13. RAILINGS

Section 13 of the LRFD Specifications addresses the design of bridge railings. "Railings" is used as a generic term in the specifications and includes traffic railings, combination railings, bicycle railings, and pedestrian railings. MnDOT further classifies railings as follows:

- Barriers – Concrete railings that have a non-vertical traffic face (e.g. - Type S).
- Parapets – Concrete railings that have a vertical traffic face and a brush curb (e.g. - Type P-4).
- Metal Railings – Steel Railings mounted on curbs, parapets, or the back of barriers (e.g. - Ornamental Metal Railing, Design T-4).

The design requirements for railings utilized on MnDOT bridges have undergone changes in recent years as the Federal Highway Administration (FHWA) established crash-testing requirements and the AASHTO Specifications were revised accordingly. Additionally, the desire for more attractive railings has influenced the style of railings on projects where aesthetics is a major consideration. Accidents involving objects thrown from overpasses onto traffic below has led to the adoption of protective screening requirements. The rapid increase in bicycle trails and traffic has increased attention on bicycle railings. This section of the LRFD Bridge Design Manual details our policies regarding the design of bridge railings for MnDOT projects.

13.1 Materials

Reinforced concrete, steel, and timber are all used for railings. The majority of traffic railings are reinforced concrete. Bridges with timber decks on low volume secondary roads may have timber railings. Pedestrian and bicycle railings are typically galvanized steel that has been painted for aesthetics.

13.2 Design Requirements

The design of newly constructed bridge railings must conform to the requirements of Section 13 of the *AASHTO LRFD Bridge Design Specifications*. This specification gives geometric and strength requirements and also describes crash test levels. FHWA requires all bridges carrying traffic on the National Highway System (NHS) to be crash tested in accordance with *NCHRP Report 350 Recommended Procedures for the Safety Performance Evaluation of Highway Features*. There are six levels of service and testing depending on vehicle size and speed.

Crash testing requirements may be waived if the railing in question is similar in geometrics to an approved crash tested rail and an analytical evaluation shows the railing to be crash worthy. This allows minor changes to crash tested railings without having to go through the time and expense of crash testing. For bridges on the NHS, any such evaluation must be approved by the FHWA.

Crash testing has shown that during impact vehicles slide along the top of the railing and parts of the vehicle, especially the boxes on trucks, extend beyond the face of the railing a considerable distance. The envelope of the vehicle encroachment beyond the face of railing is known as the zone of intrusion. Attachments to bridge railings, such as architectural metal railings or objects just behind the railing (such as light poles), must address safety concerns presented by this encroachment, which include:

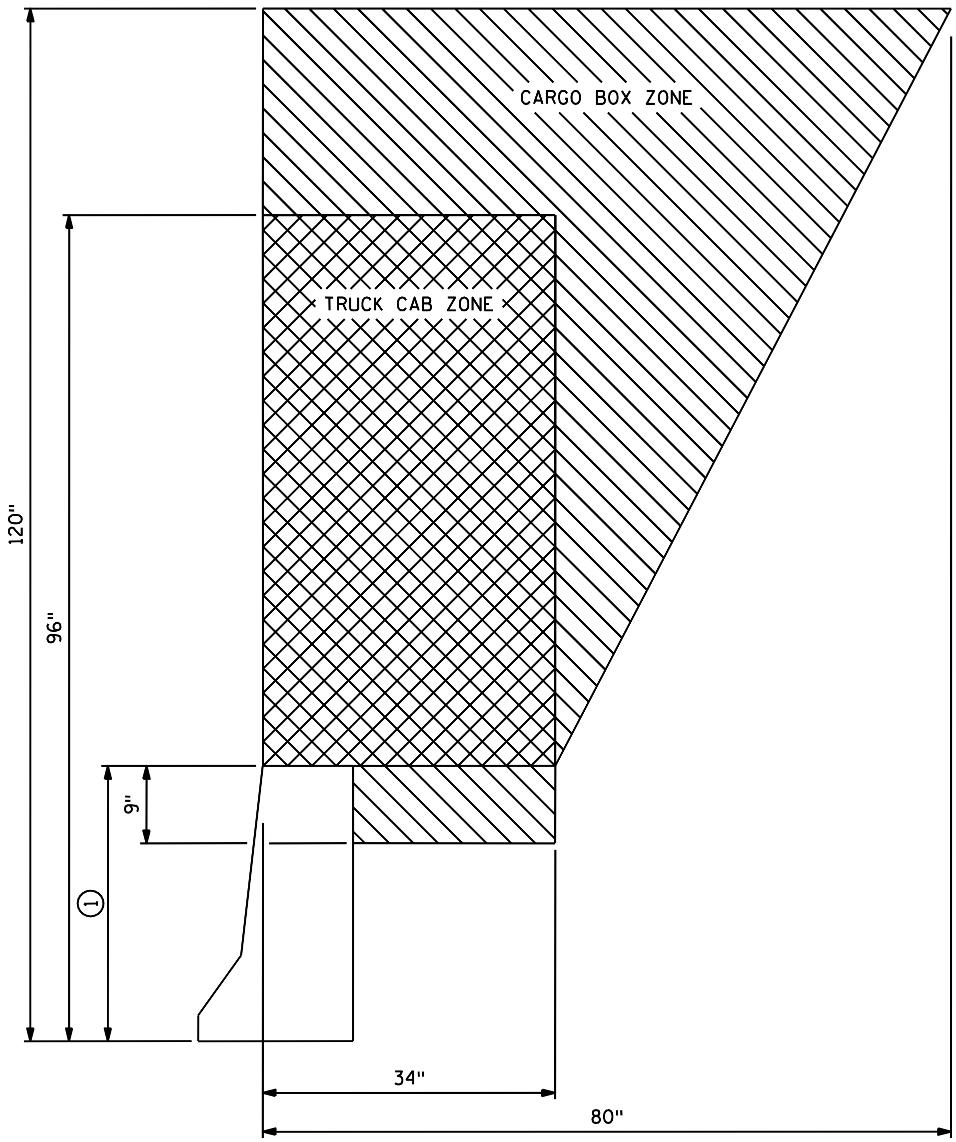
1) Snagging - which can cause the attachment or the vehicle hood to penetrate the occupant compartment.
2) Spearing – objects, such as a horizontal railing member, penetrating windshields and injuring occupants.
3) Debris falling onto traffic below.

A Midwest Roadside Safety Facility report, titled *Guidelines for Attachment to Bridge Rails and Median Barriers*, February 26, 2003, identifies zones of intrusion for several types of railings. Figure 13.2.1 shows the zone of intrusion for a Test Level 4 barrier.

Generally attachments within the zone of intrusion shall be designed to break away before severely damaging the vehicle, contain any debris from damaging traffic below, and have no members (such as rail ends) that might spear the occupant compartment of the vehicle. Ends of rails shall be sloped at 45 degrees or less to top of barrier to reduce the chance of spearing. Posts shall be set back from the face of railing to minimize snagging. (See Sections 13.2.1 and 13.2.3 for setback requirements.)

Railing designs shall include consideration of safety, cost, aesthetics and maintenance. Safety shapes (Types J and F) were developed to minimize damage to vehicles, as well as to contain and redirect vehicles back onto the roadway, and have low initial and maintenance costs. Use of designs that allow for easy replacement of damaged sections and use of standard railings can minimize maintenance costs since replacement components can be stockpiled.

Three general classes of bridge railings are Traffic Railings, Pedestrian or Bicycle Railings, and Combination Railings. Bridge cross sections showing these three classes are shown in Figure 13.2.2. Railing classes are further defined in the following sections. Also, refer to Table 13.2.1 for guidance on standard rail applications.



(1) REVIEWED TL-4 BARRIER HEIGHTS FELL IN A RANGE OF 29" TO 42"

[1] ***Figure 13.2.1***
Intrusion Zones for TL-4 Barriers

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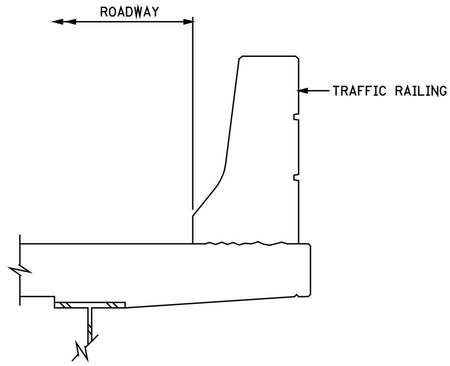
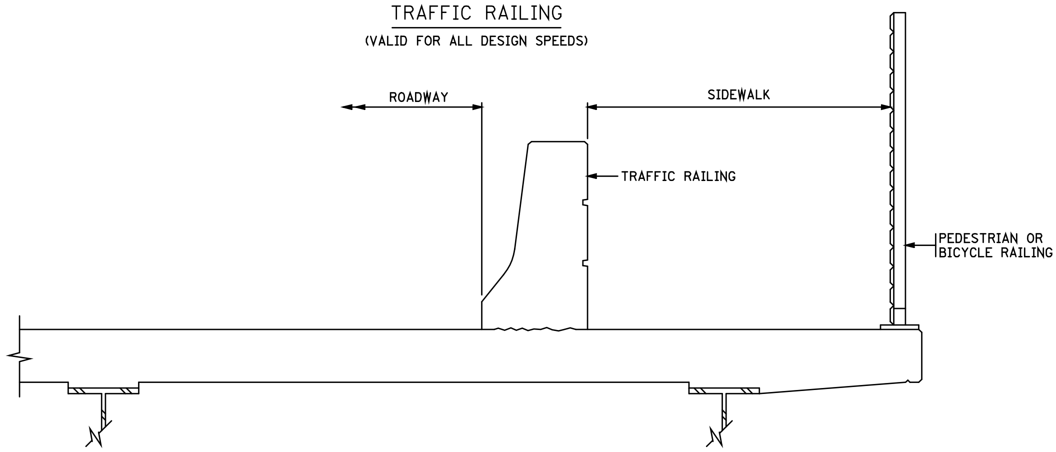
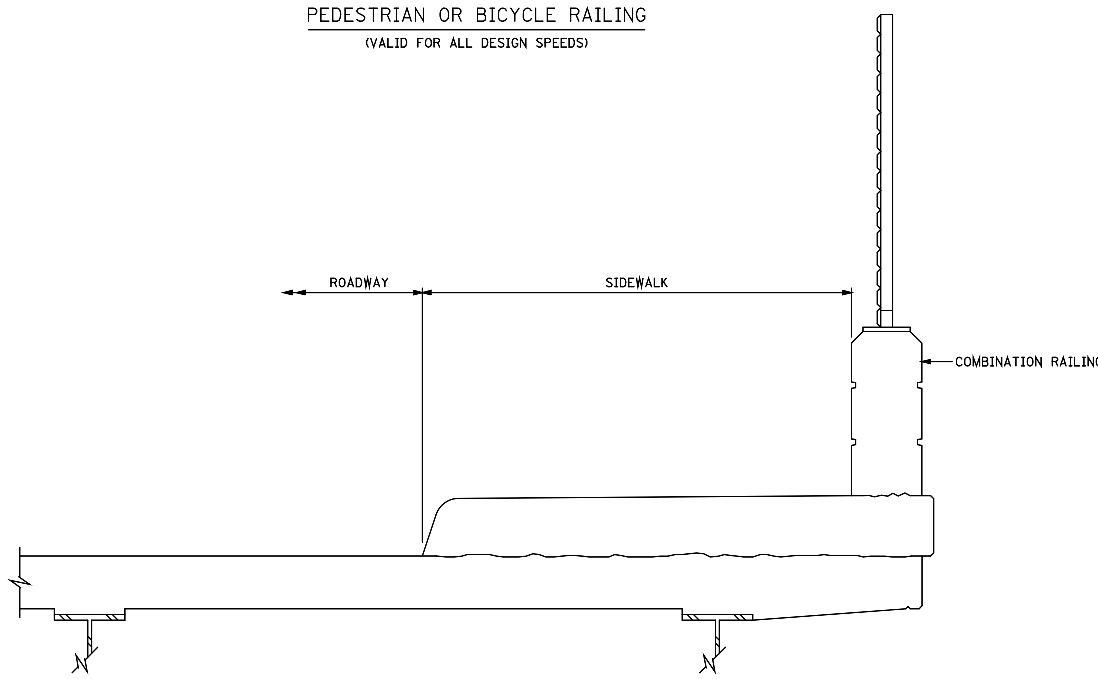
[1] Reproduced from Keller, Sicking, Faller, Polivka & Rhode, *Guidelines for Attachments to Bridge Rails and Median Barriers*, (Midwest Roadside Safety Facility, February 26, 2003), page 24.



TRAFFIC RAILING
(VALID FOR ALL DESIGN SPEEDS)



PEDESTRIAN OR BICYCLE RAILING
(VALID FOR ALL DESIGN SPEEDS)



COMBINATION RAILING
(VALID FOR DESIGN SPEEDS ≤ 40 MPH)

Figure 13.2.2

TABLE 13.2.1: Standard Rail Applications					
Rail Type	**Description**	**Test Level**	**Speed Limit**	**Application**	**Comment**
Traffic	Concrete Barrier (Type F, TL-4) 5-397.114: Separate End Post w/o W.C. 5-397.115: Integral End Post w/o W.C. 5-397.116: Separate End Post w/ W.C. 5-397.117: Integral End Post w/ W.C.	TL-4	All	Traffic Only	2'-8" tall
	Concrete Barrier (Type F, TL-5) 5-397.122: Integral End Post w/ W.C. 5-397.124: Integral End Post w/o W.C.	TL-5	> 40 mph	High Protection Area where $D_c > 5°$ and Speed > 40 mph.	3'-6" tall (Gives added protection to motorists on high speed, high curvature roadways. Modify standard to remove sidewalk.)
	Concrete Barrier (Type F, TL-5) w/ Sidewalk 5-397.125: Integral End Post w/ W.C. 5-397.126: Integral End Post w/o W.C.	TL-5	All	Between sidewalk and roadway where the shoulder is < 6'.	3'-6" tall (The additional height is to protect a bicycle rider from falling over the railing into traffic.)
	Concrete Barrier (Type F, TL-5) 5-397.128: Integral End Post w/ W.C. 5-397.129: Integral End Post w/o W.C	TL-5	All	Bridges with designated bike path or where glare screen is required.	4'-8" tall
	Solid Median Barrier (Type F, TL-4) 5-397.130: w/ W.C.	TL-4	All	Traffic Only	2'-8" tall
	Split Median Barrier (Type F, TL-4) 5-397.131: w/ W.C.	TL-4	All	Bridges with a longitudinal joint between roadways. (Usually the bridge is very wide or is to be constructed in stages.)	2'-8" tall (For stage construction, each half of barrier meets TL-4 standard.)
	Solid Median Barrier and Glare Screen (Type F, TL-4) 5-397.132: w/ W.C.	TL-4	All	Traffic Only	4'-8" tall
	Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.135: w/o W.C. 5-397.136: w/ W.C	TL-4	All	Traffic Only	4'-8" tall
	Offset Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.137: w/ W.C.	TL-4	All	Use where roadways are at different elevations. (Usually on superelevated bridges.)	4'-8" tall (Separation allows both sides to be slipformed.)

TABLE 13.2.1: Standard Rail Applications *(cont.)*

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Traffic	Concrete Barrier (Type P-2, TL-4) and Structural Tube Railing (Type T-1) 5-397.157: w/ Integral End Post	TL-4	All	Traffic Only, where an aesthetic railing is desired.	1'-3" metal railing on 1'-9" parapet (Designer must modify detail for separate end post or no W.C.)
	Concrete Barrier (Type P-4, TL-4) 5-397.173: Integral End Post w/ W.C.	TL-4	All	Traffic Only	2'-8" tall
Combination (Traffic and Ped./Bicycle)	Concrete Barrier (Type P-1, TL-2) and Wire Fence (Design W-1) 5-397.119: Integral End Post 5-397.120: Separate End Post	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	2'-4" parapet and 6' metal rail with chain link fabric.
	Concrete Barrier (Type P-1, TL-2) and Metal Railing for Bikeway (Type M-1) 5-397.154: Integral End Post	TL-2	≤ 40 mph	Outside edge of walk on highway bridges with sidewalks where bicycle traffic on the walk is expected and protective screening is not required.	2'-4" parapet with 2'-2" metal rail (Modify for separate end post.)
	Structural Tube Railing (Design T-2) 5-397.158	TL-4	All	Attachment to Type F rail for use where significant bicycle traffic will be using roadway shoulder.	Top of metal railing 1'-10½" above top of 2'-8" Type F rail (Total height of 4'-6"+ meets bicycle standard.)
	Concrete Barrier (Type P-1, TL-2) and Structural Tube Railing with Fence (Design T-3) 5-397.212	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	2'-4" parapet and 5'-8 ½" metal rail with chain link fabric
Ped./Bicycle	5' Wire Fence (Design W-1) for Pedestrian Bridges 5-397.202	Ped. & Bike	Yes	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	5' tall chain link fence
	8' Wire Fence for Pedestrian Walks 5-397.205	Ped. & Bike	Yes	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	8' tall chain link fence

NOTES:

- Crash testing levels refer to NCHRP Report 350. The structural tube traffic rail (Bridge Details Manual Part II, Fig. 5-397.157) and bicycle rail attachment to Type F rail (Bridge Details Manual Part II, Fig. 5-397.158) were developed by Minnesota and crash tested through the pooled fund program. Combination railings with the 2'-4" parapet have been judged to meet crash Test Level 2 (TL-2) by comparison to other crash tested vertical face railings.
- Railing heights are measured to the finished surface (top of wearing course).
- Information on current costs of these railings may be obtained from the Bridge Estimating Unit.
- Combination railings may also be used as bicycle/pedestrian railings. The 2'-4" parapet height permits a wider spacing of spindles (6" openings rather than the 4" openings required up to 27" above the finished surface).

TABLE 13.2.2: Non-Standard Rail Applications

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Combination (Traffic and Ped./Bicycle)	Cloquet Railing Bridge No. 09008 and 09009	TL-2	≤ 40 mph	Outside edge of walk on highway bridges with sidewalks where bicycle traffic on the walk is expected and protective screening is not required.	2'-2 $^{3}/_{4}$" metal rail on 2'-4" parapet (Sheet is metric.)
	Concrete Barrier (Type P-3, TL-2) and Ornamental Metal Railing (Type M-2)	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	3'-9" metal rail on 2'-4" parapet (Developed by City of Minneapolis for use on bridges in their city.)
	St. Peter Railing Bridge No. 27R05	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	4'-6" metal rail on 2'-4" parapet (Bridge No. 23022 has a 2'-2" height of metal rail for use where protective screening is not needed.)
	TH 100 Corridor Standard Bridge No. 27285	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	3'-9" metal rail on 2'-4" parapet
	TH 212 Corridor Standard Bridge No. 27148	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	5'-8' to 9'-2" metal rail on 2'-4" parapet
	TH 610 Corridor Standard Ornamental Metal Railing Type DWG Bridge No. 27222	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	5'-5$^{1}/_{2}$" metal rail on 2'-4" parapet (Sheet is metric.)
	Victoria Street Railing Bridge No. 62823	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	5'-8" metal rail on 2'-4" parapet with chain link fabric

TABLE 13.2.2: Non-Standard Rail Applications *(cont.)*

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Pedestrian/Bicycle	Gooseberry Falls Suspended Walkway Rail Bridge No. 38010	Ped.	N/A	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	3'-6" tall (Sheet is metric.)
	Lexington Rail Bridge No. 62823	Ped. & Bike	N/A	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	4'-6" tall (Sheet is metric.)
	St. Peter Rail Bridge No. 40002	Ped. & Bike	N/A	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	4'-6" tall (Sheet is metric.)

NOTES:

- Crash testing levels refer to NCHRP Report 350. Combination railings with the 2'-4" parapet have been judged to meet crash Test Level 2 (TL-2) by comparison to other crash tested vertical face railings.
- Railing heights are measured to the finished surface (top of wearing course).
- Information on current costs of these railings may be obtained from the Bridge Estimating Unit.
- Combination railings may also be used as pedestrian/ bicycle railings. The 2'-4" parapet height permits a wider spacing of spindles (6" openings rather than the 4", which is required in the lower 27").

13.2.1 Traffic Railing

Traffic railings are designed to contain and safely redirect vehicles. Requirements based on speed are as follows.

1) High Speed Roadways with a Design Speed > 40 mph

   MnDOT requires crash testing to Test Level 4 as the minimum standard for these roadways. Test Level 4 is run with a small car and a pickup truck at 60 mph and a single unit van truck impacting at 50 mph. This railing will normally be the 32" high Type F barrier (Bridge Details Manual Part II, Figure 5-397.114-117). Where aesthetic needs warrant, the tubular traffic railing (Bridge Details Manual Part II, Figure 5-397.157) is an acceptable alternative that provides an increased viewing opportunity to drivers crossing the bridge. It consists of a structural tube and posts mounted to the top of a 1'-9" high concrete base. Note, however, that the tubular traffic railing has higher initial and maintenance costs than the Type F barrier. Consult the Preliminary Bridge Unit for additional acceptable railings.

   MnDOT has developed a bicycle railing attachment to the Type F barrier for use where the bridge shoulders carry a bicycle route as defined in the MnDOT State Bicycle Transportation System Plan or another recognized authority. This attachment (Bridge Details Manual Part II, Figure 5-397.158) adds height to the railing to protect bicycle riders and has been crash tested to Test Level 4. It has a cable system inside the rail tubes that will contain the rail pieces in the event of an accident. It also uses weakened posts designed to lessen the impact to vehicles in the event of a hit. This railing may be applied to other traffic barriers provided that the same or greater offset distance to the face of metal rail is provided and the post attachment has the same or greater strength. The cable system must be maintained even if there is no traffic below as the cables act to keep the entire rail system intact during a crash.

   The zone of intrusion (see Section 13.2 for definition) shall be kept free of rail attachments or other features unless they have been crash tested or an analytical evaluation has shown them to be crash worthy. Exceptions to this policy include noise walls and safety features such as signs or lights. Note that light poles shall be located behind the back of the barrier. When noise walls are attached, consider using a higher Type F barrier to lessen the risk. The zone of intrusion for a TL-4 railing is shown in Figure 13.2.1.

   A more stringent rail design may be considered on a case-by-case basis for bridges with high design speeds, high truck volume, and curvature or other site-specific safety considerations. Generally a Test Level 5 railing should be considered for these sites. Test Level 5 includes a small car and a pickup truck traveling at 60 mph plus a van-type tractor trailer impacting at 50 mph. As a guide, a 42" high Type F barrier that meets TL-5 requirements is recommended for bridges having a horizontal curvature of 5 degrees and sharper on a roadway where the design speed is 45 mph or higher. The Preliminary Bridge Plans Engineer will designate the rail design on the Preliminary Bridge Plan.

2) Low Speed Roadways with a Design Speed ≤ 40 mph
MnDOT requires crash testing to Test Level 2 as the minimum standard for these roadways. Test Level 2 is run with a small car and pickup truck both impacting at a speed of 45 mph.

Normally these railings will be the same as used for higher speeds, usually the Type F concrete barrier, but with the reduced level required for crash testing more options are available. Consult the Preliminary Bridge Unit for additional acceptable railings.

If the addition of an ornamental metal railing is desired on the top of the traffic railing, a 32" high vertical faced concrete barrier (see Bridge Details Manual Part II, Figure 5-397.173) shall be used rather than the Type F barrier. The vertical face will cause more damage to a vehicle for minor hits but reduces the tendency for the vehicle to climb the face or roll over and will keep the vehicle back from the metal rail. A small 2" wide by 6" high curb is provided at the base to minimize snowplow damage to the barrier. For design speeds of 35 mph and below a metal railing may be used on the top of the concrete barrier with no minimum offset required, as it is unlikely that vehicles will contact the metal portion.[2] With a design speed of 40 mph the front face of the metal railing shall be offset a minimum of 9" from the face of barrier at the top of concrete.[3]

It is strongly recommended that a smooth face be used on the highway side of concrete barriers. Aesthetic treatments on the highway face increase the risk of vehicle snagging. In addition, in this environment the aesthetics treatment will routinely experience vehicle hits, snowplow scrapes, and high exposure to salt. As a result, their performance will be greatly reduced, causing increased maintenance costs.

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[2] Reproduced from Keller, Sicking, Faller, Polivka & Rhode, *Guidelines for Attachments to Bridge Rails and Median Barriers*, (Report dated February 26, 2003), pages 3 and 27.

[3] Reproduced from Keller, Sicking, Faller, Polivka & Rhode, *Guidelines for Attachments to Bridge Rails and Median Barriers*, (Report dated February 26, 2003), page 15 and 16. 9" offset at 40 mph judged acceptable based on 12" offset at 45 mph.

13.2.2 Pedestrian/ Bicycle Railing

Pedestrian or bicycle railings are generally located at the outside edge of a bridge sidewalk and are designed to safely contain pedestrians or bicyclists. AASHTO specifications require pedestrian railings to be at least 3'-6" in height and bicycle railings to be at least 4'-6" in height. The height is measured from the top of walkway to top of the highest horizontal rail component.

Openings between members of a pedestrian railing shall not allow a 4" sphere to pass through the lower 27" of the railing and a 6" sphere should not pass through any openings above 27". This is more restrictive than AASHTO and is intended to prevent small children from slipping through the railing. The International Building Code requires a 4" maximum opening.

13.2.3 Combination Railing

Combination railings are dual purpose railings designed to contain both vehicles and pedestrians or bicycles. These railings are generally located at the outside edge of a bridge sidewalk. A raised sidewalk is used to clearly define the walkway area and keep roadway drainage off the walkway. The sidewalk curb offers some protection to pedestrians from errant vehicles entering the walkway. There is no other barrier between the roadway and the sidewalk. Combination railings are applicable for design speeds of 40 mph and under. MnDOT requires crash testing to Test Level 2 for these railings and the strength and geometrics requirements for bicycle or pedestrian railings also apply.

Combination railings will normally consist of a 2'-4" high concrete parapet with a fence or ornamental metal railing mounted on the top. The concrete parapet serves to contain traffic and has been judged to meet crash Test Level 2. The metal railing must comply with the strength and geometric requirements for bicycle or pedestrian railings. A non-crash tested metal railing may be used on the top of the concrete barrier, as it is unlikely that vehicles will make contact with the metal portion.

For typical applications, the highway face of a concrete parapet shall be relatively smooth for ease of construction (slipforming) and maintenance. Where aesthetic needs warrant it, beveled recesses up to 2" deep may be allowed for inset panels and beveled form liner textures. Concrete posts above the parapet are acceptable but they may not project in front of the parapet.

For design speeds greater than 40 mph, a traffic railing is required between the roadway and sidewalk or bikeway. Use a 32" high Type F barrier for the traffic railing when the shoulder is 6'-0" or greater in width. If the roadway shoulder is less than 6'-0", use a 42" Type F barrier for added protection. Metal railings shall not be placed on top of a traffic railing between a sidewalk and a roadway. Although metal railings may somewhat increase protection for bicyclists, they are a risk hazard to vehicles.

13.2.4 Strength of Standard Concrete Barriers

Barrier resistance values have been determined for the standard MnDOT concrete barriers and are shown in Table 13.2.4.1. They are based on using both near and far face reinforcement as tension reinforcement. These values can be used when analyzing deck overhangs to determine reinforcement requirements. (See Section 9.2.4J for an overhang reinforcement design example.)

TABLE 13.2.4.1: Resistance Values for Standard Concrete Barriers

Description	End Panel		Interior Panel	
	L_c (ft)	R_w (kips)	L_c (ft)	R_w (kips)
Concrete Barrier (Type F, TL-4) 5-397.114: Separate End Post w/o W.C. 5-397.115: Integral End Post w/o W.C.	4.6	59.2	9.9	124.1
Concrete Barrier (Type F, TL-4) 5-397.116: Separate End Post w/ W.C. 5-397.117: Integral End Post w/ W.C.	4.6	57.2	10.2	122.9
Concrete Barrier (Type F, TL-5) 5-397.122: Integral End Post w/ W.C.	9.3	128.5	14.3	128.8
Concrete Barrier (Type F, TL-5) 5-397.124: Integral End Post w/o W.C.	9.2	133.6	14.0	131.4
Concrete Barrier (Type F, TL-5) w/ Sidewalk 5-397.125: Integral End Post w/ W.C.	9.3	128.5	14.3	128.8
Concrete Barrier (Type F, TL-5) w/Sidewalk 5-397.126: Integral End Post w/o W.C.	9.2	133.6	14.0	131.4
Concrete Barrier and Glare Screen (Type F, TL-5) 5-397.128: Integral End Post w/W.C.	9.3	128.5	14.3	128.8
Concrete Barrier and Glare Screen (Type F, TL-5) 5-397.129: Integral End Post w/o W.C.	9.2	133.6	14.0	131.4
Split Median Barrier (Type F, TL-4) 5-397.131: w/ W.C.	4.5	54.0	12.1	91.1
Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.135: w/o W.C.	4.1	55.8	9.0	106.6
Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.136: w/ W.C.	4.2	61.1	9.2	107.5

TABLE 13.2.4.1: Resistance Values for Standard Concrete Barriers				
Description	**End Panel**		**Interior Panel**	
	L_c (ft)	**R_w (kips)**	**L_c (ft)**	**R_w (kips)**
Offset Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.137: w/ W.C.	4.2	61.1	9.2	107.5
Concrete Barrier (Type P-2, TL-4) 5-397.157: w/ Integral End Post	4.6	87.7	9.0	196.7
Concrete Barrier (Type P-4, TL-4) 5-397.173: Integral End Post w/ W.C.	4.6	76.8	9.9	151.7
Concrete Barrier (Type P-1, TL-2) 5-397.119 5-397.120 5-397.154 5-397.212	4.9	50.4	9.2	103.7

13.2.5 Protective Screening

The addition of protective screening to bridge railings is a further MnDOT policy requirement. The practice of adding protective screening is common nationwide in response to accidents and fatalities that have occurred due to pedestrians throwing objects from overpasses onto vehicles below.

Protective screening must be included in the design of new bridges that accommodate pedestrians when the bridge crosses a roadway or railroad, and also when railings are replaced on existing bridges as follows:

- On bridges where a sidewalk is included in the design, incorporate a protective screening system in the design of the railing adjacent to the sidewalk.
- On pedestrian bridges, place the protective screening on both sides of the bridge.

The protective screening system will be, preferably, a chain link fence system or a railing system. The height of the fence or railing shall be 8'-0" above the top of the sidewalk. For sites with special aesthetic treatments involving ornamental railings a minimum height of 6'-0" will be allowed. However, it should be recognized that the lower railing height provides a reduced level of protection. The protective screening system shall not allow objects 6" or greater in diameter to pass through the fence or railing.

13.2.6 Architectural/ Ornamental Railings

In response to local requests, special railing designs have been incorporated in some projects to address aesthetic concerns. These ornamental architectural bridge railings have been utilized in lieu of standard combination railings for placement on the outboard side of bridge sidewalks. The Bridge Office will consider railing designs in addition to our standard railings for such locations and corridors. It is recommended that special railings incorporate features from the standard railings (such as connection details) as significant effort has gone into the development of these details.

MnDOT participation in the cost of aesthetic railings is governed by the following:

Cost Participation and Maintenance Responsibilities with Local Units of Government Manual

(To view the cost participation manual, go to: *MnDOT Policy FM011, Cost Participation for Cooperative Construction Projects and Maintenance Responsibilities Between MnDOT and Local Units of Government*, http://www.dot.state.mn.us/policy/financial/fm011.html and choose the link at the end of the Policy Statement.)

Railings are included with other aesthetic costs of the bridge. MnDOT participation is limited to 5%, 7% or 15% of the cost of a basic bridge, depending on the aesthetic level of the bridge.

Cost participation of architectural/ornamental railings on local bridges is generally funded up to the prorated cost of standard railing or chain link fence. Consult the State-Aid for Local Transportation Office for conditions on bridge funding eligibility.

13.3 Design Examples

Two design examples follow. The first illustrates the design procedures associated with a conventional Type F barrier. The second design example illustrates the steps undertaken for the design of adhesive anchors to support a metal railing.

13.3.1 Type F Barrier Design Example

This example illustrates a design check of the vertical reinforcing steel that ties a standard MnDOT Type F barrier to a concrete deck. The geometry of the barrier and the reinforcing bar sizes and types are illustrated in Bridge Details Part II Fig. 5-397.117. The configuration of the horizontal reinforcing bars in the railing is assumed fixed. The spacing of the vertical reinforcing steel is checked to ensure adequate capacity is provided. The design check uses the method described in LRFD Article A13.3.1.

A. Design Forces and Dimensions

[13.7.3.2]

MnDOT's Type F barrier satisfies the geometric height constraint of a TL-4 barrier and has satisfactorily passed crash testing to such a level. The design forces and dimensional limits for a TL-4 barrier presented in LRFD Table A13.2-1 are repeated below.

Design Forces and Designations	**TL-4 Barrier**
F_t Transverse (kip)	54
F_L Longitudinal (kip)	18
F_V Vertical/Down (kip)	18
L_t and L_L (ft)	3.5
L_V (ft)	18
H_e Minimum Height of Horizontal Loads (in)	32
H Minimum Height of Rail (in)	32

The design is based on yield line analysis methods and has three variables:

- M_b – the flexural capacity of the cap beam (if present)
- M_w – the flexural capacity of the railing about its vertical axis
- M_c – the flexural capacity of the railing about a horizontal axis

LRFD Article 13.1 cautions designers that railings placed on retaining walls or spread footings may require investigation beyond that presented in this example. The governing or controlling yield line mechanism is assumed to form in the railing. If additional mechanisms with potentially lower load capacities are possible, designers should investigate them. The yield line mechanisms vary with rail location. Interior rail regions are assumed to have three yield lines. Two of the yield lines have tension on the inside of the railing and one has tension on the outside of the railing. See Figure 13.3.1.1, reproduced from LRFD Figure CA13.3.1-1.

The assumed failure mechanism at the end of rail sections (near deflection joints, expansion joints, openings, etc.) has one yield line that produces tension on the inside face of the railing. See Figure 13.3.1.2, reproduced from LRFD Figure CA13.3.1-2.

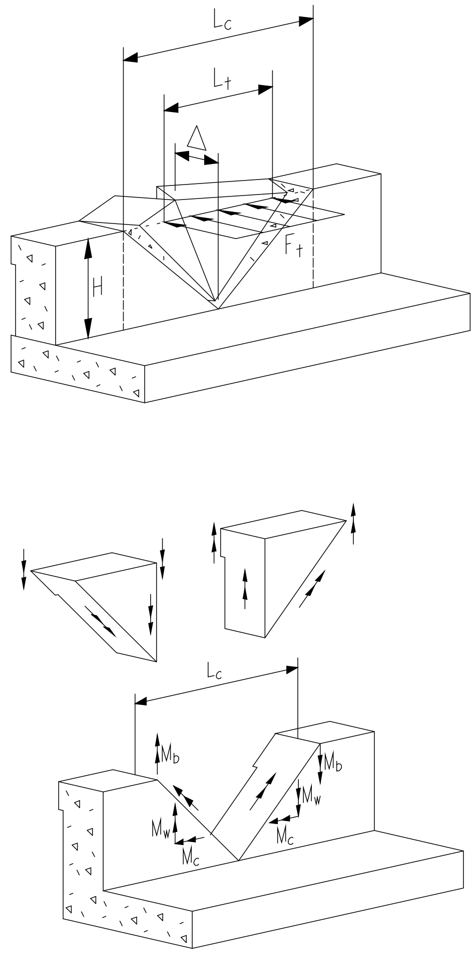


Figure 13.3.1.1
Yield Line Analysis for Interior Region

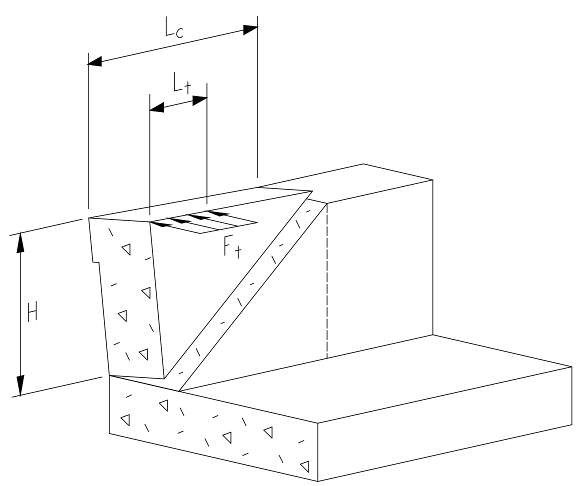


Figure 13.3.1.2
Yield Line Analysis for End Region

Figure 13.3.1.3 contains a rail elevation detail that identifies the location of interior and end regions. The length of end regions and interior regions is dependent on the relative flexural capacities of the railing (M_w and M_c). The design example uses L_{ce} to represent the length of end regions and L_{ci} to represent the length of interior yield line mechanisms. Holding M_w constant, rail sections with larger M_c resistances have shorter and steeper yield line mechanisms.

Designers should note that in addition to inclined yield lines, one-way cantilever resistance of the rail should be investigated for rail segments with lengths less than twice L_{ce}.

B. Barrier Flexural Resistance

Three section details of a Type F barrier are presented in Figure 13.3.1.4. The top section presents typical reinforcement and geometry. The horizontal reinforcement consists of eight #4 bars. Two #5 bars are used for the vertical reinforcement. The R501E bar is anchored in the deck and projects 10" into the rail. The R502E bar is a closed stirrup that laps the R501E bar.

The center detail in Figure 13.3.1.4 labels the horizontal reinforcement and identifies the "d" dimension assumed in M_w calculations. At any one yield line location four bars are assumed to provide flexural resistance and four bars are assumed available to carry shear loads via shear friction.

[CA13.3.1]

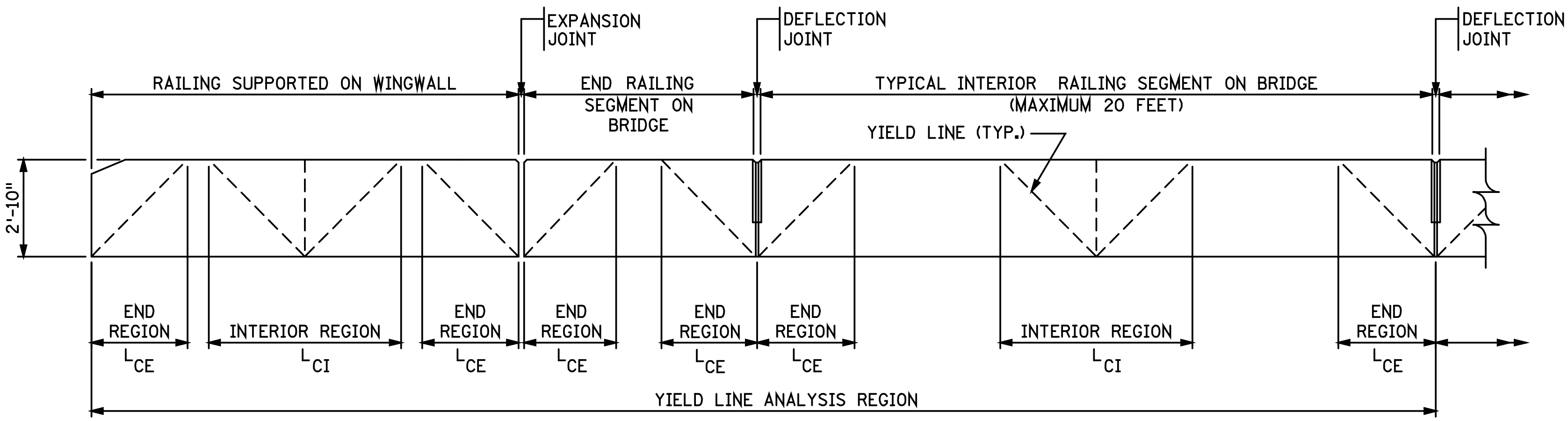
The bottom detail in Figure 13.3.1.4 identifies the "d" dimension of the vertical reinforcement at different locations. These values are averaged to compute M_c.

Determine M_b

The Type F barrier has no additional beam section at its top. Consequently, the M_b term is equal to zero in the rail resistance computations.

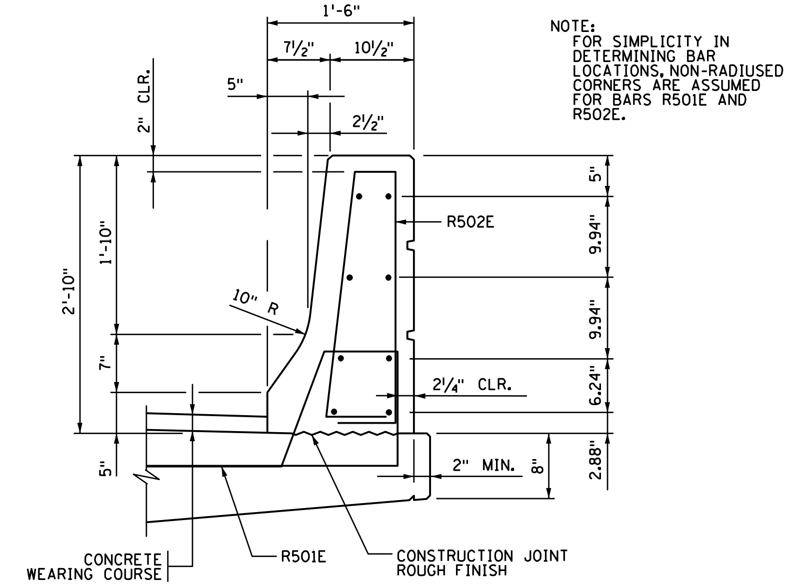
Determine M_w

Using the center detail of Figure 13.3.1.4 the flexural capacity about a vertical axis is computed. Bars 1, 3, 5, and 7 are assumed effective for yield lines that produce tension on the inside face of the rail. Bars 2, 4, 6, and 8 are assumed effective for the case where the yield line has tension on the outside face of the rail.



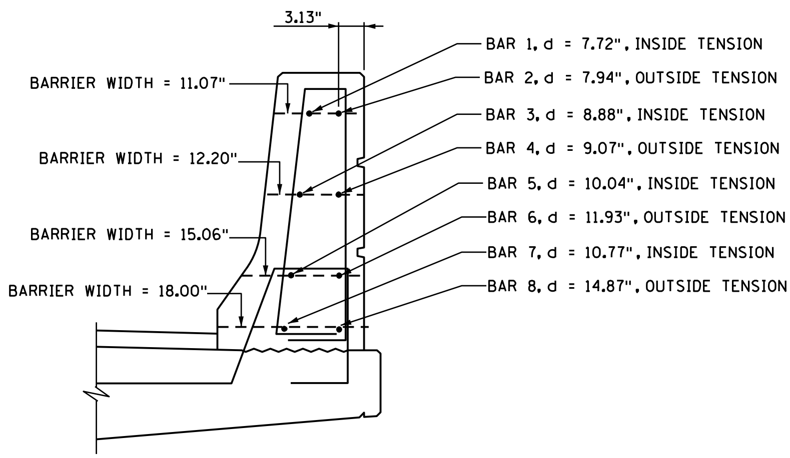
ELEVATION

Figure 13.3.1.3

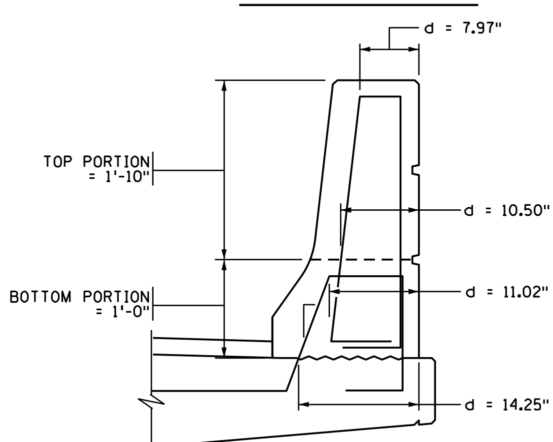


TYPICAL SECTION

DERIVED FROM FIG. 5-397.117



SECTION FOR M_w



SECTION FOR M_c & SHEAR

Figure 13.3.1.4

[5.7.3.2]
[1.3.2.1]

M_w for Interior Region

Capacities φM_n for a typical interior region are listed in the following table. The lever arm dimension of the different bars is found by subtracting half the depth of the flexural compression block.

$$\varphi M_n = \varphi A_s f_y \left(d - \frac{a}{2}\right)$$

$\varphi = 1.0$ (for Extreme Event Limit State)

$A_s = 0.20 \text{ in}^2$

$f_y = 60 \text{ ksi}$

$$a = c\beta_1 = \frac{A_{stotal} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{4 \cdot 0.20 \cdot 60}{0.85 \cdot 4.0 \cdot 34} = 0.42 \text{ in}$$

$$\frac{a}{2} = \frac{0.42}{2} = 0.21 \text{ in}$$

BAR	d (in)	Lever Arm $d - \frac{a}{2}$ (in)	φM_{ni} for Inside Face Tension (k-in)	φM_{no} for Outside Face Tension (k-in)
1	7.72	7.51	90.1	
	7.94	7.73		92.8
3	8.88	8.67	104.0	
4	9.07	8.86		106.3
5	10.04	9.83	118.0	
6	11.93	11.72		140.6
7	10.77	10.56	126.7	
8	14.87	14.66		175.9
		Totals	438.8	515.6

$$M_{wi} = \left(\frac{\varphi M_{ni}}{H}\right) = \left(\frac{438.8/12}{2.83}\right) = 12.92 \text{ kip-ft/ft}$$

$$M_{wo} = \left(\frac{\varphi M_{no}}{H}\right) = \left(\frac{515.6/12}{2.83}\right) = 15.18 \text{ kip-ft/ft}$$

For interior rail regions there is one outside tension yield line and two inside tension yield lines. Compute the average M_w:

$$M_{wint} = \frac{2 \cdot M_{wi} + 1 \cdot M_{wo}}{3} = \frac{2 \cdot 12.92 + 1 \cdot 15.18}{3} = 13.7 \text{ kip-ft/ft}$$

M_w for End Region

At end regions not all of the horizontal bars will be fully developed by the time they intersect with the anticipated yield line. Assume the L_{ce} dimension is at least 4.0 feet. The #4 bars have a development length of 12". Figure 13.3.1.5 shows the reinforcement in the end region of the rail in relation to the assumed yield line.

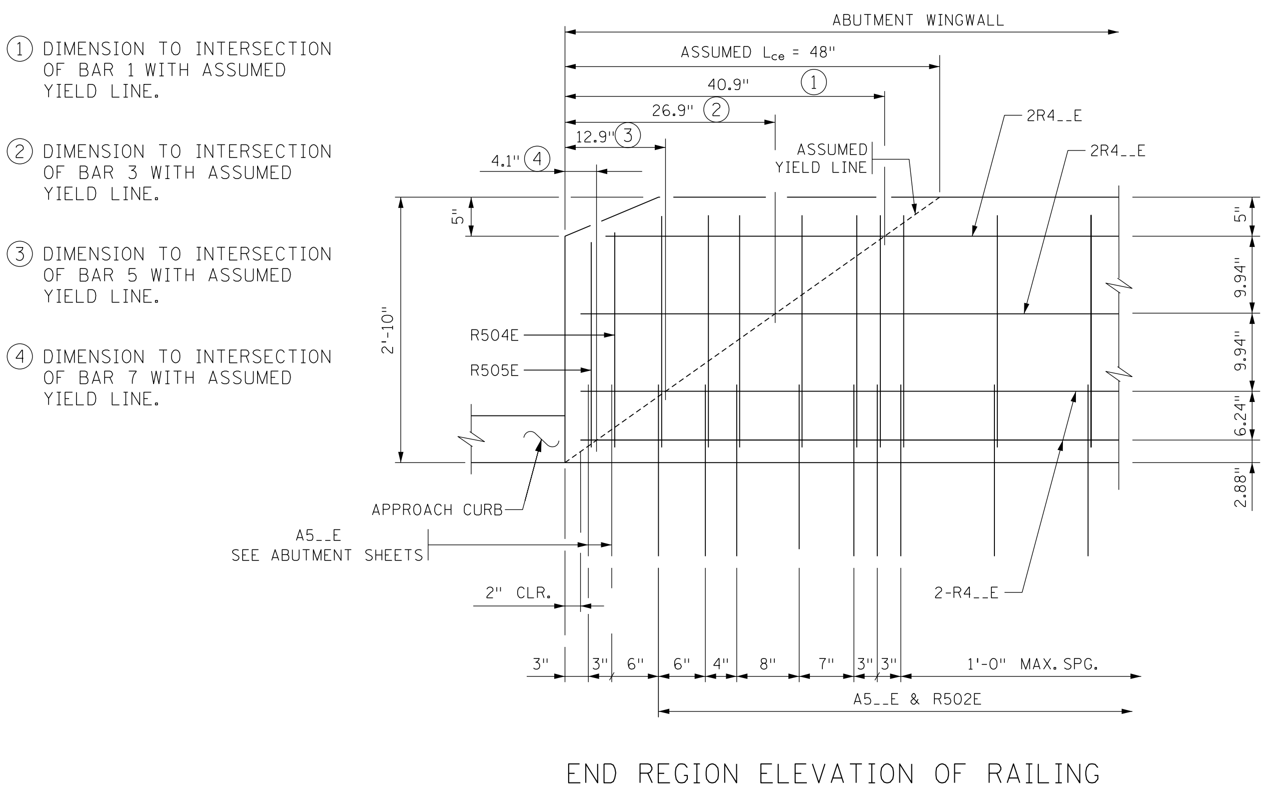


Figure 13.3.1.5

Similar to the interior region, the lever arm is found by subtracting off one half of the depth of the flexural compression block.

$$\varphi M_n = \varphi A_s f_y \left(d - \frac{a}{2}\right)$$

$\varphi = 1.0$ (for Extreme Event Limit State)

$$A_s = 0.20 \text{ in}^2$$

$$f_y = 60 \text{ ksi}$$

$$a = c\beta_1 = \frac{A_{stotal} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4.0 \cdot 34} = 0.32 \text{ in}$$

$$\frac{a}{2} = \frac{0.32}{2} = 0.16 \text{ in}$$

Capacities φM_n for the end region are listed in the following table.

BAR	Embedded Length (in)	Bar Fraction Developed	Developed Bar Area A_s (in²)	d (in)	Lever Arm $d - \frac{a}{2}$ (in)	φM_n for Inside Face Tension (k-in)
1	36	1.00	0.20	7.72	7.56	90.7
3	24.9	1.00	0.20	8.88	8.72	104.6
5	10.9	0.91	0.18	10.04	9.88	106.7
7	2.1	0.18	0.04	10.77	10.61	25.5
		Total	0.62		Total	327.5

M_w is found by averaging the capacity of the rail over the height of the rail.

$$M_{wend} = \left(\frac{\varphi M_n}{H}\right) = \left(\frac{327.5/12}{2.83}\right) = 9.6 \text{ kip-ft/ft}$$

Determine M_c

The Type F barrier does not have a uniform thickness. Consequently the "d" dimension of the vertical reinforcement varies with the vertical location in the rail. Averaged "d" dimensions are used to compute M_c separately for the top and bottom sections. Then a weighted average of the two sections is taken to determine M_c for the entire rail section. Using "d" dimensions labeled in the bottom detail of Figure 13.3.1.4, the average "d" dimensions can be found.

Location	d (in)	Average d (in)
Top	7.97	9.24
Mid Top	10.50	
Mid Bottom	11.02	12.64
Bottom	14.25	

M_c for Interior Region

The internal flexural lever arm is dependent on the amount of reinforcement in the cross section. The maximum spacing of vertical steel in interior regions is 12". Use a 12" vertical steel spacing to evaluate the interior rail region.

For the top portion, $A_{stop} = 0.31 \text{ in}^2/\text{ft}$

$$a_{top} = c\beta_1 = \frac{A_{stop} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.31 \cdot 60}{0.85 \cdot 4.0 \cdot 12.0} = 0.46 \text{ in}$$

$$M_{ctop} = \varphi M_n = \varphi A_{stop} f_y \left(d_{top} - \frac{a_{top}}{2} \right) = 1.0(0.31)(60) \cdot \left(9.24 - \frac{0.46}{2} \right) \cdot \left(\frac{1}{12} \right)$$

$$= 14.0 \text{ kip-ft/ft}$$

For the bottom portion, the R501E bars are not fully developed at the rail/deck interface. Determine bar development fraction:

For a straight #5 bar, the basic development length ℓ_{db} is:

$$\ell_{db} = \frac{1.25 A_b f_y}{\sqrt{f'_c}} = \frac{1.25(0.31)(60)}{\sqrt{4}} = 11.63 \text{ in}$$

or

$$\ell_{db} = 0.4 d_b f_y = 0.4(0.625)(60) = 15.00 \text{ in} \qquad \underline{\text{GOVERNS}}$$

Using modification factors for epoxy coating (1.2) and bar spacing > 6" with > 3" cover (0.8), the straight bar development length is:

$$\ell_{db} = 1.2(0.8)(1500) = 14.40 \text{ in}$$

For a hooked #5 bar, the basic development length ℓ_{hb} is:

$$\ell_{hb} = \frac{38.0 \cdot d_b}{\sqrt{f'_c}} = \frac{38.0(0.625)}{\sqrt{4}} = 11.88 \text{ in}$$

Using modification factors for epoxy coating (1.2) and cover (0.7), the hooked bar development length is:

$$\ell_{dh} = 1.2(0.7)(11.88) = 9.98 \text{ in}$$

Therefore, the benefit derived from the hook is:

$$14.40 - 9.98 = 4.42 \text{ in}$$

The R501E bar is hooked with a vertical embedment of 5.18 in.

Then the development fraction is:

$$F_{dev} = \frac{5.18 + 4.42}{14.40} = 0.67$$

The required extension beyond the 90° bend for a standard hook (A or G dimension) is 10" for a #5 bar. The R501E bar has an extension of 18". Because of this extra extension and the fact that the 18" extension will have to pull through the top mat of reinforcement in order for the bar to fail, assume a higher development fraction $F_{dev} = 0.75$.

Then $A_{sbot} = 0.75(0.31) = 0.23 \text{ in}^2/\text{ft}$

$$a_{bot} = c\beta_1 = \frac{A_{sbot} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.23 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.34 \text{ in in}$$

$$M_{cbot} = \varphi M_n = \varphi A_{sbot} f_y \left(d_{bot} - \frac{a_{bot}}{2}\right)$$

$$= 1.0(0.23)(60)\left(12.64 - \frac{0.34}{2}\right)\left(\frac{1}{12}\right) = 14.3 \text{ kip-ft/ft}$$

$$M_{c\,int} = \frac{14.0(1.83) + 14.3(1.00)}{2.83} = 14.1 \text{ kip-ft/ft}$$

M_c for End Region

The end region has nine A5 and nine R5 bars in the end 4.0 feet of the rail. For the last R5 bar, due to the small amount of bar extending above the yield line, consider only 8 bars to be effective in resisting load.

Then, the average $A_{stop} = 0.62 \text{ in}^2/\text{ft}$

$$a_{top} = c\beta_1 = \frac{A_{stop} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.91 \text{ in}$$

$$M_{ctop} = \varphi M_n = \varphi A_{stop} f_y \cdot \left(d_{top} - \frac{a_{top}}{2}\right) = 1.0\,(0.62)(60) \cdot \left(9.24 - \frac{0.91}{2}\right)\left(\frac{1}{12}\right)$$

$$= 27.2 \text{ kip-ft/ft}$$

The average effective $A_{sbot} = 0.75(0.62) = 0.47 \text{ in}^2/\text{ft}$

$$a_{bot} = c\beta_1 = \frac{A_{sbot} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.47 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.69 \text{ in}$$

$$M_{cbot} = \varphi M_n = \varphi A_{sbot} f_y \cdot \left(d_{bot} - \frac{a_{bot}}{2}\right) = 1.0\,(0.47)(60) \cdot \left(12.64 - \frac{0.69}{2}\right)\left(\frac{1}{12}\right)$$

$$= 28.9 \text{ kip-ft/ft}$$

Then $M_{cend} = \dfrac{27.2(1.83) + 28.9(1.00)}{2.83} = 27.8 \text{ kip-ft/ft}$

C. Flexural Capacity Check

With M_w and M_c computed for an interior and end region, the resistance of the railing can be computed with the equations in LRFD Article A13.3.1.

Check the Capacity of an Interior Region

With $M_{b\,int} = 0$, $M_{w\,int} = 13.7$ kip-ft/ft and $M_{c\,int} = 14.1$ kip-ft/ft, the length of the yield line mechanism and the resistance of the mechanism can be found:

[Eqn A13.3.1-1]

$$L_{ci} = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \left[\frac{8 \cdot H \cdot (M_{bint} + M_{wint} \cdot H)}{M_{cint}}\right]} = 9.8 \text{ ft}$$

[Eqn A13.3.1-2]

$$R_{wi} = \left(\frac{2}{2 \cdot L_{ci} - L_t}\right)\left(8 \cdot M_{bint} + 8 \cdot M_{wint} \cdot H + \frac{M_{cint} \cdot L_{ci}^2}{H}\right) = 98.0 \text{ kips}$$

which, is greater than the 54 kip extreme event design load.

Check the Capacity of the End Region

With $M_{bend} = 0$, $M_{wend} = 9.6$ kip-ft/ft and $M_{cend} = 27.8$ kip-ft/ft, the length of the yield line mechanism and the resistance of the mechanism can be found:

Eqn A13.3.1-4

$$L_{ce} = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H \cdot \left(\frac{M_{bend} + M_{wend} \cdot H}{M_{cend}}\right)} = 4.2 \text{ ft}$$

[Eqn A13.3.1-3]

$$R_{we} = \left(\frac{2}{2 \cdot L_{ce} - L_t}\right)\left(M_{bend} + M_{wend} \cdot H + \frac{M_{cend} \cdot L_{ce}^2}{H}\right) = 81.8 \text{ kips}$$

which, is also greater than the required load capacity of 54 kips.

The other end regions are to be checked similarly.

D. Shear Capacity Check

Use shear friction methods to evaluate the shear capacity of the joint between the deck and railing. Assume that F_t and F_L occur simultaneously.

The resultant shear force is:

$$V_{res} = \sqrt{F_t^2 + F_L^2} = \sqrt{54^2 + 18^2} = 56.9 \text{ kips}$$

[5.7.4]

The basic shear capacity equation for a section using shear friction is:

$$\varphi V_n = \varphi \cdot [c \cdot A_{cv} + \mu (A_{vf} \cdot f_y + P_c)]$$

Neglect cohesion and the small permanent compression across the interface due to selfweight. Conservatively assume that the interface between the railing and the deck is not roughened. The appropriate friction factor, μ, is 0.60.

Substitute V_{res} for φV_n rearranging the remaining terms, and solve for the required area of reinforcement:

$$A_{vfreq} = \left(\frac{V_{res}}{\varphi_v \cdot \mu \cdot f_y}\right) = \left(\frac{56.9}{1.0 \cdot 0.60 \cdot 60}\right) = 1.58\ \text{in}^2$$

The required number of #5 bar legs is:

$$\left(\frac{A_{vfreq}}{A_b}\right)\left(\frac{1.58}{0.31}\right) = 5.1\ \text{legs}$$

Check the interior region first. Assuming the #5 bars are at the maximum spacing of 12" and the L_{ci} dimension is 9.9 feet, 10 bars will be provided.

At the end region, nine #5 bars are provided in the end 4.2 feet (L_{ce}). Both interior and end regions have adequate shear capacity at the deck railing interface.

E. Summary

When checked in accordance with the procedure shown within this example, the capacity of the end regions adjacent to the expansion joint and deflection joints did not meet the required 54 kip load capacity.

Because the neutral axis is located very close to the outside face of the rail for determination of both M_w and M_c, all of the regions were reanalyzed to take advantage of the additional capacity provided by the outside face reinforcement. Therefore, in the second analysis, both the inside face rail reinforcement and the outside face rail reinforcement were included in the determination of the rail capacity. The revised values for the F-rail are:

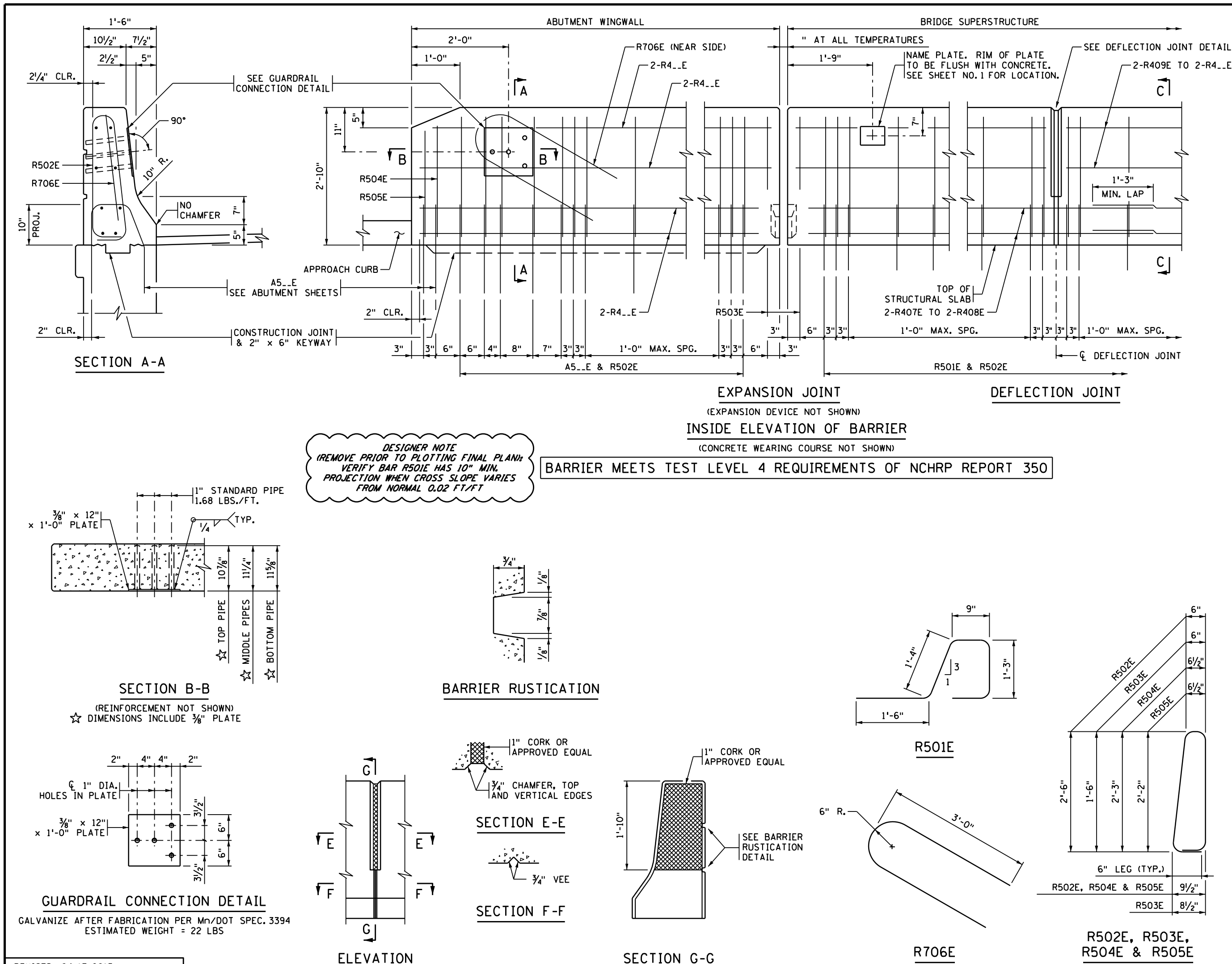
Interior Region:

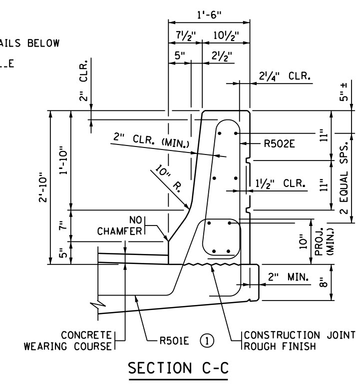
With wearing course	Without wearing course
$L_{ci} = 10.2$ ft	$L_{ci} = 9.9$ ft
$R_{wi} = 122.9$ kip	$R_{wi} = 124.1$ kip

End Region:

With wearing course	Without wearing course
$L_{ce} = 4.6$ ft	$L_{ce} = 4.6$ ft
$R_{we} = 57.2$ kip	$R_{we} = 59.2$ kip

Adequate vehicle collision load capacity is provided with the default reinforcing provided in Bridge Details Part II Figure 5-397.117. (See Figure 13.3.1.6.)





BILL OF REINFORCEMENT FOR BARRIER				
BAR	NO.	LENGTH	SHAPE	LOCATION
R501E		5'-5"		BARRIER DOWEL
R502E		6'-7"		BARRIER VERTICAL
R503E		4'-7"		BARRIER VERTICAL
R504E		6'-1"		BARRIER VERTICAL
R505E		5'-11"		BARRIER VERTICAL
R706E		6'-6"		BARRIER VERTICAL
R407E				BARRIER LONGIT.
R408E				BARRIER LONGIT.
R409E				BARRIER LONGIT.
R4_ _E				BARRIER LONGIT.
R4_ _E				BARRIER LONGIT.
R4_ _E				BARRIER LONGIT.
R4_ _E				BARRIER LONGIT.

GENERAL NOTES

LENGTH OF "TYPE F (TL-4) RAILING CONCRETE (3Y46 OR 3Y46A)" FOR PAYMENT SHALL BE MEASURED BETWEEN THE OUTSIDE FACES OF THE CONCRETE BARRIER.

CONCRETE BARRIER = 477 LBS./FT. (0.117 CU. YDS./FT.)

FINISH ALL EDGES OF BARRIER WITH 1/2" VEE, EXCEPT WHERE OTHERWISE NOTED.

MAXIMUM SPACING OF CONCRETE DEFLECTION JOINTS SHALL BE 20 FT.

SEE SUPERSTRUCTURE SHEET FOR JOINT SPACING.

GUARDRAIL CONNECTION TO BE STRUCTURAL STEEL, Mn/DOT SPEC. 3306.

GUARDRAIL CONNECTION, CORK, AND NAME PLATE TO BE CONSIDERED INCIDENTAL TO "TYPE F (TL-4) RAILING CONCRETE (3Y46 OR 3Y46A)".

BARRIER QUANTITIES ARE LISTED IN SUMMARY OF QUANTITIES FOR SUPERSTRUCTURE.

① PLACE BAR ON TOP OF BOTTOM REINFORCEMENT MAT.

FIG. 5-397.117

REVISED: 04-17-2013
APPROVED: DECEMBER 18, 2003
STATE BRIDGE ENGINEER

CERTIFIED BY ______ LICENSED PROFESSIONAL ENGINEER ______ DATE
NAME: ______ LIC. NO.

TITLE: CONCRETE BARRIER (TYPE F, TL-4)
WITH INTEGRAL END POST
(WITH CONCRETE WEARING COURSE)

DES: DR: CHK: CHK: APPROVED: BRIDGE NO.
SHEET NO. OF SHEETS

Figure 13.3.1.6

13.3.2 Railing Post and Adhesive Anchor Design Example

This example illustrates the design of a railing post, base plate, and the adhesive anchors that secure the post and base plate to a concrete curb. The railing under consideration is MnDOT Standard 5-397.163 "Ornamental Metal Railing (Design T-4 Curb Mount)", mounted on a 6 inch curb per MnDOT Standard 5-397.167 "Concrete Curb For Use With Ornamental Railing".

Figure 13.3.2.1 shows a typical post detail for the railing. The T-4 curb mount railing has three nominal height options. In this example, a nominal height of 6'-0" will be evaluated. The maximum distance L between rail posts is 10'-0".

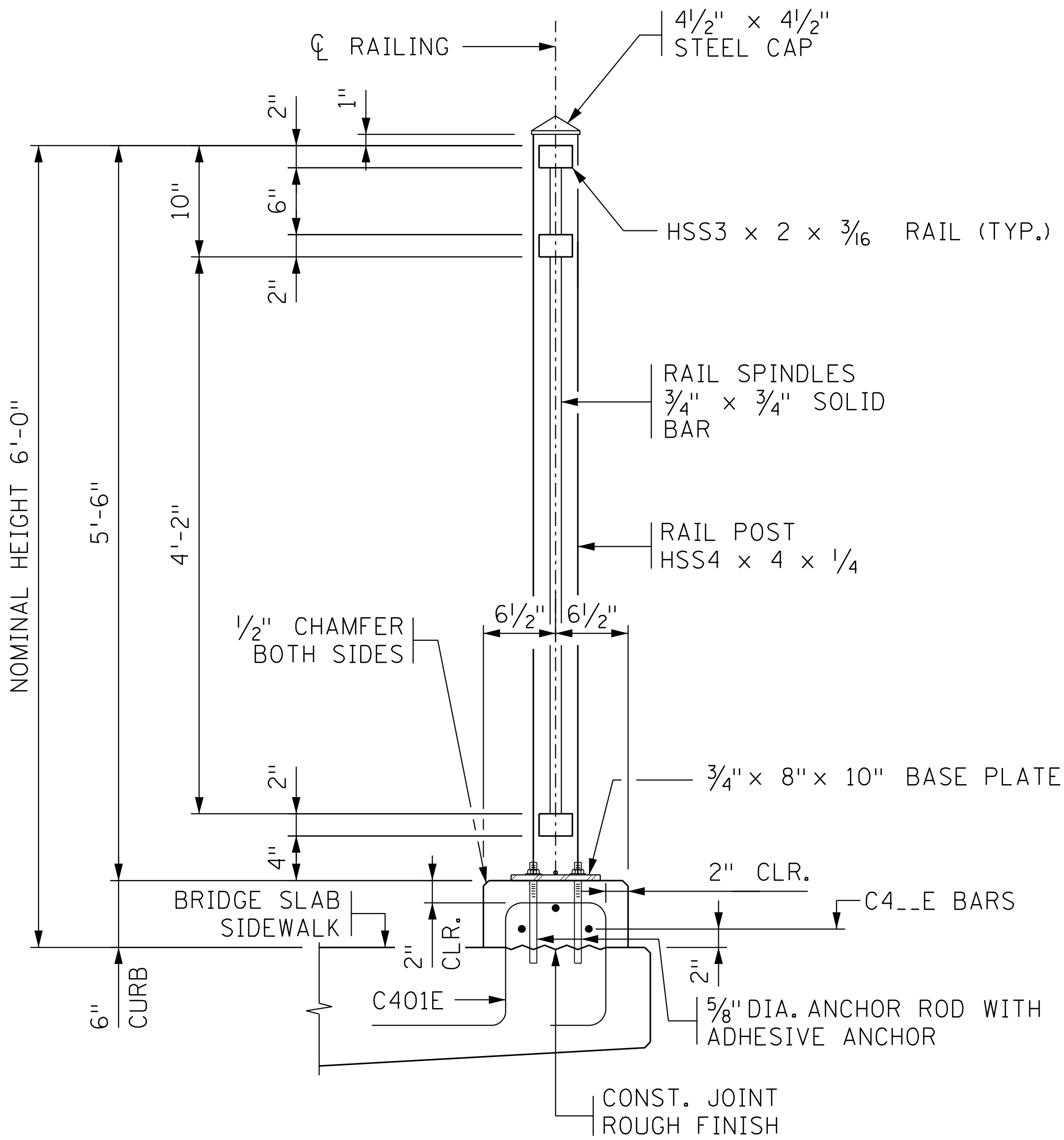


Figure 13.3.2.1

The standard anchorage elements beneath each vertical post consist of four $^{5}/_{8}$" anchor rods attached to the curb by an adhesive bonding agent.

The structural tube used for the rail post and cross members is ASTM A500, Grade B, with a yield strength, F_y, of 46 ksi. The anchor rods are MnDOT 3385, Type A, with a yield strength of 36 ksi and a tensile strength of 58 ksi. All other steel components are MnDOT 3306 with a yield strength of 36 ksi. The concrete used for the parapet has a design compressive strength, f'_c, of 4 ksi.

The example is structured in a top-down fashion. After determining the design loads, the rail post and base plate are checked. Then the shear and tensile capacity of the anchors is computed. For each of the forces, the resistance of steel, concrete, and adhesive bond is determined individually. Steel shear and tensile capacities are based on AASHTO LRFD Article 6.13.2. The equations for concrete shear capacity, concrete tensile capacity, and adhesive bond capacity, as well as modification factors presented in this example are based on AASHTO LRFD Article 5.13 and *ACI 318, Chapter 17*.

A. Design Loads [13.8.2]

Article 13.8.2 of the LRFD Specifications lists the loads to consider for the design of rail elements and posts for pedestrian and bicycle railings. Rail posts are designed to resist a concentrated live load, P_{LL}, applied transversely (inward or outward) at the center of gravity of the top rail element. However, when the center of gravity of the top rail is greater than 5′-0″ above the top surface of the sidewalk, P_{LL} is to be applied at 5′-0″ above the top surface of the sidewalk.

$$P_{LL} = 0.20 + 0.050 \cdot L = 0.20 + 0.050 \cdot 10 = 0.70 \text{ kips}$$

[Table 3.4.1-1]

Using a load factor of 1.75 for live load results in a design horizontal force, H_u, of:

$$H_u = 1.75 \cdot P_{LL} = 1.75 \cdot 0.70 = 1.23 \text{ kips}$$

The location, d_{cg}, of the center of gravity of the top rail above the top of the sidewalk is:

$$d_{cg} = (6 + 66 - 0.5 \cdot 2) / 12 = 5.92 \text{ ft} > 5.0 \text{ ft}$$

Then apply the design horizontal force at 5′-0″ above the top surface of the sidewalk. The moment arm, d, is:

$$d = 5.0 \cdot 12 - 6 = 54.0 \text{ in}$$

The design moment, M_u, is:

$$M_u = H_u \cdot d = 1.23 \cdot 54 = 66.4 \text{ kip-in}$$

B. Rail post Design Check [6.12.2.2.2]

Check the flexural resistance of the rail post, which is an HSS 4 x 4 x ¼. The nominal flexural resistance of an HSS section is dependent on the compression flange slenderness and the web slenderness.

The compression flange slenderness is:

[AISC Steel Constr. Manual Table 1-12]

$$\lambda_f = \frac{b_{fc}}{t_{fc}} = 14.2$$

$$\lambda_{pf} = 1.12 \cdot \sqrt{\frac{E}{F_y}} = 1.12 \cdot \sqrt{\frac{29000}{46}} = 28.1 > 14.2$$

Therefore, flange local buckling does not need to be considered.

The web slenderness is:

[AISC Steel Constr. Manual Table 1-12]

$$\frac{D}{t_w} = 14.2$$

$$\lambda_{pw} = 2.42 \cdot \sqrt{\frac{E}{F_y}} = 2.42 \cdot \sqrt{\frac{29000}{46}} = 60.8 > 14.2$$

Therefore, web local buckling does not need to be considered.

[6.5.4.2] For steel elements in flexure, $\phi_f = 1.00$.

[AISC Steel Constr. Manual Table 1-12] For an HSS 4 x 4 x ¼, $Z = 4.69 \text{ in}^3$.

Then:

[6.12.2.2.2]

$$\phi_f \cdot M_{npost} = \phi_f \cdot F_y \cdot Z$$
$$= 1.00 \cdot 46 \cdot 4.69 = 215.7 \text{ kip-in} > 66.4 \text{ kip-in} \qquad \text{OK}$$

[6.12.1.2.3] Now check the shear resistance of the rail post.

[6.10.9.2] The HSS is unstiffened, so k=5.

[AISC Steel Constr. Manual Table 1-12]

$$\frac{D}{t_w} = 14.2$$

[6.10.9.3.2]

$$1.12 \cdot \sqrt{\frac{E \cdot k}{F_y}} = 1.12 \cdot \sqrt{\frac{29000 \cdot 5}{46}} = 62.9 > 14.2$$

Therefore, C = 1.0

$t_w = 0.233$ in, so $D = t_w \cdot 14.2 = 0.233 \cdot 14.2 = 3.309$ in

[6.10.9.2]

Both webs are effective in resisting shear, so:

$$V_{ppost} = 0.58 \cdot F_y \cdot D \cdot 2 \cdot t_w = 0.58 \cdot 46 \cdot 3.309 \cdot 2 \cdot 0.233 = 41.1 \text{ kips}$$

[6.5.4.2]

For steel elements in shear, $\phi_v = 1.00$.

$$\phi_v \cdot C \cdot V_{ppost} = 1.00 \cdot 1.0 \cdot 41.1 = 41.1 \text{ kips} > 1.23 \text{ kips} \qquad \text{OK}$$

C. Base Plate Design Check

A plan view of the base plate is shown in Figure 13.3.2.2.

For base plates subject to bending moments, *AISC Steel Design Guide 1: Base Plate and Anchor Rod Design* suggests that designers assume a rectangular compressive bearing stress below the base plate. Refer to Figure 13.3.2.3.

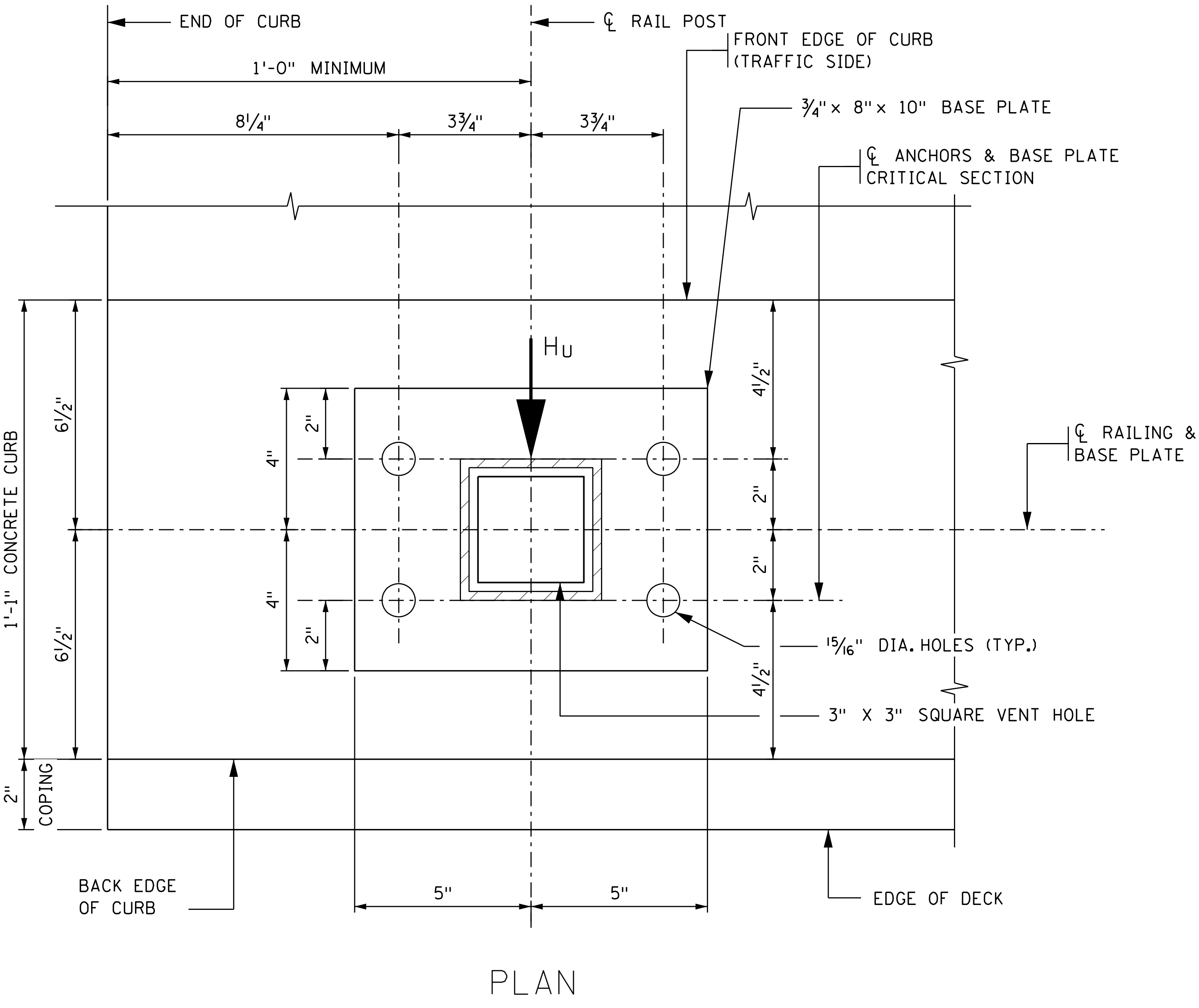
To determine the pressure distribution on the base plate, start by summing moments about the front anchor rods:

$$M_u - C \cdot \left(d - \frac{a}{2}\right) = 0$$

For concrete, $C = 0.85 \cdot f'_c \cdot L_{bp} \cdot a$

where: L_{bp} = the length of the base plate measured along the curb = 10 in

Figure 13.3.2.2



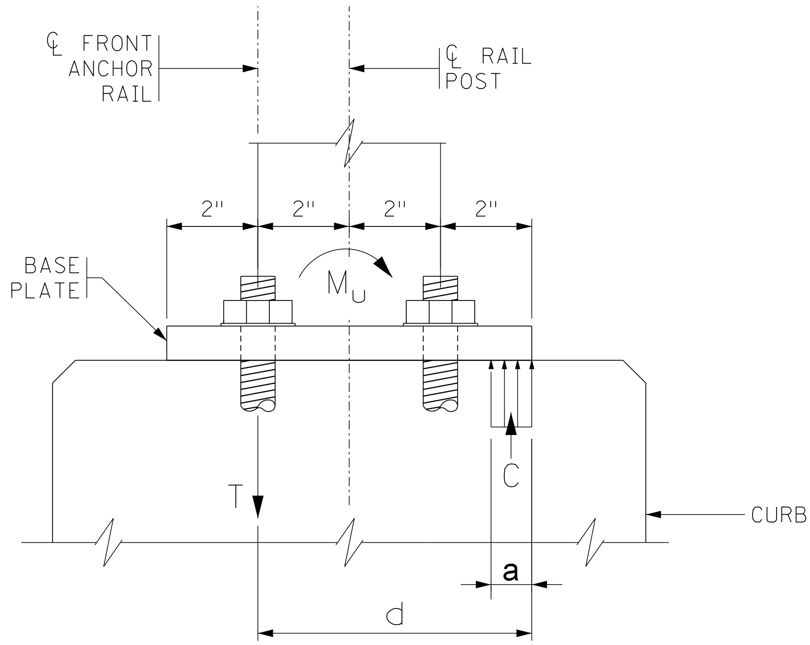


Figure 13.3.2.3

Substitute and solve the resulting quadratic for length of rectangular bearing strength a:

$$M_u - 0.85 \cdot f'_c \cdot L_{bp} \cdot a \cdot \left(d - \frac{a}{2}\right) = 0$$

$$66.4 - 0.85 \cdot 4 \cdot 10 \cdot a \cdot \left(6 - \frac{a}{2}\right) = 0$$

$$17 \cdot a^2 - 204 \cdot a + 66.4 = 0$$

$$a = \frac{204 - \sqrt{(-204)^2 - (4 \cdot 17 \cdot 66.4)}}{2 \cdot 17} = 0.33 \text{ in}$$

Then:

$$C = 0.85 \cdot 4 \cdot 10 \cdot 0.33 = 11.22 \text{ kips}$$

The critical section for bending is at the face of the post. The length of the base plate cantilever in bending, L_{bpcant}, is 2 inches.

Then the factored moment applied to the base plate, M_{ubp}, is:

$$M_{ubp} = C \cdot \left(L_{bpcant} - \frac{a}{2}\right) = 11.22 \cdot \left(2 - \frac{0.33}{2}\right) = 20.6 \text{ kip-in}$$

For design of base plates, MnDOT limits the nominal resistance to the elastic moment. The resisting moment at the face of the post is the capacity of the plate minus two anchor bolt holes.

$$S_{bp} = \frac{\left(L_{bp} - 2 \cdot d_{hole}\right) \cdot t_{bp}^2}{6} = \frac{(10 - 2 \cdot 0.94) \cdot 0.75^2}{6} = 0.76 \text{ in}^3$$

$$M_{rbp} = \phi_f \cdot S_{bp} \cdot F_y = 1.0 \cdot 0.76 \cdot 36 = 27.4 \text{ kip-in} > 20.6 \text{ kip-in} \qquad \text{OK}$$

D. Rail Post to Base Plate Weld Design

Use a fillet weld to connect the rail post to the base plate. Assume an all-around weld with a length, L_{weld}, equal to the perimeter of the rail post.

Post width, b = 4 in
Post length, h = 4 in
$L_{weld} = b \cdot h = 4 \cdot 4 = 16$ in

Calculate the section properties of the weld group by treating the weld as a line:

$$I_{weld} = \frac{2 \cdot h^3}{12} + 2 \cdot h \cdot \left(\frac{b}{2}\right)^2 = \frac{2 \cdot 4^3}{12} + 2 \cdot 4 \cdot \left(\frac{4}{2}\right)^2 = 42.7 \text{ in}^4$$

$$S_{weld} = \frac{I_{weld}}{0.5 \cdot b} = \frac{42.7}{0.5 \cdot 4} = 21.4 \text{ in}^3$$

The weld stresses are due to a combination of moment and shear at the base of the post. Calculate the weld stress, f_{ub}, caused by the moment, And, f_{uv}, caused by shear:

$$f_{ub} = \frac{M_u}{S_{weld}} = \frac{66.4}{21.4} = 3.10 \text{ kips/in}$$

$$f_{uv} = \frac{H_u}{L_{weld}} = \frac{1.23}{16} = 0.08 \text{ kips/in}$$

The resultant weld stress, f_{uweld}, equals the square root of the sum of the squares of f_{ub} and f_{uv}:

$$f_{uweld} = \sqrt{f_{ub}^2 + f_{uv}^2} = \sqrt{3.10^2 + 0.08^2} = 3.10 \text{ kips/in}$$

Now calculate the weld resistance. Try the minimum fillet weld size.

The rail post wall thickness, t_{rp}, is 0.233 inches and the base plate thickness, t_{bp}, is 0.75 inches.

[Table 6.13.3.4-1] Then the minimum weld size, a = 0.25 in

[AASHTO/AWS D1.1 Table 3.1] For ASTM A500 Grade B HSS steel that is welded to ASTM A709 Grade 36 steel, the minimum electrode classification strength, F_{exx}, is 60 ksi.

For fillet welds, resistance factor, ϕ_{e2} = 0.80.

Then the factored shear resistance of the weld is:

[6.13.3.2.4]

$$R_w = 0.6 \cdot \phi_{e2} \cdot F_{exx} \cdot A_{we}$$

[6.13.3.3] For fillet welds, the effective area of the weld, A_{we}, is:

$$A_{we} = t_{we} \cdot L_{weld}$$

where t_{we} = effective throat of fillet weld

$$= a \cdot (\cos 45) = 0.25 \cdot 0.707 = 0.18 \text{ in}$$

A_{we} = 0.18 in per inch of weld

Then:

$$R_w = 0.6 \cdot 0.80 \cdot 60 \cdot 0.18 = 5.18 \text{ kips/in} > 3.10 \text{ kips/in} \qquad \text{OK}$$

E. Adhesive Anchor Design Forces

Factored Shear Force

Consider only the two back anchors as resisting shear. Assume that the base plate engages both anchors equally. Then,

$$V_u = \frac{H_u}{2} = \frac{1.23}{2} = 0.62 \text{ kips/anchor}$$

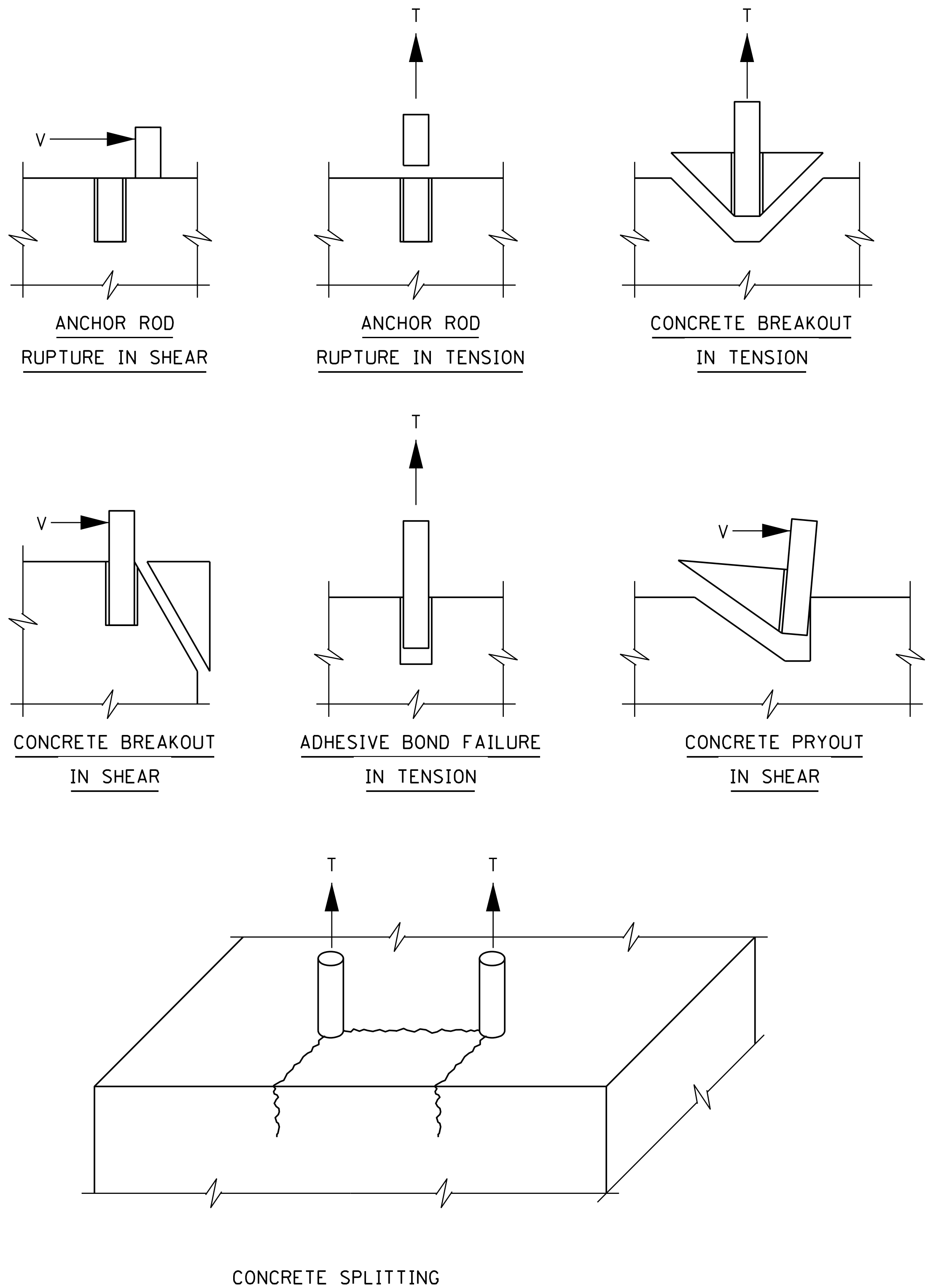
Factored Tension Force

Determine the factored tension load T_u on one anchor. Consider only the two front anchors as resisting tension, with each anchor loaded equally. The factored tension is equal to the compression force from the base plate. Then:

$$T_u = \frac{C}{2} = \frac{11.22}{2} = 5.61 \text{ kips/anchor}$$

[ACI 318 17.3.1]

For adhesive anchors, there are 7 possible failure modes that must be considered. They are illustrated in Figure 13.3.2.4:



Adhesive Anchor Failure Modes
Figure 13.3.2.4

F. Anchor Rods in Shear

Check the resistance of the anchor rods in shear. The anchor rods are assumed to have sufficient embedment to develop their shear capacity.

The material properties of MnDOT 3385, Type A anchor rods are:

F_y = 36 ksi and F_{ub} = 58 ksi

[6.5.4.2]

Type A anchor rods meet the requirements of ASTM F1554, so:

$\phi_s = 0.75$

[6.13.2.7]

The number of shear planes, N_s, is one for each anchor rod. The area, A_b, of one $^5/_8$" diameter anchor rod is 0.31 in^2. Assume that threads are included in the shear plane. The factored shear resistance is calculated as:

$R_n = 0.45 \cdot A_b \cdot F_{ub} \cdot N_s = 0.45 \cdot 0.31 \cdot 58 \cdot 1 = 8.09$ kips/anchor rod

$\phi_s \cdot R_n = 0.75 \cdot 8.09 = 6.07$ kips/anchor rod > 0.62 kips/anchor rod OK

G. Anchor Rods in Tension

Determine the capacity of the anchor rods in tension. Check if interaction effects need to be considered.

[6.13.2.11]

$$\frac{V_u}{R_n} = \frac{0.62}{8.09} = 0.08 < 0.33$$

The tension capacity can be found without considering the effects of shear.

$T_n = 0.76 \cdot A_b \cdot F_{ub} = 0.76 \cdot 0.31 \cdot 58 = 13.66$ kips

[6.5.4.2]

For an anchor rod meeting ASTM F1554, $\phi_t = 0.80$.

$\phi_t \cdot T_n = 0.80 \cdot 13.66 = 10.93$ kips > 5.61 kips OK

H. Concrete Breakout in Tension [ACI 318 17.4.2]

The resistance to concrete breakout in tension is a function of geometry and concrete compressive strength. It is based on a failure pyramid with an angle of approximately 35 degrees. For concrete breakout calculations, assume the concrete is cracked.

[ACI 318 17.4.2.2]

Start by determining the basic concrete breakout strength, N_b, of a single anchor in tension. For adhesive anchors, $k_c = 17$. Normal weight concrete is used, so $\lambda_a = 1.0$.

[ACI 318 17.3.2.3]

The limits for anchor embedment depth, h_{ef}, are:

Minimum $h_{ef} = 4 \cdot d_a = 4 \cdot 0.625 = 2.50$ in

Maximum $h_{ef} = 20 \cdot d_a = 20 \cdot 0.625 = 12.50$ in

Intuitively, designers would expect that a larger embedment depth will provide a higher strength. However, critical edge distance and critical anchor spacing are dependent on the embedment depth and large embedment may actually result in a lesser strength. For our case, it is also desireable to engage supplementary reinforcement to prevent splitting failure, therefore allowing the modification factor, $\psi_{cp,Na}$, to equal 1.0 when checking bond later on in this example. Choose a preliminary anchor embedment depth, h_{ef}, of 8 inches. This is adequate to engage the longitudinal C4__E bars in the curb that will act as supplementary reinforcement to prevent splitting failure.

Note that in ACI equations, units for concrete strength are in psi and not ksi. Then:

$$N_b = k_c \cdot \lambda_a \sqrt{f'_c} \cdot h_{ef}^{1.5} = 17 \cdot 1.0 \cdot \frac{\sqrt{4000}}{1000} \cdot 8.0^{1.5} = 24.33 \text{ kips}$$

[ACI 318 17.4.2.1] Next, calculate the area of the failure interface of a single anchor, A_{Nco}, excluding edge and group effects (refer to Figure 13.3.2.5):

$A_{Nco} = 9 \cdot h_{ef}^2 = 9 \cdot (8.0)^2 = 576.00 \text{ in}^2$

Determine the area of the actual failure interface, A_{Nc}.

[ACI 318 17.2.1.1] For concrete breakout in tension:

Critical edge distance $= 1.5 \cdot h_{ef} = 1.5 \cdot 8.00 = 12.00$ in

Critical spacing $= 3 \cdot h_{ef} = 3 \cdot 8.00 = 24.00$ in

Actual edge distances less than the critical edge distance will reduce the anchor tensile capacity. Similarly, actual anchor spacings less than the critical spacing will reduce the anchor tensile capacity.

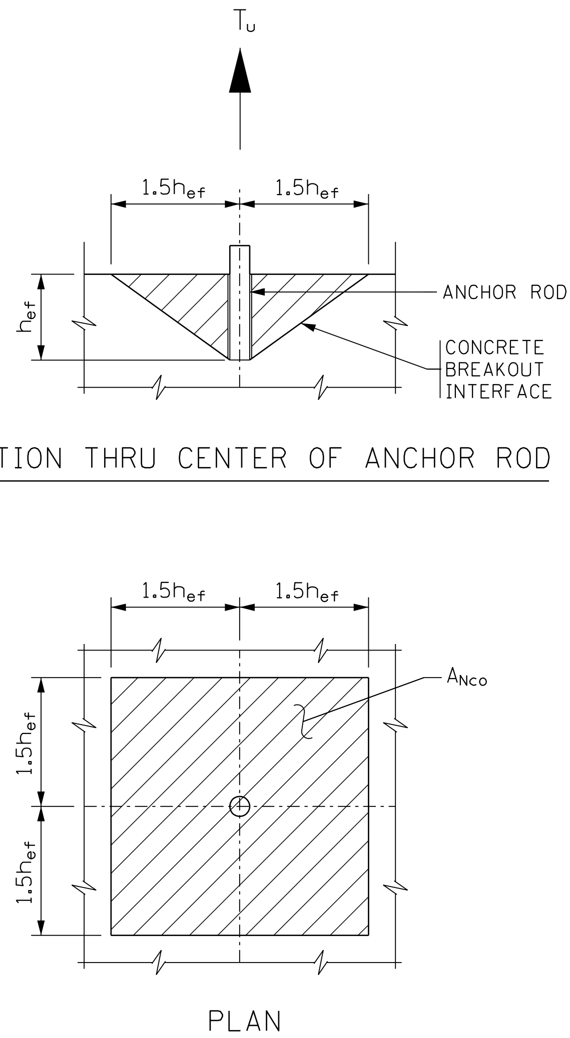


Figure 13.3.2.5

The previously calculated loads are based on a 10 ft maximum post spacing, which results in the largest load for an interior post. However, the anchors at the end post provide the least resistance due to edge effects. Rather than checking multiple cases, we will conservatively check the anchors at the end post using the loads at an interior post. Referring to Figures 13.3.2.2 and 13.3.2.6, the edge distances and anchor spacings needed to calculate A_{Nc} for the front two anchors in tension are:

Longitudinal direction dimensions:

Min. end of curb to center of left anchor c_{a1N} = 8.25 in < 12.00 in

Spacing between front two anchors s_{1N} = 7.50 in < 24.00 in

Spacing of right anchor to next anchor s_{2N} = 112.50 in > 24.00 in

Transverse direction dimensions:

Front edge of curb to center of anchors c_{a2N} = 4.50 in < 12.00 in

Back edge of curb to center of anchors c_{a3N} = 8.50 in < 12.00 in

The anchor spacing, s_{1N}, is less than the critical spacing, so the front two anchors are treated as a group.

Then:

$$A_{Nc} = [c_{a1N} + s_{1N} + 0.5 \cdot (3 \cdot h_{ef})] \cdot (c_{a2N} + c_{a3N})$$

$$= (8.25 + 7.50 + 0.5 \cdot 24.00) \cdot (4.50 + 8.50) = 360.75 \text{ in}^2$$

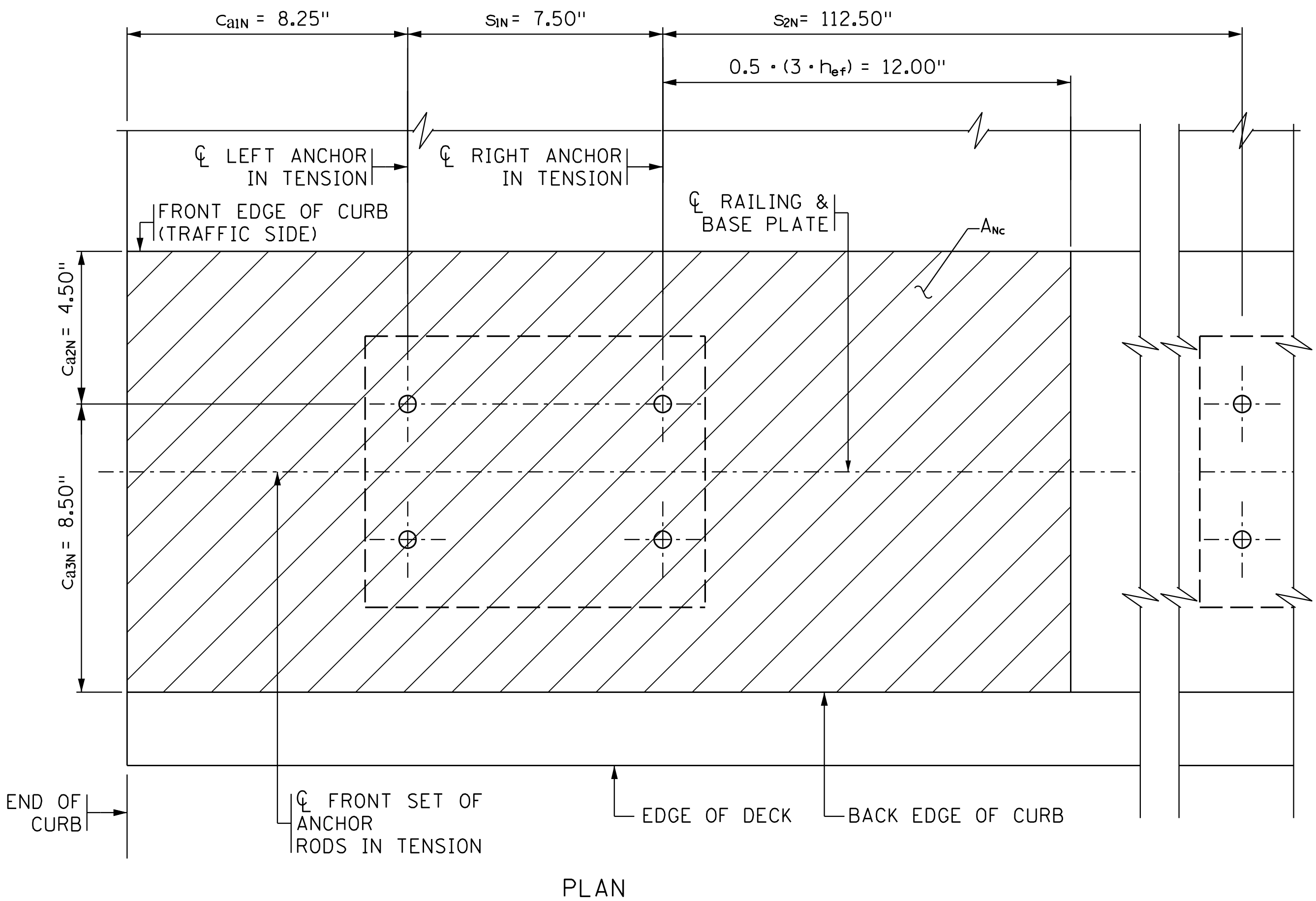


Figure 13.3.2.6

Because the equations in ACI are empirical, the resistance must be modified to account for eccentric effects, edge effects, concrete cracking, the presence or absence of reinforcement, and splitting effects.

[ACI 318 17.4.2.4] The loading is not eccentric to the anchor group, so $\psi_{ec,N}$ = 1.0.

[ACI 318 17.4.2.5] For edge effects, $c_{amin} = c_{a2N} = 4.50$ in $< 1.5 \cdot h_{ef}$, so

$$\psi_{ed,N} = 0.7 + 0.3 \cdot \frac{c_{amin}}{1.5 \cdot h_{ef}} = 0.7 + 0.3 \cdot \frac{4.50}{1.5 \cdot 8.00} = 0.81$$

[ACI 318 17.4.2.6] The concrete is assumed to be cracked, so $\psi_{c,N}$ and $\psi_{cp,N} = 1.0$.

[ACI 318 17.4.2.7]

[ACI 318 17.4.2.1] Then the nominal concrete breakout strength in tension, N_{cbg}, is:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec,N} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} \cdot N_b$$

$$= \frac{360.75}{576.00} \cdot 1.0 \cdot 0.81 \cdot 1.0 \cdot 1.0 \cdot 24.33 = 12.34 \text{ kips}$$

The concrete curb contains sufficient reinforcement to meet strength and temperature and shrinkage requirements, so anchorage Condition A applies. Per Table 1-2 found in the attachment to Technical Memorandum No. 18-11-B-01, use a resistance factor for concrete breakout in tension, $\phi_{cb,N} = 0.85$.

$\phi_{cb,N} \cdot N_{cbg} = 0.85 \cdot 12.34 = 10.49$ kips

$T_{ug} = T_u \cdot n_{anch,N} = 5.61 \cdot 2 = 11.22$ kips > 10.49 kips NO GOOD

[ACI 318 17.4.2.9] The reinforcement in the curb can act as anchor reinforcement to resist concrete breakout. To meet ACI requirements, anchor reinforcement must resist the entire tension load. The resistance factor for this check, ϕ_{anch}, is 0.75.

The reinforcement in the curb consists of 3-#4 longitudinal bars and #4 transverse bars spaced at 10 inches. During the drilling of holes for the adhesive anchors, the contractor may cut a bar, reducing its effectiveness to near zero. We will assume 2 longitudinal bars and 1 transverse bar resist the concrete breakout in tension.

For the longitudinal bars, the failure mode is direct shear through the bar, so the provisions for steel bolts with threads in the shear plane will be used:

$f_u = 90$ ksi for ASTM A615 Grade 60 bars

[6.13.2.7] $V_{long} = 0.45 \cdot A_{slong} \cdot f_u \cdot N_{blong} = 0.45 \cdot 0.20 \cdot 90 \cdot 2 = 16.20$ kips

For transverse bars:

f_y = 60 ksi

$T_{trans} = A_{strans} \cdot f_y \cdot N_{btrans} = 0.20 \cdot 60 \cdot 1 = 12.00$ kips

Then factored resistance of anchor bars, $\phi_{anch} \cdot N_{anch}$, is:

$\phi_{anch} \cdot N_{anch} = \phi_{anch} \cdot (V_{long} + T_{trans})$
$= 0.75 \cdot (16.20 + 12.00) = 21.15$ kips > 10.49 kips OK

I. Concrete Breakout in Shear [ACI 318 17.5.2]

Refer to Figures 13.3.2.2 and 13.3.2.7 for applicable dimensions. For concrete breakout in shear, two load distribution cases must be considered. Case A assumes equal distribution of forces to the front and back sets of anchors (front and back set of anchors each must resist $0.5V_u$). Case B distribution is dependent on the spacing s_{3V} between the front and back set of anchors, the distance c_{a1V} from the back set of anchors to the back edge of the curb, and whether or not the anchors are welded to a common plate.

For our case, $c_{a1V} < s_{3V}$ and the anchors are not welded to a common plate. Then for Case B, the entire shear force is applied to the back set of anchors (back set of anchors must resist V_u). By inspection, Case B governs, so only Case B is checked below.

For concrete breakout calculations, assume the concrete is cracked.

[ACI 318 17.5.2.1]

First, calculate the area of the failure interface of a single anchor, A_{Vco}, excluding edge and group effects:

$A_{Vco} = 4.5 \cdot c_{a1V}^2 = 4.5 \cdot (4.5)^2 = 91.13 \text{ in}^2$

[ACI 318 17.5.2.2]

Next, determine the basic concrete breakout strength, V_b, of a single anchor in shear.

The load bearing length, ℓ_e, is equal to the lesser of ℓ_{e1} and ℓ_{e2}:

Normally, $\ell_{e1} = h_{ef}$ = 8 inches. However, since the curb height, h_{curb}, is only 6 inches, conservatively choose to limit ℓ_{e1} to h_{curb}.

ℓ_{e1} = 6.00 in
$\ell_{e2} = 8 \cdot d_a = 8 \cdot 0.625 = 5.00$ in GOVERNS

The anchor rod diameter, d_a, is 0.625 inches. Normal weight concrete is used, so λ_a = 1.0. Note that in ACI equations, units for concrete strength are in psi and not ksi.

Then, V_b is the smaller of the following:

$$V_{b1} = 7 \cdot \left(\frac{\ell_e}{d_a}\right)^{0.2} \cdot \sqrt{d_a} \cdot \lambda_a \cdot \sqrt{f'_c} \cdot (c_{a1V})^{1.5}$$

$$= 7 \cdot \left(\frac{5.00}{0.625}\right)^{0.2} \cdot \sqrt{0.625} \cdot 1.0 \cdot \sqrt{4000} \cdot 4.50^{1.5} = 5.06 \text{ kips}$$

$$V_{b2} = 9 \cdot \lambda_a \cdot \sqrt{f'_c} \cdot (c_{a1V})^{1.5}$$

$$= 9 \cdot 1.0 \cdot \sqrt{4000} \cdot 4.50^{1.5} = 5.43 \text{ kips}$$

Then V_b = 5.06 kips

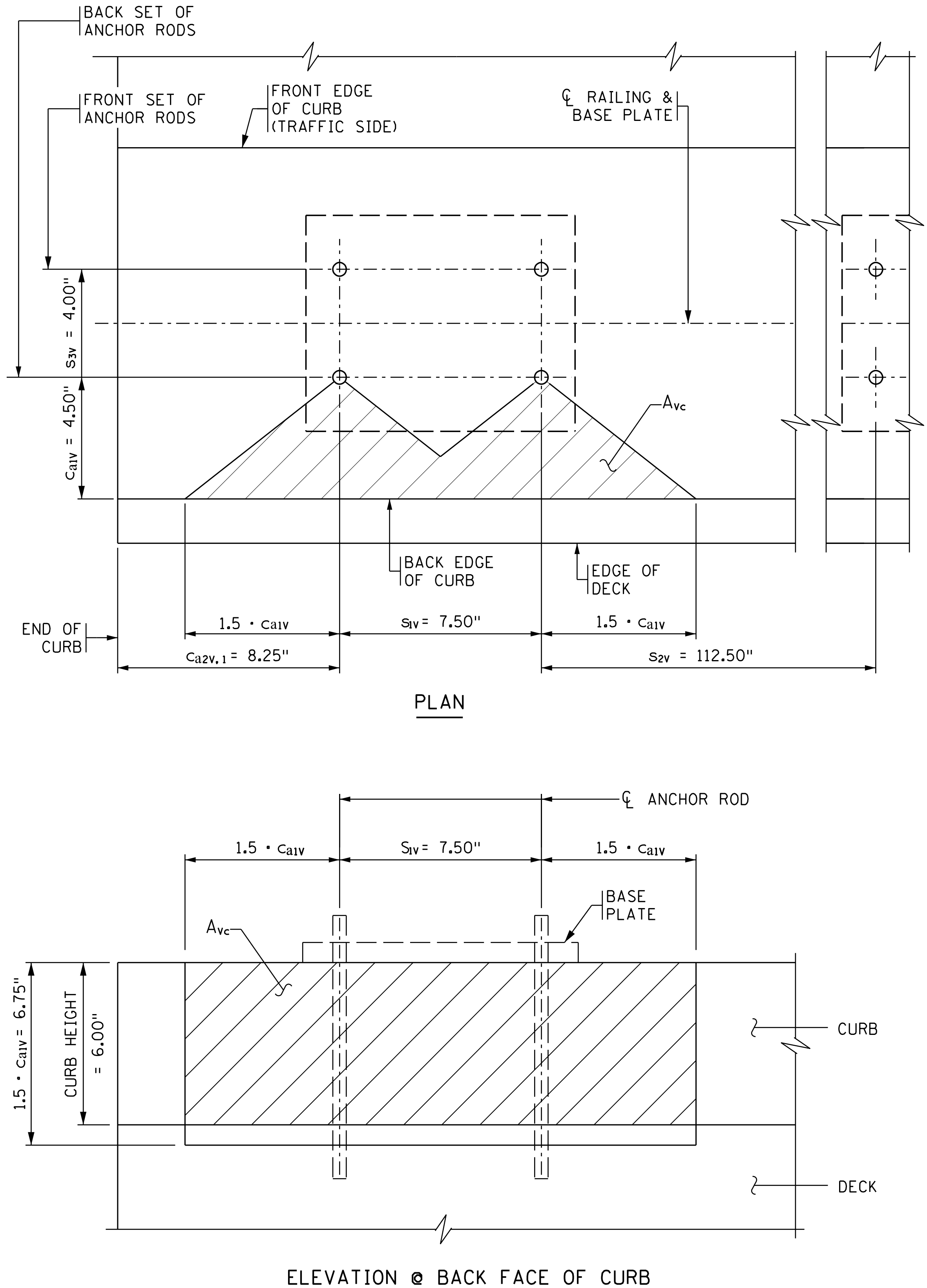


Figure 13.3.2.7

[ACI 318 17.5.2.1] Now, determine the area of the actual failure interface, A_{Vc}.

For concrete breakout in shear:

Critical edge distance = $1.5 \cdot c_{a1V} = 1.5 \cdot 4.50 = 6.75$ in

[ACI 318 17.2.1.1] Critical spacing = $3 \cdot c_{a1V} = 3 \cdot 4.50 = 13.50$ in

Actual edge distances less than the critical edge distance will reduce the anchor tensile capacity. Similarly, actual anchor spacings less than the critical spacing will reduce the anchor tensile capacity.

Referring to Figure 13.3.2.7, the longitudinal direction edge distance and anchor spacings needed to calculate A_{Vc} for the back two anchors in shear are:

Min. end of curb to center of left anchor $c_{a2V} = 8.25 \text{ in} > 6.75$ in

Spacing between front two anchors $s_{1V} = 7.50 \text{ in} < 13.50$ in

Spacing of right anchor to next anchor $s_{2V} = 112.50 \text{ in} > 13.50$ in

The depth of A_{Vc} is normally taken equal to $1.5 \cdot c_{a1V} = 6.75$ in

However, since the curb height, h_{curb}, is only 6 inches, conservatively choose to limit depth of A_{Vc} to h_{curb}.

Then:

$$A_{Vc} = (1.5 \cdot c_{a1V} + s_{1V} + 1.5 \cdot c_{a1V}) \cdot h_{curb}$$
$$= (1.5 \cdot 4.50 + 7.50 + 1.5 \cdot 4.50) \cdot 6.00 = 126.00 \text{ in}^2$$

Similar to concrete breakout in tension, modification factors must be determined for concrete breakout in shear:

[ACI 318 17.5.2.5] The loading is not eccentric to the anchor group, so $\psi_{ec,V} = 1.0$.

[ACI 318 17.5.2.6] For edge effects, $c_{a2V,1}$ and $c_{a2V,2}$ are both > c_{critV}, so $\psi_{ed,V} = 1.0$.

[ACI 318 17.5.2.7] The concrete is assumed to be cracked without supplementary reinforcement, so $\psi_{c,V} = 1.0$.

[ACI 318 17.5.2.8] The depth of the member, h_a, equals curb height plus deck thickness:

$h_a = 6.00 + 9.00 = 15 \text{ in} > 6.75$ in

Therefore $\psi_{h,V} = 1.0$.

[ACI 318 17.5.2.1] Then the nominal concrete breakout strength in shear, V_{cbg}, is:

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec,V} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot \psi_{h,V} \cdot V_b$$

$$= \frac{126.00}{91.13} \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 5.06 = 7.00 \text{ kips}$$

The concrete curb contains sufficient reinforcement to meet strength and temperature and shrinkage requirements, so anchorage Condition A applies. Per Table 1-2 found in the attachment to Technical Memorandum No. 18-11-B-01, use a resistance factor for concrete breakout in shear, $\phi_{cb,V} = 0.85$.

$$\phi_{cb,V} \cdot V_{cbg} = 0.85 \cdot 7.00 = 5.95 \text{ kips}$$

$V_{ug} = V_u \cdot n_{anch,V} = 0.62 \cdot 2 = 1.24 \text{ kips} < 5.95 \text{ kips}$ OK

J. Adhesive Anchor Bond in Tension [ACI 17.4.5]

Only adhesives found on the MnDOT Approved/Qualified Products List (see http://www.dot.state.mn.us/products/bridge/concreteanchorages.html) are allowed for use. MnDOT requires higher characteristic bond strength values than those found in ACI Table 17.4.5.2. For threaded rods used to anchor ornamental metal railings the minimum characteristic bond strength in uncracked concrete, τ_{uncr}, is 1.5 ksi.

[ACI 318 17.4.5.1]

For adhesive anchor bond in tension:

Critical edge distance, c_{Na}, is:

$$c_{Na} = 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100}} = 10 \cdot 0.625 \cdot \sqrt{\frac{1500}{1100}} = 7.30 \text{ in}$$

[ACI 318 17.2.1.1]

Critical spacing $= 2 \cdot c_{Na} = 2 \cdot 7.30 = 14.60$ in

[ACI 318 17.4.5.1]

Now calculate the area of the projected influence area of a single anchor, A_{Nao}, excluding edge and group effects:

$$A_{Nao} = (2 \cdot c_{Na})^2 = (2 \cdot 7.30)^2 = 213.16 \text{ in}^2$$

Referring to Figures 13.3.2.2 and 13.3.2.8, the edge distances and anchor spacings needed to calculate A_{Na} for the front two anchors in tension are:

Longitudinal direction dimensions:

Min. end of curb to center of left anchor $c_{a1N} = 8.25 \text{ in} > 7.30$ in

Spacing between front two anchors $s_{1N} = 7.50 \text{ in} < 14.60$ in

Spacing of right anchor to next anchor $s_{2N} = 112.50 \text{ in} > 14.60$ in

Transverse direction dimensions:

Front edge of curb to center of anchors $c_{a2N} = 4.50 \text{ in} < 7.30$ in

Back edge of curb to center of anchors $c_{a3N} = 8.50 \text{ in} > 7.30$ in

The spacing $s_{1N} < 14.60$ in, so anchors are treated as a group.

Then:

$$A_{Na} = (c_{Na} + s_{1N} + c_{Na}) \cdot (c_{a2N} + c_{Na})$$
$$= (7.30 + 7.50 + 7.30) \cdot (4.50 + 7.30) = 260.78 \text{ in}^2$$

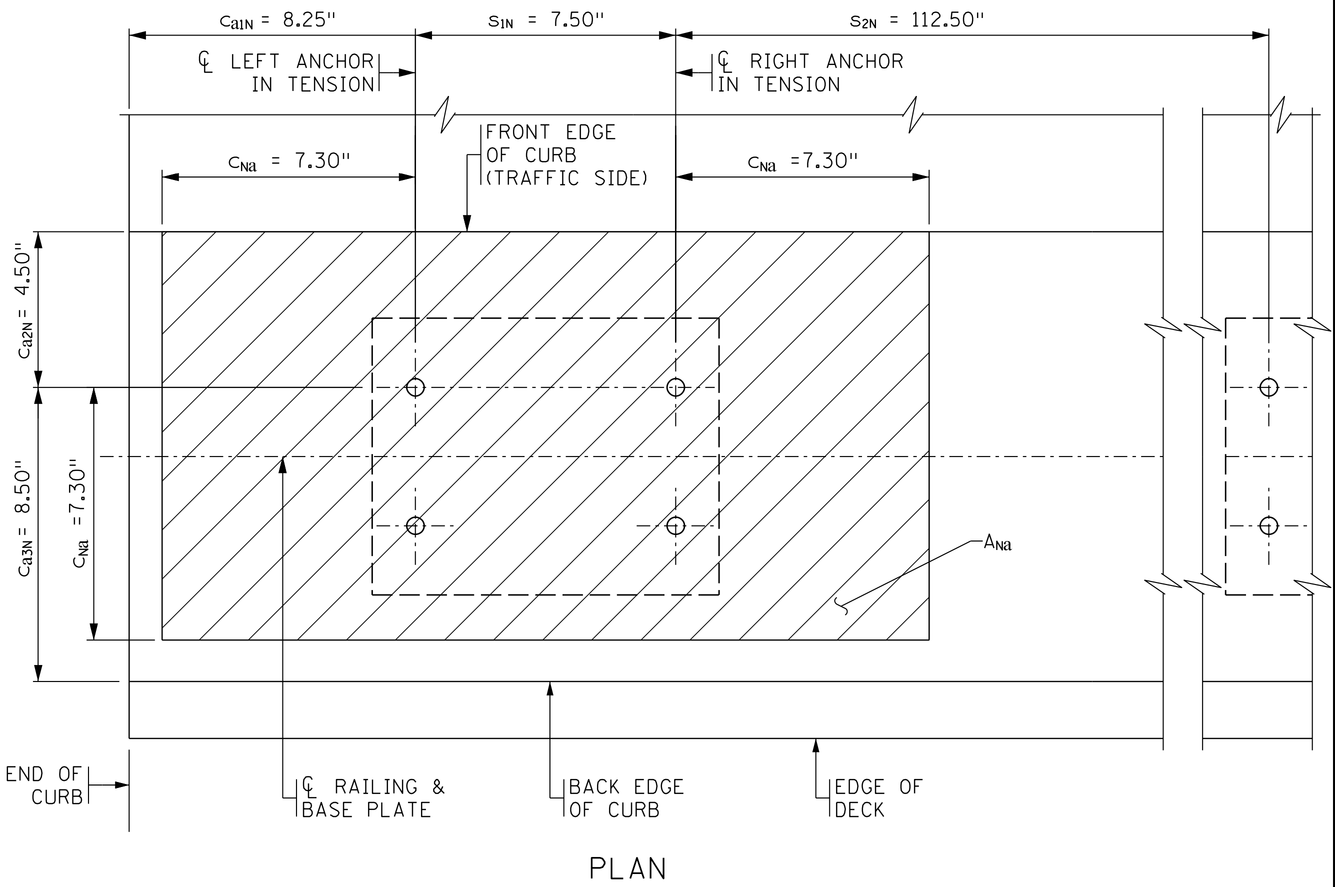


Figure 13.3.2.8

[ACI 318 17.4.5.2] Now determine the basic bond strength, N_{ba}, of a single adhesive anchor in tension. Since it is unlikely that a crack will form through the adhesive anchors, MnDOT assumes uncracked concrete for pedestrian rail anchorages attached to reinforced concrete sections (see Technical Memorandum No. 18-11-B-01).

Then:

$$N_{ba} = \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef} = 1.0 \cdot 1.5 \cdot \pi \cdot 0.625 \cdot 8 = 23.56 \text{ kips}$$

Next, determine the modification factors:

[ACI 318 17.4.5.3] The loading is not eccentric to the anchor group, so $\psi_{ec,Na} = 1.0$.

[ACI 318 17.4.5.4]

For edge effects, $c_{a,min} = c_{a2N} = 4.50$ in < 7.30 in

Then:

$$\psi_{ed,Na} = 0.7 + 0.3 \cdot \frac{c_{a,min}}{c_{Na}} = 0.7 + 0.3 \cdot \frac{4.50}{7.30} = 0.88$$

[ACI 318 17.4.5.5]

The concrete is assumed to be uncracked and has supplementary reinforcement (see discussion in BDM Article 13.3.2.H), so $\psi_{cp,Na} = 1.0$.

[ACI 318 17.4.5.1]

The nominal bond strength in tension, N_{ag}, for the group of anchors is:

$$N_{ag} = \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec,Na} \cdot \psi_{ed,Na} \cdot \psi_{cp,Na} \cdot N_{ba}$$

$$= \frac{260.78}{213.16} \cdot 1.0 \cdot 0.88 \cdot 1.0 \cdot 23.56 = 25.36 \text{ kips}$$

The concrete curb contains sufficient reinforcement to meet strength and temperature and shrinkage requirements, so anchorage Condition A applies. Per Table 1-2 found in the attachment to Technical Memorandum No. 18-11-B-01, use a resistance factor for bond, $\phi_{a,N} = 0.75$.

$\phi_{a,N} \cdot N_{ag} = 0.75 \cdot 25.36 = 19.02$ kips > 11.22 kips OK

K. Concrete Pryout in Shear
[ACI 17.5.3]

Although pryout is a shear failure mode, the pryout design strength, is based on the concrete breakout strength in tension, the adhesive bond strength, and the anchor embedment depth.

$h_{ef} = 8$ in > 2.5 in, so $k_{cp} = 2.0$

The group tension strength governed over the single anchor strength, so we will only consider the group pryout.

The basic concrete pryout strength, N_{cpg}, for the group of 2 anchors is the lesser of:

$N_{cpg1} = N_{cbg} = 12.34$ kips GOVERNS
$N_{cpg2} = N_{ag} = 25.36$ kips
Then $N_{cpg} = 12.34$ kips

The nominal pryout strength, V_{cpg}, for the group of anchors is:

$V_{cpg} = k_{cp} \cdot N_{cpg} = 2.0 \cdot 12.34 = 24.68$ kips

Condition B is always assumed for pryout. Then per Table 1-2 found in the attachment to Technical Memorandum No. 18-11-B-01, use a resistance factor for pryout, $\phi_p = 0.65$.

$\phi_p \cdot V_{cpg} = 0.65 \cdot 24.68 = 16.04$ kips > 1.24 kips OK

L. Tension/Shear Interaction [ACI 17.6]

Check whether the tension and shear interaction requirements of ACI are met.

$V_{ug} = 1.24$ kips

$\phi_p \cdot V_n$ is the smallest of:

Factored concrete breakout in shear resistance,
$\phi_{cb,V} \cdot V_{cbg} = 5.95$ kips GOVERNS
Factored pryout resistance, $\phi_p \cdot V_{cpg} = 16.04$ kips

Then:

[ACI 318 17.6.3]

$$\frac{V_{ug}}{\phi_n \cdot V_n} = \frac{1.24}{5.95} = 0.21 > 0.20, \text{ so check interaction equation}$$

$T_{ug} = 11.22$ kips

$\phi_p \cdot N_n$ is the smallest of:

Factored concrete breakout in tension resistance,
$\phi_{anch} \cdot N_{anch} = 21.15$ kips
Factored bond resistance, $\phi_a \cdot N_{ag} = 19.02$ kips GOVERNS

ACI chose a trilinear interaction approach to simplify the interaction equation calculations (see ACI Figure R17.6):

$$\frac{T_{ug}}{\phi_n \cdot N_n} + \frac{V_{ug}}{\phi_n \cdot V_n} = \frac{11.22}{19.02} + \frac{1.24}{5.95} = 0.80 < 1.2 \quad \text{OK}$$

M. Concrete Splitting [ACI 318 17.7]

Concrete splitting is not a specific check required by ACI. Instead, ACI includes minimum edge distances and minimum anchor spacing to preclude splitting failure.

[ACI 318 17.7.3]

The minimum required edge distance, $c_{aminspl}$, is:

$c_{aminspl} = 6 \cdot d_a = 6 \cdot 0.625 = 3.75$ in < 4.50 in OK

[ACI 318 17.7.1]

The minimum required anchor spacing, $s_{aminspl}$, is:

$s_{aminspl} = 6 \cdot d_a = 6 \cdot 0.625 = 3.75$ in < 7.50 in OK

N. Proof Load

In order to verify the installation and strength of the adhesive anchors at the project site, a number of anchors are subjected to a proof load test. The proof load is chosen as the smaller of:

1) T_{pr1}, equal to 80% of the anchor rod yield stress:

$$T_{pr1} = 0.80 \cdot \frac{\pi \cdot d_a^2}{4} \cdot F_y = 0.80 \cdot \frac{\pi \cdot 0.625^2}{4} \cdot 36 = 8.8 \text{ kips}$$

2) T_{pr2}, equal to the factored capacity of a single anchor in tension. For calculation of T_{pr2}, the group effects are not included since only one anchor will be tested at a time. For tension, there are 2 failure modes to consider, concrete breakout and bond. For concrete breakout in tension, it was shown earlier that reinforcement in the curb can act as anchor reinforcement to resist concrete breakout, with a factored resistance of 21.15 kips. For bond, refer to Figure 13.3.2.9. The projected influence area, A_{na}, is different from what was previously calculated:

$$A_{Na} = (c_{Na} + c_{Na}) \cdot (c_{a2N} + c_{Na})$$
$$= (7.30 + 7.30) \cdot (4.50 + 7.30) = 172.28 \text{ in}^2$$

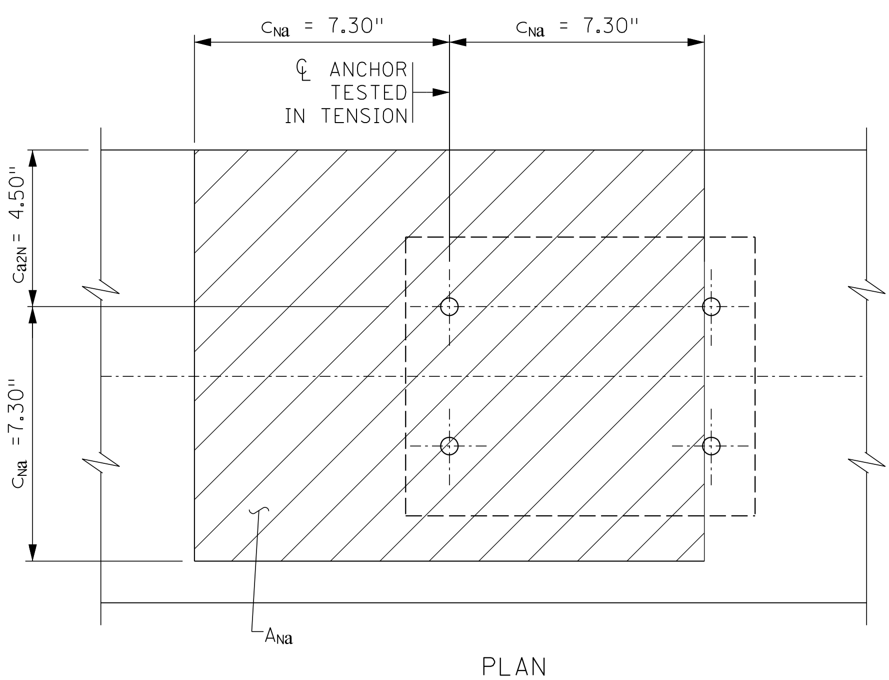


Figure 13.3.2.9

Then:

$$N_a = \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ed,Na} \cdot \psi_{cp,Na} \cdot N_{ba}$$

$$= \frac{172.28}{213.16} \cdot 0.88 \cdot 1.0 \cdot 23.56 = 16.76 \text{ kips}$$

$$\phi_{a,N} \cdot N_a = 0.75 \cdot 16.76 = 12.57 \text{ kips} < 21.15 \text{ kips}$$

$$T_{pr2} = 12.57 \text{ kips} > 8.8 \text{ kips}$$

Choose a proof load of 8.8 kips.

O. Summary

Adhesive anchors with the following properties are adequate to attach the ornamental metal railing Design T-4 to a 6 inch concrete curb:

The anchor rods are $^5/_8$" diameter, MnDOT 3385 Type A anchor rods with 8" minimum embedment.

The minimum characteristic bond strength of the adhesive in uncracked concrete is 1.5 ksi.

The proof load for field testing is 8.8 kips.