Alaska Department of Transportation & Public Facilities

Research & Technology Transfer

# Retrofit Design of the New Manual for Assessing Safety Hardward (MASH) TL4 Alaska 2-Tube Bridge Rail


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13. ABSTRACT (Maximum 200 words)

Many of bridges in the United States are obsolete with respect to the updated AASHTO strength requirements and the safety performance criteria stated in MASH. Many of these bridges utilize a wide concrete curb with a post and bridge rail system. Often, it is not economically feasible to completely replace the obsolete bridge rails with newer MASH compliant designs. State transportation agencies may need to utilize a crashworthy bridge rail retrofit design when an obsolete bridge railing needs to be replaced with a MASH compliant bridge rail system. The project scope for this project included developing details and performing the necessary engineering analyses to anchor the new Alaska 2-Tube Bridge Rail onto three existing North Dakota Department of Transportation bridges with obsolete bridge rails with curbs. One additional design involved anchoring through the curb and deck for an Alaska Department of Transportation Bridge. Details for these 4 select bridges (1 in Alaska and 3 in North Dakota) using the new MASH TL-4 Alaska 2-Tube bridge rail designed and crash-tested at TTI were developed. These new retrofit bridge rails were designed to meet the crash performance requirements of MASH TL-4 or TL-3 if necessary.

14- KEYWORDS : MASH, TL-4, Alaska 2-Tube Bridge Rail, Retrofit Design, Steel Bridge Railing, Barrier Analysis

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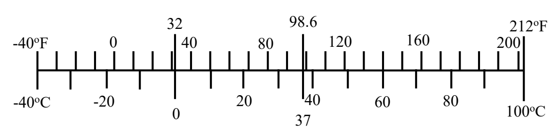
# METRIC (SI*) CONVERSION FACTORS

| APPROXIMATE CONVERSIONS TO SI UNITS | | | | | APPROXIMATE CONVERSIONS FROM SI UNITS | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Symbol | When You Know | Multiply By | To Find | Symbol | Symbol | When You Know | Multiply By | To Find | Symbol |
| | | LENGTH | | | | | LENGTH | | |
| in | inches | 25.4 | | mm | mm | millimeters | 0.039 | inches | in |
| ft | feet | 0.3048 | | m | m | meters | 3.28 | feet | ft |
| yd | yards | 0.914 | | m | m | meters | 1.09 | yards | yd |
| mi | Miles (statute) | 1.61 | | km | km | kilometers | 0.621 | Miles (statute) | mi |
| | | AREA | | | | | AREA | | |
| $in^2$ | square inches | 645.2 | millimeters squared | $cm^2$ | $mm^2$ | millimeters squared | 0.0016 | square inches | $in^2$ |
| $ft^2$ | square feet | 0.0929 | meters squared | $m^2$ | $m^2$ | meters squared | 10.764 | square feet | $ft^2$ |
| $yd^2$ | square yards | 0.836 | meters squared | $m^2$ | $km^2$ | kilometers squared | 0.39 | square miles | $mi^2$ |
| $mi^2$ | square miles | 2.59 | kilometers squared | $km^2$ | ha | hectares (10,000 $m^2$) | 2.471 | acres | ac |
| ac | acres | 0.4046 | hectares | ha | | | | | |
| | | MASS (weight) | | | | | MASS (weight) | | |
| oz | Ounces (avdp) | 28.35 | grams | g | g | grams | 0.0353 | Ounces (avdp) | oz |
| lb | Pounds (avdp) | 0.454 | kilograms | kg | kg | kilograms | 2.205 | Pounds (avdp) | lb |
| T | Short tons (2000 lb) | 0.907 | megagrams | mg | mg | megagrams (1000 kg) | 1.103 | short tons | T |
| | | VOLUME | | | | | VOLUME | | |
| fl oz | fluid ounces (US) | 29.57 | milliliters | mL | mL | milliliters | 0.034 | fluid ounces (US) | fl oz |
| gal | Gallons (liq) | 3.785 | liters | liters | liters | liters | 0.264 | Gallons (liq) | gal |
| $ft^3$ | cubic feet | 0.0283 | meters cubed | $m^3$ | $m^3$ | meters cubed | 35.315 | cubic feet | $ft^3$ |
| $yd^3$ | cubic yards | 0.765 | meters cubed | $m^3$ | $m^3$ | meters cubed | 1.308 | cubic yards | $yd^3$ |
| Note: Volumes greater than 1000 L shall be shown in $m^3$ | | | | | | | | | |
| | | TEMPERATURE (exact) | | | | | TEMPERATURE (exact) | | |
| °F | Fahrenheit temperature | 5/9 (°F-32) | Celsius temperature | °C | °C | Celsius temperature | 9/5 °C+32 | Fahrenheit temperature | °F |
| | | ILLUMINATION | | | | | ILLUMINATION | | |
| fc | Foot-candles | 10.76 | lux | lx | lx | lux | 0.0929 | foot-candles | fc |
| fl | foot-lamberts | 3.426 | candela/$m^2$ | cd/$cm^2$ | cd/$cm^2$ | candela/$m^2$ | 0.2919 | foot-lamberts | fl |
| | | FORCE and PRESSURE or STRESS | | | | | FORCE and PRESSURE or STRESS | | |
| lbf | pound-force | 4.45 | newtons | N | N | newtons | 0.225 | pound-force | lbf |
| psi | pound-force per square inch | 6.89 | kilopascals | kPa | kPa | kilopascals | 0.145 | pound-force per square inch | psi |

These factors conform to the requirement of FHWA Order 5190.1A *SI is the symbol for the International System of Measurements



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# 1. INTRODUCTION

## 1.1 Problem Statement

The American Association of State Highway and Transportation Officials (AASHTO) published an updated 2016 edition of the Manual for Assessing Safety Hardware (MASH) document [1]. Along with this new edition, the Federal Highway Administration (FHWA) and AASHTO developed a revised joint implementation agreement which establishes dates for discontinuing the use of safety hardware that has met earlier crash testing criteria for new installations and full replacements on the National Highway System (NHS).

Many of bridges in the United States are obsolete with respect to the updated AASHTO strength requirements and the safety performance criteria stated in MASH. Many of these bridges utilize a wide concrete curb with a post and bridge rail system. Often, it is not economically feasible to completely replace the obsolete bridge rails with newer MASH compliant designs. Oftentimes, state transportation agencies may need to utilize a crashworthy bridge rail retrofit design when an obsolete bridge railing needs to be replaced for a MASH compliant bridge rail system

For this project, the State of Alaska, Department of Transportation and Public Facilities (Alaska DOT & PF) contracted Texas A&M Transportation Institute (TTI) to develop effective retrofit designs that meet the current MASH's safety criteria for existing Alaska bridge rail systems. As part of this project, the TTI research team developed a retrofit design of the new MASH TL-4 Alaska 2-Tube Bridge Rail to meet the strength requirements of MASH TL-4 on the four (4) curb and deck across sections. This new MASH TL-4 Bridge Rail was designed and crash-tested by TTI in April 2019[2]. The purpose of this project was to develop retrofit designs using the new 2-Tube bridge rail anchored to the top of the existing curbs using either Hilti Adhesive anchors or with post anchor bolts that bolt through the curb and deck. TTI researchers worked closely with Alaska Department of Transportation and North Dakota Department of Transportation to develop retrofit designs for 4 bridge projects. As part of this project, the TTI research team developed details and performed strength analyses for these four designs in accordance with the current AASHTO LRFD, Section 13 Design Specifications [3].

The project scope included performing the necessary engineering analyses to anchor the new 2-Tube Bridge Rail onto the existing curbs in three (3) of the designs and anchoring through the curb and deck in one (1) of the designs. The TTI research team analyzed the designs in accordance with MASH TL-4 impact conditions and developed all the necessary details to retrofit the new MASH TL-4 Alaska 2-Tube Bridge Rail onto the designs described in this report.

## 1.2 Objective

The objective of this project was to develop details for 4 select bridges (1 in Alaska and 3 in North Dakota) using the new MASH TL-4 Alaska 2-Tube bridge rail that was designed and crash tested at TTI in April, 2019. These new retrofit designs were designed to meet the crash performance requirements of MASH TL-4 or TL-3 if necessary.

# 2. DESIGN ARTICLES

The TTI research team has received the following drawings entitled below from the technical contracts Mr. Elmer Marx of Alaska DOT & PF, and Mr. Tim Schwagler of North Dakota DOT. These drawings were used to develop the retrofit details needed for these projects. Strength analyses were performed based on the details developed for these projects.

1.) Curb Detail 94-134.115 (Figure 2-1)
2.) Curb Detail 21-106.109 (Figure 2-2)
3.) Curb Detail 2-149.663 (Figure 2-3)
4.) Alaska Steel Bridge Rail Design (Figure 2-4)

The provided details were used to further develop the retrofit designs for this project. Details of the Alaska and North Dakota bridge rails are presented along with brief descriptions for each in the following subsections.

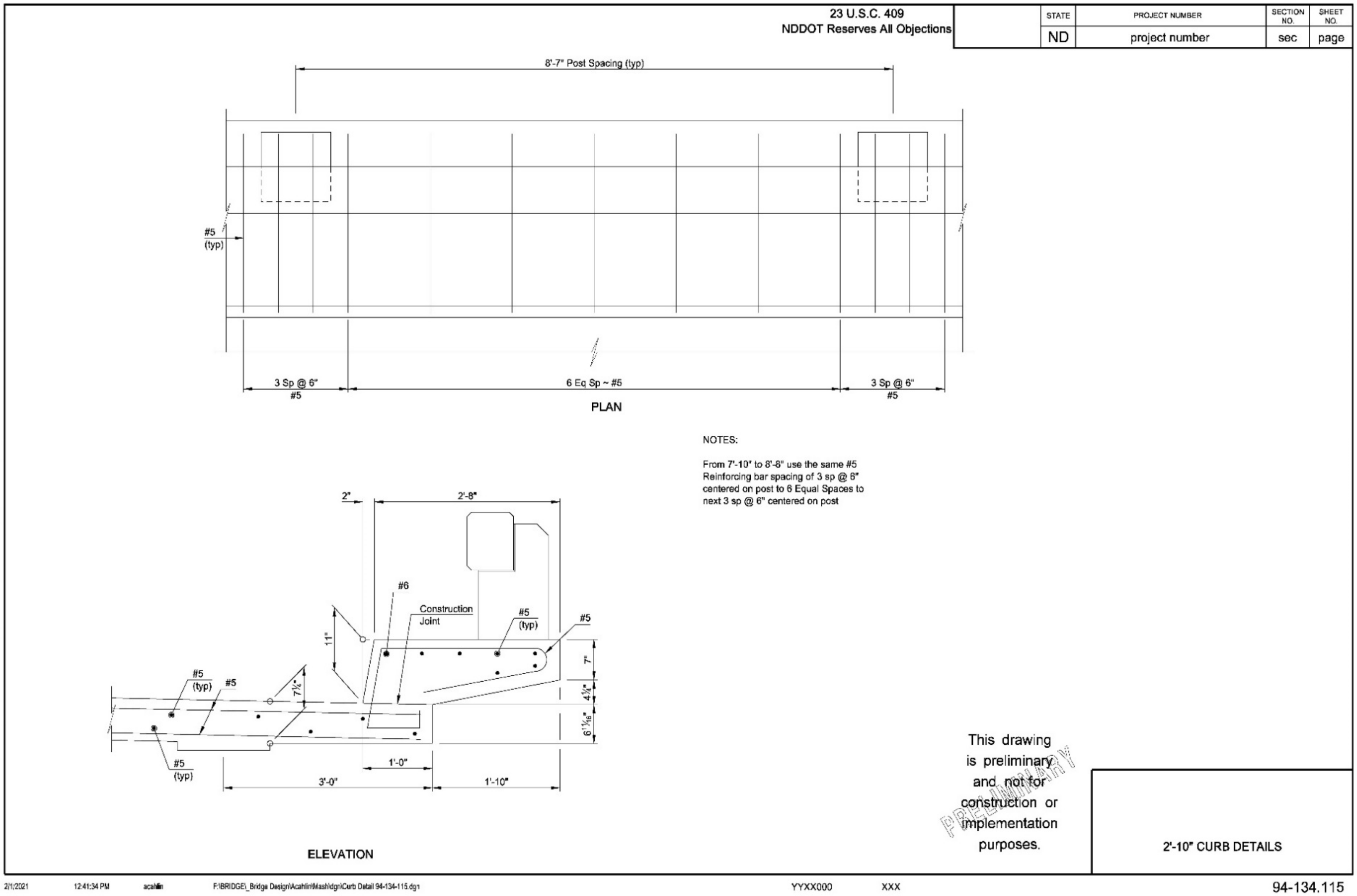



**Figure 2-1. Curb Detail 94-134.115**

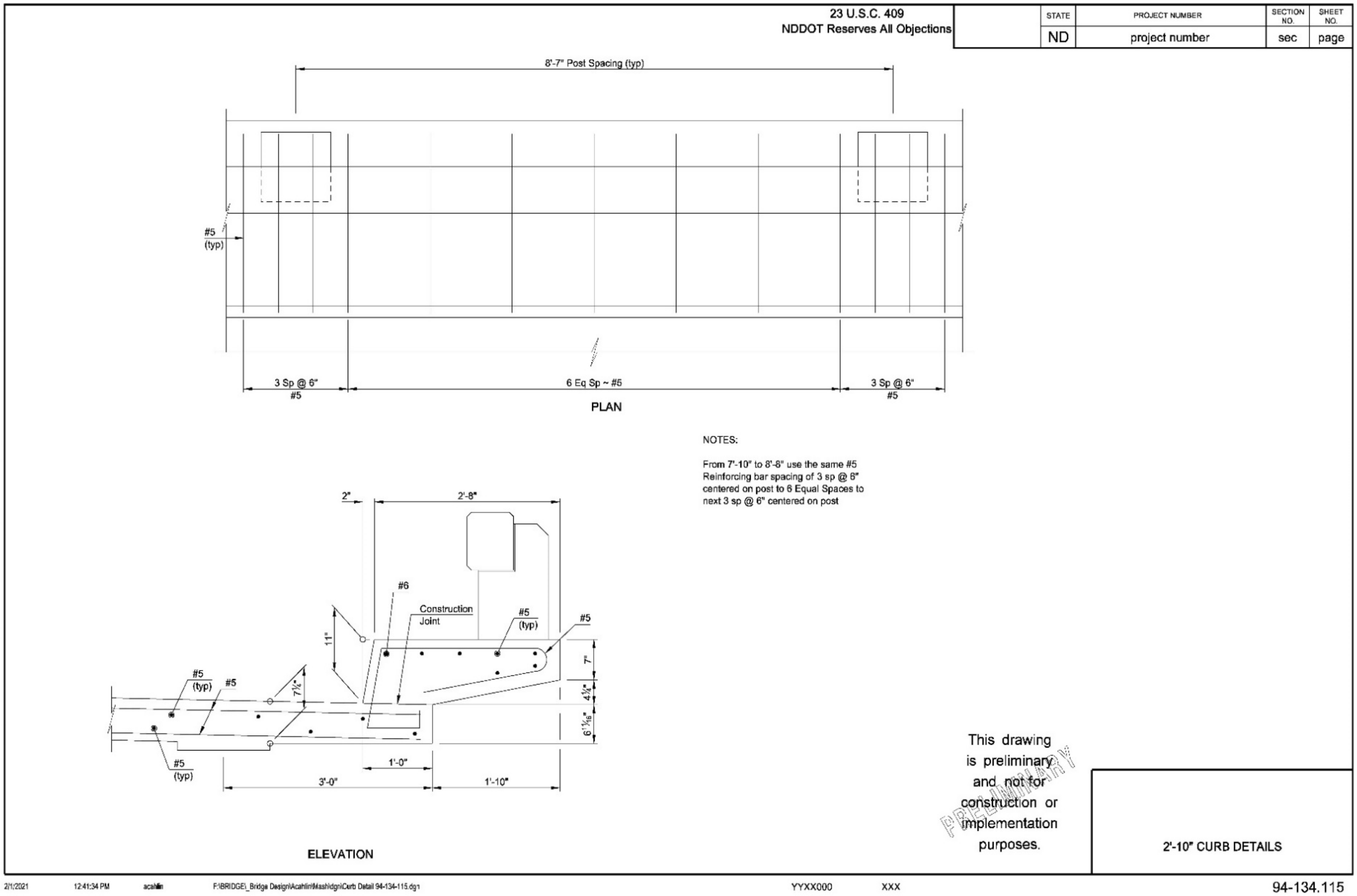



**Figure 2-2. Curb Detail 94-134.115**

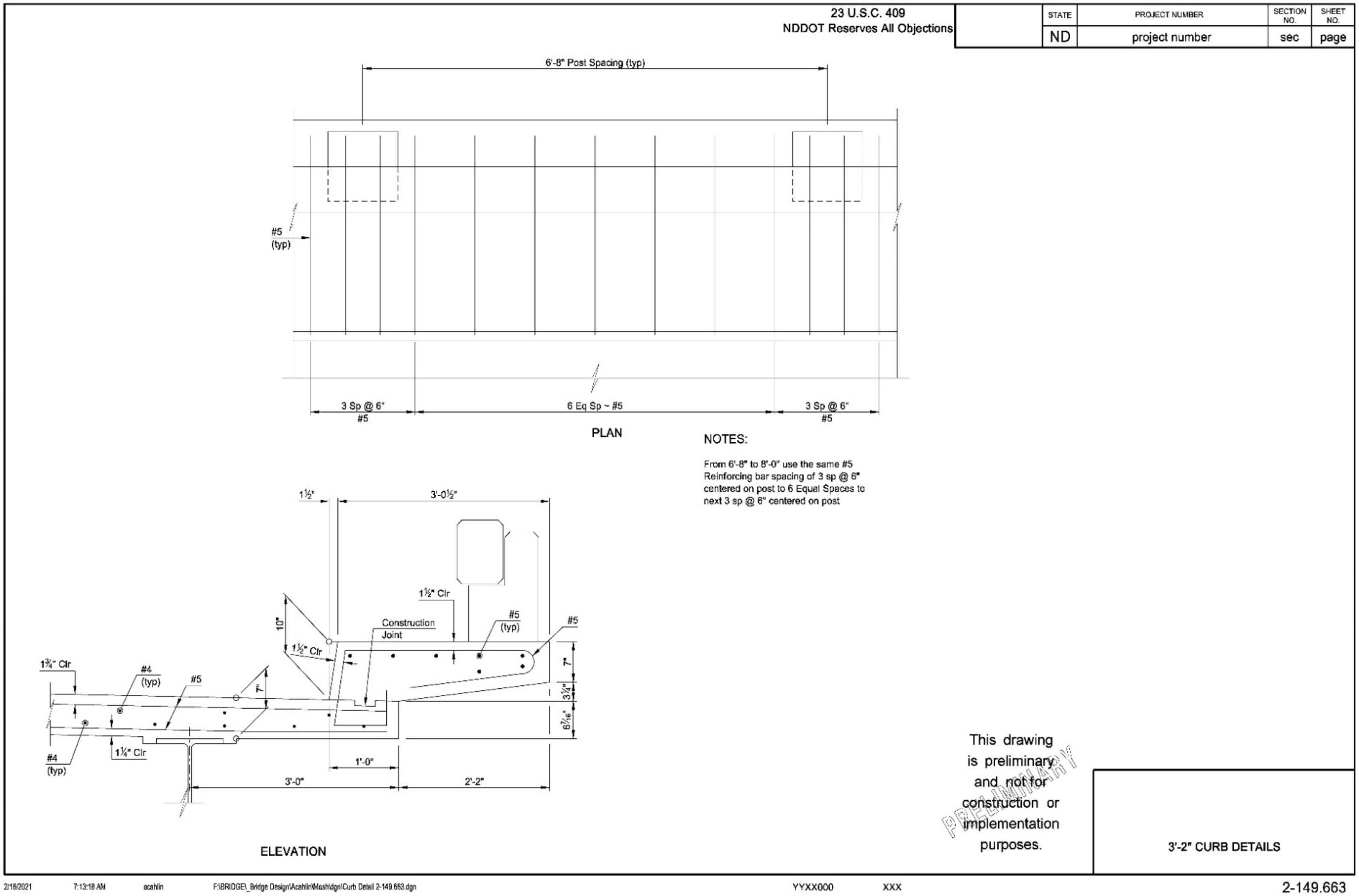



**Figure 2-3. Curb Detail 2-149.663**

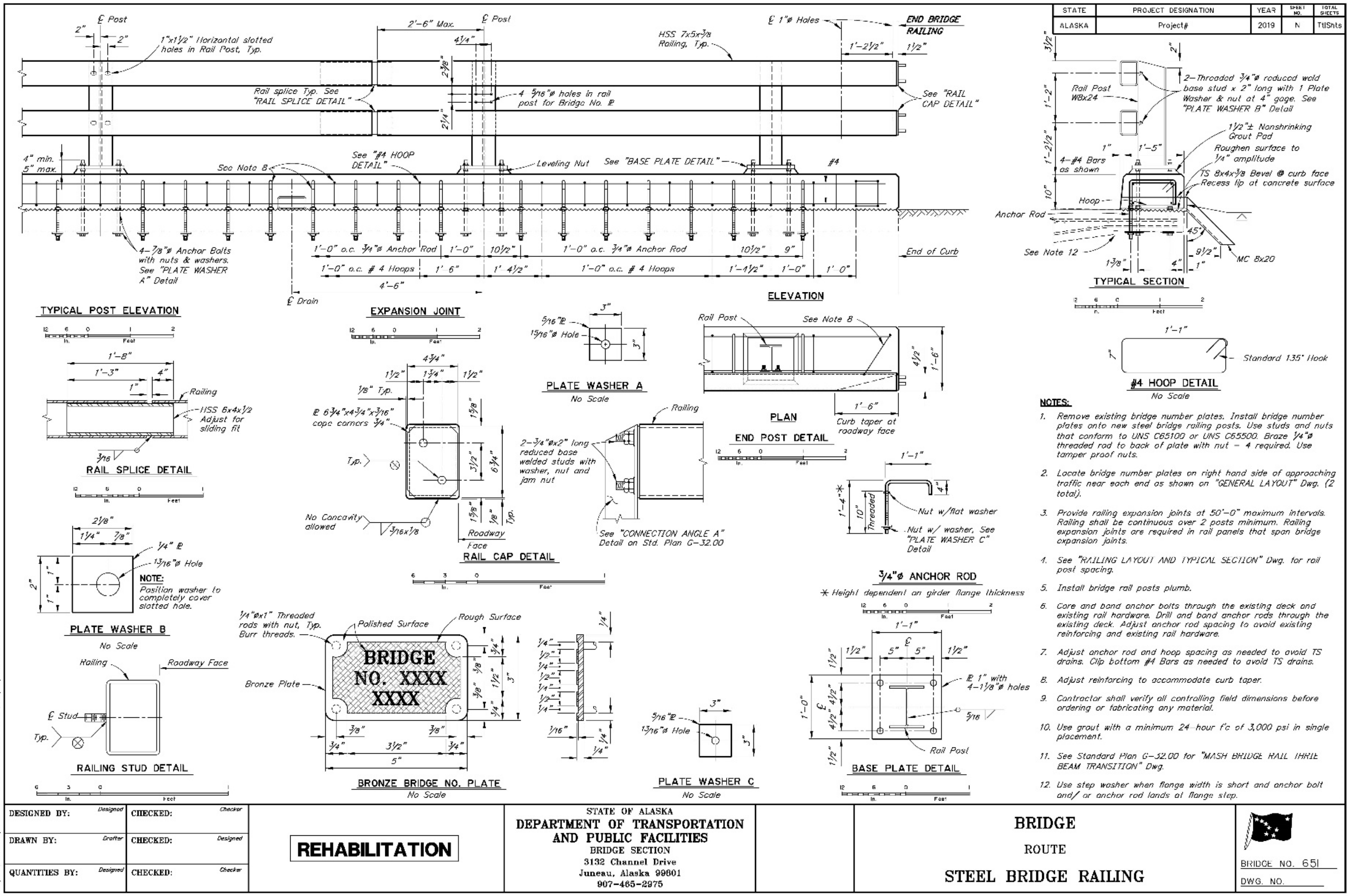



**Figure 2-4. Alaska Steel Bridge Railing Design**

## 2.1 North Dakota Bridge Rail with Curb 94-134.115

The details of deck and curb for drawings 94-134.115 are provided in Figure 2-5. As shown in the figure, the curb detail 94-134.115 has a 10-inch joint width for the curb attachment to the deck edge. The existing post and beam rail shown in these details will be removed to accommodate the new retrofit railing developed for this project.

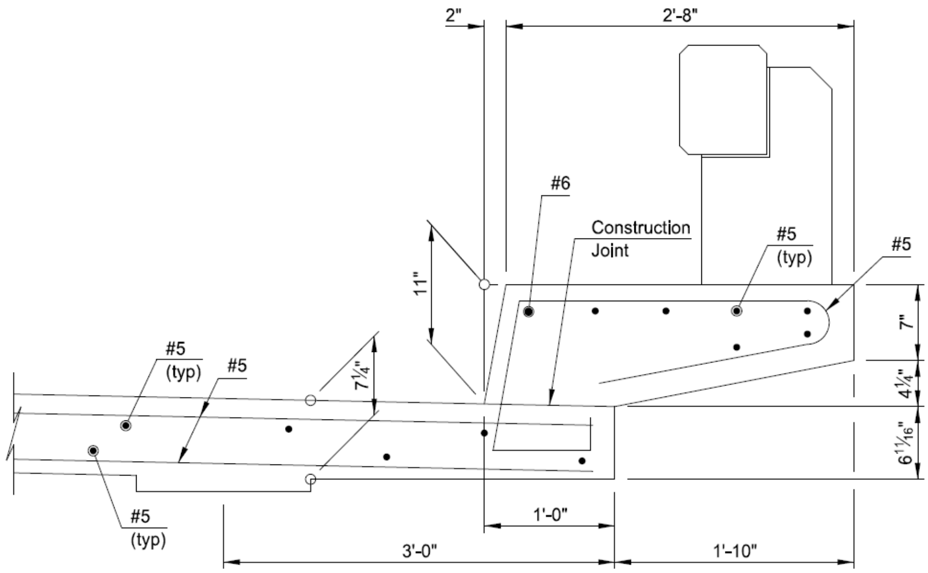



**Figure 2-5. Details of North Dakota Curb 94-134.115**

## 2.2 North Dakota Bridge Rail with Curb 21-106.109

The details of deck and curb of North Dakota 21-106.109 are provided in Figure 2-6. As shown in the figure, the curb detail 21-106.109 has an 8-inch joint width for the curb attachment to the deck edge. Of three curb details used by North Dakota DOT, this detail was considered the most critical for strength and performance It was recommended that all existing posts and rails be removed to provide sufficient space for the new retrofit design developed for this project.

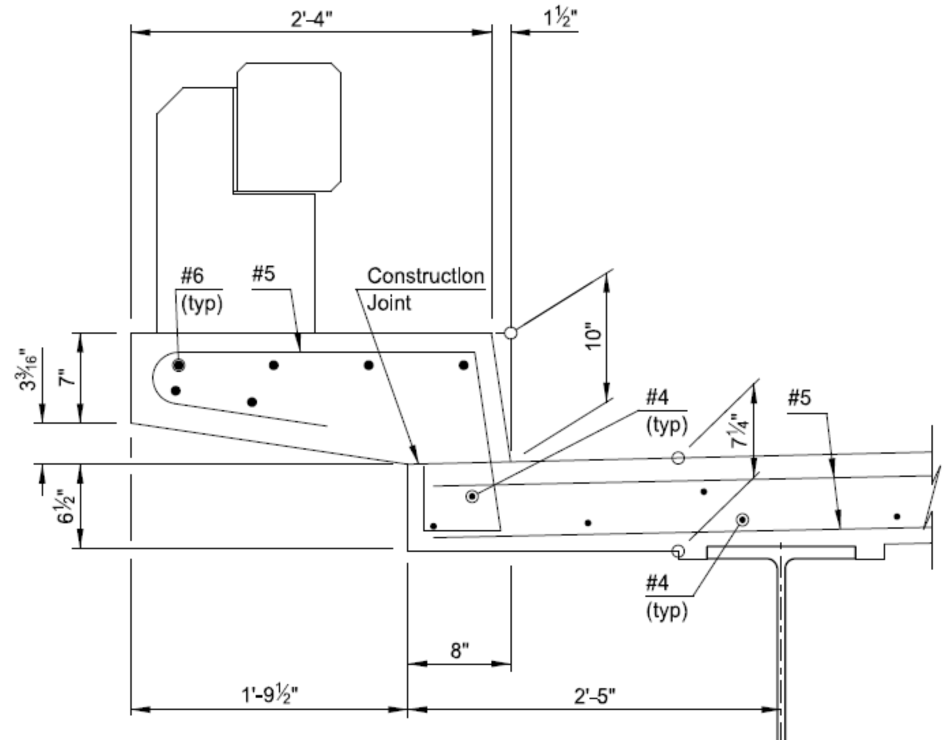



**Figure 2-6. Details of North Dakota Curb 21-106.109**

## 2.3 North Dakota Bridge Rail with Curb 2-149.663

The details of the bridge rail, deck, and curb of North Dakota 2-149.663 are provided in Figure 2-7. As shown in the figure, the curb detail 2-149.663 has a 12-inch joint width for the curb attachment to the deck edge. Of three curb details provided to us from North Dakota, this detail utilized the largest joint width connection to the deck. The details shown in Figure 2-6 were considered more critical and were used in the analyses for this project. TTI researchers recommended that the existing post and rail system be removed to accommodate the new retrofit design for this bridge rail system.

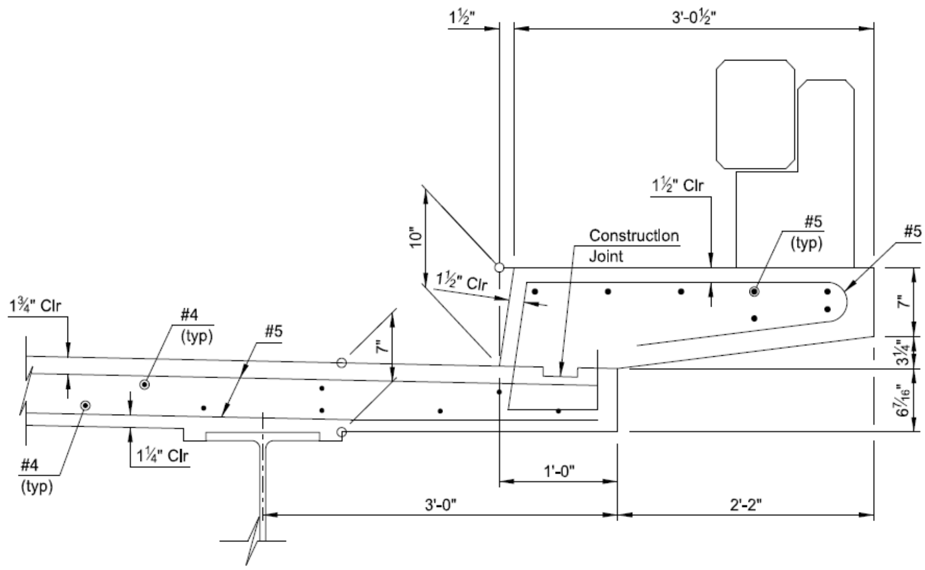



Figure 2-7. Details of North Dakota Curb 2-149.663

## 2.4 Alaska Steel Bridge Railing Design

The details of Alaska steel bridge railing design are provided in Figure 2-8. For this project, a review of the curb anchorage was performed to reduce the amount of anchorage bolting through the deck. Reducing the amount of drill-through anchorage would reduce/eliminate contaminated drilling water from entering Alaska streams and rivers.

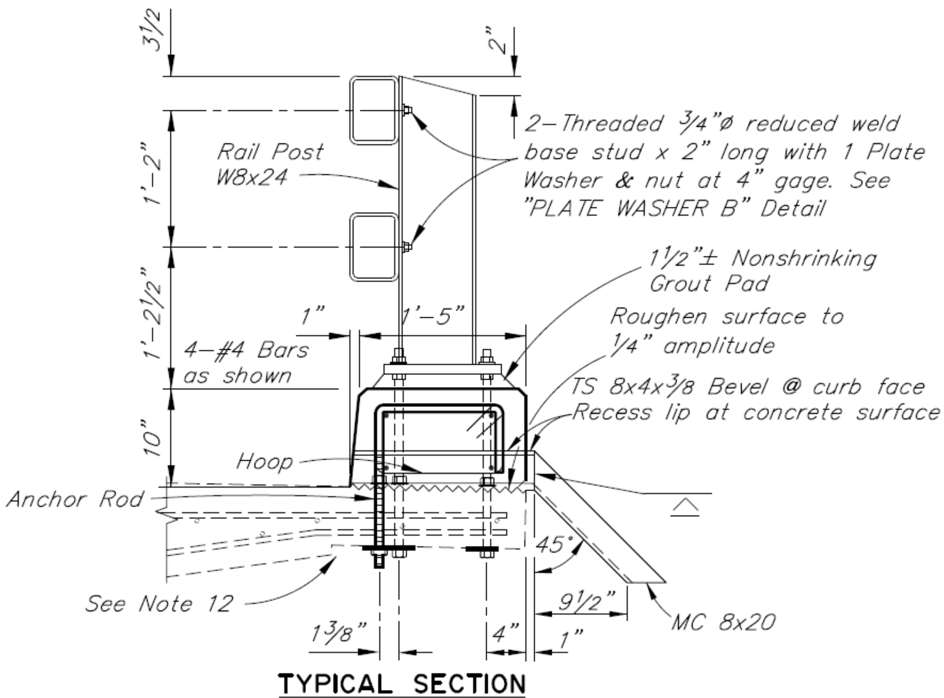



Figure 2-8. Details of Alaska steel bridge railing

# 3. TTI Retrofit Design

## 3.1 Barrier Analysis Procedure

### 3.1.1 Stability Requirements for MASH Bridge Railings

For a bridge railing to be considered a MASH acceptable system, a minimum height must be met to ensure the stability of the vehicle. Table 3-1 shows the minimum height requirements for MASH TL-2, TL-3, and TL-4 bridge traffic railings.

**Table 3-1. Minimum Height Requirements for MASH**

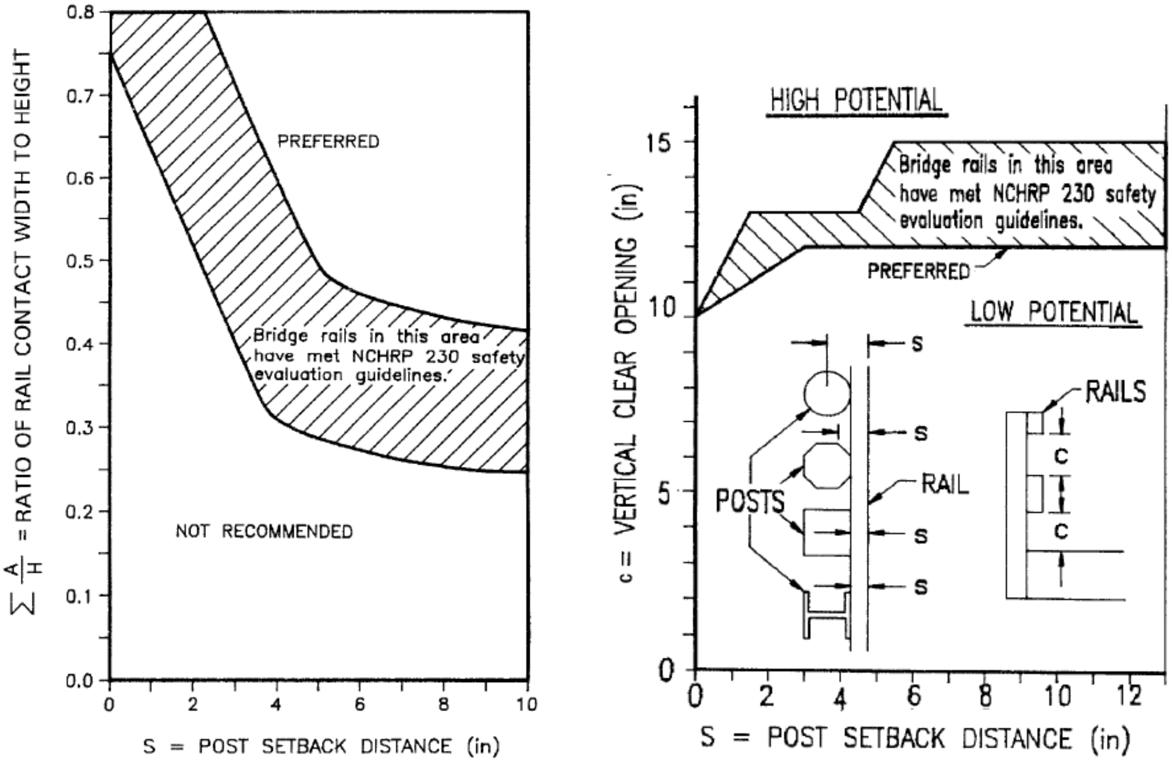
| MASH Test Level | Minimum Height (in.) |
|---|---|
| TL-2 | 18[3] |
| TL-3 | 29[4] |
| TL-4 | 36[5] |

[5] NCHRP Project 22-20(2)

The height of a bridge railing system being analyzed was acquired from the detailed drawings of that specific bridge railing and compared to the minimum height requirement for the specified test level. As specified in AASHTO Section 13 LRFD, bridge railing is measured from the top of the roadway surface or wearing course thickness to the top of the barrier. If the minimum bridge railing height was satisfied, the railing system was considered to satisfactorily meet stability requirements.

### 3.1.2 Geometric Requirements for MASH Bridge Railings

The geometric relationships for bridge railings contained in the current Section 13 AASHTO LRFD Bridge Design Specifications [3] (Figure 3-1) were applied to evaluate railing geometry. These relationships pertain to the potential for wheel, bumper, or hood snagging on elements of the bridge railing. Severe snagging can lead to a number of undesirable consequences including increased occupant compartment deformation, higher accelerations, and occupant risk indices, and vehicle instability. Both AASHTO figures were used to analyze bridge railing geometry for all MASH Test Levels for this project.




(a) Post Setback Criteria (AASHTO Figure A13.1.1-2)

(b) Snag Potential Criteria (AASHTO Figure A13.1.1-3)

**Figure 3-1. Post setback and Snag Potential Criteria per AASHTO Section 13**

For a bridge railing to be given a Satisfactory (S) designation for the geometric evaluation criteria, the bridge railings' geometric data points (i.e., post setback distance, rail contact width to height ratio, and vertical clear opening) must plot in the AASHTO Figure A13.1.1-2 and A13.1.1-3 acceptable regions. The Preferred region in AASHTO Figure A13.1.1-2 and the Low Snag Potential region in AASHTO Figure A13.1.1-3 are considered the acceptable regions. A bridge railing is given a Marginal (M) designation for the geometric evaluation criteria if the barriers' geometric data points are located between the Preferred and Not Recommended regions (Figure A13.1.1-2) or the Low Snag Potential and High Snag Potential regions (Figure A13.1.1-3). A bridge railing is given a Not Satisfactory (NS) designation for the geometric evaluation criteria if the railings' geometric data points plot in the Not Recommended region (Figure A13.1.1-2) or the High Snag Potential region (Figure A13.1.1-3).

### 3.1.3 Strength Requirements

Section 13 of the AASHTO LRFD Bridge Design Specifications contains procedures for analyzing the structural capacity of different types of bridge barriers (e.g., steel, concrete). These procedures were used to evaluate the strength of the selected bridge rail retrofits for this project. Using these procedures, an analysis of the strength of the selected bridge railings was performed using updated loads for MASH Test Level 4 impact conditions. All bridge railings analyzed for this project were evaluated with respect to current recommended MASH TL-4 impact loading conditions [3].

## 3.2 TTI Retrofit Design

The analyses procedures described in the previous section were applied to the bridge railings considered in this project. Note that the TTI researchers developed a retrofit design for each bridge railing as close to the tested design and geometry as possible by the request of Alaska DOT & PF. The analysis results of each railing system are summarized in the following subsections. The loading conditions used in the analyses for this project are provided in Table 3-2 below. This information was used for all bridge barriers analyzed and reported in NCHRP Project 20-07/Task 395[4].

**Table 3-2. Design Forces for Traffic Railings.**

| **Test Level** | **Rail Height** (in.) | $F_t$ (kip) | $F_L$ (kip) | $F_v$ (kip) | $L_t$ and $L_L$ (ft) | $L_v$ (kip) | $H_e$ (in.) | $H_{min}$ (in.) |
|---|---|---|---|---|---|---|---|---|
| TL-1[3] | 18 or above | 13.5 | 4.5 | 4.5 | 4.0 | 18.0 | 18.0 | 18.0 |
| TL-2[3] | 18 or above | 27.0 | 9.0 | 4.5 | 4.0 | 18.0 | 20.0 | 18.0 |
| TL-3[4] | 29 or above | 71.0 | 18.0 | 4.5 | 4.0 | 18.0 | 19.0 | 29.0 |
| TL-4 (a)[5] | 36 | 68.0 | 22.0 | 38.0 | 4.0 | 18.0 | 25.0 | 36.0 |
| TL-4 (b)[5] | greater than 36 | 80.0 | 27.0 | 22.0 | 5.0 | 18.0 | 30.0 | 36.0 |
| TL-5 (a)[5] | 42 | 160.0 | 41.0 | 80.0 | 10.0 | 40.0 | 35.0 | 42.0 |
| TL-5 (b)[5] | greater than 42 | 262.0 | 75.0 | 160.0 | 10.0 | 40.0 | 43.0 | 42.0 |
| TL 6[3] | 90 or above | 175.0 | 58.0 | 80.0 | 8.0 | 40.0 | 56.0 | 90.0 |

Note: [3]AASHTO LRFD Section 13 Table A13.2-1; [4]NCHRP Project 20-07/Task 395; [5]NCHRP Project 22-20(2)

### 3.2.1 North Dakota Bridge Rail with Curb 94-134.115

Figure 3-2 shows TTI's proposed retrofit design for the North Dakota's concrete curb 94-134.115. The total height of the bridge rail system is 38 in. from the roadway surface to maintain the post and rail details as tested from TTI project 608331 in accordance with the request of the client. To secure the 2-Tube Bridge Rail on the North Dakota concrete curb 94-134.115, Hilti RE500 V3 Epoxy with 7/8-in. diameter anchor bolts with 8 in. embedded length for both traffic and field sides, respectively were analyzed for this project.

Appendix A-1 and B-1 contain the suggested details and full analysis for the retrofit design of North Dakota Bridge Rail with Curb 94-134.115, respectively. Below is a summary of the evaluation results and recommendations.

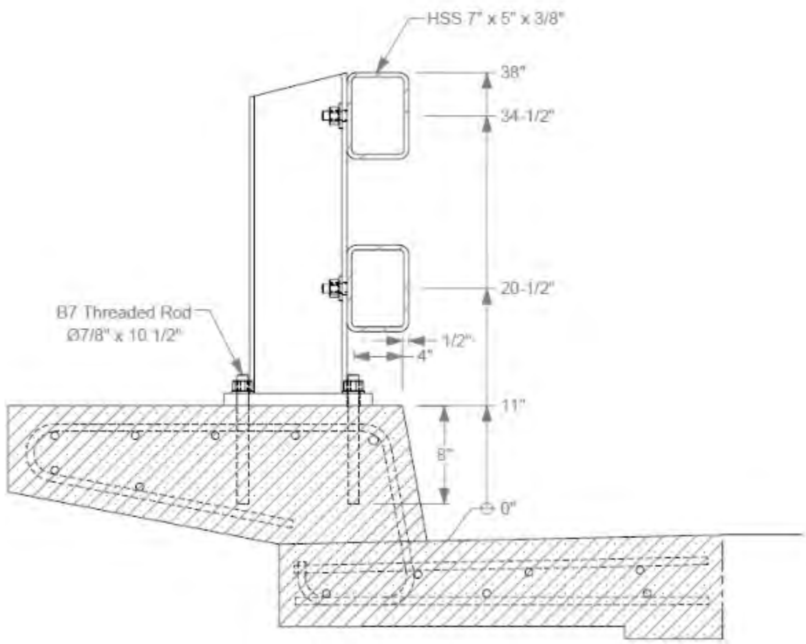



**Figure 3-2. TTI retrofit design for the North Dakota Bridge Rail with Curb 94-134.115**

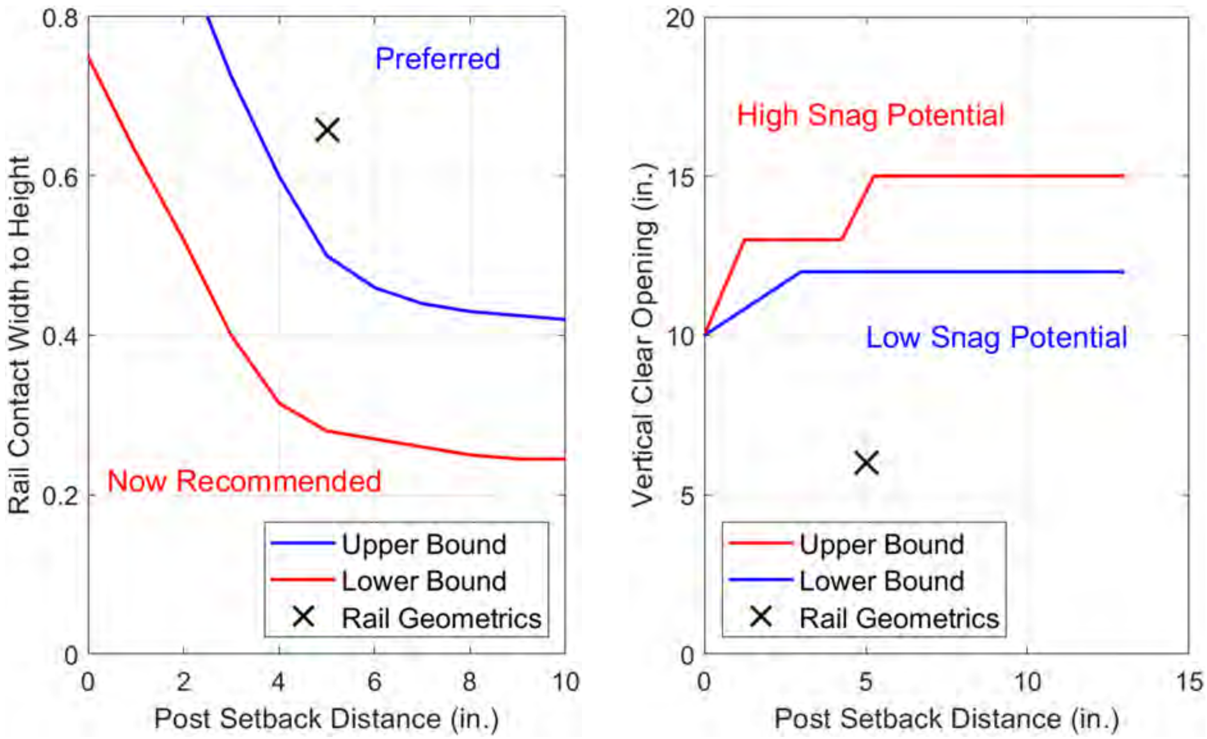
#### 3.2.1.1 Stability Evaluation

The retrofit design for North Dakota Bridge Rail with Curb 94-134.115 has a height of 38-in. The minimum height requirement for a MASH TL-4 bridge barrier is 36-in. Thus, the retrofit design for this retrofit design meets the MASH TL-4 minimum height stability criterion (Satisfactory).

#### 3.2.1.2 Geometric Evaluation

Post setback distance, ratio of contact width to height, and vertical clear opening were determined for the proposed retrofit design for the North Dakota Bridge Rail with Curb 94-134.115. The appropriate data points were plotted against the current AASHTO LRFD Section 13 geometric relationships. As seen in Figure 3-3, the geometric data points for the proposed retrofit design are located in the preferred region for the Post Setback criteria and in the acceptable region for the

Snag Potential criteria. Therefore, the retrofit design for the North Dakota Bridge Rail with Curb 94-134.115 satisfies the geometric evaluation criteria.




(a) Post Setback Criteria (b) Snag Potential Criteria

**Figure 3-3. Geometric criteria assessment of the 94-134.115 curb retrofit design**

### 3.2.1.3 Strength Evaluation

The structural capacity of the North Dakota Bridge Rail with Curb 94-134.115 retrofit design was compared to the current recommended MASH TL-4 design transverse impact load provided in Table 3-2. Since the height of the rail is 38 in. which exceeds 36 in., the desired MASH TL-4 design impact load ($F_t$) is 80 kips located at an effective height ($H_e$) of 30 in. above the roadway surface as per TL-4(b) in Table 3-2.

Table 3-3 presents the post strengths based on 9 ultimate strength load cases from the strength analysis for this retrofit design. As summarized in the table, the limiting post strength was the ultimate strength of the post based on the strength of the adhesive anchor bolts in the curb. The adhesive bond strength of the anchor in the curb governs the ultimate strength of the posts. Based on this strength, the calculated strength of the retrofit design was approximately 63 kips at $H_e$ = 30 in., over a 3-span failure mechanism. Based on this calculated strength, the proposed retrofit design does not meet the target strength of 80 kips required for MASH TL-4. TTI researchers recommend performing static load testing of the posts or full-scale crash testing for MASH TL-4. Table 3-4 summarizes the strength of the retrofit design of the North Dakota Bridge Rail with Curb 94-134.115.

**Table 3-3. Summary of post strengths based on several critical cases for 94-134.115 curb**

| | |
|---|---|
| 1) Post strength based on plastic strength of the post | 74.5 kips |
| 2) Post strength based on post fillet welds | 74.3 kips |
| 3) Post strength based on anchor bolt shear strength | 130.0 kips |
| 4) Post strength based on ultimate tensile strength of anchor bolts | 49.2 kips |
| 5) Post strength based on the curb punching shear strength | 82.8 kips |
| 6) Post strength based on lateral punching shear on the traffic side bolts | 66.1 kips |
| 7) Post strength based on flexural resistance (FS1) | 53.4 kips |
| 8) Post strength based on flexural resistance (FS2) | 38.4 kips |
| 9) Post strength based on ultimate strength of adhesive dowels in curb | **21.8 kips** |

**Table 3-4. Summary of MASH TL-3 and 4 Strength Analysis for 94-134.115 curb**

| Criteria | Required | Actual | Assessment |
|---|---|---|---|
| **TL-3** | 71.0 kips | 99 kips | **Satisfactory** |
| **TL-4** | 80.0 kips | 63 kips | **Not Satisfactory** |

### 3.2.1.4 Summary

As summarized in Table 3-5, 94-134.115 North Dakota Bridge Rail with curb 94-134.115 as shown in Figure 3-2 does not satisfy MASH TL-4 strength evaluation criteria but meets MASH TL-3 strength criteria. The assessment of occupant risk is considered satisfactory as the geometrics of the bridge rail are located in the preferred and low snag potential regions for the post setback criteria (see Figure 3-3).

Based on the analyses, the retrofit design does not the the strength requirements for MASH TL-4. However, full-scale static load testing is recommended to determine the ultimate strength of the post based can be achieved for MASH TL-4.

**Table 3-5. Summary of MASH TL-4 Assessment of 94-134.115 curb**

| | Required | Actual | Assessment |
|---|---|---|---|
| **Stability** | 36 in. | 38 in. | **Satisfactory** |
| **Geometric** | See Figure 3-3 | | **Satisfactory** |
| **Strength** | See Table 3-4 | | **Not Satisfactory (acceptable for TL-3 based on analyses)** |

### 3.2.2 North Dakota Bridge Rail with Curb 21-106.109

Figure 3-4 shows TTI's proposed retrofit design for the North Dakota's Bridge Rail with Curb 21-106.109. The total height of the bridge rail system is 38 in. from the roadway surface to maintain the post and rail details as tested from TTI project 608331. The posts for this retrofit design were anchored to the curb using 7/8-in. diameter anchor rods with Hilti RE500 V3 Epoxy. The anchor rods were embedded 8 in. and 6.5 in. on the traffic and field sides, respectively. The different embedded lengths were used to avoid interference with existing reinforcement in curb and also to secure concrete cover and construction tolerances.

Appendix A-2 and B-2 contain the suggested details and full analysis for the retrofit design of 21-106.109, respectively. Below is a summary of the evaluation results and recommendations.

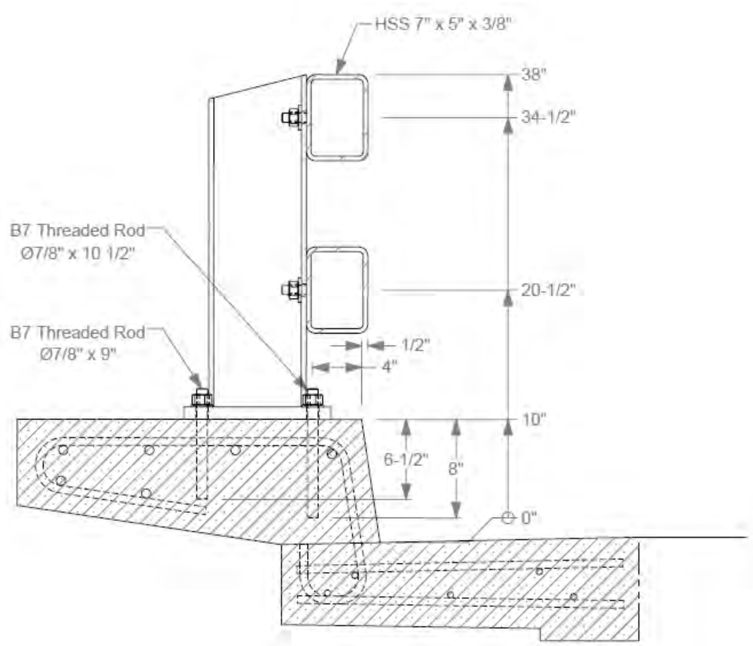


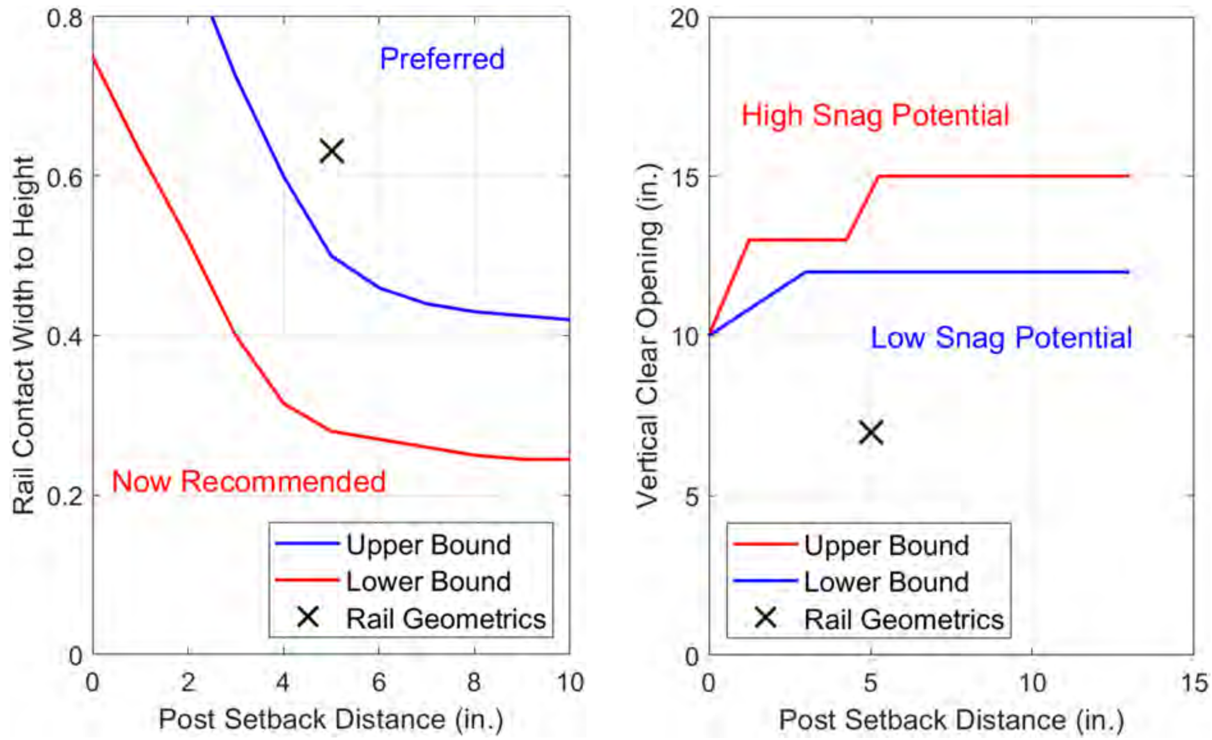

**Figure 3-4. TTI retrofit design for the 21-106.109 curb**

#### 3.2.2.1 Stability Evaluation

The retrofit design for North Dakota Bridge Rail with Concrete curb 21-106.109 has a height of 38-in. The minimum height requirement for a MASH TL-4 bridge barrier is 36-in. Thus, the retrofit design of curb 21-106.109 meets the MASH TL-4 minimum height stability criterion (Satisfactory).

#### 3.2.2.2 Geometric Evaluation

Post setback distance, ratio of contact width to height, and vertical clear opening were determined for the proposed retrofit design for the North Dakota Bridge Rail with Curb 21-106.109. The appropriate data points were plotted against the current AASHTO LRFD Section 13 geometric relationships. As seen in Figure 3-5, the bridge rails' geometric data points are located in the preferred region for the Post Setback criteria and in the acceptable region for the Snag Potential criteria. Therefore, the retrofit design satisfies the geometric evaluation criteria.




(a) Post Setback Criteria (b) Snag Potential Criteria

**Figure 3-5. Geometric criteria assessment of the 21-106.109 curb retrofit design**

### 3.2.2.3 Strength Evaluation

The structural capacity of the North Dakota Bridge Rail with Curb 21-106.109 was compared to the current recommended MASH TL-4 design transverse impact load provided in Table 3-2. Since the height of rail is 38-in. which exceeds 36 in., the desired MASH TL-4 design impact load ($F_t$) is 80 kips located at an effective height ($H_e$) of 30 in. above the roadway surface as per TL-4(b) in Table 3-2.

Table 3-3 presents the post strengths based on 9 ultimate strength load cases from the strength analysis for this retrofit design. As summarized in the table, the limiting post strength was the ultimate strength of the post based on adhesive anchor bolts in the curb. The adhesive bond strength of the anchor in the curb governs the ultimate strength of the posts. Based on this strength, the calculated strength of the retrofit design was approximately 60 kips at $H_e$ = 30 in., over a 3-span failure mechanism. Based on this calculated strength, the proposed retrofit design does not meet the target strength of 80 kips required for MASH TL-4. TTI researchers recommend performing static load testing of the posts or full-scale crash testing for MASH TL-4. Table 3-7 summarizes the strength of the retrofit design of the North Dakota Bridge Rail with Curb 21-106.109.

**Table 3-6. Summary of post strengths based on several critical cases for 21-106.109 curb**

| | |
|---|---|
| 1) Post strength based on plastic strength of the post | 70.0 kips |
| 2) Post strength based on post fillet welds | 69.8 kips |
| 3) Post strength based on anchor bolt shear strength | 130.0 kips |
| 4) Post strength based on ultimate tensile strength of anchor bolts | 46.4 kips |
| 5) Post strength based on the curb punching shear strength | 56.4 kips |
| 6) Post strength based on lateral punching shear on the traffic side bolts | 50.9 kips |
| 7) Post strength based on flexural resistance (FS1) | 31.7 kips |
| 8) Post strength based on flexural resistance (FS2) | 28.7 kips |
| 9) Post strength based on ultimate strength of adhesive dowels in curb | **20.5 kips** |

**Table 3-7. Summary of MASH TL-3 and 4 Strength Analysis for 21-106.109 curb**

| Criteria | Required | Actual | Assessment |
|---|---|---|---|
| **TL-3** | 71.0 kips | 94 kips | **Satisfactory** |
| **TL-4** | 80.0 kips | 60 kips | **Not Satisfactory** |

#### 3.2.2.4 Summary

As summarized in Table 3-8, 21-106.109 North Dakota concrete curb in Figure 3-4 does not satisfy MASH TL-4 strength evaluation criteria but meets MASH TL-3 strength criteria. The assessment of occupant risk is considered satisfactory as the geometrics of the bridge rail are located in the preferred and low snag potential regions for the post setback criteria (see Figure 3-5).

Based on the analyses, the retrofit design does not the the strength requirements for MASH TL-4. However, full-scale static load testing is recommended to determine the ultimate strength of the post based can be achieved for MASH TL-4.

**Table 3-8. Summary of MASH TL-4 Assessment of 21-106.109 curb**

| | Required | Actual | Assessment |
|---|---|---|---|
| **Stability** | 36 in. | 38 in. | **Satisfactory** |
| **Geometric** | See Figure 3-5 | | **Satisfactory** |
| **Strength** | See Table 3-7 | | **Not Satisfactory** |

### 3.2.3 North Dakota Bridge Rail with curb 2-149.663

Figure 3-6 shows TTI's proposed retrofit design for the North Dakota Bridge Rail with Curb 2-149.663. The total height of the bridge rail system is 38 in. from the roadway surface to maintain the post and rail details as tested from TTI project 608331. The posts for this retrofit design were anchored to the curb using 7/8-inch diameter anchor rods with Hilti RE500 V3 Epoxy. The anchor rods were embedded 8.0 inches and 6.5 inches on the traffic sides and field sides, respectively. The different embedded lengths are used to avoid interference with existing reinforcement in curb and also to secure concrete cover and construction tolerances.

Appendix A-3 and B-3 contain the suggested details and full analysis for the retrofit design of 2-149.663, respectively. Below is a summary of the evaluation results and recommendations.

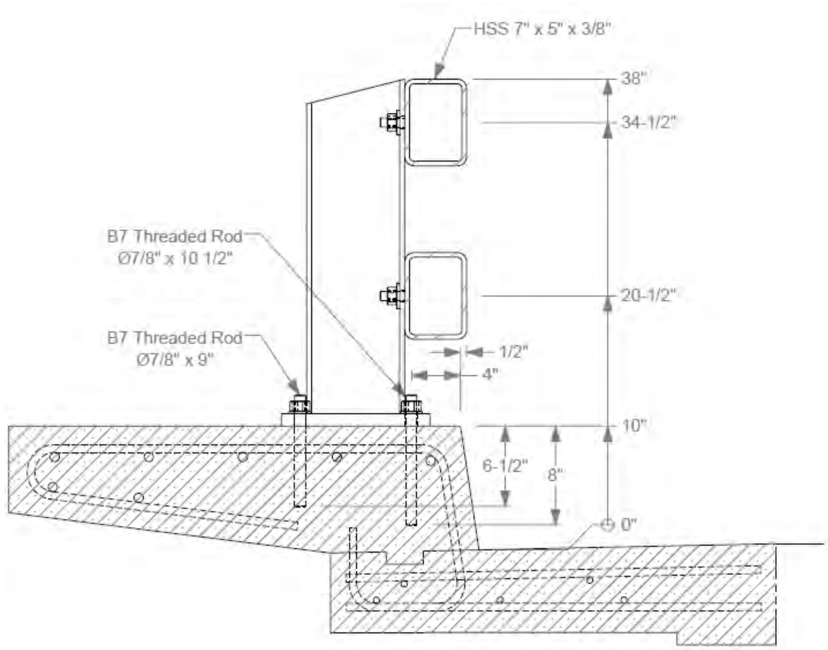


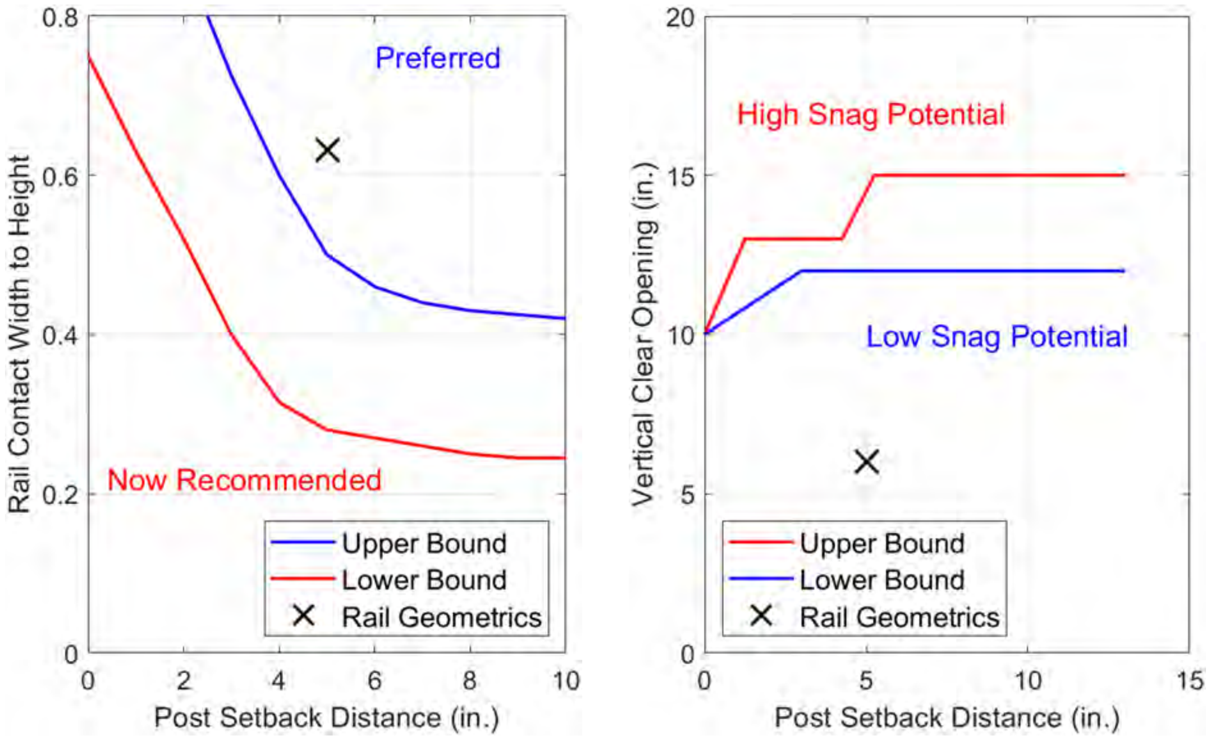

**Figure 3-6. TTI retrofit design for the 2-149.663 curb**

#### 3.2.3.1 Stability Evaluation

The retrofit design for North Dakota Bridge Rail with Curb 2-149.663 has a height of 38-in. The minimum height requirement for a MASH TL-4 bridge barrier is 36-in. Thus, the retrofit design of curb 2-149.663 meets the MASH TL-4 minimum height stability criterion (Satisfactory).

#### 3.2.3.2 Geometric Evaluation

Post setback distance, ratio of contact width to height, and vertical clear opening were determined for the proposed retrofit design for the retrofit design. The appropriate data points were plotted against the current AASHTO LRFD Section 13 geometric relationships. As seen in Figure 3-7, the bridge rails' geometric data points are located in the preferred region for the Post Setback criteria and in the acceptable region for the Snag Potential criteria. Therefore, the retrofit design of North Dakota Bridge Rail with Concrete Curb 2-149.663 satisfies the geometric evaluation criteria.




(a) Post Setback Criteria (b) Snag Potential Criteria

**Figure 3-7. Geometric criteria assessment of the 2-149.663 curb retrofit design**

### 3.2.3.3 Strength Evaluation

The structural capacity of the 2-149.663 curb's retrofit design was compared to the current recommended MASH TL-4 design transverse impact load provided in Table 3-2. Since the height of rail is 38 in. which exceeds 36 in., the desired MASH TL-4 design impact load ($F_t$) is 80 kips located at an effective height ($H_e$) of 30 in. above the roadway surface as per TL-4(b) in Table 3-2.

Table 3-3 presents the post strengths based on 9 ultimate strength load cases from the strength analysis for this retrofit design. As summarized in the table, the limiting post strength was the ultimate strength of the post based on adhesive anchor bolts in the curb. The adhesive bond strength of the anchor in the curb governs the ultimate strength of the posts. Based on this strength, the calculated strength of the retrofit design was approximately 75 kips at $H_e$ = 30 in. Based on this calculated strength, the proposed retrofit design does not meet the target strength of 80 kips required for MASH TL-4. TTI researchers recommend performing static load testing of the posts or full-scale crash testing for MASH TL-4. Table 3-10 summarizes the strength of the retrofit design the North Dakota Bridge Rail with Curb 2-149.663.

**Table 3-9. Summary of post strengths based on several critical cases for 2-149.663 curb**

| | |
|---|---|
| 1) Post strength based on plastic strength of the post | 74.5 kips |
| 2) Post strength based on post fillet welds | 74.3 kips |
| 3) Post strength based on anchor bolt shear strength | 130.0 kips |
| 4) Post strength based on ultimate tensile strength of anchor bolts | 49.2 kips |
| 5) Post strength based on the curb punching shear strength | 73.4 kips |
| 6) Post strength based on lateral punching shear on the traffic side bolts | 66.9 kips |
| 7) Post strength based on flexural resistance (FS1) | 66.2 kips |
| 8) Post strength based on flexural resistance (FS2) | 32.1 kips |
| 9) Post strength based on ultimate strength of adhesive dowels in curb | **21.8 kips** |

**Table 3-10. Summary of MASH TL-3 and 4 Strength Analysis for 2-149.663 curb**

| Criteria | Required | Actual | Assessment |
|---|---|---|---|
| **TL-3** | 71.0 kips | 118 kips | **Satisfactory** |
| **TL-4** | 80.0 kips | 75 kips | **Not Satisfactory** |

### 3.2.3.4 Summary

As summarized in Table 3-11, 2-149.663 North Dakota Bridge Rail with Curb 2-149.663 as shown in Figure 3-6 does not satisfy MASH TL-4 strength evaluation criteria but meets MASH TL-3 strength criteria. The assessment of occupant risk is considered satisfactory as the geometrics of the bridge rail are located in the preferred and low snag potential regions for the post setback criteria (see Figure 3-7).

Based on the analyses, the retrofit design does not the the strength requirements for MASH TL-4. However, full-scale static load testing is recommended to determine the ultimate strength of the post based can be achieved for MASH TL-4.

**Table 3-11. Summary of MASH TL-4 Assessment of 2-149.663 curb**

| | Required | Actual | Assessment |
|---|---|---|---|
| **Stability** | 36 in. | 38 in. | **Satisfactory** |
| **Geometric** | See Figure 3-7 | | **Satisfactory** |
| **Strength** | See Table 3-10 | | **Not Satisfactory** |

### 3.2.4 Alaska Steel Bridge Railing Design

Figure 3-8 shows TTI's proposed retrofit design for the Alaska Steel Bridge Railing. The total height of the bridge rail system is 38 in. from the roadway surface to maintain the post and rail details as tested from TTI project 608331. The retrofit design using the Alaska 2-Tube Bridge Rail incorporates anchor bolts with a single plate washer to connect curb, deck, and post.

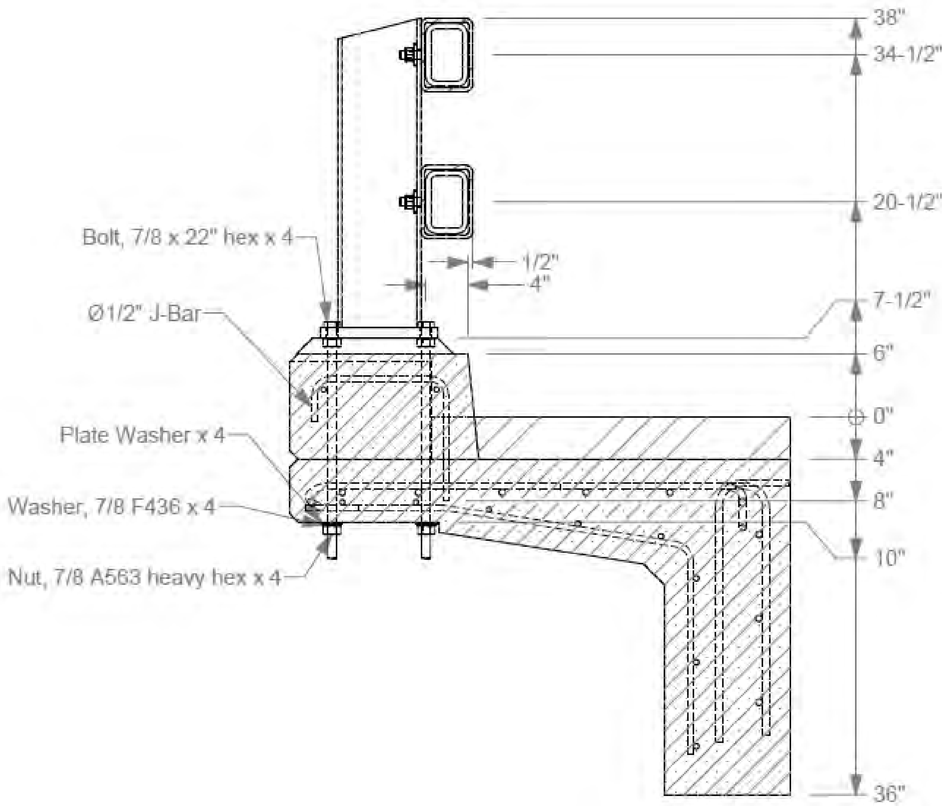



**Figure 3-8. TTI retrofit design for the Alaska steel bridge railing retrofit design**

The initial TTI design suggested using bolt-through anchors through the deck and curb with a large plate washer for 2 anchors for each post. However, it was concluded that using a single plate washer for each bolt-through anchor was preferred for ease of construction.

For this retrofit design, #4 J-dowel bars are embedded into the bridge deck using Hilti RE500 V3 Epoxy to anchor the curb to the deck. These bars can be installed at a depth of 4.0 inches to help reduce contaminated drilling water from entering into Alaska rivers and streams (preference by Alaska DOT). These #4 J-dowels bars are spaced on 16-inch center between the posts with the bars spaced on 12-inch centers away from the centerline of the posts. For anchoring the posts to the curb and deck, separate bolt through anchors were recommended.

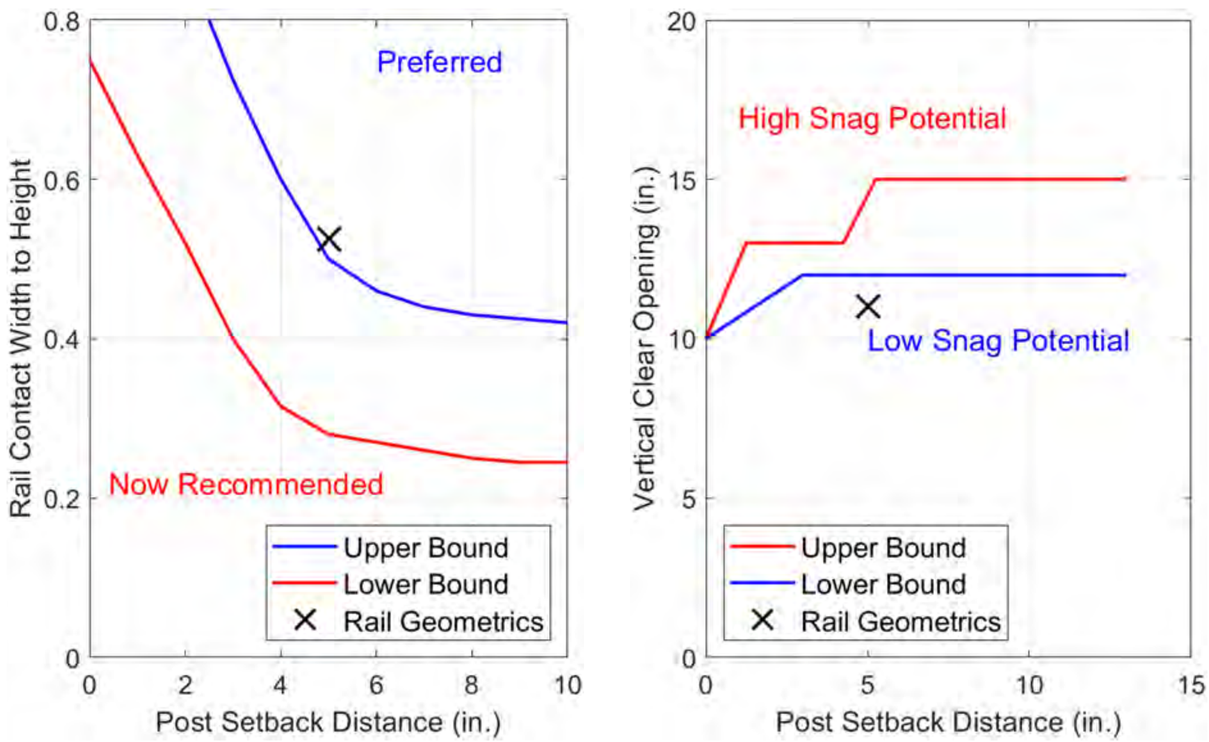
Appendix A-4 and B-4 contain the suggested details and full analysis for the retrofit design of Alaska steel bridge railing design, respectively. Below is a summary of the evaluation results and recommendations.

#### 3.2.4.1 Stability Evaluation

The retrofit design for Alaska steel bridge railing system has a height of 38-in. The minimum height requirement for a MASH TL-4 bridge barrier is 36-in. Thus, the retrofit design of Alaska steel bridge railing system meets the MASH TL-4 minimum height stability criterion (Satisfactory).

#### 3.2.4.2 Geometric Evaluation

Post setback distance, ratio of contact width to height, and vertical clear opening were determined for the proposed retrofit design for Alaska steel bridge railing system. The appropriate data points were plotted against the current AASHTO LRFD Section 13 geometric relationships. As seen in Figure 3-9, the bridge rails' geometric data points are located in the preferred region for the Post Setback criteria and in the acceptable region for the Snag Potential criteria. Therefore, the retrofit design of Alaska steel bridge railing system satisfies the geometric evaluation criteria.




(a) Post Setback Criteria (b) Snag Potential Criteria

**Figure 3-9. Geometric criteria assessment of the Alaska steel bridge railing retrofit design**

#### 3.2.4.3 Strength Evaluation

The structural capacity of Alaska steel bridge rail retrofit design was compared to the current recommended MASH TL-4 design transverse impact load provided in Table 3-2. Since the height of rail is 38 in. which exceeds 36 in., the desired MASH TL-4 design impact load ($F_t$) is 80 kips located at an effective height ($H_e$) of 30 in. above the roadway surface as per TL-4(b) in Table 3-2.

Table 3-12 presents the post strengths based on 7 critical cases from the strength analysis for the retrofit design. As summarized in the table, post strength based on flexural resistance was the governing strength. However, this strength is governed by a post located immediately adjacent to a joint in the deck. According to many crash tests conducted, deck failure is typically observed at an open joint over several spans (3 or more). Thus, computing the design strength based on flexural resistance likely may not reflect the actual strength of this deck failure mechanism. Based on the observation from the crash test, it is considered that averaging the strength of 3 posts may be a more realistic failure scenario.

When calculating the rail strength for multiple spans at $H_e$ = 30 in., a resistance for 3-span controls the strength. The strength of the retrofit design is approximately 81 kips at the TL-4 height of 30 in. The retrofit design, thus meets the target strength of 80 kips given by MASH TL-4 as shown in Table 3-13.

**Table 3-12. Summary of post strengths based on several critical cases for Alaska steel bridge railing retrofit design**

| | |
|---|---|
| 1) Post strength based on plastic strength of the post | 60.8 kips |
| 2) Post strength based on post fillet welds | 60.6 kips |
| 3) Post strength based on anchor bolt shear strength | 130.0 kips |
| 4) Post strength based on ultimate tensile strength of anchor bolts | 43.3 kips |
| 5) Post strength based on the curb punching shear strength | 57.9 kips |
| 6) Post strength based on cone-shaped punching shear | 68.2 kips |
| 7) Post strength based on flexural resistance | **31.7 kips** |

**Table 3-13. Summary of MASH TL-4 Strength Analysis for Alaska steel bridge railing retrofit design**

| Criteria | Required | Actual | Assessment |
|---|---|---|---|
| **TL-4** | 80.0 kips | 81 kips | **Satisfactory** |

#### 3.2.4.4 Summary

As summarized in Table 3-14, Alaska steel bridge railing retrofit design in Figure 3-8 satisfies MASH TL-4 strength evaluation criteria. Also, the assessment of occupant risk is considered satisfactory as the geometrics of the bridge rail are located in the preferred and low snag potential regions for the post setback criteria (see Figure 3-9). Therefore, the retrofit design for Alaska steel bridge railing meets MASH TL-4 criteria.

**Table 3-14. Summary of MASH TL-4 Assessment of Alaska steel bridge railing retrofit design**

| | **Required** | **Actual** | **Assessment** |
|---|---|---|---|
| **Stability** | 36 in. | 38 in. | **Satisfactory** |
| **Geometric** | See Figure 3-9 | | **Satisfactory** |
| **Strength** | See Table 3-10 | | **Satisfactory** |

# REFERENCES

[1] AASHTO. 2016. Manual for Assessing Safety Hardware. American Association of State Highway and Transportation Officials, Washington, DC.

[2] Williams, W. F., Menges, W. L., and Griffith, B.L. (2019). MASH TL-4 Evaluation of 2019 MASH 2-Tube Bridge Railings (Report No. 608331-01-1A-2-3). College Station, TX: Texas A&M Transportation Institute.

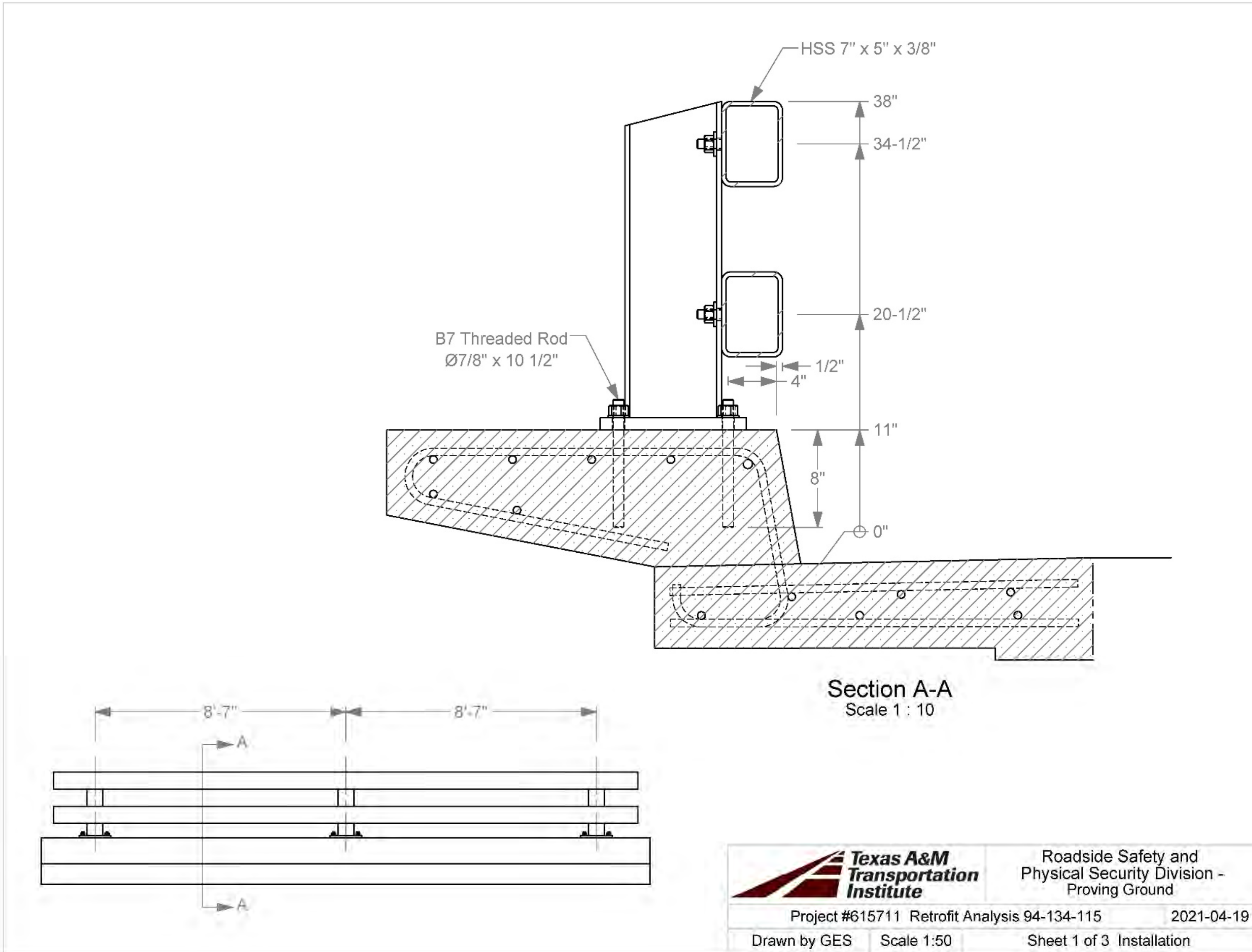
[3] AASHTO. 2020. AASHTO LRFD Bridge Design Specification, 9th ed. American Association of State Highway and Transportation Officials, Washington, DC.

[4] Bligh, R., Williams, W., Silvestri-Dobrovolny, C., Schulz, N., Moran, S., and Skinner, T. (2017). MASH Equivalency of NCHRP Report 350-Approved Bridge Railings (Report No. 607141. NCHRP Project 20-07 Task 395). College Station, TX: Texas A&M Transportation Institute.

[5] Bligh, R. P., Briaud, J. L., Abu-Odeh, A. D., Saez, O., Maddah, L. S., and Kim, K.M. (2017). Design Guidelines for Test Level 3 (TL-3) Through Test Level 5 (TL-5) Roadside Barrier Systems Placed on Mechanically Stabilized Earth (MSE) Retaining Wall (NCHRP Project No. 22-20(2)). College Station, TX: Texas A&M Transportation Institute.

# APPENDIX A. TTI Retrofit Designs

## A-1. Retrofit Design of North Dakota Curb 94-134.115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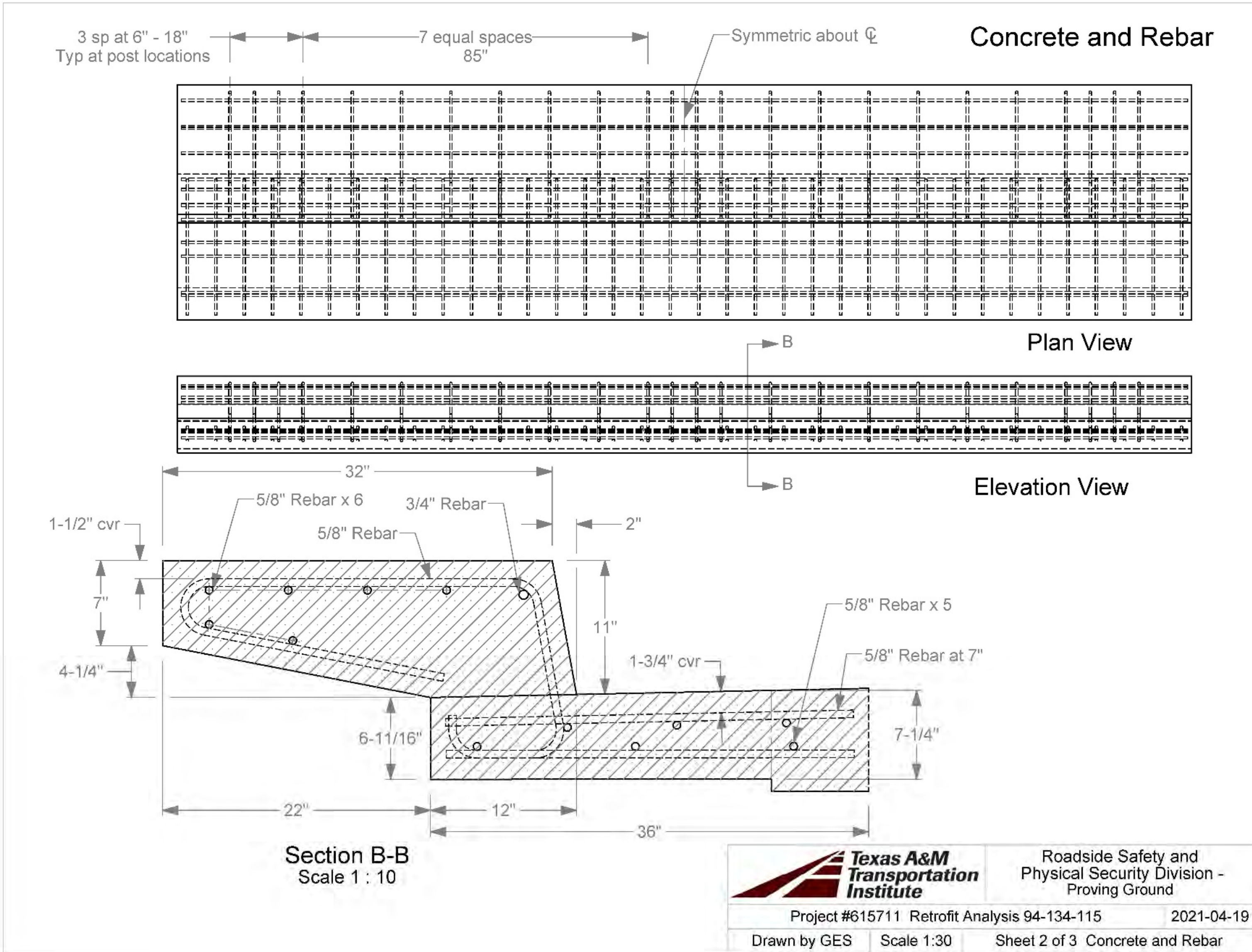
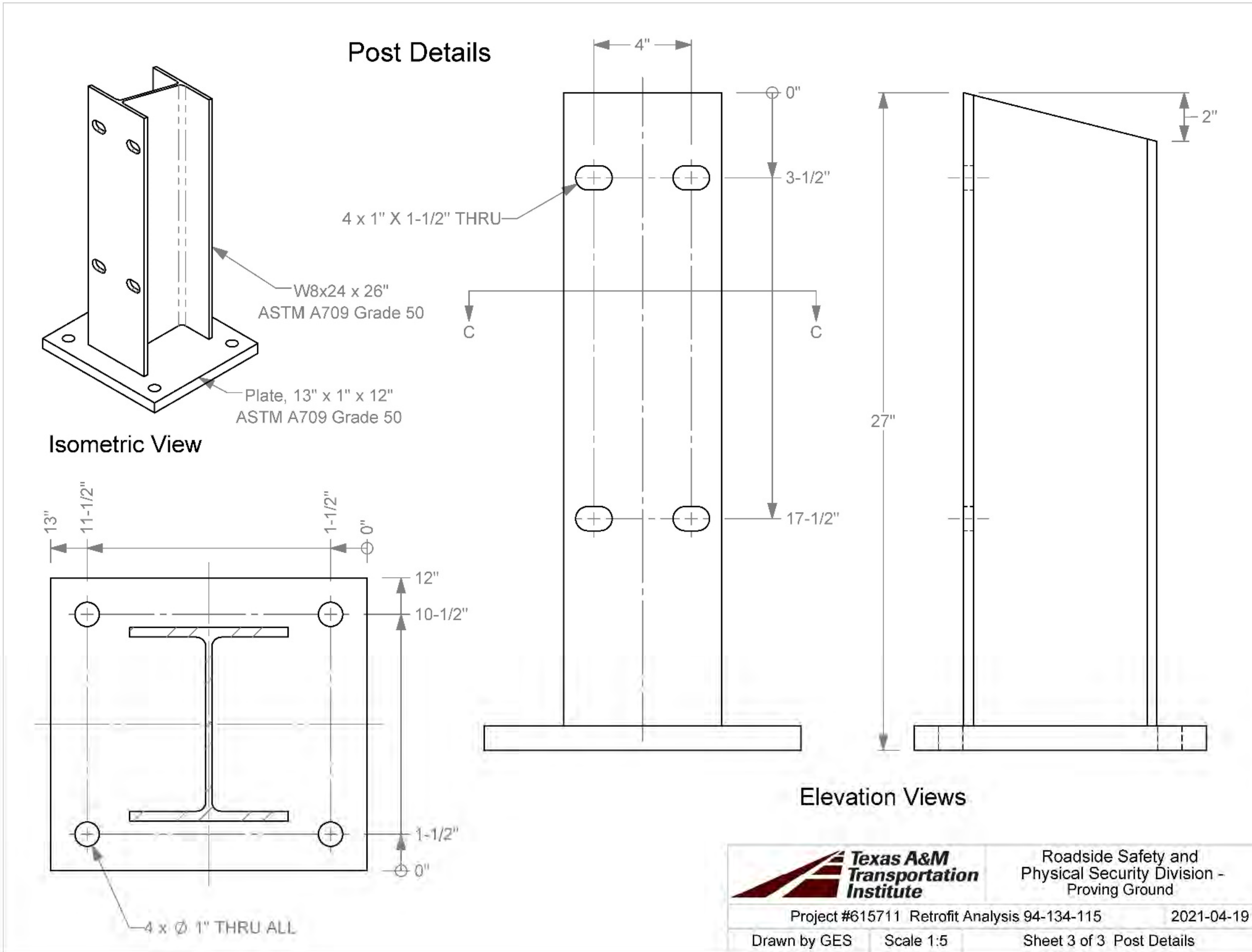
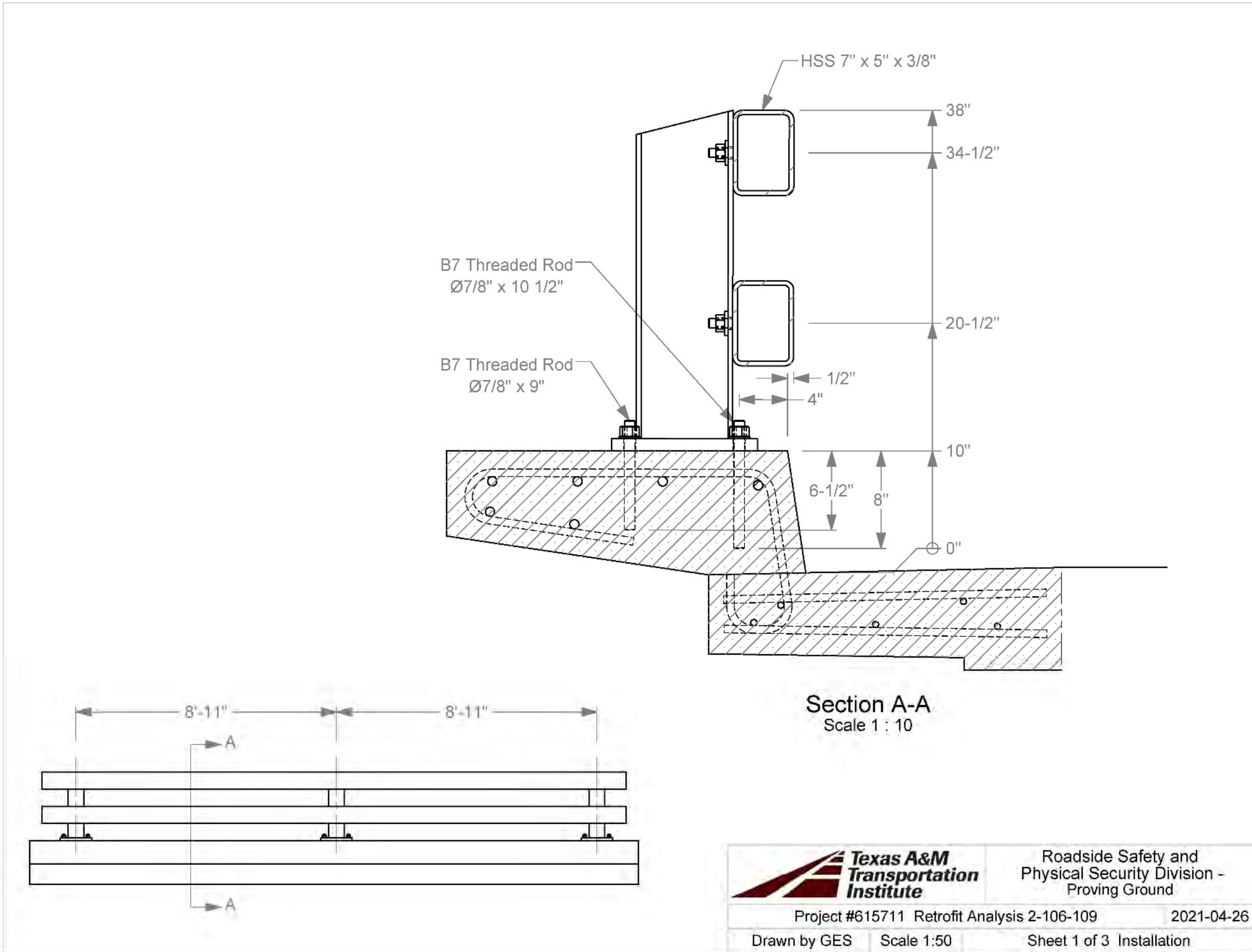
38"
34-1/2"
20-1/2"
B7 Threaded Rod
Ø7/8" x 10 1/2"
1/2"
4"
11"
8"
0"
Section A-A
Scale 1 : 10
8'-7"
8'-7"
A
A
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 94-134-115
2021-04-19
Drawn by GES
Scale 1:50
Sheet 1 of 3 Installation

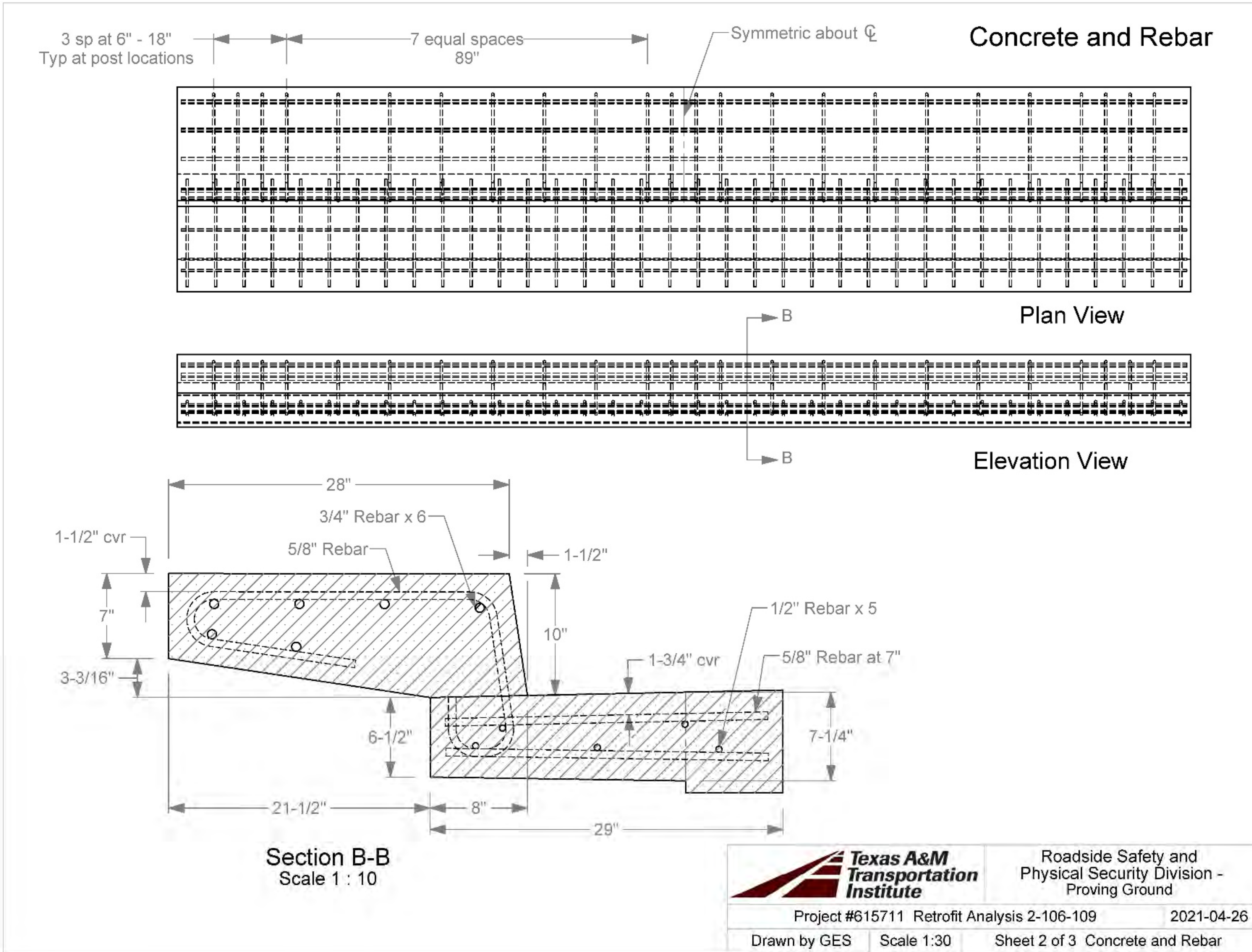


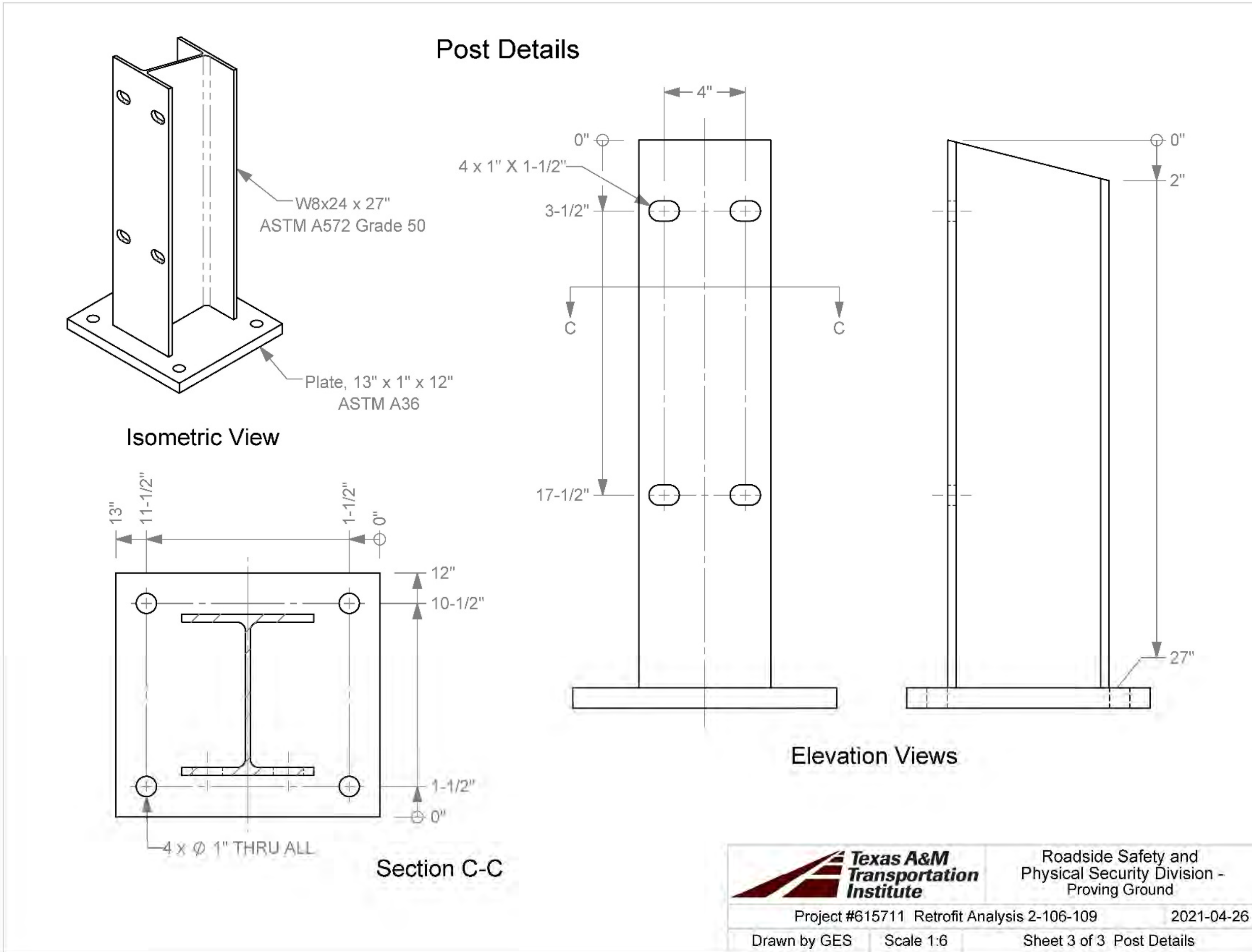
Concrete and Rebar
3 sp at 6" - 18"
Typ at post locations
7 equal spaces
85"
Symmetric about ℄
Plan View
B
B
Elevation View
32"
1-1/2" cvr
5/8" Rebar x 6
3/4" Rebar
5/8" Rebar
2"
7"
4-1/4"
11"
5/8" Rebar x 5
5/8" Rebar at 7"
1-3/4" cvr
6-11/16"
7-1/4"
22"
12"
36"
Section B-B
Scale 1 : 10
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 94-134-115
2021-04-19
Drawn by GES
Scale 1:30
Sheet 2 of 3 Concrete and Rebar


Post Details
W8x24 x 26"
ASTM A709 Grade 50
Plate, 13" x 1" x 12"
ASTM A709 Grade 50
Isometric View
4 x 1" X 1-1/2" THRU
4"
0"
3-1/2"
17-1/2"
C
C
2"
27"
13"
11-1/2"
1-1/2"
0"
12"
10-1/2"
1-1/2"
0"
4 x Ø 1" THRU ALL
Elevation Views
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 94-134-115
2021-04-19
Drawn by GES
Scale 1:5
Sheet 3 of 3 Post Details

## A-2. Retrofit Design of North Dakota Curb 21-106.109


HSS 7" x 5" x 3/8"
38"
34-1/2"
B7 Threaded Rod
Ø7/8" x 10 1/2"
20-1/2"
B7 Threaded Rod
Ø7/8" x 9"
1/2"
4"
10"
6-1/2"
8"
0"
Section A-A
Scale 1 : 10
8'-11"
8'-11"
A
A
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 2-106-109
2021-04-26
Drawn by GES
Scale 1:50
Sheet 1 of 3 Installation


Concrete and Rebar
3 sp at 6" - 18"
Typ at post locations
7 equal spaces
89"
Symmetric about ℄
Plan View
B
B
Elevation View
28"
3/4" Rebar x 6
5/8" Rebar
1-1/2" cvr
1-1/2"
7"
10"
1/2" Rebar x 5
1-3/4" cvr
5/8" Rebar at 7"
3-3/16"
6-1/2"
7-1/4"
21-1/2"
8"
29"
Section B-B
Scale 1 : 10
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 2-106-109
2021-04-26
Drawn by GES
Scale 1:30
Sheet 2 of 3 Concrete and Rebar



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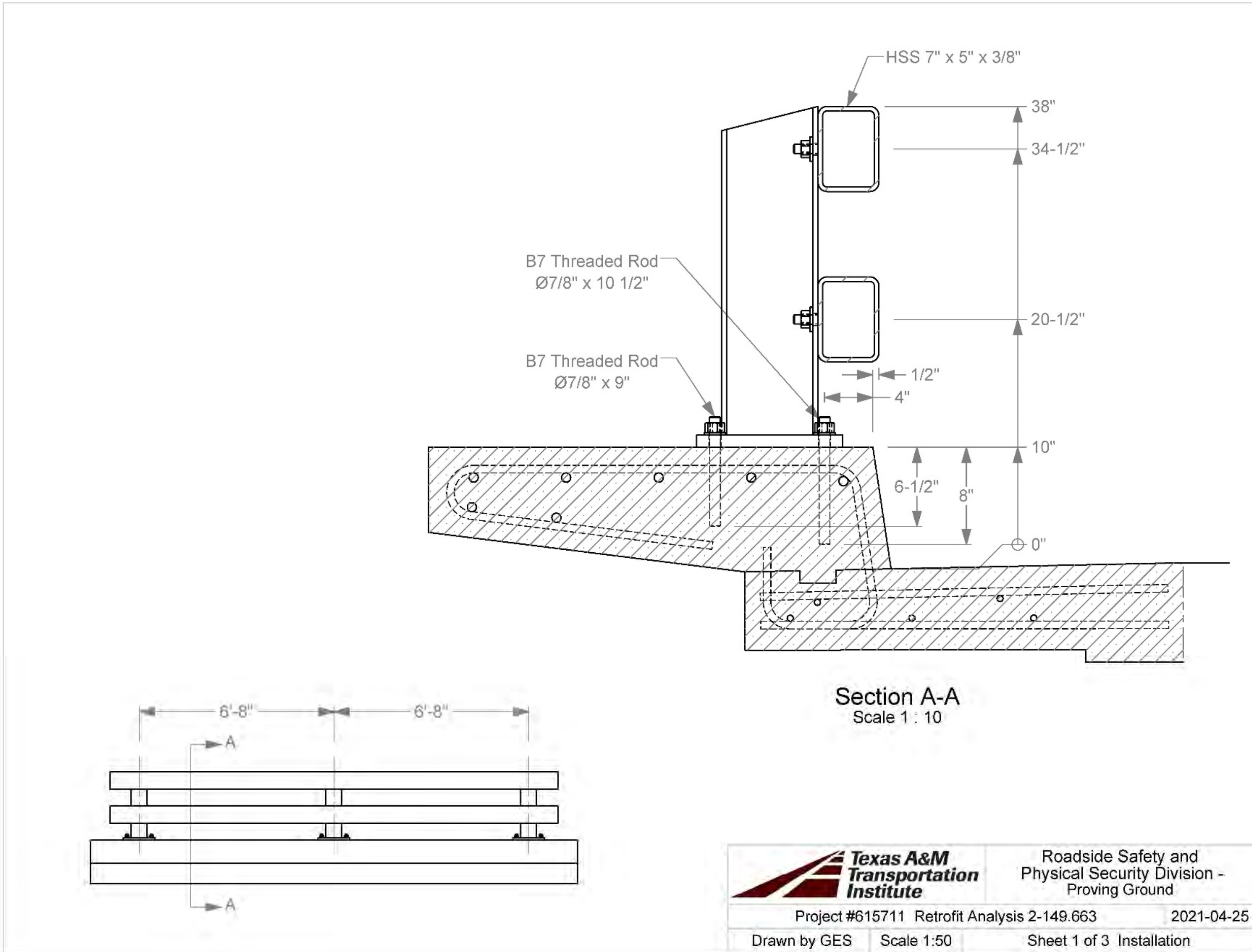
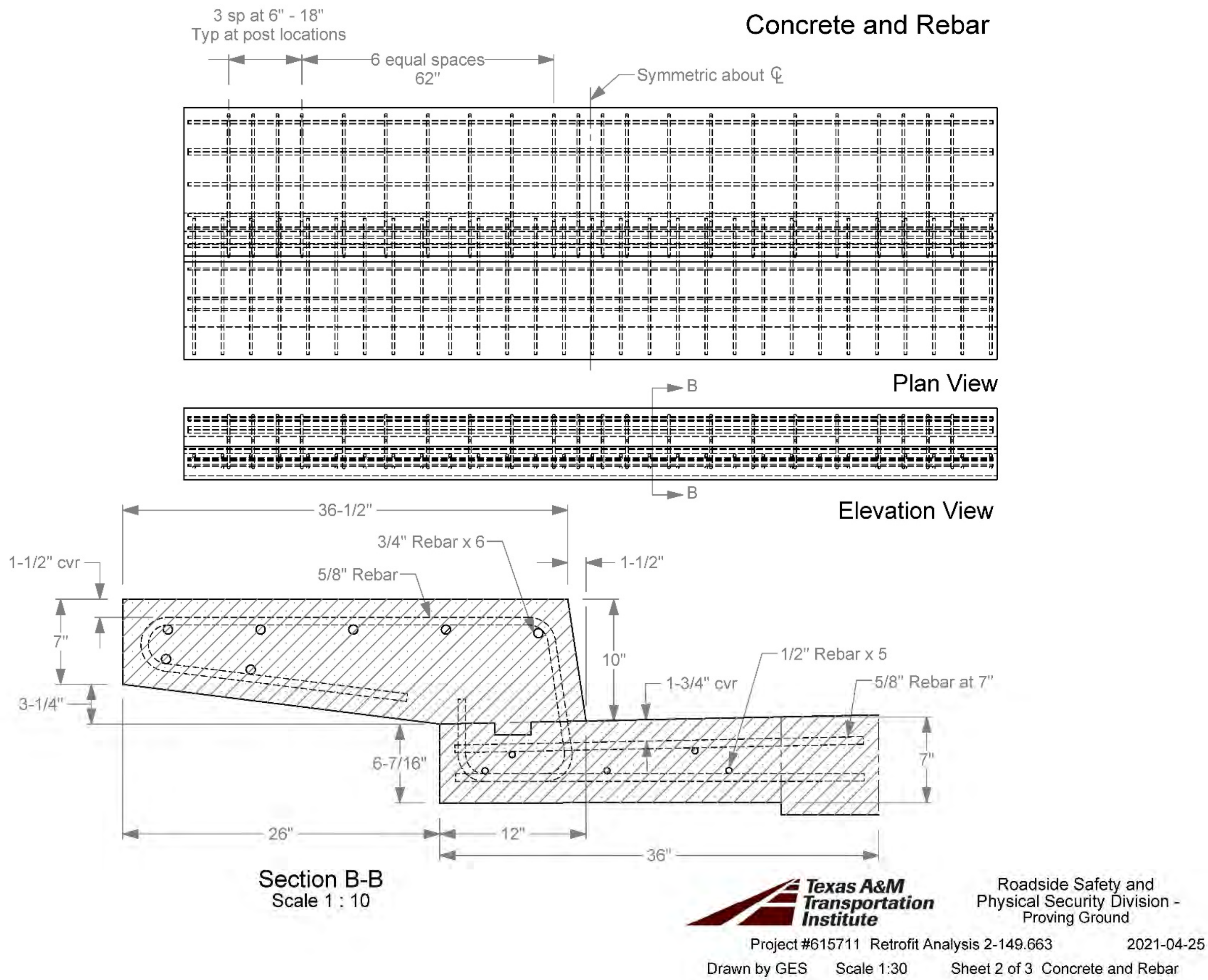
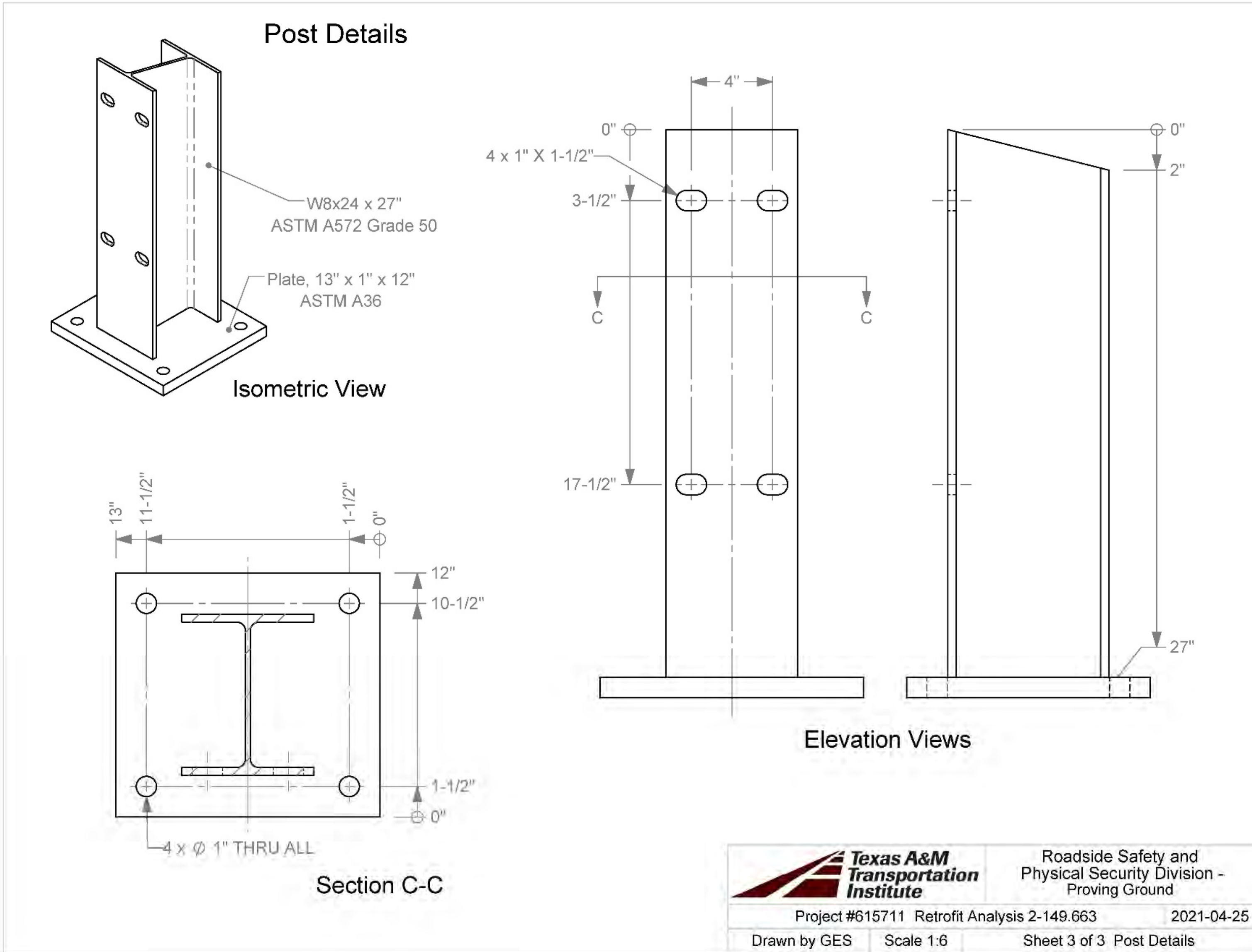
ASTM A572 Grade 50
Plate, 13" x 1" x 12"
ASTM A36
Isometric View
4"
0"
4 x 1" X 1-1/2"
3-1/2"
C
C
17-1/2"
0"
2"
27"
Elevation Views
13"
11-1/2"
1-1/2"
0"
12"
10-1/2"
1-1/2"
0"
4 x ∅ 1" THRU ALL
Section C-C
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 2-106-109
2021-04-26
Drawn by GES
Scale 1:6
Sheet 3 of 3 Post Details

## A-3. Retrofit Design of North Dakota Curb 2-149.663


HSS 7" x 5" x 3/8"
38"
34-1/2"
B7 Threaded Rod
Ø7/8" x 10 1/2"
20-1/2"
B7 Threaded Rod
Ø7/8" x 9"
1/2"
4"
10"
6-1/2"
8"
0"
Section A-A
Scale 1 : 10
6'-8"
6'-8"
A
A
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 2-149.663
2021-04-25
Drawn by GES
Scale 1:50
Sheet 1 of 3 Installation

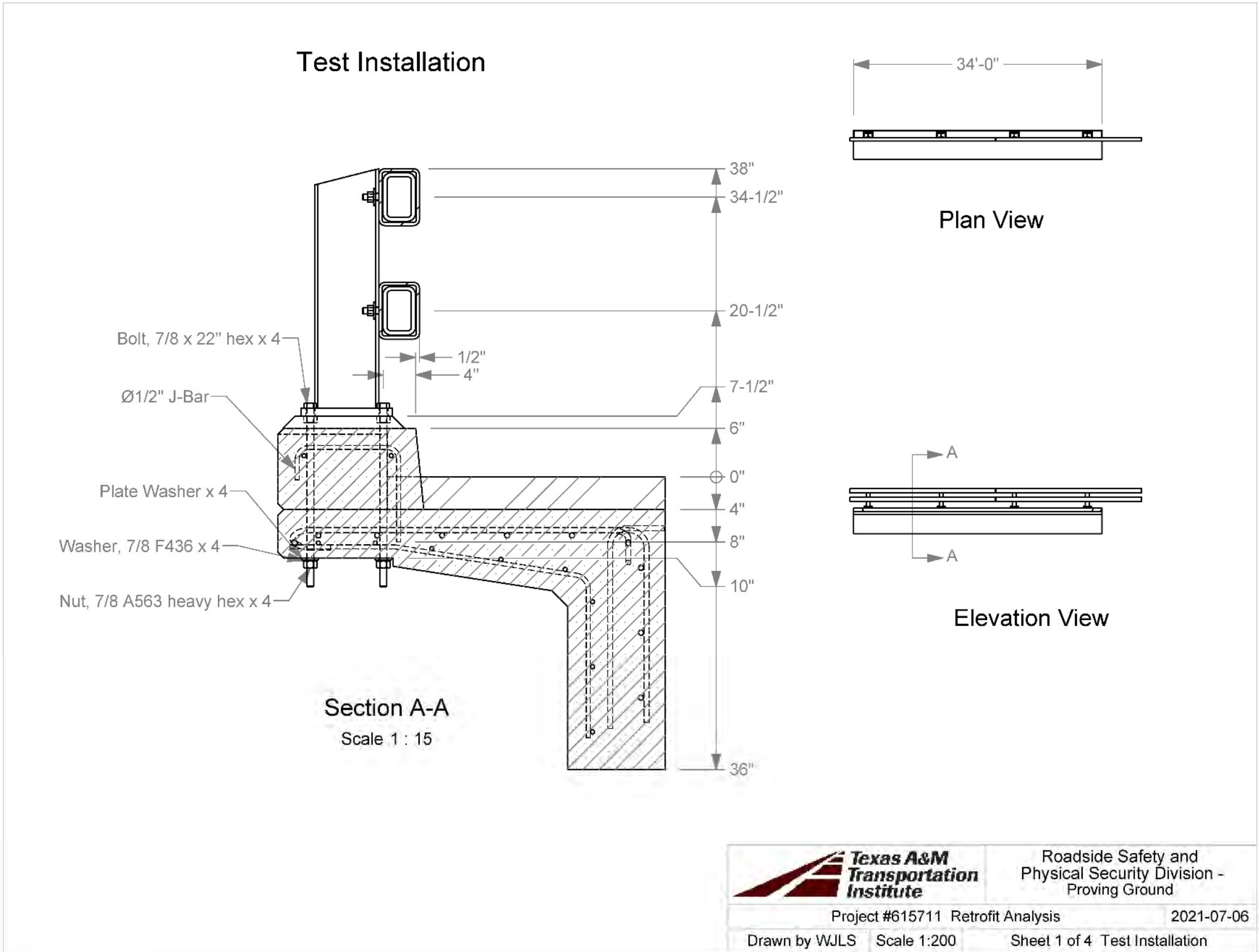


Concrete and Rebar
3 sp at 6" - 18"
Typ at post locations
6 equal spaces
62"
Symmetric about ℄
Plan View
B
B
Elevation View
36-1/2"
3/4" Rebar x 6
5/8" Rebar
1-1/2" cvr
1-1/2"
7"
10"
1/2" Rebar x 5
1-3/4" cvr
5/8" Rebar at 7"
3-1/4"
6-7/16"
7"
26"
12"
36"
Section B-B
Scale 1 : 10
Texas A&M Transportation Institute
Roadside Safety and
Physical Security Division -
Proving Ground
Project #615711 Retrofit Analysis 2-149.663 2021-04-25
Drawn by GES Scale 1:30 Sheet 2 of 3 Concrete and Rebar

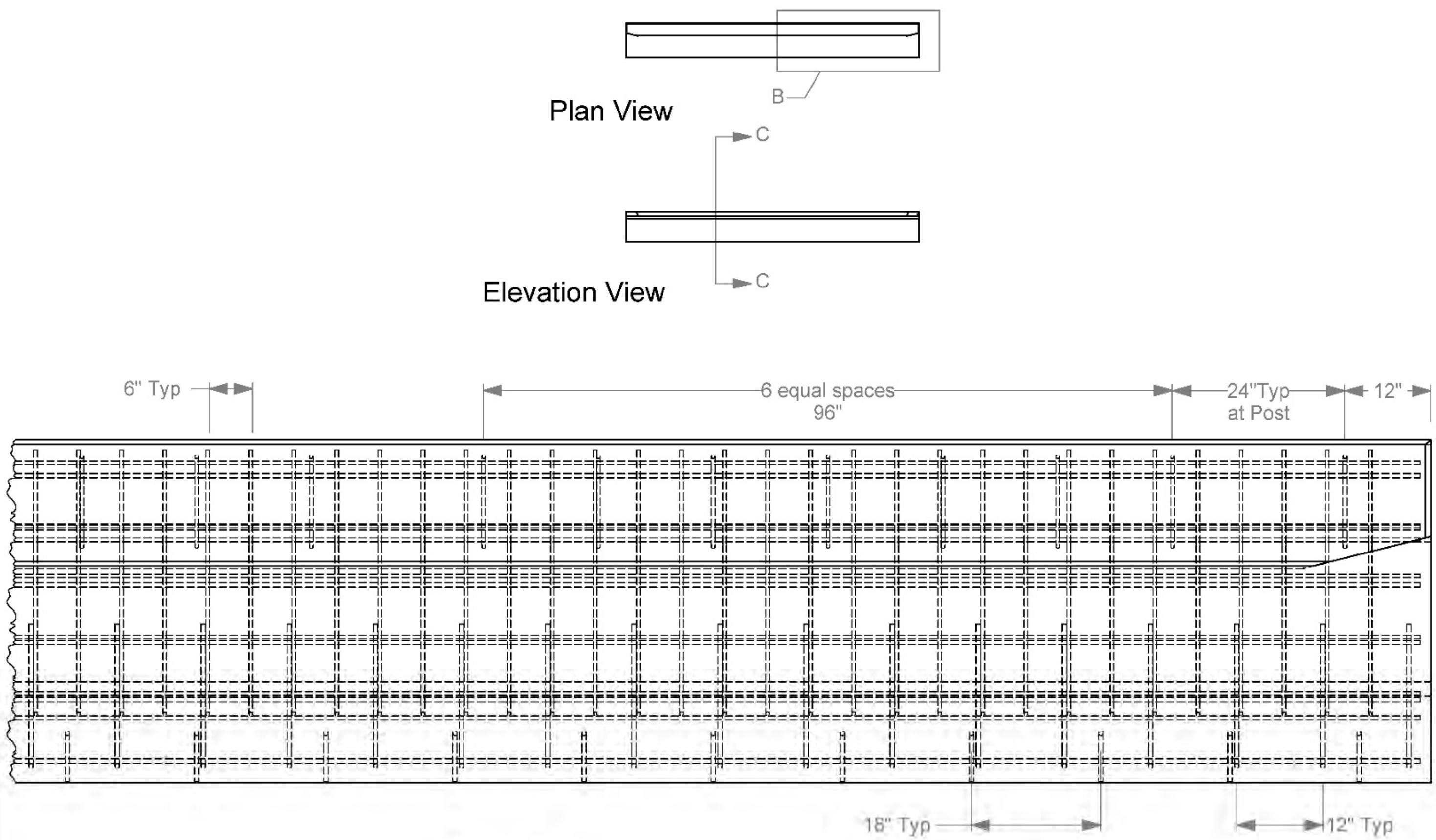


Post Details
W8x24 x 27"
ASTM A572 Grade 50
Plate, 13" x 1" x 12"
ASTM A36
Isometric View
4"
0"
4 x 1" X 1-1/2"
3-1/2"
C
C
17-1/2"
0"
2"
27"
Elevation Views
13"
11-1/2"
1-1/2"
0"
12"
10-1/2"
1-1/2"
0"
4 x ∅ 1" THRU ALL
Section C-C
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis 2-149.663
2021-04-25
Drawn by GES
Scale 1:6
Sheet 3 of 3 Post Details

## A-4. Retrofit Design of Alaska Steel Bridge Railing


Test Installation
34'-0"
Plan View
38"
34-1/2"
20-1/2"
1/2"
4"
7-1/2"
6"
0"
4"
8"
10"
36"
Bolt, 7/8 x 22" hex x 4
Ø1/2" J-Bar
Plate Washer x 4
Washer, 7/8 F436 x 4
Nut, 7/8 A563 heavy hex x 4
A
A
Elevation View
Section A-A
Scale 1 : 15
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis
2021-07-06
Drawn by WJLS
Scale 1:200
Sheet 1 of 4 Test Installation

## Concrete and Rebar




Detail B

Scale 1 : 20

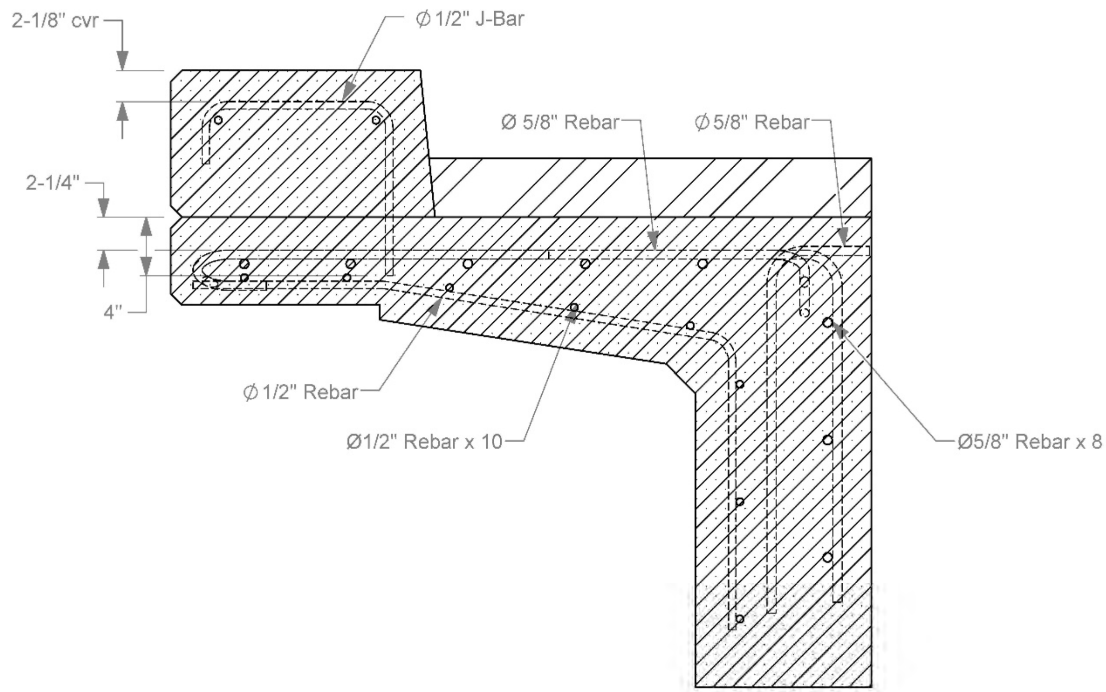
**2a.** Hook bars hidden for clarity.

| | |
|---|---|
| Texas A&M Transportation Institute | Roadside Safety and Physical Security Division - Proving Ground |

Project #615711 Retrofit Analysis 2021-07-06

Drawn by WJLS | Scale 1:200 | Sheet 2 of 4 Concrete and Rebar

## Section C-C

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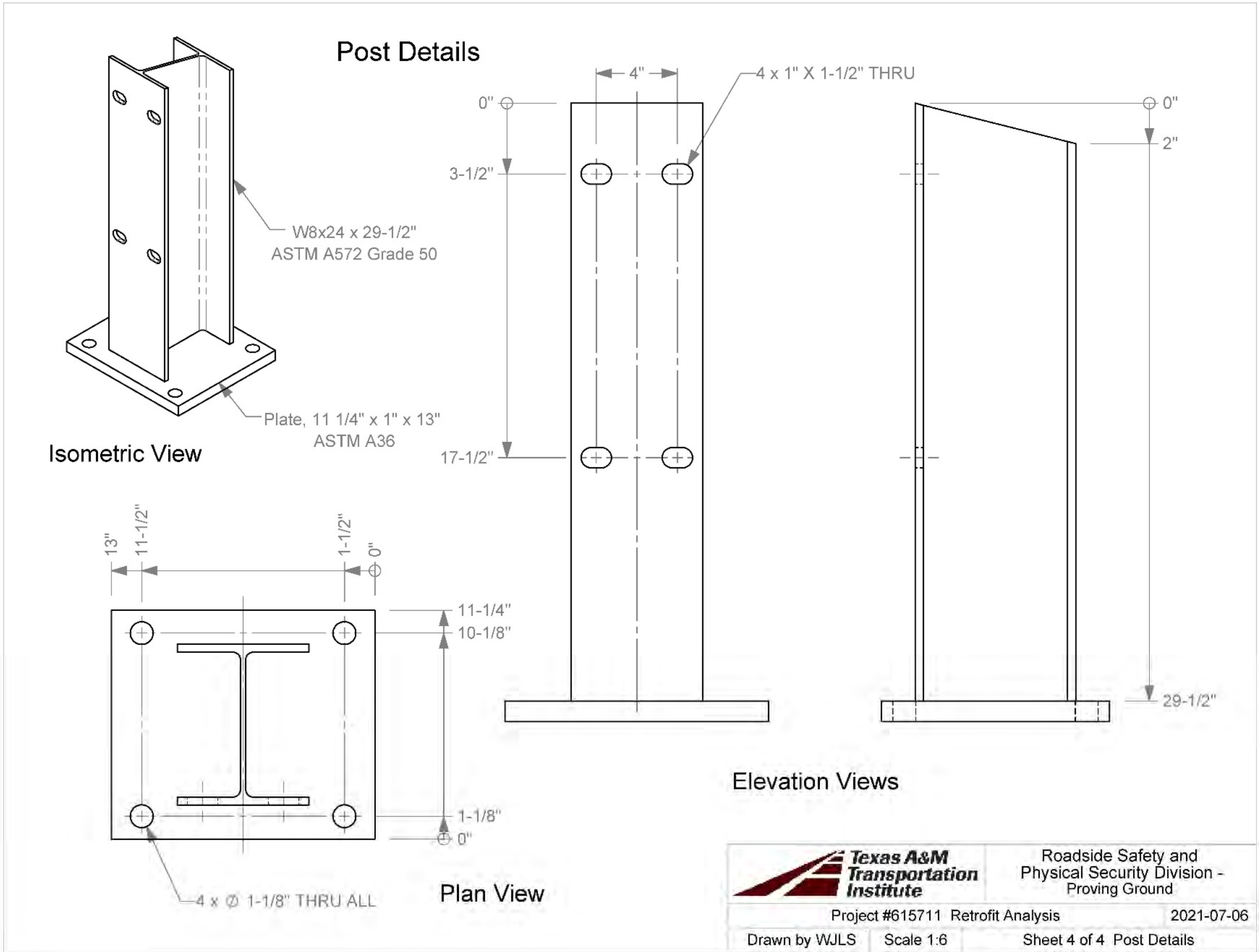
| Texas A&M Transportation Institute | Roadside Safety and Physical Security Division - Proving Ground | |
| Project #615711 Retrofit Analysis | | 2021-07-06 |
| Drawn by WJLS | Scale 1:10 | Sheet 3 of 4 Section C-C |


Post Details
W8x24 x 29-1/2"
ASTM A572 Grade 50
Plate, 11 1/4" x 1" x 13"
ASTM A36
Isometric View
4"
4 x 1" X 1-1/2" THRU
0"
3-1/2"
17-1/2"
0"
2"
29-1/2"
Elevation Views
13"
11-1/2"
1-1/2"
0"
11-1/4"
10-1/8"
1-1/8"
0"
4 x ∅ 1-1/8" THRU ALL
Plan View
Texas A&M Transportation Institute
Roadside Safety and Physical Security Division - Proving Ground
Project #615711 Retrofit Analysis
2021-07-06
Drawn by WJLS
Scale 1:6
Sheet 4 of 4 Post Details

# APPENDIX B. MATHCAD Strength Analysis Sheet

## B-1. Strength Analysis of North Dakota curb 94-134.115 retrofit design

Texas A&M Transportation Institute

**Subject** : **Retrofit Design of the MASH TL-4 Alaska 2-Tube Bridge Rail '94-134-115'**

**Sponsor** : **Alaska Department of Transportation & Public Facilities (AKDOT & PF)**

**(3-conti.) Geometric Criteria:**

$s_{post} = 5\ in$ $\quad \Sigma A = 25\ in$ $\quad H_r = 38\ in$ $\quad ratio_{\Sigma AH} := \dfrac{\Sigma A}{H_r} = 0.658$

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

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

Lower boundary for post setback criteria x and y coordinates

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

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

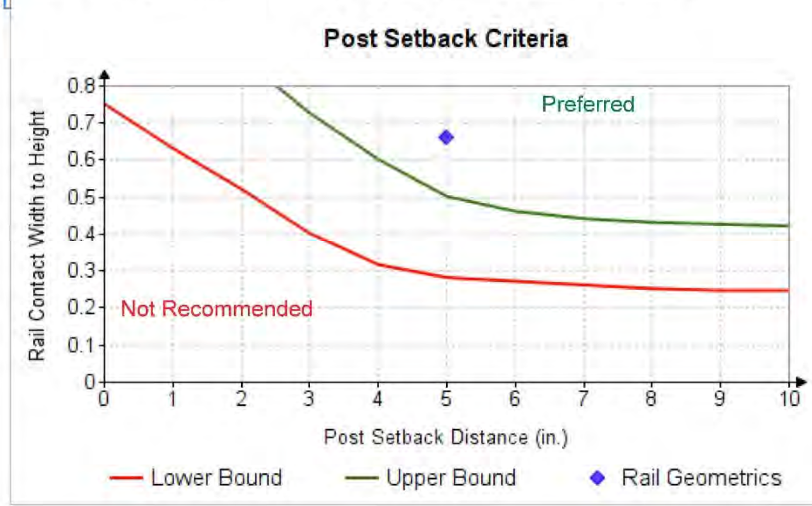
Upper boundary for post setback criteria x and y coordinates

$Set_{sys.x} := \dfrac{s_{post}}{in} = 5$ Post setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.658$ Ratio of contact width to total height rail geometric point

Inputs

$X_1 := Set_{low.x}\quad Y_1 := Set_{low.y}\qquad X_2 := Set_{up.x}\quad Y_2 := Set_{up.y}\qquad X_3 := Set_{sys.x}\quad Y_3 := Set_{sys.y}$




$NotRecommended := 1 \qquad Marginal := 2 \qquad Preferred := 3$

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

$snag_{up.x} := [0\ \ 1.25\ \ 4.25\ \ 5.25\ \ 13]$

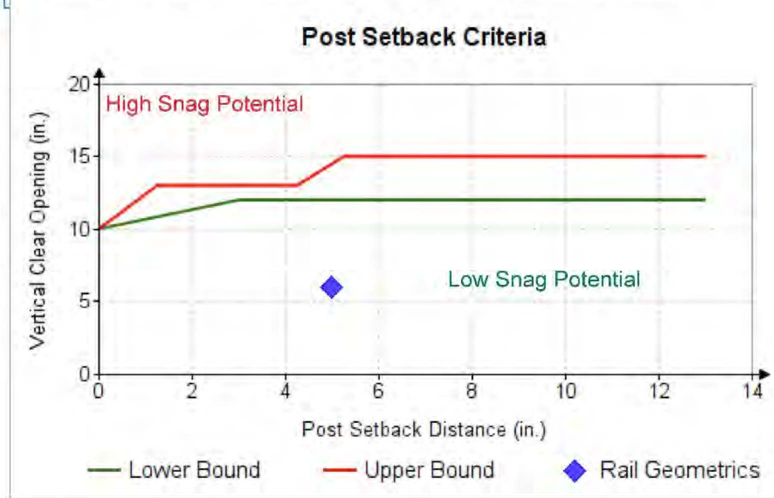
$snag_{up.y} := [10\ \ 13\ \ 13\ \ 15\ \ 15]$

Upper boundary for snag potential criteria x and y coordinates

$snag_{sys.x} := \frac{s_{post}}{in} = 5$ Post setback geometric point

$snag_{sys.y} := \frac{c_b}{in} = 6$ Vertical clear opening rail geometric point

Inputs: $X_1 := snag_{low.x}\ Y_1 := snag_{low.y}\quad X_2 := snag_{up.x}\ Y_2 := snag_{up.y}\quad X_3 := snag_{sys.x}\ Y_3 := snag_{sys.y}$




$HighSnagPotential := 1 \qquad Marginal := 2 \qquad LowSnagPotential := 3$

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

$Snag\_Potential\_Criteria\_Rail\_Geometric\_Point := LowSnagPotential$

**(3) Geometric Criteria - Summary of Results:**

Setback Criteria Check

$$Check\_Post\_Setback\_Criteria := \left\| \begin{array}{l} \text{if } Post\_Setback\_Criteria\_Rail\_Geometric\_Point \ge 2 \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right.$$

$Check\_Post\_Setback\_Criteria = \text{“OK”}$

Snag Potential Check

$$Check\_Snag\_Potential\_Criteria := \left\| \begin{array}{l} \text{if } Snag\_Potential\_Criteria\_Rail\_Geometric\_Point \ge 2 \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right.$$

$Check\_Snag\_Potential\_Criteria = \text{“OK”}$

Minimum Height Check (Stability Criteria)

$$Check\_Barrier\_Minimum\_Height := \left\| \begin{array}{l} \text{if } H_r \ge H_{min} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right.$$

$Check\_Barrier\_Minimum\_Height = \text{“OK”}$

**(4) Calculate the Ultimate transverse Load Resistance of a Single Post located @ Ybar above the deck**

(4.1) Based on the Plastic Strength of the Post: $P_{p1}$

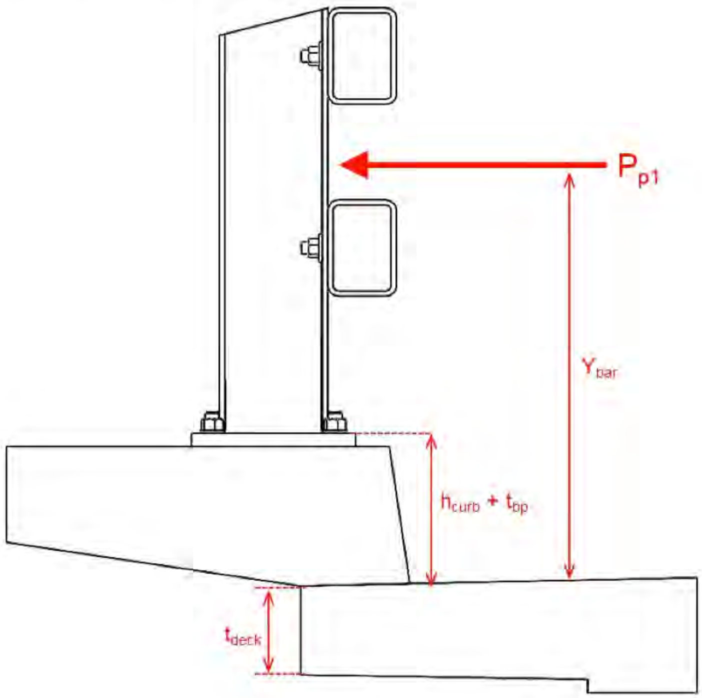



Figure. Ultimate Transverse Load Resistance Based on Plastic Strength of the Post

$$P_{p1} := \frac{Z_{post} \cdot F_{y.post}}{h_p}$$ Strength based on plastic post strength alone (ksi)

$P_{p1} = 74.516\ \boldsymbol{kip}$ Strength based on plastic post strength alone (ksi)

(4.2) Based on the Weld Strength of the Post: $P_{p2}$

Calculate fillet weld strength treated as a line in accordance with Design of Welded Structures, Blodgett, page 7.4-7, Table 5

| | |
|---|---|
| $t_{weld} := \frac{5}{16}\ in$ | Weld size (in.) |
| $b := 6.5\ in$ | Width of the welded joint (in.) |
| $d := 7.93\ in$ | Depth of the welded joint (in.) |
| $t_e := 0.707 \cdot t_{weld}$ | Throat size (in.) |
| $S_w := \left(2 \cdot b \cdot d + \frac{d^2}{3}\right) \cdot t_e = 27.408\ in^3$ | Section modulus (in3). Note: weld treated as a line |
| $F_{EXX} := 70\ ksi$ | Electrodes (ksi) |
| $\phi_{weld} := 1.0$ | Reduction factor |
| $M_{weld} := (0.60 \cdot F_{EXX}) \cdot S_w = 95.927\ kip \cdot ft$ | Moment to fail the welded joint (kip-ft) |
| $P_{p2} := \frac{M_{weld}}{h_p} = 74.266\ kip$ | Strength based on post fillet welds (kip) |
| $P_{p2} = 74.266\ kip$ | Strength based on post fillet welds (kip) |

(4.3) Based on the Ultimate Strength of the Anchor Rods in Shear: $P_{p3}$

$N_{rod}=4$ — Number of Anchor Rods

$A_{bolt}=0.601\ in^2$ — Area of single bolt (in.2)

$F_{V.bolt}:=\left(0.75\cdot A_{bolt}\right)\cdot\left(0.6\ F_{u.rod}\right)\cdot N_{rod}$ — Nominal shear strength of single bolt (kip)

$F_{V.bolt}=129.885\ kip$

$P_{p3}:=F_{V.bolt}=129.885\ kip$ — Force @ Ybar to achieve ultimate dynamic bolt shear strength

(4.4) Based on the Ultimate Strength of the Anchor Bolts in Tension: $P_{p4}$

$F_{T.bolt}:=\left(0.75\cdot F_{u.rod}\right)\cdot\left(0.75\cdot A_{bolt}\right)=40.589\ kip$ — Force to fail tension anchor bolts (kip)

$d_{T.bolt}:=10\ in$ — Moment arm for anchor bolts (in.)

$M_{T.bolt}:=2\cdot F_{T.bolt}\cdot d_{T.bolt}=67.649\ kip\cdot ft$ — Moment to fail tension anchor bolts (kip-ft)

$P_{p4}:=\dfrac{M_{T.bolt}}{Y_{bar}-h_{curb}}$ — Post strength to fail bolts in tension (kip)

$P_{p4}=49.199\ kip$

(4.5) Based on the Vertical Punching Shear in Curb: $P_{p5}$

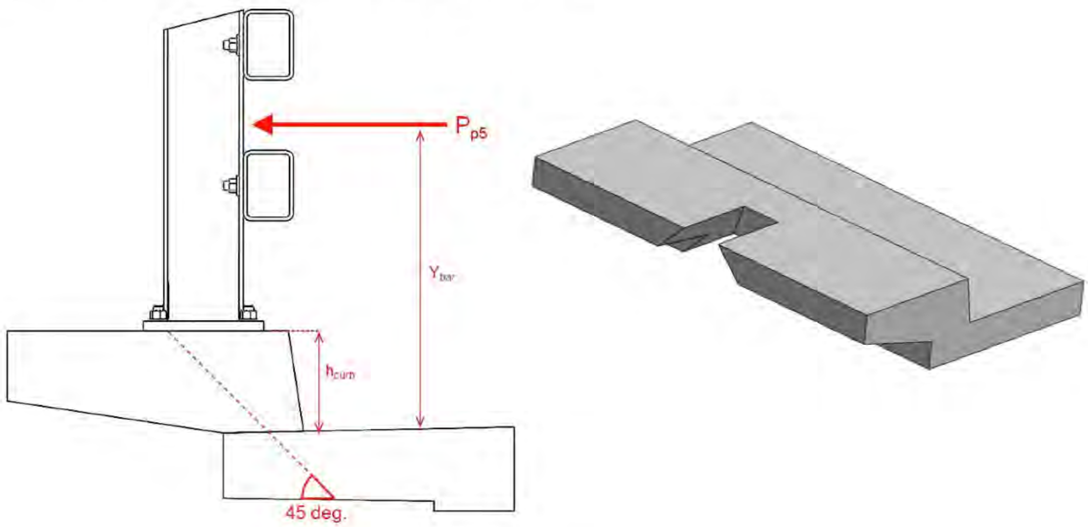



Figure. Punching Shear Failure Mechanism of the Curb

Calculate the area of the failure planes:

Note: Area deduction factor of 0.65 is used to account for concrete spalling along the steel reinforcement in the deck

$Area_{back1} := 783.18\ in^2$ Back area measured in Solidworks (in.2)

$Area_{side1} := 476.97\ in^2$ Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ Reduction factor considering concrete cover

$Area_{total1} := \phi_{punch} \cdot (Area_{back1} + 2 \cdot Area_{side1}) = (1.129 \cdot 10^3)\ in^2$ Total measured area (in.2)

Calculate the tension strength of concrete

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$\phi_c = 0.85$ — Strength reduction factor

$v_c := 2 \cdot \sqrt{\frac{f_c}{psi}}\ psi = 109.545\ psi$ — Stress corresponding to shear strength provided by concrete (psi)

Calculate the punching force to fail the curb

$V_{c.punch1} := \phi_c \cdot Area_{total1} \cdot v_c$ — Nominal shear strength provided by concrete (kip)

$V_{c.punch1} = 105.136\ kip$

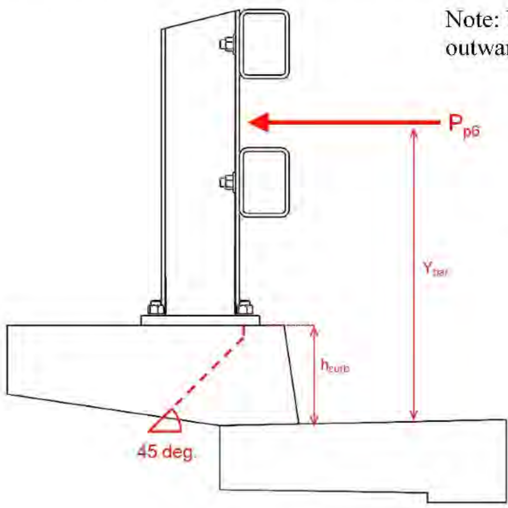
Determine force @ Ybar on post force

$d_{punch1} := 13\ in$ — Estimated moment arm for punching shear (in.)

$P_{p5} := \frac{V_{c.punch1} \cdot d_{punch1}}{h_{bp}}$ — Force to fail concrete due to vertivcal punching shear through 6" deck and curb (kip)

$P_{p5} = 82.835\ kip$ — Vertical punching shear strength (kip)

(4.6) Based on lateral punching resistance of concrete from traffic side anchor bolts: $P_{p6}$




Note: Use 3 punching shear planes extending downward and outward at 45 degees from traffic side anchor bolts

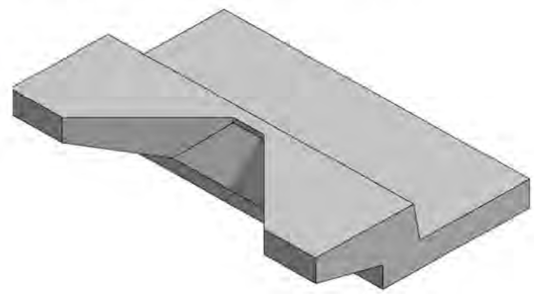


Figure. Lateral Punching Shear Failure Mechanism from Traffic Side Anchor Bolts

Calculate the area of the failure planes:

$Area_{back2} := 280.44\ in^2$ — Back area measured in Solidworks (in.2)

$Area_{side2} := 323.8\ in^2$ — Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ — Reduction factor considering concrete cover

$Area_{total2} := \phi_{punch} \cdot (Area_{back2} + 2 \cdot Area_{side2}) = 603.226\ in^2$ — Total measured area (in.2)

Calculate the punching force to fail the curb

$V_{c.punch2} := v_c \cdot (Area_{total2}) = 66.08\ kip$ — Force to fail the post due to lateral punching shear from the traffic side anchor bolts (kip)

$P_{p6} := V_{c.punch2} = 66.08\ kip$

(4.7) Based on flexural resistance of curb (Failure Section 1): $P_{p7}$

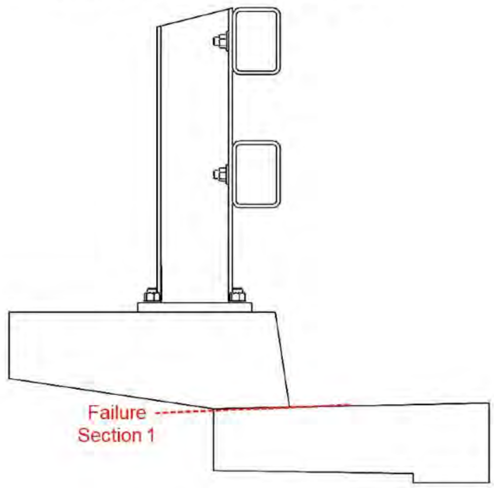



Figure. Failure section 1

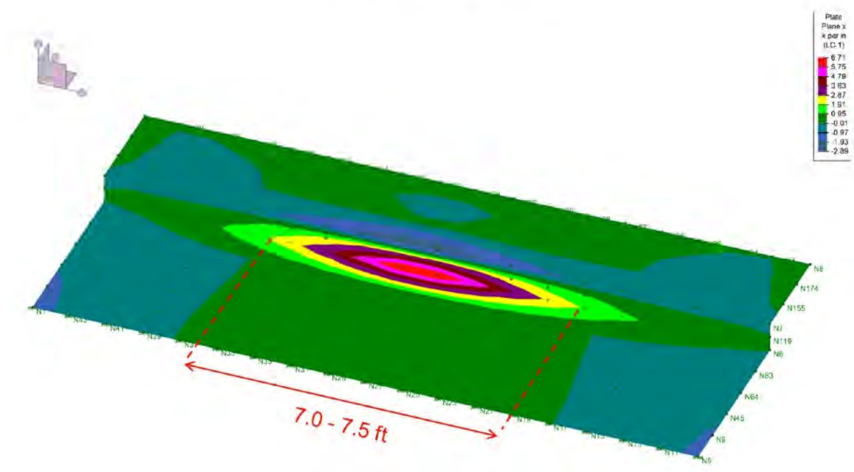



Figure. Estimated force distribution length for Failure Section 1 from structural analysis program using RISA 3D

Given design data

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS1} := 7.5\ ft$ — Longitudinal Length of Transverse Impact Force (ft)

$Y_{FS1} := Y_{bar} = 27.5\ in$ — Height measured from centroid of FS1 to resultant force of rails (in.)

$n_{FS1} := 8$ — Number of rebar within width of FS1

$A_{FS1} := n_{FS1} \cdot 0.31\ in^2$ — Area of tensile reinforcement in FS1 (in2)

$d_{FS1} := 12\ in - 1.5\ in - \frac{0.625}{2}\ in = 10.188\ in$ — Depth to tension steel from compression face of FS1 (in.)

Calculate nominal strength and post strength

$a_{FS1} := \frac{A_{FS1} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS1}} = 0.648\ in$ — Depth of Whitney stress block for FS1 (in.)

$M_{FS1} := A_{FS1} \cdot f_y \cdot \left(d_{FS1} - \frac{a_{FS1}}{2}\right) = 122.305\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p7} := \left(\frac{M_{FS1}}{Y_{FS1}}\right) = 53.37\ kip$ — Post strength based on flexural resistance of curb over the contributing length (kips)

(4.8) Based on flexural resistance of deck (Failure Section 2): $P_{p8}$

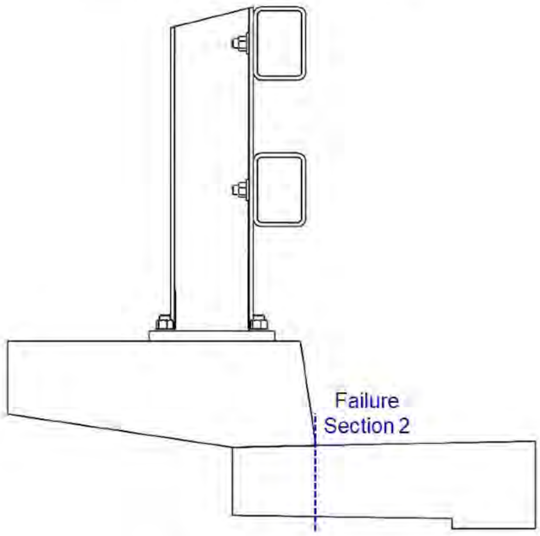



Figure. Failure section 2

The same amplified longitudinal length of distribution as 'Failure Section 1' is applied to 'Failure Section 2' because both failure sections take place where curb and deck intersect.

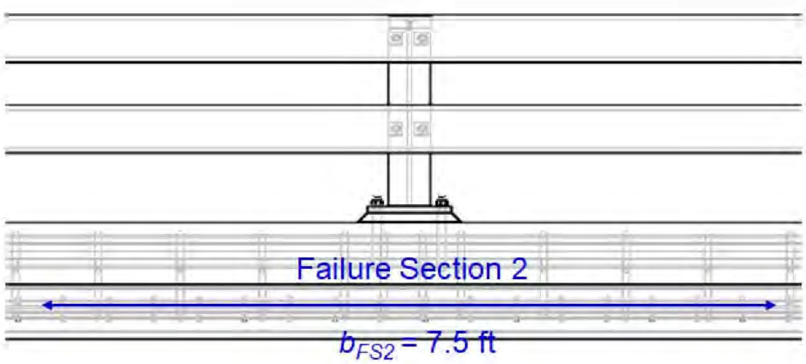



Figure. Reinforcement within Failure Section 2

Given design data

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS2} := 7.5\ ft$ — Width of FS2 (in.) Note: Width of FS2 is assumed to be the impact force projected outward at a 45 degree angle to the centroiad of FS2

$h_{FS2} := \frac{h_{deck}}{2} = 3.344\ in$ — Distance from roadway surface to centroid of FS2 (in.)

$Y_{FS2} := Y_{bar} + h_{FS2} = 30.844\ in$ — Height measured from centroid of FS2 to resultant force of rails (in.)

$n_{FS2} := 16$ — Number of rebar within width of FS2

$A_{FS2} := n_{FS2} \cdot 0.31\ in^2 = 4.96\ in^2$ — Area of tensile reinforcement in FS2 (in2)

Calculate nominal strength and post strength

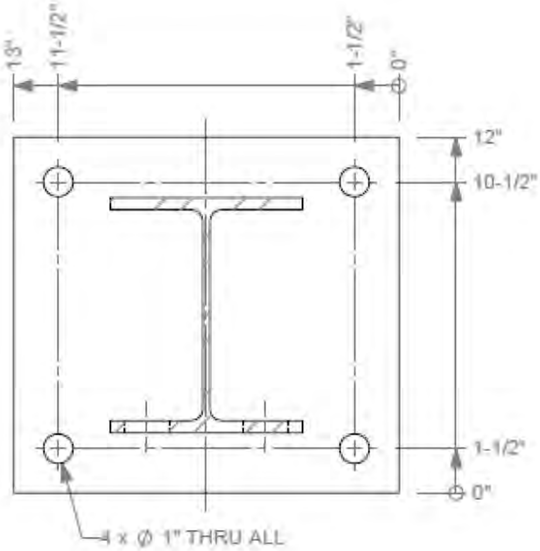
$d_{FS2} := h_{deck} - 1.75\ in - \frac{0.625}{2}\ in = 4.625\ in$ — Depth to tension steel from compression face of FS2 (in.)

$a_{FS2} := \frac{A_{FS2} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS2}} = 1.297\ in$ — Depth of Whitney stress block for FS2 (in.)

$M_{FS2} := A_{FS2} \cdot f_y \cdot \left(d_{FS2} - \frac{a_{FS2}}{2}\right) = 98.621\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p8} := \left(\frac{M_{FS2}}{Y_{FS2}}\right) = 38.369\ kip$ — Post strength based on flexural resistance of deck over the contributing length (kips)

(4.9) Based on ultimate strength of retrofit adhesive dowels in curb: $P_{p9}$




Use Hilti RE500 V3 Epoxy for 7/8" dia Anchors Embedded 8.0 inches in 3 ksi concrete curb for traffic side anchors. Note that anchors on traffic side are considered in the calculation as failure may take place at anchors in tension.

Reference Table 25 - Hilti RE-500 V3 Epoxy Adhesive Design Strength with concrete bond failure for threaded rod in uncracked concrete, 2016 Design Guide, page 151

| Nominal anchor diameter in. | Effective embedment in. (mm) | Tension — ΦN$_n$ $f'_c$ = 2,500 psi (17.2 MPa) lb (kN) | $f'_c$ = 3,000 psi (20.7 MPa) lb (kN) | $f'_c$ = 4,000 psi (27.6 MPa) lb (kN) | $f'_c$ = 6,000 psi (41.4 MPa) lb (kN) |
|---|---|---|---|---|---|
| 7/8[10] | 3-1/2 (89) | 5,105 (22.7) | 5,595 (24.9) | 6,460 (28.7) | 7,910 (35.2) |
| | 7-7/8 (200) | 17,235 (76.7) | 18,885 (84.0) | 21,805 (97.0) | 26,705 (118.8) |
| | 10-1/2 (267) | 26,540 (118.1) | 29,070 (129.3) | 33,570 (149.3) | 41,115 (182.9) |
| | 17-1/2 (445) | 57,100 (254.0) | 62,550 (278.2) | 71,740 (319.1) | 79,395 (353.2) |

$F_{T\_\phi N} := 19370$ ***lbf***

Interpolated between ultimate strengths of effective embedment of 7-7/8" and 10-1/2" for 7/8" diameter anchor

From Table 38 - Load adjustment factors for 7/8" diameter threaded rods in uncracked concrete, Hilti Design Guide, page 159

| 7/8-in. uncracked concrete | Spacing factor in tension $f_{AN}$ | | | | Edge distance factor in tension $f_{RN}$ | | | |
|---|---|---|---|---|---|---|---|---|
| Embedment in. $h_{ef}$ (mm) | 3-1/2 (89) | 7-7/8 (200) | 10-1/2 (267) | 17-1/2 (445) | 3-1/2 (89) | 7-7/8 (200) | 10-1/2 (267) | 17-1/2 (445) |
| **Spacing (s) / Edge Distance ($c_a$) / Concrete Thickness (h), - in. (mm)** | | | | | | | | |
| 1-3/4 (44) | n/a | n/a | n/a | n/a | 0.39 | 0.24 | 0.18 | 0.10 |
| 4-3/8 (111) | 0.58 | 0.58 | 0.57 | 0.54 | 0.53 | 0.31 | 0.23 | 0.13 |
| 5 (127) | 0.59 | 0.59 | 0.58 | 0.55 | 0.56 | 0.33 | 0.24 | 0.13 |
| 5-1/2 (140) | 0.60 | 0.60 | 0.59 | 0.55 | 0.58 | 0.34 | 0.25 | 0.14 |
| 6 (152) | 0.61 | 0.61 | 0.60 | 0.56 | 0.61 | 0.36 | 0.26 | 0.15 |
| 7 (178) | 0.63 | 0.63 | 0.61 | 0.57 | 0.65 | 0.39 | 0.28 | 0.16 |
| 8 (203) | 0.65 | 0.65 | 0.63 | 0.58 | 0.71 | 0.42 | 0.31 | 0.17 |
| 9 (229) | 0.67 | 0.67 | 0.64 | 0.59 | 0.76 | 0.45 | 0.33 | 0.18 |
| 9-7/8 (251) | 0.69 | 0.69 | 0.66 | 0.59 | 0.80 | 0.48 | 0.35 | 0.19 |
| 10 (254) | 0.69 | 0.69 | 0.66 | 0.60 | 0.81 | 0.49 | 0.35 | 0.19 |
| 11 (279) | 0.71 | 0.71 | 0.67 | 0.60 | 0.87 | 0.52 | 0.38 | 0.21 |
| 12 (305) | 0.73 | 0.73 | 0.69 | 0.61 | 0.92 | 0.56 | 0.40 | 0.22 |
| 12-1/2 (318) | 0.74 | 0.74 | 0.70 | 0.62 | 0.95 | 0.59 | 0.41 | 0.23 |
| 14 (356) | 0.76 | 0.76 | 0.72 | 0.63 | 1.00 | 0.66 | 0.46 | 0.25 |
| 16 (406) | 0.80 | 0.80 | 0.75 | 0.65 | | 0.75 | 0.52 | 0.29 |
| 18 (457) | 0.84 | 0.84 | 0.79 | 0.67 | | 0.84 | 0.59 | 0.32 |
| 19-1/2 (495) | 0.87 | 0.87 | 0.81 | 0.69 | | 0.92 | 0.64 | 0.35 |
| 20 (508) | 0.88 | 0.88 | 0.82 | 0.69 | | 0.94 | 0.65 | 0.36 |
| 22 (559) | 0.91 | 0.91 | 0.85 | 0.71 | | 1.00 | 0.72 | 0.40 |
| 24 (610) | 0.95 | 0.95 | 0.88 | 0.73 | | | 0.78 | 0.43 |
| 26 (660) | 0.99 | 0.99 | 0.91 | 0.75 | | | 0.85 | 0.47 |
| 28 (711) | 1.00 | 1.00 | 0.94 | 0.77 | | | 0.91 | 0.50 |
| 30 (762) | | | 0.98 | 0.79 | | | 0.98 | 0.54 |
| 36 (914) | | | 1.00 | 0.84 | | | 1.00 | 0.65 |
| > 48 (1219) | | | | 0.96 | | | | 0.86 |

$f_{AN} := 0.689$

Spacing reduction factor for uncracked concrete; 10 in. spacing is used

$f_{RN} := 0.325$

Edge distance reduction factor for uncracked concrete; 5 in.edge distance at the mid-depth of the curb is used

$f_{temp} := 1.0$

Temperature reduction factor for uncracked concrete; Temperature range A is used

$\phi_{TRE500} := 1.33$

Edge distance and spacing values from Hilti are conservative for reinforced concrete; Use factor of 1.33 for dynamic loading

$F_{TRE500} := \phi_{TRE500} \cdot F_{T\_\phi N} \cdot f_{AN} \cdot f_{RN} \cdot f_{temp} = 5.769\ \boldsymbol{kip}$

Adhesive bond strength of ahcnhor with RE500V3 for embedded depth of 8.0 in. (kip)

$d_{anchor} := 10.5\ \boldsymbol{in}$

Distance between edge of baseplate to centerline of tension bolts (in.)

$$P_{p9} := \frac{2 \cdot F_{TRE500} \cdot d_{anchor}}{h_p} = 7.816\ \boldsymbol{kip}$$

Post strength based on ultimate strength of retrofit adhesive dowels in curb (kip)

**Comments: Too conservative design due to a large factor of safety on Hilti's adhesive anchor**

Adjustment for exceedingly conservative design strength in Hilti specification using test data

Extrapolate ultimate strength for embedded depth of 8.0 in. from TTI testing of anchor with RE500V3 (in.)

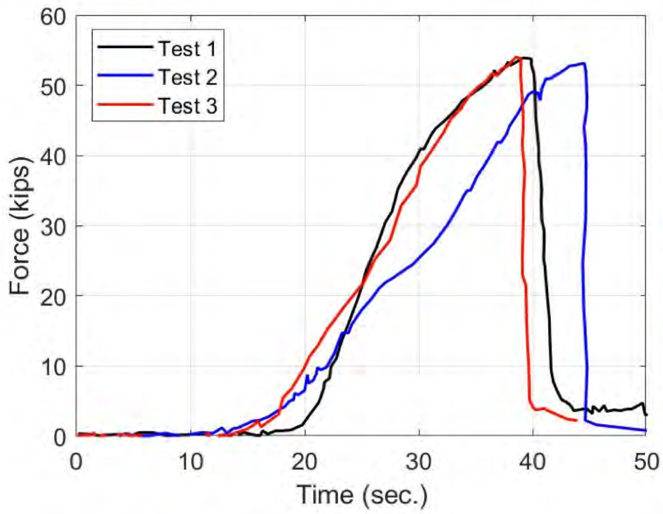



Figure. TTI test results for Hilti RE-500-V3 with 7/8 in. threaded rod and 8 in. embedment depth

Figure. Cone-shaped failure observed in TTI test

$F_{T\_\phi N} := 54\ \textbf{kip}$

Average ultimate strength of anchor with RE500V3 for embedded depth of 8.0 in. from TTI testing (in.)

$F_{TRE500} := \phi_{TRE500} \cdot F_{T\_\phi N} \cdot f_{AN} \cdot f_{RN} \cdot f_{temp} = 16.082\ \textbf{kip}$

Estimated strength to induce the cone-shaped failure (kip)

$h_p = 15.5\ \textbf{in}$

Height measured from top of base plate to resultant force of rails (in.)

$$P_{p9} := \frac{2 \cdot F_{TRE500} \cdot d_{anchor}}{h_p} = 21.789\ \textbf{kip}$$

Post strength based on ultimate strength of retrofit adhesive ahcnhors in curb (kip)

**Comment: The post strength based on the ultimate strength of the adhesive anchors here is likely conservative due to the presence of stirrup and longitudinal reinforcement. This reinforcement very likely reinforces the anchor strength for this strength check.**

**(5) Summarize post strengths and select limiting post strength for LRFD strength analysis:**

| | |
|---|---|
| $P_{p1} = 74.516\ kip$ | Post strength based on plastic strength of the post |
| $P_{p2} = 74.266\ kip$ | Post strength based on post fillet welds |
| $P_{p3} = 129.885\ kip$ | Post strenth based on anchor bolt shear strength |
| $P_{p4} = 49.199\ kip$ | Post strength based on ultimate tension strength of anchor bolts |
| $P_{p5} = 82.835\ kip$ | Post strength based on the curb punching shear strength |
| $P_{p6} = 66.08\ kip$ | Post strength based on lateral punching shear on the traffice side bolts |
| $P_{p7} = 53.37\ kip$ | Post strength based on flexural resistance (FS1) |
| $P_{p8} = 38.369\ kip$ | Post strength based on flexural resistance (FS2) |
| $P_{p9} = 21.789\ kip$ | Post strength based on ultimate strength of adhesive dowels in curb |

$$P_p := min\left(P_{p1}, P_{p2}, P_{p3}, P_{p4}, P_{p5}, P_{p6}, P_{p7}, P_{p8}, P_{p9}\right) = 21.789\ kip$$

$$P_p := P_{p9}$$

Therefore use Pp9 as limiting strength.

**(6) Calculate the rail strength for multiple spans @ He height:**

Design forces and designations for TL-4

$H_e = 30\ in$ — Height of equivalent transverse load (in.)

$P_p = 21.789\ kip$ — Governing post strength (kip)

$Y_{bar} = 27.5\ in$ — Average height of combined rail resistances (in.)

$L_{span} = 8.583\ ft$ — Span length (ft)

$L_t = 5\ ft$ — Length of loading for TL-4 (ft)

$M_p = 105.8\ kip \cdot ft$ — Total resistance of the rails (kip-ft)

** 1 Span Case

$N_1 := 1$ — 1 Span Case

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_1 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_1 = 139.134\ kip$ — Resistance for 1 Span

** 2 Span Case

$N_2 := 2$ — 2 Span Case

$$R_2 := \frac{16 \cdot M_p + {N_2}^2 \cdot P_p \cdot L_{span}}{2 \cdot N_2 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_2 = 83.212\ kip$ — Resistance for 2 Spans

** 3 Span Case

$N_3 := 3$ — 3 Span Case

$$R_3 := \frac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_3 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_3 = 68.58\ kip$ — Resistance for 3 Span

** 4 Span Case

$N_4 := 4$ — 4 Span Case

$$R_4 := \frac{16 \cdot M_p + N_4^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_4 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_4 = 73.589\ kip$ — Resistance for 4 Spans

** 5 Span Case

$N_5 := 5$ — 5 Span Case

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_5 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_5 = 76.47\ kip$ — Resistance for 3 Span

** 6 Span Case

$N_6 := 6$ — 5 Span Case

$$R_6 := \frac{16 \cdot M_p + N_6^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_6 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_6 = 85.975\ kip$ — Resistance for 3 Span

**(7) Total Ultimate Resistance (Nominal Resistance) of Railing:** $R_R$

$R_r := min(R_1, R_2, R_3, R_4, R_5, R_6) = 68.58\ kip$ — Total ultimate resistance of the bridge rail system @ ybar (kip)

$H_{e.TL4} := 30\ in$ — Height of transverse impact load (in.); MASH TL-4

$Y_{bar} = 27.5\ in$ — Height of resultant force (in.)

$F_{t.TL4} := 80\ kip$ — Transverse impact force (in.); MASH TL-4

$R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL4}}\right) = 62.865\ kip$ — Total ultimate resistance of the bridge rail system @He (kip)

$$Check\_Retrofit\_Design := \left\| \begin{array}{l} \text{if } R_R \geq F_{t.TL4} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right\| = \text{“NOT OK”}$$

**The strength of the retrofit design does not meet MASH TL-4 requirement. Check if the design meets TL-3 requirement.**

$H_{e.TL3} := 19\ in$ — Height of transverse impact load (in.)

$F_{t.TL3} := 71\ kip$ — Transverse impact force (in.)

$R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL3}}\right) = 99.261\ kip$ — Total ultimate resistance of the bridge rail system @He (kip)

$$Check\_Retrofit\_Design := \left\| \begin{array}{l} \text{if } R_R \geq F_{t.TL3} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right\| = \text{“OK”}$$

**The strength of the retrofit design meets MASH TL-3 requirement.**

## Summary:

The strength of the retrofit design is approximately 63 kips at the TL-4 height of 30 in. and this is less than the target strength of 80 kips.

Adhesive bond failure is considered the governing failure mechanism, and its design strength is significantly reduced due to conservative strength reduction factors.

However, the design does meet MASH TL-3. TTI recommends performing static load testing of posts or full-scale crash testing for MASH TL-4.

## B-2. Strength Analysis of North Dakota curb 21-106.109 retrofit design

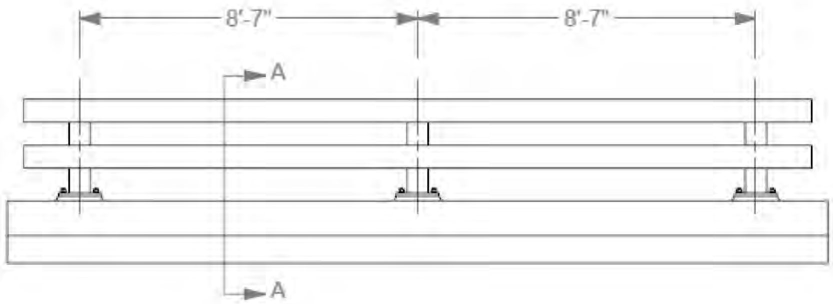
Texas A&M Transportation Institute

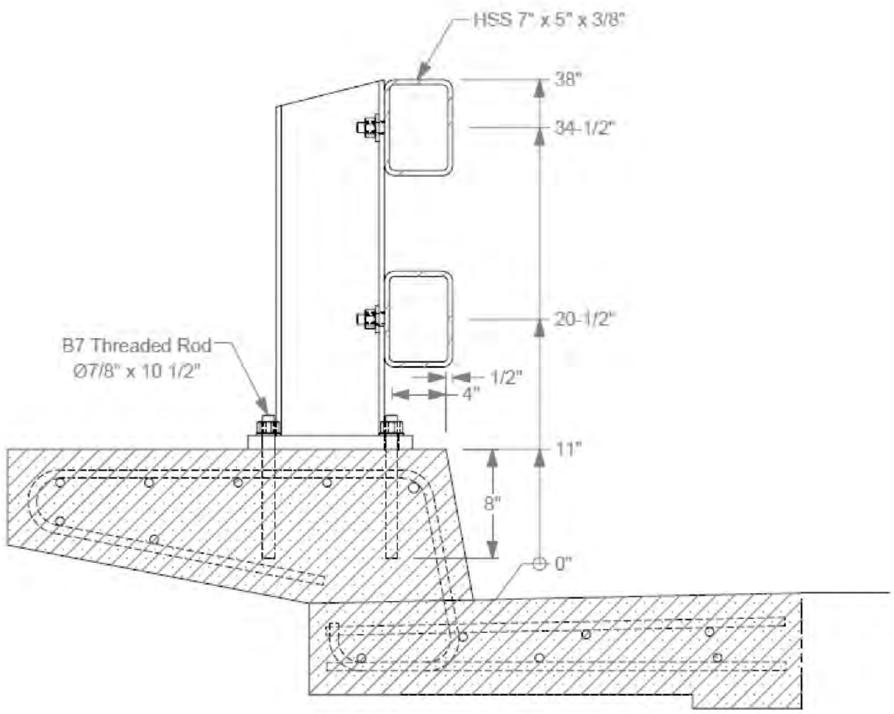
**Subject** : Retrofit Design of the MASH TL-4 Alaska 2-Tube Bridge Rail '94-134-115'

**Sponsor** : Alaska Department of Transportation & Public Facilities (AKDOT & PF)

Find strength of the Retrofit Rail system as per AASHTO LRFD Section 13, A13.3.2 Specifications

**Test Installation (Curb '94-134-115')**




Section A-A

1 of 29

## Curb and Deck Details (94-134-115)

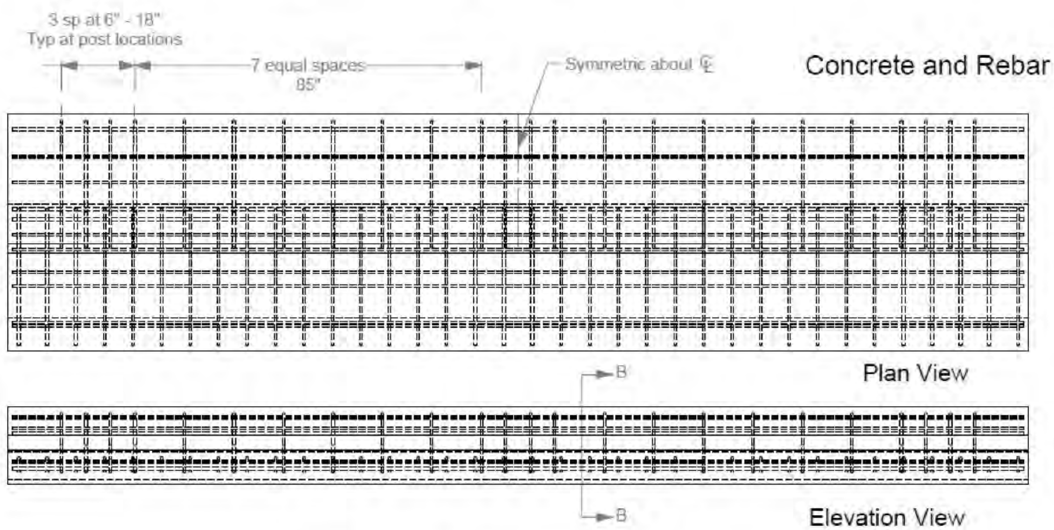



**Figure. Plan view and elevation view of curb '94-134-115'**

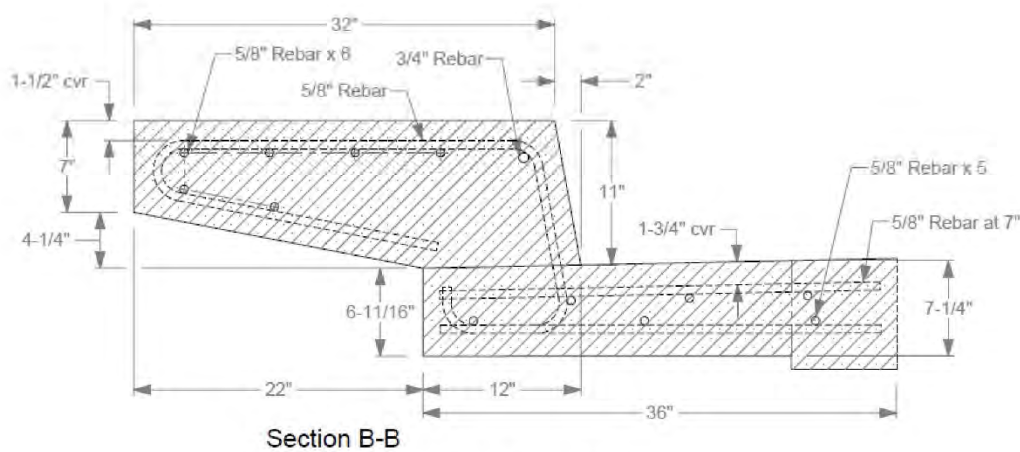
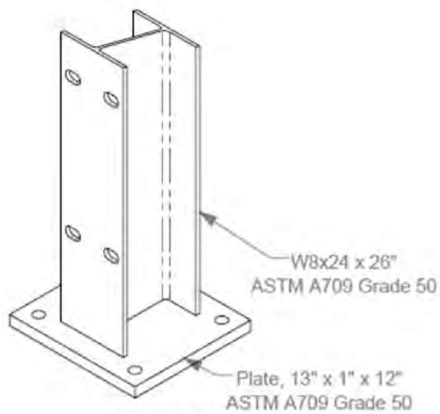
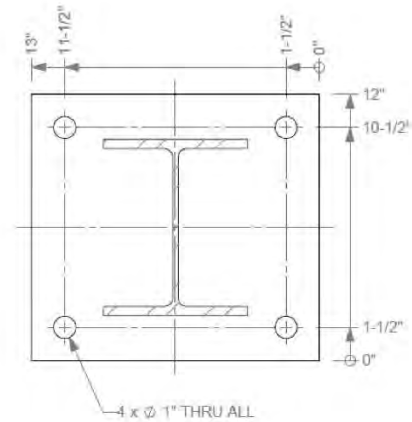



**Figure. Section B-B**

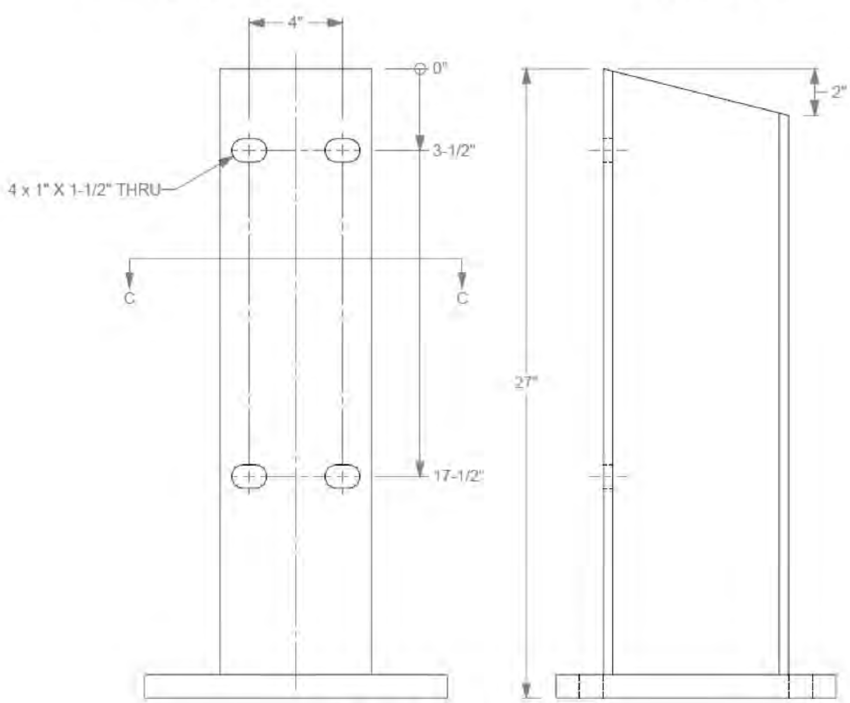
## Post Details (94-134-115)




**Isometric View**




**Section C-C**




**Elevation View**

### (1) Design Force Input:

**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 A.13.2-1
- TL-3 design forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4(a), TL-4(b), and TL-5(a), and TL-5(b) design forces are from research conducted under NCHRP Project 22-20(2)

$TL := 4$ Test level

$F_t := 80\ kip$ Transverse impact force (kips)

$L_t := 5\ ft$ Longitudinal length of distribution of impact force (ft)

$H_e := 30\ in$ Height of equivalent transverse load from top of overlay (in.)

$H_{min} := 36\ in$ Minimum height of a MASH TL-4 barrier (in.)

$L_{span} := 103\ in = 8.583\ ft$ Span length (ft)

## (2) Input Properties for Rail Components & Geometry

(2a) Input for Bridge Rail Properties

Steel rail properties and dimensions:
a) Steel rail is A500 Gr. B Material, fy=46 ksi
b) Steel rail bend about the y-axis
3) Steel rails are HSS7x5x3/8 members

Top Rail (HSS 7 x 5 x 3/8) Properties

$Z_{top.rail} := 13.8\ in^3$ — Plastic section modulus of top rail about weak axis (in^3)

$Y_{top} := 34.5\ in$ — Height to the center of the top rail (in.)

$F_{y.top} := 46\ ksi$ — Yield of middle rail material (ksi) for ASTM A500 Grade B

Bottom Rail (HSS 7 x 5 x 3/8) Properties

$Z_{bottom.rail} := 13.8\ in^3$ — Plastic section modulus of bottom rail about weak axis (in^3)

$Y_{bottom} := 20.5\ in$ — Height to the center of the bottom rail (in.)

$F_{y.bottom} := 46\ ksi$ — Yield of bottom rail material (ksi) for ASTM A500 Grade B

Calculate the Total Rail Resistance and the Average Height of all Rail Elements

$M_{p.top} := Z_{top.rail} \cdot F_{y.top} = 52.9\ kip \cdot ft$ — Resistance of the top rail (kip-ft)

$M_{p.bottom} := Z_{bottom.rail} \cdot F_{y.bottom} = 52.9\ kip \cdot ft$ — Resistance of the bottom rails(kip-ft)

$M_p := \left(M_{p.top} + M_{p.bottom}\right) = 105.8\ kip \cdot ft$ — Total resistance of the rails(kip-ft)

$$Y_{bar} := \frac{M_{p.top} \cdot Y_{top} + M_{p.bottom} \cdot Y_{bottom}}{M_p}$$

Average height of combined rail resistances from top of deck (AASHTO A13.3.3-2)

$Y_{bar} = 27.5\ in$ — Height from pavement surface to centroid of rail elemenets (in.)

(2b) Input for Post (W8x24) Properties

Steel post and base plate properties and dimensions:
a) Steel posts are A709 Gr. 50 Material, fy=50 ksi
b) Base plates are Gr. 36 Material, fy = 36 ksi
c) Base plates are 12 in. x 13 in. with 1.0 in. thickness

| | |
|---|---|
| $t_{bp} := 1.0\ \boldsymbol{in}$ | Thickness of post base plate (in.) |
| $t_w := 0.245\ \boldsymbol{in}$ | Web thickness (in.) |
| $t_f := 0.4\ \boldsymbol{in}$ | Flange thickness (in.) |
| $Z_{post} := 23.1\ \boldsymbol{in}^3$ | Plastic modulus of post about strong axis (in3) |
| $F_{y.post} := 50\ \boldsymbol{ksi}$ | Yield of post material (ksi) for ASTM A572 Grade 50 |
| $h_{curb} := 11\ \boldsymbol{in}$ | Height of curb (in.) |
| $h_{deck} := 6.6875\ \boldsymbol{in}$ | Thickness of deck (in.)<br>Note: smallest is taken for conservative design |
| $h_p := Y_{bar} - t_{bp} - h_{curb} = 15.5\ \boldsymbol{in}$ | Height measured from top of base plate to resultant force of rails (in.) |
| $h_{bp} := h_p + t_{bp} = 16.5\ \boldsymbol{in}$ | Height measured from bottom of base plate to resultant force of rails (in.) |

(2c) Input for Anchor Rods: Standard Hilti HAS-E Rod Material:

a) Anchor rods are Hilti HAS-E Rods, fu = 120 ksi
b) Anchor rods are 7/8" diameter x 10.5" embedded 8.0"

$Dia_{bolts} := 0.875\ \textbf{in}$ — Diameter of a single anchor rods (in.)

$F_{u.rod} := 120\ \textbf{ksi}$ — Tensile strength of the anchor rods (ksi)

$A_{bolt} := \dfrac{\pi \cdot (Dia_{bolts})^2}{4} = 0.601\ \textbf{in}^2$ — Area of a single anchor rod (in.)

$N_{rod} := 4$ — Number of anchor rods

$N_{rod.tension} := 2$ — Number of anchor rods in tension

$Anchor_{proj} := 2.5\ \textbf{in}$ — Projected anchor rod length (in.); anchors on traffic side are considered

$Anchor_{embed} := 8.0\ \textbf{in}$ — Embeded anchor rod length (in.); anchors on traffic side are considered

$Anchor_{length} := Anchor_{proj} + Anchor_{embed} = 10.5\ \textbf{in}$ — Total anchor rod length (in.)

(2d) Properties of concrete

$f_c := 3000\ \textbf{psi}$ — Compressive strength of concrete (psi)

$f_y := 60000\ \textbf{psi}$ — Yield strength of rebar (psi) (psi)

$\phi_c := 0.85$ — Concrete strength reduction factor

**(3) Geometric Criteria:**

$s_{post} := 5\ in$ — Post setback (in.)

$c_b := 6\ in$ — Vertical clear opening (in.)

$H_w := 11\ in$ — Height of the concrete curb measured from the top of the roadway surface (in.)

$N_R := 2$ — Number of steel rail

$h_R := 7\ in$ — Height of a single steel rail (in.)

$\Sigma A := H_w + N_R \cdot h_R = 25\ in$ — Total rail contact width (in.)

$H_r := 38\ in$ — Total height of the bridge rail measured from the top of the roadway surface/overlay (in.)

**Note**: Denoted as "H" in figure below

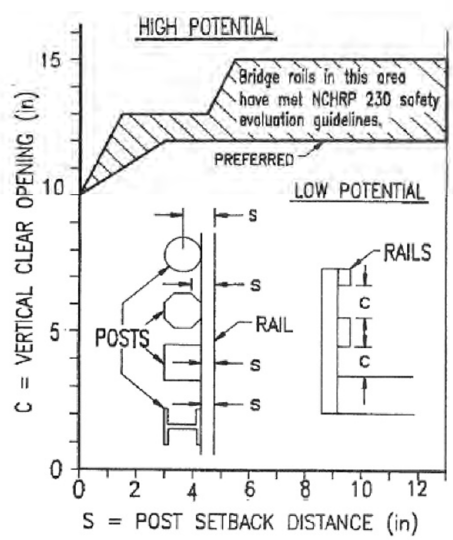
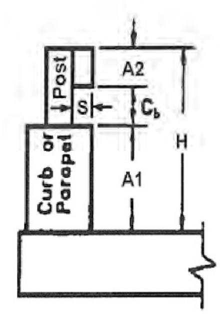


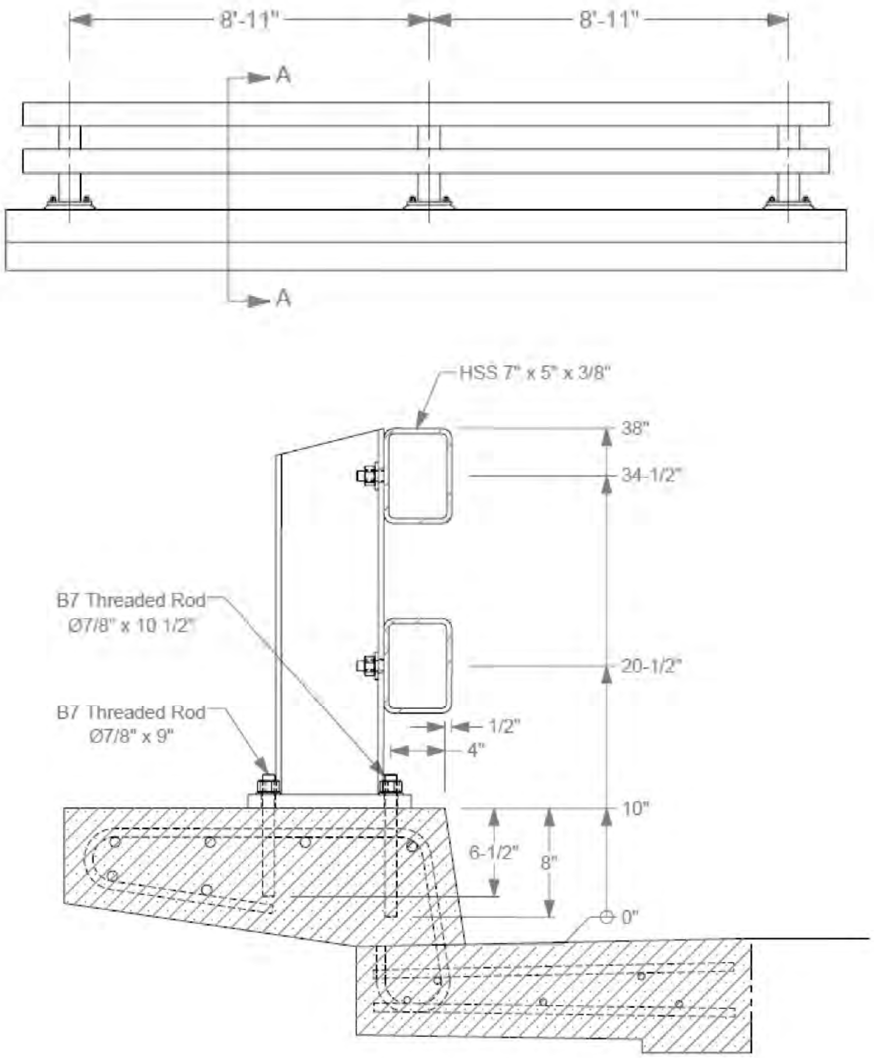

Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post




CONCRETE AND METAL RAIL

Find strength of the Retrofit Rail system as per AASHTO LRFD Section 13, A13.3.2 Specifications

## Test Installation (Curb '21-106.109')




Section A-A

## Curb and Deck Details (Curb 21-106.109)

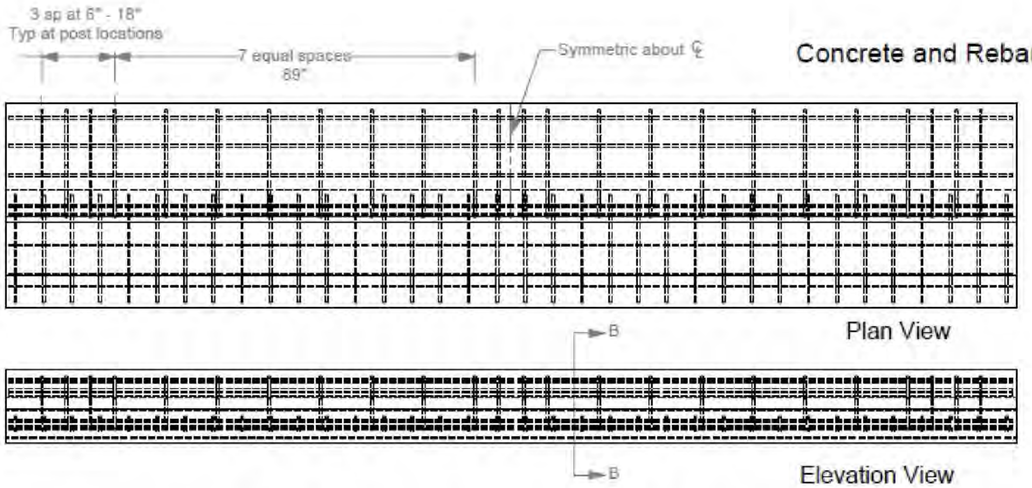



**Figure. Plan view and elevation view of curb '21-106.109'**

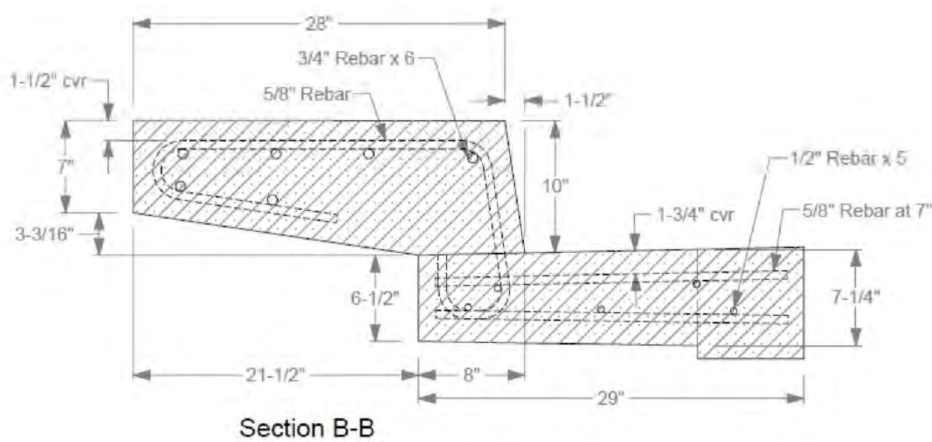
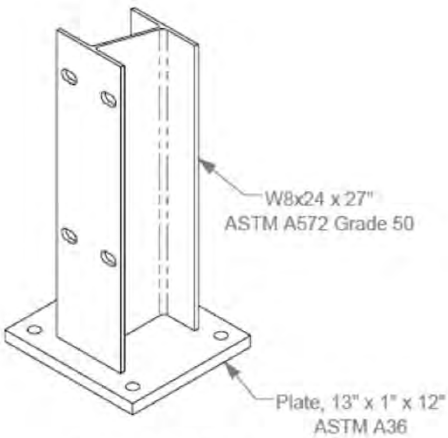
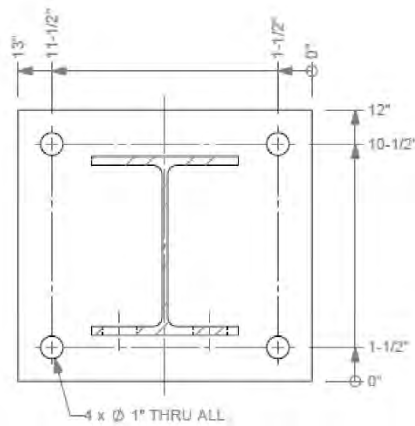



**Figure. Section B-B**

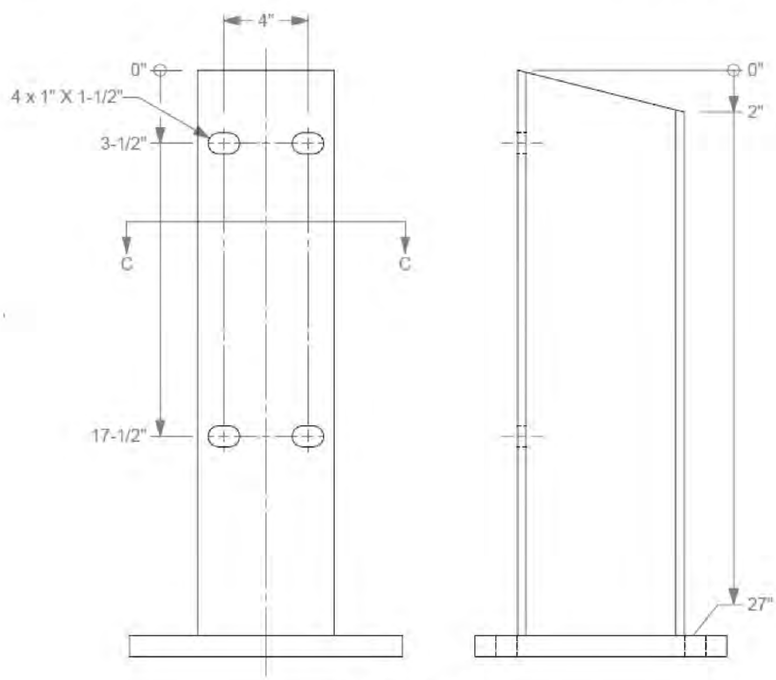
## Post Details (Curb 21-106.109)




Isometric View




Section C-C




Elevation Views

**(1) Design Force Input:**

**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 A.13.2-1
- TL-3 design forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4(a), TL-4(b), and TL-5(a), and TL-5(b) design forces are from research conducted under NCHRP Project 22-20(2)

| | |
|---|---|
| $TL := 4$ | Test level |
| $F_t := 80\ \mathbf{kip}$ | Transverse impact force (kips) |
| $L_t := 5\ \mathbf{ft}$ | Longitudinal length of distribution of impact force (ft) |
| $H_e := 30\ \mathbf{in}$ | Height of equivalent transverse load from top of overlay (in.) |
| $H_{min} := 36\ \mathbf{in}$ | Minimum height of a MASH TL-4 barrier (in.) |
| $L_{span} := 107\ \mathbf{in} = 8.917\ \mathbf{ft}$ | Span length (ft) |

**(2) Input Properties for Rail Components & Geometry**

(2a) Input for Bridge Rail Properties

Steel rail properties and dimensions:
a) Steel rail is A500 Gr. B Material, fy=46 ksi
b) Steel rail bend about the y-axis
3) Steel rails are HSS7x5x3/8 members

Top Rail (HSS 7 x 5 x 3/8) Properties

| | |
|---|---|
| $Z_{top.rail} := 13.8\ in^3$ | Plastic section modulus of top rail about weak axis (in^3) |
| $Y_{top} := 34.5\ in$ | Height to the center of the top rail (in.) |
| $F_{y.top} := 46\ ksi$ | Yield of middle rail material (ksi) for ASTM A500 Grade B |

Bottom Rail (HSS 7 x 5 x 3/8) Properties

| | |
|---|---|
| $Z_{bottom.rail} := 13.8\ in^3$ | Plastic section modulus of bottom rail about weak axis (in^3) |
| $Y_{bottom} := 20.5\ in$ | Height to the center of the bottom rail (in.) |
| $F_{y.bottom} := 46\ ksi$ | Yield of bottom rail material (ksi) for ASTM A500 Grade B |

Calculate the Total Rail Resistance and the Average Height of all Rail Elements

| | |
|---|---|
| $M_{p.top} := Z_{top.rail} \cdot F_{y.top} = 52.9\ kip \cdot ft$ | Resistance of the top rail (kip-ft) |
| $M_{p.bottom} := Z_{bottom.rail} \cdot F_{y.bottom} = 52.9\ kip \cdot ft$ | Resistance of the bottom rails(kip-ft) |
| $M_p := (M_{p.top} + M_{p.bottom}) = 105.8\ kip \cdot ft$ | Total resistance of the rails(kip-ft) |
| $Y_{bar} := \dfrac{M_{p.top} \cdot Y_{top} + M_{p.bottom} \cdot Y_{bottom}}{M_p}$ | Average height of combined rail resistances from top of deck (AASHTO A13.3.3-2) |
| $Y_{bar} = 27.5\ in$ | Height from pavement surface to centroid of rail elemenets (in.) |

(2b) Input for Post (W8x24) Properties

Steel post and base plate properties and dimensions:
a) Steel posts are A709 Gr. 50 Material, fy=50 ksi
b) Base plates are Gr. 36 Material, fy = 36 ksi
c) Base plates are 12 in. x 13 in. with 1.0 in. thickness

$t_{bp} := 1.0\ in$ — Thickness of post base plate (in.)

$t_w := 0.245\ in$ — Web thickness (in.)

$t_f := 0.4\ in$ — Flange thickness (in.)

$Z_{post} := 23.1\ in^3$ — Plastic modulus of post about strong axis (in3)

$F_{y.post} := 50\ ksi$ — Yield of post material (ksi) for ASTM A572 Grade 50

$h_{curb} := 10\ in$ — Height of curb (in.)

$h_{deck} := 6.5\ in$ — Thickness of deck (in.)
Note: smallest is taken for conservative design

$h_p := Y_{bar} - t_{bp} - h_{curb} = 16.5\ in$ — Height measured from top of base plate to resultant force of rails (in.)

$h_{bp} := h_p + t_{bp} = 17.5\ in$ — Height measured from bottom of base plate to resultant force of rails (in.)

(2c) Input for Anchor Rods: Standard Hilti HAS-E Rod Material:

a) Anchor rods are Hilti HAS-E Rods, fu = 120 ksi
b) Anchor rods are 7/8" diameter x 10.5" embedded 8.0" (consider the one in the traffic side)

| | |
|---|---|
| $Dia_{bolts} := 0.875\ in$ | Diameter of a single anchor rods (in.) |
| $F_{u.rod} := 120\ ksi$ | Tensile strength of the anchor rods (ksi) |
| $A_{bolt} := \dfrac{\pi \cdot \left(Dia_{bolts}\right)^2}{4} = 0.601\ in^2$ | Area of a single anchor rod (in.) |
| $N_{rod} := 4$ | Number of anchor rods |
| $N_{rod.tension} := 2$ | Number of anchor rods in tension |
| $Anchor_{proj} := 2.5\ in$ | Projected anchor rod length (in.); anchors on traffic side are considered |
| $Anchor_{embed} := 8.0\ in$ | Embeded anchor rod length (in.); anchors on traffic side are considered |
| $Anchor_{length} := Anchor_{proj} + Anchor_{embed} = 10.5\ in$ | Total anchor rod length (in.) |

(2d) Properties of concrete

| | |
|---|---|
| $f_c := 3000\ psi$ | Compressive strength of concrete (psi) |
| $f_y := 60000\ psi$ | Yield strength of rebar (psi) (psi) |
| $\phi_c := 0.85$ | Concrete strength reduction factor |

## (3) Geometric Criteria:

| | |
|---|---|
| $s_{post} := 5\ in$ | Post setback (in.) |
| $c_b := 7\ in$ | Vertical clear opening (in.) |
| $H_w := 10\ in$ | Height of the concrete curb measured from the top of the roadway surface (in.) |
| $N_R := 2$ | Number of steel rail |
| $h_R := 7\ in$ | Height of a single steel rail (in.) |
| $\Sigma A := H_w + N_R \cdot h_R = 24\ in$ | Total rail contact width (in.) |
| $H_r := 38\ in$ | Total height of the bridge rail measured from the top of the roadway surface/overlay (in.)<br>**Note**: Denoted as "H" in figure below |

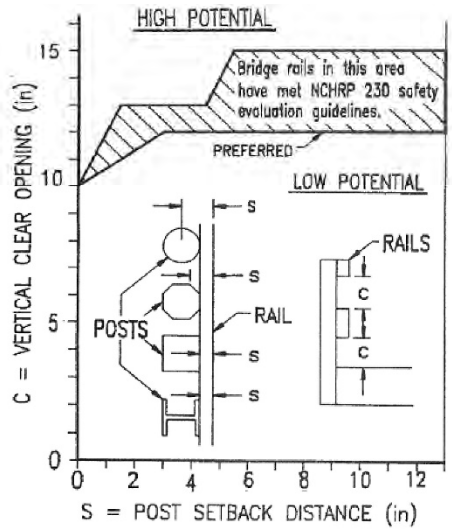
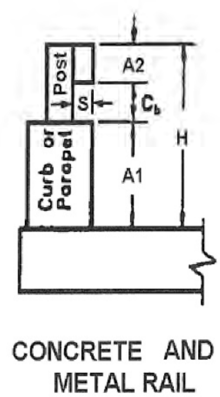



**Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post**



**(3-conti.) Geometric Criteria:**

$s_{post} = 5\ in$ $\Sigma A = 24\ in$ $H_r = 38\ in$ $ratio_{\Sigma AH} := \dfrac{\Sigma A}{H_r} = 0.632$

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

Lower boundary for post setback criteria x and y coordinates

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

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

Upper boundary for post setback criteria x and y coordinates

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

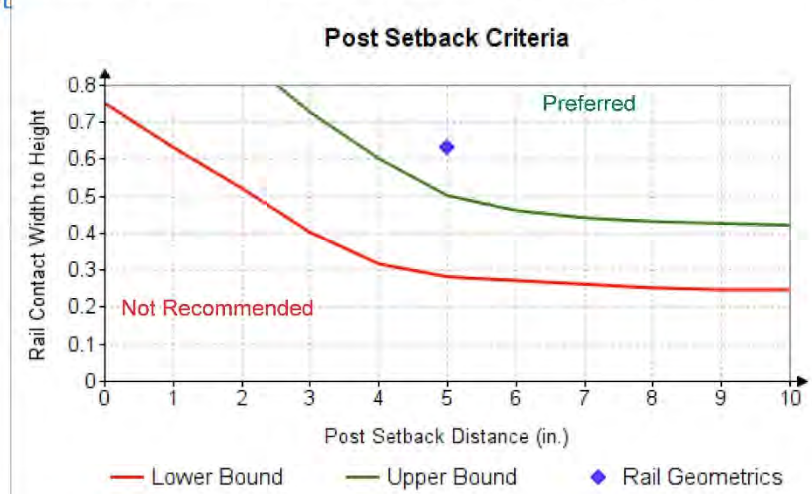
$Set_{sys.x} := \dfrac{s_{post}}{in} = 5$

Post setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.632$

Ratio of contact width to total height rail geometric point

Inputs: $X_1 := Set_{low.x}$ $Y_1 := Set_{low.y}$ $X_2 := Set_{up.x}$ $Y_2 := Set_{up.y}$ $X_3 := Set_{sys.x}$ $Y_3 := Set_{sys.y}$




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

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

$snag_{up.x} := [0 \;\; 1.25 \;\; 4.25 \;\; 5.25 \;\; 13]$

$snag_{up.y} := [10 \;\; 13 \;\; 13 \;\; 15 \;\; 15]$

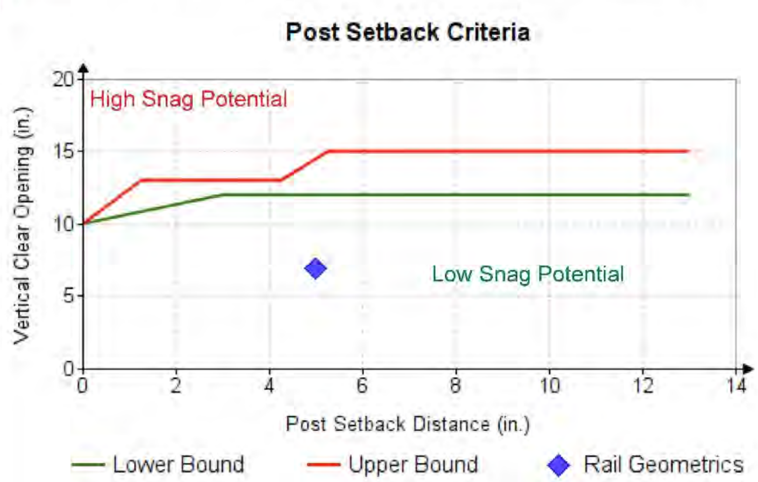
Upper boundary for snag potential criteria x and y coordinates

$snag_{sys.x} := \dfrac{s_{post}}{in} = 5$ Post setback geometric point

$snag_{sys.y} := \dfrac{c_b}{in} = 7$ Vertical clear opening rail geometric point

Inputs

$X_1 := snag_{low.x} \; Y_1 := snag_{low.y} \quad X_2 := snag_{up.x} \; Y_2 := snag_{up.y} \quad X_3 := snag_{sys.x} \; Y_3 := snag_{sys.y}$




$HighSnagPotential := 1 \qquad Marginal := 2 \qquad LowSnagPotential := 3$

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

$Snag\_Potential\_Criteria\_Rail\_Geometric\_Point := LowSnagPotential$

**(3) Geometric Criteria - Summary of Results:**

Setback Criteria Check

$$Check\_Post\_Setback\_Criteria := \begin{Vmatrix} \text{if } Post\_Setback\_Criteria\_Rail\_Geometric\_Point \geq 2 \\ \quad \Vert \text{"OK"} \\ \text{else} \\ \quad \Vert \text{"NOT OK"} \end{Vmatrix}$$

$Check\_Post\_Setback\_Criteria =$ "OK"

Snag Potential Check

$$Check\_Snag\_Potential\_Criteria := \begin{Vmatrix} \text{if } Snag\_Potential\_Criteria\_Rail\_Geometric\_Point \geq 2 \\ \quad \Vert \text{"OK"} \\ \text{else} \\ \quad \Vert \text{"NOT OK"} \end{Vmatrix}$$

$Check\_Snag\_Potential\_Criteria =$ "OK"

Minimum Height Check (Stability Criteria)

$$Check\_Barrier\_Minimum\_Height := \begin{Vmatrix} \text{if } H_r \geq H_{min} \\ \quad \Vert \text{"OK"} \\ \text{else} \\ \quad \Vert \text{"NOT OK"} \end{Vmatrix}$$

$Check\_Barrier\_Minimum\_Height =$ "OK"

**(4) Calculate the Ultimate transverse Load Resistance of a Single Post located @ Ybar above the deck**

(4.1) Based on the Plastic Strength of the Post: $P_{p1}$

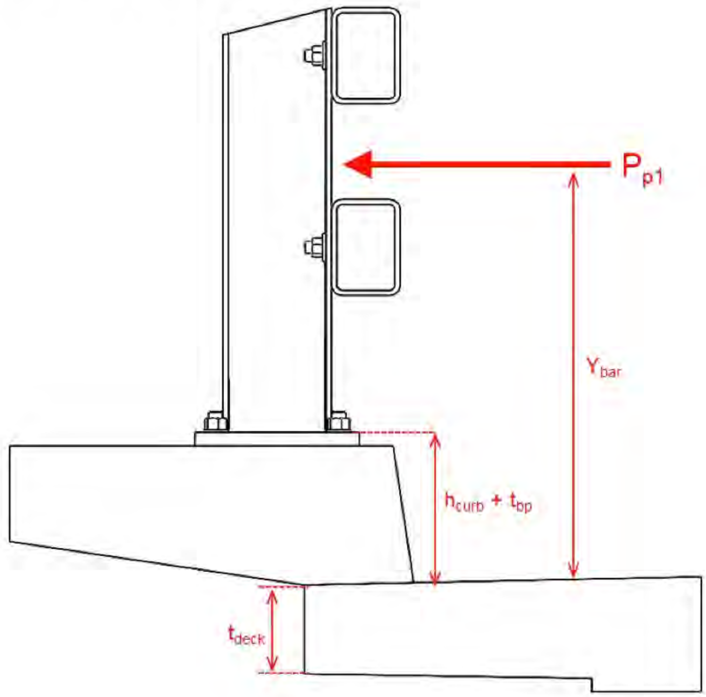



Figure. Ultimate Transverse Load Resistance Based on Plastic Strength of the Post

$$P_{p1} := \frac{Z_{post} \cdot F_{y.post}}{h_p}$$ Strength based on plastic post strength alone (ksi)

$P_{p1} = 70\ \boldsymbol{kip}$ Strength based on plastic post strength alone (ksi)

(4.2) Based on the Weld Strength of the Post: $P_{p2}$

Calculate fillet weld strength treated as a line in accordance with Design of Welded Structures, Blodgett, page 7.4-7, Table 5

| | |
|---|---|
| $t_{weld} := \frac{5}{16}\ in$ | Weld size (in.) |
| $b := 6.5\ in$ | Width of the welded joint (in.) |
| $d := 7.93\ in$ | Depth of the welded joint (in.) |
| $t_e := 0.707 \cdot t_{weld}$ | Throat size (in.) |
| $S_w := \left(2 \cdot b \cdot d + \frac{d^2}{3}\right) \cdot t_e = 27.408\ in^3$ | Section modulus (in3). Note: weld treated as a line |
| $F_{EXX} := 70\ ksi$ | Electrodes (ksi) |
| $\phi_{weld} := 1.0$ | Reduction factor |
| $M_{weld} := (0.60 \cdot F_{EXX}) \cdot S_w = 95.927\ kip \cdot ft$ | Moment to fail the welded joint (kip-ft) |
| $P_{p2} := \frac{M_{weld}}{h_p} = 69.765\ kip$ | Strength based on post fillet welds (kip) |
| $P_{p2} = 69.765\ kip$ | Strength based on post fillet welds (kip) |

(4.3) Based on the Ultimate Strength of the Anchor Rods in Shear: $P_{p3}$

$N_{rod}=4$ — Number of Anchor Rods

$A_{bolt}=0.601\ in^2$ — Area of single bolt (in.2)

$F_{V.bolt}:=(0.75 \cdot A_{bolt}) \cdot (0.6\ F_{u.rod}) \cdot N_{rod}$ — Nominal shear strength of single bolt (kip)

$F_{V.bolt}=129.885\ kip$

$P_{p3}:=F_{V.bolt}=129.885\ kip$ — Force @ Ybar to achieve ultimate dynamic bolt shear strength

(4.4) Based on the Ultimate Strength of the Anchor Bolts in Tension: $P_{p4}$

$F_{T.bolt}:=(0.75 \cdot F_{u.rod}) \cdot (0.75 \cdot A_{bolt})=40.589\ kip$ — Force to fail tension anchor bolts (kip)

$d_{T.bolt}:=10\ in$ — Moment arm for anchor bolts (in.)

$M_{T.bolt}:=2 \cdot F_{T.bolt} \cdot d_{T.bolt}=67.649\ kip \cdot ft$ — Moment to fail tension anchor bolts (kip-ft)

$P_{p4}:=\dfrac{M_{T.bolt}}{Y_{bar}-h_{curb}}$ — Post strength to fail bolts in tension (kip)

$P_{p4}=46.388\ kip$

(4.5) Based on the Vertical Punching Shear in Curb: $P_{p5}$

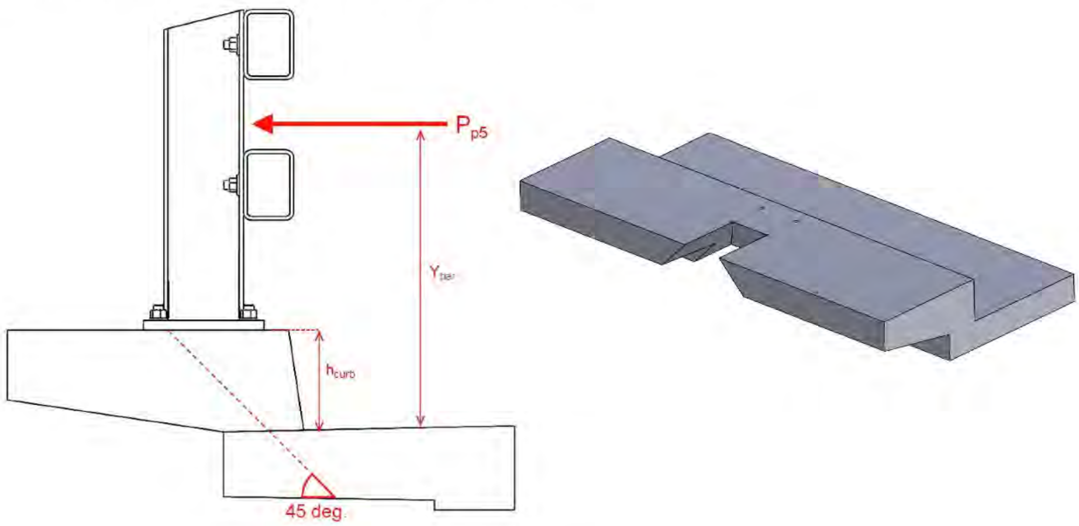



Figure. Punching Shear Failure Mechanism of the Curb

Calculate the area of the failure planes:

Note: Area deduction factor of 0.65 is used to account for concrete spalling along the steel reinforcement in the deck

$Area_{back1} := 617.7\ in^2$ Back area measured in Solidworks (in.2)

$Area_{side1} := 318.59\ in^2$ Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ Reduction factor considering concrete cover

$Area_{total1} := \phi_{punch} \cdot (Area_{back1} + 2 \cdot Area_{side1}) = 815.672\ in^2$ Total measured area (in.2)

Calculate the tension strength of concrete

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$\phi_c = 0.85$ — Strength reduction factor

$v_c := 2 \cdot \sqrt{\frac{f_c}{psi}}\ psi = 109.545\ psi$ — Stress corresponding to shear strength provided by concrete (psi)

Calculate the punching force to fail the curb

$V_{c.punch1} := \phi_c \cdot Area_{total1} \cdot v_c$ — Nominal shear strength provided by concrete (kip)

$V_{c.punch1} = 75.95\ kip$

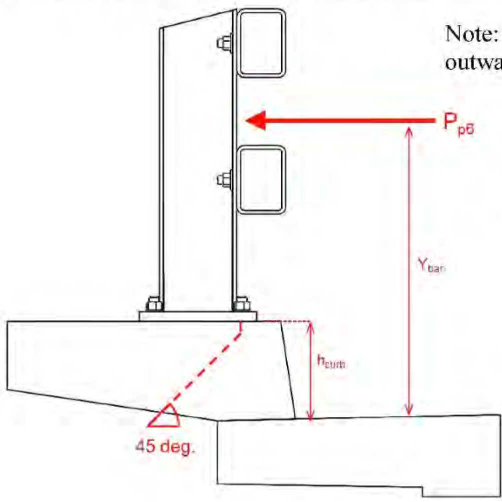
Determine force @ Ybar on post force

$d_{punch1} := 13\ in$ — Estimated moment arm for punching shear (in.)

$P_{p5} := \frac{V_{c.punch1} \cdot d_{punch1}}{h_{bp}}$ — Force to fail concrete due to vertivcal punching shear through 6" deck and curb (kip)

$P_{p5} = 56.42\ kip$ — Vertical punching shear strength (kip)

(4.6) Based on lateral punching resistance of concrete from traffic side anchor bolts: $P_{p6}$




Note: Use 3 punching shear planes extending downward and outward at 45 degees from traffic side anchor bolts

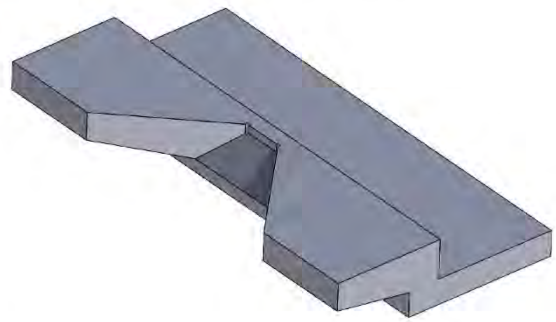


Figure. Lateral Punching Shear Failure Mechanism from Traffic Side Anchor Bolts

Calculate the area of the failure planes:

$Area_{back2} := 207.2\ in^2$ Back area measured in Solidworks (in.2)

$Area_{side2} := 254.06\ in^2$ Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ Reduction factor considering concrete cover

$Area_{total2} := \phi_{punch} \cdot (Area_{back2} + 2 \cdot Area_{side2}) = 464.958\ in^2$ Total measured area (in.2)

Calculate the punching force to fail the curb

$V_{c.punch2} := v_c \cdot (Area_{total2}) = 50.934\ kip$ Force to fail the post due to lateral punching shear from the traffic side anchor bolts (kip)

$P_{p6} := V_{c.punch2} = 50.934\ kip$

(4.7) Based on flexural resistance of curb (Failure Section 1): $P_{p7}$

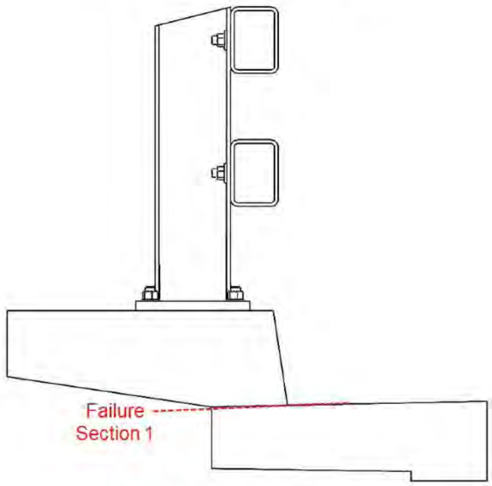



Figure. Failure section 1

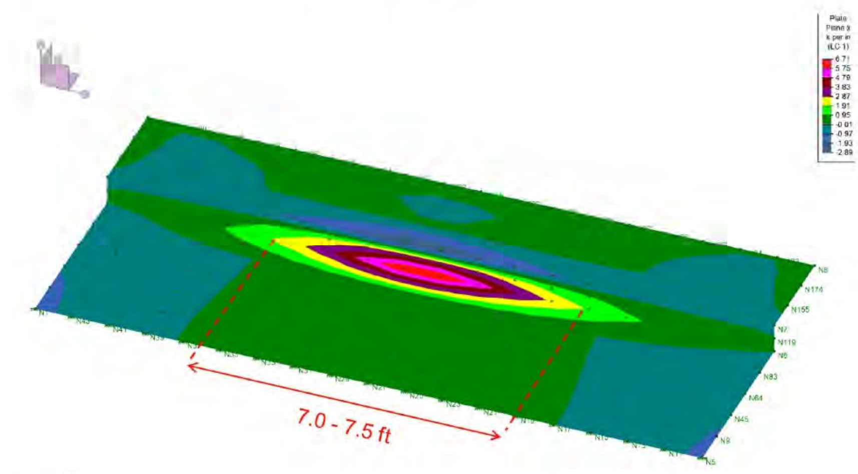



Figure. Estimated force distribution length for Failure Section 1 from structural analysis program using RISA 3D

Given design data

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS1} := 7.5\ ft$ — Amplified Longitudinal Length of Transverse Impact Force (ft)

$Y_{FS1} := Y_{bar} = 27.5\ in$ — Height measured from centroid of FS1 to resultant force of rails (in.)

$n_{FS1} := 8$ — Number of rebar within width of FS1

$A_{FS1} := n_{FS1} \cdot 0.31\ in^2$ — Area of tensile reinforcement in FS1 (in2)

$d_{FS1} := 8\ in - 1.5\ in - \frac{0.625}{2}\ in = 6.188\ in$ — Depth to tension steel from compression face of FS1 (in.)

Calculate nominal strength and post strength

$a_{FS1} := \frac{A_{FS1} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS1}} = 0.648\ in$ — Depth of Whitney stress block for FS1 (in.)

$M_{FS1} := A_{FS1} \cdot f_y \cdot \left(d_{FS1} - \frac{a_{FS1}}{2}\right) = 72.705\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p7} := \left(\frac{M_{FS1}}{Y_{FS1}}\right) = 31.726\ kip$ — Post strength based on flexural resistance of curb over the contributing length (kips)

(4.8) Based on flexural resistance of deck (Failure Section 2): $P_{p8}$

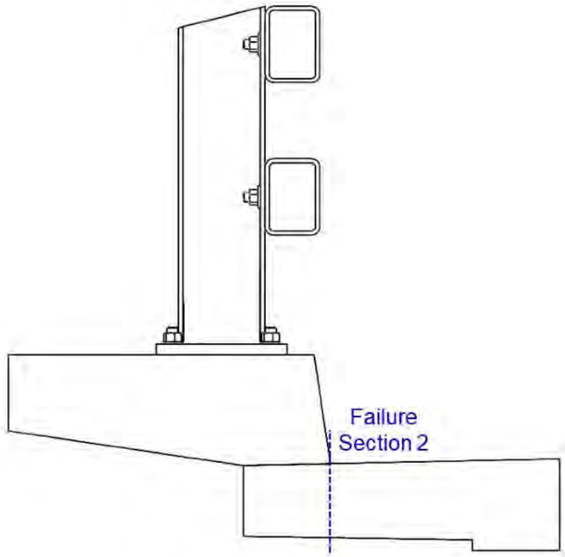



Figure. Failure section 2

The same amplified longitudinal length of distribution as 'Failure Section 1' is applied to 'Failure Section 2' because both failure sections take place where curb and deck intersect.

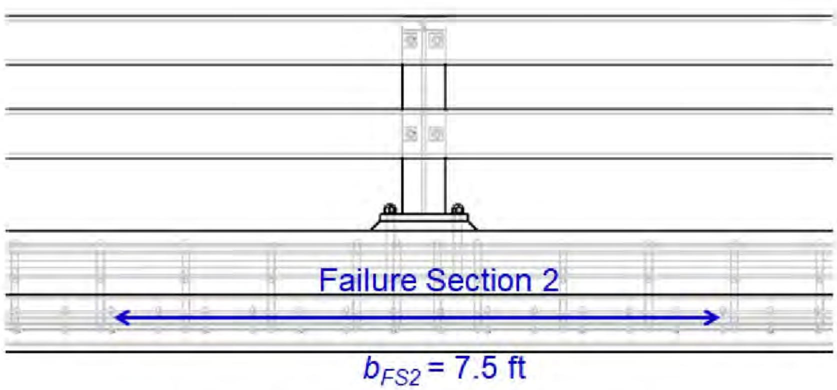



Figure. Reinforcement within Failure Section 2

Given design data

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS2} := 7.5\ ft$ — Width of FS2 (in.) Note: Width of FS2 is assumed to be the impact force projected outward at a 45 degree angle to the centroiad of FS2

$h_{FS2} := \frac{h_{deck}}{2} = 3.25\ in$ — Distance from roadway surface to centroid of FS2 (in.)

$Y_{FS2} := Y_{bar} + h_{FS2} = 30.75\ in$ — Height measured from centroid of FS2 to resultant force of rails (in.)

$n_{FS2} := 12$ — Number of rebar within width of FS2

$A_{FS2} := n_{FS2} \cdot 0.31\ in^2 = 3.72\ in^2$ — Area of tensile reinforcement in FS2 (in2)

Calculate nominal strength and post strength

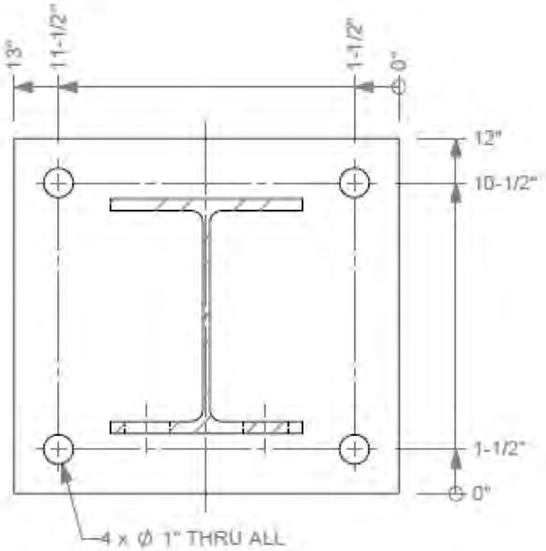
$d_{FS2} := h_{deck} - 1.75\ in - \frac{0.625}{2}\ in = 4.438\ in$ — Depth to tension steel from compression face of FS2 (in.)

$a_{FS2} := \frac{A_{FS2} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS2}} = 0.973\ in$ — Depth of Whitney stress block for FS2 (in.)

$M_{FS2} := A_{FS2} \cdot f_y \cdot \left(d_{FS2} - \frac{a_{FS2}}{2}\right) = 73.493\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p8} := \left(\frac{M_{FS2}}{Y_{FS2}}\right) = 28.68\ kip$ — Post strength based on flexural resistance of deck over the contributing length (kips)

(4.9) Based on ultimate strength of retrofit adhesive dowels in curb: $P_{p9}$




Use Hilti RE500 V3 Epoxy for 7/8" dia Anchors Embedded 8.0 inches in 3 ksi concrete curb for traffic side anchors. Note that anchors on traffic side are considered in the calculation as failure may take place at anchors in tension.

Reference Table 25 - Hilti RE-500 V3 Epoxy Adhesive Design Strength with concrete bond failure for threaded rod in uncracked concrete, 2016 Design Guide, page 151

| Nominal anchor diameter in. | Effective embedment in. (mm) | Tension — $\Phi N_n$ $f'_c$ = 2,500 psi (17.2 MPa) lb (kN) | $f'_c$ = 3,000 psi (20.7 MPa) lb (kN) | $f'_c$ = 4,000 psi (27.6 MPa) lb (kN) | $f'_c$ = 6,000 psi (41.4 MPa) lb (kN) |
|---|---|---|---|---|---|
| 7/8[10] | 3-1/2 (89) | 5,105 (22.7) | 5,595 (24.9) | 6,460 (28.7) | 7,910 (35.2) |
| | 7-7/8 (200) | 17,235 (76.7) | 18,885 (84.0) | 21,805 (97.0) | 26,705 (118.8) |
| | 10-1/2 (267) | 26,540 (118.1) | 29,070 (129.3) | 33,570 (149.3) | 41,115 (182.9) |
| | 17-1/2 (445) | 57,100 (254.0) | 62,550 (278.2) | 71,740 (319.1) | 79,395 (353.2) |

$F_{T\_\phi N} := 19370\ lbf$

Interpolated between ultimate strengths of effective embedment of 7-7/8" and 10-1/2" for 7/8" diameter anchor

From Table 38 - Load adjustment factors for 7/8" diameter threaded rods in uncracked concrete, Hilti Design Guide, page 159

| 7/8-in. uncracked concrete | | Spacing factor in tension $f_{AN}$ | | | | Edge distance factor in tension $f_{RN}$ | | | |
|---|---|---|---|---|---|---|---|---|---|
| Embedment $h_{ef}$ in. | (mm) | 3-1/2 (89) | 7-7/8 (200) | 10-1/2 (267) | 17-1/2 (445) | 3-1/2 (89) | 7-7/8 (200) | 10-1/2 (267) | 17-1/2 (445) |
| Spacing (s) / Edge Distance ($c_a$) / Concrete Thickness (h), - in. (mm) 1-3/4 | (44) | n/a | n/a | n/a | n/a | 0.39 | 0.24 | 0.18 | 0.10 |
| 4-3/8 | (111) | 0.58 | 0.58 | 0.57 | 0.54 | 0.53 | 0.31 | 0.23 | 0.13 |
| 5 | (127) | 0.59 | 0.59 | 0.58 | 0.55 | 0.56 | 0.33 | 0.24 | 0.13 |
| 5-1/2 | (140) | 0.60 | 0.60 | 0.59 | 0.55 | 0.58 | 0.34 | 0.25 | 0.14 |
| 6 | (152) | 0.61 | 0.61 | 0.60 | 0.56 | 0.61 | 0.36 | 0.26 | 0.15 |
| 7 | (178) | 0.63 | 0.63 | 0.61 | 0.57 | 0.65 | 0.39 | 0.28 | 0.16 |
| 8 | (203) | 0.65 | 0.65 | 0.63 | 0.58 | 0.71 | 0.42 | 0.31 | 0.17 |
| 9 | (229) | 0.67 | 0.67 | 0.64 | 0.59 | 0.76 | 0.45 | 0.33 | 0.18 |
| 9-7/8 | (251) | 0.69 | 0.69 | 0.66 | 0.59 | 0.80 | 0.48 | 0.35 | 0.19 |
| 10 | (254) | 0.69 | 0.69 | 0.66 | 0.60 | 0.81 | 0.49 | 0.35 | 0.19 |
| 11 | (279) | 0.71 | 0.71 | 0.67 | 0.60 | 0.87 | 0.52 | 0.38 | 0.21 |
| 12 | (305) | 0.73 | 0.73 | 0.69 | 0.61 | 0.92 | 0.56 | 0.40 | 0.22 |
| 12-1/2 | (318) | 0.74 | 0.74 | 0.70 | 0.62 | 0.95 | 0.59 | 0.41 | 0.23 |
| 14 | (356) | 0.76 | 0.76 | 0.72 | 0.63 | 1.00 | 0.66 | 0.46 | 0.25 |
| 16 | (406) | 0.80 | 0.80 | 0.75 | 0.65 | | 0.75 | 0.52 | 0.29 |
| 18 | (457) | 0.84 | 0.84 | 0.79 | 0.67 | | 0.84 | 0.59 | 0.32 |
| 19-1/2 | (495) | 0.87 | 0.87 | 0.81 | 0.69 | | 0.92 | 0.64 | 0.35 |
| 20 | (508) | 0.88 | 0.88 | 0.82 | 0.69 | | 0.94 | 0.65 | 0.36 |
| 22 | (559) | 0.91 | 0.91 | 0.85 | 0.71 | | 1.00 | 0.72 | 0.40 |
| 24 | (610) | 0.95 | 0.95 | 0.88 | 0.73 | | | 0.78 | 0.43 |
| 26 | (660) | 0.99 | 0.99 | 0.91 | 0.75 | | | 0.85 | 0.47 |
| 28 | (711) | 1.00 | 1.00 | 0.94 | 0.77 | | | 0.91 | 0.50 |
| 30 | (762) | | | 0.98 | 0.79 | | | 0.98 | 0.54 |
| 36 | (914) | | | 1.00 | 0.84 | | | 1.00 | 0.65 |
| > 48 | (1219) | | | | 0.96 | | | | 0.86 |

$f_{AN} := 0.689$

Spacing reduction factor for uncracked concrete; 10 in. spacing is used

$f_{RN} := 0.325$

Edge distance reduction factor for uncracked concrete; 5 in.edge distance at the mid-depth of the curb is used

$f_{temp} := 1.0$

Temperature reduction factor for uncracked concrete; Temperature range A is used

$\phi_{TRE500} := 1.33$

Edge distance and spacing values from Hilti are conservative for reinforced concrete; Use factor of 1.33 for dynamic loading

$F_{TRE500} := \phi_{TRE500} \cdot F_{T\_\phi N} \cdot f_{AN} \cdot f_{RN} \cdot f_{temp} = 5.769\ \textbf{kip}$

Adhesive bond strength of ahcnhor with RE500V3 for embedded depth of 8.0 in. (kip)

$d_{anchor} := 10.5\ \textbf{in}$

Distance between edge of baseplate to centerline of tension bolts (in.)

$P_{p9} := \dfrac{2 \cdot F_{TRE500} \cdot d_{anchor}}{h_p} = 7.342\ \textbf{kip}$

Post strength based on ultimate strength of retrofit adhesive dowels in curb (kip)

**Comments: Too conservative design due to a large factor of safety on Hilti's adhesive anchor**

Adjustment for exceedingly conservative design strength in Hilti specification using test data

Extrapolate ultimate strength for embedded depth of 8.0 in. from TTI testing of anchor with RE500V3 (in.)

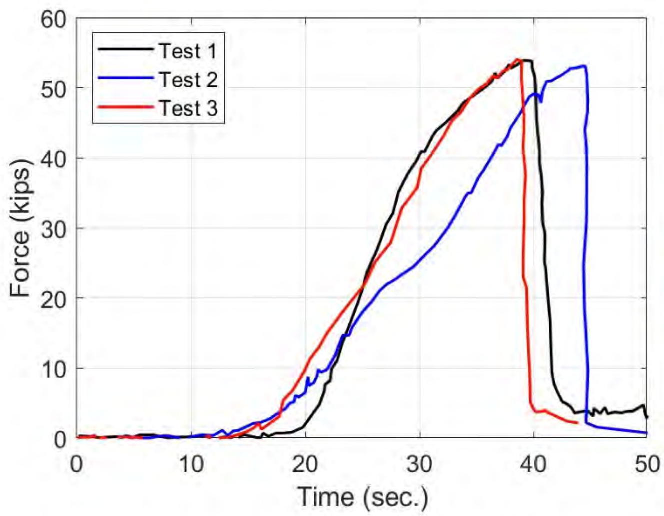



Figure. TTI test results for Hilti RE-500-V3 with 7/8 in. threaded rod and 8 in. embedment depth

Figure. Cone-shaped failure observed in TTI test

$F_{T\_\phi N} := 54\ \textbf{kip}$

Average ultimate strength of anchor with RE500V3 for embedded depth of 8.0 in. from TTI testing (in.)

$F_{TRE500} := \phi_{TRE500} \cdot F_{T\_\phi N} \cdot f_{AN} \cdot f_{RN} \cdot f_{temp} = 16.082\ \textbf{kip}$

Estimated strength to induce the cone-shaped failure (kip)

$h_p = 16.5\ \textbf{in}$

Height measured from top of base plate to resultant force of rails (in.)

$$P_{p9} := \frac{2 \cdot F_{TRE500} \cdot d_{anchor}}{h_p} = 20.468\ \textbf{kip}$$

Post strength based on ultimate strength of retrofit adhesive ahcnhors in curb (kip)

**Comment: The post strength based on the ultimate strength of the adhesive anchors here is likely conservative due to the presence of stirrup and longitudinal reinforcement. This reinforcement very likely reinforces the anchor strength for this strength check.**

**(5) Summarize post strengths and select limiting post strength for LRFD strength analysis:**

| | |
|---|---|
| $P_{p1}=70\ kip$ | Post strength based on plastic strength of the post |
| $P_{p2}=69.765\ kip$ | Post strength based on post fillet welds |
| $P_{p3}=129.885\ kip$ | Post strenth based on anchor bolt shear strength |
| $P_{p4}=46.388\ kip$ | Post strength based on ultimate tension strength of anchor bolts |
| $P_{p5}=56.42\ kip$ | Post strength based on the curb punching shear strength |
| $P_{p6}=50.934\ kip$ | Post strength based on lateral punching shear on the traffice side bolts |
| $P_{p7}=31.726\ kip$ | Post strength based on flexural resistance (FS1) |
| $P_{p8}=28.68\ kip$ | Post strength based on flexural resistance (FS2) |
| $P_{p9}=20.468\ kip$ | Post strength based on ultimate strength of adhesive dowels in curb |

$$P_p := min\left(P_{p1}, P_{p2}, P_{p3}, P_{p4}, P_{p5}, P_{p6}, P_{p7}, P_{p8}, P_{p9}\right) = 20.468\ kip$$

$$P_p := P_{p9}$$

Therefore use Pp9 as limiting strength.

**(6) Calculate the rail strength for multiple spans @ He height:**

Design forces and designations for TL-4

$H_e = 30\ \boldsymbol{in}$ — Height of equivalent transverse load (in.)

$P_p = 20.468\ \boldsymbol{kip}$ — Governing post strength (kip)

$Y_{bar} = 27.5\ \boldsymbol{in}$ — Average height of combined rail resistances (in.)

$L_{span} = 8.917\ \boldsymbol{ft}$ — Span length (ft)

$L_t = 5\ \boldsymbol{ft}$ — Length of loading for TL-4 (ft)

$M_p = 105.8\ \boldsymbol{kip \cdot ft}$ — Total resistance of the rails (kip-ft)

** 1 Span Case

$N_1 := 1$ — 1 Span Case

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_1 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_1 = 131.906\ \boldsymbol{kip}$ — Resistance for 1 Span

** 2 Span Case

$N_2 := 2$ — 2 Span Case

$$R_2 := \frac{16 \cdot M_p + {N_2}^2 \cdot P_p \cdot L_{span}}{2 \cdot N_2 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_2 = 79.006\ \boldsymbol{kip}$ — Resistance for 2 Spans

** 3 Span Case

$N_3 := 3$ — 3 Span Case

$$R_3 := \frac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_3 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_3 = 65.008$ ***kip*** — Resistance for 3 Span

** 4 Span Case

$N_4 := 4$ — 4 Span Case

$$R_4 := \frac{16 \cdot M_p + N_4^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_4 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_4 = 69.542$ ***kip*** — Resistance for 4 Spans

** 5 Span Case

$N_5 := 5$ — 5 Span Case

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_5 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_5 = 72.155$ ***kip*** — Resistance for 3 Span

** 6 Span Case

$N_6 := 6$ — 5 Span Case

$$R_6 := \frac{16 \cdot M_p + N_6^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_6 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_6 = 81.011$ ***kip*** — Resistance for 3 Span

**(7) Total Ultimate Resistance (Nominal Resistance) of Railing:** $R_R$

$R_r := min(R_1, R_2, R_3, R_4, R_5, R_6) = 65.008 \; kip$ — Total ultimate resistance of the bridge rail system @ ybar (kip)

$H_{e.TL4} := 30 \; in$ — Height of transverse impact load (in.); MASH TL-4

$Y_{bar} = 27.5 \; in$ — Height of resultant force (in.)

$F_{t.TL4} := 80 \; kip$ — Transverse impact force (in.); MASH TL-4

$R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL4}}\right) = 59.59 \; kip$ — Total ultimate resistance of the bridge rail system @He (kip)

$$Check\_Retrofit\_Design := \begin{array}{l} \text{if } R_R \geq F_{t.TL4} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} = \text{"NOT OK"}$$

**The strength of the retrofit design does not meet MASH TL-4 requirement. Check if the design meets TL-3 requirement.**

$H_{e.TL3} := 19 \; in$ — Height of transverse impact load (in.)

$F_{t.TL3} := 71 \; kip$ — Transverse impact force (in.)

$R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL3}}\right) = 94.09 \; kip$ — Total ultimate resistance of the bridge rail system @He (kip)

$$Check\_Retrofit\_Design := \begin{array}{l} \text{if } R_R \geq F_{t.TL3} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} = \text{"OK"}$$

**The strength of the retrofit design meets MASH TL-3 requirement.**

## Summary:

The strength of the retrofit design is approximately 60 kips at the TL-4 height of 30 in. and this is less than the target strength of 80 kips.

Adhesive bond failure is considered the governing failure mechanism, and its design strength is significantly reduced due to conservative strength reduction factors.

However, the design does meet MASH TL-3. TTI recommends performing static load testing of posts or full-scale crash testing for MASH TL-4.

## B-3. Strength Analysis of North Dakota curb 2-149.663 retrofit design

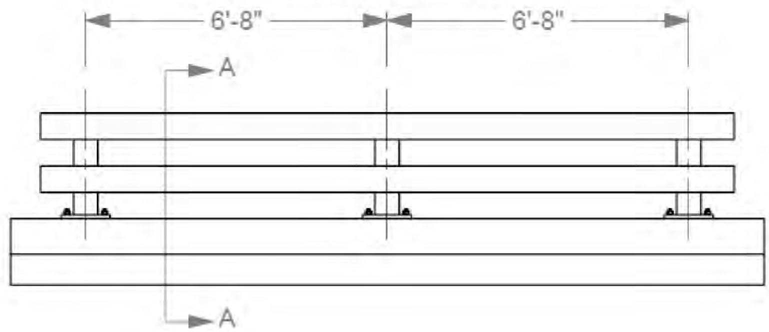
Texas A&M Transportation Institute

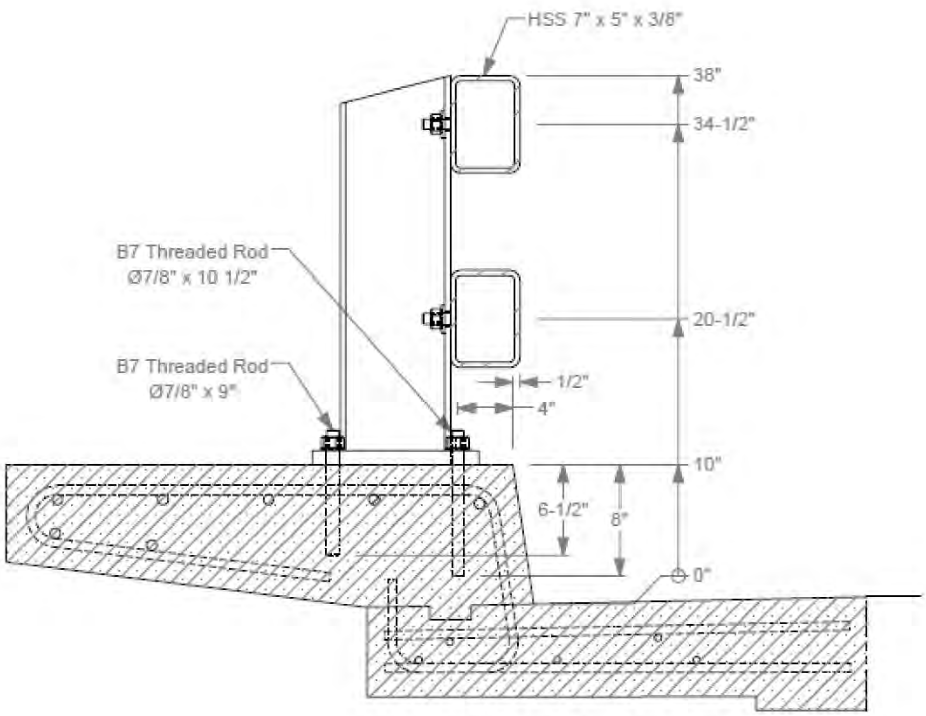
**Subject :** Retrofit Design of the MASH TL-4 Alaska 2-Tube Bridge Rail '2-149-663'

**Sponsor :** Alaska Department of Transportation & Public Facilities (AKDOT & PF)

Find strength of the Retrofit Rail system as per AASHTO LRFD Section 13, A13.3.2 Specifications

**Test Installation (Curb '2-149-663')**




Section A-A

## Curb and Deck Details (2-149-663)

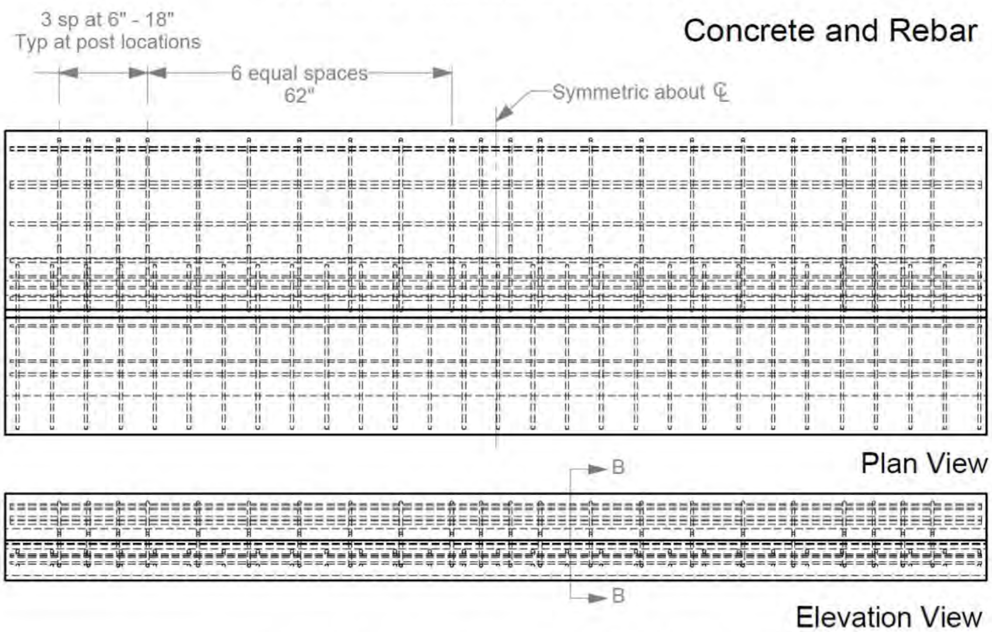



**Figure. Plan view and elevation view of curb '2-149-663'**

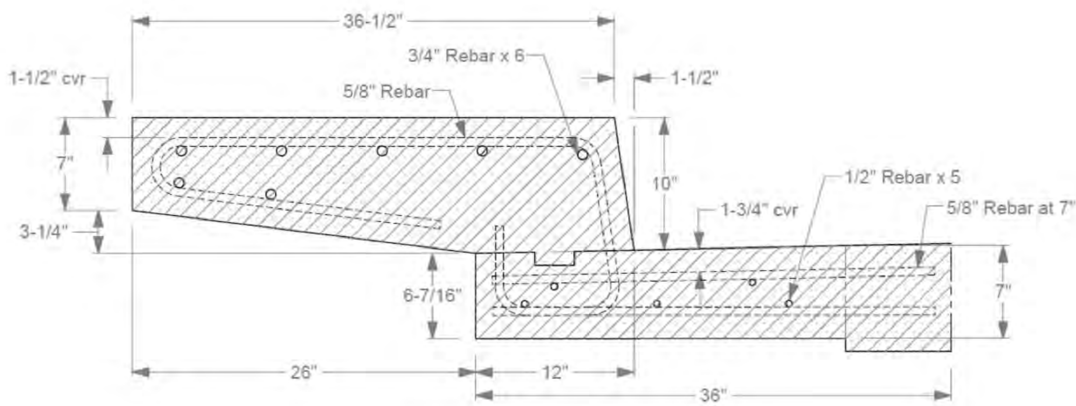
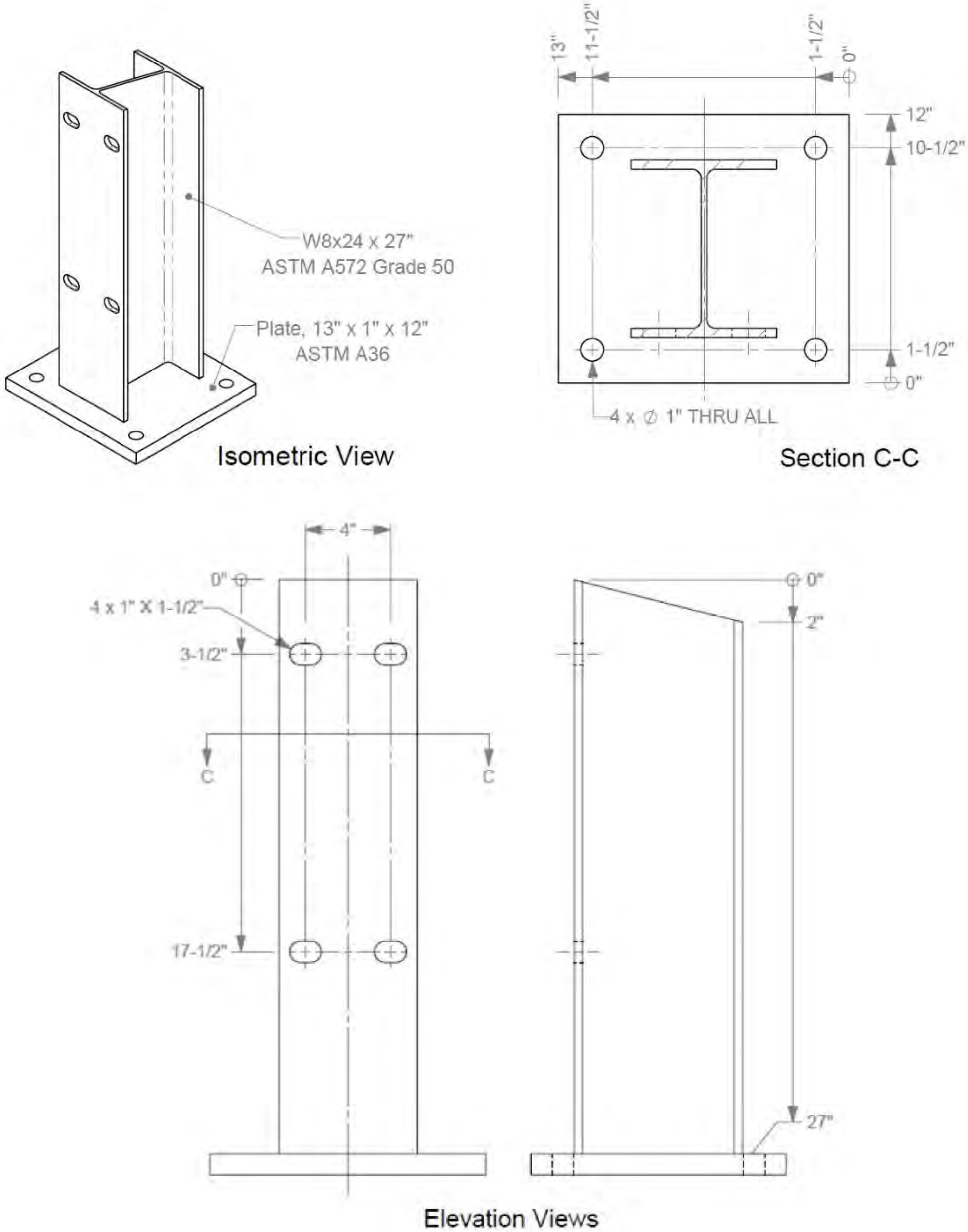



**Figure. Section B-B**

## Post Details (2-149-663)



**(1) Design Force Input:**

**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 A.13.2-1
- TL-3 design forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4(a), TL-4(b), and TL-5(a), and TL-5(b) design forces are from research conducted under NCHRP Project 22-20(2)

$TL := 4$ Test level

$F_t := 80\ kip$ Transverse impact force (kips)

$L_t := 5\ ft$ Longitudinal length of distribution of impact force (ft)

$H_e := 30\ in$ Height of equivalent transverse load from top of overlay (in.)

$H_{min} := 36\ in$ Minimum height of a MASH TL-4 barrier (in.)

$L_{span} := 80\ in = 6.667\ ft$ Span length (ft)

## (2) Input Properties for Rail Components & Geometry

### (2a) Input for Bridge Rail Properties

Steel rail properties and dimensions:
a) Steel rail is A500 Gr. B Material, fy=46 ksi
b) Steel rail bend about the y-axis
3) Steel rails are HSS7x5x3/8 members

Top Rail (HSS 7 x 5 x 3/8) Properties

$Z_{top.rail} := 13.8\ \boldsymbol{in}^3$ — Plastic section modulus of top rail about weak axis (in^3)

$Y_{top} := 34.5\ \boldsymbol{in}$ — Height to the center of the top rail (in.)

$F_{y.top} := 46\ \boldsymbol{ksi}$ — Yield of middle rail material (ksi) for ASTM A500 Grade B

Bottom Rail (HSS 7 x 5 x 3/8) Properties

$Z_{bottom.rail} := 13.8\ \boldsymbol{in}^3$ — Plastic section modulus of bottom rail about weak axis (in^3)

$Y_{bottom} := 20.5\ \boldsymbol{in}$ — Height to the center of the bottom rail (in.)

$F_{y.bottom} := 46\ \boldsymbol{ksi}$ — Yield of bottom rail material (ksi) for ASTM A500 Grade B

Calculate the Total Rail Resistance and the Average Height of all Rail Elements

$M_{p.top} := Z_{top.rail} \cdot F_{y.top} = 52.9\ \boldsymbol{kip \cdot ft}$ — Resistance of the top rail (kip-ft)

$M_{p.bottom} := Z_{bottom.rail} \cdot F_{y.bottom} = 52.9\ \boldsymbol{kip \cdot ft}$ — Resistance of the bottom rails(kip-ft)

$M_p := \left(M_{p.top} + M_{p.bottom}\right) = 105.8\ \boldsymbol{kip \cdot ft}$ — Total resistance of the rails(kip-ft)

$$Y_{bar} := \frac{M_{p.top} \cdot Y_{top} + M_{p.bottom} \cdot Y_{bottom}}{M_p}$$

Average height of combined rail resistances from top of deck (AASHTO A13.3.3-2)

$Y_{bar} = 27.5\ \boldsymbol{in}$ — Height from pavement surface to centroid of rail elemenets (in.)

(2b) Input for Post (W8x24) Properties

Steel post and base plate properties and dimensions:
a) Steel posts are A709 Gr. 50 Material, fy=50 ksi
b) Base plates are Gr. 36 Material, fy = 36 ksi
c) Base plates are 12 in. x 13 in. with 1.0 in. thickness

$t_{bp} := 1.0\ in$ — Thickness of post base plate (in.)

$t_w := 0.245\ in$ — Web thickness (in.)

$t_f := 0.4\ in$ — Flange thickness (in.)

$Z_{post} := 23.1\ in^3$ — Plastic modulus of post about strong axis (in3)

$F_{y.post} := 50\ ksi$ — Yield of post material (ksi) for ASTM A572 Grade 50

$h_{curb} := 11\ in$ — Height of curb (in.)

$h_{deck} := 6.6875\ in$ — Thickness of deck (in.)
Note: smallest is taken for conservative design

$h_p := Y_{bar} - t_{bp} - h_{curb} = 15.5\ in$ — Height measured from top of base plate to resultant force of rails (in.)

$h_{bp} := h_p + t_{bp} = 16.5\ in$ — Height measured from bottom of base plate to resultant force of rails (in.)

(2c) Input for Anchor Rods: Standard Hilti HAS-E Rod Material:

a) Anchor rods are Hilti HAS-E Rods, fu = 120 ksi
b) Anchor rods are 7/8" diameter x 10.5" embedded 8.0"

| | |
|---|---|
| $Dia_{bolts} := 0.875\ in$ | Diameter of a single anchor rods (in.) |
| $F_{u.rod} := 120\ ksi$ | Tensile strength of the anchor rods (ksi) |
| $A_{bolt} := \frac{\pi \cdot (Dia_{bolts})^2}{4} = 0.601\ in^2$ | Area of a single anchor rod (in.) |
| $N_{rod} := 4$ | Number of anchor rods |
| $N_{rod.tension} := 2$ | Number of anchor rods in tension |
| $Anchor_{proj} := 2.5\ in$ | Projected anchor rod length (in.); anchors on traffic side are considered |
| $Anchor_{embed} := 8.0\ in$ | Embeded anchor rod length (in.); anchors on traffic side are considered |
| $Anchor_{length} := Anchor_{proj} + Anchor_{embed} = 10.5\ in$ | Total anchor rod length (in.) |

(2d) Properties of concrete

| | |
|---|---|
| $f_c := 3000\ psi$ | Compressive strength of concrete (psi) |
| $f_y := 60000\ psi$ | Yield strength of rebar (psi) (psi) |
| $\phi_c := 0.85$ | Concrete strength reduction factor |

**(3) Geometric Criteria:**

$s_{post} := 5\ in$ — Post setback (in.)

$c_b := 6\ in$ — Vertical clear opening (in.)

$H_w := 10\ in$ — Height of the concrete curb measured from the top of the roadway surface (in.)

$N_R := 2$ — Number of steel rail

$h_R := 7\ in$ — Height of a single steel rail (in.)

$\Sigma A := H_w + N_R \cdot h_R = 24\ in$ — Total rail contact width (in.)

$H_r := 38\ in$ — Total height of the bridge rail measured from the top of the roadway surface/overlay (in.)

**Note**: Denoted as "H" in figure below

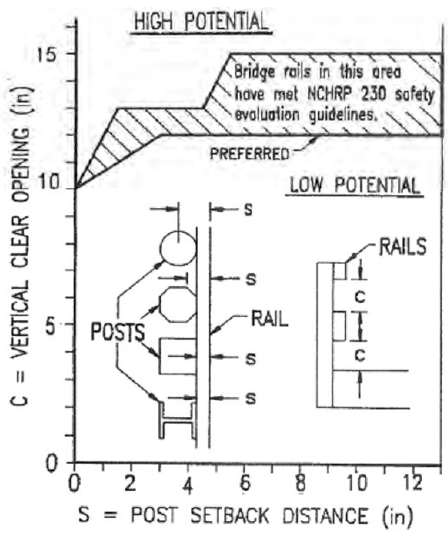
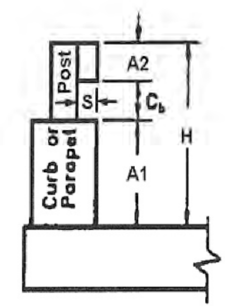



**Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post**



**(3-conti.) Geometric Criteria:**

$s_{post} = 5\ in$ $\qquad \Sigma A = 24\ in$ $\qquad H_r = 38\ in$ $\qquad ratio_{\Sigma AH} := \frac{\Sigma A}{H_r} = 0.632$

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

Lower boundary for post setback criteria x and y coordinates

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

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

Upper boundary for post setback criteria x and y coordinates

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

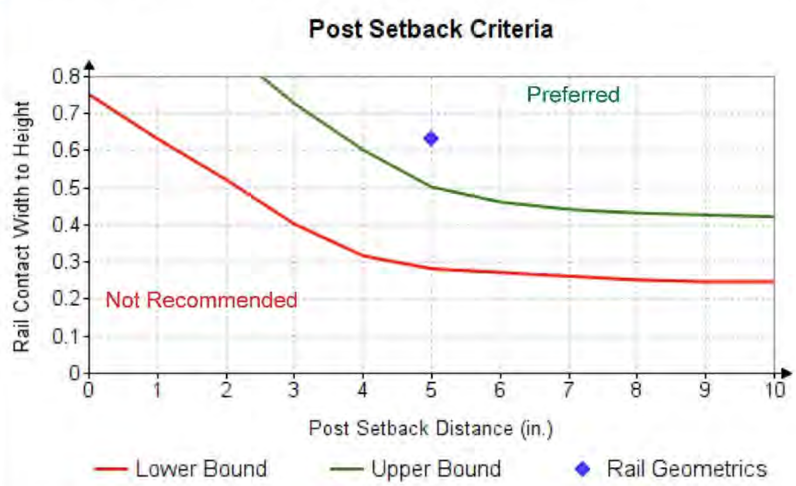
$Set_{sys.x} := \frac{s_{post}}{in} = 5$

Post setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.632$

Ratio of contact width to total height rail geometric point

Inputs $X_1 := Set_{low.x}$ $Y_1 := Set_{low.y}$ $X_2 := Set_{up.x}$ $Y_2 := Set_{up.y}$ $X_3 := Set_{sys.x}$ $Y_3 := Set_{sys.y}$




$NotRecommended := 1$ $\qquad Marginal := 2$ $\qquad Preferred := 3$

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

$snag_{sys.x} := \dfrac{s_{post}}{in} = 5$

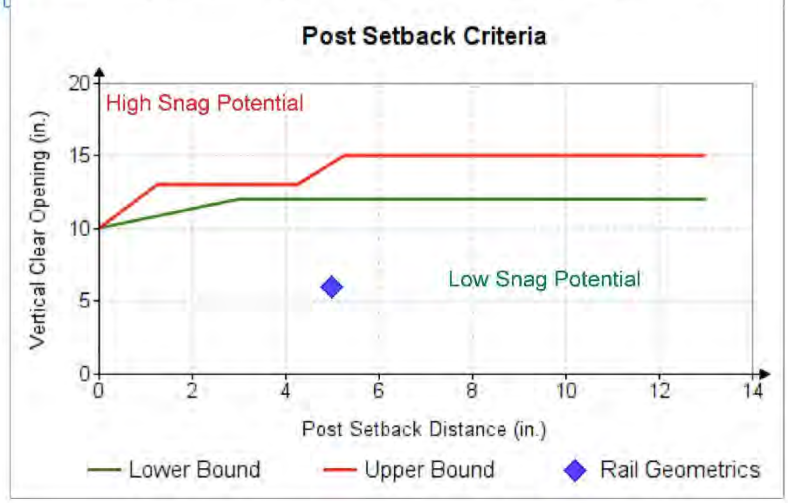
Post setback geometric point

$snag_{sys.y} := \dfrac{c_b}{in} = 6$

Vertical clear opening rail geometric point

Inputs

$X_1 := snag_{low.x}\ Y_1 := snag_{low.y}\quad X_2 := snag_{up.x}\ Y_2 := snag_{up.y}\quad X_3 := snag_{sys.x}\ Y_3 := snag_{sys.y}$




$HighSnagPotential := 1\qquad Marginal := 2\qquad LowSnagPotential := 3$

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

$Snag\_Potential\_Criteria\_Rail\_Geometric\_Point := LowSnagPotential$

**(3) Geometric Criteria - Summary of Results:**

Setback Criteria Check

$$Check\_Post\_Setback\_Criteria := \begin{array}{l} \text{if } Post\_Setback\_Criteria\_Rail\_Geometric\_Point \geq 2 \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array}$$

$$Check\_Post\_Setback\_Criteria = \text{"OK"}$$

Snag Potential Check

$$Check\_Snag\_Potential\_Criteria := \begin{array}{l} \text{if } Snag\_Potential\_Criteria\_Rail\_Geometric\_Point \geq 2 \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array}$$

$$Check\_Snag\_Potential\_Criteria = \text{"OK"}$$

Minimum Height Check (Stability Criteria)

$$Check\_Barrier\_Minimum\_Height := \begin{array}{l} \text{if } H_r \geq H_{min} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array}$$

$$Check\_Barrier\_Minimum\_Height = \text{"OK"}$$

**(4) Calculate the Ultimate transverse Load Resistance of a Single Post located @ Ybar above the deck**

(4.1) Based on the Plastic Strength of the Post: $P_{p1}$

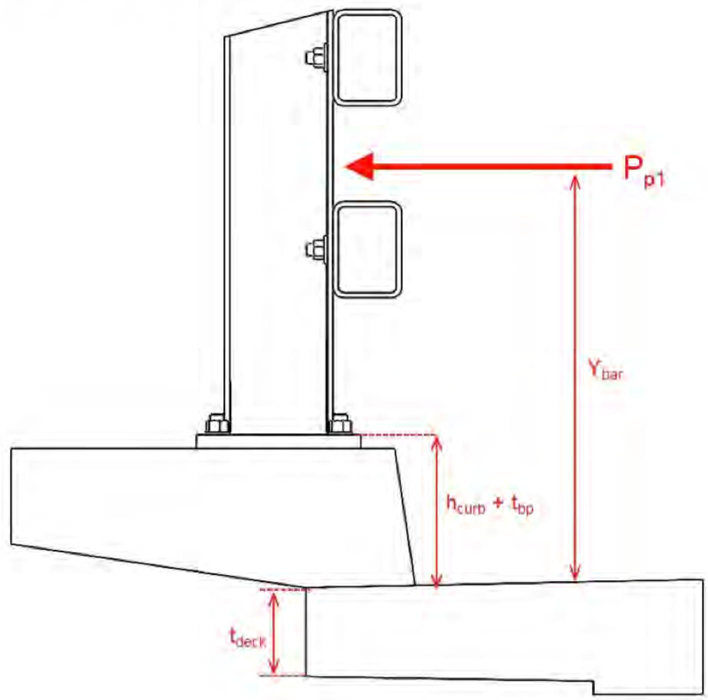



Figure. Ultimate Transverse Load Resistance Based on Plastic Strength of the Post

$$P_{p1} := \frac{Z_{post} \cdot F_{y.post}}{h_p}$$ Strength based on plastic post strength alone (ksi)

$P_{p1} = 74.516\ \boldsymbol{kip}$ Strength based on plastic post strength alone (ksi)

(4.2) Based on the Weld Strength of the Post: $P_{p2}$

Calculate fillet weld strength treated as a line in accordance with Design of Welded Structures, Blodgett, page 7.4-7, Table 5

| | |
|---|---|
| $t_{weld} := \frac{5}{16}\ in$ | Weld size (in.) |
| $b := 6.5\ in$ | Width of the welded joint (in.) |
| $d := 7.93\ in$ | Depth of the welded joint (in.) |
| $t_e := 0.707 \cdot t_{weld}$ | Throat size (in.) |
| $S_w := \left(2 \cdot b \cdot d + \frac{d^2}{3}\right) \cdot t_e = 27.408\ in^3$ | Section modulus (in3). Note: weld treated as a line |
| $F_{EXX} := 70\ ksi$ | Electrodes (ksi) |
| $\phi_{weld} := 1.0$ | Reduction factor |
| $M_{weld} := \left(0.60 \cdot F_{EXX}\right) \cdot S_w = 95.927\ kip \cdot ft$ | Moment to fail the welded joint (kip-ft) |
| $P_{p2} := \frac{M_{weld}}{h_p} = 74.266\ kip$ | Strength based on post fillet welds (kip) |
| $P_{p2} = 74.266\ kip$ | Strength based on post fillet welds (kip) |

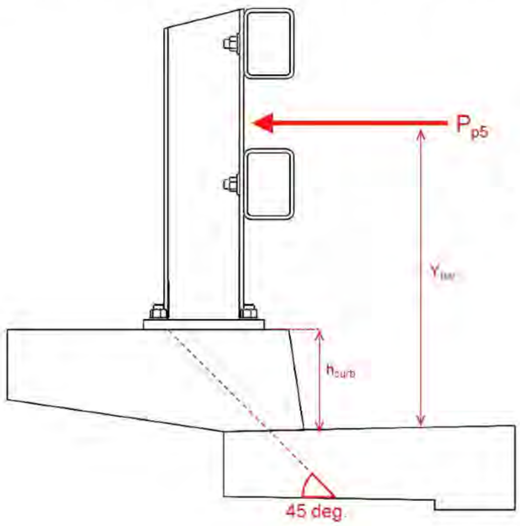
(4.3) Based on the Ultimate Strength of the Anchor Rods in Shear: $P_{p3}$

| | |
|---|---|
| $N_{rod}=4$ | Number of Anchor Rods |
| $A_{bolt}=0.601\ in^2$ | Area of single bolt (in.2) |
| $F_{V.bolt}:=(0.75\cdot A_{bolt})\cdot(0.6\ F_{u.rod})\cdot N_{rod}$ | Nominal shear strength of single bolt (kip) |
| $F_{V.bolt}=129.885\ kip$ | |
| $P_{p3}:=F_{V.bolt}=129.885\ kip$ | Force @ Ybar to achieve ultimate dynamic bolt shear strength |

(4.4) Based on the Ultimate Strength of the Anchor Bolts in Tension: $P_{p4}$

| | |
|---|---|
| $F_{T.bolt}:=(0.75\cdot F_{u.rod})\cdot(0.75\cdot A_{bolt})=40.589\ kip$ | Force to fail tension anchor bolts (kip) |
| $d_{T.bolt}:=10\ in$ | Moment arm for anchor bolts (in.) |
| $M_{T.bolt}:=2\cdot F_{T.bolt}\cdot d_{T.bolt}=67.649\ kip\cdot ft$ | Moment to fail tension anchor bolts (kip-ft) |
| $P_{p4}:=\dfrac{M_{T.bolt}}{Y_{bar}-h_{curb}}$ | Post strength to fail bolts in tension (kip) |
| $P_{p4}=49.199\ kip$ | |

(4.5) Based on the Vertical Punching Shear in Curb: $P_{p5}$



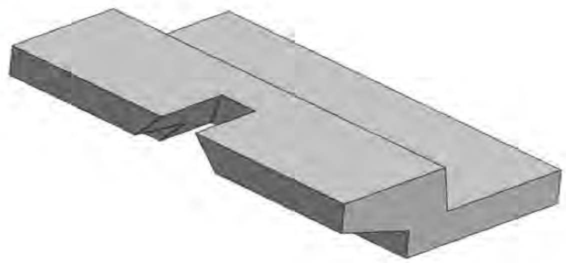



Figure. Punching Shear Failure Mechanism of the Curb

Calculate the area of the failure planes:

Note: Area deduction factor of 0.65 is used to account for concrete spalling along the steel reinforcement in the deck

$Area_{back1} := 629.82\ in^2$ — Back area measured in Solidworks (in.2)

$Area_{side1} := 455.05\ in^2$ — Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ — Reduction factor considering concrete cover

$Area_{total1} := \phi_{punch} \cdot \left(Area_{back1} + 2 \cdot Area_{side1}\right) = \left(1.001 \cdot 10^3\right)\ in^2$ — Total measured area (in.2)

Calculate the tension strength of concrete

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$\phi_c = 0.85$ — Strength reduction factor

$v_c := 2 \cdot \sqrt{\frac{f_c}{psi}}\ psi = 109.545\ psi$ — Stress corresponding to shear strength provided by concrete (psi)

Calculate the punching force to fail the curb

$V_{c.punch1} := \phi_c \cdot Area_{total1} \cdot v_c$ — Nominal shear strength provided by concrete (kip)

$V_{c.punch1} = 93.201\ kip$

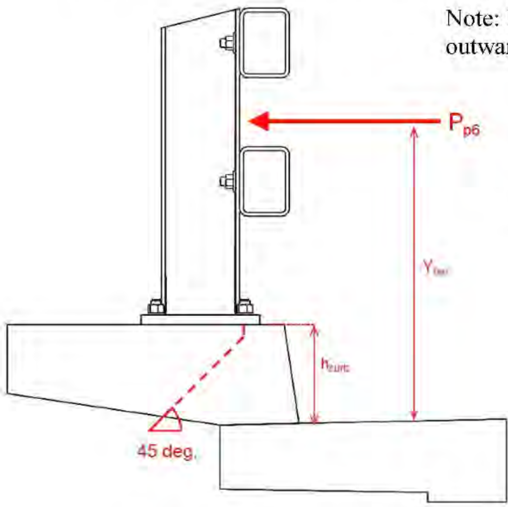
Determine force @ Ybar on post force

$d_{punch1} := 13\ in$ — Estimated moment arm for punching shear (in.)

$P_{p5} := \frac{V_{c.punch1} \cdot d_{punch1}}{h_{bp}}$ — Force to fail concrete due to vertivcal punching shear through 6" deck and curb (kip)

$P_{p5} = 73.431\ kip$ — Vertical punching shear strength (kip)

(4.6) Based on lateral punching resistance of concrete from traffic side anchor bolts: $P_{p6}$




Note: Use 3 punching shear planes extending downward and outward at 45 degees from traffic side anchor bolts

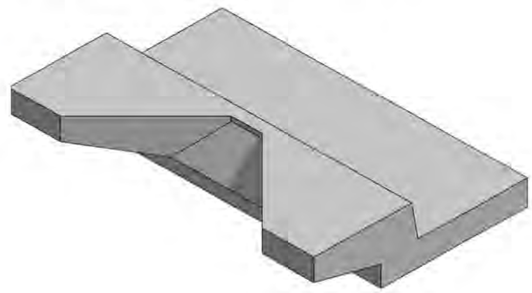


Figure. Lateral Punching Shear Failure Mechanism from Traffic Side Anchor Bolts

Calculate the area of the failure planes:

$Area_{back2} := 281.83\ in^2$ — Back area measured in Solidworks (in.2)

$Area_{side2} := 329.08\ in^2$ — Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ — Reduction factor considering concrete cover

$Area_{total2} := \phi_{punch} \cdot (Area_{back2} + 2 \cdot Area_{side2}) = 610.994\ in^2$ — Total measured area (in.2)

Calculate the punching force to fail the curb

$V_{c.punch2} := v_c \cdot (Area_{total2}) = 66.931\ kip$ — Force to fail the post due to lateral punching shear from the traffic side anchor bolts (kip)

$P_{p6} := V_{c.punch2} = 66.931\ kip$

(4.7) Based on flexural resistance of curb (Failure Section 1): $P_{p7}$

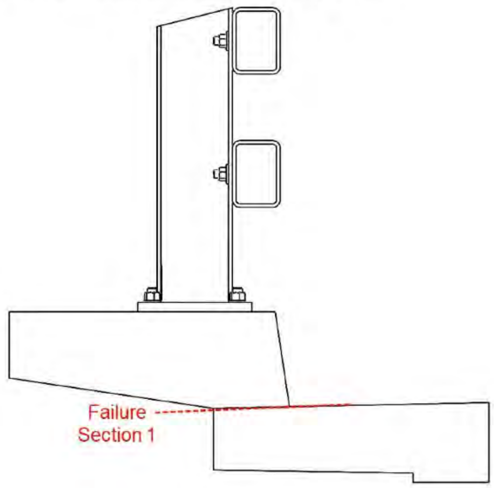



Figure. Failure section 1

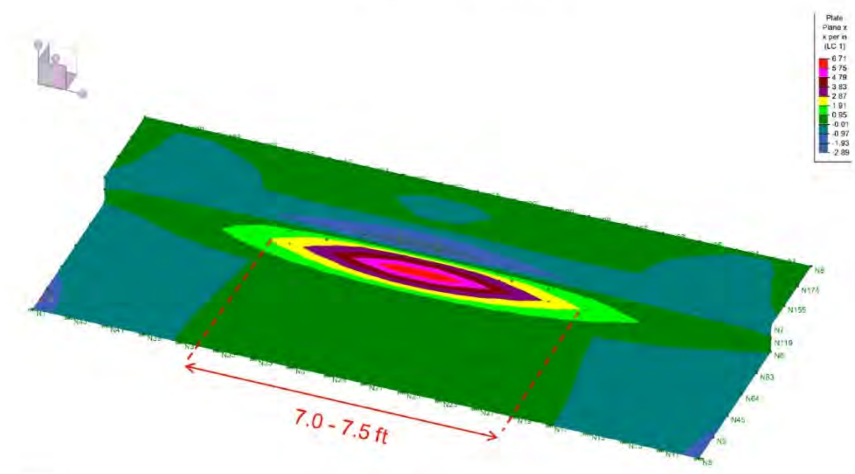



Figure. Estimated force distribution length for Failure Section 1 from structural analysis program using RISA 3D

Given design data

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS1} := 7.5\ ft$ — Longitudinal Length of Transverse Impact Force (ft)

$Y_{FS1} := Y_{bar} = 27.5\ in$ — Height measured from centroid of FS1 to resultant force of rails (in.)

$n_{FS1} := 10$ — Number of rebar within width of FS1

$A_{FS1} := n_{FS1} \cdot 0.31\ in^2$ — Area of tensile reinforcement in FS1 (in2)

$d_{FS1} := 12\ in - 1.5\ in - \frac{0.625}{2}\ in = 10.188\ in$ — Depth to tension steel from compression face of FS1 (in.)

Calculate nominal strength and post strength

$a_{FS1} := \frac{A_{FS1} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS1}} = 0.81\ in$ — Depth of Whitney stress block for FS1 (in.)

$M_{FS1} := A_{FS1} \cdot f_y \cdot \left(d_{FS1} - \frac{a_{FS1}}{2}\right) = 151.625\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p7} := \left(\frac{M_{FS1}}{Y_{FS1}}\right) = 66.164\ kip$ — Post strength based on flexural resistance of curb over the contributing length (kips)

(4.8) Based on flexural resistance of deck (Failure Section 2): $P_{p8}$

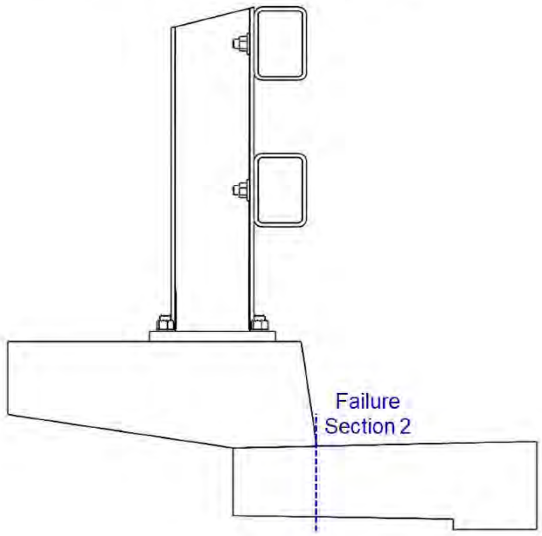



Figure. Failure section 2

The same amplified longitudinal length of distribution as 'Failure Section 1' is applied to 'Failure Section 2' because both failure sections take place where curb and deck intersect.

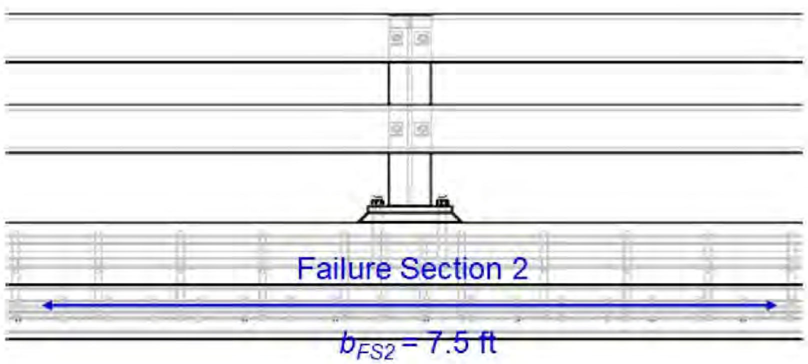



Figure. Reinforcement within Failure Section 2

Given design data

$f_c = (3 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS2} := 7.5\ ft$ — Width of FS2 (in.) Note: Width of FS2 is assumed to be the impact force projected outward at a 45 degree angle to the centroiad of FS2

$h_{FS2} := \dfrac{h_{deck}}{2} = 3.344\ in$ — Distance from roadway surface to centroid of FS2 (in.)

$Y_{FS2} := Y_{bar} + h_{FS2} = 30.844\ in$ — Height measured from centroid of FS2 to resultant force of rails (in.)

$n_{FS2} := 13$ — Number of rebar within width of FS2

$A_{FS2} := n_{FS2} \cdot 0.31\ in^2 = 4.03\ in^2$ — Area of tensile reinforcement in FS2 (in2)

Calculate nominal strength and post strength

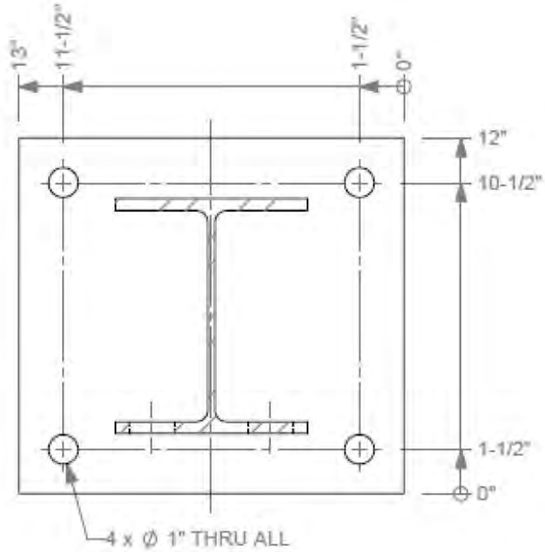
$d_{FS2} := h_{deck} - 1.75\ in - \dfrac{0.625}{2}\ in = 4.625\ in$ — Depth to tension steel from compression face of FS2 (in.)

$a_{FS2} := \dfrac{A_{FS2} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS2}} = 1.054\ in$ — Depth of Whitney stress block for FS2 (in.)

$M_{FS2} := A_{FS2} \cdot f_y \cdot \left(d_{FS2} - \dfrac{a_{FS2}}{2}\right) = 82.579\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p8} := \left(\dfrac{M_{FS2}}{Y_{FS2}}\right) = 32.128\ kip$ — Post strength based on flexural resistance of deck over the contributing length (kips)

(4.9) Based on ultimate strength of retrofit adhesive dowels in curb: $P_{p9}$




Use Hilti RE500 V3 Epoxy for 7/8" dia Anchors Embedded 8.0 inches in 3 ksi concrete curb for traffic side anchors. Note that anchors on traffic side are considered in the calculation as failure may take place at anchors in tension.

Reference Table 25 - Hilti RE-500 V3 Epoxy Adhesive Design Strength with concrete bond failure for threaded rod in uncracked concrete, 2016 Design Guide, page 151

| Nominal anchor diameter in. | Effective embedment in. (mm) | Tension — $\Phi N_n$ | | | |
|---|---|---|---|---|---|
| | | $f'_c$ = 2,500 psi (17.2 MPa) lb (kN) | $f'_c$ = 3,000 psi (20.7 MPa) lb (kN) | $f'_c$ = 4,000 psi (27.6 MPa) lb (kN) | $f'_c$ = 6,000 psi (41.4 MPa) lb (kN) |
| 7/8[10] | 3-1/2 (89) | 5,105 (22.7) | 5,595 (24.9) | 6,460 (28.7) | 7,910 (35.2) |
| | 7-7/8 (200) | 17,235 (76.7) | 18,885 (84.0) | 21,805 (97.0) | 26,705 (118.8) |
| | 10-1/2 (267) | 26,540 (118.1) | 29,070 (129.3) | 33,570 (149.3) | 41,115 (182.9) |
| | 17-1/2 (445) | 57,100 (254.0) | 62,550 (278.2) | 71,740 (319.1) | 79,395 (353.2) |

$F_{T\_\phi N} := 19370\ \mathbf{lbf}$

Interpolated between ultimate strengths of effective embedment of 7-7/8" and 10-1/2" for 7/8" diameter anchor

From Table 38 - Load adjustment factors for 7/8" diameter threaded rods in uncracked concrete, Hilti Design Guide, page 159

| 7/8-in. uncracked concrete | | Spacing factor in tension $f_{AN}$ | | | | Edge distance factor in tension $f_{RN}$ | | | |
|---|---|---|---|---|---|---|---|---|---|
| Embedment $h_{ef}$ | in. (mm) | 3-1/2 (89) | 7-7/8 (200) | 10-1/2 (267) | 17-1/2 (445) | 3-1/2 (89) | 7-7/8 (200) | 10-1/2 (267) | 17-1/2 (445) |
| Spacing (s) / Edge Distance ($c_a$) / Concrete Thickness (h), - in. (mm) | 1-3/4 (44) | n/a | n/a | n/a | n/a | 0.39 | 0.24 | 0.18 | 0.10 |
| | 4-3/8 (111) | 0.58 | 0.58 | 0.57 | 0.54 | 0.53 | 0.31 | 0.23 | 0.13 |
| | 5 (127) | 0.59 | 0.59 | 0.58 | 0.55 | 0.56 | 0.33 | 0.24 | 0.13 |
| | 5-1/2 (140) | 0.60 | 0.60 | 0.59 | 0.55 | 0.58 | 0.34 | 0.25 | 0.14 |
| | 6 (152) | 0.61 | 0.61 | 0.60 | 0.56 | 0.61 | 0.36 | 0.26 | 0.15 |
| | 7 (178) | 0.63 | 0.63 | 0.61 | 0.57 | 0.65 | 0.39 | 0.28 | 0.16 |
| | 8 (203) | 0.65 | 0.65 | 0.63 | 0.58 | 0.71 | 0.42 | 0.31 | 0.17 |
| | 9 (229) | 0.67 | 0.67 | 0.64 | 0.59 | 0.76 | 0.45 | 0.33 | 0.18 |
| | 9-7/8 (251) | 0.69 | 0.69 | 0.66 | 0.59 | 0.80 | 0.48 | 0.35 | 0.19 |
| | 10 (254) | 0.69 | 0.69 | 0.66 | 0.60 | 0.81 | 0.49 | 0.35 | 0.19 |
| | 11 (279) | 0.71 | 0.71 | 0.67 | 0.60 | 0.87 | 0.52 | 0.38 | 0.21 |
| | 12 (305) | 0.73 | 0.73 | 0.69 | 0.61 | 0.92 | 0.56 | 0.40 | 0.22 |
| | 12-1/2 (318) | 0.74 | 0.74 | 0.70 | 0.62 | 0.95 | 0.59 | 0.41 | 0.23 |
| | 14 (356) | 0.76 | 0.76 | 0.72 | 0.63 | 1.00 | 0.66 | 0.46 | 0.25 |
| | 16 (406) | 0.80 | 0.80 | 0.75 | 0.65 | | 0.75 | 0.52 | 0.29 |
| | 18 (457) | 0.84 | 0.84 | 0.79 | 0.67 | | 0.84 | 0.59 | 0.32 |
| | 19-1/2 (495) | 0.87 | 0.87 | 0.81 | 0.69 | | 0.92 | 0.64 | 0.35 |
| | 20 (508) | 0.88 | 0.88 | 0.82 | 0.69 | | 0.94 | 0.65 | 0.36 |
| | 22 (559) | 0.91 | 0.91 | 0.85 | 0.71 | | 1.00 | 0.72 | 0.40 |
| | 24 (610) | 0.95 | 0.95 | 0.88 | 0.73 | | | 0.78 | 0.43 |
| | 26 (660) | 0.99 | 0.99 | 0.91 | 0.75 | | | 0.85 | 0.47 |
| | 28 (711) | 1.00 | 1.00 | 0.94 | 0.77 | | | 0.91 | 0.50 |
| | 30 (762) | | | 0.98 | 0.79 | | | 0.98 | 0.54 |
| | 36 (914) | | | 1.00 | 0.84 | | | 1.00 | 0.65 |
| | > 48 (1219) | | | | 0.96 | | | | 0.86 |

$f_{AN} := 0.689$

Spacing reduction factor for uncracked concrete; 10 in. spacing is used

$f_{RN} := 0.325$

Edge distance reduction factor for uncracked concrete; 5 in.edge distance at the mid-depth of the curb is used

$f_{temp} := 1.0$

Temperature reduction factor for uncracked concrete; Temperature range A is used

$\phi_{TRE500} := 1.33$

Edge distance and spacing values from Hilti are conservative for reinforced concrete; Use factor of 1.33 for dynamic loading

$F_{TRE500} := \phi_{TRE500} \cdot F_{T\_\phi N} \cdot f_{AN} \cdot f_{RN} \cdot f_{temp} = 5.769\ \boldsymbol{kip}$

Adhesive bond strength of ahcnhor with RE500V3 for embedded depth of 8.0 in. (kip)

$d_{anchor} := 10.5\ \boldsymbol{in}$

Distance between edge of baseplate to centerline of tension bolts (in.)

$$P_{p9} := \frac{2 \cdot F_{TRE500} \cdot d_{anchor}}{h_p} = 7.816\ \boldsymbol{kip}$$

Post strength based on ultimate strength of retrofit adhesive dowels in curb (kip)

**Comments: Too conservative design due to a large factor of safety on Hilti's adhesive anchor**

Adjustment for exceedingly conservative design strength in Hilti specification using test data

Extrapolate ultimate strength for embedded depth of 8.0 in. from TTI testing of anchor with RE500V3 (in.)

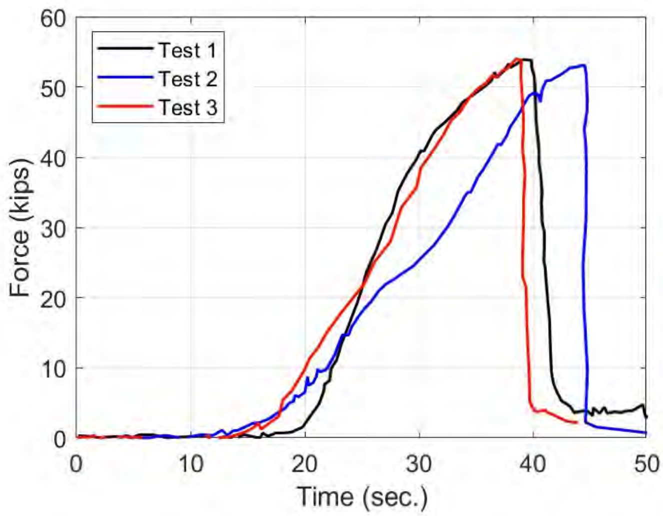



Figure. TTI test results for Hilti RE-500-V3 with 7/8 in. threaded rod and 8 in. embedment depth

Figure. Cone-shaped failure observed in TTI test

$F_{T\_\phi N} := 54\ \boldsymbol{kip}$ — Average ultimate strength of anchor with RE500V3 for embedded depth of 8.0 in. from TTI testing (in.)

$F_{TRE500} := \phi_{TRE500} \cdot F_{T\_\phi N} \cdot f_{AN} \cdot f_{RN} \cdot f_{temp} = 16.082\ \boldsymbol{kip}$ — Estimated strength to induce the cone-shaped failure (kip)

$h_p = 15.5\ \boldsymbol{in}$ — Height measured from top of base plate to resultant force of rails (in.)

$P_{p9} := \dfrac{2 \cdot F_{TRE500} \cdot d_{anchor}}{h_p} = 21.789\ \boldsymbol{kip}$ — Post strength based on ultimate strength of retrofit adhesive ahcnhors in curb (kip)

**Comment: The post strength based on the ultimate strength of the adhesive anchors here is likely conservative due to the presence of stirrup and longitudinal reinforcement. This reinforcement very likely reinforces the anchor strength for this strength check.**

**(5) Summarize post strengths and select limiting post strength for LRFD strength analysis:**

$P_{p1} = 74.516\ \boldsymbol{kip}$ Post strength based on plastic strength of the post

$P_{p2} = 74.266\ \boldsymbol{kip}$ Post strength based on post fillet welds

$P_{p3} = 129.885\ \boldsymbol{kip}$ Post strenth based on anchor bolt shear strength

$P_{p4} = 49.199\ \boldsymbol{kip}$ Post strength based on ultimate tension strength of anchor bolts

$P_{p5} = 73.431\ \boldsymbol{kip}$ Post strength based on the curb punching shear strength

$P_{p6} = 66.931\ \boldsymbol{kip}$ Post strength based on lateral punching shear on the traffice side bolts

$P_{p7} = 66.164\ \boldsymbol{kip}$ Post strength based on flexural resistance (FS1)

$P_{p8} = 32.128\ \boldsymbol{kip}$ Post strength based on flexural resistance (FS2)

$P_{p9} = 21.789\ \boldsymbol{kip}$ Post strength based on ultimate strength of adhesive dowels in curb

$$P_p := min\left(P_{p1}, P_{p2}, P_{p3}, P_{p4}, P_{p5}, P_{p6}, P_{p7}, P_{p8}, P_{p9}\right) = 21.789\ \boldsymbol{kip}$$

$P_p := P_{p9}$

Therefore use Pp9 as limiting strength.

**(6) Calculate the rail strength for multiple spans @ $H_e$ height:**

Design forces and designations for TL-4

$H_e = 30\ in$ Height of equivalent transverse load (in.)

$P_p = 21.789\ kip$ Governing post strength (kip)

$Y_{bar} = 27.5\ in$ Average height of combined rail resistances (in.)

$L_{span} = 6.667\ ft$ Span length (ft)

$L_t = 5\ ft$ Length of loading for TL-4 (ft)

$M_p = 105.8\ kip \cdot ft$ Total resistance of the rails (kip-ft)

** 1 Span Case

$N_1 := 1$ 1 Span Case

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_1 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_1 = 203.136\ kip$ Resistance for 1 Span

** 2 Span Case

$N_2 := 2$ 2 Span Case

$$R_2 := \frac{16 \cdot M_p + {N_2}^2 \cdot P_p \cdot L_{span}}{2 \cdot N_2 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_2 = 104.946\ kip$ Resistance for 2 Spans

** 3 Span Case

$N_3 := 3$ — 3 Span Case

$$R_3 := \frac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_3 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_3 = 81.568$ ***kip*** — Resistance for 3 Span

** 4 Span Case

$N_4 := 4$ — 4 Span Case

$$R_4 := \frac{16 \cdot M_p + N_4^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_4 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_4 = 83.109$ ***kip*** — Resistance for 4 Spans

** 5 Span Case

$N_5 := 5$ — 5 Span Case

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_5 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_5 = 83.984$ ***kip*** — Resistance for 3 Span

** 6 Span Case

$N_6 := 6$ — 5 Span Case

$$R_6 := \frac{16 \cdot M_p + N_6^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_6 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_6 = 92.295$ ***kip*** — Resistance for 3 Span

**(7) Total Ultimate Resistance (Nominal Resistance) of Railing:** $R_R$

$R_r := min(R_1, R_2, R_3, R_4, R_5, R_6) = 81.568\ kip$ — Total ultimate resistance of the bridge rail system @ ybar (kip)

$H_{e.TL4} := 30\ in$ — Height of transverse impact load (in.); MASH TL-4

$Y_{bar} = 27.5\ in$ — Height of resultant force (in.)

$F_{t.TL4} := 80\ kip$ — Transverse impact force (in.); MASH TL-4

$R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL4}}\right) = 74.771\ kip$ — Total ultimate resistance of the bridge rail system @He (kip)

$$Check\_Retrofit\_Design := \begin{array}{l} \text{if } R_R \ge F_{t.TL4} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} = \text{"NOT OK"}$$

**The strength of the retrofit design does not meet MASH TL-4 requirement. Check if the design meets TL-3 requirement.**

$H_{e.TL3} := 19\ in$ — Height of transverse impact load (in.)

$F_{t.TL3} := 71\ kip$ — Transverse impact force (in.)

$R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL3}}\right) = 118.059\ kip$ — Total ultimate resistance of the bridge rail system @He (kip)

$$Check\_Retrofit\_Design := \begin{array}{l} \text{if } R_R \ge F_{t.TL3} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} = \text{"OK"}$$

**The strength of the retrofit design meets MASH TL-3 requirement.**

## Summary:

The strength of the retrofit design is approximately 75 kips at the TL-4 height of 30 in. and this is less than the target strength of 80 kips.

Adhesive bond failure is considered the governing failure mechanism, and its design strength is significantly reduced due to conservative strength reduction factors.

However, the design does meet MASH TL-3. TTI recommends performing static load testing of posts or full-scale crash testing for MASH TL-4.

## B-4. Strength Analysis of Alaska Steel Bridge Railing Retrofit Design

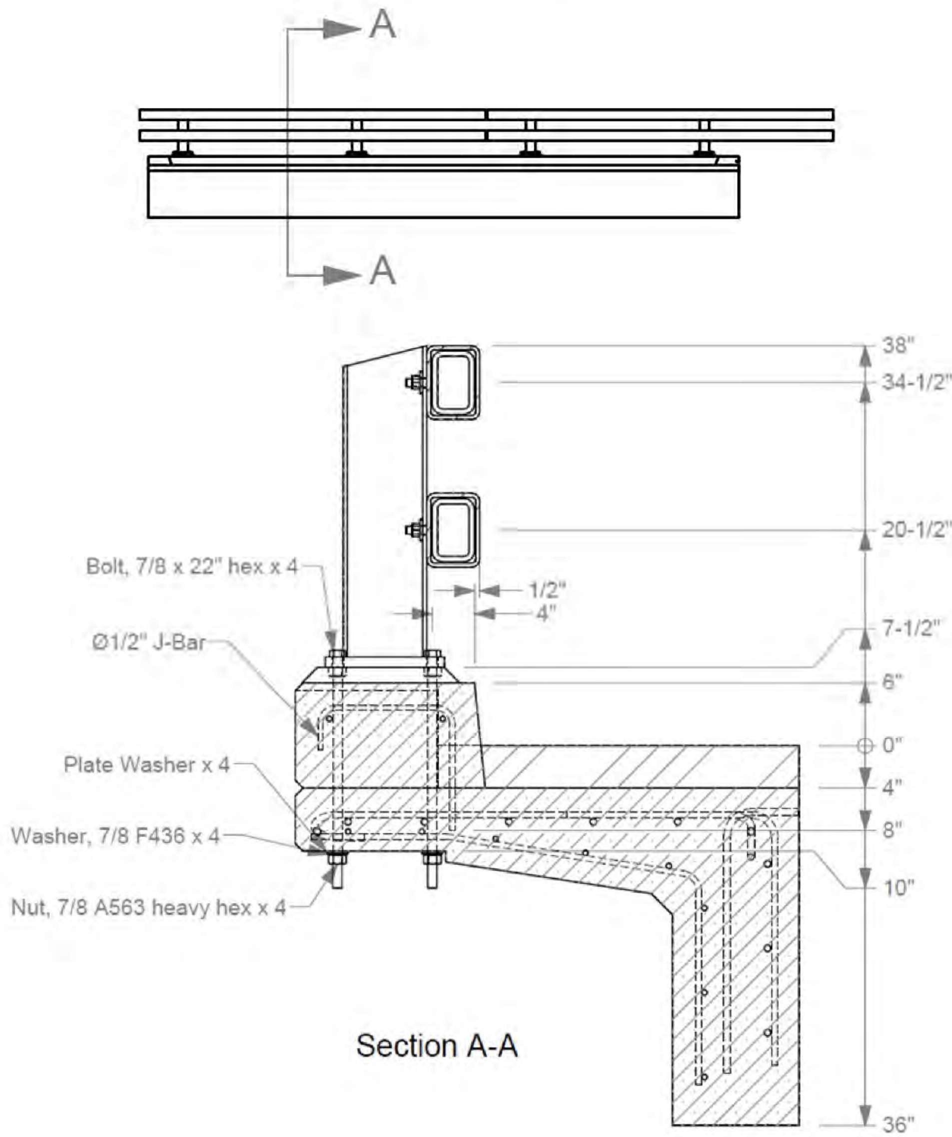
Texas A&M Transportation Institute

**Subject :** Retrofit Design of the MASH TL-4 Alaska 2-Tube Bridge Rail

**Sponsor :** Alaska Department of Transportation & Public Facilities (AKDOT & PF)

Find strength of the Retrofit Rail system as per AASHTO LRFD Section 13, A13.3.2 Specifications

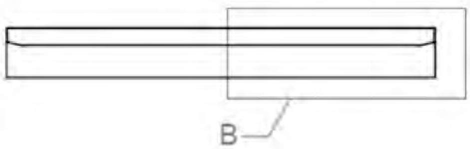
**Test Installation**



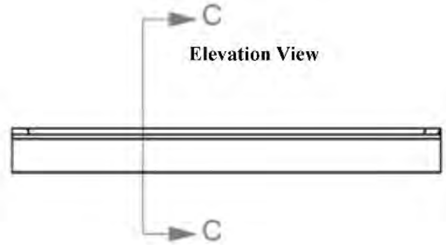

Section A-A

## Curb and Deck Details

Plan View




Elevation View



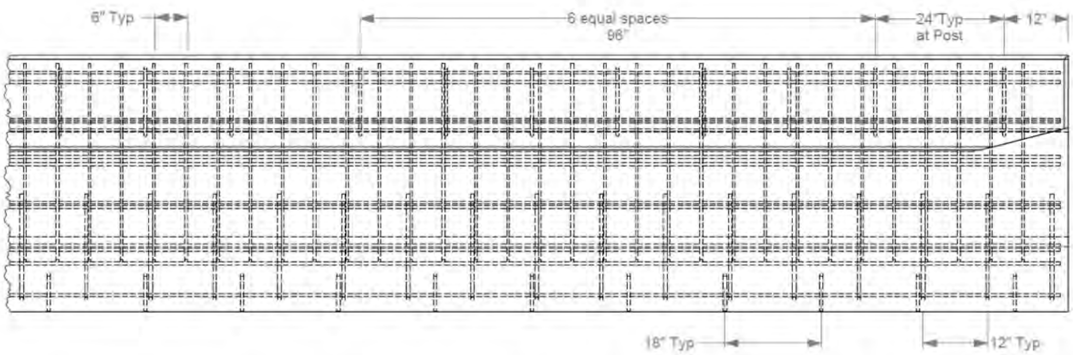




**Figure. Detail B (Hook bars hidden for clarity)**

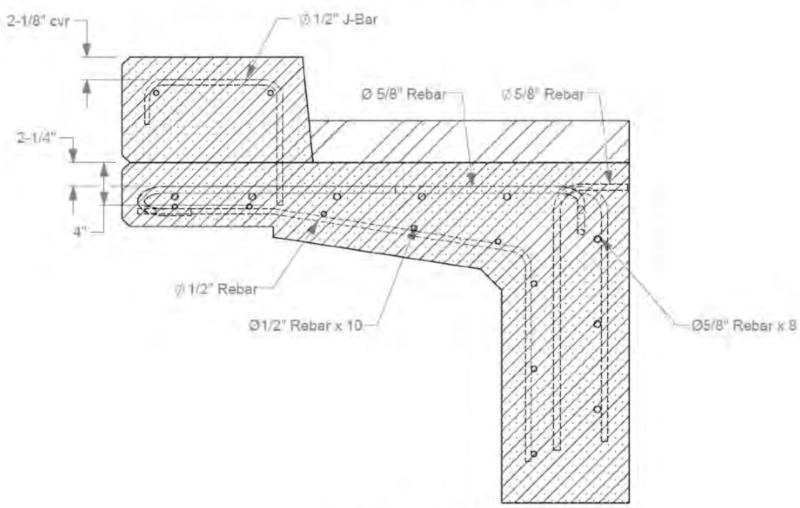
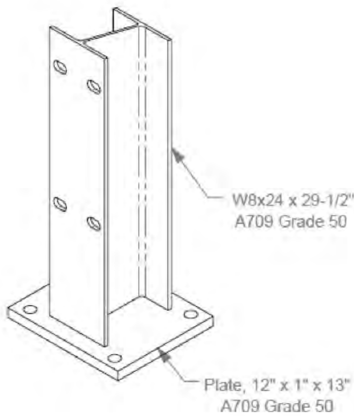
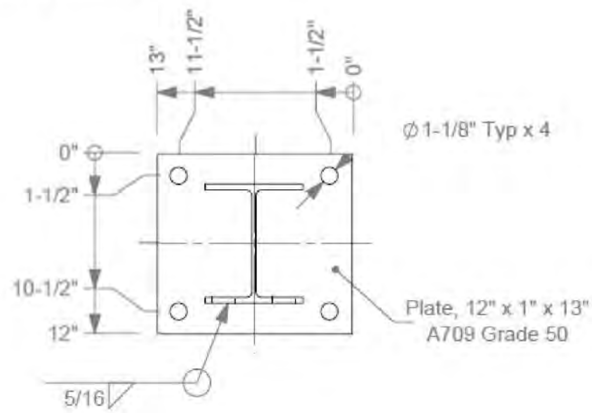



**Figure. Section C-C**

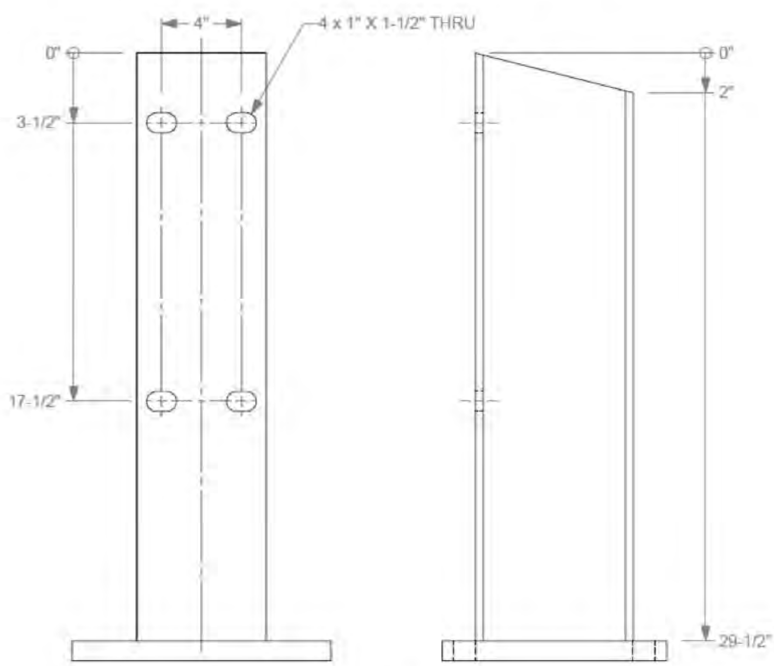
## Post Details




**Isometric View**




**Plan View**




**Elevation View**

## (1) Design Force Input:

### 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 A.13.2-1
- TL-3 design forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4(a), TL-4(b), and TL-5(a), and TL-5(b) design forces are from research conducted under NCHRP Project 22-20(2)

$TL := 4$ Test level

$F_t := 80\ \boldsymbol{kip}$ Transverse impact force (kips)

$L_t := 5\ \boldsymbol{ft}$ Longitudinal length of distribution of impact force (ft)

$H_e := 30\ \boldsymbol{in}$ Height of equivalent transverse load from top of overlay (in.)

$H_{min} := 36\ \boldsymbol{in}$ Minimum height of a MASH TL-4 barrier (in.)

$L_{span} := 120\ \boldsymbol{in} = 10\ \boldsymbol{ft}$ Span length (ft)

## (2) Input Properties for Rail Components & Geometry

(2a) Input for Bridge Rail Properties

Steel rail properties and dimensions:
a) Steel rail is A500 Gr. B Material, fy=46 ksi
b) Steel rail bend about the y-axis
3) Steel rails are HSS7x5x3/8 members

Top Rail (HSS 7 x 5 x 3/8) Properties

$Z_{top.rail} := 13.8\ in^3$ — Plastic section modulus of top rail about weak axis (in^3)

$Y_{top} := 34.5\ in$ — Height to the center of the top rail (in.)

$F_{y.top} := 46\ ksi$ — Yield of middle rail material (ksi) for ASTM A500 Grade B

Bottom Rail (HSS 7 x 5 x 3/8) Properties

$Z_{bottom.rail} := 13.8\ in^3$ — Plastic section modulus of bottom rail about weak axis (in^3)

$Y_{bottom} := 20.5\ in$ — Height to the center of the bottom rail (in.)

$F_{y.bottom} := 46\ ksi$ — Yield of bottom rail material (ksi) for ASTM A500 Grade B

Calculate the Total Rail Resistance and the Average Height of all Rail Elements

$M_{p.top} := Z_{top.rail} \cdot F_{y.top} = 52.9\ kip \cdot ft$ — Resistance of the top rail (kip-ft)

$M_{p.bottom} := Z_{bottom.rail} \cdot F_{y.bottom} = 52.9\ kip \cdot ft$ — Resistance of the bottom rails(kip-ft)

$M_p := \left(M_{p.top} + M_{p.bottom}\right) = 105.8\ kip \cdot ft$ — Total resistance of the rails(kip-ft)

$$Y_{bar} := \frac{M_{p.top} \cdot Y_{top} + M_{p.bottom} \cdot Y_{bottom}}{M_p}$$

Average height of combined rail resistances from top of deck (AASHTO A13.3.3-2)

$Y_{bar} = 27.5\ in$ — Height from pavement surface to centroid of rail elemenets (in.)

(2b) Input for Post (W8x24) Properties

Steel post and base plate properties and dimensions:
a) Steel posts are ASTM A709 Grade 50; Material, fy=50 ksi
b) Base plates are Gr. 50 Material, fy = 50 ksi
c) Base plates are 12 in. x 13 in. with 1.0 in. thickness

| | |
|---|---|
| $t_{bp} := 1.0\ in$ | Thickness of post base plate (in.) |
| $t_w := 0.245\ in$ | Web thickness (in.) |
| $t_f := 0.4\ in$ | Flange thickness (in.) |
| $Z_{post} := 23.1\ in^3$ | Plastic modulus of post about strong axis (in3) |
| $F_{y.post} := 50\ ksi$ | Yield of post material (ksi) for ASTM A572 Grade 50 |
| $h_{curb} := 6\ in$ | Height of curb measured from the top of the asphalt overlay (in.) |
| $h_{deck} := 6.0\ in$ | Thickness of deck (in.)<br>Note: smallest is taken for conservative design |
| $t_{grout} := 1.5\ in$ | Thickness of grout (in.) |
| $h_{overlay} := 4\ in$ | Height of asphalt overlay (in.) |
| $h_p := Y_{bar} - t_{bp} - t_{grout} - h_{curb} = 19\ in$ | Height measured from top of base plate to resultant force of rails (in.) |
| $h_{bp} := h_p + t_{bp} = 20\ in$ | Height measured from bottom of base plate to resultant force of rails (in.) |

(2c) Input for Threaded Rods

a) Anchor rods are 7/8" diameter x 22" embedded through the curb and deck
b) Single anchor washer (F436) is used for each anchor at the bottom of the deck

| | |
|---|---|
| $Dia_{bolts} := 0.875\ in$ | Diameter of a single anchor rods (in.) |
| $F_{u.rod} := 120\ ksi$ | Tensile strength of the anchor rods (ksi) |
| $A_{bolt} := \dfrac{\pi \cdot \left(Dia_{bolts}\right)^2}{4} = 0.601\ in^2$ | Area of a single anchor rod (in.) |
| $N_{rod} := 4$ | Number of anchor rods |
| $N_{rod.tension} := 2$ | Number of anchor rods in tension |
| $Anchor_{proj} := 6\ in$ | Projected anchor rod length (in.); anchors on traffic side are considered |
| $Anchor_{embed} := 16\ in$ | Embeded anchor rod length (in.); anchors on traffic side are considered |
| $Anchor_{length} := Anchor_{proj} + Anchor_{embed} = 22\ in$ | Total anchor rod length (in.) |

(2d) Properties of concrete

| | |
|---|---|
| $f_c := 5000\ psi$ | Compressive strength of concrete of deck (psi) |
| $f_y := 60000\ psi$ | Yield strength of rebar (psi) |
| $\phi_c := 0.85$ | Concrete strength reduction factor |

**(3) Geometric Criteria:**

$s_{post} := 5\ in$ — Post setback (in.)

$c_b := 11\ in$ — Vertical clear opening (in.)

$H_w := 6\ in$ — Height of the concrete curb measured from the top of the roadway surface (in.)

$N_R := 2$ — Number of steel rail

$h_R := 7\ in$ — Height of a single steel rail (in.)

$\Sigma A := H_w + N_R \cdot h_R = 20\ in$ — Total rail contact width (in.)

$H_r := 38\ in$ — Total height of the bridge rail measured from the top of the roadway surface/overlay (in.)

**Note**: Denoted as "H" in figure below

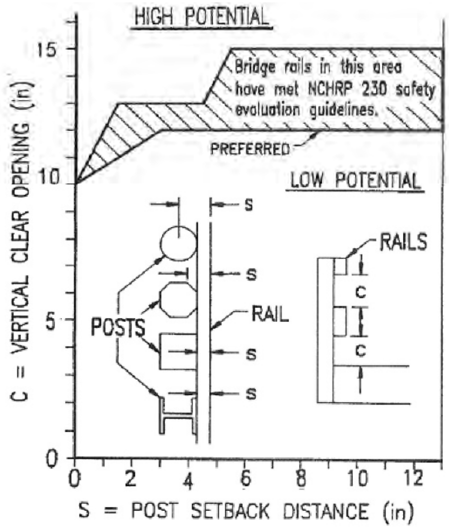
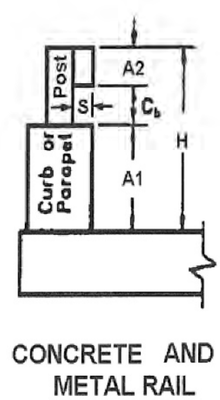



**Figure A13.1.1-2—Potential for Wheel, Bumper, or Hood Impact with Post**



**(3-conti.) Geometric Criteria:**

$s_{post} = 5\ in$ $\quad \Sigma A = 20\ in$ $\quad H_r = 38\ in$ $\quad ratio_{\Sigma AH} := \dfrac{\Sigma A}{H_r} = 0.526$

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

Lower boundary for post setback criteria x and y coordinates

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

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

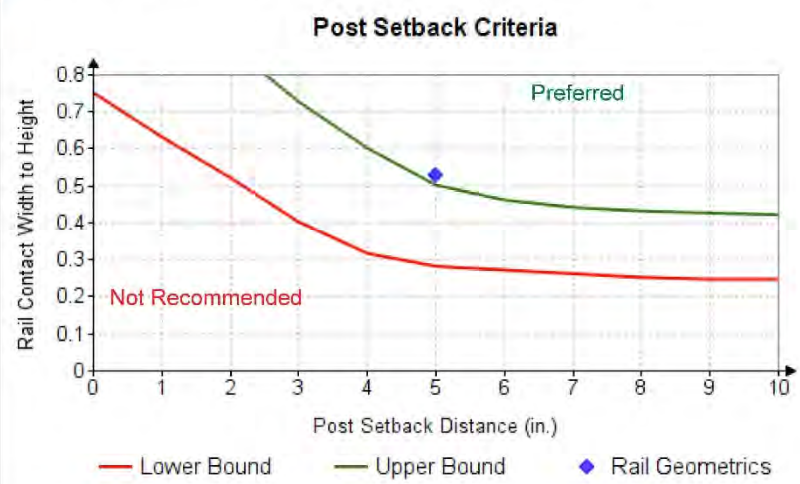
Upper boundary for post setback criteria x and y coordinates

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

$Set_{sys.x} := \dfrac{s_{post}}{in} = 5$ — Post setback rail geometric point

$Set_{sys.y} := ratio_{\Sigma AH} = 0.526$ — Ratio of contact width to total height rail geometric point

Inputs: $X_1 := Set_{low.x}$ $\ Y_1 := Set_{low.y}$ $\quad X_2 := Set_{up.x}$ $\ Y_2 := Set_{up.y}$ $\quad X_3 := Set_{sys.x}$ $\ Y_3 := Set_{sys.y}$




$NotRecommended := 1$ $\quad Marginal := 2$ $\quad Preferred := 3$

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

$Post\_Setback\_Criteria\_Rail\_Geometric\_Point := Preferred$

**(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

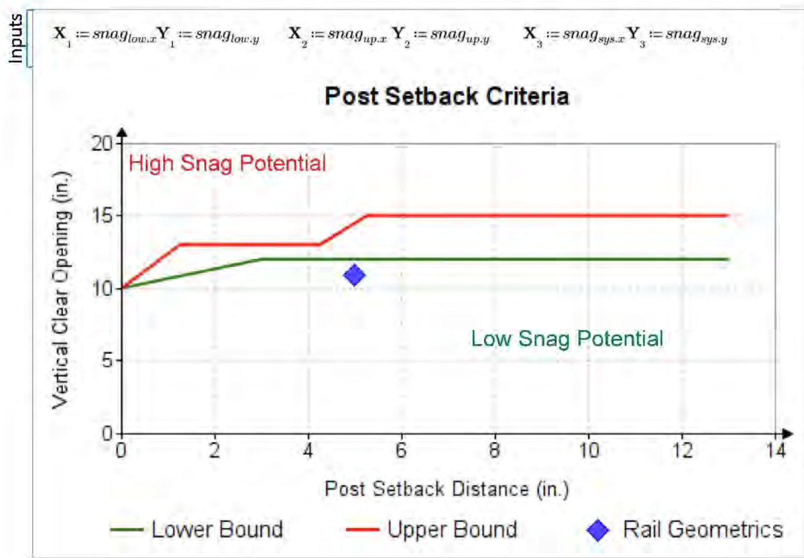
$snag_{up.x} := [0\ \ 1.25\ \ 4.25\ \ 5.25\ \ 13]$

$snag_{up.y} := [10\ \ 13\ \ 13\ \ 15\ \ 15]$

Upper boundary for snag potential criteria x and y coordinates

$snag_{sys.x} := \frac{s_{post}}{in} = 5$ Post setback geometric point

$snag_{sys.y} := \frac{c_b}{in} = 11$ 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:**

Setback Criteria Check

$$Check\_Post\_Setback\_Criteria := \begin{Vmatrix} \text{if } Post\_Setback\_Criteria\_Rail\_Geometric\_Point \geq 2 \\ \quad \Vert \text{“OK”} \\ \text{else} \\ \quad \Vert \text{“NOT OK”} \end{Vmatrix}$$

$Check\_Post\_Setback\_Criteria =$ “OK”

Snag Potential Check

$$Check\_Snag\_Potential\_Criteria := \begin{Vmatrix} \text{if } Snag\_Potential\_Criteria\_Rail\_Geometric\_Point \geq 2 \\ \quad \Vert \text{“OK”} \\ \text{else} \\ \quad \Vert \text{“NOT OK”} \end{Vmatrix}$$

$Check\_Snag\_Potential\_Criteria =$ “OK”

Minimum Height Check (Stability Criteria)

$$Check\_Barrier\_Minimum\_Height := \begin{Vmatrix} \text{if } H_r \geq H_{min} \\ \quad \Vert \text{“OK”} \\ \text{else} \\ \quad \Vert \text{“NOT OK”} \end{Vmatrix}$$

$Check\_Barrier\_Minimum\_Height =$ “OK”

**(4) Calculate the Ultimate transverse Load Resistance of a Single Post located @ Ybar above the deck**

(4.1) Based on the Plastic Strength of the Post: $P_{p1}$

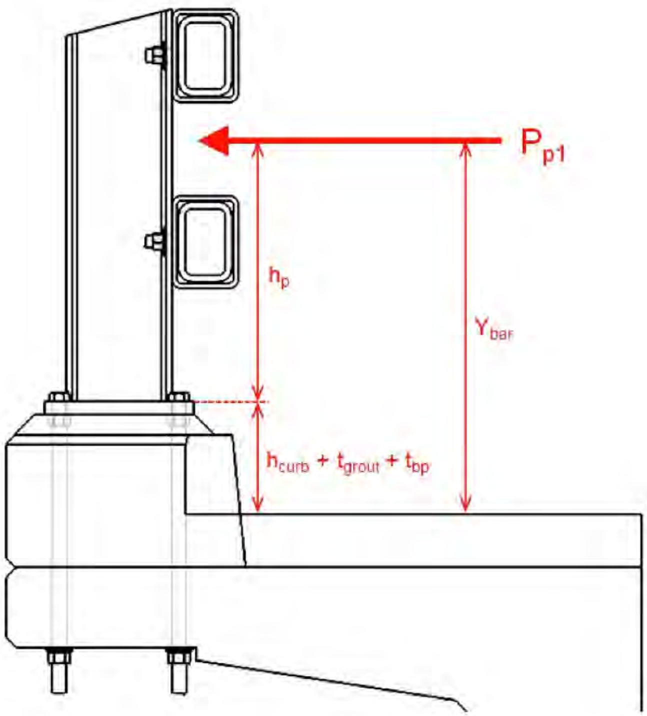



Figure. Ultimate Transverse Load Resistance Based on Plastic Strength of the Post

$$P_{p1} := \frac{Z_{post} \cdot F_{y.post}}{h_p}$$ Strength based on plastic post strength alone (ksi)

$P_{p1} = 60.789\ \boldsymbol{kip}$ Strength based on plastic post strength alone (ksi)

(4.2) Based on the Weld Strength of the Post: $P_{p2}$

Calculate fillet weld strength treated as a line in accordance with Design of Welded Structures, Blodgett, page 7.4-7, Table 5

| | |
|---|---|
| $t_{weld} := \frac{5}{16}\ in$ | Weld size (in.) |
| $b := 6.5\ in$ | Width of the welded joint (in.) |
| $d := 7.93\ in$ | Depth of the welded joint (in.) |
| $t_e := 0.707 \cdot t_{weld}$ | Throat size (in.) |
| $S_w := \left(2 \cdot b \cdot d + \frac{d^2}{3}\right) \cdot t_e = 27.408\ in^3$ | Section modulus (in3). Note: weld treated as a line |
| $F_{EXX} := 70\ ksi$ | Electrodes (ksi) |
| $\phi_{weld} := 1.0$ | Reduction factor |
| $M_{weld} := \left(0.60 \cdot F_{EXX}\right) \cdot S_w = 95.927\ kip \cdot ft$ | Moment to fail the welded joint (kip-ft) |
| $P_{p2} := \frac{M_{weld}}{h_p} = 60.585\ kip$ | Strength based on post fillet welds (kip) |
| $P_{p2} = 60.585\ kip$ | Strength based on post fillet welds (kip) |

(4.3) Based on the Ultimate Strength of the Anchor Rods in Shear: $P_{p3}$

$N_{rod} = 4$ — Number of Anchor Rods

$A_{bolt} = 0.601\ \boldsymbol{in}^2$ — Area of single bolt (in.2)

$F_{V.bolt} := (0.75 \cdot A_{bolt}) \cdot (0.6\ F_{u.rod}) \cdot N_{rod}$ — Nominal shear strength of single bolt (kip)

$F_{V.bolt} = 129.885\ \boldsymbol{kip}$

$P_{p3} := F_{V.bolt} = 129.885\ \boldsymbol{kip}$ — Force @ Ybar to achieve ultimate dynamic bolt shear strength

(4.4) Based on the Ultimate Strength of the Anchor Bolts in Tension: $P_{p4}$

$F_{T.bolt} := (0.75 \cdot F_{u.rod}) \cdot (0.75 \cdot A_{bolt}) = 40.589\ \boldsymbol{kip}$ — Force to fail tension anchor bolts (kip)

$d_{T.bolt} := 10.125\ \boldsymbol{in}$ — Moment arm for anchor bolts (in.)

$M_{T.bolt} := 2 \cdot F_{T.bolt} \cdot d_{T.bolt} = 68.494\ \boldsymbol{kip \cdot ft}$ — Moment to fail tension anchor bolts (kip-ft)

$$P_{p4} := \frac{M_{T.bolt}}{h_p}$$ — Post strength to fail bolts in tension (kip)

$P_{p4} = 43.259\ \boldsymbol{kip}$

(4.5) Based on the Vertical Punching Shear in Curb: $P_{p5}$

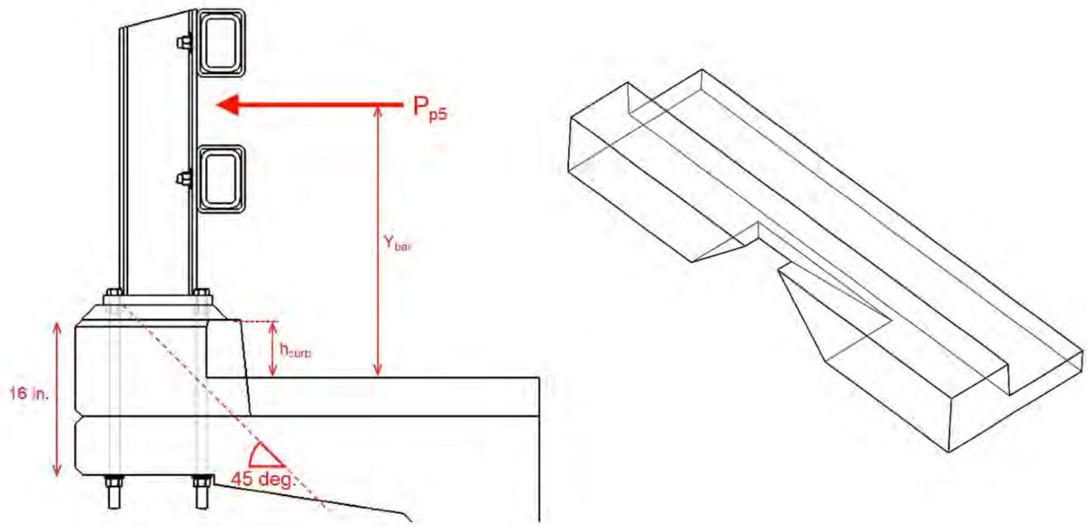



Figure. Punching Shear Failure Mechanism of the Curb

Calculate the area of the failure planes:

Note: Area deduction factor of 0.65 is used to account for concrete spalling along the steel reinforcement in the deck

$Area_{back1} := 592.84\ in^2$ — Back area measured in Solidworks (in.2)

$Area_{side1} := 273.79\ in^2$ — Side area measured in Solidworks (in.2)

$\phi_{punch} := 0.65$ — Reduction factor considering concrete cover

$Area_{total1} := \phi_{punch} \cdot (Area_{back1} + 2 \cdot Area_{side1}) = 741.273\ in^2$ — Total measured area (in.2)

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Calculate the tension strength of concrete

$f_c = (5 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$\phi_c = 0.85$ — Strength reduction factor

$v_c := 2 \cdot \sqrt{\dfrac{f_c}{psi}}\ psi = 141.421\ psi$ — Stress corresponding to shear strength provided by concrete (psi)

Calculate the punching force to fail the curb

$V_{c.punch1} := \phi_c \cdot Area_{total1} \cdot v_c$ — Nominal shear strength provided by concrete (kip)

$V_{c.punch1} = 89.107\ kip$

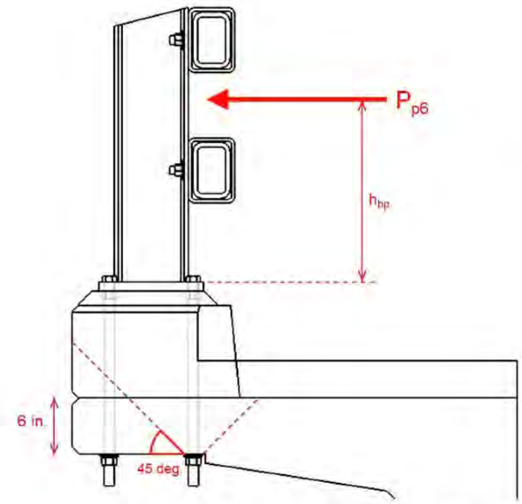
Determine force @ Ybar on post force

$d_{punch1} := 13\ in$ — Estimated moment arm for punching shear (in.)

$P_{p5} := \dfrac{V_{c.punch1} \cdot d_{punch1}}{h_{bp}}$ — Force to fail concrete due to vertivcal punching shear through 6" deck and curb (kip)

$P_{p5} = 57.92\ kip$ — Vertical punching shear strength (kip)

(4.6) Based on the Cone-shaped Punching Shear in Deck: $P_{p6}$



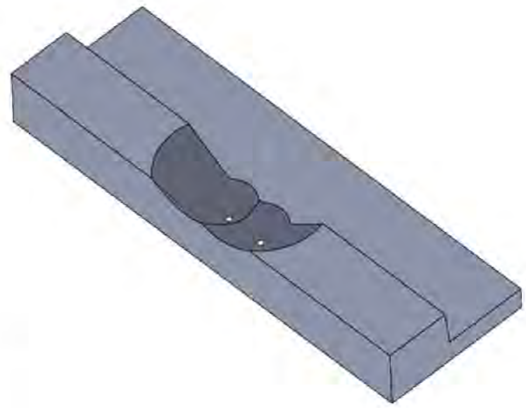



Figure. Cone-Shaped Punching Shear Failure Mechanism in the Deck

Calculate the area of the failure planes:

$Area_{cone} := 521.64\ in^2$ — Cone-shaped concrete area measured in Solidworks (in.2); Note: overlapped area is deducted

$Area_{cone.total} := 2 \cdot Area_{cone} = (1.043 \cdot 10^3)\ in^2$ — Total measured area (in.2)

Calculate the shear contribution by concrete

$f_c = (5 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$v_c := 2 \cdot \sqrt{\frac{f_c}{psi}}\ psi = 141.421\ psi$ — Stress corresponding to shear strength provided by concrete (psi)

$\phi_{cone} := 0.65$ — Reduction factor concrete spalling

$V_{c.cone} := \phi_{cone} \cdot Area_{cone.total} \cdot v_c = 95.902\ kip$ — Nominal shear strength provided by concrete (kip)

Calculate the shear contribution by steel (J-bars)

Reference Table 3 - Hilti RE-500 V3 Epoxy Adhesive Design Strength with concrete bond failure for rebar in uncracked concrete, 2016 Design Guide, page 138

| Rebar size | Effective embedment in. (mm) | Shear — $\phi V_n$ $f'_c$ = 2,500 psi (17.2 MPa) lb (kN) | $f'_c$ = 3,000 psi (20.7 MPa) lb (kN) | $f'_c$ = 4,000 psi (27.6 MPa) lb (kN) | $f'_c$ = 6,000 psi (41.4 MPa) lb (kN) |
|---|---|---|---|---|---|
| #4 | 4-1/2 (114) | 16,035 (71.3) | 17,570 (78.2) | 19,365 (86.1) | 21,430 (95.3) |
| | 6 (152) | 22,960 (102.1) | 24,030 (106.9) | 25,820 (114.9) | 28,575 (127.1) |
| | 10 (254) | 38,265 (170.2) | 40,050 (178.2) | 43,035 (191.4) | 47,625 (211.8) |

$n_{s.cone} := 2$ Number of J-bars penetrating the cone shape

$V_{s.Jbar} := 19.365\ kip$ Shear strength for a single J-bar (kip)

$V_{s.cone} := n_{s.cone} \cdot V_{s.Jbar} = 38.73\ kip$ Nominal shear strength provided by J-bar (kip)

Determine force @ Ybar on post force

$d_{punch1} := 10.125\ in$ Estimated moment arm for cone-shaped punching shear failure; This is a distance between bearing edge and centerline anchor on traffic side (in.)

$$P_{p6} := \frac{(V_{c.cone} + V_{s.cone}) \cdot d_{punch1}}{h_{bp}}$$ Force to fail concrete due to cone-shaped punching shear through 6" deck (kip)

$P_{p6} = 68.158\ kip$ Cone-shaped punching shear strength (kip)

(4.7) Based on flexural resistance of curb (Flexure Critical Section 1): $P_{p7}$

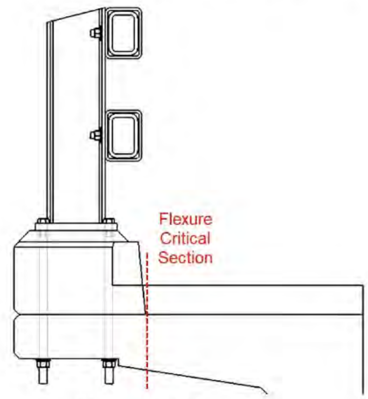



Figure. Failure section

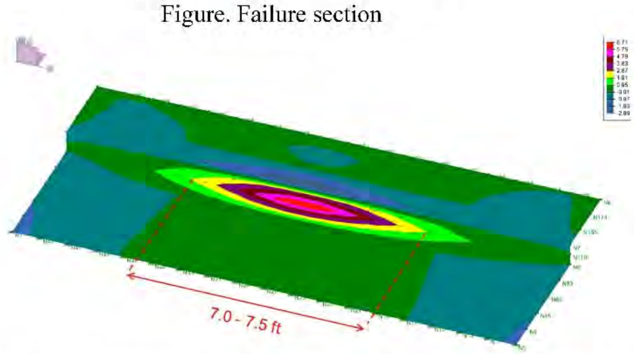



Figure. Estimated force distribution length for Flexure Critical Section from structural analysis program using RISA 3D

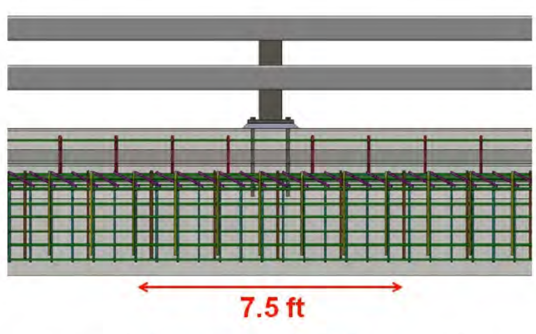



Figure. Reinforcement within Failure Section

Given design data

$f_c = (5 \cdot 10^3)\ psi$ — Concrete strength (ksi)

$f_y = (6 \cdot 10^4)\ psi$ — Yield strength of rebar (ksi)

$b_{FS2} := 7.5\ ft$ — Width of FS2 (in.) Note: Width of FS2 is assumed to be the impact force projected outward at a 45 degree angle to the centroiad of FS2

$h_{FS2} := 7.5\ in$ — Deck thickness at flexure critical section2 (in.)

$Y_{FS2} := Y_{bar} + h_{overlay} + \frac{h_{FS2}}{2} = 35.25\ in$ — Height measured from centroid of FS1 to resultant force of rails (in.)

$n_{FS2} := 13$ — Number of rebar (#5 hook bar) within width of FS2 Bars at the both ends are excluded for conservative design

$A_{FS2} := n_{FS2} \cdot 0.31\ in^2 = 4.03\ in^2$ — Area of tensile reinforcement in FS2 (in2)

Calculate nominal strength and post strength

$d_{FS2} := h_{FS2} - 2.25\ in - \frac{0.625}{2}\ in = 4.938\ in$ — Depth to tension steel from compression face of FS2 (in.)

$a_{FS2} := \frac{A_{FS2} \cdot f_y}{0.85 \cdot f_c \cdot b_{FS2}} = 0.632\ in$ — Depth of Whitney stress block for FS2 (in.)

$M_{FS2} := A_{FS2} \cdot f_y \cdot \left(d_{FS2} - \frac{a_{FS2}}{2}\right) = 93.122\ kip \cdot ft$ — Flexural resistance of the deck (kip-ft)

$P_{p7} := \left(\frac{M_{FS2}}{Y_{FS2}}\right) = 31.701\ kip$ — Post strength based on flexural resistance of deck over the contributing length (kips)

**(5) Summarize post strengths and select limiting post strength for LRFD strength analysis:**

| | |
|---|---|
| $P_{p1} = 60.789\ kip$ | Post strength based on plastic strength of the post |
| $P_{p2} = 60.585\ kip$ | Post strength based on post fillet welds |
| $P_{p3} = 129.885\ kip$ | Post strenth based on anchor bolt shear strength |
| $P_{p4} = 43.259\ kip$ | Post strength based on ultimate tension strength of anchor bolts |
| $P_{p5} = 57.92\ kip$ | Post strength based on the curb punching shear strength |
| $P_{p6} = 68.158\ kip$ | Post strength based on cone-shaped punching failure |
| $P_{p7} = 31.701\ kip$ | Post strength based on flexural resistance |

Therefore use Pp7 as limiting strength.

According to many crash test conducted, deck failure was observed at an open joint, but it was unlikely over 2 or 3 spans. Thus, computing the design strength using Pp7 only may not reflect the actual failure mechanism. Based on the observaion from the crash test, it is considered that averaging the strength of 3 posts (Pp7+ 2Pp4) may be a more realistic failure scenario. Additional calculation is conducted for this perspective.

$$P_p := \frac{2 \cdot P_{p4} + P_{p7}}{3} = 39.407\ kip$$

**(6) Calculate the rail strength for multiple spans @ He height:**

Design forces and designations for TL-4

$H_e = 30\ in$ — Height of equivalent transverse load (in.)

$P_p = 39.407\ kip$ — Governing post strength (kip)

$Y_{bar} = 27.5\ in$ — Average height of combined rail resistances (in.)

$L_{span} = 10\ ft$ — Span length (ft)

$L_t = 5\ ft$ — Length of loading for TL-4 (ft)

$M_p = 105.8\ kip \cdot ft$ — Total resistance of the rails (kip-ft)

** 1 Span Case

$N_1 := 1$ — 1 Span Case

$$R_1 := \frac{16 \cdot M_p + (N_1 - 1) \cdot (N_1 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_1 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_1 = 112.853\ kip$ — Resistance for 1 Span

** 2 Span Case

$N_2 := 2$ — 2 Span Case

$$R_2 := \frac{16 \cdot M_p + N_2^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_2 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_2 = 93.402\ kip$ — Resistance for 2 Spans

** 3 Span Case

$N_3 := 3$ — 3 Span Case

$$R_3 := \frac{16 \cdot M_p + (N_3 - 1) \cdot (N_3 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_3 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_3 = 88.097\ \boldsymbol{kip}$ — Resistance for 3 Span

** 4 Span Case

$N_4 := 4$ — 4 Span Case

$$R_4 := \frac{16 \cdot M_p + N_4^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_4 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-2

$R_4 = 106.638\ \boldsymbol{kip}$ — Resistance for 4 Spans

** 5 Span Case

$N_5 := 5$ — 5 Span Case

$$R_5 := \frac{16 \cdot M_p + (N_5 - 1) \cdot (N_5 + 1) \cdot P_p \cdot L_{span}}{2 \cdot N_5 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_5 = 117.373\ \boldsymbol{kip}$ — Resistance for 3 Span

** 6 Span Case

$N_6 := 6$ — 5 Span Case

$$R_6 := \frac{16 \cdot M_p + N_6^{\ 2} \cdot P_p \cdot L_{span}}{2 \cdot N_6 \cdot L_{span} - L_t}$$

Total ultimate resistance (kip)
Eq. A13.3.2-1

$R_6 = 138.08\ \boldsymbol{kip}$ — Resistance for 3 Span

**(7) Total Ultimate Resistance (Nominal Resistance) of Railing:** $R_R$

| | |
|---|---|
| $R_r := min\left(R_1, R_2, R_3, R_4, R_5, R_6\right) = 88.097\ \boldsymbol{kip}$ | Total ultimate resistance of the bridge rail system @ ybar (kip) |
| $H_{e.TL4} := 30\ \boldsymbol{in}$ | Height of transverse impact load (in.); MASH TL-4 |
| $Y_{bar} = 27.5\ \boldsymbol{in}$ | Height of resultant force (in.) |
| $F_{t.TL4} := 80\ \boldsymbol{kip}$ | Transverse impact force (in.); MASH TL-4 |
| $R_R := R_r \cdot \left(\frac{Y_{bar}}{H_{e.TL4}}\right) = 80.756\ \boldsymbol{kip}$ | Total ultimate resistance of the bridge rail system @He (kip) |

$$Check\_Retrofit\_Design := \begin{Vmatrix} \text{if } R_R \geq F_{t.TL4} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{Vmatrix} = \text{"OK"}$$

**Summary:**

The strength of the retrofit design is approximately 80.8 kips at the TL-4 height of 30 in. and this is slightly greater than the target strength of 80 kips. Thus, the design does meet MASH TL-4.