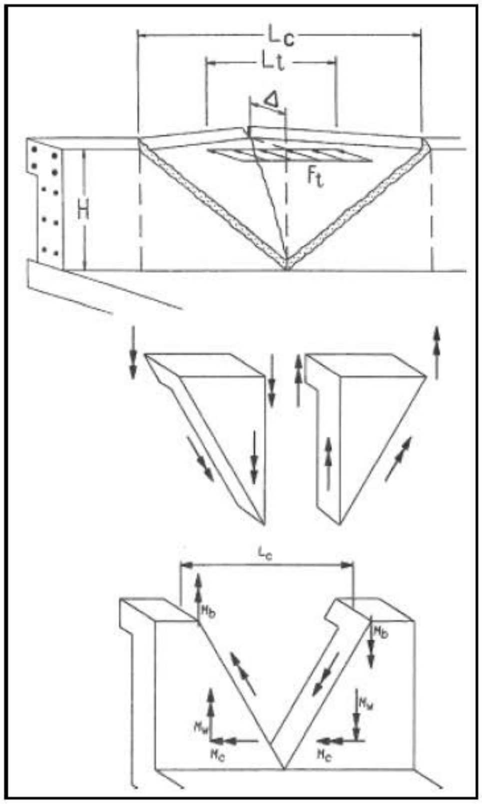

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 13.112 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 31.618 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.446 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 92.893 \cdot kip$$ (Equation A13.3.1-1)

(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

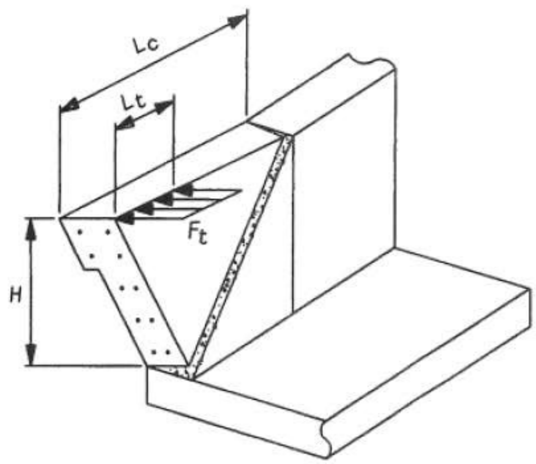

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)
Note: H_w = H in Figure 3e

$M_B = 0$ — No additional beam strength

$M_w = 31.618 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 13.112 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 5.23 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\ 2}\right)}{h_w}\right] = 51.429 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$H_w = 32 \cdot in$ — Height of the Concrete Barrier measured from the top of the roadway surface (in.)

$R_{wmid} = 92.893 \cdot kip$ — Ultimate Resistance of the wall at midspan (kip)

$R_{wend} = 51.429 \cdot kip$ — Ultimate Resistance of the wall at the end of the wall or at a joint (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$R_{R.mid} := R_{wmid} \cdot \left(\frac{h_w}{h_e}\right) = 156.451 \cdot kip$ — Structural Capacity of the Barrier at midspan located at H_e (kip)

$R_{R.end} := R_{wend} \cdot \left(\frac{h_w}{h_e}\right) = 86.617 \cdot kip$ — Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.mid} \geq F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} = \text{"OK"}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.end} \geq F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} = \text{"OK"}$$

(4) Strength Analysis of the Integral End Post:

(4a) Bending Capacity of the End Post and Conjoining Barrier Segment about the Long Axis: $M_{c.ipost}$

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

Bending Capacity of End Post and Conjoining Barrier Segment Considering only the Parapet Vertical Reinf.:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{v1.ipost} := 0.31 in^2$ — Average area of one vertical reinforcement leg in the tension zone at the end post and conjoining barrier segment (in^2)

$s_{v.ipost} := 8.6875 in$ — Average spacing of vertical reinforcement at the end post and conjoining barrier segment (in.) (7 bars over 4 feet)

$$A_{v.ipost} := \left(\frac{b_c}{s_{v.ipost}}\right) \cdot A_{v1.ipost} = 0.428 \cdot in^2$$

Total Area of vertical reinforment per unit length of the wall at the end post and conjoining barrier segment (in^2)

$$a_{c.ipost} := \frac{A_{v.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.63 \cdot in$$

Depth of Whitney Stress Block (in.)

$d_{c.ipost} := 8.6875 in$ — Average extreme distance of tension parapet vertical reinforcement at the end post and conjoining barrier segment (in.)

$$M_{c.ipost} := \frac{\left[A_{v.ipost} \cdot f_y \cdot \left(d_{c.ipost} - \frac{a_{c.ipost}}{2}\right)\right]}{b_c} = 17.926 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis when considering the critical reinforcment specified in Article A13.3.1 (k-ft/ft)

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.141
MASH Compliance Assessment

(4b) Bending Capacity of the End Post and Conjoining Barrier Segment about the Vert. Axis: $M_{w.ipost}$

$A_{w1.ipost} := 0.2in^2$ — Area of one longitudinal reinforcement bar in tension (in^2)

$n_{w.ipost} := 4$ — Number of longitudinal reinforcement bars acting in tension

$A_{w.ipost} := A_{w1.ipost} \cdot n_{w.ipost} = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 32 \cdot in$ — Total height of the barrier (in.)

$$a_{w.ipost} := \frac{A_{w.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.441 \cdot in$$

Depth of the Whitney Stress Block (in.)

$d_{w.ipost} := 8.125in$ — Average extreme distance of tension longitudinal reinforcement (in.)

$$M_{w.ipost} := A_{w.ipost} \cdot f_y \cdot \left(d_{w.ipost} - \frac{a_{w.ipost}}{2}\right) = 31.618 \cdot kip \cdot ft$$

Flexural Resistance of the end post and conjoining barrier segment about the Vertical Axis specified in Article A13.3.1 (k-ft)

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.141
MASH Compliance Assessment

(4c) Determine the Ultimate Resistance of the End Post and Conjoining Barrier Segment: $R_{w.ipost}$

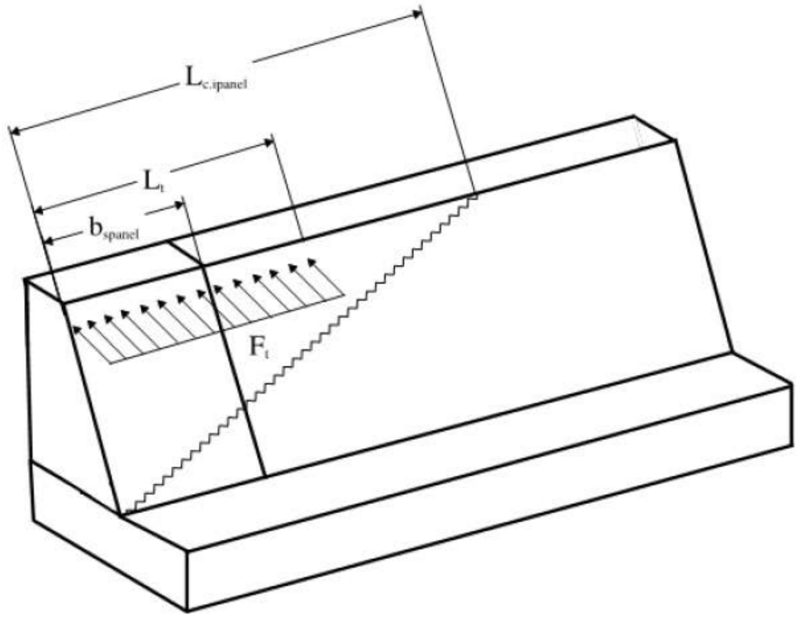


Figure 4c. Yield Line Analysis of the End Post and Conjoining Barrier Segment.

$M_{c.ipost} = 17.926 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis specified in Article A13.4.2 (k-ft/ft)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$h_w = 32 \cdot in$ — Total Height of the Barrier (in.)

$$L_{c.ipost} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_{w.ipost}}{M_{c.ipost}}\right)} = 4.95 \cdot ft$$ (Equation A13.3.1-4)

$$R_{w.ipost} := \left(\frac{2}{2 \cdot L_{c.ipost} - L_t}\right) \cdot \left[M_{w.ipost} + \frac{\left(M_{c.ipost} \cdot L_{c.ipost}^{\ 2}\right)}{h_w}\right] = 66.552 \cdot kip$$ (Equation A13.3.1-3)

(4) Structural Capacity of the End Post and Conjoining Segment of Barrier - Summary of Results:

$h_w = 32 \cdot in$

Total Height of the barrier measured from the top of the roadway surface/overlay (in.)

$R_{w.ipost} = 66.552 \cdot kip$

Ultimate Resistance of the end post and conjoining barrier segment (kip)

$h_e = 19 \cdot in$

Total Height of the Transverse Impact Force, F_t(in.)

$$R_{R.ipost} := R_{w.ipost} \cdot \left(\frac{h_w}{h_e}\right) = 112.088 \cdot kip$$

Structural Capacity of the end post and conjoining barrier segment located at h_e (kip)

$F_t = 71 \cdot kip$

Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_Conjoining\_Barrier\_Segment\_Check} := \left(\begin{array}{|l} \text{"OK"} \quad \text{if } R_{R.ipost} \geq F_t \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array} \right)$$

Structural_Capacity_of_End_Post_and_Conjoining_Barrier_Segment_Check = "OK"

Texas A&M Transportation Institute

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.141
MASH Compliance Assessment

(5) Shear Capacity of the Barrier:

$\lambda := 1.0$	Concrete Modification Factor
$T_w := 9in$	Top Width of the parapet (in.)
$h_c := 15in$	Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)
$d_c := 6.0in$	Distance from compression face to the tension reinforcement (in.)
$L_t = 4 \cdot ft$	Length of the distribution of the impact force (ft.)
$f'_c = 4 \cdot ksi$	Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right)\cdot T_w\right] + 2\cdot\left[\left(h_c + \frac{d_c}{2}\right)\cdot T_w\right] = 810\cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in2)

$$V_{c.int} := 2\cdot\lambda\cdot\left[\left(\sqrt{\frac{f'_c}{psi}}\right)\cdot psi\right]\cdot A_{c.int} = 102.458\cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right)\cdot T_w + \left(h_c + \frac{d_c}{2}\right)\cdot T_w = 621\cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in2)

$$V_{c.end} := 2\cdot\lambda\cdot\left[\left(\sqrt{\frac{f'_c}{psi}}\right)\cdot psi\right]\cdot A_{c.end} = 78.551\cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 78.551\cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$$F_t = 71\cdot kip$$

Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } V_c \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

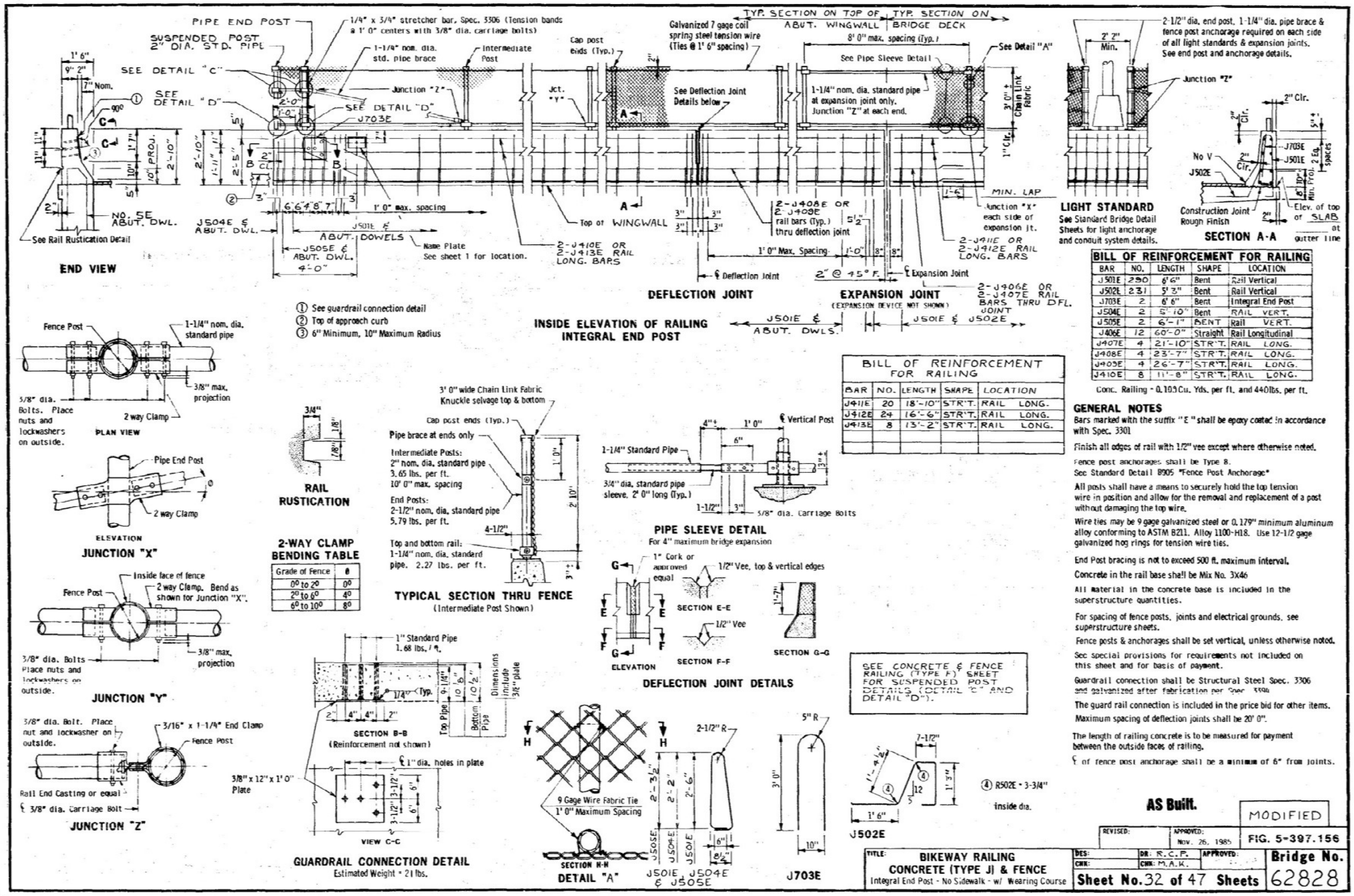
Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

Structural_Capacity_of_End_Post_and_Conjoining_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier from Figure 5-397.141 does satisfy all MASH TL-3 Criteria

APPENDIX B7: J BARRIER ON BRIDGE NO. 62828 (FIGURE 5-397-156 MODIFIED)



BILL OF REINFORCEMENT FOR RAILING				
BAR	NO.	LENGTH	SHAPE	LOCATION
J501E	290	6' 6"	Bent	Rail Vertical
J502E	231	5' 3"	Bent	Rail Vertical
J703E	2	6' 6"	Bent	Integral End Post
J504E	2	5'-10"	Bent	RAIL VERT.
J505E	2	6'-1"	BENT	Rail VERT.
J406E	12	60'-0"	Straight	Rail Longitudinal
J407E	4	21'-10"	STR'T.	RAIL LONG.
J408E	4	23'-7"	STR'T.	RAIL LONG.
J409E	4	26'-7"	STR'T.	RAIL LONG.
J410E	8	11'-8"	STR'T.	RAIL LONG.

Conc. Railing - 0.103 Cu. Yds. per ft. and 440 lbs. per ft.

BILL OF REINFORCEMENT FOR RAILING				
BAR	NO.	LENGTH	SHAPE	LOCATION
J411E	20	18'-10"	STR'T.	RAIL LONG.
J412E	24	16'-6"	STR'T.	RAIL LONG.
J413E	8	13'-2"	STR'T.	RAIL LONG.

GENERAL NOTES

Bars marked with the suffix "E" shall be epoxy coated in accordance with Spec. 3301

Finish all edges of rail with 1/2" vee except where otherwise noted.

Fence post anchorages shall be Type 8.
See Standard Detail B905 "Fence Post Anchorage"

All posts shall have a means to securely hold the top tension wire in position and allow for the removal and replacement of a post without damaging the top wire.

Wire ties may be 9 gage galvanized steel or 0.179" minimum aluminum alloy conforming to ASTM B211. Alloy 1100-H18. Use 12-1/2 gage galvanized hog rings for tension wire ties.

End Post bracing is not to exceed 500 ft. maximum interval.

Concrete in the rail base shall be Mix No. 3X46

All material in the concrete base is included in the superstructure quantities.

For spacing of fence posts, joints and electrical grounds, see superstructure sheets.

Fence posts & anchorages shall be set vertical, unless otherwise noted.

See special provisions for requirements not included on this sheet and for basis of payment.

Guardrail connection shall be Structural Steel Spec. 3306 and galvanized after fabrication per Spec. 3394

The guard rail connection is included in the price bid for other items.

Maximum spacing of deflection joints shall be 20' 0".

The length of railing concrete is to be measured for payment between the outside faces of railing.

℄ of fence post anchorage shall be a minimum of 6" from joints.

AS Built.

MODIFIED

APPROVED: Nov. 26, 1985

FIG. 5-397.156

TITLE: BIKEWAY RAILING
CONCRETE (TYPE J) & FENCE
Integral End Post - No Sidewalk - w/ Wearing Course

DR: R.C.F.
CHK: M.A.K.

Sheet No. 32 of 47 Sheets

Bridge No. 62828

SUBJECT: **MnDOT J-Barrier on Figure 5-397.156 Modified MASH TL-3 Compliance Assessment**

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 32in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 2in$	Thickness of overlay (in.)
$h_w := H_w + t_o$	Total height of the barrier (in.)

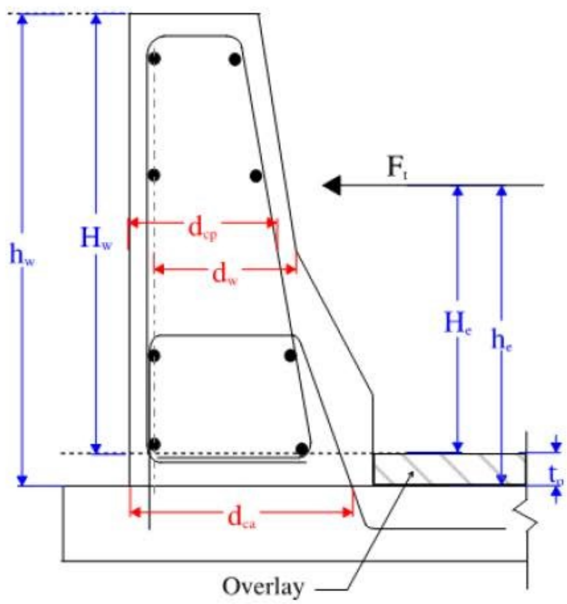


Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b) Concrete Parapet Inputs:

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{vp.mid} := 12in$	Spacing of parapet vertical reinforcement at midspan (in.)
$d_{cp.mid} := 9in$	Extreme distance of parapet vertical reinforcement in tension at midspan (in.)
$A_{vp1.end} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{vp.end} := 9.6in$	Spacing of parapet vertical reinforcement at joints/ends (in.)
$d_{cp.end} := 9in$	Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$	Area of longitudinal reinforcement bars in tension (in^2)
$d_w := 8in$	Extreme distance of tension longitudinal reinforcement of wall (in.)

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(1b-conti.) **Concrete Parapet Inputs:**

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_J502E} := 10in$ — Projected length of J502E reinforcement over the slab (in.)

$L_{wid\_J502E} := 7.5in$ — Outer width of J502E reinforcement (in.)

$Cover := 2in$ — Cover clear distance (in.)

$Ratio_{J502E} := \frac{5}{12}$ — Inclined angle of R502E reinforcement

$d_{b\_J502E} := 0.625in$ — Nominal diameter of J502E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_J502E} + L_{proj\_J502E} \cdot Ratio_{J502E} + Cover - \frac{1}{2} d_{b\_J502E} = 13.354 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ — Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 9.6in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := d_{ca} = 13.354 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

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(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71\text{kip}$ Transverse Impact Force

$L_t := 4\text{ft}$ Longitudinal Length of Distribution of Impact Force

$H_e := 19\text{in}$ Height of Equivalent Transverse Load

$H_{min} := 29\text{in}$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot \text{in}$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

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SUBJECT: **MnDOT J-Barrier on Figure 5-397.156 Modified MASH TL-3 Compliance Assessment**

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$ — Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$ — Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 9 \cdot in$ — Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 13.597 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

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(3a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 13.354 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 20.346 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 13.597 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 9.6 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.388 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.57 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 9 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 16.885 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 9.6 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.388 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.57 \cdot in$ — Depth of Whitney Stress Block (in.)

(3b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 13.354 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 25.322 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 16.885 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(3c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 8 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 34 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.415 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 31.17 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

(3d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

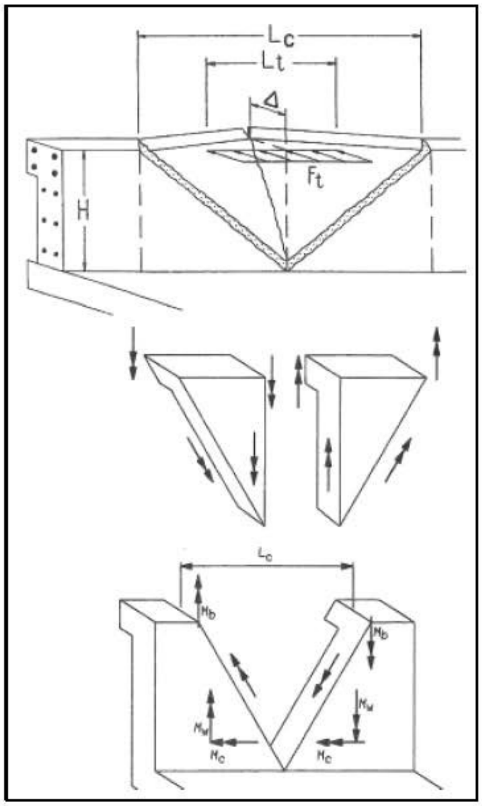

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_W = H in Figure 3d

$M_B := 0kip \cdot ft$ — No additional beam strength

$M_{cmid} = 13.597 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 31.17 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot H_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.274 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{H_w}\right]\right] = 94.568 \cdot kip$$ (Equation A13.3.1-1)

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(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

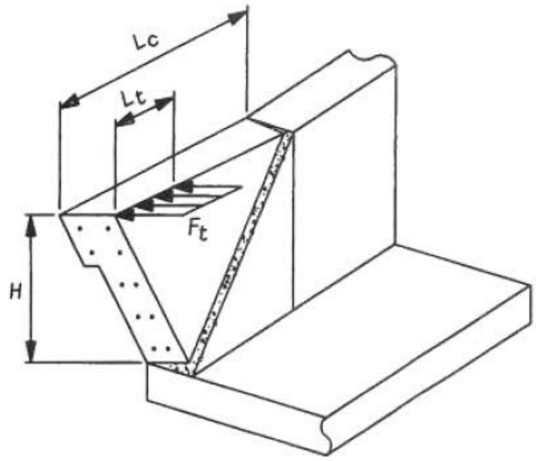

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 32 \cdot in$	Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3e
$M_B = 0$	No additional beam strength
$M_w = 31.17 \cdot kip \cdot ft$	Flex. Resistance of the Wall about the Vert. Axis (k-ft)
$L_t = 4 \cdot ft$	Longitudinal length of distribution of impact force (ft.)
$M_{cend} = 16.885 \cdot \frac{kip \cdot ft}{ft}$	Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 4.987 \cdot ft$$ (Equation A13.3.1-4)

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^2\right)}{H_w}\right] = 63.157 \cdot kip$$ (Equation A13.3.1-3)

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$H_w = 32 \cdot in$ — Height of the Concrete Barrier measured from the top of the roadway surface (in.)

$R_{wmid} = 94.568 \cdot kip$ — Ultimate Resistance of the wall at midspan (kip)

$R_{wend} = 63.157 \cdot kip$ — Ultimate Resistance of the wall at the end of the wall or at a joint (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$R_{R.mid} := R_{wmid} \cdot \left(\frac{H_w}{H_e}\right) = 159.272 \cdot kip$ — Structural Capacity of the Barrier at midspan located at H_e (kip)

$R_{R.end} := R_{wend} \cdot \left(\frac{H_w}{H_e}\right) = 106.369 \cdot kip$ — Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{array}{|l} \text{"OK"} \quad \text{if } R_{R.mid} \ge F_t \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} = \text{"OK"}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} := \begin{array}{|l} \text{"OK"} \quad \text{if } R_{R.end} \ge F_t \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} = \text{"OK"}$$

(4) Strength Analysis of the Integral End Post:

(4a) Bending Capacity of the End Post and Conjoining Barrier Segment about the Long Axis: $M_{c.ipost}$

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

Bending Capacity of End Post and Conjoining Barrier Segment Considering only the Parapet Vertical Reinf.:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{vp1.ipost} := 0.31 in^2$ — Average area of one parapet vertical reinforcement leg in the tension zone at the end post and conjoining barrier segment (in^2)

$s_{vp.ipost} := 5in$ — Average spacing of parapet vertical reinforcement at the end post and conjoining barrier segment (in.)

$$A_{vp.ipost} := \left(\frac{b_c}{s_{vp.ipost}}\right) \cdot A_{vp1.ipost} = 0.744 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at the end post and conjoining barrier segment (in^2)

$$a_{cp.ipost} := \frac{A_{vp.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.094 \cdot in$$

Depth of Whitney Stress Block (in.)

$d_{cp.ipost} := 9in$ — Average extreme distance of tension parapet vertical reinforcement at the end post and conjoining barrier segment (in.)

$$M_{cp.ipost} := \frac{\left[A_{vp.ipost} \cdot f_y \cdot \left(d_{cp.ipost} - \frac{a_{cp.ipost}}{2}\right)\right]}{b_c} = 31.445 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(4a-conti.) <u>Bending Capacity of the End Post and Conjoining Barrier Segment about the Long. Axis</u>: $M_{c.ipost}$

<u>Bending Capacity of End Post and Conjoining Barrier Segment Considering only the Deck Anchorage Vertical Reinf.:</u>
<u>Note:</u> See *Figure 1* for a visual representation of the reinforcement bars.

$A_{va1.ipost} := 0.31in^2$ — Average area of one deck anchorage vertical reinforcement leg in the tension zone at the end post and conjoining barrier segment (in^2)

$s_{va.ipost} := 5in$ — Average spacing of deck anchorage vertical reinforcement at the end post and conjoining barrier segment (in.)

$A_{va.ipost} := \left(\frac{b_c}{s_{va.ipost}}\right) \cdot A_{va1.ipost} = 0.744 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at the end post and conjoining barrier segment (in^2)

$a_{ca.ipost} := \frac{A_{va.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.094 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{ca.ipost} := 14in$ — Average extreme distance of tension anchorage vertical reinforcement at the end post and conjoining barrier segment (in.)

$$M_{ca.ipost} := \frac{\left[A_{va.ipost} \cdot f_y \cdot \left(d_{ca.ipost} - \frac{a_{ca.ipost}}{2}\right)\right]}{b_c} = 50.045 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{c.ipost} := \min\left(M_{cp.ipost}, M_{ca.ipost}\right) = 31.445 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

(4b) Bending Capacity of the End Post and Conjoining Barrier Segment about the Vert. Axis: $M_{w.ipost}$

$A_{w1.ipost} := 0.2in^2$ — Area of one longitudinal reinforcement bar in tension (in^2)

$n_{w.ipost} := 4$ — Number of longitudinal reinforcement bars acting in tension

$A_{w.ipost} := A_{w1.ipost} \cdot n_{w.ipost} = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 34 \cdot in$ — Total height of the barrier (in.)

$a_{w.ipost} := \dfrac{A_{w.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.415 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{w.ipost} := 9in$ — Average extreme distance of tension longitudinal reinforcement (in.)

$M_{w.ipost} := A_{w.ipost} \cdot f_y \cdot \left(d_{w.ipost} - \dfrac{a_{w.ipost}}{2}\right) = 35.17 \cdot kip \cdot ft$ — Flexural Resistance of the end post and conjoining barrier segment about the Vertical Axis specified in Article A13.3.1 (k-ft)

(4c) Determine the Ultimate Resistance of the End Post and Conjoining Barrier Segment: $R_{w.ipost}$

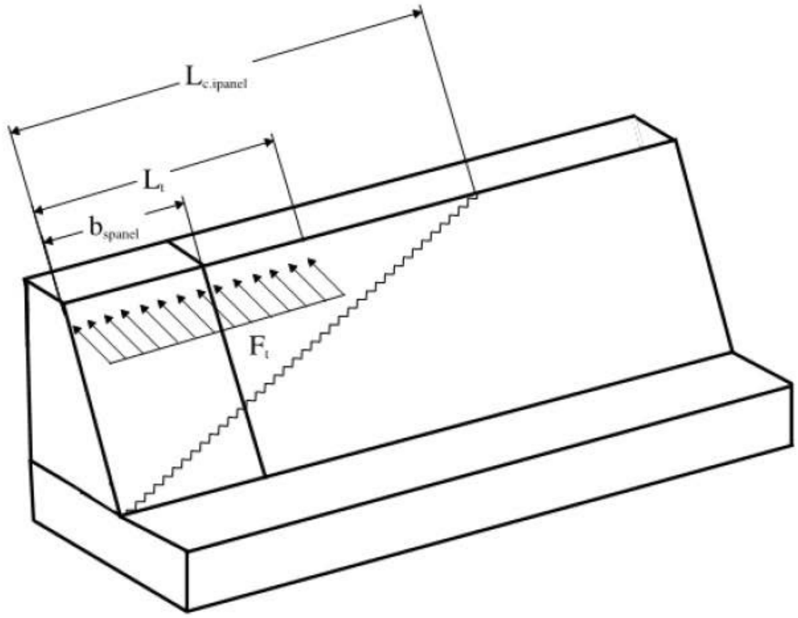

Figure 4c. Yield Line Analysis of the End Post and Conjoining Barrier Segment.

$M_{c.ipost} = 31.445 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis specified in Article A13.4.2 (k-ft/ft)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$H_w = 32 \cdot in$ — Total Height of the Barrier (in.)

$$L_{c.ipost} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H_w \cdot \left(\frac{M_{w.ipost}}{M_{c.ipost}}\right)} = 4.642 \cdot ft$$ (Equation A13.3.1-4)

$$R_{w.ipost} := \left(\frac{2}{2 \cdot L_{c.ipost} - L_t}\right) \cdot \left[M_{w.ipost} + \frac{\left(M_{c.ipost} \cdot L_{c.ipost}^{\ 2}\right)}{H_w}\right] = 109.486 \cdot kip$$ (Equation A13.3.1-3)

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(4) Structural Capacity of the End Post and Conjoining Segment of Barrier - Summary of Results:

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the roadway surface/overlay (in.)

$R_{w.ipost} = 109.486 \cdot kip$ — Ultimate Resistance of the end post and conjoining barrier segment (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$R_{R.ipost} := R_{w.ipost} \cdot \left(\frac{H_w}{H_e}\right) = 184.398 \cdot kip$ — Structural Capacity of the end post and conjoining barrier segment located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_Conjoining\_Barrier\_Segment\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.ipost} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_Conjoining\_Barrier\_Segment\_Check} = \text{"OK"}$$

(5) Shear Capacity of the Barrier:

$\lambda := 1.0$	Concrete Modification Factor
$T_w := 9in$	Top Width of the parapet (in.)
$h_c := 15in$	Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)
$d_c := 8.25in$	Distance from compression face to the tension reinforcement (in.)
$L_t = 4 \cdot ft$	Length of the distribution of the impact force (ft.)
$f'_c = 4 \cdot ksi$	Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right) \cdot T_w\right] + 2 \cdot \left[\left(h_c + \frac{d_c}{2}\right) \cdot T_w\right] = 850.5 \cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in^2)

$$V_{c.int} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.int} = 107.581 \cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right) \cdot T_w + \left(h_c + \frac{d_c}{2}\right) \cdot T_w = 641.25 \cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in^2)

$$V_{c.end} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.end} = 81.112 \cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 81.112 \cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$$F_t = 71 \cdot kip$$

Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } V_c \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

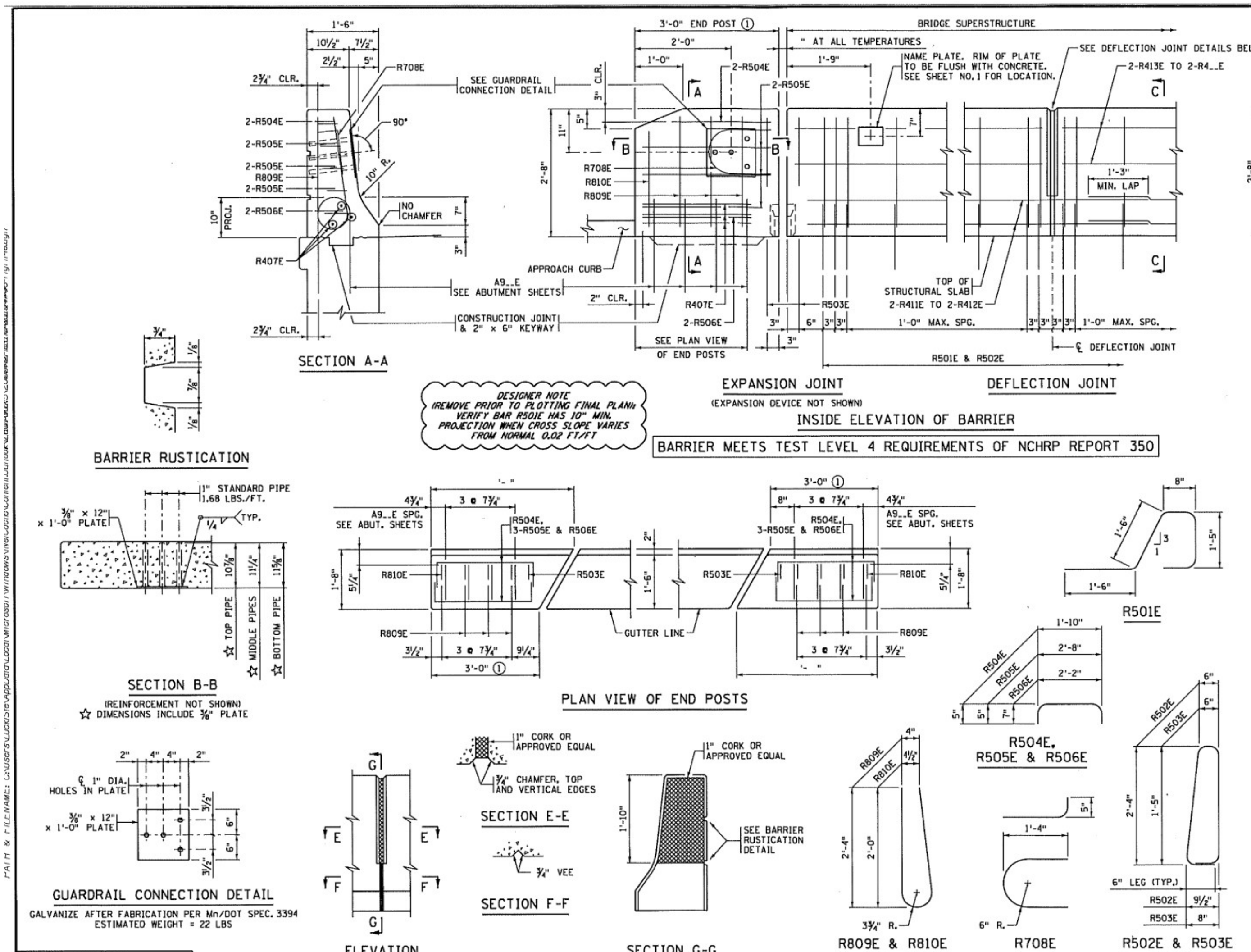
Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

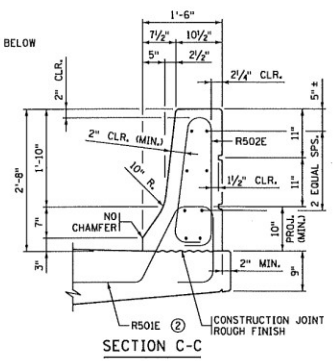
Structural_Capacity_of_End_Post_and_Conjoining_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier on Bridge No. 62828 (Type J) does satisfy all MASH TL-3 Criteria

APPENDIX B8: F BARRIER ON FIGURE 5-397.114





BILL OF REINFORCEMENT FOR BARRIER

BAR	NO.	LENGTH	SHAPE	LOCATION
R501E		5'-7"		BARRIER DOWEL
R502E		6'-3"		BARRIER VERTICAL
R503E		4'-5"		BARRIER VERTICAL
R504E		2'-8"		END POST
R505E		3'-6"		END POST
R506E		3'-4"		END POST
R407E		2'-3"		END POST
R708E		4'-1"		END POST
R809E		5'-0"		END POST
R810E		4'-5"		END POST
R411E				BARRIER LONGIT.
R412E				BARRIER LONGIT.
R413E				BARRIER LONGIT.
R4__E				BARRIER LONGIT.
R4__E				BARRIER LONGIT.

GENERAL NOTES

LENGTH OF "TYPE F (TL-4) RAILING CONCRETE (3Y46 OR 3Y46A)" FOR PAYMENT SHALL BE MEASURED BETWEEN THE OUTSIDE FACES OF THE CONCRETE BARRIER.

CONCRETE BARRIER = 439 LBS./FT. (0.108 CU. YDS./FT.)

FINISH ALL EDGES OF BARRIER AND END POST WITH 1/2" VEE, EXCEPT WHERE OTHERWISE NOTED.

MAXIMUM SPACING OF CONCRETE DEFLECTION JOINTS SHALL BE 20 FT.

SEE SUPERSTRUCTURE SHEET FOR JOINT SPACING.

GUARDRAIL CONNECTION TO BE STRUCTURAL STEEL, Mn/DOT SPEC. 3306.

GUARDRAIL CONNECTION, CORK, AND NAME PLATE TO BE CONSIDERED INCIDENTAL TO "TYPE F (TL-4) RAILING CONCRETE (3Y46 OR 3Y46A)"

BARRIER QUANTITIES ARE LISTED IN SUMMARY OF QUANTITIES FOR SUPERSTRUCTURE.

① END POST DIMENSIONS ARE ALONG GUTTER LINE OR OUTSIDE FACE OF END POST.

② PLACE BAR ON TOP OF BOTTOM REINFORCEMENT MAT.

FIG. 5-397.114

REVISED: 04-17-2013

APPROVED: DECEMBER 18, 2003

STATE BRIDGE ENGINEER

CERTIFIED BY ______ LICENSED PROFESSIONAL ENGINEER ______ DATE

NAME: ______ LIC. NO.

CONCRETE BARRIER (TYPE F, TL-4)
WITH SEPARATE END POST
(WITHOUT CONCRETE WEARING COURSE)

DES: | DR: | CHK: | CHK: | APPROVED:

SHEET NO. OF SHEETS

BRIDGE NO.

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 32in$	Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)
$t_o := 0in$	Thickness of overlay (in)
$h_w := H_w + t_o$	Total height of the barrier (in.)

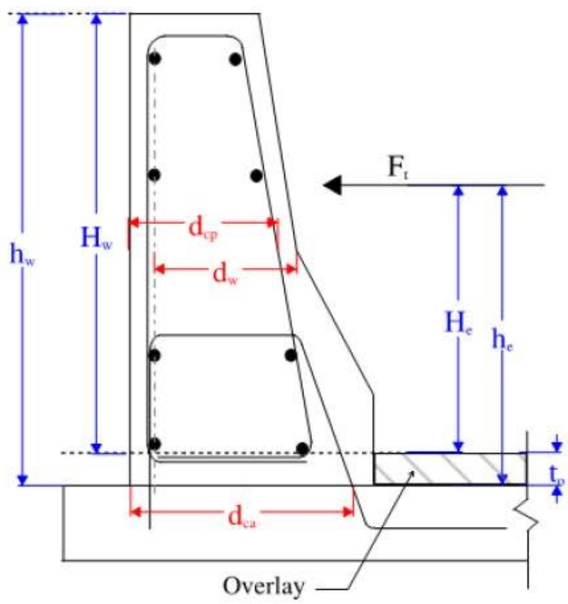


Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

SUBJECT: MnDOT F-Barrier
Figure 5-397.114
MASH Compliance Assessment

Texas A&M Transportation Institute

(1b) Concrete Parapet Inputs:

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$ Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{vp.mid} := 12in$ Spacing of parapet vertical reinforcement at midspan (in.)

$d_{cp.mid} := 10.688in$ Extreme distance of parapet vertical reinforcement in tension at midspan (in.)

$A_{vp1.end} := 0.31in^2$ Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 9.6in$ Spacing of parapet vertical reinforcement at joints/ends (in.) (5 bars over 48 inches)

$d_{cp.end} := 10.688in$ Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$ Area of longitudinal reinforcement bars in tension (in^2)

$d_w := 9.875in$ Extreme distance of tension longitudinal reinforcement of wall (in.)

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SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH Compliance Assessment

(1b-conti.) Concrete Parapet Inputs:

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R501E} := 10in$ — Projected length of R502E reinforcement over the slab (in.)

$L_{wid\_R501E} := 8in$ — Width of R502E reinforcement (in.)

$Cover := 2.25in$ — Cover clear distance (in.)

$Ratio_{R501E} := \frac{1}{3}$ — Inclined angle of R502E reinforcement

$d_{b\_R501E} := 0.625in$ — Nominal diameter of R501E reinforcement (#5 bar)

$$d_{ca} := L_{wid\_R501E} + L_{proj\_R501E} \cdot Ratio_{R501E} + Cover - \frac{1}{2} d_{b\_R501E} = 13.271 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ — Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 13.271 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 9.6in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.) (5 bars over 4 feet average)

$d_{ca.end} := d_{ca} = 13.271 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$h_e := H_e + t_o$ Total Height of Equivalent Trans. Load

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

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SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(3a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$ — Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$ — Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 10.688 \cdot in$ — Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 16.213 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(3a-conti.) <u>Bending Capacity of the Wall about the Longitudinal Axis at Midspan:</u> M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 13.271 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$

Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 20.216 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 16.213 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(3b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 9.6 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.388 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.57 \cdot in$$

Depth of Whitney Stress Block (in.)

$d_{cp.end} = 10.688 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 20.156 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 9.6 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.388 \cdot in^2$$

Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.57 \cdot in$$

Depth of Whitney Stress Block (in.)

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(3b-conti.) <u>Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends:</u> M_{cend}

$d_{ca.end} = 13.271 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 25.16 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 20.156 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(3c) <u>Bending Capacity of the Wall about the Vertical Axis:</u> M_w

$d_w = 9.875 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 32 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.441 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 38.618 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(3d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

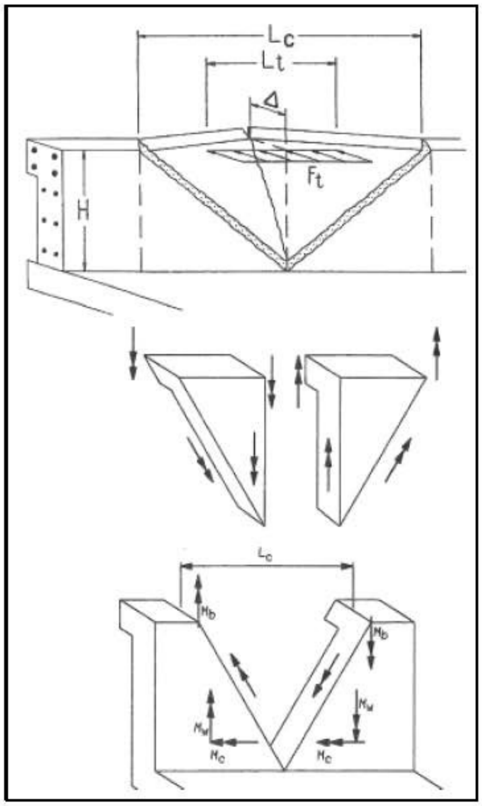

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)
Note: $H_w = H$ in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 16.213 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 38.618 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.404 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 114.346 \cdot kip$$ (Equation A13.3.1-1)

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(3e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

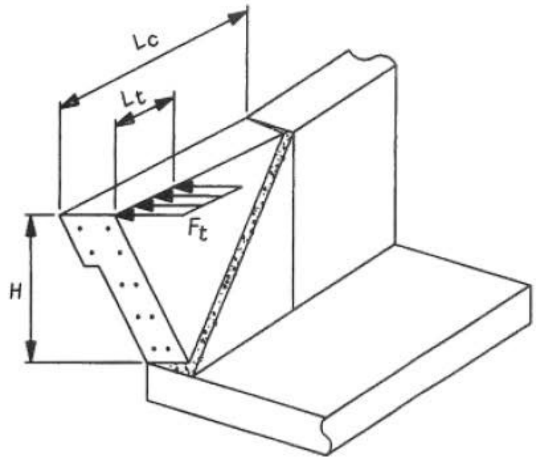

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 32 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3e

$M_B = 0$ — No additional beam strength

$M_w = 38.618 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 20.156 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 5.018 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\,2}\right)}{h_w}\right] = 75.859 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

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SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(3) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$H_w = 32 \cdot in$ — Height of the Concrete Barrier measured from the top of the roadway surface (in.)

$R_{wmid} = 114.346 \cdot kip$ — Ultimate Resistance of the wall at midspan (kip)

$R_{wend} = 75.859 \cdot kip$ — Ultimate Resistance of the wall at the end of the wall or at a joint (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$$R_{R.mid} := R_{wmid} \cdot \left(\frac{h_w}{H_e + t_o}\right) = 192.583 \cdot kip$$

Structural Capacity of the Barrier at midspan located at H_e (kip)

$$R_{R.end} := R_{wend} \cdot \left(\frac{H_w}{H_e + t_o}\right) = 127.763 \cdot kip$$

Structural Capacity of the Barrier at the end of the wall or at a joint located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.mid} \ge F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Ends\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.end} \ge F_t \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

SUBJECT: **MnDOT F-Barrier**
Figure 5-397.114
MASH TL-3 Compliance Assessment

(4) Strength Analysis of the Seperate End Post:

(4a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

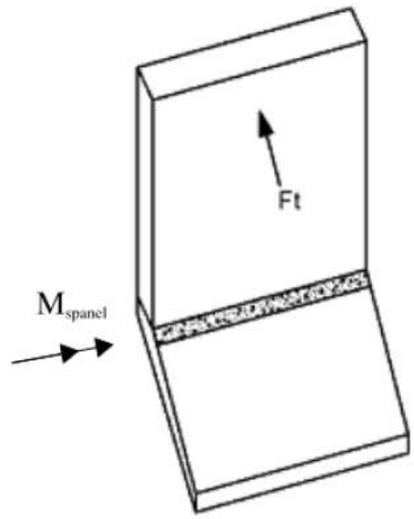

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 32in$ — Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 36in$ — Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 0.79in^2$ — Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
#8 Bars

$n_{p.spost} := 4$ — Number of parapet vertical reinforcement in the end post (in.)

(4a-conti.) <u>Bending Capacity of the End Post about the Longitudinal Axis</u>: $M_{s.post}$

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 3.16 \cdot in^2$ — Total Area of parapet vertical reinforment in the tension zone of the end post (in^2)

$a_{p.spost} := \dfrac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.549 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 9.75in$ — Average extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{p.spost}}{2}\right) = 141.813 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

<u>Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:</u>
<u>Note:</u> See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1in^2$ — Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2)
#9 Bars

$n_{a.spost} := 4$ — Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 4 \cdot in^2$ — Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.961 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 11in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 200.392 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 141.813 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

(4a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t(in.)

$M_{s.post} = 141.813 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \dfrac{M_{s.post}}{H_e + t_o} = 89.566 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \left(\begin{array}{ll} \text{"OK"} & \text{if } R_{s.post} \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"OK"}$$

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Figure 5-397.114
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(4b) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

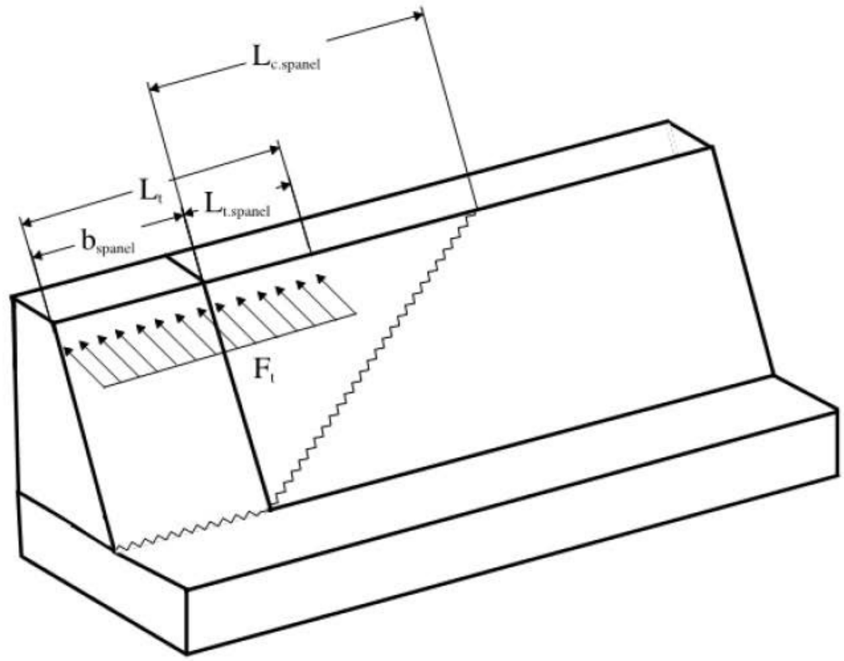

Figure 4b. Flexural Strength and Yield Line Analysis of the End Post and the Contributing Barrier Segment.

Note: $R_{R.spost}$ is equal to the structural capacity of the end post plus the structural capacity of the end of the barrier considering a reduced L_t ($L_{t.spost}$).

Structural Capacity at the End of the Barrier: ($R_{w.spost}$)

Note: $R_{w.spost}$ considers a reduced L_t called $L_{t.spost}$

$b_{spost} = 3 \cdot ft$ — Width of the End Post (ft.)

$L_t = 4 \cdot ft$ — Length of the Distribution of the Impact Force (ft.)

$L_{t.spost} := L_t - b_{spost} = 1 \cdot ft$ — Distribution Length of the Impact Force acting at the End of the Barrier (ft.)

SUBJECT: **MnDOT F-Barrier**
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(4b-conti.) Structural Capacity of the End Post and the End of the Barrier: $R_{R.spost}$

$H_w = 32 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay (in.)

$M_{cend} = 20.156 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$M_B = 0$ — No beam addition to the barrier

$M_w = 38.618 \cdot kip \cdot ft$ — Flexural Resistance of the barrier about the vertical axis (kip-ft)

$$L_{c.spost} := \frac{L_{t.spost}}{2} + \sqrt{\left(\frac{L_{t.spost}}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 2.815 \cdot ft$$

Length of the ultimate resistance at the end of the barrier segment (ft.)
-Modified Equation A13.3.1-4

$$R_{end} := \left(\frac{2}{2 \cdot L_{c.spost} - L_{t.spost}}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{c.spost}^{\ 2}\right)}{h_w}\right] = 42.554 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_W (kip)
-Modified Equation A13.3.1-3

$$R_{w.spost} := R_{end} \cdot \left(\frac{h_w}{H_e + t_o}\right) = 71.67 \cdot kip$$

Structural Capacity at the end of the barrier segment using L_{tspost} located at H_e (kip)

$R_{s.post} = 89.566 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$R_{R.spost} := R_{w.spost} + R_{s.post} = 161.236 \cdot kip$ — Structural Capacity of the end post and contributing segment of barrier (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} := \left(\begin{array}{ll} \text{"OK"} & \text{if } R_{R.spost} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_End\_of\_Barrier\_Segment\_Check} = \text{"OK"}$$

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(5) Shear Capacity of the Barrier:

$\lambda := 1.0$	Concrete Modification Factor
$T_W := 9in$	Top Width of the parapet (in.)
$h_c := 15in$	Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)
$d_c := 8in$	Distance from compression face to the tension reinforcement (in.)
$L_t = 4 \cdot ft$	Length of the distribution of the impact force (ft.)
$f'_c = 4 \cdot ksi$	Concrete parapet compressive strength (ksi)

(5a) Shear Capacity of an Interior Segment of the Barrier: V_{cint}

$$A_{c.int} := \left[\left(L_t + d_c\right) \cdot T_W\right] + 2 \cdot \left[\left(h_c + \frac{d_c}{2}\right) \cdot T_W\right] = 846 \cdot in^2$$

Concrete Parapet Shear Zone Area of an Interior Segment of the Barrier (in^2)

$$V_{c.int} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.int} = 107.011 \cdot kip$$

Shear Capacity of an Interior Segment of the Barrier (kip)

(5b) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right) \cdot T_W + \left(h_c + \frac{d_c}{2}\right) \cdot T_W = 639 \cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in^2)

$$V_{c.end} := 2 \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{psi}}\right) \cdot psi\right] \cdot A_{c.end} = 80.828 \cdot kip$$

Shear Capacity of an End Segment of the Barrier (kip)

$$V_c := \min\left(V_{c.int}, V_{c.end}\right) = 80.828 \cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$$F_t = 71 \cdot kip$$

Transverse Impact Force (kip)

$$\text{Shear\_Capacity\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } V_c \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_Ends_Check = "OK"

Structural_Capacity_of_End_Post_Check = "OK"

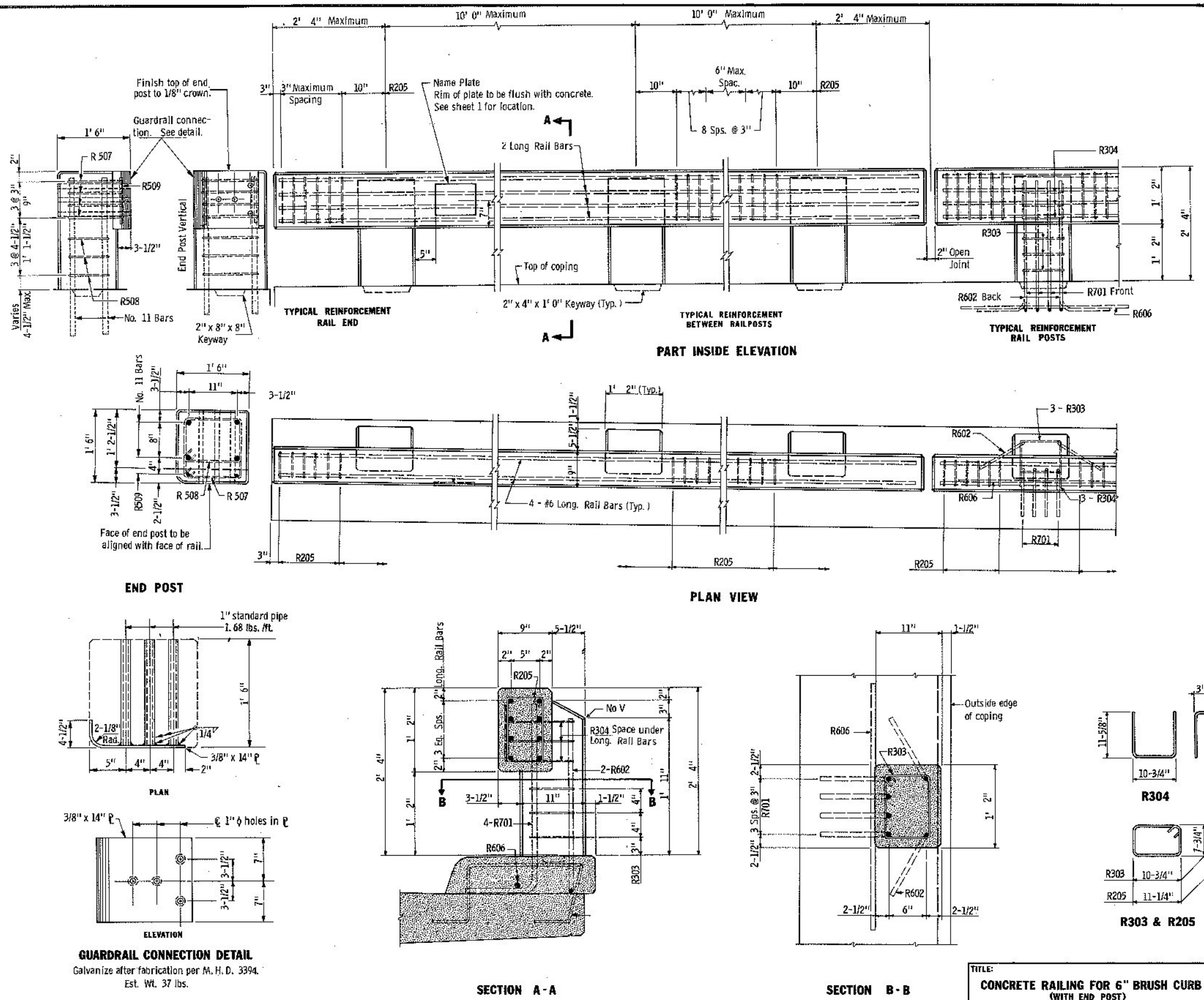
Structural_Capacity_of_End_Post_and_End_of_Barrier_Segment_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The F-Barrier from Figure 5-397.114 does satisfy all MASH TL-3 Criteria

APPENDIX C: ONE-LINE RAILING ANALYSES

APPENDIX C1: ONE-LINE BRIDGE RAIL ON FIGURE 5-397.102



BILL OF REINFORCEMENT FOR RAILING				
MARK	NO	LENGTH	SHAPE	LOCATION
R701		3' 8"	Bent	Railpost
R602		3' 4"	"	"
R303		3' 9"	"	"
R304		3' 4"	"	"
R205		3' 6"	"	Railing
R606		3' 6"	Str.	Curb
R507		6' 0"	Bent	End Post
R508		5' 4"	Bent	End Post
R509		0' 11"	Str.	End Post

See special provisions for railing ties alternate.

GENERAL NOTES:

Railposts and railing to be concrete mix No. 3Y46A.

All railposts normal to grade, except as noted.

Finish all edges of railing and railposts to a 1/2" V except where otherwise noted.

Special care shall be exercised in finishing the surface of the concrete slab in the area of the posts so that the vertical reinforcement for the posts can be properly placed with respect to depth of anchorage.

No. 2 reinforcement bars shall comply with A. S. T. M. A306.

Place the No. 5 end post stirrups when the No. 11 vertical bars are cast in the abutment.

Guardrail connection to be included in weight of structural steel M. H. D. 3306.

Railing quantities are included in summary of quantities for superstructure.

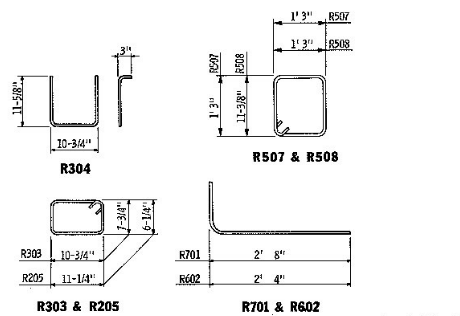


Fig. 5-397.102

August 6, 1970

TITLE: CONCRETE RAILING FOR 6" BRUSH CURB (WITH END POST)

DES: DR: CHK: CHK: APPROVED:

Sheet No. of Sheets

Bridge No.

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(1) General Information and Inputs:

1) Reference: AASHTO MASH TL-3 Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$ Compressive Strength of Concrete (psi)

$f_y := 40ksi$ Yield Strength of Concrete Reinforcing Steel, (ksi)

$E_s := 29000ksi$ Modulus of Elasticity of Steel (ksi)

$H_w := 34.25in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$t_o := 0in$ Thickness of overlay (in.)

$h_w := H_w + t_o = 34.25 \cdot in$ Total height of the barrier (in.)

(1b) Concrete Rail Inputs:

$b_{cr} := 14in$ Width of the Concrete Rail (in.)

$A_{cr} := 1.76in^2$ Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)

$d_{cr} := 7in$ Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)

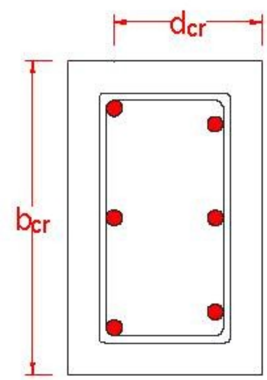

Figure 1b. Profile View showing Concrete Rail Inputs

$y_{cr} := 27.25in$ Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)

(1c) Concrete Post Inputs:

$b_p := 14in$	Width of Concrete Post (in.)
$A_p := 2.4in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p := 8.5in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$h_{curb} := 6.25in$	Height of curb (in.)
$L_p := 10ft$	Spacing of Concrete Posts (ft.)

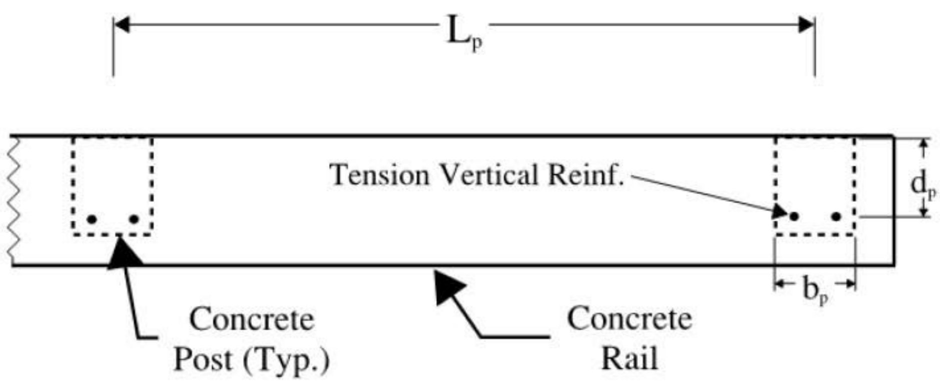

Figure 1c. Plan View of a Concrete Post-and-Beam Railing showing Concrete Post Inputs

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(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 34.25 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ — Minimum height of a MASH TL-3 barrier (in.)

$H_w = 34.25 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

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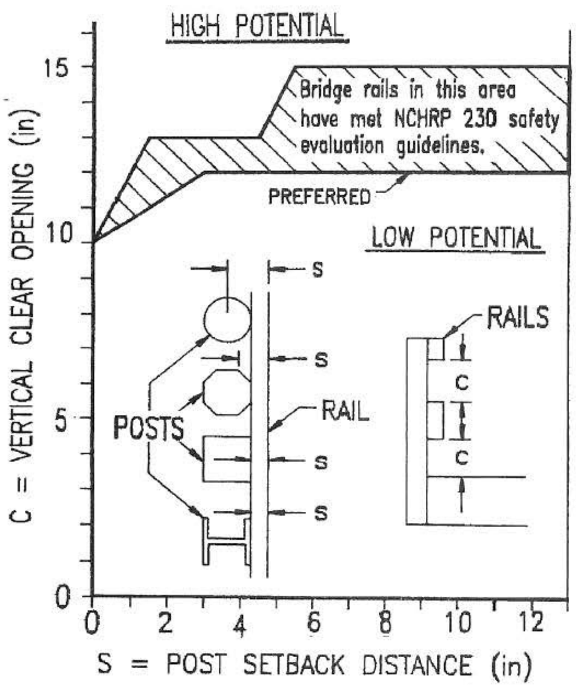
(3) Geometric Criteria:

$S_{post} := 3.5in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 14in$	Vertical Clear Opening (in.)
$\Sigma A := 20in$	Total Rail Contact Width (in.)
$H_w = 34.25 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.



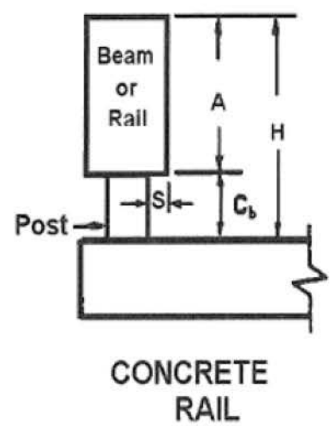


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

(3-conti.) Geometric Criteria:

$S_{post} = 3.5 \cdot in$ $\quad \Sigma A = 20 \cdot in$ $\quad H_w = 34.25 \cdot in$ $\quad ratio_{\Sigma AH} := \frac{\Sigma A}{H_w} = 0.584$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

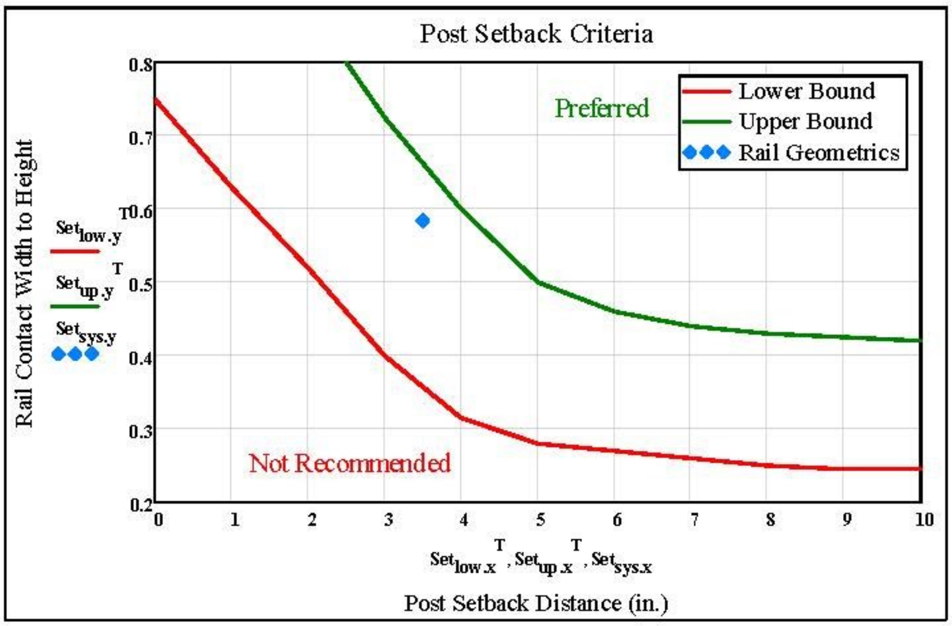
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 3.5$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.584$ Ratio of Contact Width to Total Height rail geometric point



NotRecommended := 1 $\quad$ **Marginal** := 2 $\quad$ **Preferred** := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Post_Setback_Criteria_Rail_Geometric_Point := **Marginal**

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0 \;\; 3 \;\; 13)$

$snag_{low.y} := (10 \;\; 12 \;\; 12)$

Lower Boundary for Snag Potential Criteria x and y coordinates

$snag_{up.x} := (0 \;\; 1.25 \;\; 4.25 \;\; 5.25 \;\; 13)$

$snag_{up.y} := (10 \;\; 13 \;\; 13 \;\; 15 \;\; 15)$

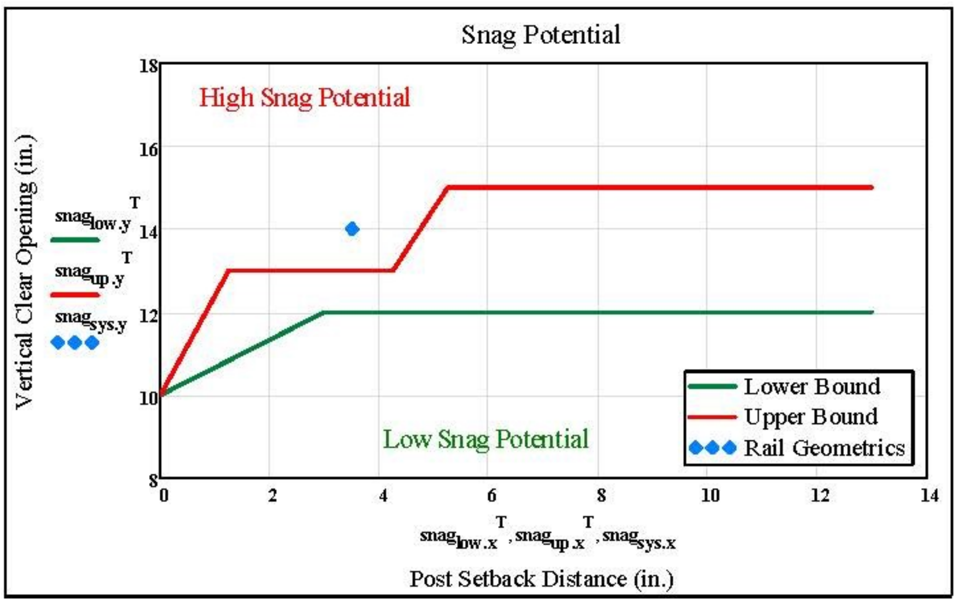
Upper Boundary for Snag Potential Criteria x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 3.5$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 14$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1 Marginal = 2 LowSnagPotential := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := HighSnagPotential

(3) Geometric Criteria - Summary of Results:

Post_Setback_Criteria_Check := | "OK" if Post_Setback_Criteria_Rail_Geometric_Point = Preferred
| "NOT OK" otherwise

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check := | "OK" if Snag_Potential_Criteria_Rail_Geometric_Point = LowSnagPotential
| "NOT OK" otherwise

Snag_Potential_Criteria_Check = "NOT OK"

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Flexural Capacity of the Concrete Rail: M_{cr}

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_{cr} = 14 \cdot in$	Width of the Concrete Rail (in.)
$A_{cr} = 1.76 \cdot in^2$	Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)
$d_{cr} = 7 \cdot in$	Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)
$a_{cr} := \dfrac{A_{cr} \cdot f_y}{0.85 f'_c \cdot b_{cr}} = 1.479 \cdot in$	Whitney Stress Block Depth (in.)
$M_{cr} := A_{cr} \cdot f_y \cdot \left(d_{cr} - \dfrac{a_{cr}}{2}\right) = 36.728 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Rail (kip-ft)
$y_{cr} = 27.25 \cdot in$	Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)
$y_{bar} := y_{cr} = 27.25 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.) Note: $y_{bar} = y_{cr}$, since the only rail is the concrete rail

(4b) Post Strength: P_p

$f_y = 40 \cdot ksi$ — Yield Strength of Concrete Reinforcing Steel (ksi)

$f'_c = 4 \cdot ksi$ — Concrete Compressive Strength

$b_p = 14 \cdot in$ — Width of Concrete Post (in.)

$A_p = 2.4 \cdot in^2$ — Area of Concrete Post reinforcement acting in tension (in^2)

$d_p = 8.5 \cdot in$ — Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)

$$a_p := \frac{A_p \cdot f_y}{0.85 \cdot f'_c \cdot b_p} = 2.017 \cdot in$$

Whitney Stress Block Depth (in.)

$$M_{post} := A_p \cdot f_y \cdot \left(d_p - \frac{a_p}{2}\right) = 59.933 \cdot kip \cdot ft$$

Flexural Capacity of the Concrete Post (kip-ft)

$y_{bar} = 27.25 \cdot in$ — Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$h_{curb} = 6.25 \cdot in$ — Height of curb (in.)

$$y_p := y_{bar} - h_{curb} = 21 \cdot in$$

Height measured from the bottom of the Concrete Post to the Resultant Force of all Rails (in.)

$$P_p := \frac{M_{post}}{y_p} = 34.247 \cdot kip$$

Post Strength (kip)

(4c) **Ultimate Resistance (Nominal Resistance) of the Railing for a Single-Span Failure Mode:** R_1

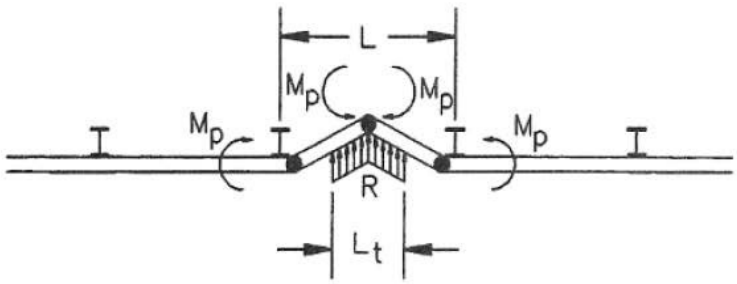


Figure 4c. Single-Span Failure Mode for Post-and-Beam Railings

$P_p = 34.247 \cdot kip$ — Post Strength (kip)

$N_1 := 1$ — Number of Failure Spans

$M_p := M_{cr} = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)
Note: $M_p = M_{cr}$, since the only rail is the concrete rail

$L_p = 10 \cdot ft$ — Post Spacing (ft.)
Note: $L_p = L$ in Figure 4c.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_p}{2 \cdot N_1 \cdot L_p - L_t} = 36.728 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode (kip)
- Eqn. A13.3.2-1

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(4d) Ultimate Resistance (Nominal Resistance) of the Railing for a Two-Span Failure Mode: R_2

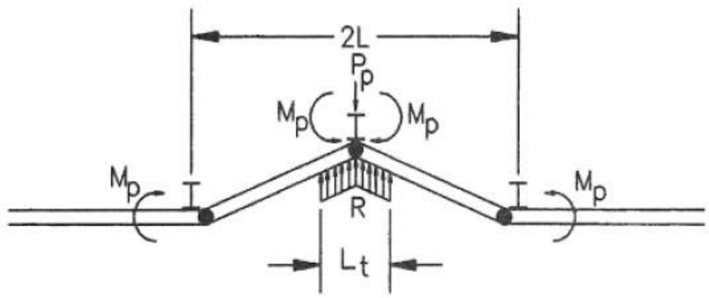

Figure 4d. Two-Span Failure Mode for Post-and-Beam Railings

$P_p = 34.247 \cdot kip$	Post Strength (kip)
$N_2 := 2$	Number of Failure Spans
$M_p = 36.728 \cdot kip \cdot ft$	Flexural Capacity of all Rails (kip-ft)
$L_p = 10 \cdot ft$	Post Spacing (ft.) Note: L_p = L in Figure 4d.
$L_t = 4 \cdot ft$	Longitudinal Length of Distribution of the Impact Force
$R_2 := \dfrac{16 \cdot M_p + N_2^{\ 2} \cdot P_p \cdot L_p}{2 \cdot N_2 \cdot L_p - L_t} = 54.376 \cdot kip$	Ultimate Resistance of the Railing for a Two-Span Failure Mode (kip) - Eqn. A13.3.2-2

(4e) Ultimate Resistance (Nominal Resistance) of the Railing for a Three-Span Failure Mode: R_3

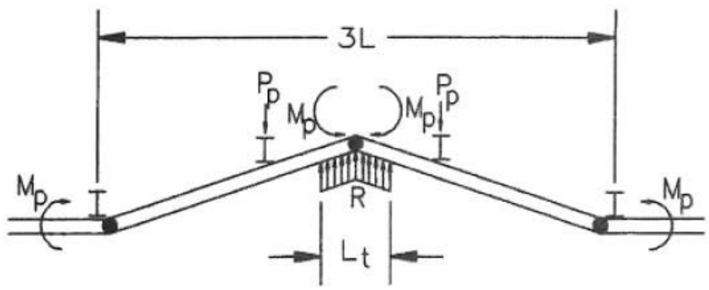

Figure 4e. Three-Span Failure Mode for Post-and-Beam Railings

$P_p = 34.247 \cdot kip$	Post Strength (kip)
$N_3 := 3$	Number of Failure Spans
$M_p = 36.728 \cdot kip \cdot ft$	Flexural Capacity of all Rails (kip-ft)
$L_p = 10 \cdot ft$	Post Spacing (ft.) Note: L_p = L in Figure 4e.
$L_t = 4 \cdot ft$	Longitudinal Length of Distribution of the Impact Force
$R_3 := \dfrac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_p}{2 \cdot N_3 \cdot L_p - L_t} = 59.419 \cdot kip$	Ultimate Resistance of the Railing for a Three-Span Failure Mode (kip) - Eqn. A13.3.2-1

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(4f) Ultimate Resistance (Nominal Resistance) of the Railing for a 4-8 Span Failure Mode: R_4 - R_8

$P_p = 34.247 \cdot kip$ $\quad L_p = 10\,ft$ $\quad M_p = 36.728 \cdot kip \cdot ft$ $\quad L_t = 4\,ft$

$N_4 := 4$ $\quad N_5 := 5$ $\quad N_6 := 6$ $\quad N_7 := 7$ $\quad N_8 := 8$

$$R_4 := \frac{16 \cdot M_p + N_4^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_4 \cdot L_p - L_t} = 79.832 \cdot kip$$

Ultimate Resistance of the Railing for a Four-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_p}{2 \cdot N_5 \cdot L_p - L_t} = 91.74 \cdot kip$$

Ultimate Resistance of the Railing for a Five-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_6 := \frac{16 \cdot M_p + N_6^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_6 \cdot L_p - L_t} = 111.351 \cdot kip$$

Ultimate Resistance of the Railing for a Six-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_7 := \frac{16 \cdot M_p + (N_7 - 1) \cdot (N_7 + 1) \cdot P_p \cdot L_p}{2 \cdot N_7 \cdot L_p - L_t} = 125.194 \cdot kip$$

Ultimate Resistance of the Railing for a Seven-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_8 := \frac{16 \cdot M_p + N_8^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_8 \cdot L_p - L_t} = 144.269 \cdot kip$$

Ultimate Resistance of the Railing for a Eight-Span Failure Mode (kip)
- Eqn. A13.3.2-2

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$R_r := \min(R_1, R_2, R_3, R_4, R_5, R_6, R_7, R_8) = 36.728 \cdot \text{kip}$

Total Ultimate Resistance of the railing @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_1 - R_8)

$H_e = 19 \cdot \text{in}$

Height of Equivalent Transverse Load

$y_{bar} = 27.25 \cdot \text{in}$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$F_t = 71 \cdot \text{kip}$

Transverse Impact Force

$$R_R := R_r \cdot \left(\frac{y_{bar}}{H_e}\right) = 52.676 \cdot \text{kip}$$

Total Ultimate Resistance of the railing @ H_e (kip)

Structural_Capacity_of_Barrier_Check := | "OK" if $R_R \geq F_t$
| "NOT OK" otherwise

Structural_Capacity_of_Barrier_Check = "NOT OK"

(5) Strength Analysis of the Barrier at an End Section or Joint:

(5a) Ultimate Resistance of the Railing at an End Section or Joint for a Single-Span Failure Mode: R_{1end}

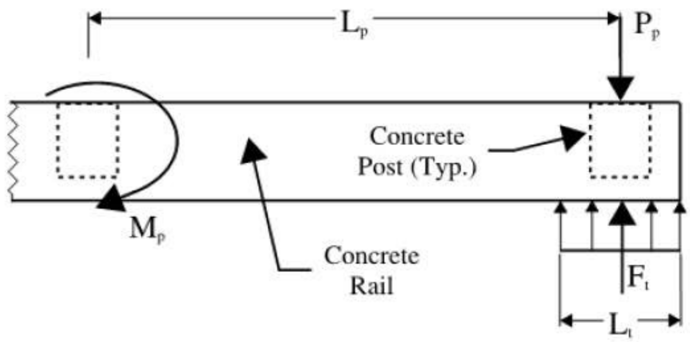

Figure 5a. Single-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 34.247 \cdot kip$ — Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10\,ft$ — Post Spacing (ft)

$R_{1end} := P_p + \dfrac{M_p}{L_p} = 37.92 \cdot kip$ — Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

<u>Using the Approach from AASHTO Section 13:</u> $N_1 = 1$

$$R_{1A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_1} i\right)}{2N_1 \cdot L_p - L_t} = 47.4 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$R_{r1.end} := \min\left(R_{1end}, R_{1A.end}\right) = 37.92 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

(5b) Ultimate Resistance of the Railing at an End Section or Joint for a Two-Span Failure Mode: R_{2end}

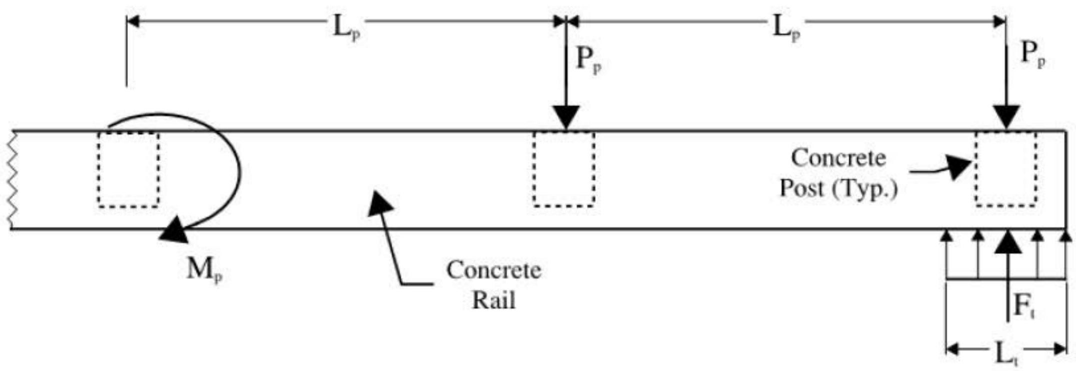

Figure 5b. Two-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 34.247 \cdot kip$ — Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10\,ft$ — Post Spacing (ft)

$$R_{2end} := \frac{P_p \cdot 3L_p + M_p}{2L_p} = 53.207 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

Using AASHTO Section 13: $N_2 = 2$

$$R_{2A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_2} i\right)}{2N_2 \cdot L_p - L_t} = 59.119 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r2.end} := \min\left(R_{2end}, R_{2A.end}\right) = 53.207 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

(5) Strength Analysis of the Barrier at an End Section or Joint-Summary of Results:

$R_{r1.end} = 37.92 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

$R_{r2.end} = 53.207 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

$R_{rend} := \min\left(R_{r1.end}, R_{r2.end}\right) = 37.92 \cdot kip$ — Total Ultimate Resistance of the railing at an End Section or Joint @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_{r1end} & R_{r2end})

$y_{bar} = 27.25 \cdot in$ — Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$H_e = 19 \cdot in$ — Height of Equivalent Transverse Load

$F_t = 71 \cdot kip$ — Transverse Impact Force

$R_{Rend} := R_{rend} \cdot \left(\frac{y_{bar}}{H_e}\right) = 54.385 \cdot kip$ — Total Ultimate Resistance of the railing at an End Section or Joint @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{Rend} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} = \text{"NOT OK"}$$

(6) Strength Analysis of the Seperate End Post:

(6a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

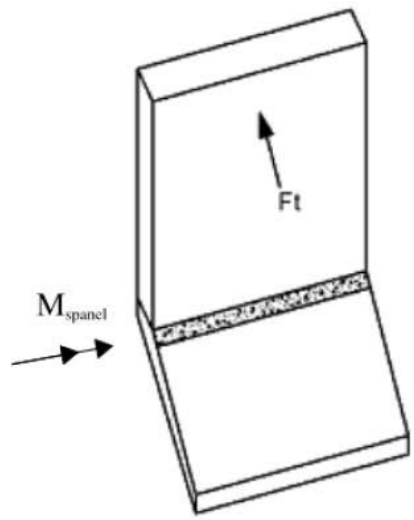

Figure 6a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 35in$ Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 18in$ Width of the end post (in.)

$A_{1.spost} := 1.56in^2$ Area of one vertical reinforcement leg in the tension zone in the end post (in^2)

$n_{spost} := 2$ Number of vertical reinforcement in the end post (in.)

(6a-conti.) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

$A_{spost} := n_{spost} \cdot A_{1.spost} = 3.12 \cdot in^2$ — Total Area of vertical reinforment in the tension zone of the end post (in2)

$a_{spost} := \dfrac{A_{spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 2.039 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{spost} := 11.5in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{s.post} := A_{spost} \cdot f_y \cdot \left(d_{spost} - \dfrac{a_{spost}}{2}\right) = 108.996 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis(kip-ft)

SUBJECT: MnDOT One-Line Barrier
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(6) Strength Analysis of the End Post-Summary of Results:

$H_e = 19 \cdot in$

Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 108.996 \cdot kip \cdot ft$

Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$$R_{s.post} := \frac{M_{s.post}}{H_e} = 68.84 \cdot kip$$

Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$

Transverse Impact Force located at H_e (kip)

$$Structural\_Capacity\_of\_End\_Post\_Check := \left(\begin{array}{ll} \text{"OK"} & \text{if } R_{s.post} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right)$$

$$Structural\_Capacity\_of\_End\_Post\_Check = \text{"NOT OK"}$$

(7) **Conclusions:**

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check = "NOT OK"

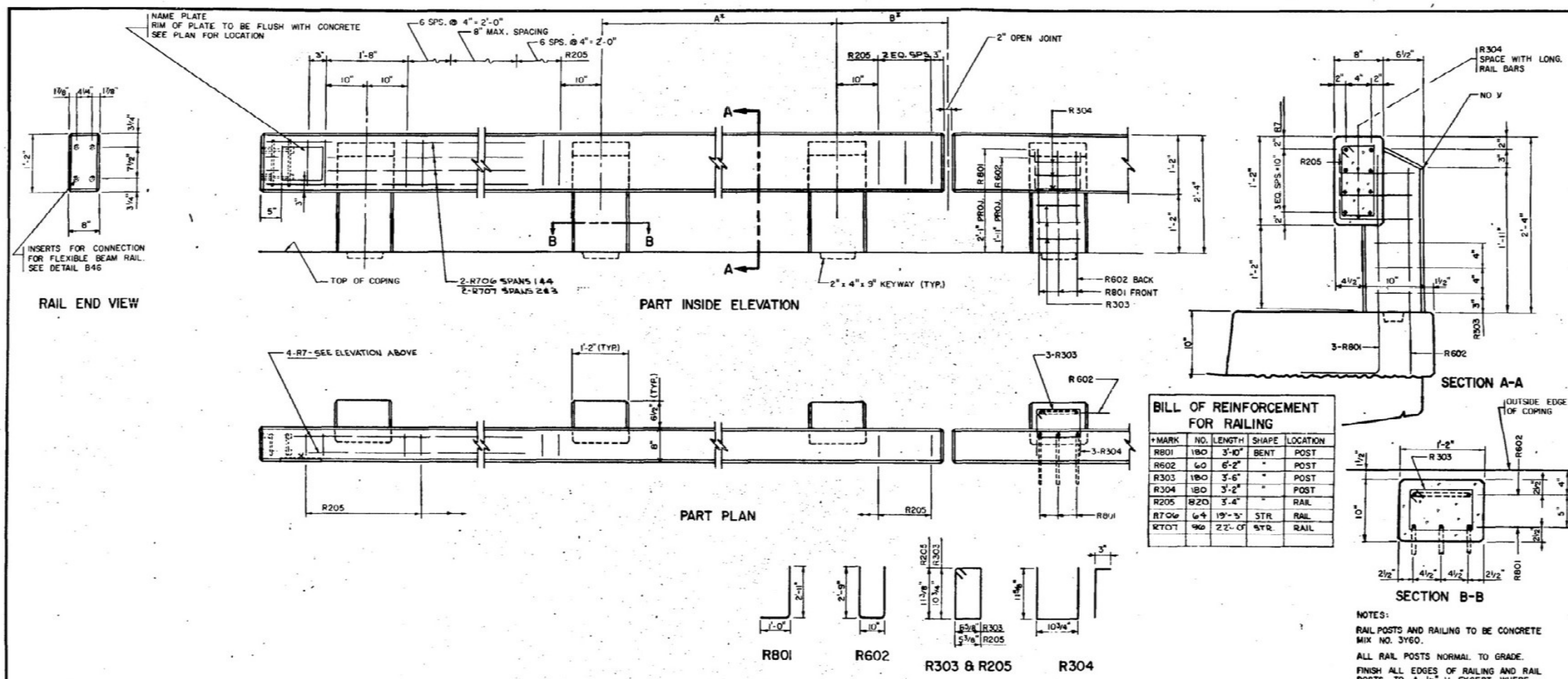
Structural_Capacity_of_Barrier_Check = "NOT OK"

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

Structural_Capacity_of_End_Post_Check = "NOT OK"

The One-Line barrier on Figure 5-397.102 does not satisfy all MASH TL-3 Criteria

APPENDIX C2: ONE-LINE BRIDGE RAIL ON BRIDGE NO. 70802



BILL OF REINFORCEMENT FOR RAILING

+MARK	NO.	LENGTH	SHAPE	LOCATION
R801	180	3'-10"	BENT	POST
R602	60	6'-2"	"	POST
R303	180	3'-6"	"	POST
R304	180	3'-2"	"	POST
R205	820	3'-4"	"	RAIL
R706	64	19'-3"	STR.	RAIL
R707	96	22'-0"	STR.	RAIL

NOTES:

RAIL POSTS AND RAILING TO BE CONCRETE MIX NO. 3Y60.

ALL RAIL POSTS NORMAL TO GRADE.

FINISH ALL EDGES OF RAILING AND RAIL POSTS TO A ½" V EXCEPT WHERE OTHERWISE NOTED.

RAILING QUANTITIES ARE INCLUDED IN SUMMARY OF QUANTITIES FOR SUPERSTRUCTURE.

ALL BAR DIMENSIONS ARE OUT TO OUT.

ALL BAR DETAILS SHALL CONFORM TO ACI 315-57.

*DIMENSION "A" NOT OVER 10'-0".

*DIMENSION "B" NOT OVER 2'-6".

+FIRST NUMBER IN BAR MARK INDICATES SIZE OF BAR.

DRAWN 7-22-63
REVISED 8-6-63
REVISED 8-13-63
REVISED 8-27-63

STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS

BRIDGE NO. 70802

CONCRETE RAILING FOR 10" HIGH CURB

APPROVED: 2-4-64

SHEET 9 OF 16 SHEETS | 70802

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$ — Compressive Strength of Concrete (psi)

$f_y := 40ksi$ — Yield Strength of Concrete Reinforcing Steel, (ksi)

$E_s := 29000ksi$ — Modulus of Elasticity of Steel (ksi)

$H_w := 38.25in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$t_o := 0in$ — Thickness of overlay (in.)

$h_w := H_w + t_o = 38.25 \cdot in$ — Total height of the barrier (in.)

(1b) Concrete Rail Inputs:

$b_{cr} := 14in$ — Width of the Concrete Rail (in.)

$A_{cr} := 2.4in^2$ — Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)

$d_{cr} := 6in$ — Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)

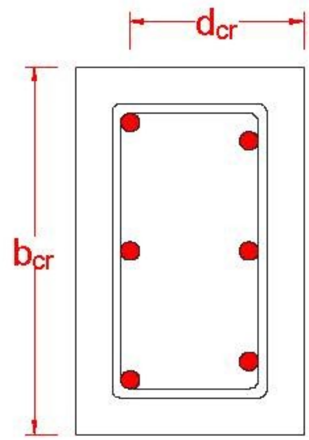


Figure 1b. Profile View showing Concrete Rail Inputs

$y_{cr} := 31in$ — Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)

(1c) Concrete Post Inputs:

$b_p := 14in$	Width of Concrete Post (in.)
$A_p := 2.37in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p := 7.5in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$h_{curb} := 10in$	Height of curb (in.)
$L_p := 10ft$	Spacing of Concrete Posts (ft.)

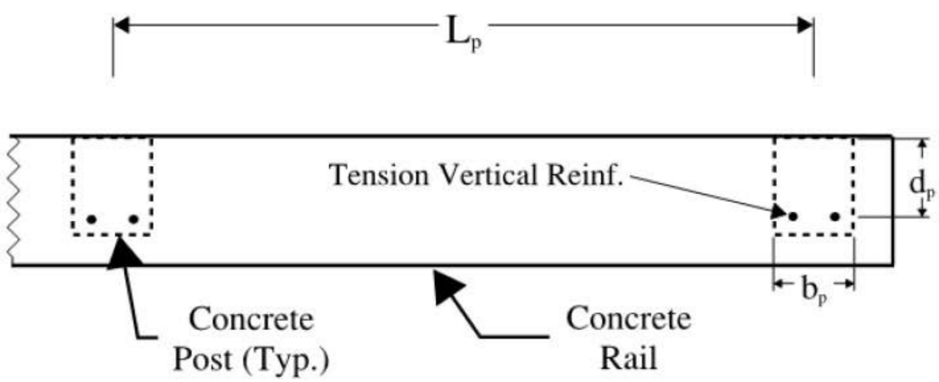


Figure 1c. Plan View of a Concrete Post-and-Beam Railing showing Concrete Post Inputs

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Bridge No. 70802
MASH TL-3 Compliance Assessment

(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$H_{min} := 29in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 38.25 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

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(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 38.25 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$$Minimum\_Height\_of\_Barrier\_Check := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$Minimum\_Height\_of\_Barrier\_Check = \text{"OK"}$$

(3) Geometric Criteria:

$S_{post} := 4.5in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 14in$	Vertical Clear Opening (in.)
$\Sigma A := 24in$	Total Rail Contact Width (in.)
$H_w = 38.25 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.

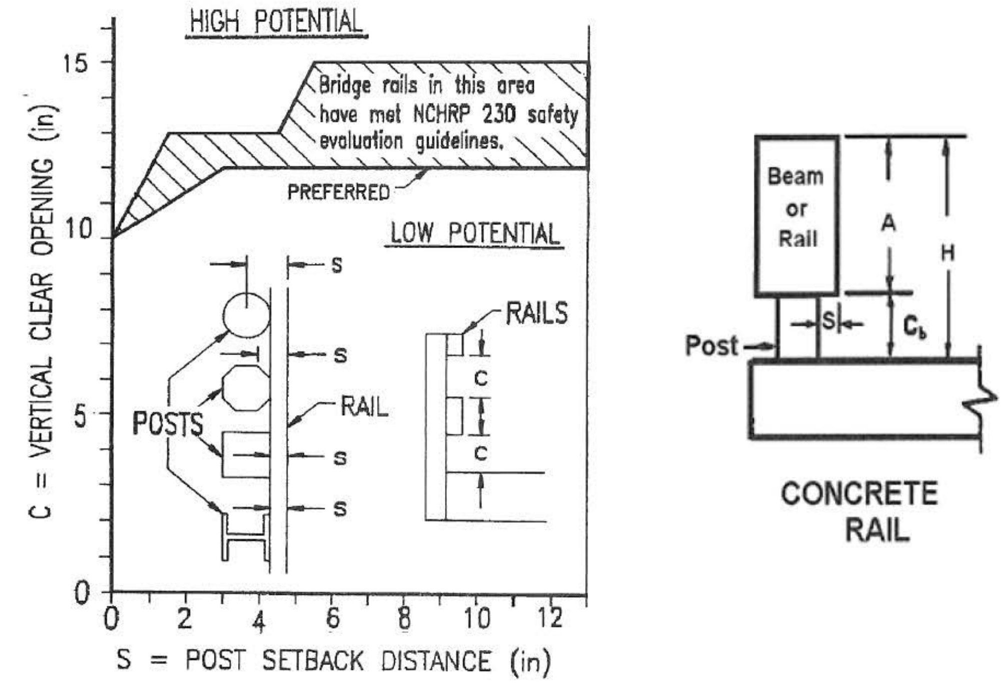


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

(3-conti.) Geometric Criteria:

$S_{post} = 4.5 \cdot in$ $\Sigma A = 24 \cdot in$ $H_w = 38.25 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_w} = 0.627$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

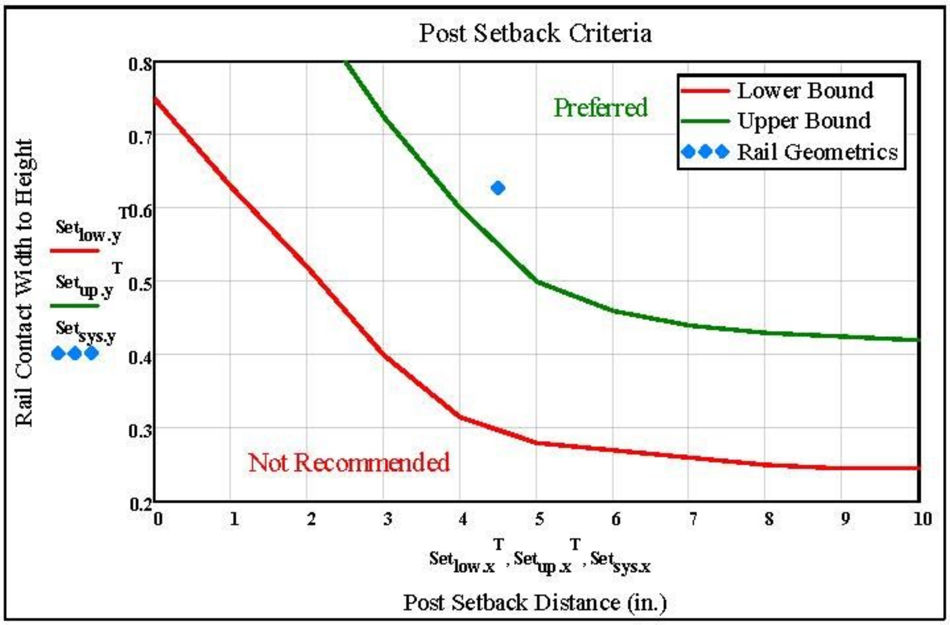
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 4.5$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.627$ Ratio of Contact Width to Total Height rail geometric point



NotRecommended := 1 Marginal := 2 Preferred := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Post_Setback_Criteria_Rail_Geometric_Point := Preferred

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0 \;\; 3 \;\; 13)$

$snag_{low.y} := (10 \;\; 12 \;\; 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0 \;\; 1.25 \;\; 4.25 \;\; 5.25 \;\; 13)$

$snag_{up.y} := (10 \;\; 13 \;\; 13 \;\; 15 \;\; 15)$

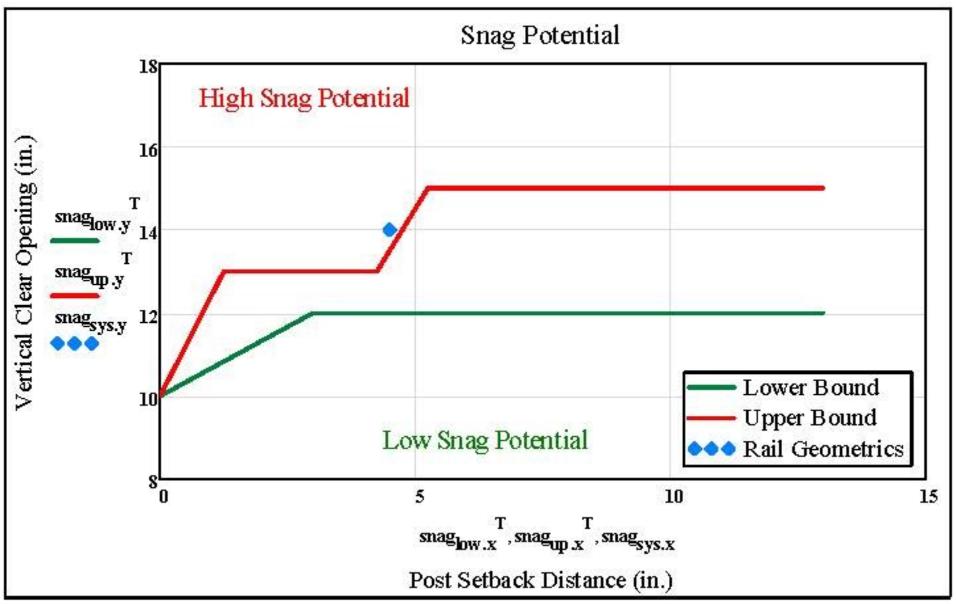
Upper Boundary for Snag Potential Criteria
x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 4.5$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 14$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1 Marginal = 2 LowSnagPotential := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := HighSnagPotential

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(3) Geometric Criteria - Summary of Results:

$$\text{Post\_Setback\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if Post\_Setback\_Criteria\_Rail\_Geometric\_Point} = \text{Preferred} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Post\_Setback\_Criteria\_Check} = \text{"OK"}$$

$$\text{Snag\_Potential\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if Snag\_Potential\_Criteria\_Rail\_Geometric\_Point} = \text{LowSnagPotential} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Snag\_Potential\_Criteria\_Check} = \text{"NOT OK"}$$

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Flexural Capacity of the Concrete Rail: M_{cr}

$f_y = 40 \cdot ksi$ — Yield Strength of Concrete Reinforcing Steel, (ksi)

$f'_c = 4 \cdot ksi$ — Concrete Compressive Strength

$b_{cr} = 14 \cdot in$ — Width of the Concrete Rail (in.)

$A_{cr} = 2.4 \cdot in^2$ — Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)

$d_{cr} = 6 \cdot in$ — Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)

$$a_{cr} := \frac{A_{cr} \cdot f_y}{0.85 f'_c \cdot b_{cr}} = 2.017 \cdot in$$

Whitney Stress Block Depth (in.)

$$M_{cr} := A_{cr} \cdot f_y \cdot \left(d_{cr} - \frac{a_{cr}}{2} \right) = 39.933 \cdot kip \cdot ft$$

Flexural Capacity of the Concrete Rail (kip-ft)

$y_{cr} = 31 \cdot in$ — Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)

$y_{bar} := y_{cr} = 31 \cdot in$ — Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

Note: $y_{bar} = y_{cr}$, since the only rail is the concrete rail

(4b) Post Strength: P_p

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_p = 14 \cdot in$	Width of Concrete Post (in.)
$A_p = 2.37 \cdot in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p = 7.5 \cdot in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$a_p := \dfrac{A_p \cdot f_y}{0.85 \cdot f'_c \cdot b_p} = 1.992 \cdot in$	Whitney Stress Block Depth (in.)
$M_{post} := A_p \cdot f_y \cdot \left(d_p - \dfrac{a_p}{2}\right) = 51.383 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Post (kip-ft)
$y_{bar} = 31 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)
$h_{curb} = 10 \cdot in$	Height of curb (in.)
$y_p := y_{bar} - h_{curb} = 21 \cdot in$	Height measured from the bottom of the Concrete Post to the Resultant Force of all Rails (in.)
$P_p := \dfrac{M_{post}}{y_p} = 29.362 \cdot kip$	Post Strength (kip)

(4c) Ultimate Resistance (Nominal Resistance) of the Railing for a Single-Span Failure Mode: R_1

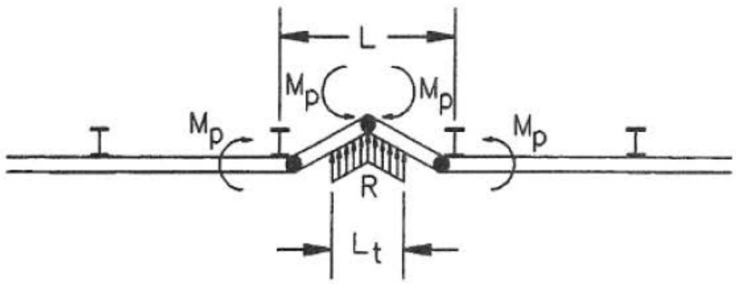


Figure 4c. Single-Span Failure Mode for Post-and-Beam Railings

$P_p = 29.362 \cdot kip$ — Post Strength (kip)

$N_1 := 1$ — Number of Failure Spans

$M_p := M_{cr} = 39.933 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)
Note: $M_p = M_{cr}$, since the only rail is the concrete rail

$L_p = 10 \cdot ft$ — Post Spacing (ft.)
Note: $L_p = L$ in Figure 4c.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_p}{2 \cdot N_1 \cdot L_p - L_t} = 39.933 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode (kip)
- Eqn. A13.3.2-1

(4d) Ultimate Resistance (Nominal Resistance) of the Railing for a Two-Span Failure Mode: R_2

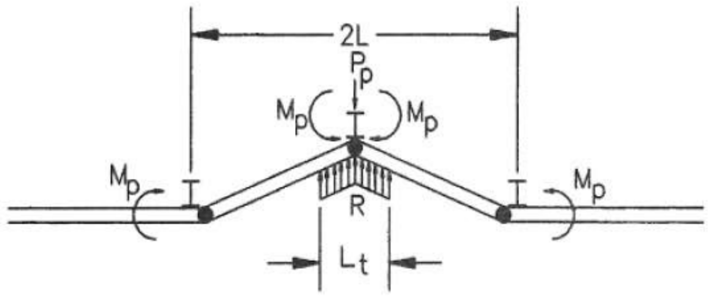


Figure 4d. Two-Span Failure Mode for Post-and-Beam Railings

$P_p = 29.362 \cdot kip$ — Post Strength (kip)

$N_2 := 2$ — Number of Failure Spans

$M_p = 39.933 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10 \cdot ft$ — Post Spacing (ft.)
Note: L_p = L in Figure 4d.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_2 := \frac{16 \cdot M_p + N_2^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_2 \cdot L_p - L_t} = 50.372 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode (kip)
- Eqn. A13.3.2-2

(4e) Ultimate Resistance (Nominal Resistance) of the Railing for a Three-Span Failure Mode: R_3

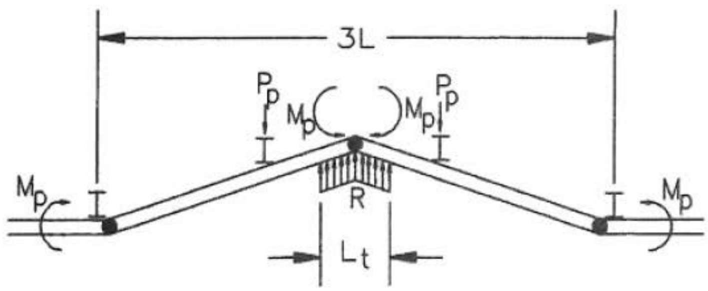


Figure 4e. Three-Span Failure Mode for Post-and-Beam Railings

$P_p = 29.362 \cdot kip$ — Post Strength (kip)

$N_3 := 3$ — Number of Failure Spans

$M_p = 39.933 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10 \cdot ft$ — Post Spacing (ft.)
Note: L_p = L in Figure 4e.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_3 := \frac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_p}{2 \cdot N_3 \cdot L_p - L_t} = 53.355 \cdot kip$$

Ultimate Resistance of the Railing for a Three-Span Failure Mode (kip)
- Eqn. A13.3.2-1

(4f) Ultimate Resistance (Nominal Resistance) of the Railing for a 4-8 Span Failure Mode: R_4 - R_8

$P_p = 29.362 \cdot kip$ $\quad L_p = 10\,ft$ $\quad M_p = 39.933 \cdot kip \cdot ft$ $\quad L_t = 4\,ft$

$N_4 := 4$ $\quad N_5 := 5$ $\quad N_6 := 6$ $\quad N_7 := 7$ $\quad N_8 := 8$

$$R_4 := \frac{16 \cdot M_p + N_4^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_4 \cdot L_p - L_t} = 70.221 \cdot kip$$

Ultimate Resistance of the Railing for a Four-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_p}{2 \cdot N_5 \cdot L_p - L_t} = 80.06 \cdot kip$$

Ultimate Resistance of the Railing for a Five-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_6 := \frac{16 \cdot M_p + N_6^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_6 \cdot L_p - L_t} = 96.631 \cdot kip$$

Ultimate Resistance of the Railing for a Six-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_7 := \frac{16 \cdot M_p + (N_7 - 1) \cdot (N_7 + 1) \cdot P_p \cdot L_p}{2 \cdot N_7 \cdot L_p - L_t} = 108.328 \cdot kip$$

Ultimate Resistance of the Railing for a Seven-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_8 := \frac{16 \cdot M_p + N_8^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_8 \cdot L_p - L_t} = 124.554 \cdot kip$$

Ultimate Resistance of the Railing for a Eight-Span Failure Mode (kip)
- Eqn. A13.3.2-2

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$R_r := \min(R_1, R_2, R_3, R_4, R_5, R_6, R_7, R_8) = 39.933 \cdot kip$

Total Ultimate Resistance of the railing @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_1 - R_8)

$H_e = 19 \cdot in$

Height of Equivalent Transverse Load

$y_{bar} = 31 \cdot in$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$F_t = 71 \cdot kip$

Transverse Impact Force

$R_R := R_r \cdot \left(\frac{y_{bar}}{H_e}\right) = 65.153 \cdot kip$

Total Ultimate Resistance of the railing @ H_e (kip)

Structural_Capacity_of_Barrier_Check := | "OK" if $R_R \geq F_t$
| "NOT OK" otherwise

Structural_Capacity_of_Barrier_Check = "NOT OK"

(5) Strength Analysis of the Barrier at an End Section or Joint:

(5a) Ultimate Resistance of the Railing at an End Section or Joint for a Single-Span Failure Mode: R_{1end}

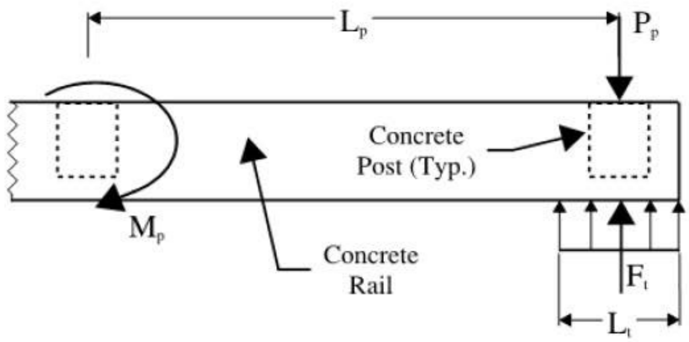


Figure 5a. Single-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 29.362 \cdot kip$ — Post Strength (kip)

$M_p = 39.933 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10\,ft$ — Post Spacing (ft.)

$$R_{1end} := P_p + \frac{M_p}{L_p} = 33.355 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

Using the Approach from AASHTO Section 13:

$N_1 = 1$

$$R_{1A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_1} i\right)}{2N_1 \cdot L_p - L_t} = 41.694 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r1.end} := \min\left(R_{1end}, R_{1A.end}\right) = 33.355 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

(5b) Ultimate Resistance of the Railing at an End Section or Joint for a Two-Span Failure Mode: R_{2end}

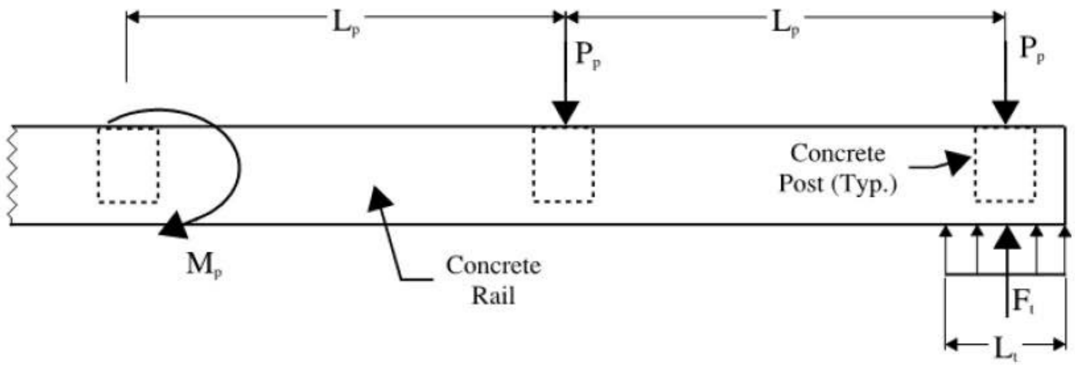


Figure 5b. Two-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 29.362 \cdot kip$ — Post Strength (kip)

$M_p = 39.933 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10\,ft$ — Post Spacing (ft.)

$$R_{2end} := \frac{P_p \cdot 3L_p + M_p}{2L_p} = 46.039 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

Using AASHTO Section 13: $N_2 = 2$

$$R_{2A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_2} i\right)}{2N_2 \cdot L_p - L_t} = 51.155 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r2.end} := \min\left(R_{2end}, R_{2A.end}\right) = 46.039 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

(5) Strength Analysis of the Barrier at an End Section or Joint-Summary of Results:

$R_{r1.end} = 33.355 \cdot kip$

Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

$R_{r2.end} = 46.039 \cdot kip$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

$R_{rend} := \min\left(R_{r1.end}, R_{r2.end}\right) = 33.355 \cdot kip$

Total Ultimate Resistance of the railing at an End Section or Joint @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_{r1end} & R_{r2end})

$y_{bar} = 31 \cdot in$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$H_e = 19 \cdot in$

Height of Equivalent Transverse Load

$F_t = 71 \cdot kip$

Transverse Impact Force

$R_{Rend} := R_{rend} \cdot \left(\frac{y_{bar}}{H_e}\right) = 54.421 \cdot kip$

Total Ultimate Resistance of the railing at an End Section or Joint @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} := \begin{vmatrix} \text{"OK"} & \text{if } R_{Rend} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{vmatrix}$$

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

(7) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "OK"

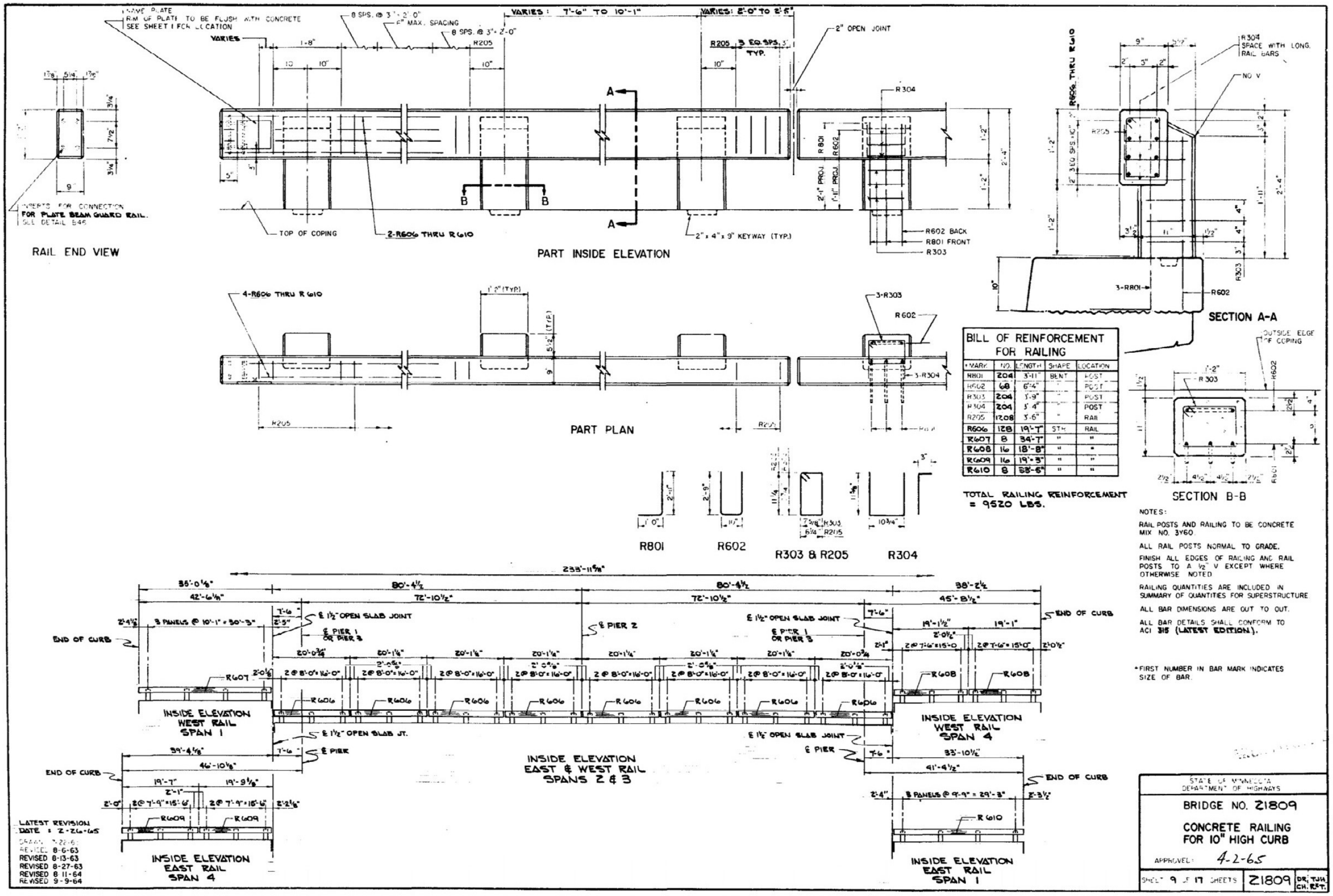
Snag_Potential_Criteria_Check = "NOT OK"

Structural_Capacity_of_Barrier_Check = "NOT OK"

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

The One Line barrier on Bridge No. 70802 does not satisfy all MASH TL-3 Criteria

APPENDIX C3: ONE-LINE BRIDGE RAIL ON BRIDGE NO. 21809



BILL OF REINFORCEMENT FOR RAILING

*MARK	NO.	LENGTH	SHAPE	LOCATION
R801	204	3'-11"	BENT	POST
R602	68	6'-4"	"	POST
R303	204	3'-9"	"	POST
R304	204	3'-4"	"	POST
R205	1208	3'-6"	"	RAIL
R606	128	19'-7"	STR.	RAIL
R607	8	34'-7"	"	"
R608	16	18'-8"	"	"
R609	16	19'-3"	"	"
R610	8	33'-5"	"	"

TOTAL RAILING REINFORCEMENT = 9520 LBS.

NOTES:

RAIL POSTS AND RAILING TO BE CONCRETE MIX NO. 3Y60.

ALL RAIL POSTS NORMAL TO GRADE.

FINISH ALL EDGES OF RAILING AND RAIL POSTS TO A ½" V EXCEPT WHERE OTHERWISE NOTED.

RAILING QUANTITIES ARE INCLUDED IN SUMMARY OF QUANTITIES FOR SUPERSTRUCTURE.

ALL BAR DIMENSIONS ARE OUT TO OUT.

ALL BAR DETAILS SHALL CONFORM TO ACI 315 (LATEST EDITION).

*FIRST NUMBER IN BAR MARK INDICATES SIZE OF BAR.

LATEST REVISION
DATE : 2-26-65

DRAWN 7-22-63
REVISED 8-6-63
REVISED 8-13-63
REVISED 8-27-63
REVISED 8-11-64
REVISED 9-9-64

STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS

BRIDGE NO. 21809

CONCRETE RAILING FOR 10" HIGH CURB

APPROVED: 4-2-65

SHEET 9 OF 17 SHEETS | 21809 | DR. TJH CH. RST.

SUBJECT: **MnDOT One-Line**
Bridge No. 21809
MASH Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$ — Compressive Strength of Concrete (psi)

$f_y := 40ksi$ — Yield Strength of Concrete Reinforcing Steel, (ksi)

$E_s := 29000ksi$ — Modulus of Elasticity of Steel (ksi)

$H_w := 38.25in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$t_o := 0in$ — Thickness of overlay (in.)

$h_w := H_w + t_o = 38.25 \cdot in$ — Total height of the barrier (in.)

(1b) Concrete Rail Inputs:

$b_{cr} := 14in$ — Width of the Concrete Rail (in.)

$A_{cr} := 1.76in^2$ — Total area of the reinforcement bars acting in tension in the Concrete Rail (in2)

$d_{cr} := 7in$ — Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)

$y_{cr} := 31in$ — Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)

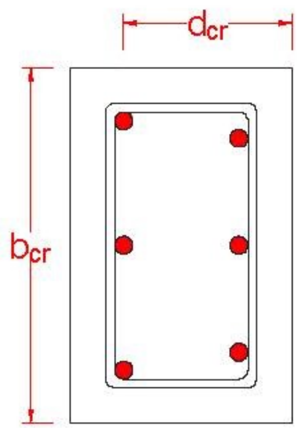


Figure 1b. Profile View showing Concrete Rail Inputs

(1c) <u>Concrete Post Inputs:</u>

$b_p := 14in$	Width of Concrete Post (in.)
$A_p := 2.37in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p := 8.5in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$h_{curb} := 10in$	Height of curb (in.)
$L_p := 10ft + 1in = 10.083\,ft$	Spacing of Concrete Posts (ft.)

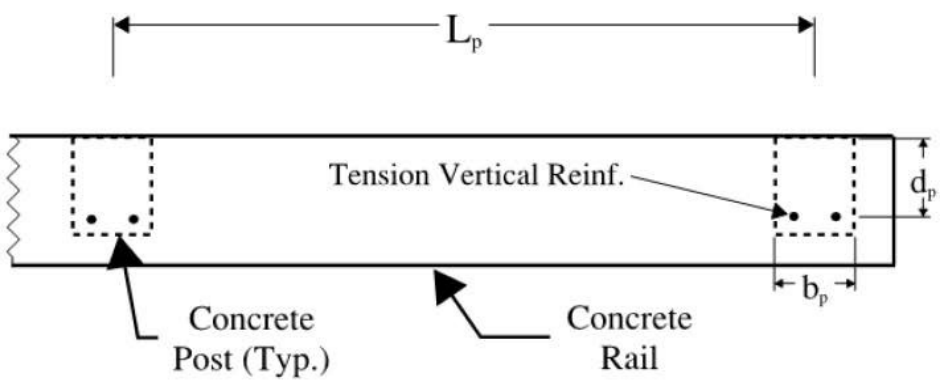


Figure 1c. Plan View of a Concrete Post-and-Beam Railing showing Concrete Post Inputs

SUBJECT: **MnDOT One-Line**
Bridge No. 21809
MASH TL-3 Compliance Assessment

(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$\mathbf{TL := 3}$ Test Level

$\mathbf{F_t := 71kip}$ Transverse Impact Force

$\mathbf{L_t := 4ft}$ Longitudinal Length of Distribution of Impact Force

$\mathbf{H_e := 19in}$ Height of Equivalent Transverse Load

$\mathbf{H_{min} := 29in}$ Minimum height of a MASH TL-3 barrier (in.)

$\mathbf{H_w = 38.25 \cdot in}$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ — Minimum height of a MASH TL-3 barrier (in.)

$H_w = 38.25 \cdot in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

(3) Geometric Criteria:

$S_{post} := 3.5in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 14in$	Vertical Clear Opening (in.)
$\Sigma A := 24in$	Total Rail Contact Width (in.)
$H_w = 38.25 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.

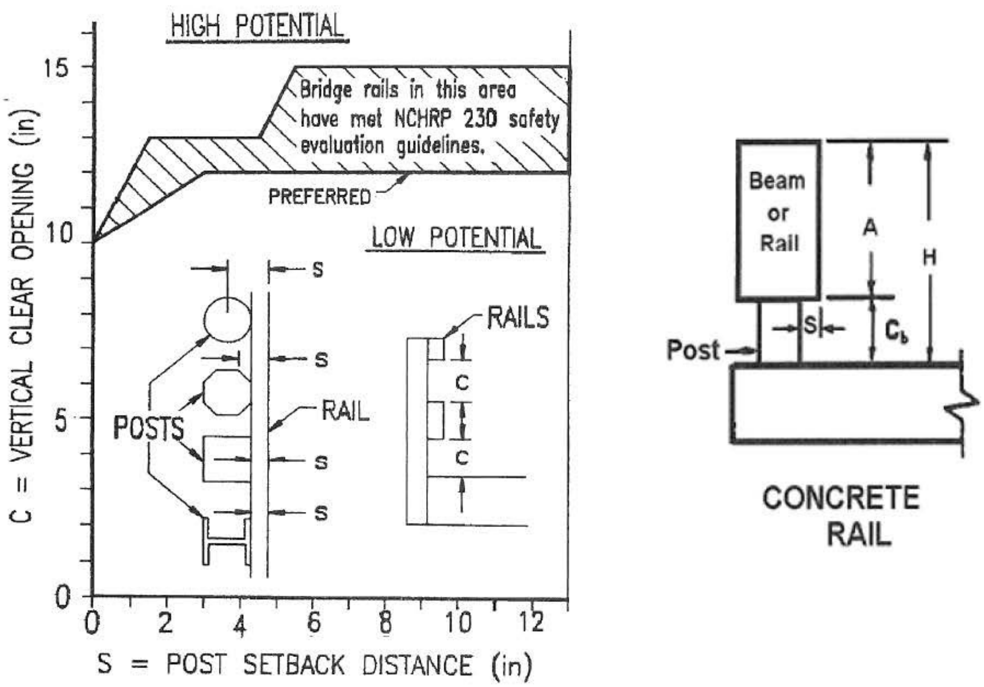


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

(3-conti.) Geometric Criteria:

$S_{post} = 3.5 \cdot in$ $\Sigma A = 24 \cdot in$ $H_w = 38.25 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_w} = 0.627$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

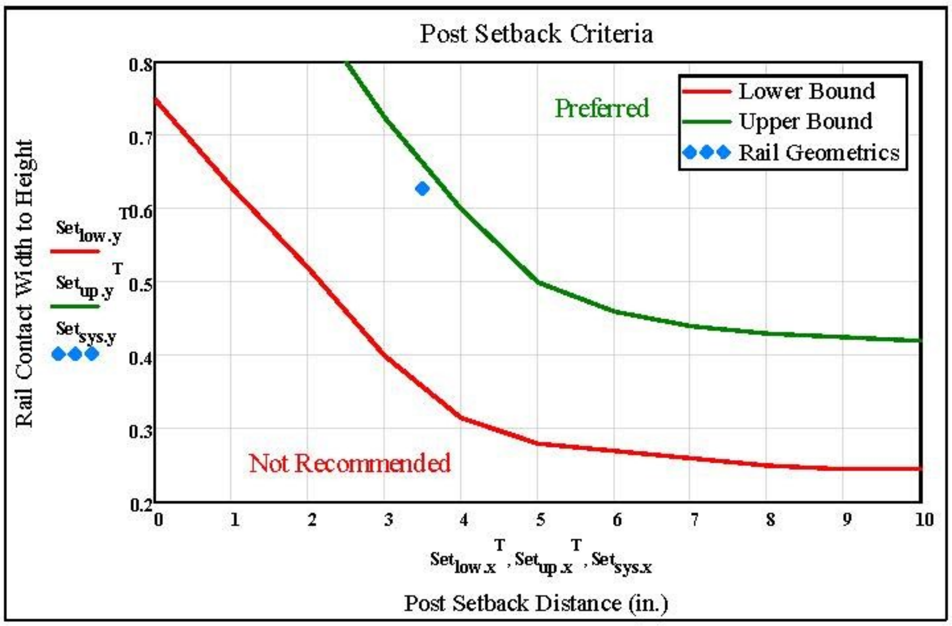
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 3.5$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.627$ Ratio of Contact Width to Total Height rail geometric point



$NotRecommended := 1$ $Marginal := 2$ $Preferred := 3$

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Post_Setback_Criteria_Rail_Geometric_Point := Marginal

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MASH Compliance Assessment

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0\ \ 3\ \ 13)$

$snag_{low.y} := (10\ \ 12\ \ 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0\ \ 1.25\ \ 4.25\ \ 5.25\ \ 13)$

$snag_{up.y} := (10\ \ 13\ \ 13\ \ 15\ \ 15)$

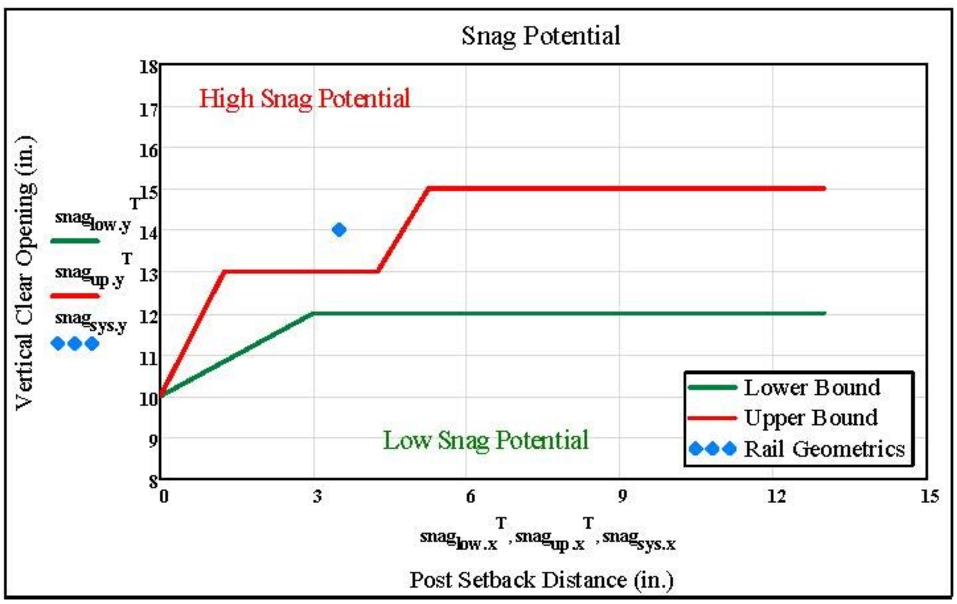
Upper Boundary for Snag Potential Criteria
x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 3.5$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 14$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1 **Marginal = 2** **LowSnagPotential := 3**

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := HighSnagPotential

(3) Geometric Criteria - Summary of Results:

$$\text{Post\_Setback\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if } \text{Post\_Setback\_Criteria\_Rail\_Geometric\_Point} = \text{Preferred} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Post\_Setback\_Criteria\_Check} = \text{"NOT OK"}$$

$$\text{Snag\_Potential\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if } \text{Snag\_Potential\_Criteria\_Rail\_Geometric\_Point} = \text{LowSnagPotential} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Snag\_Potential\_Criteria\_Check} = \text{"NOT OK"}$$

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Bridge No. 21809
MASH TL-3 Compliance Assessment

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Flexural Capacity of the Concrete Rail: M_{cr}

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_{cr} = 14 \cdot in$	Width of the Concrete Rail (in.)
$A_{cr} = 1.76 \cdot in^2$	Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)
$d_{cr} = 7 \cdot in$	Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)
$a_{cr} := \dfrac{A_{cr} \cdot f_y}{0.85 f'_c \cdot b_{cr}} = 1.479 \cdot in$	Whitney Stress Block Depth (in.)
$M_{cr} := A_{cr} \cdot f_y \cdot \left(d_{cr} - \dfrac{a_{cr}}{2}\right) = 36.728 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Rail (kip-ft)
$y_{cr} = 31 \cdot in$	Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)
$y_{bar} := y_{cr} = 31 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.) Note: $y_{bar} = y_{cr}$, since the only rail is the concrete rail

(4b) Post Strength: P_p

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_p = 14 \cdot in$	Width of Concrete Post (in.)
$A_p = 2.37 \cdot in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p = 8.5 \cdot in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$a_p := \frac{A_p \cdot f_y}{0.85 \cdot f'_c \cdot b_p} = 1.992 \cdot in$	Whitney Stress Block Depth (in.)
$M_{post} := A_p \cdot f_y \cdot \left(d_p - \frac{a_p}{2}\right) = 59.283 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Post (kip-ft)
$y_{bar} = 31 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)
$h_{curb} = 10 \cdot in$	Height of curb (in.)
$y_p := y_{bar} - h_{curb} = 21 \cdot in$	Height measured from the bottom of the Concrete Post to the Resultant Force of all Rails (in.)
$P_p := \frac{M_{post}}{y_p} = 33.876 \cdot kip$	Post Strength (kip)

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(4c) **Ultimate Resistance (Nominal Resistance) of the Railing for a Single-Span Failure Mode:** R_1

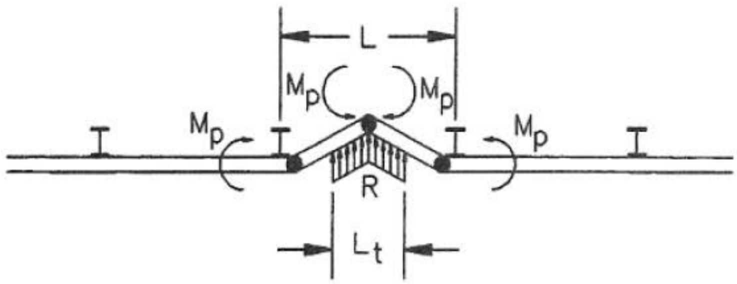

Figure 4c. Single-Span Failure Mode for Post-and-Beam Railings

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$N_1 := 1$ — Number of Failure Spans

$M_p := M_{cr} = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)
Note: $M_p = M_{cr}$, since the only rail is the concrete rail

$L_p = 10.083 \cdot ft$ — Post Spacing (ft.)
Note: $L_p = L$ in Figure 4c.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_p}{2 \cdot N_1 \cdot L_p - L_t} = 36.35 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode (kip)
- Eqn. A13.3.2-1

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(4d) Ultimate Resistance (Nominal Resistance) of the Railing for a Two-Span Failure Mode: R_2

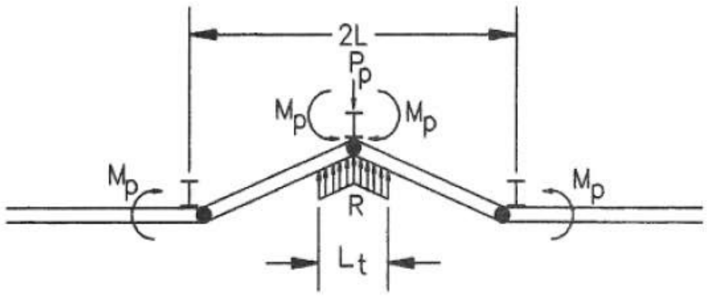


Figure 4d. Two-Span Failure Mode for Post-and-Beam Railings

$P_p = 33.876 \cdot kip$	Post Strength (kip)
$N_2 := 2$	Number of Failure Spans
$M_p = 36.728 \cdot kip \cdot ft$	Flexural Capacity of all Rails (kip-ft)
$L_p = 10.083 \cdot ft$	Post Spacing (ft.) Note: L_p = L in Figure 4d.
$L_t = 4 \cdot ft$	Longitudinal Length of Distribution of the Impact Force
$R_2 := \dfrac{16 \cdot M_p + N_2^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_2 \cdot L_p - L_t} = 53.78 \cdot kip$	Ultimate Resistance of the Railing for a Two-Span Failure Mode (kip) - Eqn. A13.3.2-2

(4e) Ultimate Resistance (Nominal Resistance) of the Railing for a Three-Span Failure Mode: R_3

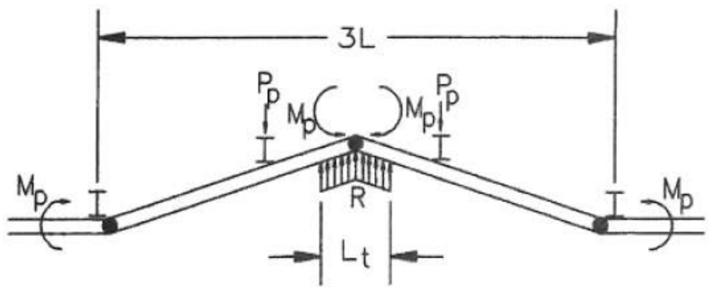


Figure 4e. Three-Span Failure Mode for Post-and-Beam Railings

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$N_3 := 3$ — Number of Failure Spans

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10.083 \cdot ft$ — Post Spacing (ft.)
Note: L_p = L in Figure 4e.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_3 := \frac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_p}{2 \cdot N_3 \cdot L_p - L_t} = 58.767 \cdot kip$$

Ultimate Resistance of the Railing for a Three-Span Failure Mode (kip)
- Eqn. A13.3.2-1

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$R_r := \min(R_1, R_2, R_3) = 36.35 \cdot kip$

Total Ultimate Resistance of the railing @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_1 - R_8)

$H_e = 19 \cdot in$

Height of Equivalent Transverse Load

$y_{bar} = 31 \cdot in$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$F_t = 71 \cdot kip$

Transverse Impact Force

$R_R := R_r \cdot \left(\frac{y_{bar}}{H_e}\right) = 59.307 \cdot kip$

Total Ultimate Resistance of the railing @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_Check} := \begin{vmatrix} \text{"OK"} & \text{if } R_R \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{vmatrix}$$

Structural_Capacity_of_Barrier_Check = "NOT OK"

(5) Strength Analysis of the Barrier at an End Section or Joint:

(5a) Ultimate Resistance of the Railing at an End Section or Joint for a Single-Span Failure Mode: R_{1end}

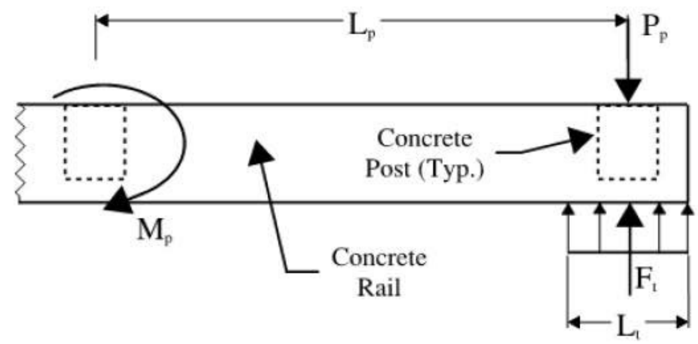


Figure 5a. Single-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10.083\, ft$ — Post Spacing (ft.)

$$R_{1end} := P_p + \frac{M_p}{L_p} = 37.519 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

Using the Approach from AASHTO Section 13: $N_1 = 1$

$$R_{1A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_1} i\right)}{2N_1 \cdot L_p - L_t} = 46.802 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r1.end} := \min\left(R_{1end}, R_{1A.end}\right) = 37.519 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

(5b) Ultimate Resistance of the Railing at an End Section or Joint for a Two-Span Failure Mode: R_{2end}

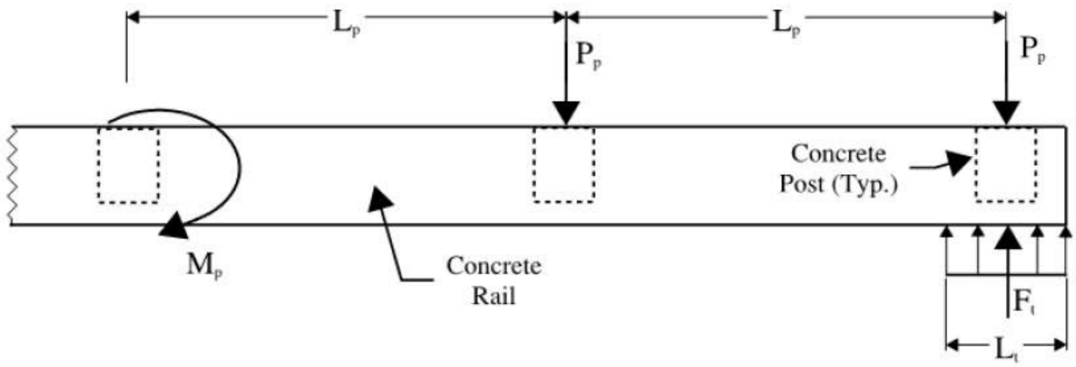


Figure 5b. Two-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 33.876 \cdot kip$ Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ Flexural Capacity of all Rails (kip-ft)

$L_p = 10.083\,ft$ Post Spacing (ft)

$$R_{2end} := \frac{P_p \cdot 3L_p + M_p}{2L_p} = 52.635 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

Using AASHTO Section 13: $N_2 = 2$

$$R_{2A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_2} i\right)}{2N_2 \cdot L_p - L_t} = 58.43 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r2.end} := \min\left(R_{2end}, R_{2A.end}\right) = 52.635 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

(5) Strength Analysis of the Barrier at an End Section or Joint-Summary of Results:

$R_{r1.end} = 37.519 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

$R_{r2.end} = 52.635 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

$R_{rend} := \min\left(R_{r1.end}, R_{r2.end}\right) = 37.519 \cdot kip$ — Total Ultimate Resistance of the railing at an End Section or Joint @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_{r1end} & R_{r2end})

$y_{bar} = 31 \cdot in$ — Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$H_e = 19 \cdot in$ — Height of Equivalent Transverse Load

$F_t = 71 \cdot kip$ — Transverse Impact Force

$R_{Rend} := R_{rend} \cdot \left(\frac{y_{bar}}{H_e}\right) = 61.215 \cdot kip$ — Total Ultimate Resistance of the railing at an End Section or Joint @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{Rend} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

(7) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "NOT OK"

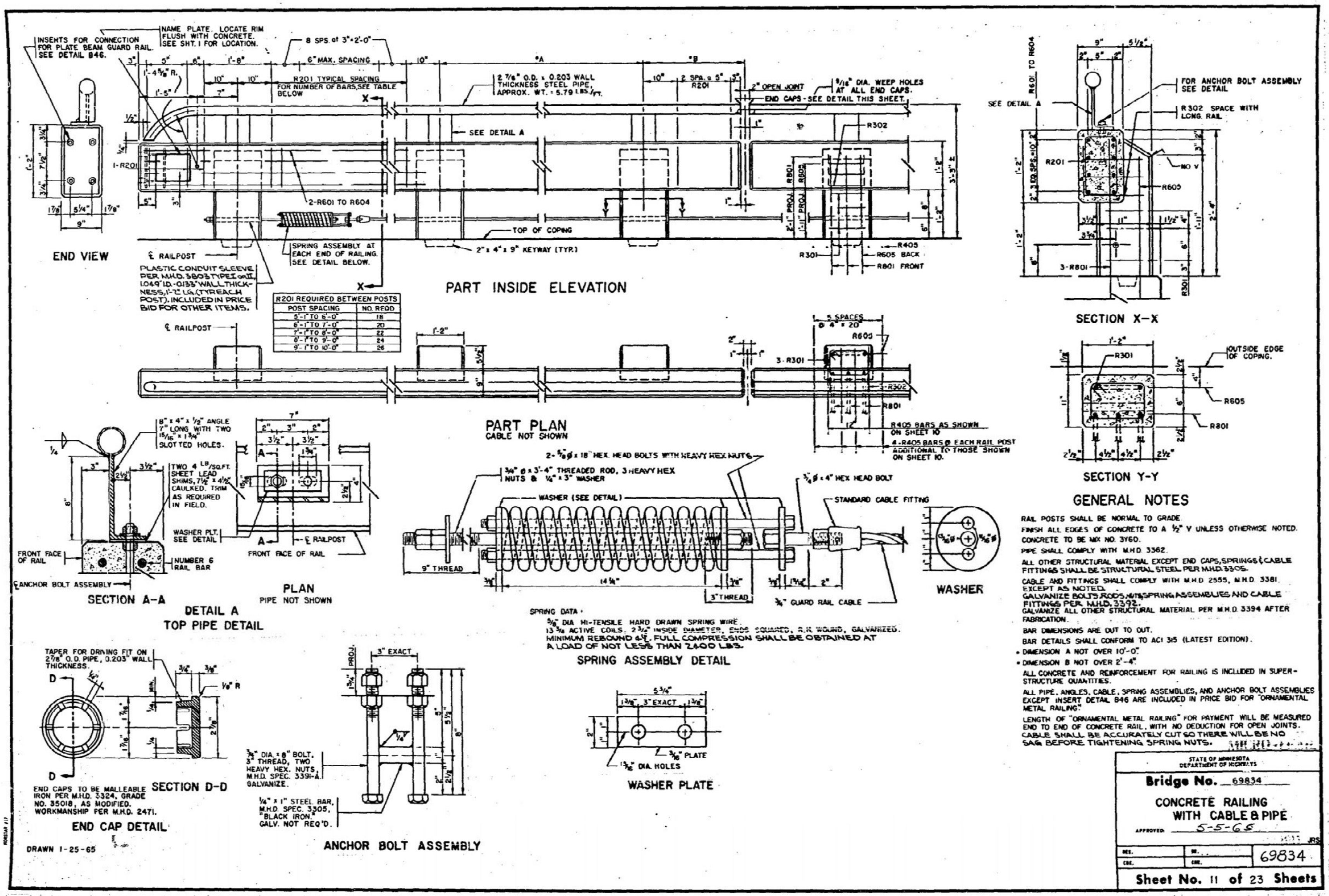
Snag_Potential_Criteria_Check = "NOT OK"

Structural_Capacity_of_Barrier_Check = "NOT OK"

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

The One Line Barrier on Bridge No. 21809 does not satisfy all MASH TL-3 Criteria

APPENDIX C4: ONE-LINE BRIDGE RAIL ON BRIDGE NO. 69834


END VIEW
PART INSIDE ELEVATION
SECTION X-X
PART PLAN
CABLE NOT SHOWN
SECTION Y-Y
SECTION A-A
DETAIL A
TOP PIPE DETAIL
PLAN
PIPE NOT SHOWN
SPRING ASSEMBLY DETAIL
WASHER
WASHER PLATE
SECTION D-D
END CAP DETAIL
ANCHOR BOLT ASSEMBLY
R201 REQUIRED BETWEEN POSTS
POST SPACING | NO. REQD
5'-1" TO 6'-0" | 18
6'-1" TO 7'-0" | 20
7'-1" TO 8'-0" | 22
8'-1" TO 9'-0" | 24
9'-1" TO 10'-0" | 26
GENERAL NOTES
RAIL POSTS SHALL BE NORMAL TO GRADE.
FINISH ALL EDGES OF CONCRETE TO A ½" V UNLESS OTHERWISE NOTED.
CONCRETE TO BE MIX NO. 3Y60.
PIPE SHALL COMPLY WITH M.H.D. 3362.
ALL OTHER STRUCTURAL MATERIAL EXCEPT END CAPS, SPRINGS & CABLE FITTINGS SHALL BE STRUCTURAL STEEL PER M.H.D. 3305.
CABLE AND FITTINGS SHALL COMPLY WITH M.H.D. 2555, M.H.D. 3381 EXCEPT AS NOTED.
GALVANIZE BOLTS, RODS, NUTS, SPRING ASSEMBLIES AND CABLE FITTINGS PER M.H.D. 3392.
GALVANIZE ALL OTHER STRUCTURAL MATERIAL PER M.H.D. 3394 AFTER FABRICATION.
BAR DIMENSIONS ARE OUT TO OUT.
BAR DETAILS SHALL CONFORM TO ACI 315 (LATEST EDITION).
• DIMENSION A NOT OVER 10'-0".
• DIMENSION B NOT OVER 2'-4".
ALL CONCRETE AND REINFORCEMENT FOR RAILING IS INCLUDED IN SUPERSTRUCTURE QUANTITIES.
ALL PIPE, ANGLES, CABLE, SPRING ASSEMBLIES, AND ANCHOR BOLT ASSEMBLIES EXCEPT INSERT DETAIL B46 ARE INCLUDED IN PRICE BID FOR "ORNAMENTAL METAL RAILING".
LENGTH OF "ORNAMENTAL METAL RAILING" FOR PAYMENT WILL BE MEASURED END TO END OF CONCRETE RAIL, WITH NO DEDUCTION FOR OPEN JOINTS.
CABLE SHALL BE ACCURATELY CUT SO THERE WILL BE NO SAG BEFORE TIGHTENING SPRING NUTS.
SPRING DATA:
5/8" DIA. HI-TENSILE HARD DRAWN SPRING WIRE.
13 ¾ ACTIVE COILS. 2 ¾" INSIDE DIAMETER, ENDS SQUARED, R.H. WOUND, GALVANIZED.
MINIMUM REBOUND 4½". FULL COMPRESSION SHALL BE OBTAINED AT A LOAD OF NOT LESS THAN 2400 LBS.
END CAPS TO BE MALLEABLE IRON PER M.H.D. 3324, GRADE NO. 35018, AS MODIFIED. WORKMANSHIP PER M.H.D. 2471.
DRAWN 1-25-65
STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS
Bridge No. 69834
CONCRETE RAILING
WITH CABLE & PIPE
APPROVED: 5-5-65
69834
Sheet No. 11 of 23 Sheets

SUBJECT: **MnDOT One-Line**
Bridge No. 69834
MASH Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$ Compressive Strength of Concrete (psi)

$f_y := 40ksi$ Yield Strength of Concrete Reinforcing Steel, (ksi)

$E_s := 29000ksi$ Modulus of Elasticity of Steel (ksi)

$H_w := 28in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$t_o := 0in$ Thickness of overlay (in.)

$h_w := H_w + t_o = 28 \cdot in$ Total height of the barrier (in.)

(1b) Concrete Rail Inputs:

$b_{cr} := 14in$ Width of the Concrete Rail (in.)

$A_{cr} := 1.76in^2$ Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)

$d_{cr} := 7in$ Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)

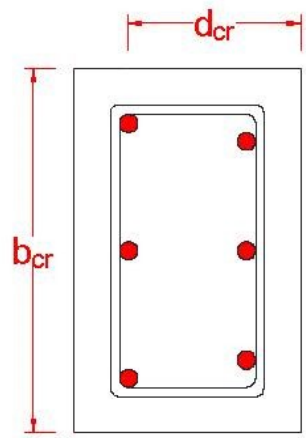


Figure 1b. Profile View showing Concrete Rail Inputs

$y_{cr} := 21in$ Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)

(1c) Concrete Post Inputs:

$b_p := 14in$ Width of Concrete Post (in.)

$A_p := 2.37in^2$ Area of Concrete Post reinforcement acting in tension (in^2)

$d_p := 8.5in$ Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)

$h_{curb} := 0in$ Height of curb (in.)

$L_p := 10ft$ Spacing of Concrete Posts (ft.)

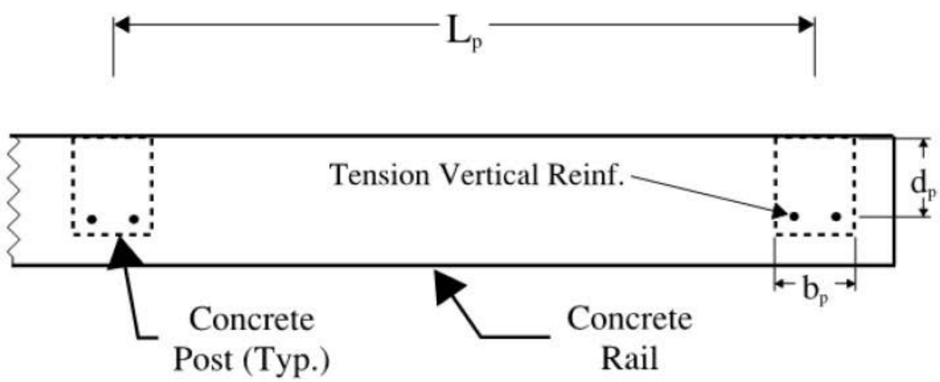


Figure 1c. Plan View of a Concrete Post-and-Beam Railing showing Concrete Post Inputs

SUBJECT: **MnDOT One-Line**
Bridge No. 69834
MASH Compliance Assessment

(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load

$H_{min} := 29in$ Minimum height of a MASH TL-2 barrier (in.)

$H_w = 28 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-2 barrier (in.)

$H_w = 28 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$$Minimum\_Height\_of\_Barrier\_Check := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$Minimum\_Height\_of\_Barrier\_Check = \text{"NOT OKAY"}$$

SUBJECT: MnDOT One-Line
Bridge No. 69834
MASH Compliance Assessment

(3) Geometric Criteria:

$S_{post} := 3.5in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 14in$	Vertical Clear Opening (in.)
$\Sigma A := 14in$	Total Rail Contact Width (in.)
$H_w = 28 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.

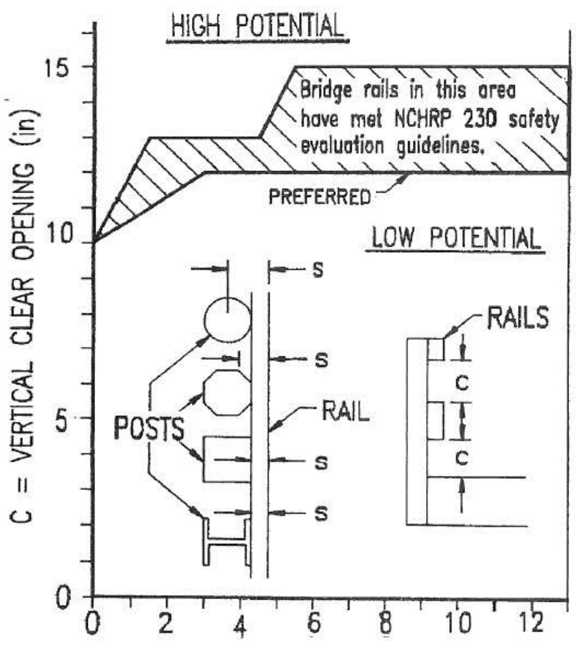
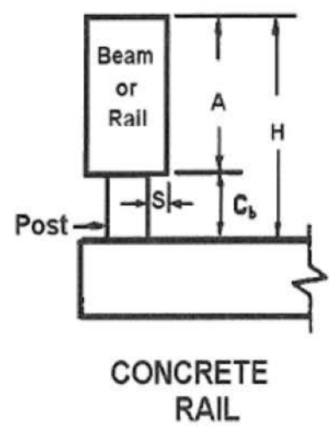


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post



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SUBJECT: MnDOT One-Line On
Bridge No. 69834
MASH Compliance Assessment

(3-conti.) Geometric Criteria:

$S_{post} = 3.5 \cdot in$ $\Sigma A = 14 \cdot in$ $H_w = 28 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_w} = 0.5$

$Set_{low.x} := (0\ 1\ 2\ 3\ 4\ 5\ 6\ 7\ 8\ 9\ 10)$

$Set_{low.y} := (0.75\ 0.63\ 0.52\ 0.4\ 0.315\ 0.28\ 0.27\ 0.26\ 0.25\ 0.245\ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

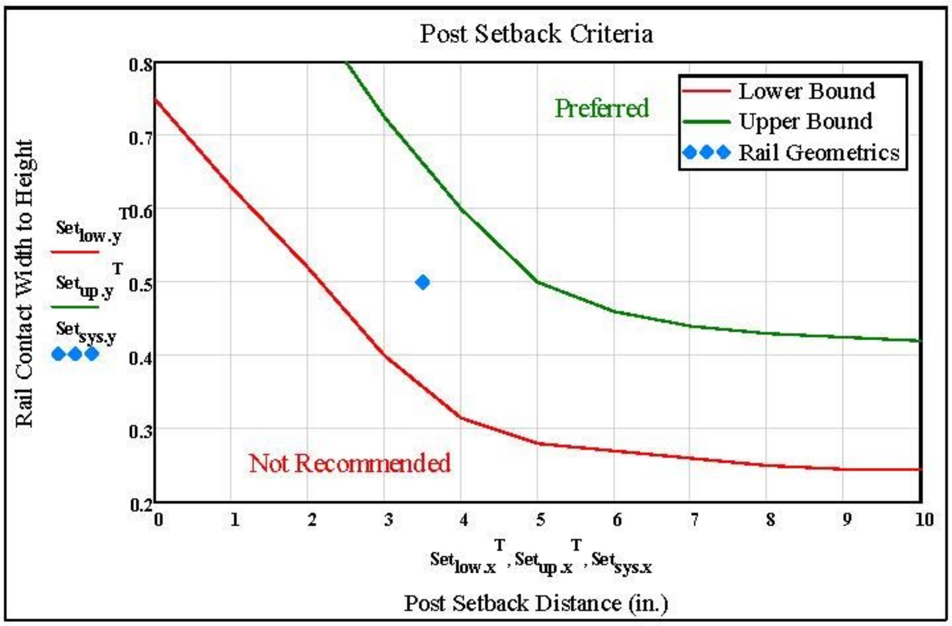
$Set_{up.x} := (2.5\ 3\ 4\ 5\ 6\ 7\ 8\ 9\ 10)$

$Set_{up.y} := (0.8\ 0.725\ 0.6\ 0.5\ 0.46\ 0.44\ 0.43\ 0.425\ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 3.5$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.5$ Ratio of Contact Width to Total Height rail geometric point



$NotRecommended := 1$ $Marginal := 2$ $Preferred := 3$

Region Designation
Note: Marginal region is between Lower and Upper Bounds

$Post\_Setback\_Criteria\_Rail\_Geometric\_Point := Marginal$

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0\ \ 3\ \ 13)$

$snag_{low.y} := (10\ \ 12\ \ 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0\ \ 1.25\ \ 4.25\ \ 5.25\ \ 13)$

$snag_{up.y} := (10\ \ 13\ \ 13\ \ 15\ \ 15)$

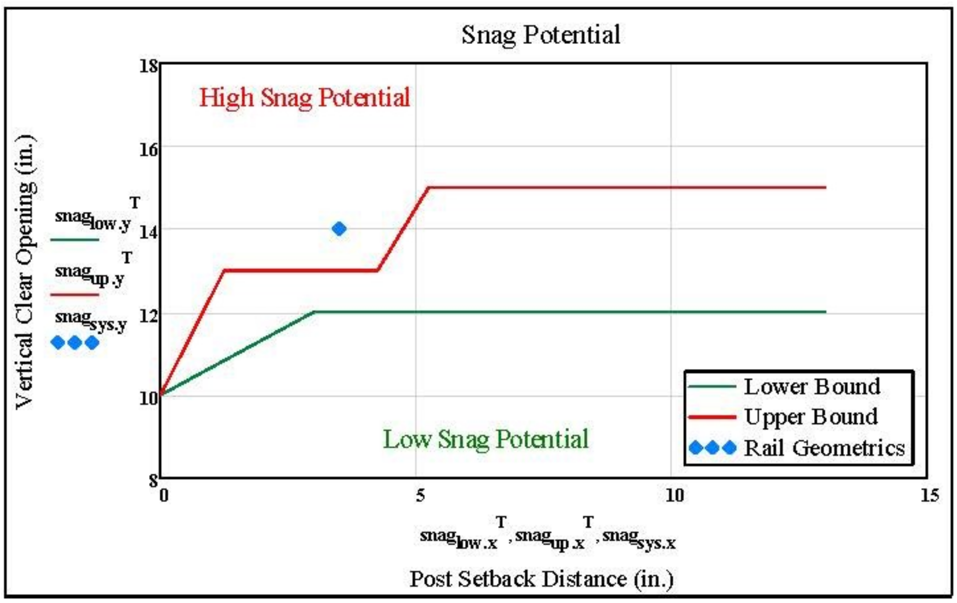
Upper Boundary for Snag Potential Criteria
x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 3.5$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 14$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1   Marginal = 2   LowSnagPotential := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := HighSnagPotential

(3) Geometric Criteria - Summary of Results:

$$\text{Post\_Setback\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if } \text{Post\_Setback\_Criteria\_Rail\_Geometric\_Point} = \text{Preferred} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Post\_Setback\_Criteria\_Check} = \text{"NOT OK"}$$

$$\text{Snag\_Potential\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if } \text{Snag\_Potential\_Criteria\_Rail\_Geometric\_Point} = \text{LowSnagPotential} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Snag\_Potential\_Criteria\_Check} = \text{"NOT OK"}$$

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Flexural Capacity of the Concrete Rail: M_{cr}

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_{cr} = 14 \cdot in$	Width of the Concrete Rail (in.)
$A_{cr} = 1.76 \cdot in^2$	Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)
$d_{cr} = 7 \cdot in$	Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)
$a_{cr} := \dfrac{A_{cr} \cdot f_y}{0.85 f'_c \cdot b_{cr}} = 1.479 \cdot in$	Whitney Stress Block Depth (in.)
$M_{cr} := A_{cr} \cdot f_y \cdot \left(d_{cr} - \dfrac{a_{cr}}{2}\right) = 36.728 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Rail (kip-ft)
$y_{cr} = 21 \cdot in$	Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)
$y_{bar} := y_{cr} = 21 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.) Note: $y_{bar} = y_{cr}$, since the only rail is the concrete rail

(4b) Post Strength: P_p

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_p = 14 \cdot in$	Width of Concrete Post (in.)
$A_p = 2.37 \cdot in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p = 8.5 \cdot in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$a_p := \dfrac{A_p \cdot f_y}{0.85 \cdot f'_c \cdot b_p} = 1.992 \cdot in$	Whitney Stress Block Depth (in.)
$M_{post} := A_p \cdot f_y \cdot \left(d_p - \dfrac{a_p}{2}\right) = 59.283 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Post (kip-ft)
$y_{bar} = 21 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)
$h_{curb} = 0 \cdot in$	Height of curb (in.)
$y_p := y_{bar} - h_{curb} = 21 \cdot in$	Height measured from the bottom of the Concrete Post to the Resultant Force of all Rails (in.)
$P_p := \dfrac{M_{post}}{y_p} = 33.876 \cdot kip$	Post Strength (kip)

(4c) **Ultimate Resistance (Nominal Resistance) of the Railing for a Single-Span Failure Mode:** R_1

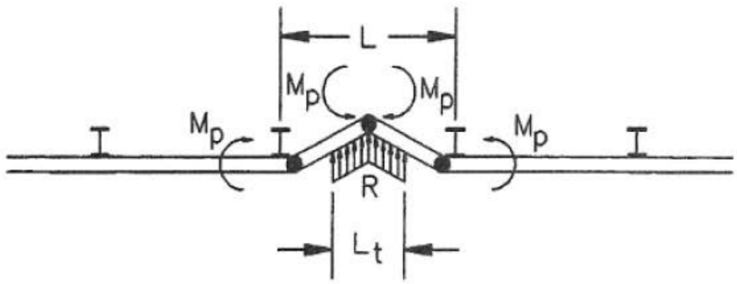


Figure 4c. Single-Span Failure Mode for Post-and-Beam Railings

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$N_1 := 1$ — Number of Failure Spans

$M_p := M_{cr} = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)
Note: $M_p = M_{cr}$, since the only rail is the concrete rail

$L_p = 10 \cdot ft$ — Post Spacing (ft.)
Note: $L_p = L$ in Figure 4c.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_p}{2 \cdot N_1 \cdot L_p - L_t} = 36.728 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode (kip)
- Eqn. A13.3.2-1

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SUBJECT: **MnDOT One-Line**
Bridge No. 69834
MASH Compliance Assessment

(4d) Ultimate Resistance (Nominal Resistance) of the Railing for a Two-Span Failure Mode: R_2

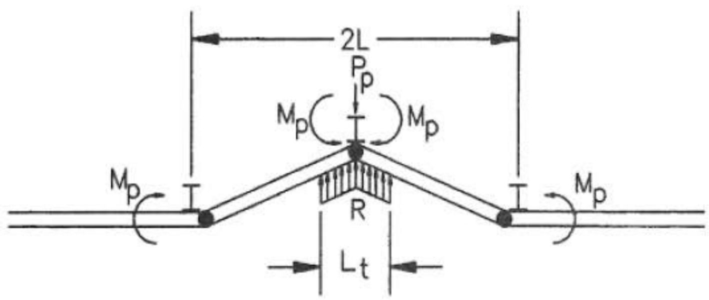


Figure 4d. Two-Span Failure Mode for Post-and-Beam Railings

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$N_2 := 2$ — Number of Failure Spans

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10 \cdot ft$ — Post Spacing (ft.)
Note: L_p = L in Figure 4d.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_2 := \frac{16 \cdot M_p + N_2^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_2 \cdot L_p - L_t} = 53.964 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode (kip)
- Eqn. A13.3.2-2

(4e) Ultimate Resistance (Nominal Resistance) of the Railing for a Three-Span Failure Mode: R_3

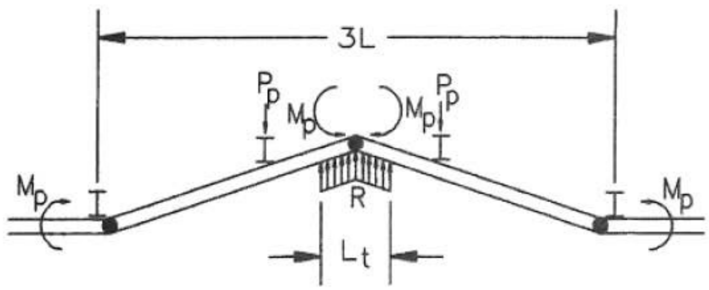


Figure 4e. Three-Span Failure Mode for Post-and-Beam Railings

$P_p = 33.876 \cdot kip$	Post Strength (kip)
$N_3 := 3$	Number of Failure Spans
$M_p = 36.728 \cdot kip \cdot ft$	Flexural Capacity of all Rails (kip-ft)
$L_p = 10 \cdot ft$	Post Spacing (ft.) Note: L_p = L in Figure 4e.
$L_t = 4 \cdot ft$	Longitudinal Length of Distribution of the Impact Force
$R_3 := \dfrac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_p}{2 \cdot N_3 \cdot L_p - L_t} = 58.888 \cdot kip$	Ultimate Resistance of the Railing for a Three-Span Failure Mode (kip) - Eqn. A13.3.2-1

(4f) Ultimate Resistance (Nominal Resistance) of the Railing for a 4-8 Span Failure Mode: R_4 - R_8

$P_p = 33.876 \cdot kip$ $L_p = 10\,ft$ $M_p = 36.728 \cdot kip \cdot ft$ $L_t = 4\,ft$

$N_4 := 4$ $N_5 := 5$ $N_6 := 6$ $N_7 := 7$ $N_8 := 8$

$$R_4 := \frac{16 \cdot M_p + N_4^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_4 \cdot L_p - L_t} = 79.05 \cdot kip$$

Ultimate Resistance of the Railing for a Four-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_p}{2 \cdot N_5 \cdot L_p - L_t} = 90.812 \cdot kip$$

Ultimate Resistance of the Railing for a Five-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_6 := \frac{16 \cdot M_p + N_6^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_6 \cdot L_p - L_t} = 110.199 \cdot kip$$

Ultimate Resistance of the Railing for a Six-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_7 := \frac{16 \cdot M_p + (N_7 - 1) \cdot (N_7 + 1) \cdot P_p \cdot L_p}{2 \cdot N_7 \cdot L_p - L_t} = 123.884 \cdot kip$$

Ultimate Resistance of the Railing for a Seven-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_8 := \frac{16 \cdot M_p + N_8^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_8 \cdot L_p - L_t} = 142.746 \cdot kip$$

Ultimate Resistance of the Railing for a Eight-Span Failure Mode (kip)
- Eqn. A13.3.2-2

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$R_r := \min(R_1, R_2, R_3, R_4, R_5, R_6, R_7, R_8) = 36.728 \cdot kip$

Total Ultimate Resistance of the railing @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_1 - R_8)

$H_e = 19 \cdot in$

Height of Equivalent Transverse Load

$y_{bar} = 21 \cdot in$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$F_t = 71 \cdot kip$

Transverse Impact Force

$R_R := R_r \cdot \left(\frac{y_{bar}}{H_e}\right) = 40.594 \cdot kip$

Total Ultimate Resistance of the railing @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } R_R \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_Check = "NOT OK"

(5) Strength Analysis of the Barrier at an End Section or Joint:

(5a) Ultimate Resistance of the Railing at an End Section or Joint for a Single-Span Failure Mode: R_{1end}

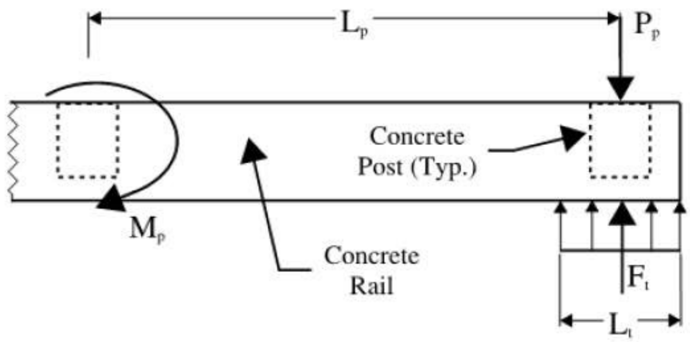


Figure 5a. Single-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10 \, ft$ — Post Spacing (ft)

$$R_{1end} := P_p + \frac{M_p}{L_p} = 37.549 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

<u>Using the Approach from AASHTO Section 13:</u> $N_1 = 1$

$$R_{1A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_1} i\right)}{2N_1 \cdot L_p - L_t} = 46.936 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r1.end} := \min\left(R_{1end}, R_{1A.end}\right) = 37.549 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

(5b) Ultimate Resistance of the Railing at an End Section or Joint for a Two-Span Failure Mode: R_{2end}

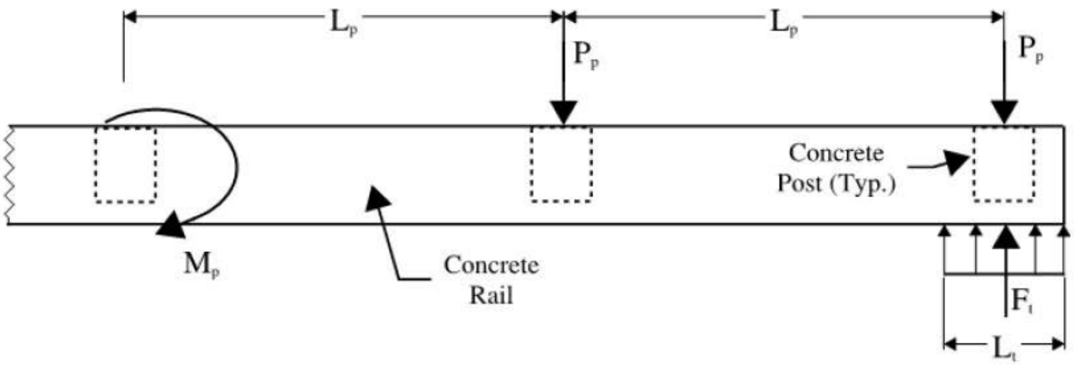


Figure 5b. Two-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 33.876 \cdot kip$ — Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10 ft$ — Post Spacing (ft.)

$$R_{2end} := \frac{P_p \cdot 3 L_p + M_p}{2 L_p} = 52.651 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

Using AASHTO Section 13: $N_2 = 2$

$$R_{2A.end} := \frac{2 \cdot M_p + 2 P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_2} i \right)}{2 N_2 \cdot L_p - L_t} = 58.501 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r2.end} := \min\left(R_{2end}, R_{2A.end}\right) = 52.651 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

(5) Strength Analysis of the Barrier at an End Section or Joint-Summary of Results:

$R_{r1.end} = 37.549 \cdot kip$

Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

$R_{r2.end} = 52.651 \cdot kip$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

$R_{rend} := \min\left(R_{r1.end}, R_{r2.end}\right) = 37.549 \cdot kip$

Total Ultimate Resistance of the railing at an End Section or Joint @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_{r1end} & R_{r2end})

$y_{bar} = 21 \cdot in$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$H_e = 19 \cdot in$

Height of Equivalent Transverse Load

$F_t = 71 \cdot kip$

Transverse Impact Force

$R_{Rend} := R_{rend} \cdot \left(\frac{y_{bar}}{H_e}\right) = 41.501 \cdot kip$

Total Ultimate Resistance of the railing at an End Section or Joint @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{Rend} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

(7) **Conclusions:**

Minimum_Height_of_Barrier_Check = "NOT OKAY"

Post_Setback_Criteria_Check = "NOT OK"

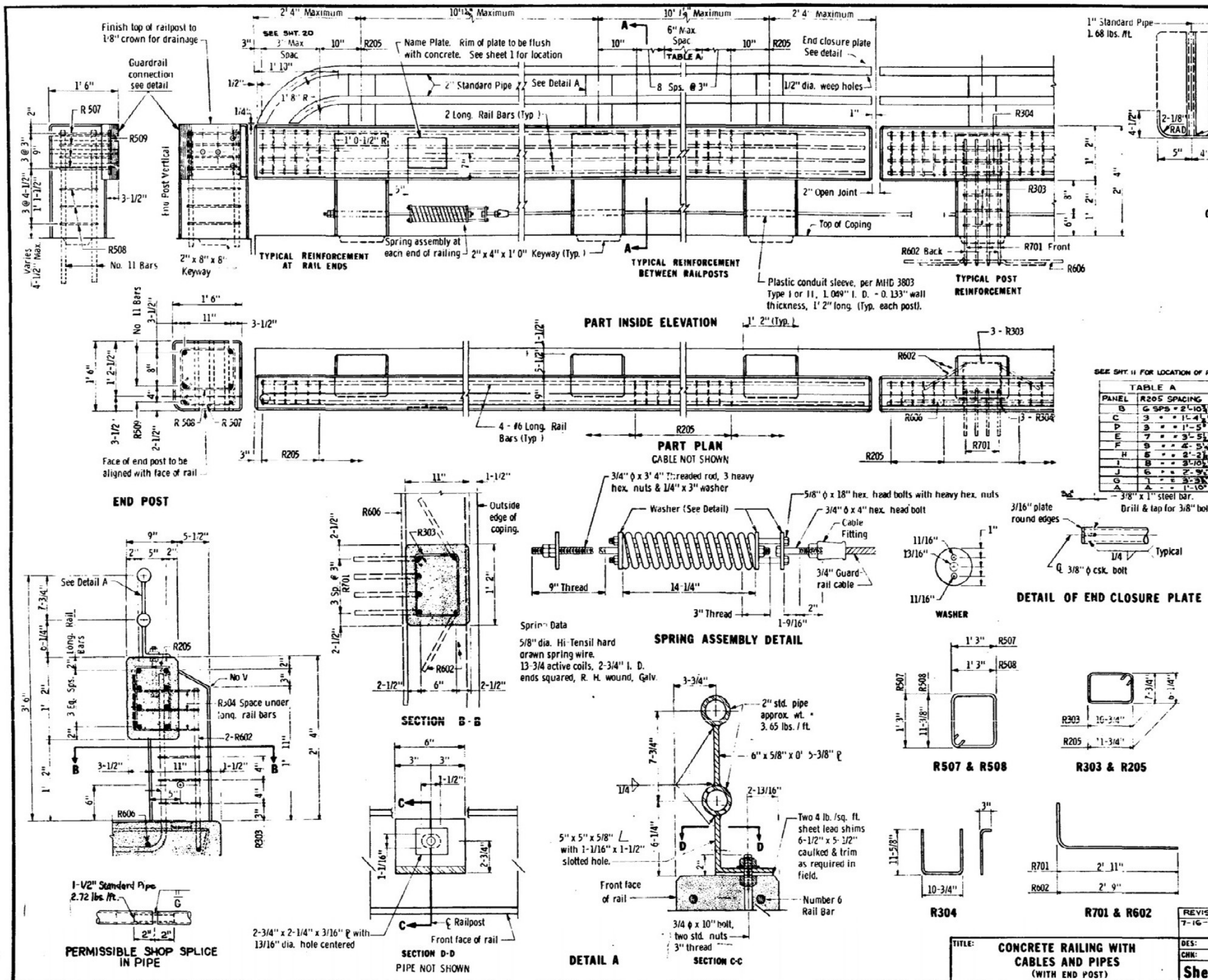
Snag_Potential_Criteria_Check = "NOT OK"

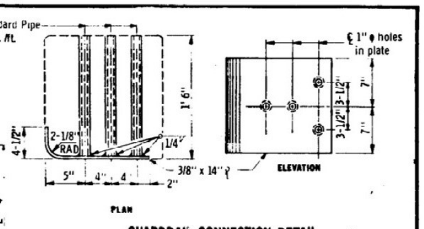
Structural_Capacity_of_Barrier_Check = "NOT OK"

Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

The One Line barrier on Bridge No. 69834 does not satisfy all MASH TL-3 Criteria

APPENDIX C5: ONE-LINE BRIDGE RAIL ON FIGURE 5-397.104





GUARDRAIL CONNECTION DETAIL
GALVANIZE AFTER FABRICATION PER M. H. D. 3394
Est. Wt. 37 Lbs.

SEE SHT. 11 FOR LOCATION OF PANELS

TABLE A	
PANEL	R205 SPACING
B	6 SPS @ 2'-10¾"
C	3 " " 1'-4½"
D	3 " " 1'-5"
E	7 " " 3'-5½"
F	9 " " 4'-5¼"
H	5 " " 2'-2⅛"
I	8 " " 3'-10½"
J	6 " " 2'-9½"
G	7 " " 3'-3½"
A	4 " " 1'-10"

BILL OF REINFORCEMENT FOR RAILING

MARK	NO	LENGTH	SHAPE	LOCATION
R701	4?8	3' 11"	Bent	Railpost
R602	204	3' 9"	"	"
R303	306	3' 9"	"	"
R304	306	3' 4"	"	"
R205	1878	3' 6"	"	Railing
R606	102	3' 6"	Str.	Curb
R507	16	6' 0"	Bent	End Post
R508	12	5' 4"	Bent	End Post
R509	8	0' 11"	Str.	End Post
R610	8	19'-3"	"	PANEL A LONG.
R611	8	20'-11"	"	" B "
R612	96	17'-11"	"	" C,D "
R613	112	22'-1"	"	" E "
R614	8	24'-8"	"	" F "
R615	8	22'-4"	"	" G "
R616	8	19'-8"	"	" H "
R617	16	22'-11"	"	" I "
R618	8	21'-2"	"	" J "

See special provisions for railing ties alternate.

GENERAL NOTES

All railposts shall be normal to grade, except as noted.

Finish all edges of concrete to a 1/2" V unless otherwise noted.

Special care shall be exercised in finishing the surface of the concrete slab in the area of the posts so that the vertical reinforcement for the posts can be properly placed with respect to depth of anchorage.

Place the No. 5 end post ties when the No. 11 vertical bars are cast in the abutment.

No. 2 reinforcement bars shall comply with A. S. T. M. A306.

Pipe shall comply with M. H. D. 3362, Grade A or B, or M. H. D. 3306. All other structural material except springs and cable fittings shall be structural steel per M. H. D. 3306. Cable and fittings shall comply with M. H. D. 2554, M. H. D. 3381, except as noted.

Galvanize bolts, rods, nuts, spring assemblies and cable fittings per M. H. D. 3392. Galvanize all structural material per M. H. D. 3394 after fabrication.

Guard rail connection to be included in weight of structural steel M. H. D. 3306.

Railing quantities are included in Summary of Quantities for Superstructure.

Length of "Ornamental Metal Railing" for payment will be measured end to end of concrete rail, with no deduction for open joints. Cable shall be accurately cut so there will be no sag before tightening spring nuts.

Price bid for ornamental metal railing includes the cable, spring assemblies, plastic sleeves, 3/4" ϕ x 10" bolts, and all material above the concrete rail.

All concrete shall be Concrete Mix No. 3Y46A.

REVISION		
7-16-71	END PANELS & REF. REVISED	R.P.C.

Fig. 5-397.104
August 6, 1970

TITLE: **CONCRETE RAILING WITH CABLES AND PIPES** (WITH END POST)

DES: | DR: | APPROVED: 11-2-70
CHK: | CHK: | REVISED 7-16-71

Sheet No. 18R of 26 **Sheets**

Bridge No. 43004

SUBJECT: **MnDOT One-Line**
Figure 5-397.104
MASH Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$ — Compressive Strength of Concrete (psi)

$f_y := 40ksi$ — Yield Strength of Concrete Reinforcing Steel, (ksi)

$E_s := 29000ksi$ — Modulus of Elasticity of Steel (ksi)

$H_w := 28in$ — Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$t_o := 0in$ — Thickness of overlay (in.)

$h_w := H_w + t_o = 28 \cdot in$ — Total height of the barrier (in.)

(1b) Concrete Rail Inputs:

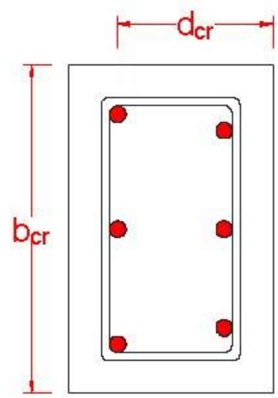


Figure 1b. Profile View showing Concrete Rail Inputs

$b_{cr} := 14in$ — Width of the Concrete Rail (in.)

$A_{cr} := 1.76in^2$ — Total area of the reinforcement bars acting in tension in the Concrete Rail (in2)

$d_{cr} := 7in$ — Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)

$y_{cr} := 21in$ — Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)

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SUBJECT: **MnDOT One-Line**
Figure 5-397.104
MASH Compliance Assessment

(1c) <u>Concrete Post Inputs:</u>

$b_p := 14in$	Width of Concrete Post (in.)
$A_p := 2.4in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p := 8.5in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$h_{curb} := 0in$	Height of curb (in.)
$L_p := 10ft + 1in = 10.083\,ft$	Spacing of Concrete Posts (ft.)

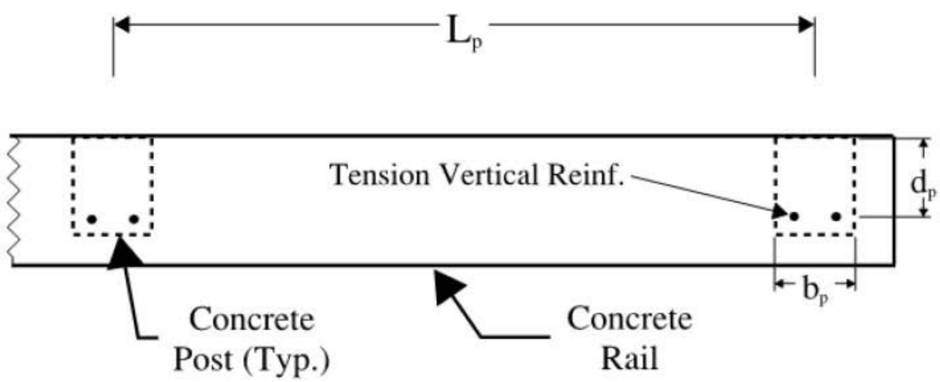


Figure 1c. Plan View of a Concrete Post-and-Beam Railing showing Concrete Post Inputs

SUBJECT: **MnDOT One-Line**
Figure 5-397.104
MASH Compliance Assessment

(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71kip$ Transverse Impact Force

$L_t := 4ft$ Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ Height of Equivalent Transverse Load above pavement surface (in)

$h_e := H_e + t_o = 19 \cdot in$ Total height of equivalent transverse load (in).

$H_{min} := 29in$ Minimum height of a MASH TL-2 barrier (in.)

$H_w = 28 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

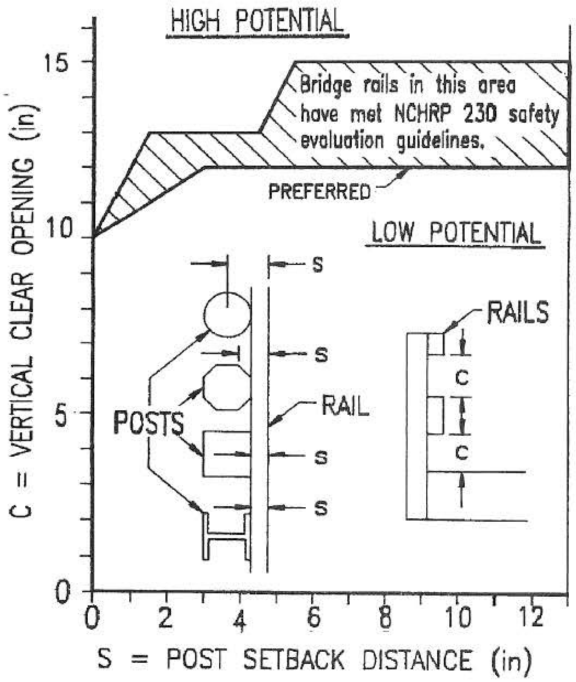
$H_w = 28 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/overlay to the top of the highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{vmatrix} \text{"OK"} \quad \text{if} \quad H_w \geq H_{min} \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{vmatrix}$$

Minimum_Height_of_Barrier_Check = "NOT OKAY"

(3) Geometric Criteria:

$S_{post} := 3.5in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 14in$	Vertical Clear Opening (in.)
$\Sigma A := 14in$	Total Rail Contact Width (in.)
$H_w = 28 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.



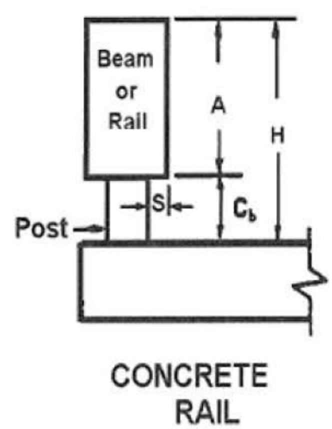


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

(3-conti.) Geometric Criteria:

$S_{post} = 3.5 \cdot in$ $\Sigma A = 14 \cdot in$ $H_w = 28 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_w} = 0.5$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

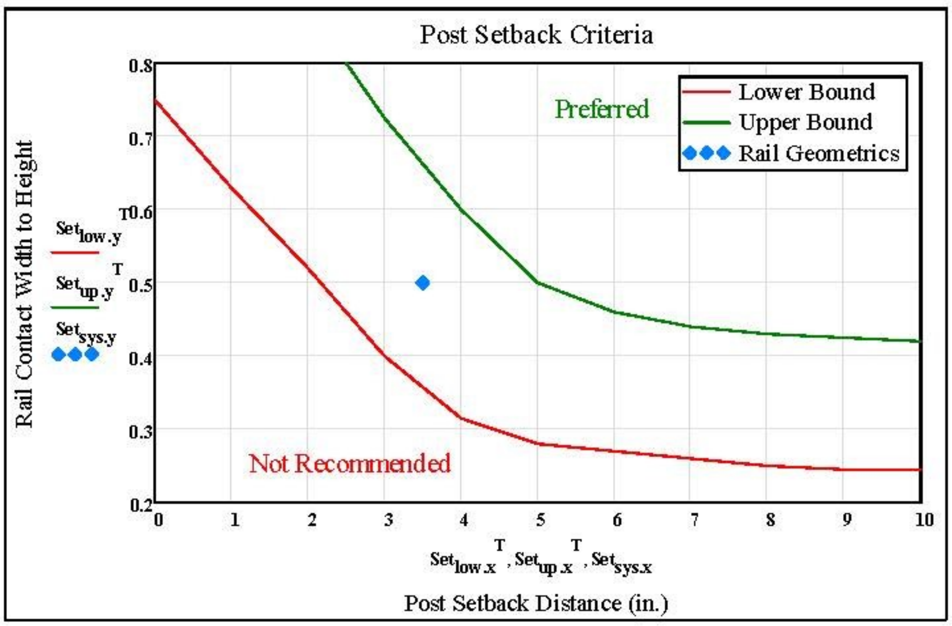
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 3.5$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.5$ Ratio of Contact Width to Total Height rail geometric point



$NotRecommended := 1$ $Marginal := 2$ $Preferred := 3$

Region Designation
Note: Marginal region is between Lower and Upper Bounds

$Post\_Setback\_Criteria\_Rail\_Geometric\_Point := Marginal$

SUBJECT: **MnDOT One-Line**
Figure 5-397.104
MASH Compliance Assessment

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0\ \ 3\ \ 13)$

$snag_{low.y} := (10\ \ 12\ \ 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0\ \ 1.25\ \ 4.25\ \ 5.25\ \ 13)$

$snag_{up.y} := (10\ \ 13\ \ 13\ \ 15\ \ 15)$

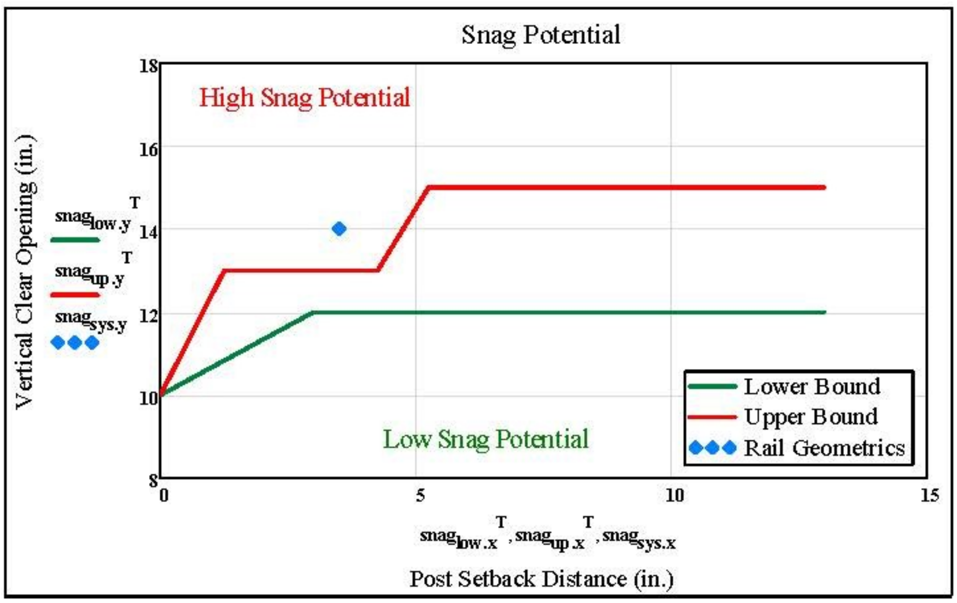
Upper Boundary for Snag Potential Criteria
x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 3.5$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 14$

Vertical Clear Opening rail geometric point



$HighSnagPotential := 1$ $\quad Marginal = 2$ $\quad LowSnagPotential := 3$

Region Designation
Note: Marginal region is between Lower and Upper Bounds

$Snag\_Potential\_Criteria\_Rail\_Geometric\_Point := HighSnagPotential$

(3) Geometric Criteria - Summary of Results:

Post_Setback_Criteria_Check := | "OK" if Post_Setback_Criteria_Rail_Geometric_Point = Preferred
| "NOT OK" otherwise

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check := | "OK" if Snag_Potential_Criteria_Rail_Geometric_Point = LowSnagPotential
| "NOT OK" otherwise

Snag_Potential_Criteria_Check = "NOT OK"

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Flexural Capacity of the Concrete Rail: M_{cr}

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_{cr} = 14 \cdot in$	Width of the Concrete Rail (in.)
$A_{cr} = 1.76 \cdot in^2$	Total area of the reinforcement bars acting in tension in the Concrete Rail (in^2)
$d_{cr} = 7 \cdot in$	Distance from the compression face of the Concrete Rail to the tension reinforcement bars (in.)
$a_{cr} := \frac{A_{cr} \cdot f_y}{0.85 f'_c \cdot b_{cr}} = 1.479 \cdot in$	Whitney Stress Block Depth (in.)
$M_{cr} := A_{cr} \cdot f_y \cdot \left(d_{cr} - \frac{a_{cr}}{2}\right) = 36.728 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Rail (kip-ft)
$y_{cr} = 21 \cdot in$	Height of the Concrete Rail measured from the top of the roadway surface/overlay to the centroid of the rail (in.)
$y_{bar} := y_{cr} = 21 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.) Note: $y_{bar} = y_{cr}$, since the only rail is the concrete rail

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Figure 5-397.104
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(4b) <u>Post Strength:</u> P_p

$f_y = 40 \cdot ksi$	Yield Strength of Concrete Reinforcing Steel (ksi)
$f'_c = 4 \cdot ksi$	Concrete Compressive Strength
$b_p = 14 \cdot in$	Width of Concrete Post (in.)
$A_p = 2.4 \cdot in^2$	Area of Concrete Post reinforcement acting in tension (in^2)
$d_p = 8.5 \cdot in$	Distance from the compression face of the Concrete Post to the tension reinforcement bars (in.)
$a_p := \dfrac{A_p \cdot f_y}{0.85 \cdot f'_c \cdot b_p} = 2.017 \cdot in$	Whitney Stress Block Depth (in.)
$M_{post} := A_p \cdot f_y \cdot \left(d_p - \dfrac{a_p}{2}\right) = 59.933 \cdot kip \cdot ft$	Flexural Capacity of the Concrete Post (kip-ft)
$y_{bar} = 21 \cdot in$	Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)
$h_{curb} = 0 \cdot in$	Height of curb (in.)
$y_p := y_{bar} - h_{curb} = 21 \cdot in$	Height measured from the bottom of the Concrete Post to the Resultant Force of all Rails (in.)
$P_p := \dfrac{M_{post}}{y_p} = 34.247 \cdot kip$	Post Strength (kip)

(4c) <u>Ultimate Resistance (Nominal Resistance) of the Railing for a Single-Span Failure Mode</u>: R_1

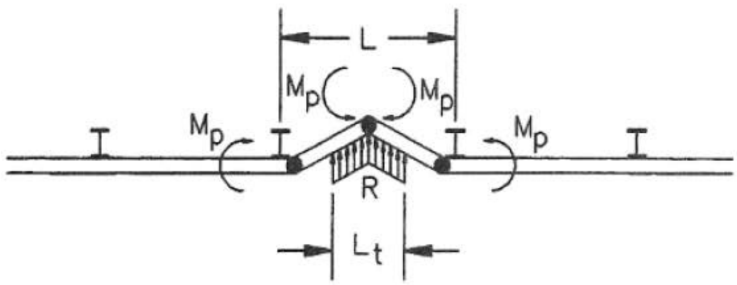

Figure 4c. Single-Span Failure Mode for Post-and-Beam Railings

$P_p = 34.247 \cdot kip$ — Post Strength (kip)

$N_1 := 1$ — Number of Failure Spans

$M_p := M_{cr} = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)
<u>Note</u>: $M_p = M_{cr}$, since the only rail is the concrete rail

$L_p = 10.083 \cdot ft$ — Post Spacing (ft.)
<u>Note</u>: $L_p = L$ in Figure 4c.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_p}{2 \cdot N_1 \cdot L_p - L_t} = 36.35 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode (kip)
- Eqn. A13.3.2-1

(4d) Ultimate Resistance (Nominal Resistance) of the Railing for a Two-Span Failure Mode: R_2

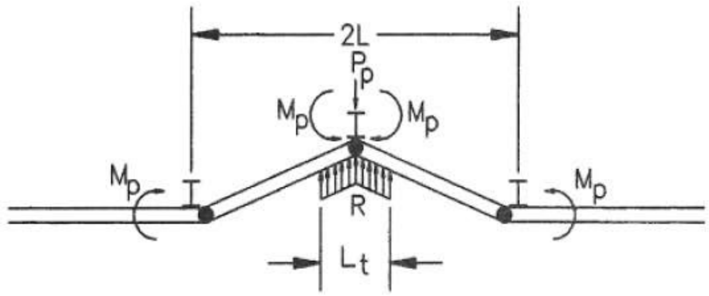

Figure 4d. Two-Span Failure Mode for Post-and-Beam Railings

$P_p = 34.247 \cdot kip$ — Post Strength (kip)

$N_2 := 2$ — Number of Failure Spans

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10.083 \cdot ft$ — Post Spacing (ft.)
Note: L_p = L in Figure 4d.

$L_t = 4 \cdot ft$ — Longitudinal Length of Distribution of the Impact Force

$$R_2 := \frac{16 \cdot M_p + N_2^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_2 \cdot L_p - L_t} = 54.192 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode (kip)
- Eqn. A13.3.2-2

(4e) Ultimate Resistance (Nominal Resistance) of the Railing for a Three-Span Failure Mode: R_3

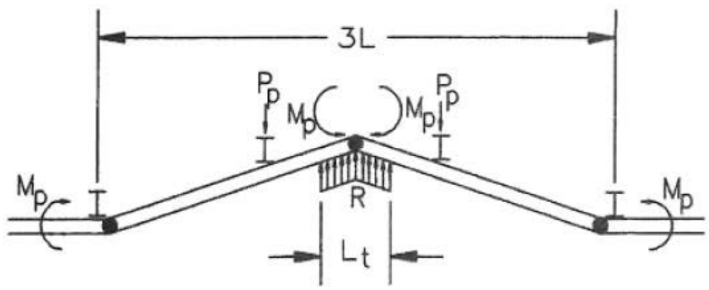

Figure 4e. Three-Span Failure Mode for Post-and-Beam Railings

$P_p = 34.247 \cdot \mathbf{kip}$	Post Strength (kip)
$N_3 := 3$	Number of Failure Spans
$M_p = 36.728 \cdot \mathbf{kip} \cdot \mathbf{ft}$	Flexural Capacity of all Rails (kip-ft)
$L_p = 10.083 \cdot \mathbf{ft}$	Post Spacing (ft.) Note: L_p = L in Figure 4e.
$L_t = 4 \cdot \mathbf{ft}$	Longitudinal Length of Distribution of the Impact Force
$R_3 := \dfrac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_p}{2 \cdot N_3 \cdot L_p - L_t} = 59.297 \cdot \mathbf{kip}$	Ultimate Resistance of the Railing for a Three-Span Failure Mode (kip) - Eqn. A13.3.2-1

(4f) Ultimate Resistance (Nominal Resistance) of the Railing for a 4-8 Span Failure Mode: R_4 - R_8

$P_p = 34.247 \cdot kip$ $\quad L_p = 10.083\,ft$ $\quad M_p = 36.728 \cdot kip \cdot ft$ $\quad L_t = 4\,ft$

$N_4 := 4$ $\quad N_5 := 5$ $\quad N_6 := 6$ $\quad N_7 := 7$ $\quad N_8 := 8$

$$R_4 := \frac{16 \cdot M_p + N_4^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_4 \cdot L_p - L_t} = 79.733 \cdot kip$$

Ultimate Resistance of the Railing for a Four-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_p}{2 \cdot N_5 \cdot L_p - L_t} = 91.657 \cdot kip$$

Ultimate Resistance of the Railing for a Five-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_6 := \frac{16 \cdot M_p + N_6^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_6 \cdot L_p - L_t} = 111.277 \cdot kip$$

Ultimate Resistance of the Railing for a Six-Span Failure Mode (kip)
- Eqn. A13.3.2-2

$$R_7 := \frac{16 \cdot M_p + (N_7 - 1) \cdot (N_7 + 1) \cdot P_p \cdot L_p}{2 \cdot N_7 \cdot L_p - L_t} = 125.128 \cdot kip$$

Ultimate Resistance of the Railing for a Seven-Span Failure Mode (kip)
- Eqn. A13.3.2-1

$$R_8 := \frac{16 \cdot M_p + N_8^{\,2} \cdot P_p \cdot L_p}{2 \cdot N_8 \cdot L_p - L_t} = 144.207 \cdot kip$$

Ultimate Resistance of the Railing for a Eight-Span Failure Mode (kip)
- Eqn. A13.3.2-2

SUBJECT: **MnDOT One-Line**
Figure 5-397.104
MASH Compliance Assessment

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$R_r := \min(R_1, R_2, R_3, R_4, R_5, R_6, R_7, R_8) = 36.35 \cdot kip$

Total Ultimate Resistance of the railing @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_1 - R_8)

$H_e = 19 \cdot in$

Height of Equivalent Transverse Load

$h_e = 19 \cdot in$

Total height of equivalent transverse load (top of concrete deck)

$y_{bar} = 21 \cdot in$

Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$F_t = 71 \cdot kip$

Transverse Impact Force

$$R_R := R_r \cdot \left(\frac{y_{bar} + t_o}{h_e} \right) = 40.176 \cdot kip$$

Total Ultimate Resistance of the railing @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_Check} := \begin{array}{|l} \text{"OK"} \quad \text{if} \quad R_R \ge F_t \\ \text{"NOT OK"} \quad \text{otherwise} \end{array}$$

Structural_Capacity_of_Barrier_Check = "NOT OK"

(5) Strength Analysis of the Barrier at an End Section or Joint:

(5a) Ultimate Resistance of the Railing at an End Section or Joint for a Single-Span Failure Mode: R_{1end}

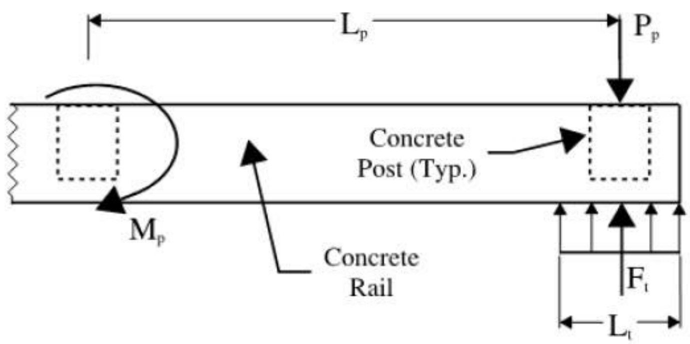

Figure 5a. Single-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 34.247 \cdot kip$ — Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ — Flexural Capacity of all Rails (kip-ft)

$L_p = 10.083\,ft$ — Post Spacing (ft.)

$$R_{1end} := P_p + \frac{M_p}{L_p} = 37.89 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

Using the Approach from AASHTO Section 13:

$N_1 = 1$

$$R_{1A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_1} i\right)}{2N_1 \cdot L_p - L_t} = 47.265 \cdot kip$$

Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r1.end} := \min\left(R_{1end}, R_{1A.end}\right) = 37.89 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

(5b) Ultimate Resistance of the Railing at an End Section or Joint for a Two-Span Failure Mode: R_{2end}

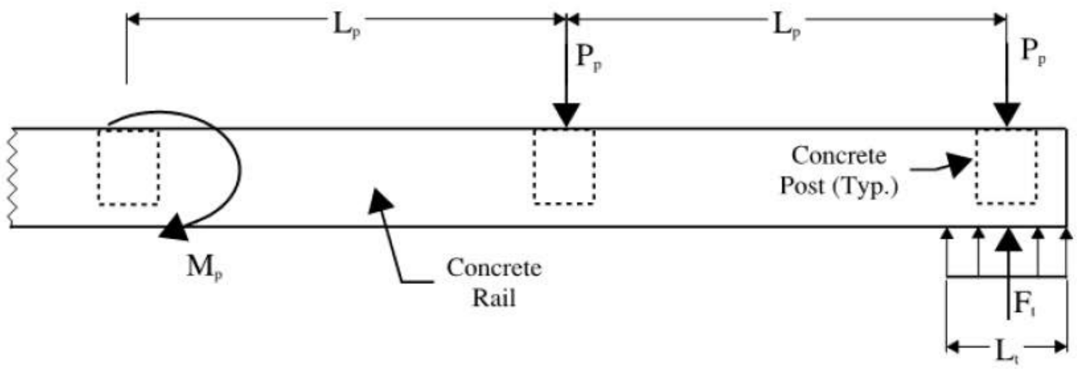

Figure 5b. Two-Span Failure Mode for Post-and-Beam Railings at an End Section or Joint

$P_p = 34.247 \cdot kip$ Post Strength (kip)

$M_p = 36.728 \cdot kip \cdot ft$ Flexural Capacity of all Rails (kip-ft)

$L_p = 10.083\,ft$ Post Spacing (ft)

$$R_{2end} := \frac{P_p \cdot 3L_p + M_p}{2L_p} = 53.192 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

Using AASHTO Section 13: $N_2 = 2$

$$R_{2A.end} := \frac{2 \cdot M_p + 2P_p \cdot L_p \cdot \left(\sum_{i=1}^{N_2} i\right)}{2N_2 \cdot L_p - L_t} = 59.048 \cdot kip$$

Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint using AASHTO Eqn. A13.3.2-3 (kip)

$$R_{r2.end} := \min\left(R_{2end}, R_{2A.end}\right) = 53.192 \cdot kip$$

Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

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SUBJECT: **MnDOT One-Line**
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(5) Strength Analysis of the Barrier at an End Section or Joint-Summary of Results:

$R_{r1.end} = 37.89 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Single-Span Failure Mode at an End Section or Joint (kip)

$R_{r2.end} = 53.192 \cdot kip$ — Critical Ultimate Resistance of the Railing for a Two-Span Failure Mode at an End Section or Joint (kip)

$R_{rend} := \min\left(R_{r1.end}, R_{r2.end}\right) = 37.89 \cdot kip$ — Total Ultimate Resistance of the railing at an End Section or Joint @ y_{bar} (kip)
Note: The total Ultimate Resistance of the railing is the critical span failure mode (i.e., the minimum value of R_{r1end} & R_{r2end})

$y_{bar} = 21 \cdot in$ — Height of the Resultant Force of all Rails measured from the top of the roadway surface/overlay (in.)

$H_e = 19 \cdot in$ — Height of Equivalent Transverse Load

$F_t = 71 \cdot kip$ — Transverse Impact Force

$R_{Rend} := R_{rend} \cdot \left(\frac{y_{bar}}{H_e}\right) = 41.878 \cdot kip$ — Total Ultimate Resistance of the railing at an End Section or Joint @ H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{Rend} \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_Barrier\_at\_End\_Section\_Check} = \text{"NOT OK"}$$

SUBJECT: MnDOT One-Line
Figure 5-397.104
MASH Compliance Assessment

(6) Strength Analysis of the Seperate End Post:

(6a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

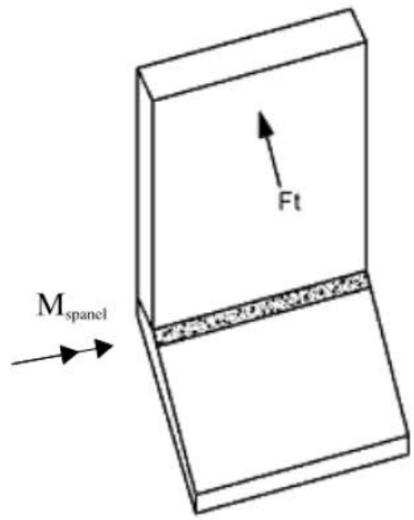

Figure 6a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 29in$	Height of the end post measured from the top of the roadway/surface (in.)
$b_{spost} := 18in$	Width of the end post (in.)
$A_{1.spost} := 1.56in^2$	Area of one vertical reinforcement leg in the tension zone of the end post (in^2)
$n_{spost} := 2$	Number of vertical reinforcement in the end post (in.)

(5a-conti.) <u>Bending Capacity of the End Post about the Longitudinal Axis</u>: $M_{s.post}$

$A_{spost} := n_{spost} \cdot A_{1.spost} = 3.12 \cdot in^2$

Total Area of vertical reinforment in the tension zone of the end post (in^2)

$a_{spost} := \dfrac{A_{spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 2.039 \cdot in$

Depth of the Whitney Stress Block (in.)

$d_{spost} := 11.5in$

Average extreme distance of tension vertical reinforcement in the end post (in.)

$M_{s.post} := A_{spost} \cdot f_y \cdot \left(d_{spost} - \dfrac{a_{spost}}{2} \right) = 108.996 \cdot kip \cdot ft$

Flexural Capacity of the End Post about the Longitudinal Axis when considering only the vertical reinforcment (kip-ft)

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Figure 5-397.104
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(6) Strength Analysis of the End Post-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$M_{s.post} = 108.996 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$$R_{spost} := \frac{M_{s.post}}{H_e} = 68.84 \cdot kip$$

Structural Capacity of the End Post

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \left(\begin{array}{l} \text{"OK"} \quad \text{if} \ R_{spost} \geq F_t \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"NOT OKAY"}$$

(7) **Conclusions:**

Minimum_Height_of_Barrier_Check = "NOT OKAY"

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check = "NOT OK"

Structural_Capacity_of_Barrier_Check = "NOT OK"

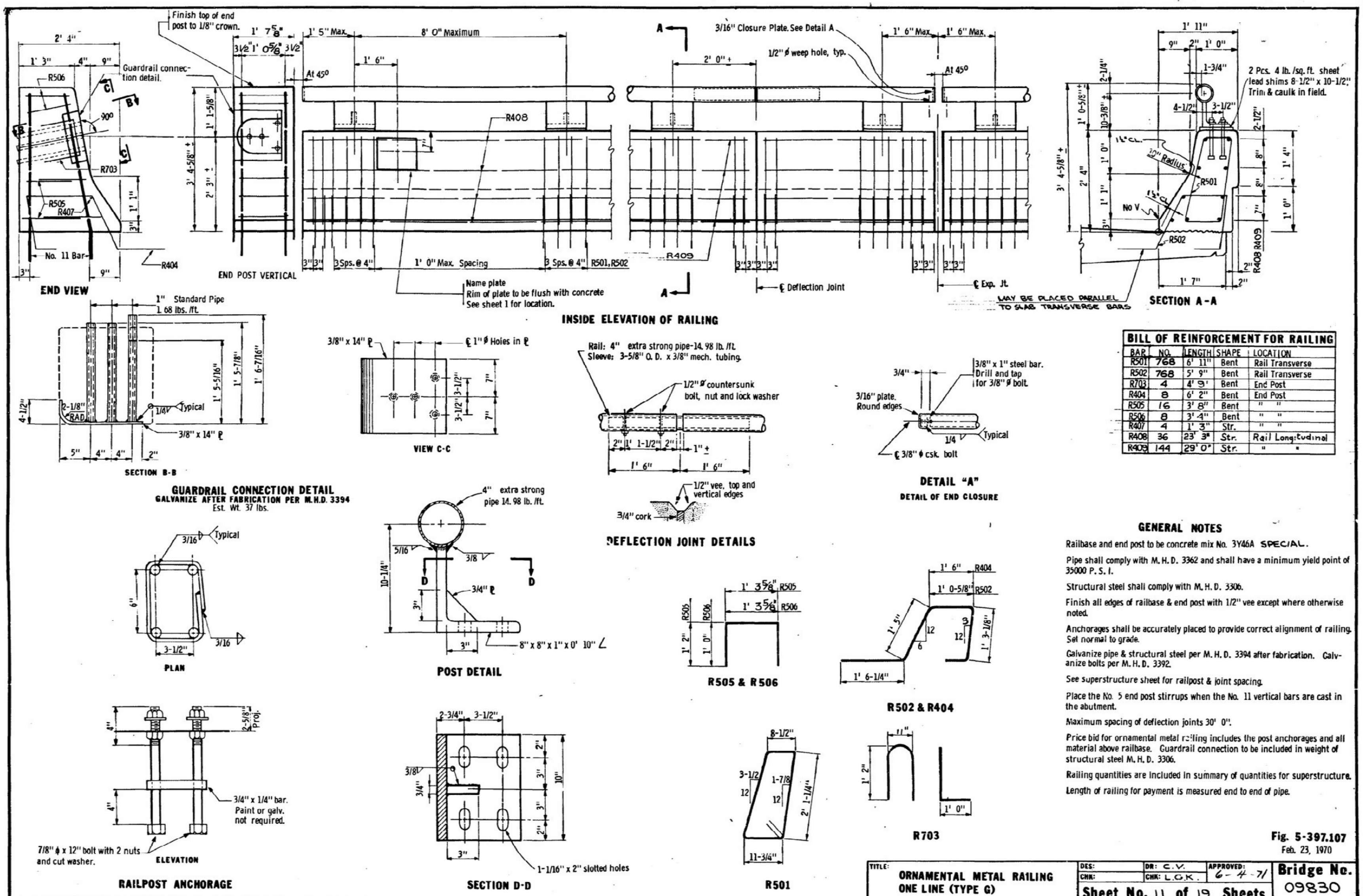
Structural_Capacity_of_Barrier_at_End_Section_Check = "NOT OK"

Structural_Capacity_of_End_Post_Check = "NOT OKAY"

The One Line barrier from Figure 5-397.104 does not satisfy all MASH TL-3 Criteria

APPENDIX D: G BARRIER ANALYSES

APPENDIX D1: G BARRIER ON BRIDGE NO. 09830 (FIGURE 5-397.107)



BILL OF REINFORCEMENT FOR RAILING

BAR	NO.	LENGTH	SHAPE	LOCATION
R501	768	6' 11"	Bent	Rail Transverse
R502	768	5' 9"	Bent	Rail Transverse
R703	4	4' 9'	Bent	End Post
R404	8	6' 2"	Bent	End Post
R505	16	3' 8"	Bent	" "
R506	8	3' 4"	Bent	" "
R407	4	1' 3"	Str.	" "
R408	36	23' 3"	Str.	Rail Longitudinal
R409	144	29' 0"	Str.	" "

GENERAL NOTES

Railbase and end post to be concrete mix No. 3Y46A SPECIAL.

Pipe shall comply with M.H.D. 3362 and shall have a minimum yield point of 35000 P.S.I.

Structural steel shall comply with M.H.D. 3306.

Finish all edges of railbase & end post with 1/2" vee except where otherwise noted.

Anchorages shall be accurately placed to provide correct alignment of railing. Set normal to grade.

Galvanize pipe & structural steel per M.H.D. 3394 after fabrication. Galvanize bolts per M.H.D. 3392.

See superstructure sheet for railpost & joint spacing.

Place the No. 5 end post stirrups when the No. 11 vertical bars are cast in the abutment.

Maximum spacing of deflection joints 30' 0".

Price bid for ornamental metal railing includes the post anchorages and all material above railbase. Guardrail connection to be included in weight of structural steel M.H.D. 3306.

Railing quantities are included in summary of quantities for superstructure.

Length of railing for payment is measured end to end of pipe.

Fig. 5-397.107
Feb. 23, 1970

TITLE: ORNAMENTAL METAL RAILING ONE LINE (TYPE G)	DES: / CHK:	DR: C.V. / CHK: L.G.K.	APPROVED: 6-4-71	Bridge No. 09830
	Sheet No. 11 of 19 Sheets			

SUBJECT: **MnDOT G-Barrier on Bridge No. 09830**
Figure 5-397.107
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH TL-3 Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 40ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$H_r := 40.625in$	Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)
$H_R := 38.375in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$t_o := 0in$	Thickness of overlay (in.)

(1b) Concrete Parapet Inputs:

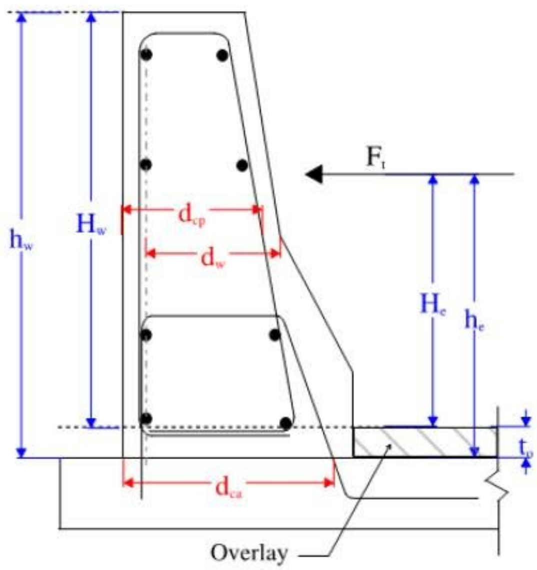


Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b-conti.) Concrete Parapet Inputs:

$H_w := 28in$ — Height of the concrete parapet/wall measured from the top of the roadway surface/overlay (in.)

$h_w := H_w + t_o = 28 \cdot in$ — Total height of the concrete parapet/wall (in.)

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{vp.mid} := 12in$ — Average Spacing of parapet vertical reinforcement at midspan (in.)

$d_{cp.mid} := 10.187in$ — Extreme distance of parapet vertical reinforcement in tension at midspan (in.)

$A_{vp1.end} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 9.6in$ — Average Spacing of parapet vertical reinforcement at joints/ends (in.)

$d_{cp.end} := 10.187in$ — Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$ — Area of longitudinal reinforcement bars in tension (in^2)

$d_w := 12in$ — Extreme distance of tension longitudinal reinforcement of wall (in.)

(1b-conti.) **Concrete Parapet Inputs:**

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502} := 10in$ — Projected length of R502E reinforcement over the slab (in.)

$L_{wid\_R502} := 12.625in$ — Outer width of R502E reinforcement (in.)

$Cover := 2in$ — Cover clear distance (in.)

$Ratio_{R502} := \frac{6}{12}$ — Inclined angle of R502E reinforcement

$d_{b\_R502} := 0.625in$ — Nominal diameter of R502E reinforcement (#5 bar)

$Coping := 2in$ — Coping on the back of the barrier

$$d_{ca} := L_{wid\_R502} + L_{proj\_R502} \cdot Ratio_{R502} + Cover - \frac{1}{2} d_{b\_R502} - Coping = 17.313 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ — Average Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 17.313 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

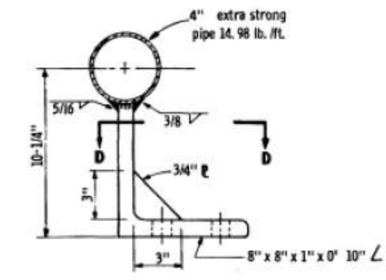
$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 9.6in$ — Average Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := d_{ca} = 17.313 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

SUBJECT: **MnDOT G-Barrier on Bridge No. 09830**
Figure 5-397.107
MASH TL-3 Compliance Assessment

(1b) Steel Rail, Post, and Anchor Rod Inputs:



POST DETAIL

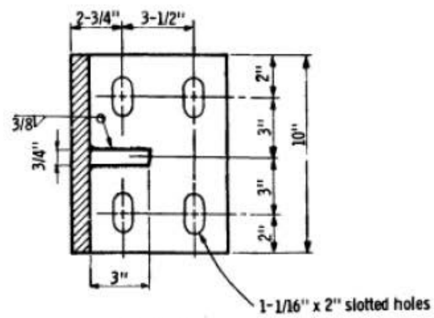
Steel Rail Inputs:

a) Steel Tube Rail is M.H.D. 3362 material, Fy=35ksi
b) Steel Tube Rail is a 4" extra strong pipe

$F_{yR} := 35ksi$	Yield Strength of Steel Tube Rail (ksi)
$d_{oR} := 4.5in$	Outside diameter of Steel Tube Rail (in.)
$d_{iR} := 3.83in$	Inside diameter of Steel Tube Rail (in.)

Steel Post Inputs:

a) Steel Post is M.H.D. 3306 material, Fy= 36ksi
b) Steel Post is a 8"x8"x1"x10" Angle Member

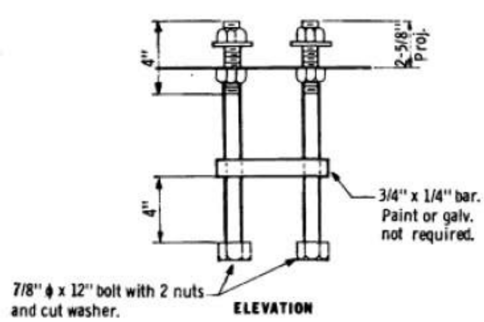


SECTION D-D

$F_{yp} := 36ksi$	Yield Strength of Steel Post (ksi)
$w_p := 10in$	Width of Steel Post about the bending axis (in.)
$t_p := 1in$	Thickness of Steel Post (in.)
$h_p := 9.25in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$L_p := 8ft$	Steel Post Spacing (ft.)

Anchor Rod Inputs:

a) Anchor Rods are F1554 Gr. 36 material, Fu=58 ksi
b) Anchor Rods are 7/8" ϕ x 12"



RAILPOST ANCHORAGE

$F_{u.rod} := 58ksi$	Tensile Strength of Anchor Rods (ksi)
$N_{rod} := 4$	Number of Anchor Rods
$N_{rod.tension} := 2$	Number of Anchor Rods in Tension
$d_{rod} := 5.25in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$\phi_{rod} := \frac{7}{8}in$	Diameter of Anchor Rods (in)

(1c) Design Force Inputs:

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ — Test Level

$F_t := 71kip$ — Transverse Impact Force

$L_t := 4ft$ — Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ — Height of Equivalent Transverse Load

$H_{min} := 29in$ — Minimum height of a MASH TL-3 barrier (in.)

$H_r = 40.625 \cdot in$ — Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_r = 40.625 \cdot in$ Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_r \geq H_{min} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

(4) Geometric Criteria:

$S_{post} := 1.75in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 10.375in$	Vertical Clear Opening (in.)
$\Sigma A := H_w + \frac{d_{oR}}{2} = 30.25 \cdot in$	Total Rail Contact Width (in.)
$H_r = 40.625 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.

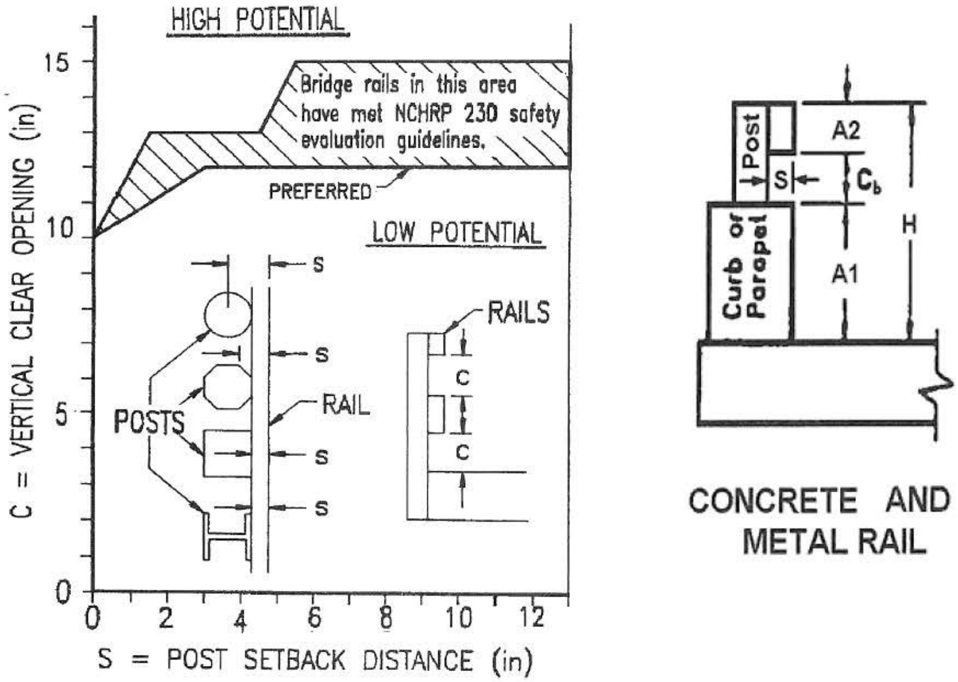


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

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(3-conti.) Geometric Criteria:

$S_{post} = 1.75 \cdot in$ $\Sigma A = 30.25 \cdot in$ $H_r = 40.625 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_r} = 0.745$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

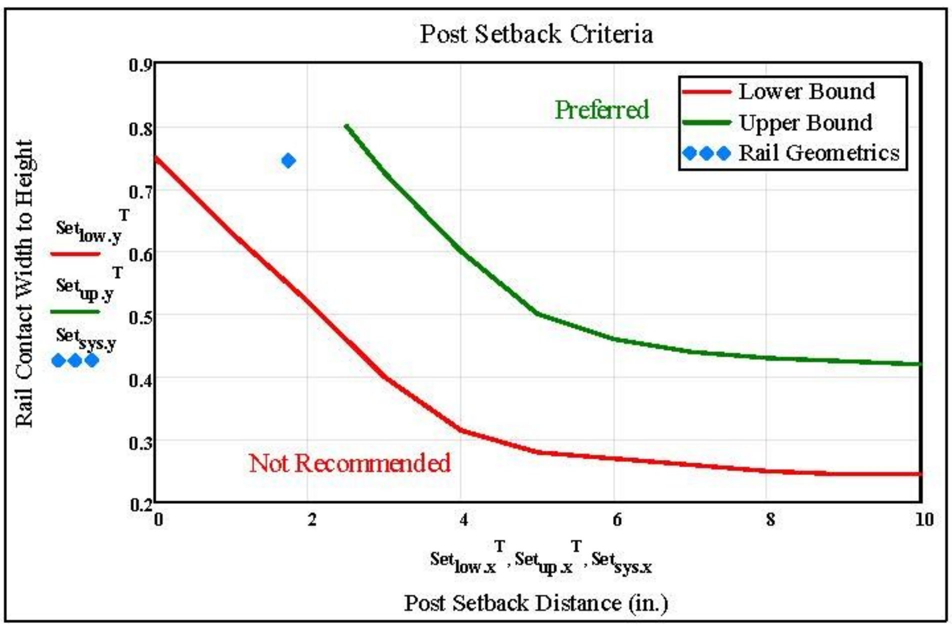
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 1.75$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.745$ Ratio of Contact Width to Total Height rail geometric point



$NotRecommended := 1$ $Marginal := 2$ $Preferred := 3$

Region Designation
Note: Marginal region is between Lower and Upper Bounds

$Post\_Setback\_Criteria\_Rail\_Geometric\_Point := Marginal$

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(3-conti.) Geometric Criteria:

$snag_{low.x} := (0 \quad 3 \quad 13)$

$snag_{low.y} := (10 \quad 12 \quad 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0 \quad 1.25 \quad 4.25 \quad 5.25 \quad 13)$

$snag_{up.y} := (10 \quad 13 \quad 13 \quad 15 \quad 15)$

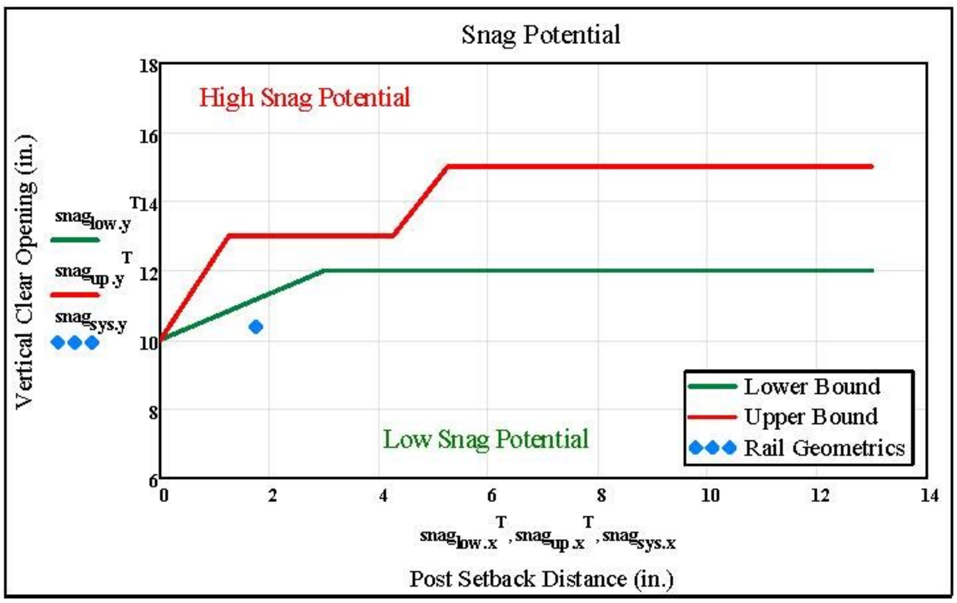
Upper Boundary for Snag Potential Criteria
x and y coordinates

$$snag_{sys.x} := \frac{S_{post}}{in} = 1.75$$

Post Setback rail geometric point

$$snag_{sys.y} := \frac{C_b}{in} = 10.375$$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1 Marginal = 2 LowSnagPotential := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := LowSnagPotential

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(3) Geometric Criteria - Summary of Results:

Post_Setback_Criteria_Check := | "OK" if Post_Setback_Criteria_Rail_Geometric_Point = Preferred
"NOT OK" otherwise

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check := | "OK" if Snag_Potential_Criteria_Rail_Geometric_Point = LowSnagPotential
"NOT OK" otherwise

Snag_Potential_Criteria_Check = "OK"

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$ — Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone (in2)

$s_{vp.mid} = 12 \cdot in$ — Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in2)

$d_{cp.mid} = 10.187 \cdot in$ — Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.304 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 10.37 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(4a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 17.313 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.304 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 17.733 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 10.37 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(4b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 9.6 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.388 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.38 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 10.187 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 12.913 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 9.6 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.388 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.38 \cdot in$ — Depth of Whitney Stress Block (in.)

(4b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 17.313 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 22.117 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 12.913 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(4c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 12 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 28 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.336 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 31.552 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

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(4d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

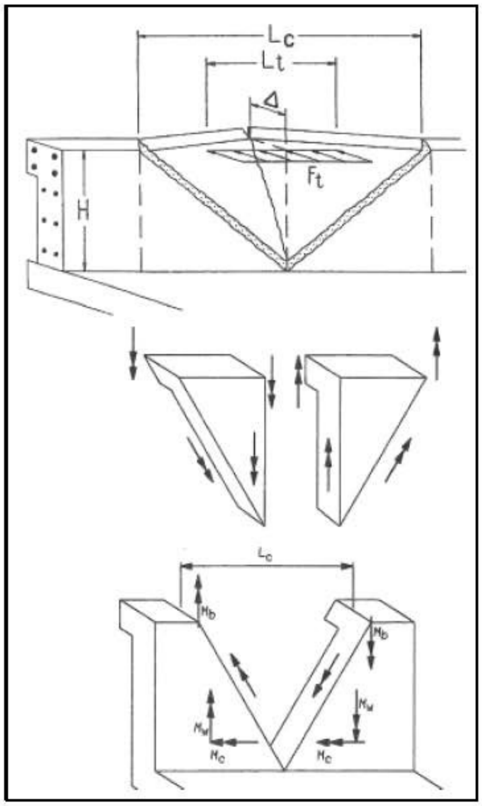

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 10.37 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 31.552 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.797 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 87.08 \cdot kip$$ (Equation A13.3.1-1)

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(4e) **Determine the Ultimate Resistance of the Wall at Joints/Ends:** R_{wend}

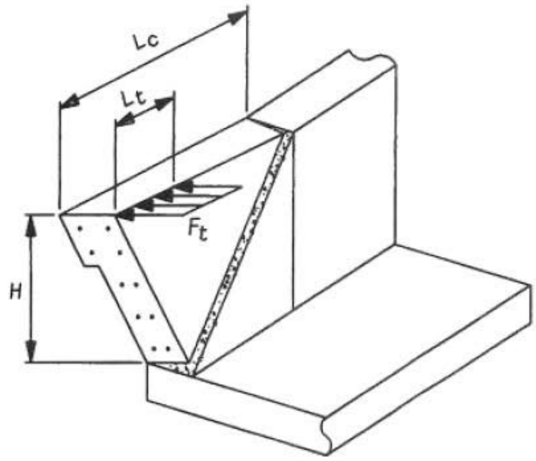

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)
Note: H_w = H in Figure 3e

$M_B = 0$ — No additional beam strength

$M_w = 31.552 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$M_{cend} = 12.913 \cdot \frac{kip \cdot ft}{ft}$ — Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 5.115 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\ 2}\right)}{h_w}\right] = 56.61 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

(4f) Steel Rail & Post Strength Analysis:

$F_{yR} = 35 \cdot ksi$ — Yield Strength of Steel Tube Rail (ksi)

$d_{oR} = 4.5 \cdot in$ — Outside diameter of Steel Tube Rail (in.)

$d_{iR} = 3.83 \cdot in$ — Inside diameter of Steel Tube Rail (in.)

$Z_R := \frac{\left(d_{oR}^3 - d_{iR}^3\right)}{6} = 5.824 \cdot in^3$ — Plastic Sectional Modulus of the Steel Tube Rail (in^3)

$M_p := F_{yR} \cdot Z_R = 16.986 \cdot kip \cdot ft$ — Plastic Moment Capacity of the Steel Tube Rail (kip-ft)

Calculate the Plastic Strength of the Post: P_{P1}

$w_p = 10 \cdot in$ — Width of Steel Post about the bending axis (in.)

$t_p = 1 \cdot in$ — Thickness of Steel Post (in.)

$Z_p := \frac{w_p \cdot t_p^2}{4} = 2.5 \cdot in^3$ — Plastic Sectional Modulus of the Steel Post about the bending axis (in.)

$F_{yp} = 36 \cdot ksi$ — Yield Strength of Steel Post (ksi)

$M_{post} := F_{yp} \cdot Z_p = 7.5 \cdot kip \cdot ft$ — Plastic Moment Capacity of the Steel Post (kip-ft)

$h_p = 9.25 \cdot in$ — Height from the bottom of the post to the centroid of the steel tube rail (in.)

$P_{P1} := \frac{M_{post}}{h_p} = 9.73 \cdot kip$ — Post Strength based on the Plastic Failure of a Steel Post (kip)

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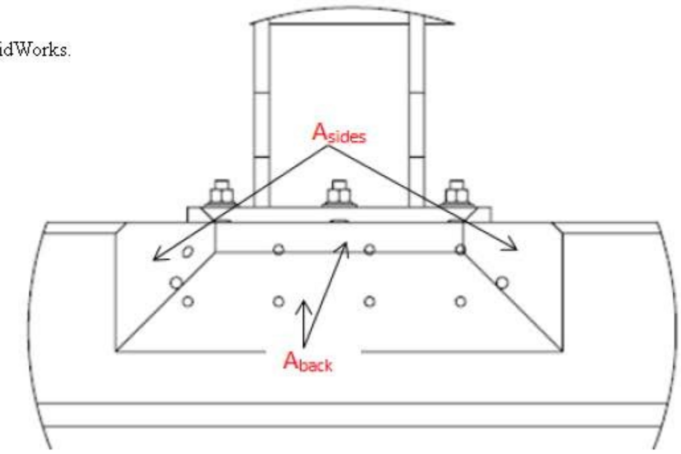
Calculate the Post Strength based on the Ultimate Strength of the Anchor Rods: P_{P2}

$F_{u.rod} = 58 \cdot ksi$	Tensile Strength of the Anchor Rods (ksi)
$\phi_{rod} = 0.875 \cdot in$	Diameter of Anchor Rods (in)
$A_{rod} := \frac{\pi}{4} \cdot \phi_{rod}{}^2 = 0.601 \cdot in^2$	Area of a Anchor Rod (in^2)
$R_{nt} := F_{u.rod} \cdot (0.75 \cdot A_{rod}) = 26.157 \cdot kip$	Nominal strength of one Anchor Rod in Tension (kip)
$N_{rod.tension} = 2$	Number of Anchor Rods acting in tension
$d_{rod} = 5.25 \cdot in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$d_b := 1.5in$	Length of the steel plate bearing pressure acting on the concrete parapet (in.)
$w_{rod} := d_{rod} - \frac{d_b}{3} = 4.75 \cdot in$	Distance from anchor rods acting in tension to the centroid of the bearing pressure acting on the concrete parapet (in.)
$M_{t.rod} := w_{rod} \cdot R_{nt} \cdot N_{rod.tension} = 20.708 \cdot kip \cdot ft$	Moment strength of Post based on tensile capacity of Anchor Rods (k-ft)
$h_p = 9.25 \cdot in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$P_{t.rod} := \frac{M_{t.rod}}{h_p} = 26.864 \cdot kip$	Post Strength based on the tensile capacity of Anchor Rods (kip)
$R_{nv} := F_{u.rod} \cdot (0.45 \cdot A_{rod}) = 15.694 \cdot kip$	Nominal strength of one anchor rod in Shear w/h threads in shear plane (kip)
$P_{v.rod} := N_{rod} \cdot R_{nv} = 62.778 \cdot kip$	Post Strength based on the shear capacity of Anchor Rods (kip)
$P_{P2} := \min(P_{t.rod}, P_{v.rod}) = 26.864 \cdot kip$	Post Strength based on the Ultimate Strength of the Anchor Rods (kip)

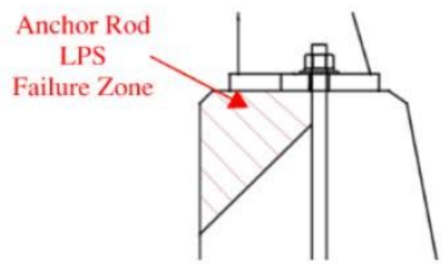
Calculate the Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods: P_{P3}

Note: This failure mechanism was modeled in SolidWorks.



$A_{sides} := 62 in^2$

$A_{back} := 130 in^2$



$\phi_v := 0.75$ — Shear Strength Reduction Factor

$A_{LPS} := A_{sides} + A_{back} = 0.124\ m^2$ — Total Area of Failure Planes due to Lateral Punching Shear Failure (in^2)

$f'_c = 4 \cdot ksi$ — Concrete Compressive Strength (ksi)

$V_{c.lat} := \phi_v \cdot 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi = 94.87 \cdot psi$ — Concrete Stress from Block Shear of Anchor Rods (psi)
-ACI 318-14 Eqn. 22.5.5.1

$P_{P3} := V_{c.lat} \cdot A_{LPS} = 18.215 \cdot kip$ — Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)

Determine the Limiting ("Worst Case") Post Strength (kips): P_P

$P_{P1} = 9.73 \cdot kip$ — Post Strength based on the Plastic Failure of a Steel Post (kip)

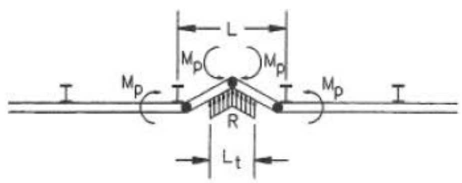
$P_{P2} = 26.864 \cdot kip$ — Post Strength based on the Ultimate Strength of the Anchor Rods (kip)

$P_{P3} = 18.215 \cdot kip$ — Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)

$P_P := \min(P_{P1}, P_{P2}, P_{P3}) = 9.73 \cdot kip$ — Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

Determine Ultimate Capacity of Steel Rail for a Single and Double Span Failure Mode:

$L_t = 4 \cdot ft$	Length of the Distribution of the Transverse Impact Force (ft.)
$L_p = 8 \cdot ft$	Steel Post Spacing (ft.)
$M_p = 16.986 \cdot kip \cdot ft$	Flexural Capacity of the Steel Tube Rail (kip-ft)
$P_p = 9.73 \cdot kip$	Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

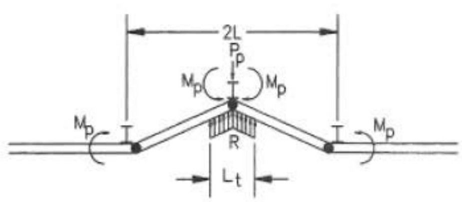


Single–Span Failure Mode

$N_1 := 1$ Number of Spans

$$R_R := \frac{(16 \cdot M_p) + \left[(N_1 - 1)(N_1 + 1) \cdot P_p \cdot L_p\right]}{(2 \cdot N_1 \cdot L_p) - L_t} = 22.648 \cdot kip$$

Ultimate Capacity of rail over one span (kip)



Two–Span Failure Mode

$N_2 := 2$ Number of Spans

$$R_{R'} := \frac{16 \cdot M_p + \left(N_2^{\,2} \cdot P_p \cdot L_p\right)}{(2 N_2 \cdot L_p) - L_t} = 20.826 \cdot kip$$

Ultimate Capacity of rail over two spans (kip)

(4g) Determine the Combined Resultant Strength of the Bridge Rail System:

Determine the Resultant Strength of the Bridge Rail System at Midspan: (R_1)

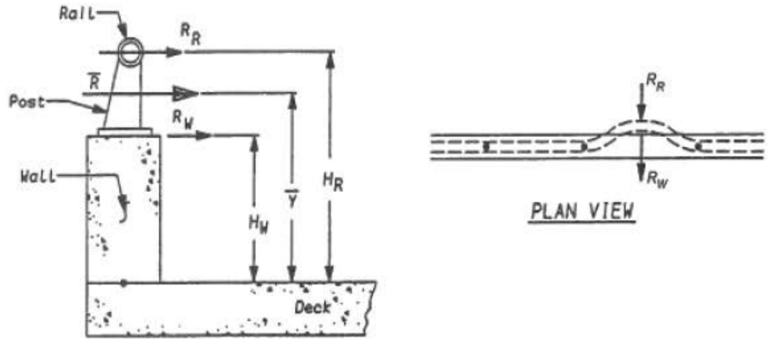


Figure A13.3.3-1—Concrete Wall and Metal Rail Evaluation—Impact at Midspan of Rail

$R_R = 22.648 \cdot kip$	Ultimate Capacity of rail over one span (kip)
$H_R = 38.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 87.08 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$R_{bar1} := R_R + R_{wmid} = 109.728 \cdot kip$	Resultant Strength of the Bridge Rail System Located at y_{bar1} AASHTO Eqn. A13.3.3-1
$y_{bar1} := \dfrac{R_R \cdot H_R + R_{wmid} \cdot H_w}{R_{bar1}} = 30.141 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-2
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load
$R_1 := R_{bar1} \cdot \left(\dfrac{y_{bar1}}{H_e}\right) = 174.072 \cdot kip$	Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

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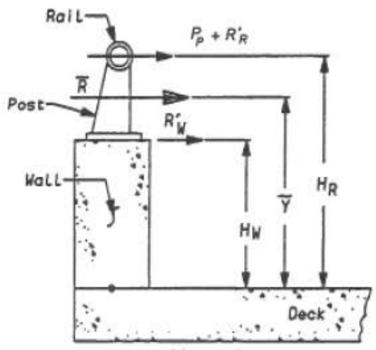
Determine the Resultant Strength of the Bridge Rail System at a Post: (R_2)



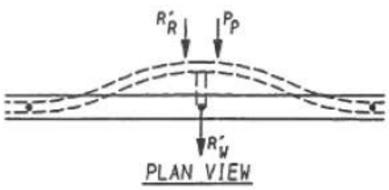


Figure A13.3.3-2—Concrete Wall and Metal Rail Evaluation—Impact at Post

$R_{R'} = 20.826 \cdot kip$	Ultimate Capacity of rail over two spans (kip)
$H_R = 38.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 87.08 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$R_{w'} := \dfrac{R_{wmid} \cdot H_w - P_P \cdot H_R}{H_w} = 73.745 \cdot kip$	Reduced Wall Strength (kip) AASHTO Eqn. A13.3.3-5
$R_{bar2} := P_P + R_{R'} + R_{w'} = 104.301 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at y_{bar2} AASHTO Eqn. A13.3.3-3
$y_{bar2} := \dfrac{H_R \cdot (P_P + R_{R'}) + R_{w'} \cdot H_w}{R_{bar2}} = 31.039 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-4
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load (in.)
$R_2 := R_{bar2} \cdot \left(\dfrac{y_{bar2}}{H_e}\right) = 170.392 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

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Figure 5-397.107
MASH TL-3 Compliance Assessment

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$R_1 = 174.072 \cdot kip$ — Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_1 \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$R_2 = 170.392 \cdot kip$ — Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_a\_Post\_Check} := \begin{cases} \text{"OK"} & \text{if } R_2 \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

(5) Strength Analysis of the Seperate End Post:

(5a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

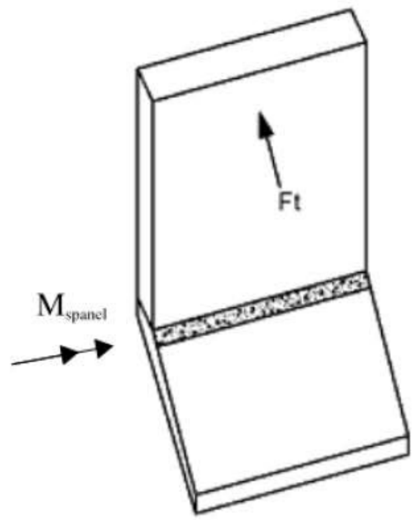

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 40.625in$ — Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 19.625in$ — Width of the end post (in.)

$A_{v1.spost} := 1.56in^2$ — Area of one vertical reinforcement leg in the tension zone of the end post (in^2)
#11 Bars

$n_{v.spost} := 2$ — Number of vertical reinforcement in the end post (in.)

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(5a-conti.) <u>Bending Capacity of the End Post about the Longitudinal Axis</u>: $M_{s.post}$

$A_{v.spost} := n_{v.spost} \cdot A_{v1.spost} = 3.12 \cdot in^2$

Total Area of vertical reinforment in the tension zone of the end post (in^2)

$a_{v.spost} := \dfrac{A_{v.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.87 \cdot in$

Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 19in$

Average extreme distance of tension vertical reinforcement in the end post (in.)

$M_{s.post} := A_{v.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{v.spost}}{2}\right) = 187.874 \cdot kip \cdot ft$

Flexural Capacity of the End Post about the Longitudinal Axis when considering only the vertical reinforcment (kip-ft)

(5a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 187.874 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \frac{M_{s.post}}{H_e} = 118.657 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$b_{spost} = 1.635 \cdot ft$ — Width of the End Post (ft.)

$F_{tr} := F_t \cdot \left(\frac{b_{spost}}{L_t}\right) = 29.029 \cdot kip$ — Ratioed Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check\_Against\_Ftr} := \left(\begin{array}{l} \text{"OK"} \quad \text{if} \quad R_{s.post} \ge F_{tr} \\ \text{"NOT OK"} \quad \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check\_Against\_Ftr} = \text{"OK"}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check\_Against\_Ft} := \left(\begin{array}{l} \text{"OK"} \quad \text{if} \quad R_{s.post} \ge F_t \\ \text{"NOT OK"} \quad \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check\_Against\_Ft} = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

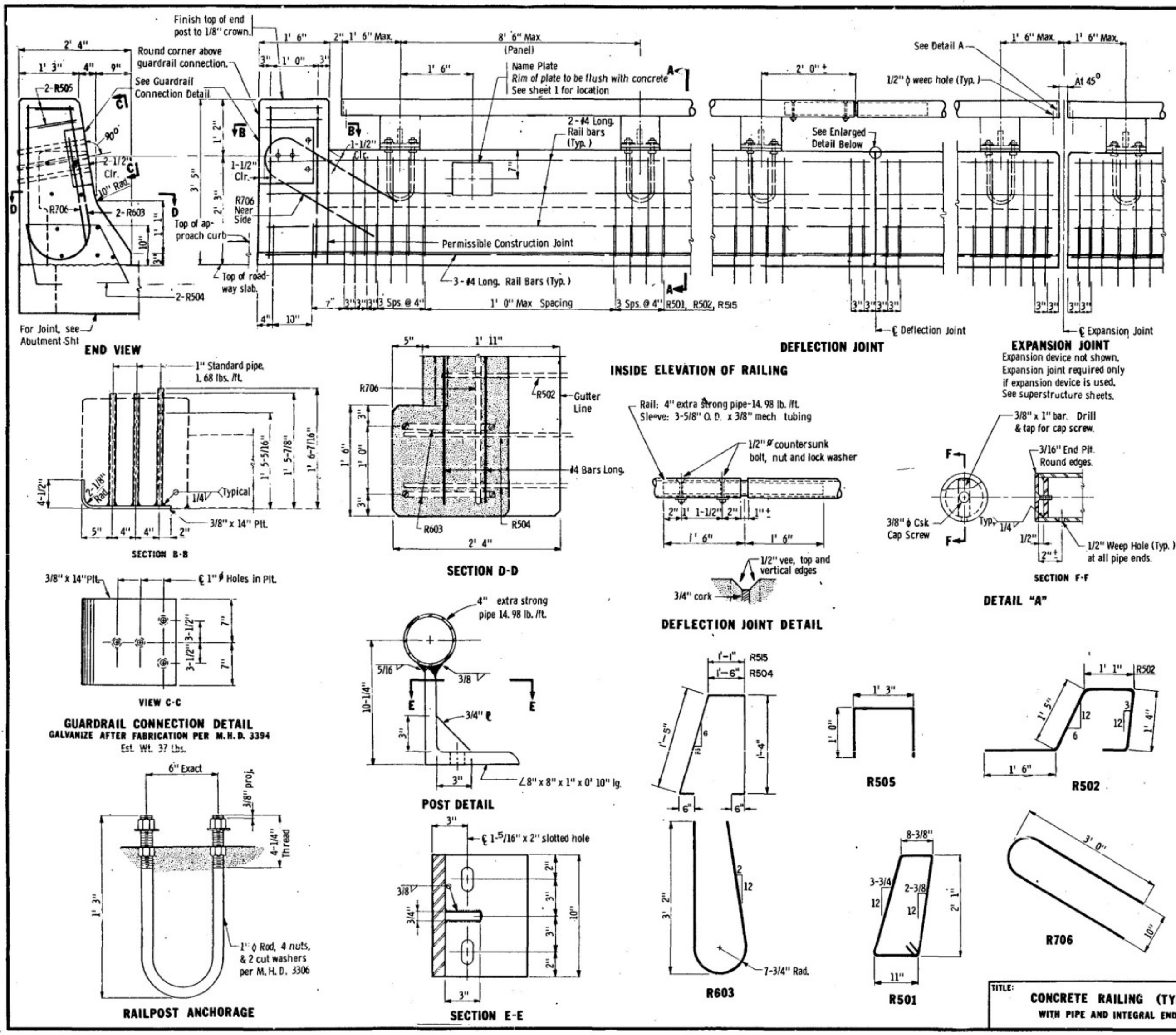
Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

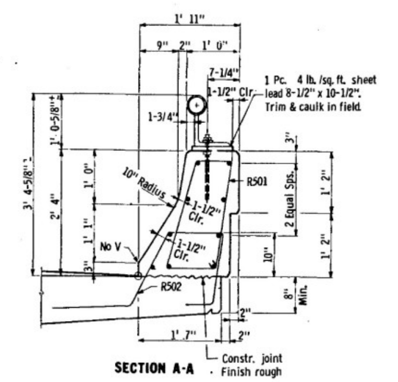
Structural_Capacity_of_End_Post_Check_Against_Ftr = "OK"

Structural_Capacity_of_End_Post_Check_Against_Ft = "OK"

The G-Barrier on Bridge No. 09830 (Fig. 5-397.107) has not satisfied all MASH TL-3 Criteria

APPENDIX D2: G BARRIER ON BRIDGE NO. 19021 (FIGURE 5-397.109)





BILL OF REINFORCEMENT FOR RAILING

BAR	NO.	LENGTH	SHAPE	LOCATION
R501	356	6' 11"	Bent	Rail Vertical
R502	232	5' 10"	Bent	Rail Vertical
R603	4	7' 1"	Bent	End Vertical
R504	4	5'-3"	Bent	End Vertical
R505	8	3' 3"	Bent	End Long.
R706	2	6' 6"	Bent	End at G. R. Conn.
R407	18	19'-6"	Str't.	Rail Long.
R408	54	26'-9"	"	" "
R414	9	19'-7"	"	" "
R515	124	4'-10"	Bent	Rail Transv
R418	9	21'-0"	Str't	Rail Long.

GENERAL NOTES

Railbase to be Concrete Mix No. 3Y46A.

Pipe shall comply with M. H. D. 3362. The 4" ϕ pipe shall have a minimum yield point of 35000 P. S. I.

Structural steel shall comply with M. H. D. 3306.

Finish all edges of railbase and end post with 1/2" vee, except where otherwise noted.

Anchorages shall be accurately placed to provide correct alignment of railing. Set normal to grade.

Galvanize pipe and structural steel per M. H. D. 3394 after fabrication. Galvanize bolts and anchorages per M. H. D. 3392

See superstructure sheet for joint spacing.

Maximum spacing of concrete deflection joints shall be 30' 0". Railpipe deflection joints shall be placed in same panel as concrete deflection joints.

Price bid for ornamental metal railing includes the post anchorages and all material above railbase. Guardrail connection to be included in weight of structural steel M. H. D. 3306

Length of ornamental metal railing for payment is measured end to end of pipe.

Railing quantities are included in summary of quantities for superstructure.

Fig. 5-397.109

June 2, 1971

TITLE:	DES: RAH	DR: SRH	APPROVED:	Bridge No.
CONCRETE RAILING (TYPE G) WITH PIPE AND INTEGRAL END POST	CHK: JC	CHK: JC	4-7-72	19021
	Sheet No. 20 of 26 Sheets			

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SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH TL-3 Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$H_r := 40.625in$	Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)
$H_R := 38.375in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$t_o := 0in$	Thickness of overlay (in.)

(1b) Concrete Parapet Inputs:

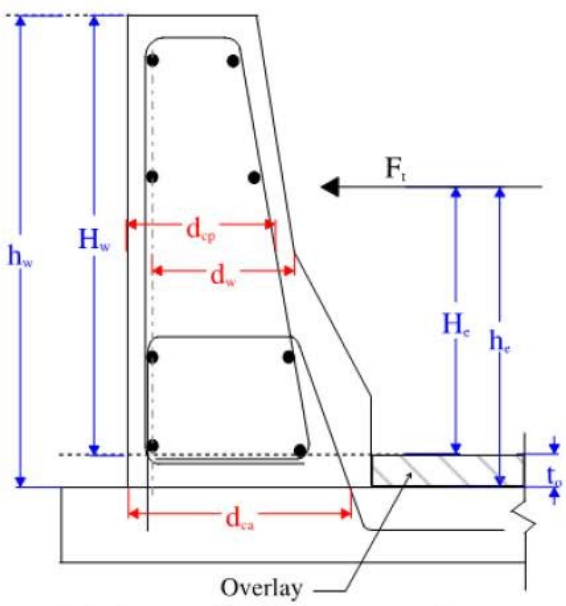

Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(1b-conti.) Concrete Parapet Inputs:

$H_w := 28in$	Height of the concrete parapet/wall measured from the top of the roadway surface/overlay (in.)
$h_w := H_w + t_o = 28 \cdot in$	Total height of the concrete parapet/wall (in.)

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)
$s_{vp.mid} := 12in$	Average Spacing of parapet vertical reinforcement at midspan (in.)
$d_{cp.mid} := 10.1875in$	Extreme distance of parapet vertical reinforcement in tension at midspan (in.)
$A_{vp1.end} := 0.31in^2$	Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)
$s_{vp.end} := 4in$	Average Spacing of parapet vertical reinforcement at joints/ends (in.)
$d_{cp.end} := 10.1875in$	Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$	Area of longitudinal reinforcement bars in tension (in^2)
$d_w := 9.625in$	Extreme distance of tension longitudinal reinforcement of wall (in.)

(1b-conti.) **Concrete Parapet Inputs:**

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502} := 10in$ Projected length of R502 reinforcement over the slab (in.)

$L_{wid\_R502} := 13in$ Outer width of R502 reinforcement (in.)

$Cover := 2in$ Cover clear distance (in.)

$Ratio_{R502} := \frac{6}{12}$ Inclined angle of R502E reinforcement

$d_{b\_R502} := 0.625in$ Nominal diameter of R502E reinforcement (#5 bar)

$Coping := 2in$ Coping on the back of the barrier

$$d_{ca} := L_{wid\_R502} + L_{proj\_R502} \cdot Ratio_{R502} + Cover - \frac{1}{2} d_{b\_R502} - Coping = 17.688 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ Average Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 17.688 \cdot in$ Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

$A_{va1.end} := 0.31in^2$ Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 4in$ Average Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := d_{ca} = 17.688 \cdot in$ Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

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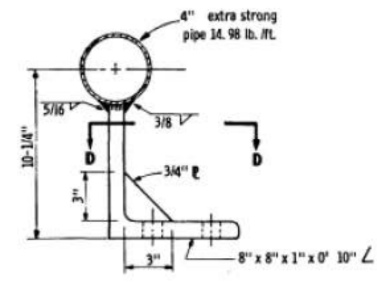
SUBJECT: **MnDOT G-Barrier on Bridge No. 19021**
Figure 5-397.109
MASH TL-3 Compliance Assessment

(1b) Steel Rail, Post, and Anchor Rod Inputs:

Steel Rail Inputs:
a) Steel Tube Rail is M.H.D. 3362 material, Fy=35ksi
b) Steel Tube Rail is a 4" extra strong pipe

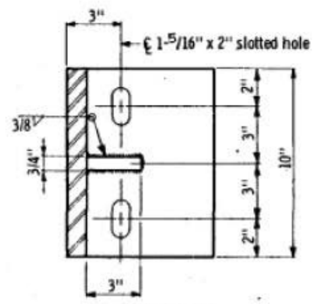
$F_{yR} := 35ksi$	Yield Strength of Steel Tube Rail (ksi)
$d_{oR} := 4.5in$	Outside diameter of Steel Tube Rail (in.)
$d_{iR} := 3.83in$	Inside diameter of Steel Tube Rail (in.)



POST DETAIL

Steel Post Inputs:
a) Steel Post is M.H.D. 3306 material, Fy= 36ksi
b) Steel Post is a 8"x8"x1"x10" Angle Member

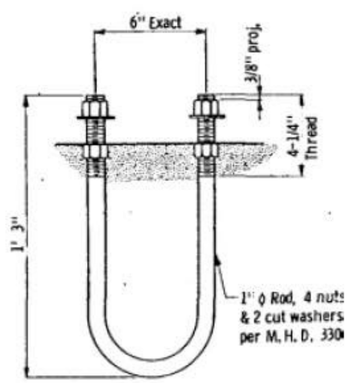
$F_{yp} := 36ksi$	Yield Strength of Steel Post (ksi)
$w_p := 10in$	Width of Steel Post about the bending axis (in.)
$t_p := 1in$	Thickness of Steel Post (in.)
$h_p := 10.25in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$L_p := 8.5ft$	Steel Post Spacing (ft.)



SECTION E-E

Anchor Rod Inputs:
a) Anchor Rods are F1554 Gr. 36 material, Fu=58 ksi
b) Anchor Rods are 1"ϕ x 15" U bolt

$F_{u.rod} := 58ksi$	Tensile Strength of Anchor Rods (ksi)
$N_{rod} := 1$	Number of Anchor Rods
$N_{rod.tension} := 2$	Number of Anchor Rods in Tension
$d_{rod} := 5in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$\phi_{rod} := 1in$	Diameter of Anchor Rods (in)



RAILPOST ANCHORAGE

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SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ — Test Level

$F_t := 71kip$ — Transverse Impact Force

$L_t := 4ft$ — Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ — Height of Equivalent Transverse Load

$H_{min} := 29in$ — Minimum height of a MASH TL-3 barrier (in.)

$H_r = 40.625 \cdot in$ — Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ — Minimum height of a MASH TL-3 barrier (in.)

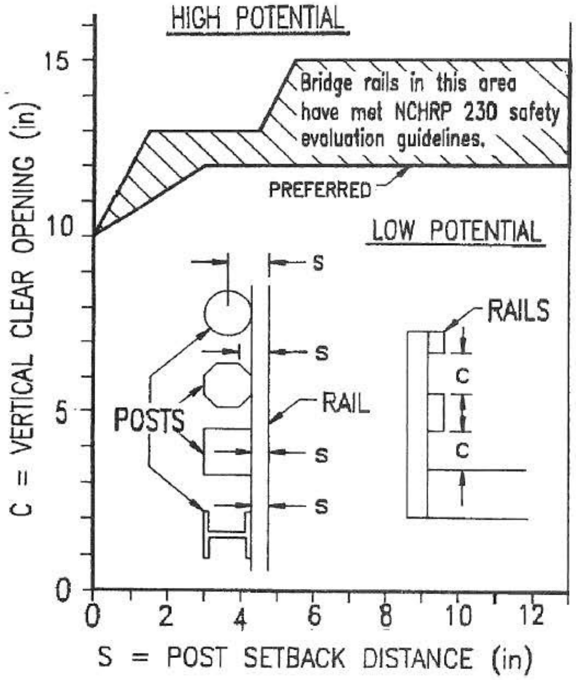
$H_r = 40.625 \cdot in$ — Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_r \geq H_{min} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

(4) Geometric Criteria:

$S_{post} := 1.75in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 10.375in$	Vertical Clear Opening (in.)
$\Sigma A := H_w + \frac{d_{oR}}{2} = 30.25 \cdot in$	Total Rail Contact Width (in.)
$H_r = 40.625 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.



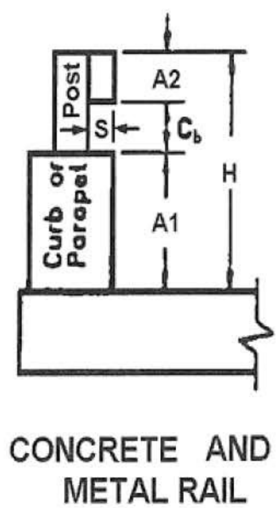


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

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SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(3-conti.) Geometric Criteria:

$S_{post} = 1.75 \cdot in$ $\Sigma A = 30.25 \cdot in$ $H_r = 40.625 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_r} = 0.745$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

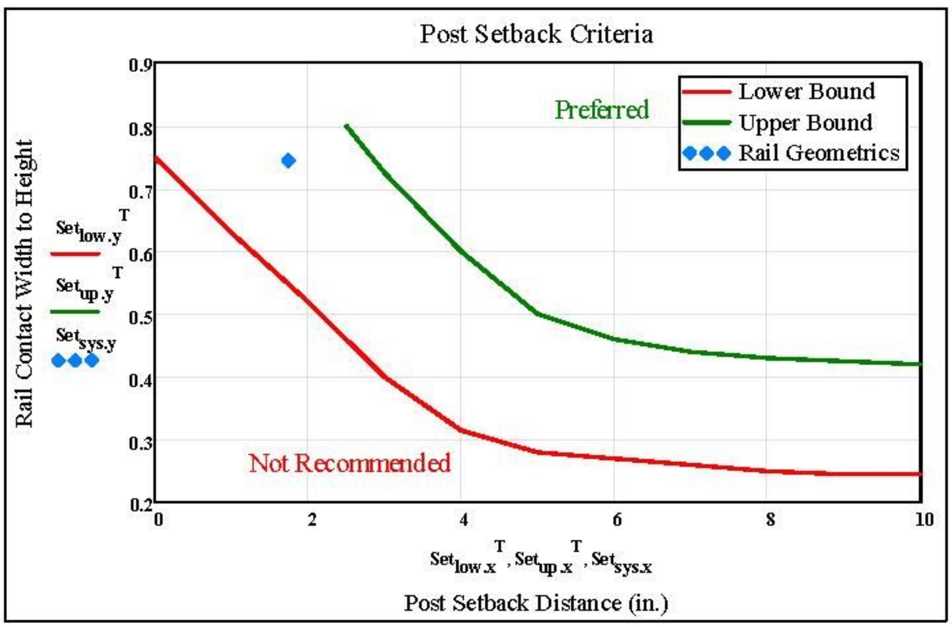
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 1.75$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.745$ Ratio of Contact Width to Total Height rail geometric point



NotRecommended := 1 Marginal := 2 Preferred := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Post_Setback_Criteria_Rail_Geometric_Point := Marginal

SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(3-conti.) Geometric Criteria:

$snag_{low.x} := (0\ \ 3\ \ 13)$

$snag_{low.y} := (10\ \ 12\ \ 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0\ \ 1.25\ \ 4.25\ \ 5.25\ \ 13)$

$snag_{up.y} := (10\ \ 13\ \ 13\ \ 15\ \ 15)$

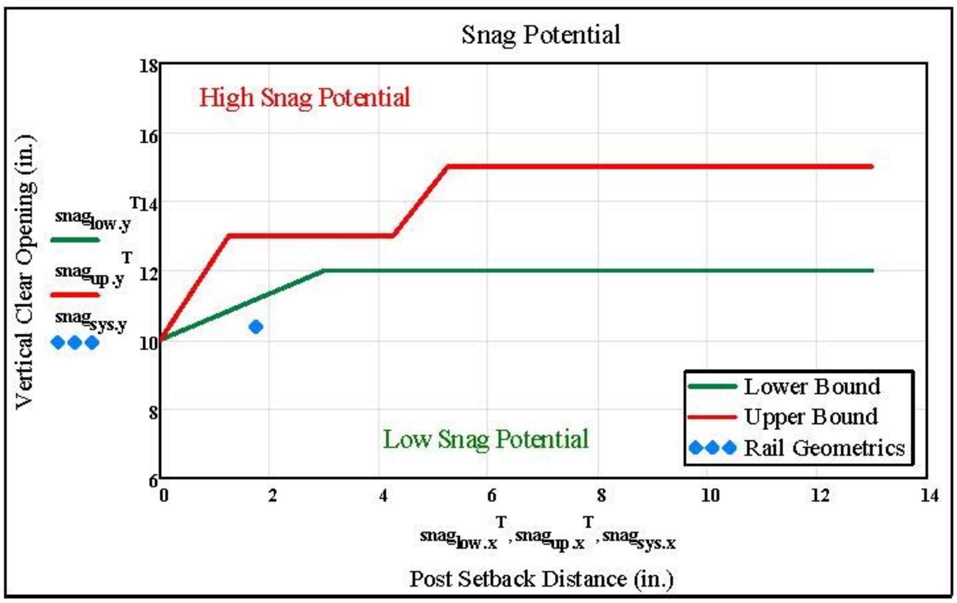
Upper Boundary for Snag Potential Criteria
x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 1.75$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 10.375$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1 Marginal = 2 LowSnagPotential := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := LowSnagPotential

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SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(3) Geometric Criteria - Summary of Results:

Post_Setback_Criteria_Check := | "OK" if Post_Setback_Criteria_Rail_Geometric_Point = Preferred
| "NOT OK" otherwise

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check := | "OK" if Snag_Potential_Criteria_Rail_Geometric_Point = LowSnagPotential
| "NOT OK" otherwise

Snag_Potential_Criteria_Check = "OK"

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one parapet vertical reinforcement leg in the tension zone (in2)

$s_{vp.mid} = 12 \cdot in$

Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in2)

$d_{cp.mid} = 10.188 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 15.437 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

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SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(4a-conti.) **Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)**

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in2)

$d_{ca.mid} = 17.688 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$

Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 27.062 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 15.437 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

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SUBJECT: MnDOT G-Barrier on Bridge No. 19021
Figure 5-397.109
MASH TL-3 Compliance Assessment

(4b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 4 \cdot in$ — Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.93 \cdot in^2$ — Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.368 \cdot in$ — Depth of Whitney Stress Block (in.)

$d_{cp.end} = 10.188 \cdot in$ — Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 44.192 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 4 \cdot in$ — Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.93 \cdot in^2$ — Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.368 \cdot in$ — Depth of Whitney Stress Block (in.)

(4b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 17.688 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 79.067 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 44.192 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(4c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 9.625 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 28 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.504 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 37.492 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

(4d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

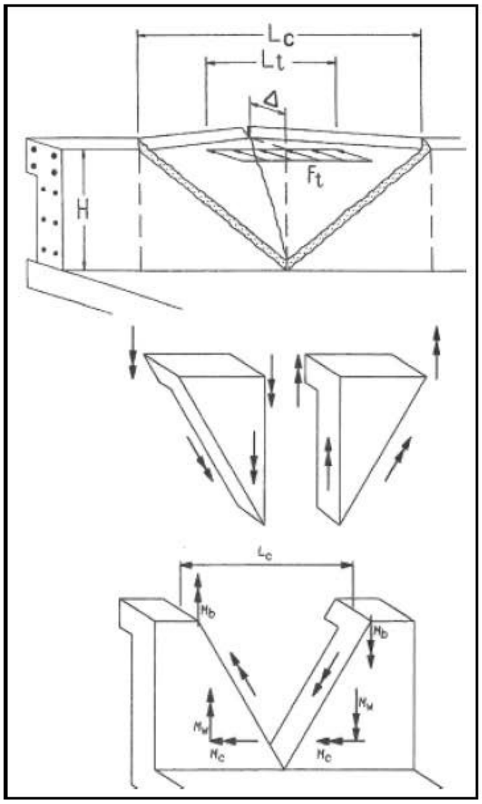

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3d

$M_B := 0kip \cdot ft$ — No additional beam strength

$M_{cmid} = 15.437 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 37.492 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.024 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 119.403 \cdot kip$$ (Equation A13.3.1-1)

(4e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

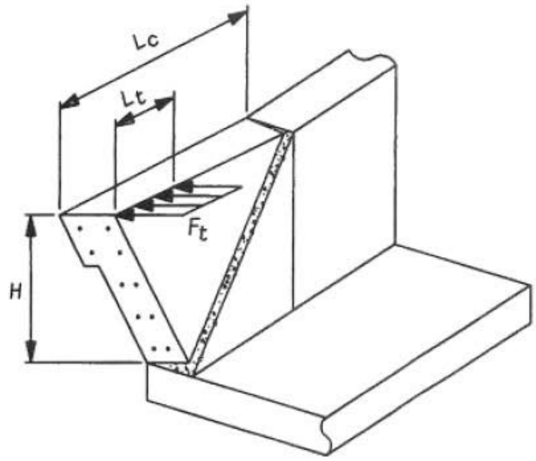

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 28 \cdot in$	Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3e
$M_B = 0$	No additional beam strength
$M_w = 37.492 \cdot kip \cdot ft$	Flex. Resistance of the Wall about the Vert. Axis (k-ft)
$L_t = 4 \cdot ft$	Longitudinal length of distribution of impact force (ft.)
$M_{cend} = 44.192 \cdot \frac{kip \cdot ft}{ft}$	Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 4.445 \cdot ft \qquad \text{(Equation A13.3.1-4)}$$

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\ 2}\right)}{h_w}\right] = 168.384 \cdot kip \qquad \text{(Equation A13.3.1-3)}$$

(4f) Steel Rail & Post Strength Analysis:

$F_{yR} = 35 \cdot ksi$ — Yield Strength of Steel Tube Rail (ksi)

$d_{oR} = 4.5 \cdot in$ — Outside diameter of Steel Tube Rail (in.)

$d_{iR} = 3.83 \cdot in$ — Inside diameter of Steel Tube Rail (in.)

$$Z_R := \frac{\left(d_{oR}^{\,3} - d_{iR}^{\,3}\right)}{6} = 5.824 \cdot in^3$$ Plastic Sectional Modulus of the Steel Tube Rail (in^3)

$M_p := F_{yR} \cdot Z_R = 16.986 \cdot kip \cdot ft$ — Plastic Moment Capacity of the Steel Tube Rail (kip-ft)

Calculate the Plastic Strength of the Post: P_{P1}

$w_p = 10 \cdot in$ — Width of Steel Post about the bending axis (in.)

$t_p = 1 \cdot in$ — Thickness of Steel Post (in.)

$$Z_P := \frac{w_p \cdot t_p^{\,2}}{4} = 2.5 \cdot in^3$$ Plastic Sectional Modulus of the Steel Post about the bending axis (in.)

$F_{yp} = 36 \cdot ksi$ — Yield Strength of Steel Post (ksi)

$M_{post} := F_{yp} \cdot Z_P = 7.5 \cdot kip \cdot ft$ — Plastic Moment Capacity of the Steel Post (kip-ft)

$h_p = 10.25 \cdot in$ — Height from the bottom of the post to the centroid of the steel tube rail (in.)

$$P_{P1} := \frac{M_{post}}{h_p} = 8.78 \cdot kip$$ Post Strength based on the Plastic Failure of a Steel Post (kip)

Calculate the Post Strength based on the Ultimate Strength of the Anchor Rods: P_{P2}

$F_{u.rod} = 58 \cdot ksi$ — Tensile Strength of the Anchor Rods (ksi)

$\phi_{rod} = 1 \cdot in$ — Diameter of Anchor Rods (in)

$A_{rod} := \frac{\pi}{4} \cdot \phi_{rod}^2 = 0.785 \cdot in^2$ — Area of a Anchor Rod (in^2)

$R_{nt} := F_{u.rod} \cdot (0.75 \cdot A_{rod}) = 34.165 \cdot kip$ — Nominal strength of one Anchor Rod in Tension (kip)

$N_{rod.tension} = 2$ — Number of Anchor Rods acting in tension

$d_{rod} = 5 \cdot in$ — Distance from the anchor rods acting in tension to the back of the steel plate (in.)

$d_b := 1.5in$ — Length of the steel plate bearing pressure acting on the concrete parapet (in.)

$w_{rod} := d_{rod} - \frac{d_b}{3} = 4.5 \cdot in$ — Distance from anchor rods acting in tension to the centroid of the bearing pressure acting on the concrete parapet (in.)

$M_{t.rod} := w_{rod} \cdot R_{nt} \cdot N_{rod.tension} = 25.624 \cdot kip \cdot ft$ — Moment strength of Post based on tensile capacity of Anchor Rods (k-ft)

$h_p = 10.25 \cdot in$ — Height from the bottom of the post to the centroid of the steel tube rail (in.)

$P_{t.rod} := \frac{M_{t.rod}}{h_p} = 29.998 \cdot kip$ — Post Strength based on the tensile capacity of Anchor Rods (kip)

$R_{nv} := F_{u.rod} \cdot (0.45 \cdot A_{rod}) = 20.499 \cdot kip$ — Nominal strength of one anchor rod in Shear w/h threads in shear plane (kip)

$P_{v.rod} := N_{rod} \cdot R_{nv} = 20.499 \cdot kip$ — Post Strength based on the shear capacity of Anchor Rods (kip)

$P_{P2} := \min(P_{t.rod}, P_{v.rod}) = 20.499 \cdot kip$ — Post Strength based on the Ultimate Strength of the Anchor Rods (kip)

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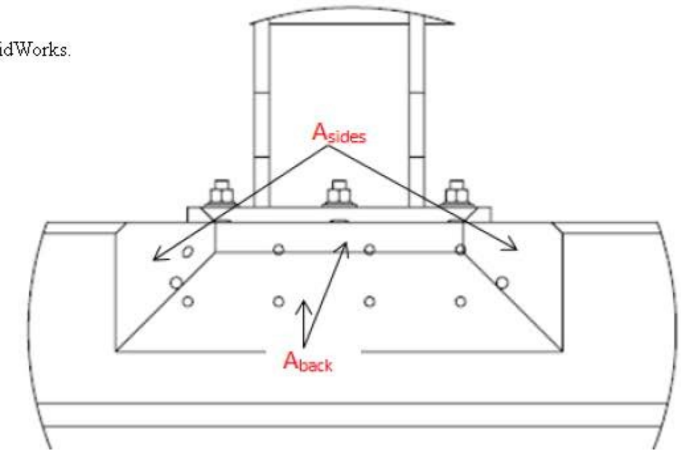
MASH TL-3 Compliance Assessment

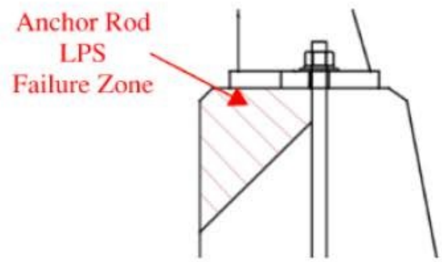
Calculate the Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods: P_{P3}

Note: This failure mechanism was modeled in SolidWorks.



$A_{sides} := 59 in^2$

$A_{back} := 120 in^2$



$\phi_v := 0.75$ — Shear Strength Reduction Factor

$A_{LPS} := A_{sides} + A_{back} = 0.115\ m^2$ — Total Area of Failure Planes due to Lateral Punching Shear Failure (in^2)

$f'_c = 4 \cdot ksi$ — Concrete Compressive Strength (ksi)

$V_{c.lat} := \phi_v \cdot 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi = 94.87 \cdot psi$ — Concrete Stress from Block Shear of Anchor Rods (psi) -ACI 318-14 Eqn. 22.5.5.1

$P_{P3} := V_{c.lat} \cdot A_{LPS} = 16.981 \cdot kip$ — Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)

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MASH TL-3 Compliance Assessment

Determine the Limiting ("Worst Case") Post Strength (kips): P_P

$P_{P1} = 8.78 \cdot kip$ — Post Strength based on the Plastic Failure of a Steel Post (kip)

$P_{P2} = 20.499 \cdot kip$ — Post Strength based on the Ultimate Strength of the Anchor Rods (kip)

$P_{P3} = 16.981 \cdot kip$ — Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)

$P_P := \min(P_{P1}, P_{P2}, P_{P3}) = 8.78 \cdot kip$ — Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

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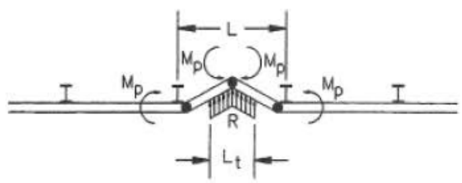
Figure 5-397.109
MASH TL-3 Compliance Assessment

Determine Ultimate Capacity of Steel Rail for a Single and Double Span Failure Mode:

$L_t = 4 \cdot ft$	Length of the Distribution of the Transverse Impact Force (ft.)
$L_p = 8.5 \cdot ft$	Steel Post Spacing (ft.)
$M_p = 16.986 \cdot kip \cdot ft$	Flexural Capacity of the Steel Tube Rail (kip-ft)
$P_p = 8.78 \cdot kip$	Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

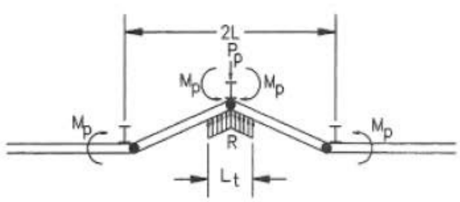


Single–Span Failure Mode

$N_1 := 1$ Number of Spans

$$R_R := \frac{(16 \cdot M_p) + \left[(N_1 - 1)(N_1 + 1) \cdot P_p \cdot L_p\right]}{(2 \cdot N_1 \cdot L_p) - L_t} = 20.906 \cdot kip$$

Ultimate Capacity of rail over one span (kip)



Two–Span Failure Mode

$N_2 := 2$ Number of Spans

$$R_{R'} := \frac{16 \cdot M_p + \left(N_2^{\,2} \cdot P_p \cdot L_p\right)}{(2N_2 \cdot L_p) - L_t} = 19.011 \cdot kip$$

Ultimate Capacity of rail over two spans (kip)

(4g) Determine the Combined Resultant Strength of the Bridge Rail System:

Determine the Resultant Strength of the Bridge Rail System at Midspan: (R_1)

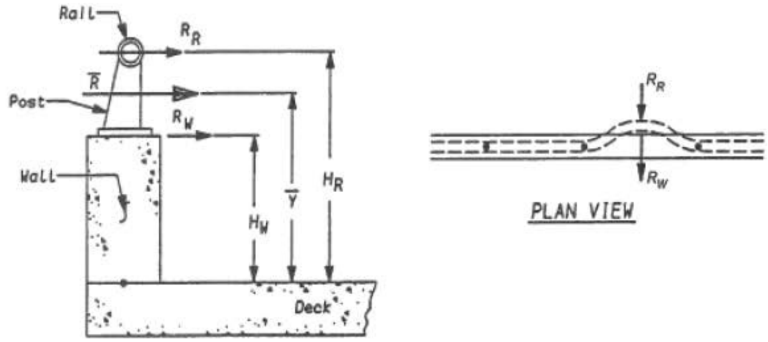


Figure A13.3.3-1—Concrete Wall and Metal Rail Evaluation—Impact at Midspan of Rail

$R_R = 20.906 \cdot kip$ — Ultimate Capacity of rail over one span (kip)

$H_R = 38.375 \cdot in$ — Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)

$R_{wmid} = 119.403 \cdot kip$ — Ultimate Resistance of the concrete parapet at midspan (kip)

$H_w = 28 \cdot in$ — Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)

$R_{bar1} := R_R + R_{wmid} = 140.31 \cdot kip$ — Resultant Strength of the Bridge Rail System Located at y_{bar1} AASHTO Eqn. A13.3.3-1

$$y_{bar1} := \frac{R_R \cdot H_R + R_{wmid} \cdot H_w}{R_{bar1}} = 29.546 \cdot in$$

Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-2

$H_e = 19 \cdot in$ — Height of Equivalent Transverse Load

$$R_1 := R_{bar1} \cdot \left(\frac{y_{bar1}}{H_e}\right) = 218.188 \cdot kip$$

Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

Determine the Resultant Strength of the Bridge Rail System at a Post: (R_2)

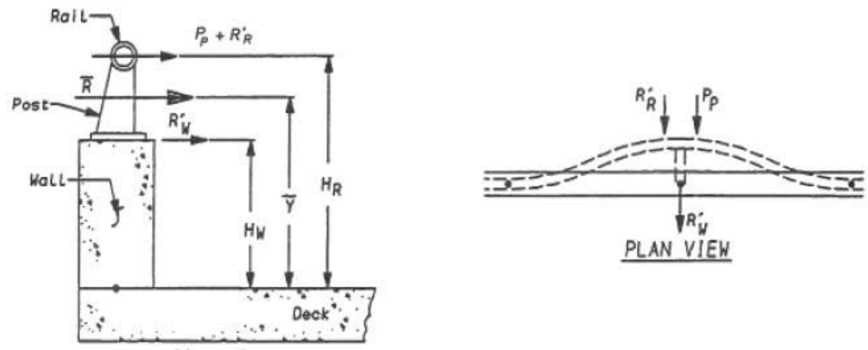


Figure A13.3.3-2—Concrete Wall and Metal Rail Evaluation—Impact at Post

$R_{R'} = 19.011 \cdot kip$	Ultimate Capacity of rail over two spans (kip)
$H_R = 38.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 119.403 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$R_{w'} := \dfrac{R_{wmid} \cdot H_w - P_P \cdot H_R}{H_w} = 107.37 \cdot kip$	Reduced Wall Strength (kip) AASHTO Eqn. A13.3.3-5
$R_{bar2} := P_P + R_{R'} + R_{w'} = 135.161 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at y_{bar2} AASHTO Eqn. A13.3.3-3
$y_{bar2} := \dfrac{H_R \cdot (P_P + R_{R'}) + R_{w'} \cdot H_w}{R_{bar2}} = 30.133 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-4
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load (in.)
$R_2 := R_{bar2} \cdot \left(\dfrac{y_{bar2}}{H_e}\right) = 214.359 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$R_1 = 218.188 \cdot kip$ — Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_Midspan\_Check} := \begin{cases} \text{"OK"} & \text{if } R_1 \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$R_2 = 214.359 \cdot kip$ — Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

$$\text{Structural\_Capacity\_of\_Barrier\_at\_a\_Post\_Check} := \begin{cases} \text{"OK"} & \text{if } R_2 \ge F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

(4) Strength Analysis of the Integral End Post:

(4a) Bending Capacity of the End Post and Conjoining Barrier Segment about the Long Axis: $M_{c.ipost}$

$b_c = 12 \cdot in$ — Unit Width of Wall (in.)

Bending Capacity of End Post and Conjoining Barrier Segment Considering only the Parapet Vertical Reinf.:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{v1.ipost} := 0.44in^2$ — Average area of one vertical reinforcement leg in the tension zone at the end post and conjoining barrier segment (in^2)

$s_{v.ipost} := 10in$ — Average spacing of vertical reinforcement at the end post and conjoining barrier segment (in.)

$$A_{v.ipost} := \left(\frac{b_c}{s_{v.ipost}}\right) \cdot A_{v1.ipost} = 0.528 \cdot in^2$$

Total Area of vertical reinforment per unit length of the wall at the end post and conjoining barrier segment (in^2)

$$a_{c.ipost} := \frac{A_{v.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.776 \cdot in$$

Depth of Whitney Stress Block (in.)

$d_{c.ipost} := 16in$ — Average extreme distance of tension vertical reinforcement at the end post and conjoining barrier segment (in.)

$$M_{c.ipost} := \frac{\left[A_{v.ipost} \cdot f_y \cdot \left(d_{c.ipost} - \frac{a_{c.ipost}}{2}\right)\right]}{b_c} = 41.215 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis when considering the critical reinforcment specified in Article A13.3.1 (k-ft/ft) (k-ft/ft)

(4b) Bending Capacity of the End Post and Conjoining Barrier Segment about the Vert. Axis: $M_{w.ipost}$

$A_{w1.ipost} := 0.2in^2$ Area of one longitudinal reinforcement bar in tension (in^2)

$n_{w.ipost} := 4$ Number of longitudinal reinforcement bars acting in tension

$A_{w.ipost} := A_{w1.ipost} \cdot n_{w.ipost} = 0.8 \cdot in^2$ Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 28 \cdot in$ Total height of the barrier (in.)

$a_{w.ipost} := \dfrac{A_{w.ipost} \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.504 \cdot in$ Depth of the Whitney Stress Block (in.)

$d_{w.ipost} := 11.625in$ Average extreme distance of tension longitudinal reinforcement (in.)

$M_{w.ipost} := A_{w.ipost} \cdot f_y \cdot \left(d_{w.ipost} - \dfrac{a_{w.ipost}}{2}\right) = 45.492 \cdot kip \cdot ft$ Flexural Resistance of the end post and conjoining barrier segment about the Vertical Axis specified in Article A13.3.1 (k-ft)

(4c) Determine the Ultimate Resistance of the End Post and Conjoining Barrier Segment: $R_{w.ipost}$

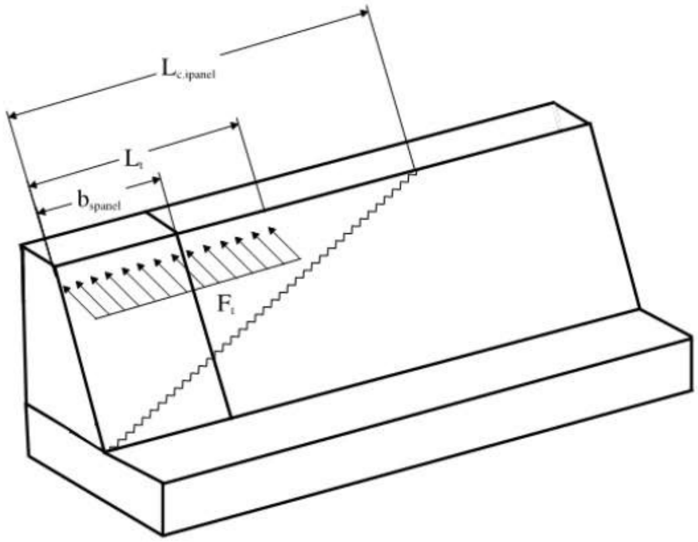


Figure 4c. Yield Line Analysis of the End Post and Conjoining Barrier Segment.

$$M_{w.ipost} = 13.866\,m \cdot \frac{kip \cdot ft}{ft}$$

$$M_{c.ipost} = 41.215 \cdot \frac{kip \cdot ft}{ft}$$ Flexural Resistance of the end post and conjoining barrier segment about the Longitudinal Axis specified in Article A13.4.2 (k-ft/ft)

$L_t = 4 \cdot ft$ Distribution Length of the Impact Force (ft.)

$H_w = 28 \cdot in$ Total Height of the Barrier (in.)

$$L_{c.ipost} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H_w \cdot \left(\frac{M_{w.ipost}}{M_{c.ipost}}\right)} = 4.564 \cdot ft$$ (Equation A13.3.1-4)

$$R_{w.ipost} := \left(\frac{2}{2 \cdot L_{c.ipost} - L_t}\right) \cdot \left[M_{w.ipost} + \frac{\left(M_{c.ipost} \cdot L_{c.ipost}^{\;2}\right)}{H_w}\right] = 161.243 \cdot kip$$ (Equation A13.3.1-3)

(4) Structural Capacity of the End Post and Conjoining Segment of Barrier - Summary of Results:

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the roadway surface/overlay (in.)

$R_{w.ipost} = 161.243 \cdot kip$ — Ultimate Resistance of the end post and conjoining barrier segment (kip)

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$R_{R.ipost} := R_{w.ipost} \cdot \left(\frac{H_w}{H_e}\right) = 237.621 \cdot kip$ — Structural Capacity of the end post and conjoining barrier segment located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_Conjoining\_Barrier\_Segment\_Check} := \begin{cases} \text{"OK"} & \text{if } R_{R.ipost} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Structural\_Capacity\_of\_End\_Post\_and\_Conjoining\_Barrier\_Segment\_Check} = \text{"OK"}$$

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MASH TL-3 Compliance Assessment

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check = "OK"

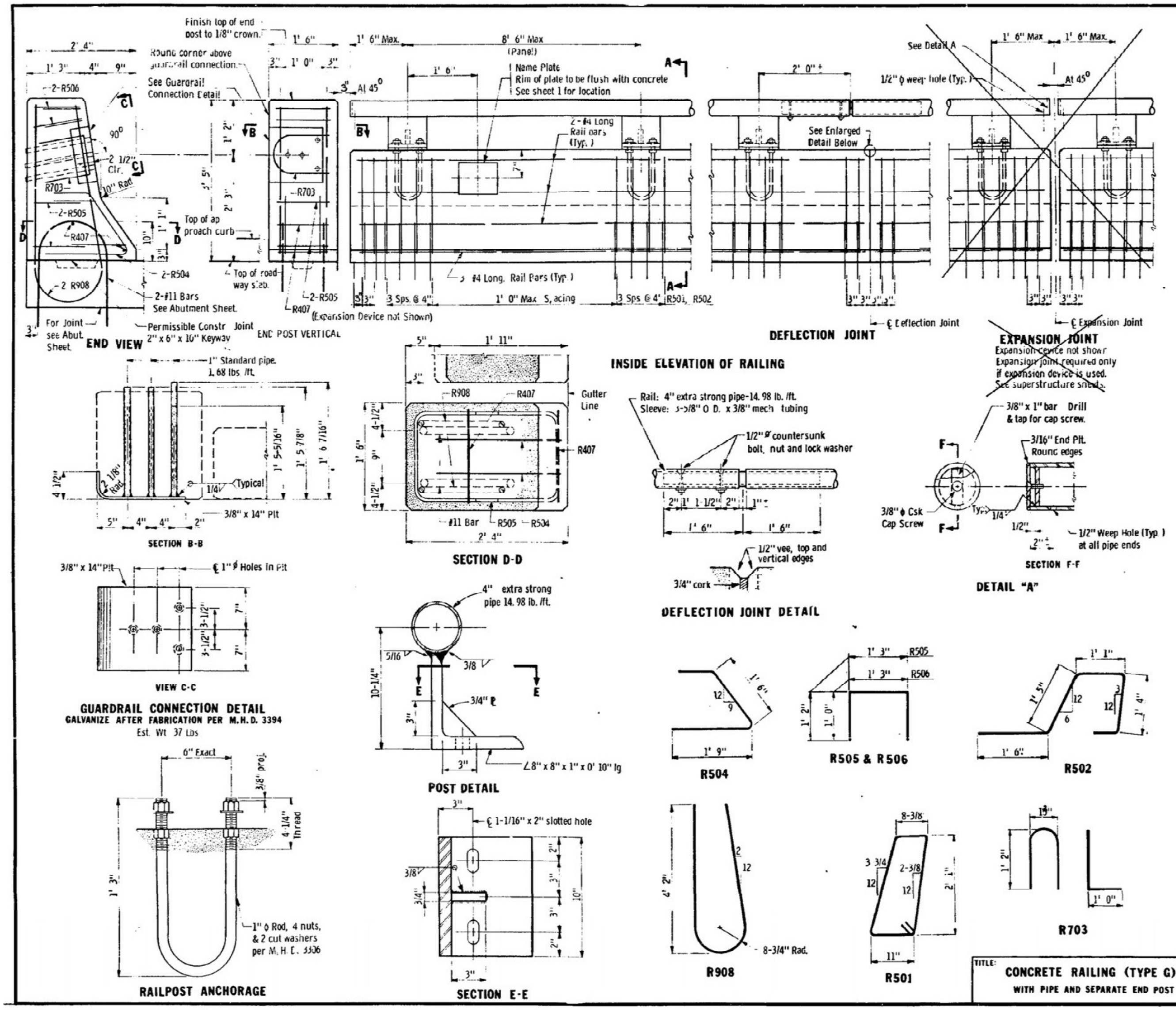
Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

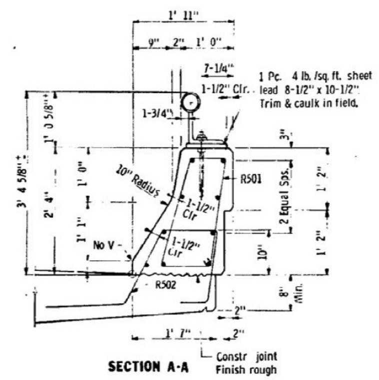
Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

Structural_Capacity_of_End_Post_and_Conjoining_Barrier_Segment_Check = "OK"

The G-Barrier on Bridge No. 09830 (Fig. 5-397.107) has <u>not satisfied</u> all MASH TL-3 Criteria

APPENDIX D3: G BARRIER ON BRIDGE NO. 86812 (FIGURE 5-397.107)





BILL OF REINFORCEMENT FOR RAILING				
BAR	NO	LENGTH	SHAPE	LOCATION
R501	736	6' 11"	Bent	Rail Vertical
R502	736	5' 10"	Bent	Rail Vertical
R703	4	4' 10"	Bent	End Post
R504	8	4' 0"	Bent	End Post
R505	16	3' 7"	Bent	" "
R506	16	3' 3"	Bent	" "
R407	8	1' 3"	Str'l	" "
R908	8	8' 4"	Bent	" "
R409	54	26'-10"	Str'L	Rail Long.
R410	108	25'-2"	"	" "
R411	18	27'-2"	"	" "

GENERAL NOTES:

Railbase and end post to be Concrete Mix No. 3Y46A SPECIAL

Pipe shall comply with M.H.D. 3362. The 4" ϕ pipe shall have a minimum yield point of 35000 P.S.I.

Structural steel shall comply with M.H.D. 3306.

Finish all edges of railbase and end post with 1/2" vee, except where otherwise noted.

Anchorages shall be accurately placed to provide correct alignment of railing. Set normal to grade.

Galvanize pipe and structural steel per M.H.D. 3394 after fabrication. Galvanize bolts and anchorages per M.H.D. 3392

See superstructure sheet for joint spacing.

Maximum spacing of concrete deflection joints shall be 30' 0". Railpipe deflection joints shall be placed in same panel as concrete deflection joints

Price bid for ornamental metal railing includes the post anchorages and all material above railbase. Guardrail connection to be included in weight of structural steel M.H.D. 3306

Length of ornamental metal railing for payment is measured end to end of pipe.

Railing quantities are included in summary of quantities for superstructure.

Fig. 5-397.107
June 2, 1971

TITLE: **CONCRETE RAILING (TYPE G)**
WITH PIPE AND SEPARATE END POST

DES:	DR: R.E.H.	APPROVED: 8-5-71
CHK:	CHK:	

Sheet No. 12 of 20 Sheets

Bridge No. 86812

Texas A&M Transportation Institute

SUBJECT: **MnDOT G-Barrier on Bridge No. 86812**
Figure 5-397.107
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH TL-3 Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$H_r := 40.625in$	Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)
$H_R := 38.375in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$t_o := 0in$	Thickness of overlay (in.)

(1b) Concrete Parapet Inputs:

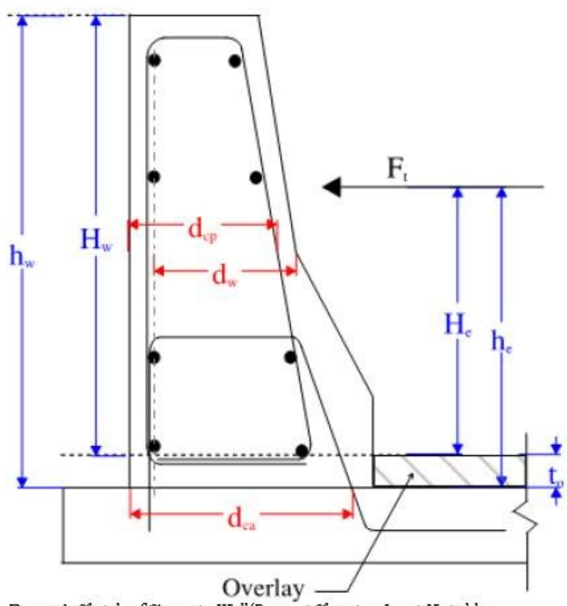


Figure 1. Sketch of Concrete Wall/Parapet Showing Input Variables

SUBJECT: **MnDOT G-Barrier on Bridge No. 86812**
Figure 5-397.107
MASH TL-3 Compliance Assessment

(1b-conti.) Concrete Parapet Inputs:

$H_w := 28in$ — Height of the concrete parapet/wall measured from the top of the roadway surface/overlay (in.)

$h_w := H_w + t_o = 28 \cdot in$ — Total height of the concrete parapet/wall (in.)

Parapet Vertical Reinforcement Inputs:

$A_{vp1.mid} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{vp.mid} := 12in$ — Average Spacing of parapet vertical reinforcement at midspan (in.)

$d_{cp.mid} := 10.1875in$ — Extreme distance of parapet vertical reinforcement in tension at midspan (in.)

$A_{vp1.end} := 0.31in^2$ — Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 4in$ — Average Spacing of parapet vertical reinforcement at joints/ends (in.)

$d_{cp.end} := 10.1875in$ — Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Longitudinal Reinforcement Inputs:

$A_w := 0.8in^2$ — Area of longitudinal reinforcement bars in tension (in^2)

$d_w := 9.625in$ — Extreme distance of tension longitudinal reinforcement of wall (in.)

(1b-conti.) Concrete Parapet Inputs:

Deck Anchorage Vertical Reinforcement Inputs:

$L_{proj\_R502} := 10in$ — Projected length of R502 reinforcement over the slab (in.)

$L_{wid\_R502} := 13in$ — Outer width of R502 reinforcement (in.)

$Cover := 2in$ — Cover clear distance (in.)

$Ratio_{R502} := \frac{6}{12}$ — Inclined angle of R502 reinforcement

$d_{b\_R502} := 0.625in$ — Nominal diameter of R502 reinforcement (#5 bar)

$Coping := 2in$ — Coping on the back of the barrier

$$d_{ca} := L_{wid\_R502} + L_{proj\_R502} \cdot Ratio_{R502} + Cover - \frac{1}{2} d_{b\_R502} - Coping = 17.688 \cdot in$$

Extreme distance of tension deck anchorage vertical reinforcement (in,)

$A_{va1.mid} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at midspan (in^2)

$s_{va.mid} := 12in$ — Average Spacing of deck anchorage vertical reinforcement at midspan (in.)

$d_{ca.mid} := d_{ca} = 17.688 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement of the wall at midspan(in.)

$A_{va1.end} := 0.31in^2$ — Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 4in$ — Average Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := d_{ca} = 17.688 \cdot in$ — Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

SUBJECT: **MnDOT G-Barrier on Bridge No. 86812**
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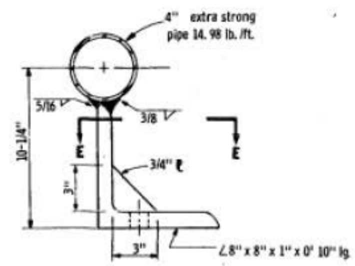
(1b) Steel Rail, Post, and Anchor Rod Inputs:

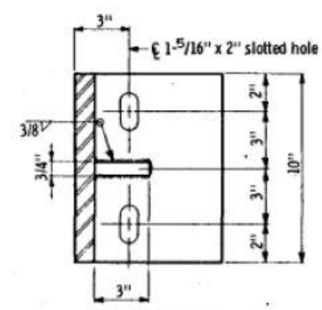
Steel Rail Inputs:
a) Steel Tube Rail is M.H.D. 3362 material, Fy=35ksi
b) Steel Tube Rail is a 4" extra strong pipe



POST DETAIL

$F_{yR} := 35ksi$	Yield Strength of Steel Tube Rail (ksi)
$d_{oR} := 4.5in$	Outside diameter of Steel Tube Rail (in.)
$d_{iR} := 3.83in$	Inside diameter of Steel Tube Rail (in.)

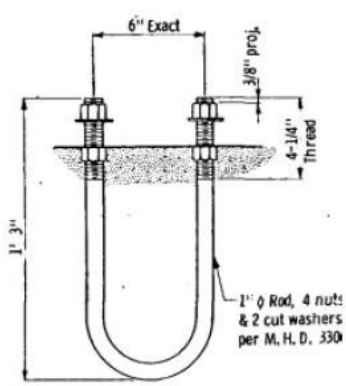
Steel Post Inputs:
a) Steel Post is M.H.D. 3306 material, Fy= 36ksi
b) Steel Post is a 8"x8"x1"x10" Angle Member



SECTION E-E

$F_{yp} := 36ksi$	Yield Strength of Steel Post (ksi)
$w_p := 10in$	Width of Steel Post about the bending axis (in.)
$t_p := 1in$	Thickness of Steel Post (in.)
$h_p := 10.25in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$L_p := 8.5ft$	Steel Post Spacing (ft.)

Anchor Rod Inputs:
a) Anchor Rods are F1554 Gr. 36 material, Fu=58 ksi
b) Anchor Rods are 1"ϕ x 15" U bolt



RAILPOST ANCHORAGE

$F_{u.rod} := 58ksi$	Tensile Strength of Anchor Rods (ksi)
$N_{rod} := 1$	Number of Anchor Rods
$N_{rod.tension} := 2$	Number of Anchor Rods in Tension
$d_{rod} := 5.0in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$\phi_{rod} := 1in$	Diameter of Anchor Rods (in)

(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ — Test Level

$F_t := 71kip$ — Transverse Impact Force

$L_t := 4ft$ — Longitudinal Length of Distribution of Impact Force

$H_e := 19in$ — Height of Equivalent Transverse Load

$H_{min} := 29in$ — Minimum height of a MASH TL-3 barrier (in.)

$H_r = 40.625 \cdot in$ — Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

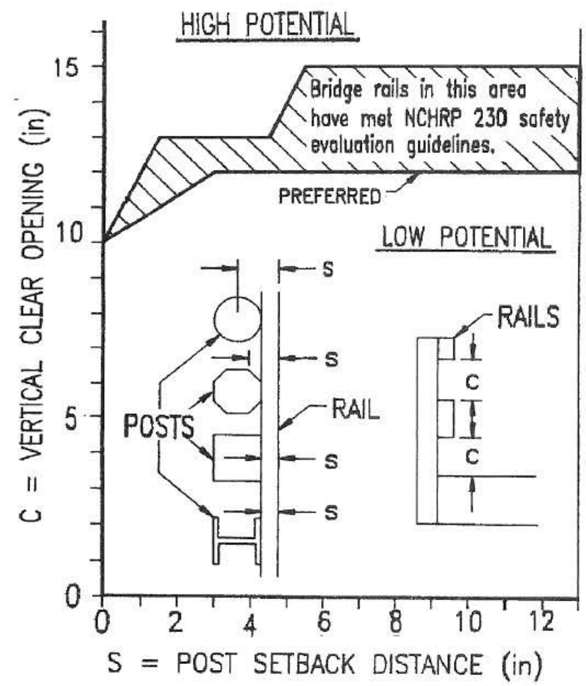
$H_r = 40.625 \cdot in$ Total height of bridge rail system measured from the top of the roadway surface/overlay to the top of highest rail (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_r \geq H_{min} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

Minimum_Height_of_Barrier_Check = "OK"

(4) Geometric Criteria:

$S_{post} := 1.75in$	Post Setback (in.) Note: Denoted as "S" in figure below.
$C_b := 10.375in$	Vertical Clear Opening (in.)
$\Sigma A := H_w + \frac{d_{oR}}{2} = 30.25 \cdot in$	Total Rail Contact Width (in.)
$H_r = 40.625 \cdot in$	Total height of the bridge rail measured from the top of the roadway surface/overlay (in.) Note: Denoted as "H" in figure below.



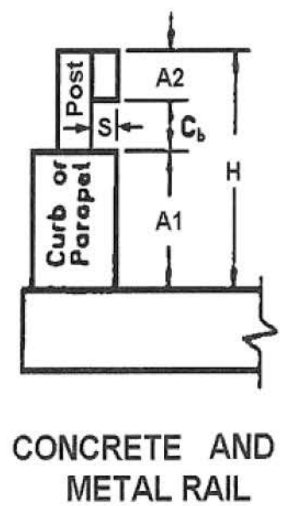


Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post

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SUBJECT: MnDOT G-Barrier on Bridge No. 86812
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(3-conti.) Geometric Criteria:

$S_{post} = 1.75 \cdot in$ $\Sigma A = 30.25 \cdot in$ $H_r = 40.625 \cdot in$ $ratio_{\Sigma AH} := \frac{\Sigma A}{H_r} = 0.745$

$Set_{low.x} := (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{low.y} := (0.75\ \ 0.63\ \ 0.52\ \ 0.4\ \ 0.315\ \ 0.28\ \ 0.27\ \ 0.26\ \ 0.25\ \ 0.245\ \ 0.245)$

Lower Boundary for Post Setback Criteria x and y coordinates

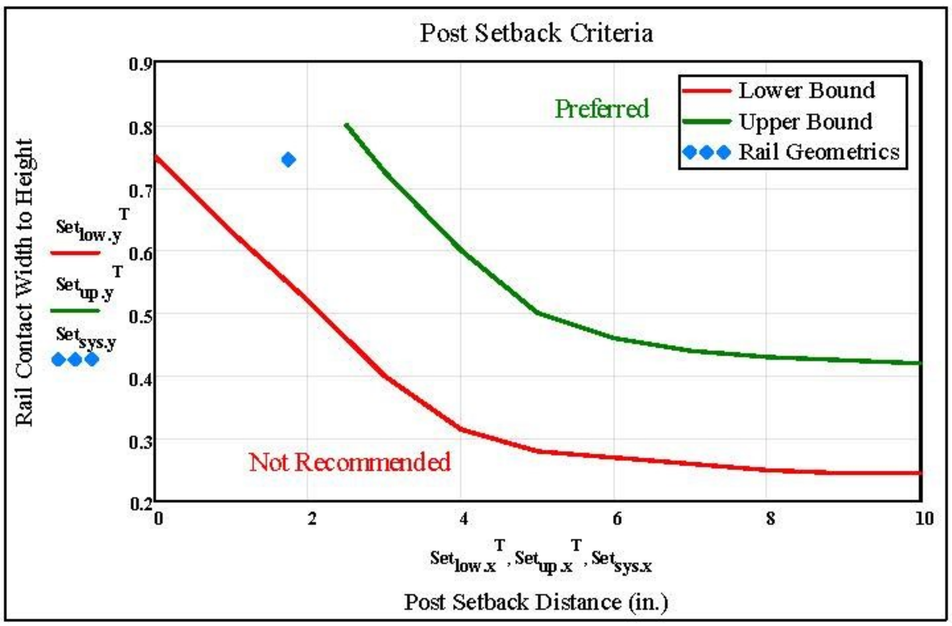
$Set_{up.x} := (2.5\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10)$

$Set_{up.y} := (0.8\ \ 0.725\ \ 0.6\ \ 0.5\ \ 0.46\ \ 0.44\ \ 0.43\ \ 0.425\ \ 0.42)$

Upper Boundary for Post Setback Criteria x and y coordinates

$Set_{sys.x} := \frac{S_{post}}{in} = 1.75$ Post Setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.745$ Ratio of Contact Width to Total Height rail geometric point



$NotRecommended := 1$ $Marginal := 2$ $Preferred := 3$

Region Designation
Note: Marginal region is between Lower and Upper Bounds

$Post\_Setback\_Criteria\_Rail\_Geometric\_Point := Marginal$

SUBJECT: MnDOT G-Barrier on Bridge No. 86812
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MASH TL-3 Compliance Assessment

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(3-conti.) Geometric Criteria:

$snag_{low.x} := (0 \;\; 3 \;\; 13)$

$snag_{low.y} := (10 \;\; 12 \;\; 12)$

Lower Boundary for Snag Potential Criteria
x and y coordinates

$snag_{up.x} := (0 \;\; 1.25 \;\; 4.25 \;\; 5.25 \;\; 13)$

$snag_{up.y} := (10 \;\; 13 \;\; 13 \;\; 15 \;\; 15)$

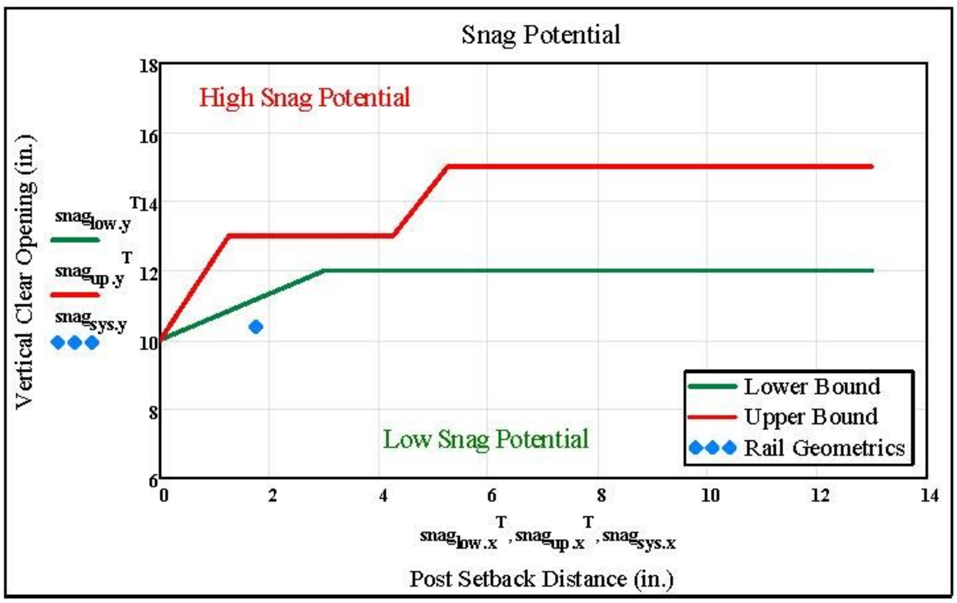
Upper Boundary for Snag Potential Criteria
x and y coordinates

$snag_{sys.x} := \frac{S_{post}}{in} = 1.75$

Post Setback rail geometric point

$snag_{sys.y} := \frac{C_b}{in} = 10.375$

Vertical Clear Opening rail geometric point



HighSnagPotential := 1 Marginal = 2 LowSnagPotential := 3

Region Designation
Note: Marginal region is between Lower and Upper Bounds

Snag_Potential_Criteria_Rail_Geometric_Point := LowSnagPotential

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SUBJECT: MnDOT G-Barrier on Bridge No. 86812
Figure 5-397.107
MASH TL-3 Compliance Assessment

(3) Geometric Criteria - Summary of Results:

$$\text{Post\_Setback\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if Post\_Setback\_Criteria\_Rail\_Geometric\_Point = Preferred} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Post\_Setback\_Criteria\_Check} = \text{"NOT OK"}$$

$$\text{Snag\_Potential\_Criteria\_Check} := \begin{cases} \text{"OK"} & \text{if Snag\_Potential\_Criteria\_Rail\_Geometric\_Point = LowSnagPotential} \\ \text{"NOT OK"} & \text{otherwise} \end{cases}$$

$$\text{Snag\_Potential\_Criteria\_Check} = \text{"OK"}$$

(4) LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications:

(4a) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$b_c := 12in$

Unit Width of Wall (in.)
Note: b_c is taken as 1ft per AASHTO Section 13 procedure

$A_{vp1.mid} = 0.31 \cdot in^2$

Area of one parapet vertical reinforcement leg in the tension zone (in^2)

$s_{vp.mid} = 12 \cdot in$

Spacing of parapet vertical reinforcement at midspan (in.)

$$A_{vp.mid} := \left(\frac{b_c}{s_{vp.mid}}\right) \cdot A_{vp1.mid} = 0.31 \cdot in^2$$

Total Area of parapet vertical reinforment per unit length of the wall at midspan (in^2)

$d_{cp.mid} = 10.188 \cdot in$

Average extreme distance of parapet vertical reinforcement in tension (in.)

$$a_{cp.mid} := \frac{A_{vp.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$$

Depth of Whitney Stress Block (in.)

$$M_{cp.mid} := \frac{\left[A_{vp.mid} \cdot f_y \cdot \left(d_{cp.mid} - \frac{a_{cp.mid}}{2}\right)\right]}{b_c} = 15.437 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

(4a-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Midspan: M_{cmid} (k-ft/ft)

$A_{va1.mid} = 0.31 \cdot in^2$

Area of one deck anchorage vertical reinforcement leg in the tension zone (in^2)

$s_{va.mid} = 12 \cdot in$

Spacing of deck anchorage vertical reinforcement at midspan (in.)

$A_{va.mid} := \left(\frac{b_c}{s_{va.mid}}\right) \cdot A_{va1.mid} = 0.31 \cdot in^2$

Total Area of deck anchorage vertical reinforment per unit length of the wall at midspan (in^2)

$d_{ca.mid} = 17.688 \cdot in$

Extreme distance of tension deck anchorage vertical reinforcement of the wall (in.)

$a_{ca.mid} := \frac{A_{va.mid} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 0.456 \cdot in$

Depth of Whitney Stress Block (in.)

$M_{ca.mid} := \frac{\left[A_{va.mid} \cdot f_y \cdot \left(d_{ca.mid} - \frac{a_{ca.mid}}{2}\right)\right]}{b_c} = 27.062 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$M_{cmid} := \min\left(M_{cp.mid}, M_{ca.mid}\right) = 15.437 \cdot \frac{kip \cdot ft}{ft}$

Flexural Resistance of the Wall about the Longitudinal Axis at Midspan when considering the critical reinforcment (k-ft/ft)

(4b) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$b_c = 12 \cdot in$ Unit Width of Wall (in.)

$A_{vp1.end} = 0.31 \cdot in^2$ Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} = 4 \cdot in$ Spacing of parapet vertical reinforcement at joints/ends (in.)

$A_{vp.end} := \left(\frac{b_c}{s_{vp.end}}\right) \cdot A_{vp1.end} = 0.93 \cdot in^2$ Total Area of parapet vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{cp.end} := \frac{A_{vp.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.368 \cdot in$ Depth of Whitney Stress Block (in.)

$d_{cp.end} = 10.188 \cdot in$ Average extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

$$M_{cp.end} := \frac{\left[A_{vp.end} \cdot f_y \cdot \left(d_{cp.end} - \frac{a_{cp.end}}{2}\right)\right]}{b_c} = 44.192 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends when considering only the parapet vertical reinforcment specified in Article A13.3.1 (k-ft/ft)

$A_{va1.end} = 0.31 \cdot in^2$ Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} = 4 \cdot in$ Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$A_{va.end} := \left(\frac{b_c}{s_{va.end}}\right) \cdot A_{va1.end} = 0.93 \cdot in^2$ Total Area of deck anchorage vertical reinforment per unit length of the wall at joints/ends (in^2)

$a_{ca.end} := \frac{A_{va.end} \cdot f_y}{0.85 \cdot f'_c \cdot b_c} = 1.368 \cdot in$ Depth of Whitney Stress Block (in.)

(4b-conti.) Bending Capacity of the Wall about the Longitudinal Axis at Joints/Ends: M_{cend}

$d_{ca.end} = 17.688 \cdot in$ — Extreme distance of tension anchorage vertical reinforcement at joints/ends (in.)

$$M_{ca.end} := \frac{\left[A_{va.end} \cdot f_y \cdot \left(d_{ca.end} - \frac{a_{ca.end}}{2}\right)\right]}{b_c} = 79.067 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering only the deck anchorage reinforcment specified in Article A13.3.1 (k-ft/ft)

$$M_{cend} := \min\left(M_{cp.end}, M_{ca.end}\right) = 44.192 \cdot \frac{kip \cdot ft}{ft}$$

Flexural Resistance of the Wall about the Longitudinal Axis at joints/ends when considering the critical reinforcment (k-ft/ft)

(4c) Bending Capacity of the Wall about the Vertical Axis: M_w

$d_w = 9.625 \cdot in$ — Average extreme distance of tension longitudinal reinforcement of wall (in.)

$A_w = 0.8 \cdot in^2$ — Total Area of longitudinal reinforcement bars acting in tension (in^2)

$h_w = 28 \cdot in$ — Total height of the barrier (in.)

$$a_w := \frac{A_w \cdot f_y}{0.85 \cdot f'_c \cdot h_w} = 0.504 \cdot in$$

Depth of the Whitney Stress Block (in.)

$$M_w := A_w \cdot f_y \cdot \left(d_w - \frac{a_w}{2}\right) = 37.492 \cdot kip \cdot ft$$

Flexural Resistance of the Wall about the Vertical Axis specified in Article A13.3.1 (k-ft)

(4d) Determine the Ultimate Resistance of the Wall at Midspan: R_{wmid}

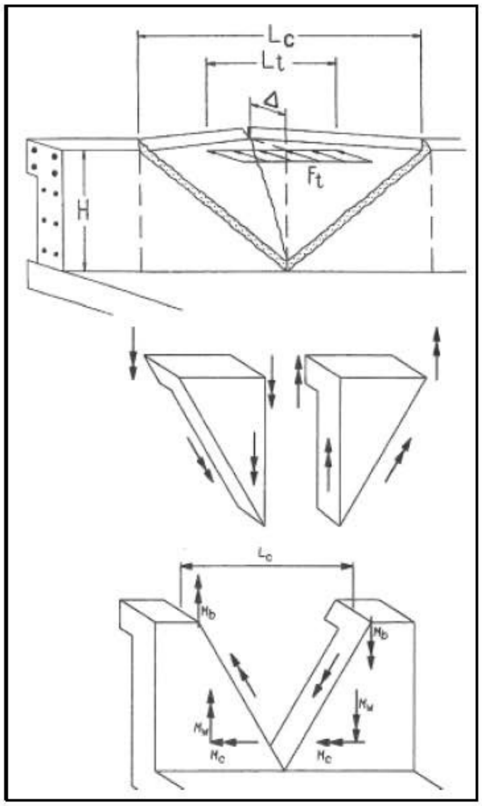

Figure 3d. Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment.

$H_w = 28 \cdot in$ — Height of the barrier measured from the top of the overlay or roadway surface (in.)
Note: H_w = H in Figure 3d

$M_B := 0 kip \cdot ft$ — No additional beam strength

$M_{cmid} = 15.437 \cdot \frac{kip \cdot ft}{ft}$ — Flex. Resistance of the Wall about the Long. Axis at Midspan (k-ft/ft)

$M_w = 37.492 \cdot kip \cdot ft$ — Flex. Resistance of the Wall about the Vert. Axis (k-ft)

$L_t = 4 \cdot ft$ — Longitudinal length of distribution of impact force (ft.)

$$L_{cmid} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{\left[8 \cdot h_w \cdot \left(M_B + M_w\right)\right]}{M_{cmid}}} = 9.024 \cdot ft$$ (Equation A13.3.1-2)

$$R_{wmid} := \left[\left(\frac{2}{2 \cdot L_{cmid} - L_t}\right) \cdot \left[8 \cdot M_B + 8 \cdot M_w + \frac{M_{cmid} \cdot \left(L_{cmid}\right)^2}{h_w}\right]\right] = 119.403 \cdot kip$$ (Equation A13.3.1-1)

(4e) Determine the Ultimate Resistance of the Wall at Joints/Ends: R_{wend}

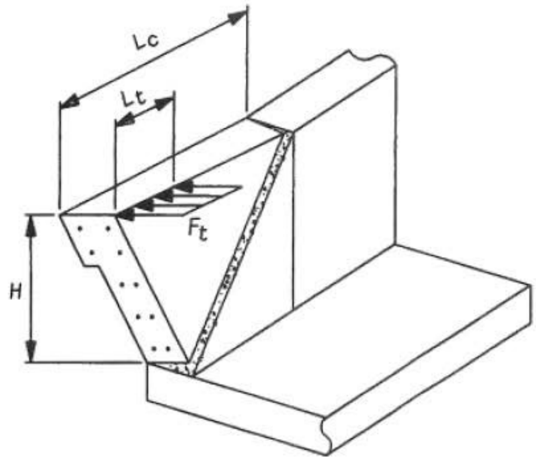

Figure 3e. Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

$H_w = 28 \cdot in$	Height of the barrier measured from the top of the overlay or roadway surface (in.) Note: H_w = H in Figure 3e
$M_B = 0$	No additional beam strength
$M_w = 37.492 \cdot kip \cdot ft$	Flex. Resistance of the Wall about the Vert. Axis (k-ft)
$L_t = 4 \cdot ft$	Longitudinal length of distribution of impact force (ft.)
$M_{cend} = 44.192 \cdot \frac{kip \cdot ft}{ft}$	Flexural Resistance of the Wall about the Longitudinal Axis at Joints/Ends specified in Article A13.4.2 (k-ft/ft)

$$L_{cend} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + h_w \cdot \left(\frac{M_B + M_w}{M_{cend}}\right)} = 4.445 \cdot ft$$ (Equation A13.3.1-4)

$$R_{wend} := \left(\frac{2}{2 \cdot L_{cend} - L_t}\right) \cdot \left[M_B + M_w + \frac{\left(M_{cend} \cdot L_{cend}^{\ 2}\right)}{h_w}\right] = 168.384 \cdot kip$$ (Equation A13.3.1-3)

(4f) Steel Rail & Post Strength Analysis:

$F_{yR} = 35 \cdot ksi$	Yield Strength of Steel Tube Rail (ksi)
$d_{oR} = 4.5 \cdot in$	Outside diameter of Steel Tube Rail (in.)
$d_{iR} = 3.83 \cdot in$	Inside diameter of Steel Tube Rail (in.)
$Z_R := \frac{\left(d_{oR}^3 - d_{iR}^3\right)}{6} = 5.824 \cdot in^3$	Plastic Sectional Modulus of the Steel Tube Rail (in^3)
$M_p := F_{yR} \cdot Z_R = 16.986 \cdot kip \cdot ft$	Plastic Moment Capacity of the Steel Tube Rail (kip-ft)

Calculate the Plastic Strength of the Post: P_{P1}

$w_p = 10 \cdot in$	Width of Steel Post about the bending axis (in.)
$t_p = 1 \cdot in$	Thickness of Steel Post (in.)
$Z_p := \frac{w_p \cdot t_p^2}{4} = 2.5 \cdot in^3$	Plastic Sectional Modulus of the Steel Post about the bending axis (in.)
$F_{yp} = 36 \cdot ksi$	Yield Strength of Steel Post (ksi)
$M_{post} := F_{yp} \cdot Z_p = 7.5 \cdot kip \cdot ft$	Plastic Moment Capacity of the Steel Post (kip-ft)
$h_p = 10.25 \cdot in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$P_{P1} := \frac{M_{post}}{h_p} = 8.78 \cdot kip$	Post Strength based on the Plastic Failure of a Steel Post (kip)

Calculate the Post Strength based on the Ultimate Strength of the Anchor Rods: P_{P2}

$F_{u.rod} = 58 \cdot ksi$	Tensile Strength of the Anchor Rods (ksi)
$\phi_{rod} = 1 \cdot in$	Diameter of Anchor Rods (in)
$A_{rod} := \frac{\pi}{4} \cdot \phi_{rod}^{\;2} = 0.785 \cdot in^2$	Area of a Anchor Rod (in^2)
$R_{nt} := F_{u.rod} \cdot (0.75 \cdot A_{rod}) = 34.165 \cdot kip$	Nominal strength of one Anchor Rod in Tension (kip)
$N_{rod.tension} = 2$	Number of Anchor Rods acting in tension
$d_{rod} = 5 \cdot in$	Distance from the anchor rods acting in tension to the back of the steel plate (in.)
$d_b := 1.5in$	Length of the steel plate bearing pressure acting on the concrete parapet (in.)
$w_{rod} := d_{rod} - \frac{d_b}{3} = 4.5 \cdot in$	Distance from anchor rods acting in tension to the centroid of the bearing pressure acting on the concrete parapet (in.)
$M_{t.rod} := w_{rod} \cdot R_{nt} \cdot N_{rod.tension} = 25.624 \cdot kip \cdot ft$	Moment strength of Post based on tensile capacity of Anchor Rods (k-ft)
$h_p = 10.25 \cdot in$	Height from the bottom of the post to the centroid of the steel tube rail (in.)
$P_{t.rod} := \frac{M_{t.rod}}{h_p} = 29.998 \cdot kip$	Post Strength based on the tensile capacity of Anchor Rods (kip)
$R_{nv} := F_{u.rod} \cdot (0.45 \cdot A_{rod}) = 20.499 \cdot kip$	Nominal strength of one anchor rod in Shear w/h threads in shear plane (kip)
$P_{v.rod} := N_{rod} \cdot R_{nv} = 20.499 \cdot kip$	Post Strength based on the shear capacity of Anchor Rods (kip)
$P_{P2} := \min(P_{t.rod}, P_{v.rod}) = 20.499 \cdot kip$	Post Strength based on the Ultimate Strength of the Anchor Rods (kip)

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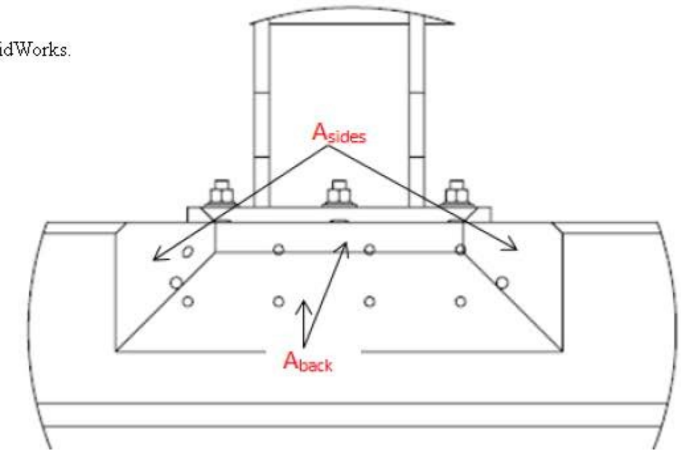
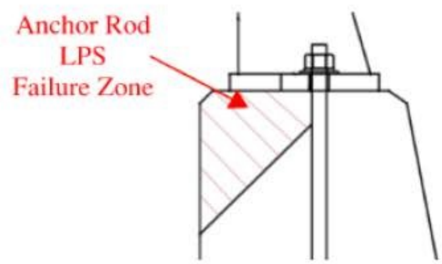
Calculate the Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods: P_{P3}

Note: This failure mechanism was modeled in SolidWorks.

$A_{sides} := 59in^2$

$A_{back} := 125in^2$





$\phi_v := 0.75$ Shear Strength Reduction Factor

$A_{LPS} := A_{sides} + A_{back} = 184 \cdot in^2$ Total Area of Failure Planes due to Lateral Punching Shear Failure (in^2)

$f'_c = 4 \cdot ksi$ Concrete Compressive Strength (ksi)

$V_{c.lat} := \phi_v \cdot 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi = 94.87 \cdot psi$ Concrete Stress from Block Shear of Anchor Rods (psi)
-ACI 318-14 Eqn. 22.5.5.1

$P_{P3} := V_{c.lat} \cdot A_{LPS} = 17.456 \cdot kip$ Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)

Determine the Limiting ("Worst Case") Post Strength (kips): P_P

$P_{P1} = 8.78 \cdot kip$	Post Strength based on the Plastic Failure of a Steel Post (kip)
$P_{P2} = 20.499 \cdot kip$	Post Strength based on the Ultimate Strength of the Anchor Rods (kip)
$P_{P3} = 17.456 \cdot kip$	Post Strength based on the Lateral Punching Shear Resistance of Concrete from Traffic Side Anchor Rods (kip)
$P_P := \min(P_{P1}, P_{P2}, P_{P3}) = 8.78 \cdot kip$	Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

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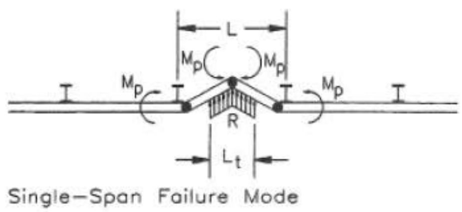
Determine Ultimate Capacity of Steel Rail for a Single and Double Span Failure Mode:

$L_t = 4 \cdot ft$ — Length of the Distribution of the Transverse Impact Force (ft.)

$L_p = 8.5 \cdot ft$ — Steel Post Spacing (ft.)

$M_p = 16.986 \cdot kip \cdot ft$ — Flexural Capacity of the Steel Tube Rail (kip-ft)

$P_p = 8.78 \cdot kip$ — Post Strength found by using the Limiting ("Worst Case") Post Strength (kips)

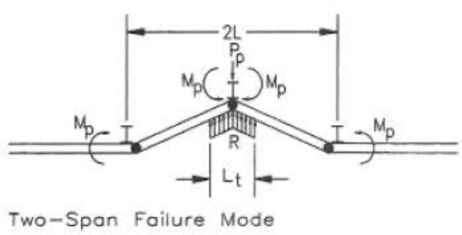


Single–Span Failure Mode

$N_1 := 1$ — Number of Spans

$$R_R := \frac{(16 \cdot M_p) + \left[(N_1 - 1)(N_1 + 1) \cdot P_p \cdot L_p\right]}{(2 \cdot N_1 \cdot L_p) - L_t} = 20.906 \cdot kip$$

Ultimate Capacity of rail over one span (kip)



Two–Span Failure Mode

$N_2 := 2$ — Number of Spans

$$R_{R'} := \frac{16 \cdot M_p + \left(N_2^{\,2} \cdot P_p \cdot L_p\right)}{(2 N_2 \cdot L_p) - L_t} = 19.011 \cdot kip$$

Ultimate Capacity of rail over two spans (kip)

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(4g) Determine the Combined Resultant Strength of the Bridge Rail System:

Determine the Resultant Strength of the Bridge Rail System at Midspan: (R_1)

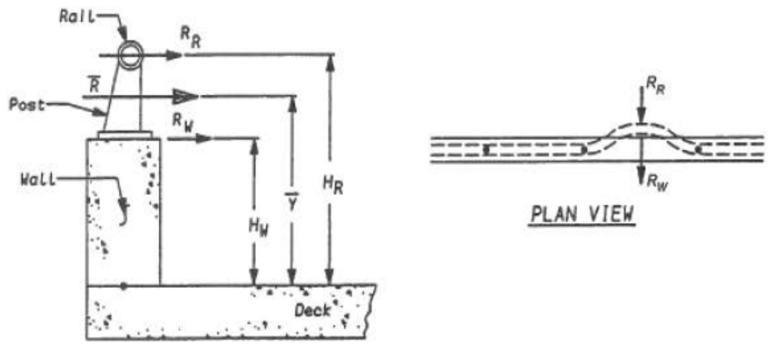

Figure A13.3.3-1—Concrete Wall and Metal Rail Evaluation—Impact at Midspan of Rail

$R_R = 20.906 \cdot kip$	Ultimate Capacity of rail over one span (kip)
$H_R = 38.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 119.403 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$R_{bar1} := R_R + R_{wmid} = 140.31 \cdot kip$	Resultant Strength of the Bridge Rail System Located at y_{bar1} AASHTO Eqn. A13.3.3-1
$y_{bar1} := \dfrac{R_R \cdot H_R + R_{wmid} \cdot H_w}{R_{bar1}} = 29.546 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-2
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load
$R_1 := R_{bar1} \cdot \left(\dfrac{y_{bar1}}{H_e}\right) = 218.188 \cdot kip$	Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

Determine the Resultant Strength of the Bridge Rail System at a Post: (R_2)

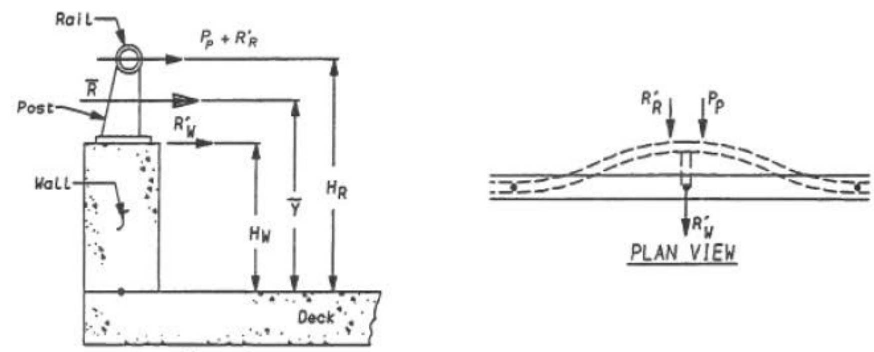

Figure A13.3.3-2—Concrete Wall and Metal Rail Evaluation—Impact at Post

$R_{R'} = 19.011 \cdot kip$	Ultimate Capacity of rail over two spans (kip)
$H_R = 38.375 \cdot in$	Height of the bridge rail system measured from the top of the roadway surface/overlay to the centroid of the steel tube rail (in.)
$R_{wmid} = 119.403 \cdot kip$	Ultimate Resistance of the concrete parapet at midspan (kip)
$H_w = 28 \cdot in$	Height of the concrete parapet measured fom the top of the roadway surface/overlay (in.)
$R_{w'} := \dfrac{R_{wmid} \cdot H_w - P_P \cdot H_R}{H_w} = 107.37 \cdot kip$	Reduced Wall Strength (kip) AASHTO Eqn. A13.3.3-5
$R_{bar2} := P_P + R_{R'} + R_{w'} = 135.161 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at y_{bar2} AASHTO Eqn. A13.3.3-3
$y_{bar2} := \dfrac{H_R \cdot (P_P + R_{R'}) + R_{w'} \cdot H_w}{R_{bar2}} = 30.133 \cdot in$	Effective Height of the Resultant Strength (in.) AASHTO Eqn. A13.3.3-4
$H_e = 19 \cdot in$	Height of Equivalent Transverse Load (in.)
$R_2 := R_{bar2} \cdot \left(\dfrac{y_{bar2}}{H_e}\right) = 214.359 \cdot kip$	Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

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(4) **LRFD Strength Analysis of the Barrier per AASHTO Section 13 Specifications - Summary of Results:**

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$R_1 = 218.188 \cdot kip$ — Resultant Strength of the Bridge Rail System at midspan located at H_e (kip)

Structural_Capacity_of_Barrier_at_Midspan_Check := | "OK" if $R_1 \geq F_t$
| "NOT OK" otherwise

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

$R_2 = 214.359 \cdot kip$ — Resultant Strength of the Bridge Rail System at a post located at H_e (kip)

Structural_Capacity_of_Barrier_at_a_Post_Check := | "OK" if $R_2 \geq F_t$
| "NOT OK" otherwise

Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

(5) Strength Analysis of the Seperate End Post:

(5a) Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

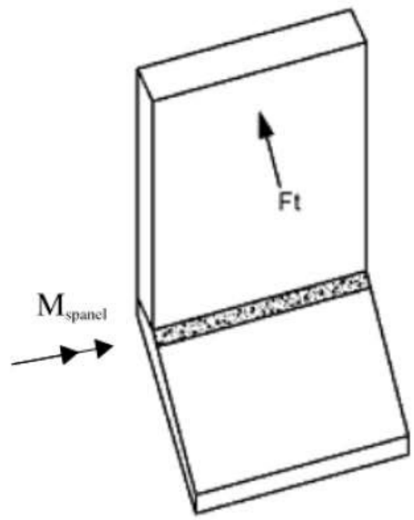

Figure 4a. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 41in$ — Height of the end post measured from the top of the roadway/surface (in.)

$b_{spost} := 18in$ — Width of the end post (in.)

Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:
Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{p1.spost} := 1in^2$ — Area of one parapet vertical reinforcement leg in the tension zone of the end post (in^2)
2~#9 Bars

$n_{p.spost} := 2$ — Number of parapet vertical reinforcement in the end post (in.)

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(5a-conti.) Bending Capacity of the End Post about the Longitudinal Axis: M_{spost}

$A_{p.spost} := n_{p.spost} \cdot A_{p1.spost} = 2 \cdot in^2$ — Total Area of parapet vertical reinforment in the tension zone of the end post (in^2)

$a_{p.spost} := \dfrac{A_{p.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 1.961 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{p.spost} := 15in$ — extreme distance of tension parapet vertical reinforcement in the end post (in.)

$M_{p.spost} := A_{p.spost} \cdot f_y \cdot \left(d_{p.spost} - \dfrac{a_{p.spost}}{2}\right) = 140.196 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the parapet vertical reinforcment (kip-ft)

Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:

Note: See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 1.56in^2$ — Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2) #11 Bars

$n_{a.spost} := 2$ — Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 3.12 \cdot in^2$ — Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 3.059 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 20.75in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 299.841 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 140.196 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

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SUBJECT: MnDOT G-Barrier on Bridge No. 86812
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(5a) **Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:**

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t (in.)

$M_{s.post} = 140.196 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \frac{M_{s.post}}{H_e} = 88.545 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force located at H_e (kip)

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} := \left(\begin{array}{ll} \text{"OK"} & \text{if } R_{s.post} \geq F_t \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right)$$

$$\text{Structural\_Capacity\_of\_End\_Post\_Check} = \text{"OK"}$$

(6) Conclusions:

Minimum_Height_of_Barrier_Check = "OK"

Post_Setback_Criteria_Check = "NOT OK"

Snag_Potential_Criteria_Check = "OK"

Structural_Capacity_of_Barrier_at_Midspan_Check = "OK"

Structural_Capacity_of_Barrier_at_a_Post_Check = "OK"

Structural_Capacity_of_End_Post_Check = "OK"

The G-Barrier on Bridge No. 86812 (Fig. 5-397.107) has <u>not satisfied</u> all MASH TL-3 Criteria

APPENDIX E: ANALYSIS CALCULATIONS FOR RETROFIT DESIGNS

APPENDIX E1: ANALYSIS CALCULATIONS FOR END POST RETROFIT WITH DRILLED SHAFT DESIGN

Given the Proposed Design Details of New End Post with Drilled Shaft:

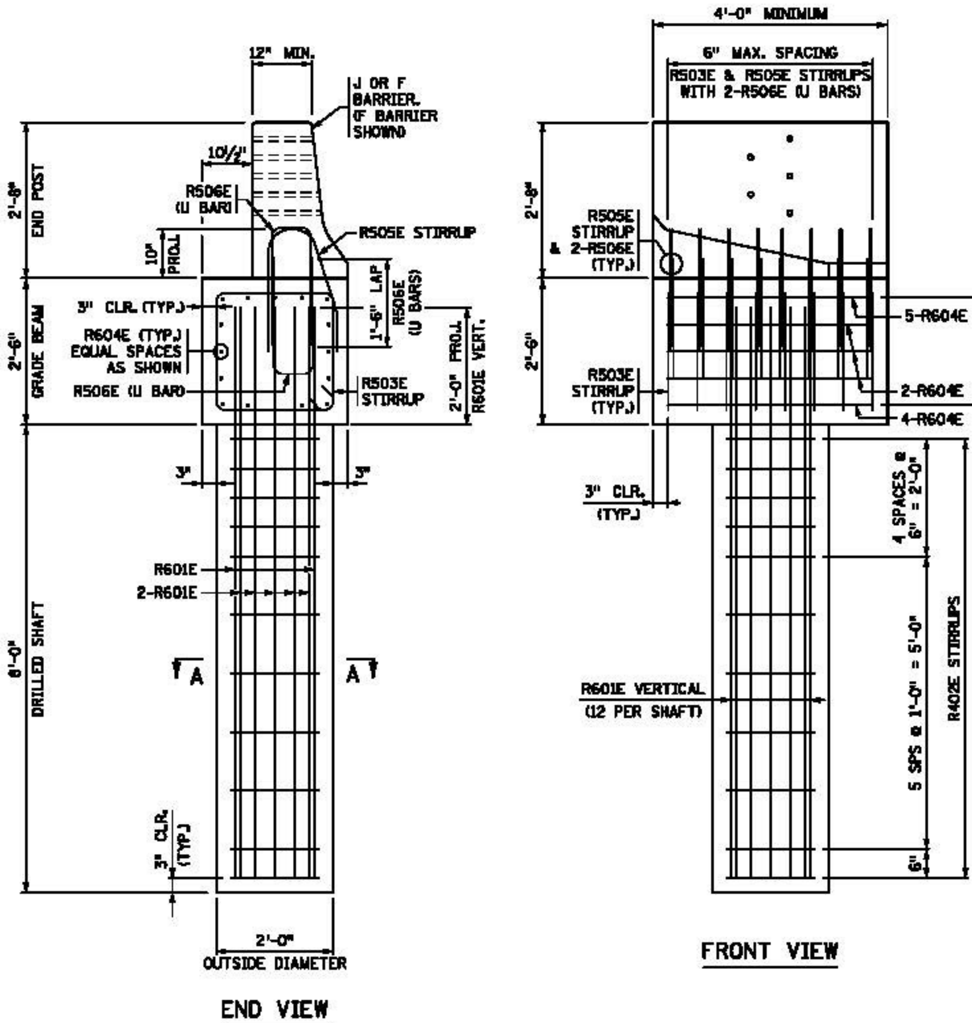

Figure 1. Design Details of New End Post

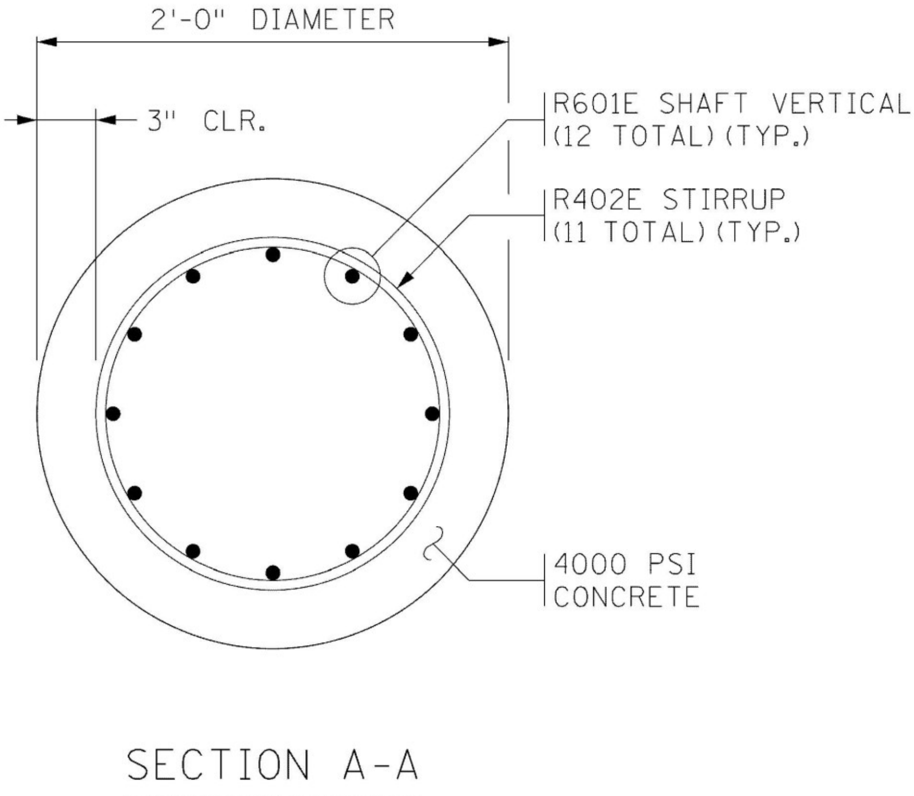

Figure 2. Post details

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.116
MASH TL-3 Compliance Assessment

(1) General Information and Inputs:

1) Reference: AASHTO MASH Conditions.
2) Assess the adequacy of the barrier based on AASHTO LRFD Section 13 criteria.

(1a) General Inputs:

$f'_c := 4000 \cdot psi$	Compressive Strength of Concrete (psi)
$f_y := 60ksi$	Yield Strength of Concrete Reinforcing Steel, (ksi)
$E_s := 29000ksi$	Modulus of Elasticity of Steel (ksi)
$H_w := 32in$	Height of the concrete barrier measured from the top of the roadway surface (in.)
$t_o := 0in$	Thickness of overlay (in)
$h_w := H_w + t_o$	Total height of the barrier (in.)

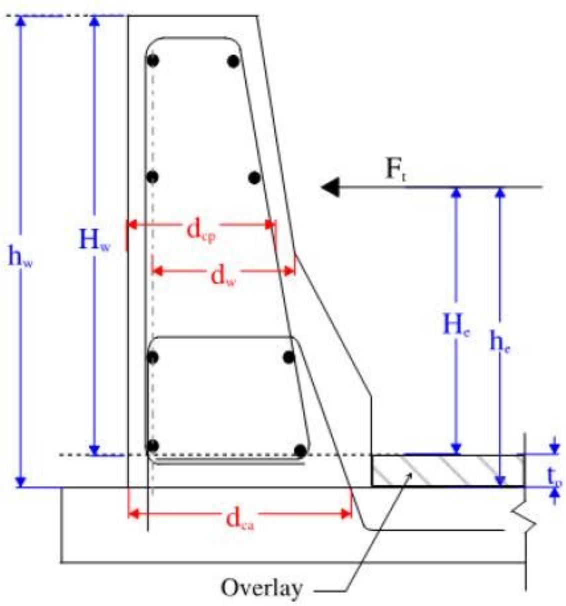

Figure 3. Sketch of Concrete Wall/Parapet Showing Input Variables

(1b) Concrete Parapet Inputs:

Typical Parapet Vertical Reinforcement Inputs:

***** End Post Parameters *****

$A_{vp1.end} := 0.31in^2$ Area of one parapet vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{vp.end} := 6in$ Spacing of parapet vertical reinforcement at joints/ends (in.)

$d_{cp.end} := 11.0in$ Extreme distance of tension parapet vertical reinforcement at joints/ends (in.)

Deck Anchorage Vertical Reinforcement Inputs:

***** End Post Parameters *****

$A_{va1.end} := 0.31in^2$ Area of one deck anchorage vertical reinforcement leg in the tension zone at joints/ends (in^2)

$s_{va.end} := 6in$ Spacing of deck anchorage vertical reinforcement at joints/ends (in.)

$d_{ca.end} := 11.0in$ Extreme distance of tension deck anchorage vertical reinforcement at joints/ends (in.)

SUBJECT: **MnDOT J-Barrier**
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(1c) **Design Force Inputs:**

Design Forces for Traffic Railings

Test Level	Rail Height (in.)	F_t (kip)	F_L (kip)	F_v (kip)	L_t/L_L (ft)	L_v (ft)	H_e (in)	H_{min} (in)
TL-1	18 or above	13.5	4.5	4.5	4.0	18.0	18.0	18.0
TL-2	18 or above	27.0	9.0	4.5	4.0	18.0	20.0	18.0
TL-3	29 or above	71.0	18.0	4.5	4.0	18.0	19.0	29.0
TL-4 (a)	36	68.0	22.0	38.0	4.0	18.0	25.0	36.0
TL-4 (b)	between 36 and 42	80.0	27.0	22.0	5.0	18.0	30.0	36.0
TL-5 (a)	42	160.0	41.0	80.0	10.0	40.0	35.0	42.0
TL-5 (b)	greater than 42	262.0	75.0	160.0	10.0	40.0	43.0	42.0
TL 6		175.0	58.0	80.0	8.0	40.0	56.0	90.0

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

$TL := 3$ Test Level

$F_t := 71\text{kip}$ Transverse Impact Force

$L_t := 4\text{ft}$ Longitudinal Length of Distribution of Impact Force

$H_e := 19\text{in}$ Height of Equivalent Transverse Load from top of Asphalt

$h_e := H_e + t_o$ Total equivalent transverse impact height (in.)

$H_{min} := 29\text{in}$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot \text{in}$ Height of the concrete barrier measured from the top of the roadway surface/asphalt overlay (in.)

(2) Stability Criteria:

$H_{min} = 29 \cdot in$ Minimum height of a MASH TL-3 barrier (in.)

$H_w = 32 \cdot in$ Height of the concrete barrier measured from the top of the roadway surface/asphalt overlay (in.)

$$\text{Minimum\_Height\_of\_Barrier\_Check} := \begin{cases} \text{"OK"} & \text{if } H_w \geq H_{min} \\ \text{"NOT OKAY"} & \text{otherwise} \end{cases}$$

$$\text{Minimum\_Height\_of\_Barrier\_Check} = \text{"OK"}$$

AASHTO LRFD EQUATIONS

(3) Strength Analysis of the Seperate End Post:

Bending Capacity of the End Post about the Longitudinal Axis: $M_{s.post}$

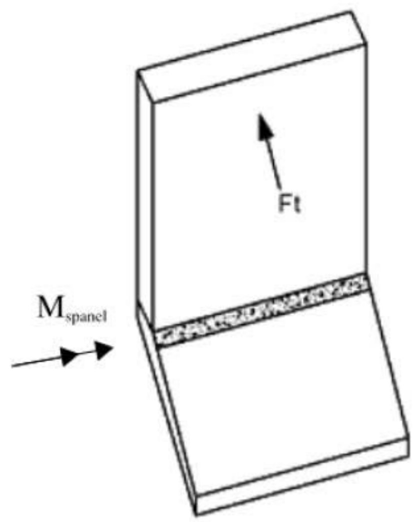

Figure 4. Flexural Strength Analysis of the End Post about the Longitudinal Axis.

$H_{spost} := 32in$ Height of the end post measured from the top of the roadway/asphalt surface (in.)

$b_{spost} := 48in$ Width of the end post (in.)

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(3a-conti.) <u>Bending Capacity of the End Post about the Longitudinal Axis</u>: $M_{s.post}$

<u>Bending Capacity of End Post Considering only the Deck Anchorage Vertical Reinforcement:</u>

<u>Note:</u> See *Figure 1* for a visual representation of the reinforcement bars.

$A_{a1.spost} := 0.31 in^2$ — Area of one anchorage vertical reinforcement leg in the tension zone in the end post (in^2) (#5 Bars are used)

$n_{a.spost} := 8$ — Number of anchorage vertical reinforcement in the end post (in.)

$A_{a.spost} := n_{a.spost} \cdot A_{a1.spost} = 2.48 \cdot in^2$ — Total Area of deck anchorage vertical reinforment in the tension zone of the end post (in^2)

$a_{a.spost} := \dfrac{A_{a.spost} \cdot f_y}{0.85 \cdot f'_c \cdot b_{spost}} = 0.912 \cdot in$ — Depth of the Whitney Stress Block (in.)

$d_{a.spost} := 10.625 in$ — Extreme distance of tension deck anchorage vertical reinforcement in the end post (in.)

$M_{a.spost} := A_{a.spost} \cdot f_y \cdot \left(d_{a.spost} - \dfrac{a_{a.spost}}{2}\right) = 126.097 \cdot kip \cdot ft$ — Flexural Capacity of the End Post about the Longitudinal Axis when considering only the deck anchorage vertical reinforcment (kip-ft)

<u>Bending Capacity of End Post Considering only the Parapet Vertical Reinforcement:</u>

<u>Note:</u> See *Figure 1* for a visual representation of the reinforcement bars.

Since the same steel reinforcement is provided at the same spacing for both deck and parapet, the bending capacity shall be the same and this calculation is unnecessary

$M_{p.spost} := M_{a.spost} = 126.097 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$M_{s.post} := \min\left(M_{p.spost}, M_{a.spost}\right) = 126.097 \cdot kip \cdot ft$

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(3a) Bending Capacity of the End Post about the Longitudinal Axis-Summary of Results:

$H_e = 19 \cdot in$ — Height of the Transverse Impact Force, F_t(in.)

$M_{s.post} = 126.097 \cdot kip \cdot ft$ — Flexural Resistance of the End Post about the Longitudinal Axis when considering the critical reinforcment (k-ft/ft)

$R_{s.post} := \dfrac{M_{s.post}}{H_e} = 79.64 \cdot kip$ — Structural Capacity of the End Post located at H_e (kip)

$F_t = 71 \cdot kip$ OK! — Transverse Impact Force located at H_e (kip)

$L_t = 4 \cdot ft$ — Distribution Length of the Impact Force (ft.)

$$Structural\_Capacity\_of\_End\_Post\_Check := \left(\begin{array}{l} \text{"OK"} \quad \text{if } R_{s.post} \geq F_t \\ \text{"NOT OK"} \quad \text{otherwise} \end{array} \right)$$

$$Structural\_Capacity\_of\_End\_Post\_Check = \text{"OK"}$$

(4) Shear Capacity of the Barrier:

$\lambda := 1.0$ — Concrete Modification Factor

$T_w := 12in$ — Top Width of the parapet (in.)

$h_c := 15in$ — Depth of the shear zone at the critical segment (top most portion) of the barrier (in.)

$d_c := 10in$ — Distance from compression face to the tension reinforcement (in.)

$L_t = 4 \cdot ft$ — Length of the distribution of the impact force (ft.)

$f'_c = 4 \cdot ksi$ — Concrete parapet compressive strength (ksi)

$\beta := 2.0$ — factor for shear; β=2.0 is used for nonprestressed section (AASHTO 5.8.3.4.1)

(4a) Shear Capacity of an End Segment of the Barrier: V_{cend}

$$A_{c.end} := \left(L_t + \frac{d_c}{2}\right) \cdot T_w = 636 \cdot in^2$$

Concrete Parapet Shear Zone Area of an End Segment of the Barrier (in^2)

$$V_{c.end} := 0.0316\beta \cdot \lambda \cdot \left[\left(\sqrt{\frac{f'_c}{ksi}}\right) \cdot ksi\right] \cdot A_{c.end} = 80.39 \cdot kip$$

One Way Shear Capacity of an End Segment of the Barrier (kip) (AASHTO 5.8.3.3)

$$V_c := \min\left(V_{c.end}\right) = 80.39 \cdot kip$$

Critical Shear Capacity of the Barrier (kip)

$F_t = 71 \cdot kip$ — Transverse Impact Force (kip)

$$Shear\_Capacity\_of\_Barrier\_Check := \left(\begin{array}{|l} \text{"OK"} \quad \text{if } V_c \ge F_t \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array} \right) = \text{"OK"}$$

(5) Conclusions of End Post & Barrier Design:

Minimum_Height_of_Barrier_Check = "OK"

Structural_Capacity_of_End_Post_Check = "OK"

Shear_Capacity_of_Barrier_Check = "OK"

The J-Barrier from Figure 5-397.116 does satisfy all MASH TL-3 Criteria

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SUBJECT: **MnDOT J-Barrier**
Figure 5-397.116
MASH TL-3 Compliance Assessment

6.) Design Drilled Shaft for Bending capacity & size reinforcing steel:

Data from Basic Limits Table C4-60, "Design Handbook Volume 2 Columns"
In Accordance with ACI 318-83 page 152, **** Circular Pier/Column *********

$f'_c := 4ksi$ — Compressive Strength of Concrete

$f_y := 60ksi$ — Yield Strength of rebar

$h := 24in$ — Diameter of pier

$cover := 3in$ — Clear cover

$A_g := \frac{\pi h^2}{4}$ — Gross Area (in^2)

$\phi := 1.0$ — Strength Reduction Factor

$Tie_{dia} := \frac{4}{8} in$ — Tie/Spiral Diameter

$Vertical_{dia} := \frac{6}{8} in$ — Size of Vertical Steel

$No_{verts} := 12$ — Number of Vertical Bars

$A_{st} := \frac{\pi (Vertical_{dia})^2}{4} \cdot No_{verts} = 5.301 \cdot in^2$ — Total Area of Steel Provided (in^2)

$\rho_{act} := \frac{A_{st}}{A_g} = 0.012$ — Reinforcement ratio "ρ"

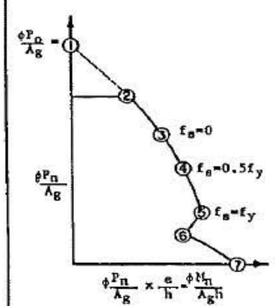
$\gamma h := h - 2 \cdot cover - 2 \cdot Tie_{dia} - Vertical_{dia} = 16.25 \cdot in$ — Distance between the outer layers of reinforcement in a column

$\gamma := \frac{\gamma h}{h} = 0.677$ — Ratio of the distance between the outer layers of reinforcement in a column to the overall depth of the column

6-conti.) Design Drilled Shaft for Bending capacity & size reinforcing steel:

COLUMNS 8.22—Basic limits of factored axial load and factored moment for C4-60 columns (Design load and moment strengths)

References: ACI 318-83, Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182

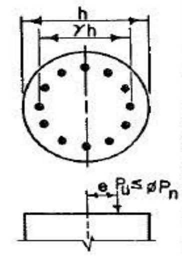


Coordinates tabulated below provide more precise values for critical points on corresponding interaction diagrams (COLUMNS 7).

units: $\frac{\phi P_n}{A_g}$, ksi $\frac{\phi M_n}{A_g h}$, ksi

BASIC LIMITS TABLE C4-60

Circular spiral column



$f_c' = 4$ ksi

$f_y = 60$ ksi

ρ_g	② maximum P_u/A_g $\frac{\phi P_n}{A_g}$	② $\frac{\phi M_n}{A_g h}$	③ $f_s = 0$ $\frac{\phi P_n}{A_g}$	③ $\frac{\phi M_n}{A_g h}$	④ $f_s = 0.5f_y$ $\frac{\phi P_n}{A_g}$	④ $\frac{\phi M_n}{A_g h}$	⑤ $f_s = f_y$ $\frac{\phi P_n}{A_g}$	⑤ $\frac{\phi M_n}{A_g h}$	⑤ $\frac{e}{h}$	⑥ ϕ changes $\frac{\phi P_n}{A_g}$	⑥ $\frac{\phi M_n}{A_g h}$	⑦ $P_u = 0$ $\frac{\phi M_n}{A_g h}$
C4-60.45 γ = 0.45												
.01	2.528	.140	1.829	.271	1.171	.297	.724	.278	.384	.400	.229	.1867
.02	2.889	.142	2.009	.293	1.202	.326	.606	.315	.519	.400	.306	.2962
.03	3.250	.146	2.190	.315	1.234	.354	.489	.352	.719	.400	.349	.3753
.04	3.611	.151	2.370	.337	1.265	.383	.372	.388	1.045	.372	.388	.4402
.05	3.972	.156	2.550	.359	1.297	.412	.254	.425	1.673	.254	.425	.5015
.06	4.332	.162	2.730	.381	1.328	.441	.137	.462	3.378	.137	.462	.5050
.07	4.693	.167	2.910	.403	1.359	.470	.019	.499	25.782	.019	.499	.5050
.08	5.054	.174	3.091	.425	1.391	.499	-.098	.536	-5.461	-.098	.536	.5336
C4-60.60 γ = 0.60												
.01	2.528	.149	2.064	.254	1.381	.318	.914	.315	.345	.400	.262	.1907
.02	2.889	.158	2.281	.288	1.470	.365	.868	.375	.432	.400	.344	.3284
.03	3.250	.168	2.499	.323	1.558	.412	.822	.434	.529	.400	.411	.4319
.04	3.611	.179	2.716	.357	1.646	.459	.776	.494	.637	.400	.485	.5289
.05	3.972	.190	2.934	.391	1.734	.506	.730	.554	.759	.400	.552	.6230
.06	4.332	.201	3.151	.425	1.822	.553	.684	.614	.898	.400	.613	.7152
.07	4.693	.212	3.369	.460	1.910	.600	.637	.673	1.056	.400	.674	.7499
.08	5.054	.224	3.586	.494	1.998	.647	.591	.733	1.240	.400	.733	.8802
C4-60.75 γ = 0.75												
.01	2.528	.156	2.271	.224	1.572	.330	1.082	.348	.322	.400	.277	.2001
.02	2.889	.170	2.507	.269	1.698	.394	1.083	.432	.399	.400	.388	.3645
.03	3.250	.186	2.744	.313	1.823	.458	1.084	.516	.476	.400	.484	.5047
.04	3.611	.201	2.981	.357	1.949	.522	1.085	.599	.552	.400	.586	.6298
.05	3.972	.217	3.218	.402	2.074	.586	1.086	.683	.629	.400	.690	.7521
.06	4.332	.233	3.455	.446	2.200	.650	1.087	.766	.705	.400	.771	.8726
.07	4.693	.249	3.692	.490	2.325	.714	1.088	.850	.781	.400	.876	.9920
.08	5.054	.265	3.928	.535	2.451	.778	1.089	.934	.858	.400	.962	1.1105
C4-60.90 γ = 0.90												
.01	2.528	.162	2.453	.185	1.745	.333	1.242	.376	.303	.400	.289	.2157
.02	2.889	.181	2.699	.238	1.892	.411	1.280	.482	.376	.400	.427	.4052
.03	3.250	.200	2.945	.290	2.039	.490	1.318	.588	.446	.400	.547	.5829
.04	3.611	.220	3.192	.343	2.185	.568	1.356	.693	.511	.400	.673	.7367
.05	3.972	.239	3.438	.395	2.332	.647	1.394	.799	.573	.400	.774	.8652
.06	4.332	.259	3.684	.448	2.478	.725	1.433	.905	.632	.400	.907	1.0317
.07	4.693	.278	3.931	.500	2.625	.804	1.471	1.011	.687	.400	1.012	1.1770
.08	5.054	.298	4.177	.552	2.771	.883	1.509	1.117	.740	.400	1.149	1.3213

6-conti.) Design Drilled Shaft for Bending capacity & size reinforcing steel:

Interpolate values in the table at γ=0.60 and 0.75 to get values at γ=0.677

γ = 0.60 Case $\qquad$ γ = 0.75 Case

$$\rho_g := \begin{pmatrix} 0.01 \\ 0.02 \\ 0.03 \\ 0.04 \\ 0.05 \\ 0.06 \\ 0.07 \\ 0.08 \end{pmatrix} \qquad \phi M_{n\_\gamma 0.60} := \begin{pmatrix} 0.1907 \\ 0.3284 \\ 0.4319 \\ 0.5289 \\ 0.6230 \\ 0.7152 \\ 0.7499 \\ 0.8802 \end{pmatrix} \qquad \phi M_{n\_\gamma 0.75} := \begin{pmatrix} 0.2001 \\ 0.3645 \\ 0.5047 \\ 0.6298 \\ 0.7521 \\ 0.8726 \\ 0.9920 \\ 1.1105 \end{pmatrix}$$

(Data from Table C4-60 page 152 Case 7: Moment only and no Axial Loading)

$$Increase\_rate := \frac{\left(\phi M_{n\_\gamma 0.75} - \phi M_{n\_\gamma 0.60}\right)}{0.15} = \begin{pmatrix} 0.063 \\ 0.241 \\ 0.485 \\ 0.673 \\ 0.861 \\ 1.049 \\ 1.614 \\ 1.535 \end{pmatrix}$$

Calculate increasing rate by 1.0

$$\phi M_{n\_\gamma 0.677} := \phi M_{n\_\gamma 0.60} + (0.677 - 0.6) \cdot Increase\_rate = \begin{pmatrix} 0.196 \\ 0.347 \\ 0.469 \\ 0.581 \\ 0.689 \\ 0.796 \\ 0.874 \\ 0.998 \end{pmatrix}$$

Interpolate $\phi M_n/A_g h$ at γ = 0.677

6-conti.) Design Drilled Shaft for Bending capacity & size reinforcing steel:

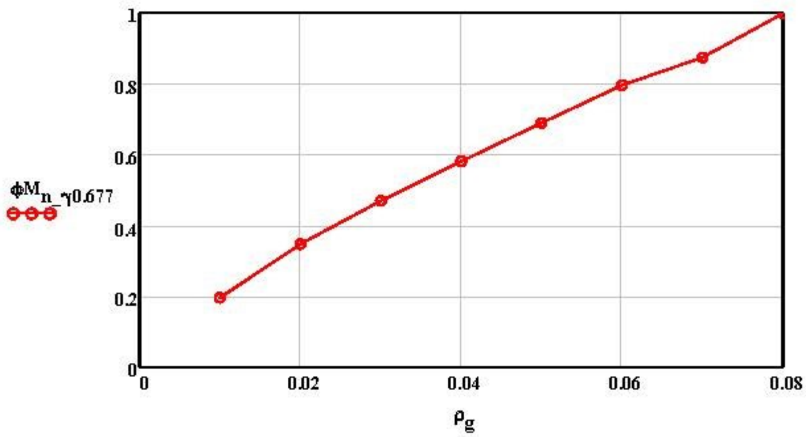


Figure 5. ρ_g vs $\phi M_{n\_\gamma 0.677}$ for interpolation

$\phi M_{nact\_hAg} := \text{linterp}(\rho_g, \phi M_{n\_\gamma 0.677}, \rho_{act}) = 0.222$

Interpolate to get $\phi M_n/A_g h$ at the steel ratio, ρ_{act}

$\phi M_{nPier} := \phi M_{nact\_hAg} \cdot A_g \cdot h \cdot \frac{kip}{in^2} = 200.452 \cdot kip \cdot ft$

Calculate factored nominal capacity of pier, ϕM_{nPier}

$V_F := F_t = 71 \cdot kip$

Transverse impact force causing moment

$H_e = 19 \cdot in$

Height of the transverse impact force

$M_{applied} := V_F \cdot (H_e + 12in) = 183.417 \cdot kip \cdot ft$

Moment applied. Note: An additional 12 inches of eccentricity length was added to account for moment in the shaft below grade.

Drilled_Shaft_Flexure_Check := | "Okay" if $\phi M_{nPier} > M_{applied}$
| "Not Okay" otherwise

Drilled_Shaft_Flexure_Check = "Okay"

7.) Determine/Design Pier Depth Using Brom's Method:

"The Broms method is a simplified limit equilibrium solution that is suitable for simple analysis of relatively short, stiff drilled shafts subjected to lateral shear and overturning moments. The method is most suited to analysis of strength limit states"; Brown, D.A., J.P. Turner, and R.J.Castelli (2010). "Drilled Shafts: Construction procedures and LRFD Design Methods," (NHI Course No. 132014: Geotechnical Engineering Circular No.10, Federal Highway Administration Report No. FHWA NHI-10-016). Washington, DC

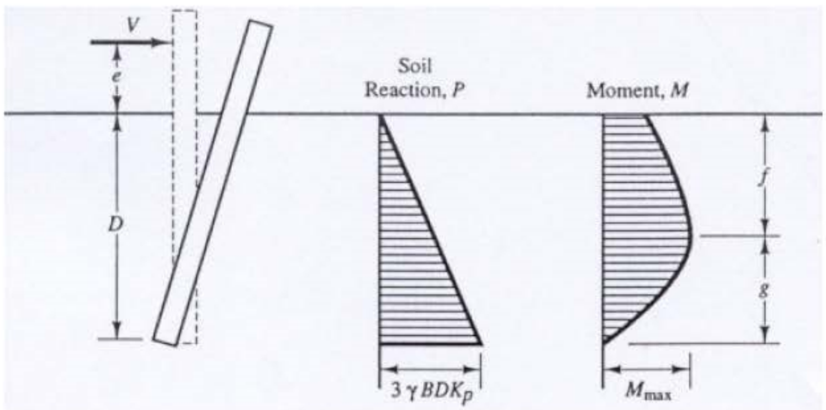


Figure 5. Soil reactions and foundation moments for free-head foundations in cohesionless soil

Requirements of overall moment equilibrium are applied in order to determine the minimum length of the shaft, L_{min}, to satisfy geotechnical strength requirements. At the base of the shaft:

$$\Sigma M_b = 0 = M_t + P_t L_{min} - 3B_b\, \gamma\, L_{min}\, K_P\, (L_{min} / 2)\, (L_{min} / 3)$$

$$= M_t + P_t L_{min} - 1/2 B_b\, \gamma\, (L_{min})^3\, K_P$$

where ΣM_b = sum of the moment at the shaft; P_t = applied shear force at the top; M_t = induced moment at the top; L_{min} = minimum length of the shaft satisfying the moment equilibrium; K_p = Rankine coefficient of passive earth pressure; B_b = width of the shaft; and γ = unit weight of soil

By solving this equation, the minimum required length, L_{min}, of the shaft can be obtained

7. cont.) Determine/Design Pier Depth Using Brom's Method:

$i := 1..7$ $\quad$ $ORIGIN := 1$ — Range variable for depth matrix

$kips \equiv 1000lbf$

$$L_{pier} := \begin{pmatrix} 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 11 \\ 12 \end{pmatrix} ft$$

Vector for pile length underneath the ground

$$Impact_{limit} := \begin{pmatrix} 71 \\ 71 \\ 71 \\ 71 \\ 71 \\ 71 \\ 71 \end{pmatrix} kip$$

Vector for impact load

$\phi_1 := 25deg$ — Effective friction angle of the cohesionless soil for loose Sand. See Joseph Bowles, FOundation Analysis & Design, page 100, Table 3-2 Empirical Values for phi and Unit Weight for Granular Soils

$K_p := \tan\left(45 + \dfrac{\phi_1}{2}\right)^2$ — Rankine passive earth pressure coefficient for Brom's Method

$\gamma_1 := 110pcf$ — Dry unit weight of cohesionless material (pcf)

$B_b := 24in$ — Width of the pier

$H_e = 19 \cdot in$ — Height of Impact Load for MASH TL-3

$F_t = 71 \cdot kip$ — MASH TL-3 Impact Load on End Post

$M_t := F_t \cdot H_e = 112.417 \cdot kip \cdot ft$ — Design Moment used in the Analyses (kip-ft.). M_t is conservative design loading considering impact conditions over short time duration (0.20 seconds) and the disspation of impact loading over a larger area due to the addition of the grade beam in the upper portion of the shaft.

$$P_{t_i} := \frac{\gamma_1 \cdot B_b \cdot \left(L_{pier_i}\right)^2 \cdot K_p}{2} + \frac{M_t}{L_{pier_i}}$$

Corresponding force for given lengths (kip.)

7-cont.) Determine/Design Pier Depth Using Brom's Method:

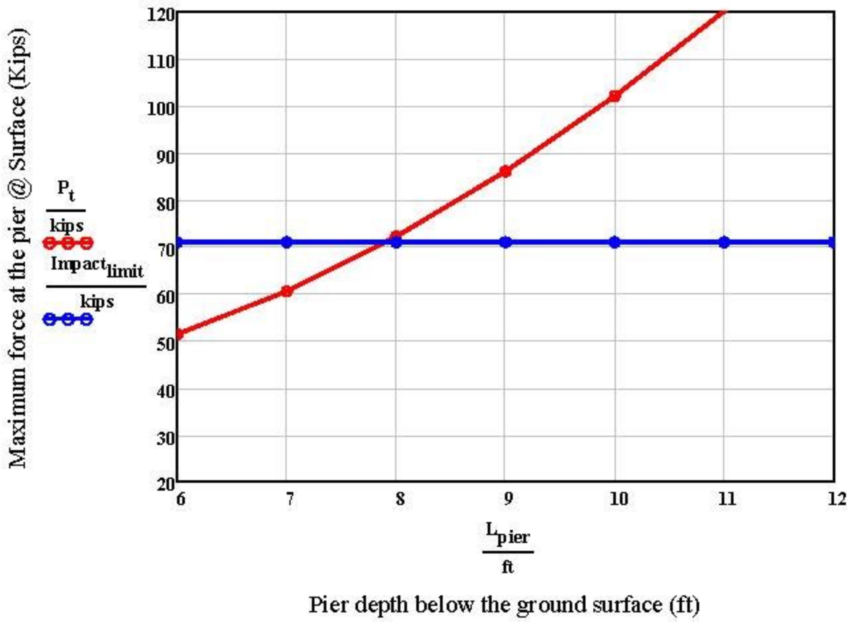


Figure 6. Pier depth vs. Maximum force at pier

<u>Therefore, use 8 ft. pier beneath grade beam</u>

<u>Use 2'-0" Dia. x 8'-0" Deep Drilled Shaft</u>
<u>with #4 Circular stirrups @ 6" O.C. upper 24 inches</u>
<u>#4 Circular stirrups @ 12" O.C. from 2'-0" depth</u>
<u>with 12 ~ #6 Vertical bars evenly spaced for Pier Foundation.</u>

8.) Check a development length of rebar in the drilled shaft

8a.) Determine Basis Development length factors and calculate development length (l_d):

$f'_c = 4 \cdot ksi$ $f_y = 60 \cdot ksi$ for # 6 reinforcing steel $d_b := \frac{6}{8} \cdot in$ $A_b := 0.44 in^2$

Development length is in accordance with AASHTO LRFD 8th provision (2017)

For a conservative design, tension development length (AASHTO 5.10.8.2.1) is considered.

The provisions may be used for:

a) No. 11 bars or smaller **Okay**

b) normal weight concrete with a compressive strength of concrete up to 15 ksi. **Okay**

The basic tension development length, l_{db} shall be take as:

$$l_{db} := 2.4 \cdot d_b \cdot \frac{\frac{f_y}{ksi}}{\sqrt{\left(\frac{f'_c}{ksi}\right)}} = 54 \cdot in$$

8b.) Determine Modification Factors:

Increasing Factors (AASHTO 5.10.8.2.1b)

- Reinforcement location factor (λ_{rl})

  For horizontal reinforcement, placed such that more than 12.0 in. of fresh concrete is cast below the reinforcement : $\lambda_{rl} = 1.3$

  For horizontal reinforcement, placed such that no more than 12.0 in. of concrete is cast below the reinforcement and f_c is greater than 10.0 ksi: $\lambda_{rl} = 1.3$

  $\lambda_{rl} := 1.0$ AASHTO C.5.10.8.2.1b)

- Concrete density modification factor (λ)

  For lightweight concrete use λ as specified in Article 5.4.2.8.

  $\lambda_c := 1.0$ For normal weight concrete (AASHTO 5.4.2.8)

8-Cont.) Check a development length of rebar in the drilled shaft

- Coating factor (λ_{cf})

For epoxy-coated bars with cover less than 3db or with clear spacing between bars less than 6db: λ_{cf}= 1.5

For epoxy-coated bars not covered above: λ_{cf}= 1.2

$\lambda_{cf} := 1.2$

The product λ_{rl} x λ_{cf} need not be taken greater than 1.7

$\lambda_{inc} := \lambda_{rl} \cdot \lambda_{cf} = 1.2$

$$\text{Increasing\_Factor\_Check} := \left(\begin{array}{l} \text{"OK"} \quad \text{if } \lambda_{inc} < 1.7 \\ \text{"NOT OKAY"} \quad \text{otherwise} \end{array} \right) = \text{"OK"}$$

Decreasing Factors (AASHTO 5.10.8.2.1c)

- Reinforcement confinement factor (λ_{rc})

$\lambda_{rc} := 0.4$ — For bar sizes of No.8 and smaller (AASHTO C5.10.8.2.1c)

- Excess reinforcement factor (λ_{er})

Where anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural member is in excess of that required by analysis :

$\lambda_{er} = A_s^{required} / A_s^{provided}$ — AASHTO 5.10.8.2.1c-4

$\lambda_{er} := 1.0$ — Use 1.0 for the conservative design

The modified tension development length shall be taken as:

$$l_d := l_{db} \cdot \left(\frac{\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er}}{\lambda_c} \right) = 25.92 \cdot in$$

**Use 24in. development length: This is slightly less than what is required based on AASHTO.
However, this value is acceptable considering the input parameters and impact loading conditions.**

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SUBJECT: MnDOT J-Barrier
Figure 5-397.116
MASH TL-3 Compliance Assessment

9.) Design Grade Beam for Impact Force (Shear Loading)

9a.) Longitudinal reinforcement

Assume impact force @ very corner of barrier

$F_{beam1} := F_t = 71 \cdot kip$

Eccentricity of 12 inches used ((4 feet grade beam width - 2 feet shaft diameter)/ 2 sides)

$M_{u\_beam} := F_{beam1} \cdot 12in = 71 \cdot kip \cdot ft$

Corresponding moment @ very corner of barrier

$A_{beam} := 5 \cdot 0.2in^2$

5 ~ #4 Horizontal Bars each side of grade beam for bending about the vertical axis due to the impact load applied at the edge of the barrier/grade beam (transverse loading = 71 kips @ 12 inches)

$b_{beam} := 30in$ Width of the grade beam

$\phi_f := 0.9$

Reduction factor (AASHTO 5.5.4.2)

$d_{beam} := 30in - 3in - 0.5in - .25in = 26.25 \cdot in$

Effective depth of the grade beam

$$a_{beam} := \frac{A_{beam} \cdot f_y}{0.85 \cdot f'_c \cdot b_{beam}} = 0.588 \cdot in$$

Depth of the equivalent compression block

$$M_{n\_beam} := A_{beam} \cdot f_y \cdot \left(d_{beam} - \frac{a_{beam}}{2}\right) = 129.779 \cdot kip \cdot ft$$

Nominal flexural resistance

$M_{r\_beam} := \phi_f \cdot M_{n\_beam} = 116.801 \cdot kip \cdot ft$

Factored flexural resistance

$$\text{Beam\_Flexure\_Check} := \begin{cases} \text{"Okay"} & \text{if } M_{r\_beam} \geq M_{u\_beam} \\ \text{"Not Okay"} & \text{otherwise} \end{cases}$$

Beam_Flexure_Check = "Okay"

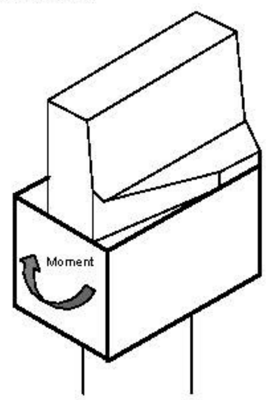


Figure 7. Moment on the grade beam

<u>Therefore, providing 5 - #4 bars on each side of the grade beam is adequate</u>

SUBJECT: **MnDOT J-Barrier**
Figure 5-397.116
MASH TL-3 Compliance Assessment

9.) Design Grade Beam for Impact Force (Shear Loading)

9b.) Transverse reinforcement

Use sectional design method for shear in accordance to AASHTO 5.8.3.4.1 (Simplified Procedure for Nonprestressed Sections)

$\beta_{beam} := 2.0$ — Concrete factor (= 2.0 as per AASHTO 5.8.3.4.1)

$\theta_{beam} := 45deg$ — Angle of inclination (= 45 deg. as per AASHTO 5.8.3.4.1)

$A_{v\_beam} := 2 \cdot 0.2in^2 = 0.4 \cdot in^2$ — Area of transverse reinforcement (2 legs of #4)

$s_{beam} := 6in$ — Spacing of transverse reinforcement

$\phi_v := 0.9$ — Reduction factor (AASHTO 5.5.4.2)

$V_{c\_beam} := 0.0316 \cdot \beta \cdot \sqrt{\left(\frac{f'_c}{ksi}\right)} \cdot ksi \cdot b_{beam} \cdot d_{beam} = 99.54 \cdot kip$ — Nominal shear resistance by concretes

$V_{s\_beam} := \frac{\left(A_{v\_beam} \cdot f_y \cdot d_{beam} \cdot \cot\left(\theta_{beam}\right)\right)}{s_{beam}} = 105 \cdot kip$ — Nominal shear resistance by transverse reinforcement

$V_{n\_beam} := V_{c\_beam} + V_{s\_beam} = 204.54 \cdot kip$ — Nominal shear resistance

$V_r := \phi_v \cdot V_{n\_beam} = 184.086 \cdot kip$ — Factored shear resistance

$V_{u\_beam} := F_t = 71 \cdot kip$ — Shear force applied

$$\text{Beam\_Shear\_Check} := \begin{cases} \text{"Okay"} & \text{if } V_r \geq V_{u\_beam} \\ \text{"Not Okay"} & \text{otherwise} \end{cases} = \text{"Okay"}$$

Providing #4 stirrups at 6 in. spacing for the 30"x30" grade beam is adequate

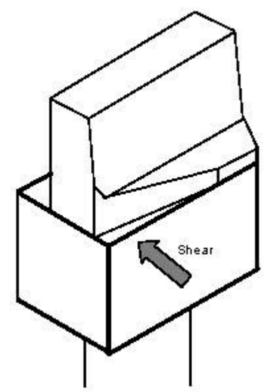


Figure 8. Shear on the grade beam

APPENDIX E2: Analysis Calculations for HSS Tube Section Retrofit

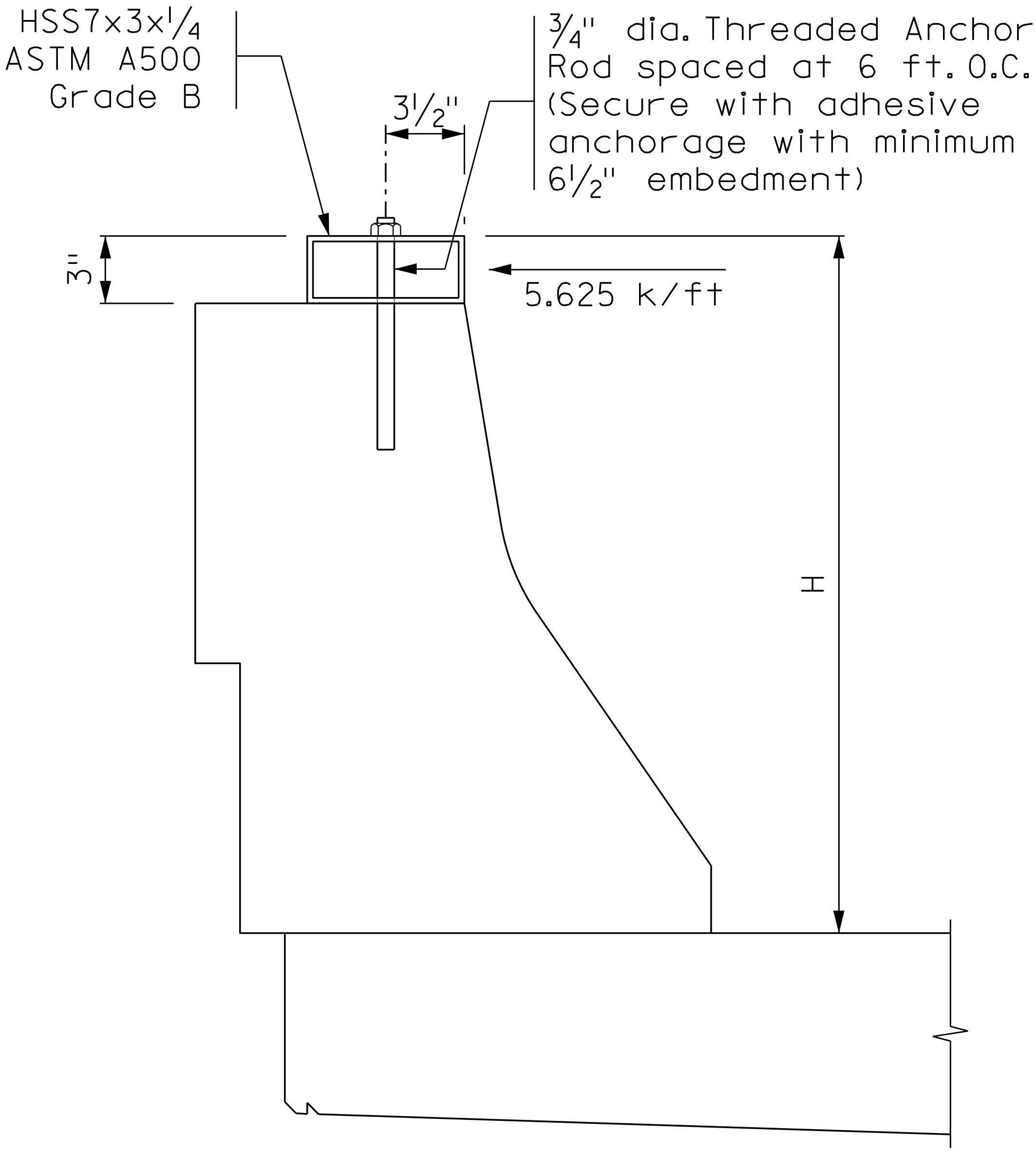


Figure E2-1 HSS Tube Retrofit Details.

Design Loads		
Note: Assuming that the load is distributed at the midspan of a rail section supported by two anchor rods. Assuming the critical condition of a single span, simply supported beam (See Figure B2).		
L_{rod} =	6	Anchor rod spacing (ft)
w_t =	5.625	Distributed load acting at the top of the barrier system (kip/ft)
L_t =	4	Length of the distributed load, w_t (ft)
R_v =	11.25	Shear force acting on each anchor rod (kip)
M_u =	22.5	Maximum moment force acting on the rail (kip-ft)

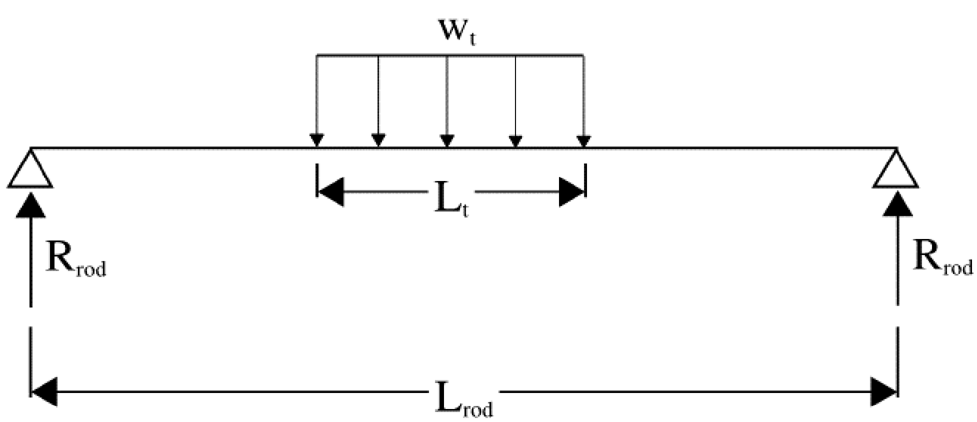


Figure E2-2 Critical Condition Load Application on Steel Rail.

Anchor Rod Capacity		
A_{LPS} =	125	Area of Lateral Punching Shear (LPS) caused by a single anchor rod (in^2) - See Figure B3
f'_c =	4000	Concrete Compressive Srength (psi)
$R_{rod.LPS}$ =	15.81	Lateral Punching Shear (LPS) resistance caused by a single Anchor Rod (kip) = 2 A_{LPS}*SQRT(f'c)
d_{rod} =	0.75	Diameter of Anchor Rods (in.)
$F_{u.rod}$ =	125	Ultimate Strength of Anchor Rods (ksi)
A_{rod} =	0.442	Area of a Single Anchor Rod (in^2)
$R_{rod.v}$ =	24.85	Shear Strength of a single Anchor Rod (kip) = $0.45 A_{rod} F_u$
R_{rod} =	15.81	Limiting ("worst case") capacity of a single Anchor Rod (kip)
R_v =	11.25	Shear force acting on each Anchor Rod (kip)
CHECK	**OK**	**OK if: $R_{rod} \geq R_v$**

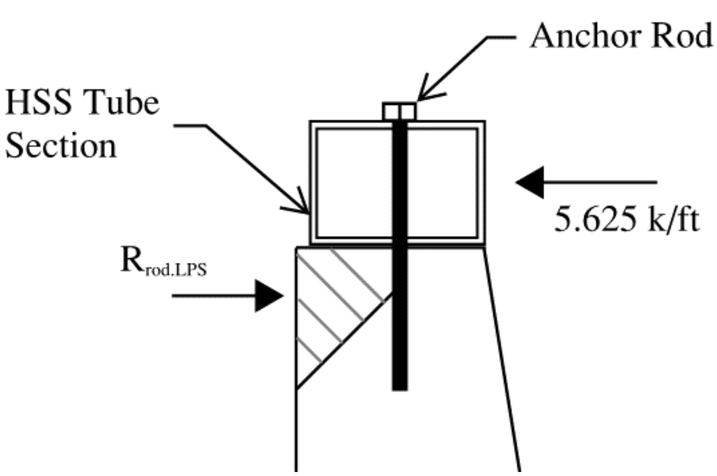
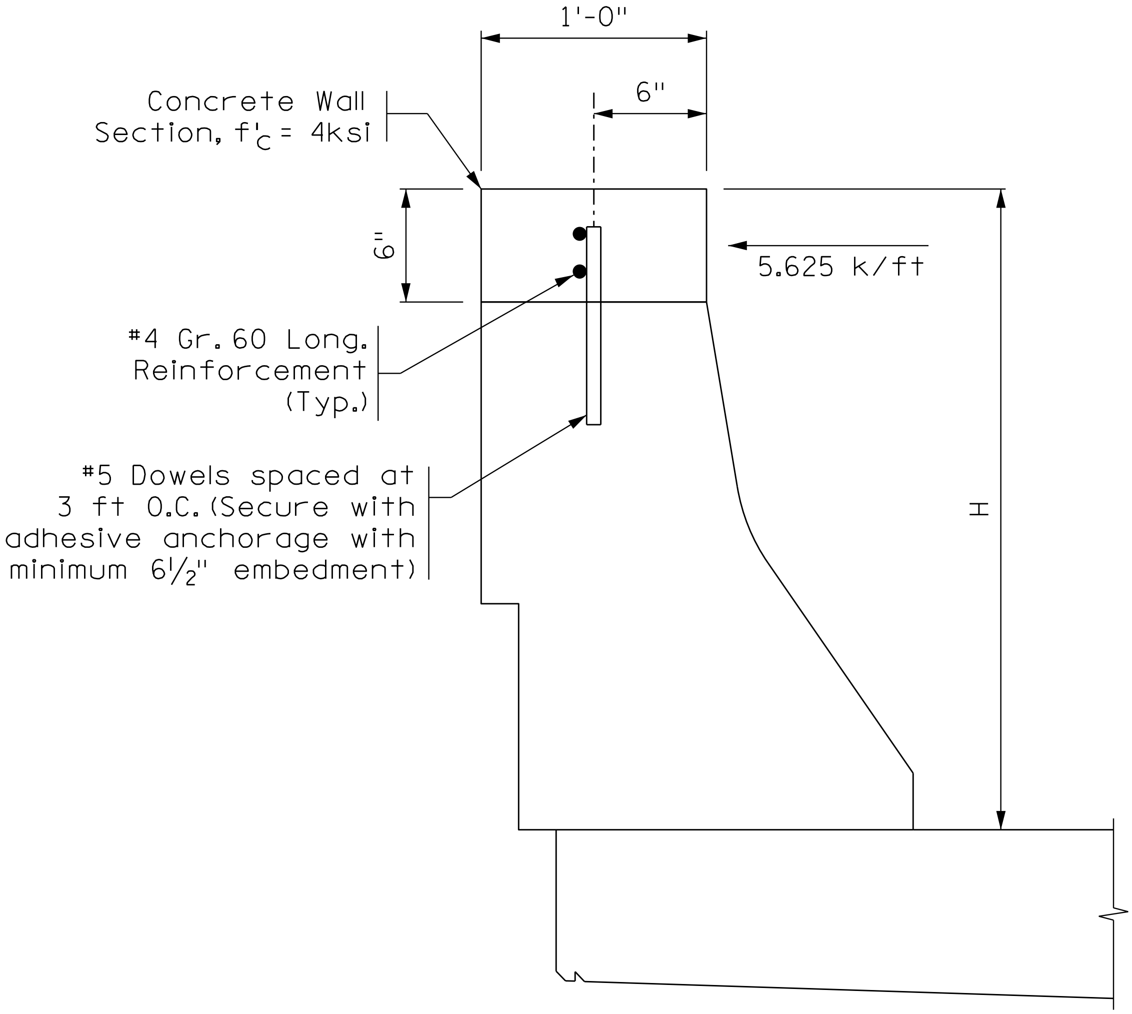


Figure E2-3 Lateral Punching Shear (LPS) Failure Mechanism.

Steel Rail Capacity		
Z_r =	9.22	Plastic Section Modulus of Steel Rail about the bending axis (in^3)
F_{yr} =	46	Yield Strength of Steel Rail (ksi)
M_r =	35.34	Plastic Moment Strength of the Steel Rail (kip-ft) = $Z_r F_{yr}$
M_u =	22.50	Maximum moment force acting on the rail (kip-ft)
CHECK	**OK**	**OK if: $M_r \geq M_u$**

APPENDIX E3: Analysis Calculations for Concrete Wall Section Retrofit



Note: Roughen and clean existing concrete. Apply approved bonding agent such as Shep-Weld or equivalent concrete bonding agent prior to new construction as per manufacturers specifications.

Figure E3-1 Concrete Wall Section Retrofit Details.

Design Loads		
<u>Note</u>: Assuming a one-way shear failure mechanism of the concrete wall retrofit. Considering the critical condition of the load distributed at an end section of the concrete wall retrofit.		
L_{rod} =	3.0	Anchor bar spacing (ft)
d_c =	6.625	Distance from compression face of concrete wall to the tension reinforcement (in.)
w_t =	5.625	Distributed load acting at the top of the barrier system (kip/ft)
L_t =	4	Length of the distributed load, w_t (ft)
L_c =	4.28	Length of the one-way shear zone resulting from the distributed load acting at an end segment (ft)
R_v =	22.5	Shear force acting on the concrete wall retrofit (kip)

Anchorage Capacity		
A_{LPS} =	275	Area of Lateral Punching Shear (LPS) caused by the anchor bars within the applied distributed load (in^2) - See Figure C2
f'_c =	4000	Concrete Compressive Srength (psi)
$R_{bar.LPS}$ =	34.79	Lateral Punching Shear (LPS) resistance of the Anchor Bars in the one-way shear zone (kip) = 2 A_{LPS}*SQRT(f'c)
d_{bar} =	0.625	Diameter of Anchor Bars (in.)
n_{bar} =	2	Number of Anchor Bars in the one-way shear zone
$F_{u.bar}$ =	90	Ultimate Tensile Strength of Anchor Bars (ksi)
A_{bar} =	0.307	Area of a Single Anchor Bar (in^2)
$R_{bar.v}$ =	24.85	Shear Strength of the Anchor Bars in the one-way shear zone (kip) = 0.45 n_{bar} $F_{u.bar}$ A_{bar}
R_{bar} =	24.85	Limiting ("worst case") capacity of the Anchor Bars in the one-way shear zone (kip)
R_v =	22.50	Shear force acting on the concrete wall retrofit (kip)
CHECK	**OK**	**OK if: $R_{bar} \geq R_v$**

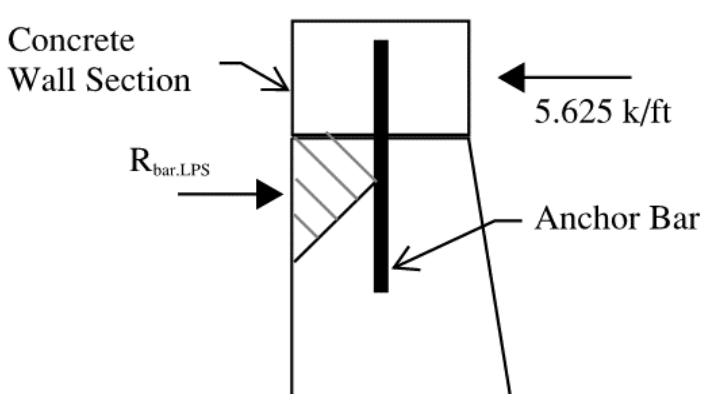


Figure E3-2 Lateral Punching Shear (LPS) Failure Mechanism.

Concrete Wall Capacity		
T_w =	12	Top Width of the Concrete Wall (in)
h_c =	6	Height of the Concrete Wall (in)
$A_{c.v}$ =	687.8	Concrete Wall area resisting one-way shear (in^2) = $L_c T_w + T_w h_c$
$\mathbf{V_c}$ =	86.99	Shear Capicity of the Concrete Wall (kip) = 2 $A_{c.v}$*SQRT(f'c)
$\mathbf{R_v}$ =	22.50	Shear force acting on the concrete wall retrofit (kip)
CHECK	**OK**	**OK if: $V_c \geq R_v$**