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Strengthening of Existing Inverted-T Bent Caps: Design Recommendations and Examples

Technical Report 0-6893-P1

Cooperative Research Program

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16. Abstract

Inverted-T bent caps have been widely used in Texas to reduce the overall elevation of bridges, to improve the available clearance beneath the beams, and to improve aesthetics. Since bridge design criteria have been improved and modified over the decades, many of the early inverted-T bent caps are deficient when evaluated against the current design approach and/or lack adequate strength to support planned increases to live load demands. Therefore, techniques for strengthening these bent caps are needed.

In this volume, the recommendations for identifying deficiencies of in-service bent caps are addressed along with the recommendations for selecting retrofit solutions to use. Based on the observations of the tests described in Volume 2, design procedures for six solutions are provided: end-region stiffener, clamped threadbar with channel, load-balancing post-tensioning, concrete infill with partial- and full-depth fiber reinforced polymer anchored by steel waling for the bent, and large bearing pad. Step-by-step procedures for each solution and design examples of each solution for the bent caps described are provided.

17. Key Words		18. Distribution Statement	
Inverted-T Beam, Bent Cap, Bridge, Ledge, Hanger, Punching Shear, Concrete Shear Friction, Ledge Flexure, Retrofit, Strengthening, FRP Wrap, External Post-Tensioning, Prestress, PT Bar, Bearing, Bearing Pad		No restrictions. This document is available to the public through NTIS: National Technical Information Service Alexandria, Virginia http://www.ntis.gov	
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DISCLAIMER

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This report is not intended for construction, bidding, or permit purposes. The researcher in charge of the project was Stefan Hurlebaus. The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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CHAPTER 1: INTRODUCTION

1.1 Overview

Inverted-T bent caps have been widely used in Texas to reduce the overall elevation of bridges, to improve the available clearance beneath the beams, and to improve aesthetics. The structural behavior of inverted-T bent caps is different than that of conventional top-loaded beams since the loads are introduced into the bottom flange rather than the top of the beam. The flange of the inverted-T serves as a shallow ledge to seat the bridge girders, while the web of the inverted-T, rising above the ledge, provides the required depth to deliver sufficient flexure and shear strength and stiffness.

Diagonal cracks at the reentrant corners between the cantilever ledges and the web in older, existing inverted-T bent caps have been reported throughout the state of Texas. Since bridge design criteria have been improved and modified over the decades, many of the early inverted-T bent caps are deficient when evaluated against the current design approach and/or lack adequate strength to support planned increases in live load demands. One example is the substructure supporting the IH 35 upper deck through downtown Austin (Figure 1.1).

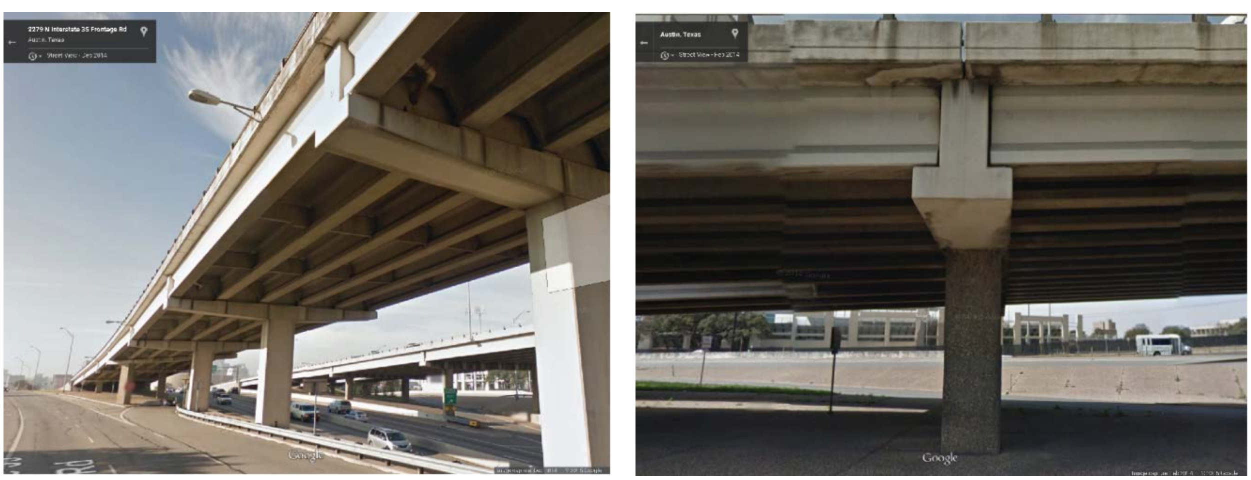


Figure 1.1. Inverted-T Bent Cap in Downtown Austin (Source: Google Maps).

Replacement of deficient bent caps is not always practical due to cost, interruption to traffic, and the acceptable condition of other parts of the structure. Therefore, techniques for strengthening these bent caps are needed. However, despite the need for robust, proven strengthening techniques of inverted-T bent caps in Texas, no formal guidance is available in current standards. As such, there is a need to investigate the effectiveness of retrofit solutions that adequately address the design deficiencies and observed in-service damage of existing inverted-T bent caps.

1.2 Project Objective

This project focused on the design and validation of satisfactory performance of strengthening existing inverted-T bent cap ledges through experimental testing. A primary objective was to demonstrate and validate, through experimental testing, the satisfactory performance of strengthening existing inverted-T bent caps. The research objectives were to:

- Evaluate existing inverted-T bent caps based on field visits and current design methodologies.
- Propose technical concepts to retrofit inverted-T bent caps found to be deficient using current design methodologies.
- Evaluate the proposed retrofit solutions and make recommendations to test.
- Conduct experimental tests on half-scaled specimens and analyze the results.
- Develop design recommendations and provide design examples for the tested retrofit solutions.

The solutions developed in this research are expected to provide increased capacity of existing substructure components on numerous direct connectors and other bridges including the highly congested IH 35 upper deck through downtown Austin.

1.3 Summary of Volume 1 and 2

Details of this project are reported in three volumes. Volume 1 (Hurlebaus et al., 2018a) presented (a) a literature review to guide the analysis of inverted-T bent caps and develop retrofit solutions; (b) an evaluation of in-service inverted-T bent caps in Austin, Texas; (c) retrofit solutions for strengthening inverted-T bent caps with deficient capacity; and (d) an evaluation of the proposed retrofit solutions.

Following is a summary of the evaluation of in-service inverted-T bent caps and retrofit solutions:

- During field inspections, the types of cracks observed on the end face of the in-service inverted-T bent caps were diagonal cracks, horizontal cracks, vertical cracks, or a combination of these initiated from the ledge-web interface.

- Typical double- and single-column bents were found to be deficient for increased traffic when evaluated with modern codes.
- Eighteen retrofit solutions were proposed, including external post-tensioning (PT), steel bracket to provide supplementary load path, fiber reinforced polymer (FRP) wrap with anchors, concrete masonry column, and increased bearing pad size.
- The proposed solutions were designed to address ledge flexure, punching shear, and hanger failure modes.
- Proposed solutions were evaluated in terms of six criteria: strength increase, total cost, constructability, clearance constraints, durability, and ease of monitoring.
- Using a weighted sum model with specified weight factors, the retrofit solutions were rated and ranked to create a decision matrix to choose the most viable solutions.
- Top-ranked solutions were selected to test in the lab: end-region stiffener (Solution 3), clamped threadbar with channel (Solution 8), load-balancing PT (Solution 14), concrete infill wall with partial- and full-depth FRP anchored with steel waling (Solutions 16 and 17), and large bearing pad (Solution 18). Schematics of each solution are shown in Figure 1.2.

In Volume 2 (Hurlebaus et al., 2018b), an experimental test program to investigate the ability of retrofit solutions to strengthen inverted-T bent caps was conducted. Thirty-three tests were conducted on eight half-scale inverted-T bent specimens (five ledge-deficient and three hanger-deficient specimens). It was found that the cracks observed from the experimental test matched field observations. The diagonal cracks were observed on the punching shear reference specimen, while horizontal cracks were observed on the hanger reference specimen. Vertical and a combination of vertical and diagonal cracks were observed on the ledge reference specimen.

Hanger-deficient specimens were strengthened with end-region stiffener (Solutions 3), clamped threadbar with channel (Solution 8), load-balancing PT (Solution 14), and concrete infill with full-depth FRP anchored with steel waling (Solution 17) to evaluate the performance of these retrofit methods in improving hanger capacity. The largest exterior hanger capacity increase was provided by Solution 8 (61 percent), and the smallest was provided by Solution 3 (18 percent). The largest interior capacity increase was provided by Solution 17 (23 percent), which resulted in a shift in failure mode from hanger to ledge flexure. Solution 14 (interior and exterior) provided substantial reduction in damage at loads expected on in-service bent caps; this retrofit was not tested to failure.

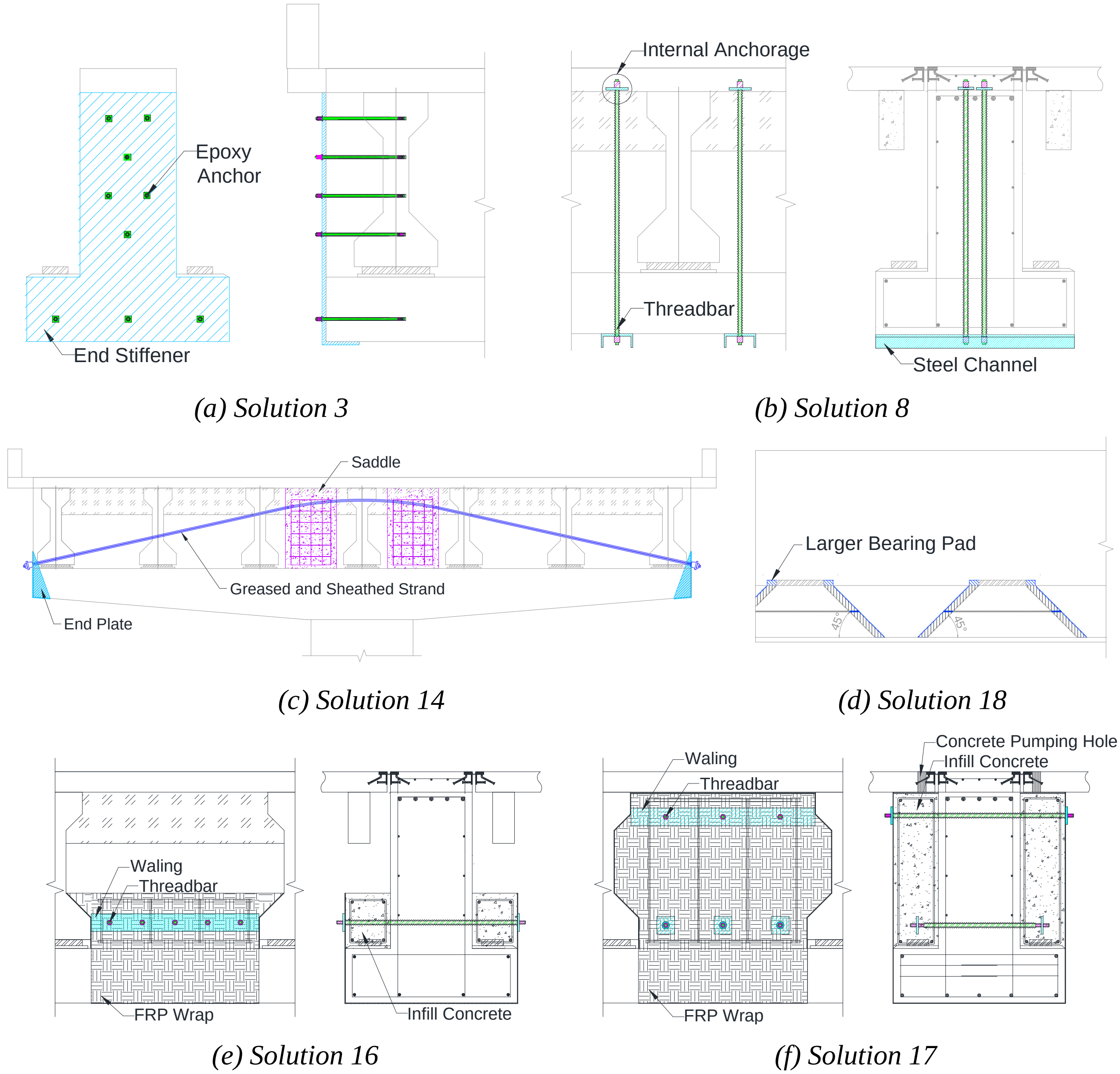


Figure 1.2. Schematics of Selected Retrofit Solutions.

Ledge-deficient specimens were strengthened with end-region stiffener (Solutions 3), clamped threadbar with channel (Solution 8), load-balancing PT (Solution 14), and concrete infill with partial- and full-depth FRP anchored with steel waling (Solutions 16 and 17) to assess the effectiveness of the solutions in increasing the ledge flexure capacity. The largest exterior ledge capacity increase was provided by Solution 17 (82 percent), and the smallest was provided by Solution 8 (21 percent for one threadbar, 36 percent for two threadbars). The interior ledge capacity increase was investigated by two solutions, with Solution 16 (21 percent) providing a greater increase in capacity than Solution 8 (16 percent).

Punching shear tests were conducted on ledge-deficient specimens to assess the effect of bearing pad size (Solution 18) on punching shear capacity. Larger pads increased the exterior capacity by 14 percent, but there was slight or no improvement in the interior capacity.

In Volume 2 (Hurlebaus et al., 2018b), the accuracy of American Association of State Highway and Transportation Officials load and resistance factor design (LRFD) (AASHTO, 2014) procedures in estimating the capacity of inverted-T bent caps was evaluated. Rational modifications for some AASHTO LRFD equations were provided. Modifications were proposed for exterior distribution widths for ledge flexure and ledge shear friction and for the angle of truncated pyramid for punching shear capacity based on the test results.

1.4 Overview of Volume 3

In this volume, the results of the experimental test program are used to develop recommendations for evaluation of in-service inverted-T bent caps and design of select retrofit solutions. To demonstrate implementation of the proposed design recommendations, two design examples are provided for double-column bents for each solution considered, and, where appropriate, for single-column bents.

1.5 Report Outline

Chapter 2 addresses the recommendations for identifying deficiencies of in-service bent caps. In-service bent caps used for the examples are summarized along with the calculated deficiencies of the bent caps. In Chapter 3, recommendations are provided for selecting retrofit solutions. Chapter 4 through 9 describe design procedures for six solutions: end-region stiffener (Solution 3), clamped threadbar with channel (Solution 8), load-balancing PT (Solution 14), concrete infill with partial- and full-depth FRP anchored by steel waling (Solutions 16 and 17), and large bearing pad (Solution 18). Step-by-step procedures for each solution are addressed in each chapter. Calculations of deficiencies are provided in Appendix A. Design examples of each solution for the bent caps are presented in Appendix B and Appendix C. Finally, Chapter 10 provides a summary of findings.

CHAPTER 2: BENT CAP ANALYSIS FOR DESIGN EXAMPLES

2.1 Overview

To establish the strength deficiencies that must be addressed in the design of inverted-T strengthening solutions, the bent caps must be analyzed. The potential need to strengthen these particular structures is the result of (a) changes in design provisions since the time construction in the late 1960s, and (b) interest in increasing the number of lanes on the bridge, thereby increasing the demands.

In this chapter, the capacity (*C*) of inverted-T bent caps is calculated using AASHTO LRFD (2014) sectional methods with modifications by Hurlebaus et al. (2018b) and compared to the demands (*D*). When there is insufficient capacity, the amount of additional strength needed, referred to as the deficiency, is calculated as:

$$Deficiency = D/\phi - C \tag{2.1}$$

where ϕ = strength reduction factor, 0.9.

Based on the recommendations to analyze in-service bent caps, the bent caps used for design examples are evaluated to identify the deficiencies. In Section 2.4, the bent caps are briefly summarized including structural characteristics and capacity.

2.2 Demands

Bent cap demands are characterized by girder loads for the most critical failure mechanisms: hanger, ledge shear friction, ledge flexure, punching shear, and bearing.

Dead loads include the self-weight of the girder, deck, and any overlay that may be present. The weight of the rails is distributed evenly among the stringers, up to three stringers per rail. To account for the additional dead load from the haunch of the column to the slab ends, the dead load of the slab is increased by 10 percent (Texas Department of Transportation [TxDOT], 2015).

Live loads are computed in accordance with Sections 3.6.1.2.2 and 3.6.1.2.4 of the AASHTO LRFD (2014) specifications. The vehicular live loading on the roadway consists of a combination of the design truck or the design tandem and the design lane load. The maximum live load is always governed by the design truck over the design tandem for spans greater than 26 ft. Figure 2.1 shows the locations of the HL-93 design truck on the interior girder, which generates the maximum load effect as described in Section 3.6.1.3.1 of AASHTO LRFD (2014). When the length of the spans is different, and the longer span (Span 2) is shorter than twice the short span (Span 1) length, the middle axle (32 kips) is placed over the interior support, the front axle (8 kips) is placed on the short span (Span 1), and the rear axle (32 kips) is placed on the long span (Span 2). To account for wheel load impact from moving vehicles, the live load without the design lane load is increased by applying dynamic load allowance factors, which are listed in Table 3.6.2.1-1 of AASHTO LRFD (2014). The load effects from the design lane load are subject to multiple presence factors (AASHTO, 2014). The live load applied to the slab is distributed to the beams by assuming the slab is hinged at each beam except the outside beam (TxDOT, 2015).

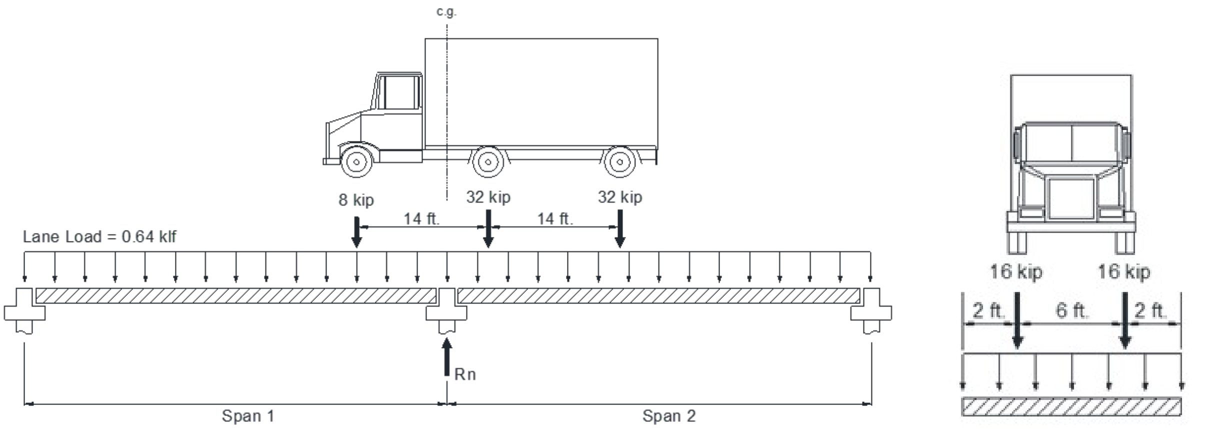
The girder reaction (V_u in Figure 2.1[d]) is the factored load on each ledge of the inverted-T bent caps. The live load for the girder reaction is maximized by placing the rear axial (32 kips) of the HL-93 truck model over the support, as shown in Figure 2.1(c), and multiplied by the shear live load distribution factor. The limit state factors listed in Table 3.4.1-1 of AASHTO (2014) are multiplied to obtain girder reaction. The Service I limit state factors are 1.0 for dead and live load. The Strength I limit state factors are 1.25 and 1.75 for dead and live load, respectively.

2.3 Capacity

AASHTO LRFD (2014) specifies the design methods for the beam ledges in Section 5.13.2.5. Figure 2.2 shows potential cracks and their locations on the ledge of an inverted-T bent cap. AASHTO LRFD (2014) indicates that the beam ledges must resist (a) flexure, shear, and horizontal forces; (b) tension force in the supporting element; (c) punching shear at points of loading; and (d) bearing force. The cracks specified in Figure 2.2 are referred to as “Ledge Shear Friction and Ledge Flexure (1),” “Hanger (2),” “Punching Shear (3),” and “Bearing (4).” Requirements to address the specific conditions of the inverted-T bent cap ledge component are outlined in Articles 5.13.2.5.2 through 5.13.2.5.5.

2.3.1 Evaluation of Hanger Capacity

Hanger reinforcement must have sufficient capacity to transmit the vertical forces from the ledges to the web. The hangers should resist tension forces at the location of Crack 2 in Figure 2.2. Hanger capacity of the inverted-T bent caps is calculated and evaluated for both service limit state and strength limit state.



(a) Standard live load model *(b) Slab live load model*

(c) Live load model for girder reaction *(d) Girder reactions*

Figure 2.1. Live Load Models on Girder Used for the Computation of Girder Reaction.

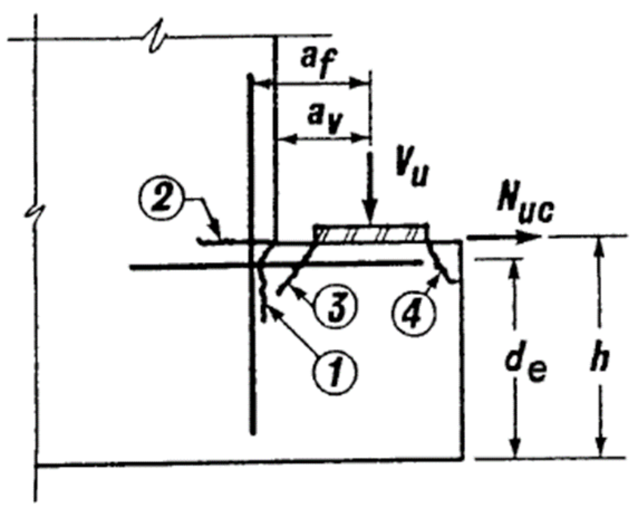
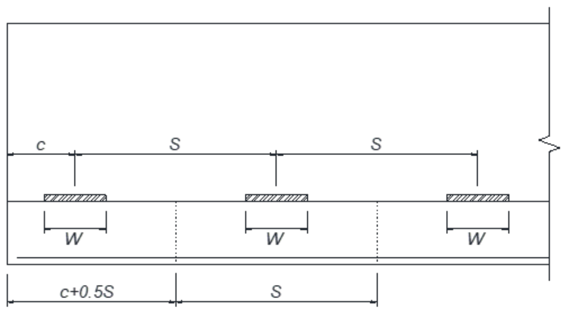
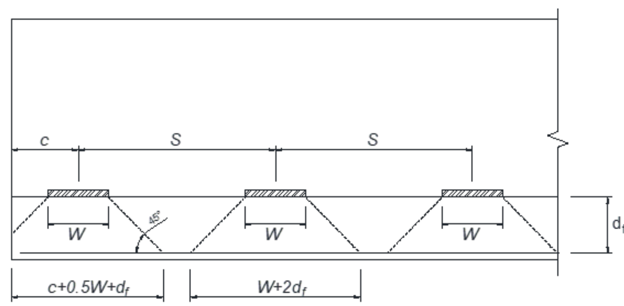


Figure 2.2. Notation and Potential Crack Locations for Ledge Beams (AASHTO, 2014).

The distribution width represents the length of the ledge considered capable of distributing the concentrated load longitudinally among the hanger reinforcements along the web. The longitudinal distance will be limited either by the longitudinal center-to-center girder spacing, *S*, which is shown in Figure 2.3(a), or by the capacity of the ledge to distribute the applied force to the hangers, also known as the flexural-shear resistance of the hangers. The latter is limited by the concrete shear capacity combined with the tensile capacity of the hangers within the distribution width of $W+2d_f$, as shown in Figure 2.3(b). For the service limit state, the distribution width is only *S*, which does not account for flexure-shear of the hanger, while the lesser of the capacity with the distribution width of *S* or $W+2d_f$ is taken for the strength limit state.



(a) Shear distribution width



(b) Flexural-shear distribution width

Figure 2.3. Parameters for Calculation of Hanger Capacity.

For the nominal shear resistance of the hanger at the service limit state, TxDOT uses $2/3f_y$ from the study of Furlong and Mirza (1974) instead of $0.5f_y$ from AASHTO LRFD (2014) Equation 5.1.2.5.5-1 (TxDOT, 2015). Thus, for this research, hanger capacity at the service limit state is the lesser of:

$$V_n = \frac{A_{hr}(\frac{2}{3}f_y)}{s}(W + 3a_v) \tag{2.2}$$

$$V_n = \frac{A_{hr}(\frac{2}{3}f_y)}{s}S \tag{2.3}$$

where A_{hr} = area of hanger reinforcement and s = spacing of hanger reinforcements.

For the strength limit state, the hanger capacity is the lesser of the following two AASHTO LRFD (2014) equations:

$$V_n = \frac{A_{hr}f_y}{s}S \tag{2.4}$$

$$V_n = 0.063\sqrt{f_c'}b_f d_f + \frac{A_{hr} f_y}{s}\left(W + 2d_f\right) \tag{2.5}$$

where b_f = width of the bottom flange.

For exterior girders, to consider the limitation of the distribution width to the edge of the cap, TxDOT provides modified equations for the shear resistance of exterior hangers. The exterior girder shear resistance for hanger at the service limit state is the lesser of:

$$V_n = \frac{A_{hr}(\frac{2}{3} f_y)}{s}\left(\frac{W + 3a_v}{2} + c\right) \tag{2.6}$$

$$V_n = \frac{A_{hr}(\frac{2}{3} f_y)}{s}\left(\frac{S}{2} + c\right) \tag{2.7}$$

For the strength limit state, the hanger resistance is taken as the lesser of the following two equations:

$$V_n = \frac{A_{hr} f_y}{s}\left(\frac{S}{2} + c\right) \tag{2.8}$$

$$V_n = 0.063\sqrt{f_c'}b_f d_f + \frac{A_{hr} f_y}{s}\left(\frac{W + 2d_f}{2} + c\right) \tag{2.9}$$

2.3.2 Evaluation of Ledge Shear Friction and Flexure Capacity

Figure 2.4 shows the reinforcement details of the inverted-T bent cap specified in AASHTO LRFD (2014). The top layer of the ledge reinforcement (red) is defined as the primary tension reinforcement, A_s, to sustain concurrent flexural-tension force at the face of the web. The remainder of the ledge reinforcement (blue) is defined as the auxiliary reinforcement, A_h, which only resists shear friction acting normal to the face of the web.

Nominal ledge shear friction (or interface shear) capacity for normal weight concrete is obtained using Equations 5.13.2.4.2-1 and 5.13.2.4.2-2 from AASHTO LRFD (2014). The ledge shear friction capacity is the lesser of:

$$C_s = min\begin{cases} V_n = 0.2f_c' b_w d_e & (2.10) \\ V_n = 0.8 b_w d_e & (2.11) \end{cases}$$

where f_c' = specified concrete strength; b_w = distribution width for the shear friction, as specified in Figure 2.5(a); c = distance from the center of the bearing pad to the end of the bent cap; W = width of the bearing pad; S = girder spacing; a_v = distance from the center of the bearing pad to the face of the web of the bent cap; and d_e = depth of the center of gravity of the negative flexural reinforcements, as shown in Figure 2.4.

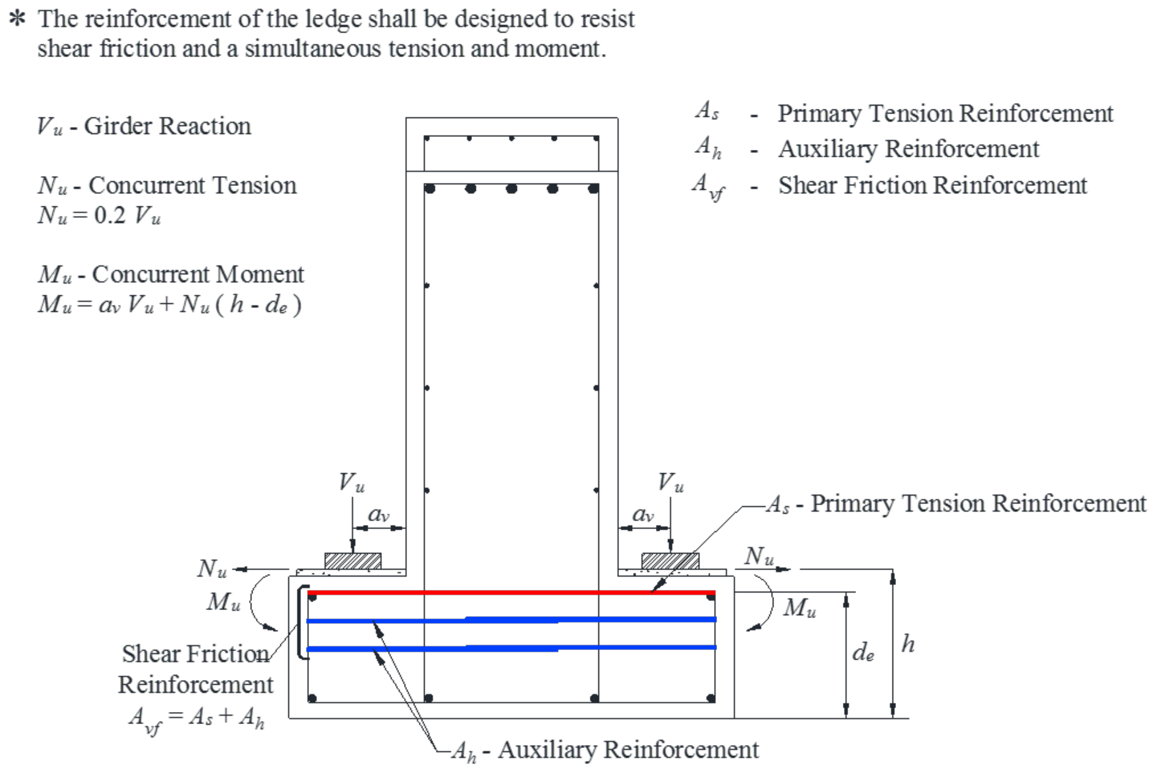
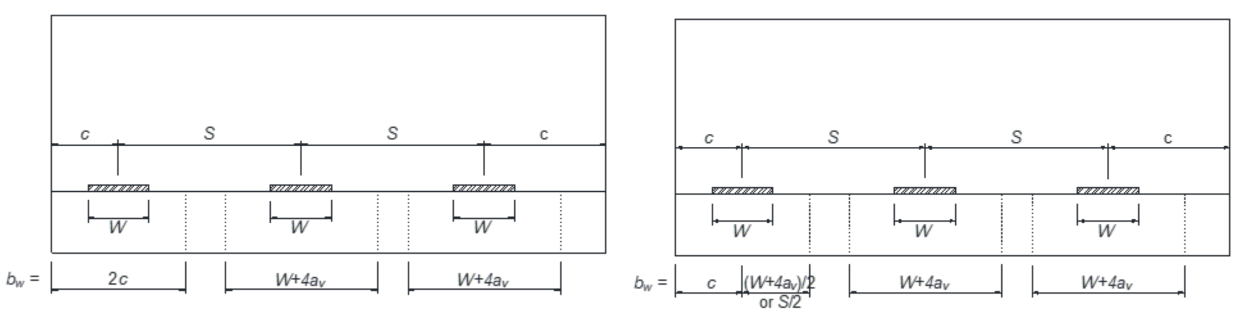


Figure 2.4. Ledge Reinforcements and Notations.

Figure 2.5 shows the distribution width, b_w, for shear friction. The AASHTO distribution width of the concrete assumed to participate in the resistance to interface shear friction is the lesser of S and (W+$4a_v$) for interior girders. For exterior girders, a rational modification shown in Figure 2.5(b) is proposed for identifying the ledge shear friction capacity of an inverted-T bent cap: the lesser of S, c+$S/2$, (W+$4a_v$), or c+(W+$4a_v$)/2 as shown in Figure 2.5(b). This proposed modification is based on test results reported by Hurlebaus et al. (2018b).



(a) AASHTO distribution widths *(b) Proposed distribution widths*

Figure 2.5. Parameters for Calculation of Ledge Shear Friction Capacity.

The ledge must simultaneously resist a factored girder reaction force, V_u, a factored concurrent horizontal tensile force, N_u, and a factored concurrent moment, M_u. The concurrent horizontal tensile force is regarded as a live load (AASHTO, 2014) and determined by:

$$N_u = 0.2V_u \tag{2.12}$$

The factored concurrent moment M_u is determined using:

$$M_u = V_u a_v + N_u(h - d_e) \tag{2.13}$$

where h = depth of the ledge; and d_e = effective depth of the ledge from the extreme compression fiber to the centroid of the tensile force N_u.

Based on Article 3.7.3.2 of AASHTO LRFD (2014), the nominal flexural resistance of the ledge section should be taken as:

$$M_n = A_s f_y \left(d_e - \frac{a}{2}\right) \tag{2.14}$$

where A_s = area of ledge flexure reinforcement specified in Figure 2.4; f_y = yield stress of ledge flexure reinforcement; and a = depth of the equivalent stress block.

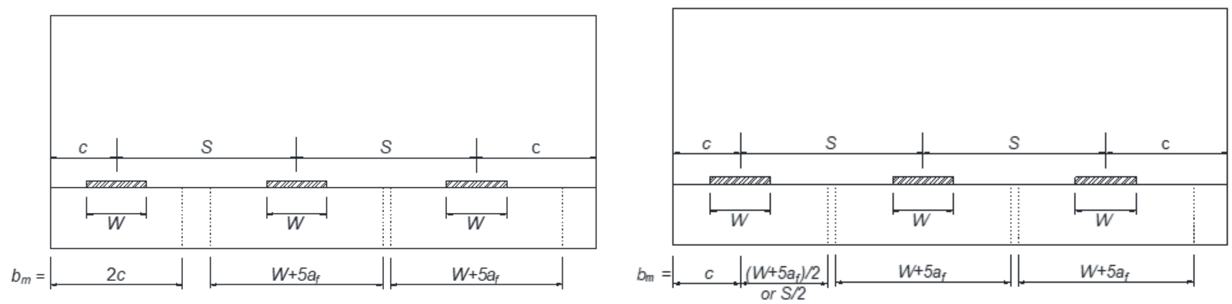
The depth of the equivalent stress block, a, should take account the concurrent horizontal axial tension, N_u, since it already exists. This axial force increases the depth of the equivalent stress block, a, and decreases the ledge flexure capacity based on the equilibrium equation:

$$\frac{N_u}{\phi} + A_s f_y = 0.85 f_c' a b_m \tag{2.15}$$

where b_m = distribution width for ledge flexure and axial tension, as shown in Figure 2.6(a); and a_f = distance from the center of the bearing pad to the center of the nearest stirrup.
Therefore, the depth of the equivalent stress block, a, with axial tension is obtained by:

$$a = \frac{\frac{N_u}{\phi} + A_s f_y}{0.85 f_c' b_m} \tag{2.16}$$

The AASHTO distribution width is taken as the lesser of S and (W+5a_f) for interior girders. A rational modification of the distribution width for exterior ledge flexure is proposed as the lesser of S, $c+S/2$, ($W+5a_f$), and $c+(W+5a_f)/2$, as shown in Figure 2.6(b). The proposed distribution widths are verified based on the test results presented by Hurlebaus et al. (2018b).



(a) AASHTO distribution widths

(b) Proposed distribution widths

Figure 2.6. Parameters for Calculation of Ledge Flexure with Axial Tension Capacity.

2.3.3 Evaluation of Punching Shear and Bearing Capacity

Punching failure can occur if the girder reactions are sufficient enough to punch out a truncated pyramid of concrete beneath the bearing pad. The area of the truncated pyramid shown in Figure 2.7 is approximated as the average of the perimeter of the bearing pad and the perimeter at depth, d_f, assuming 45-degree slopes in the *Bridge Design Manual—LRFD* (BDM-LRFD; TxDOT, 2015).

Based on the punching shear tests, the truncated pyramids were shaped with 35-degree angles, which was the average of all the tests for both exterior and interior ledges as described by Hurlebaus et al. (2018b). Therefore, the rational modified punching shear capacity equations are proposed with the measured angle. Since TxDOT equations use a 45-degree slope for the area of the truncated pyramid as the average of the perimeter of the bearing pad, the modified equations are obtained by multiplying cot(35°) = 1.43 by d_f for the perimeter as follows:

$$V_n = 0.125\sqrt{f_c'}\left(W + 2L + 2d_f \cot(35°)\right)d_f \tag{2.17a}$$

$$= 0.125\sqrt{f_c'}\left(W + 2L + 2.86d_f\right)d_f \tag{2.17b}$$

$$V_n = 0.125\sqrt{f_c'}\left(\frac{W}{2} + L + d_f \cot(35°) + C\right)d_f \tag{2.18a}$$

$$= 0.125\sqrt{f_c'}\left(\frac{W}{2} + L + 1.43d_f + C\right)d_f \tag{2.18b}$$

where L = length of the bearing pad. For interior girder location, Equation (2.17a) is used. For exterior girder location, the lesser of Equation (2.17a) or (2.18a) is used. When c is less than $W/2 + L + d_f$, Equation (2.18a) controls the punching shear capacity.

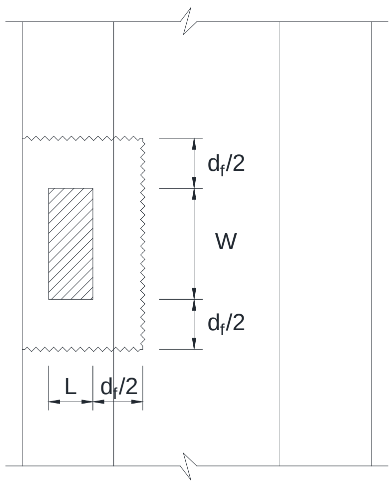
The ledge of the inverted-T bent cap should have sufficient bearing capacity to resist the load on the bearing pad at the location of Crack 4 shown in Figure 2.2. The load on the bearing pad distributes along a truncated pyramid, as shown in Figure 2.8. The ledge of the bent cap should have bearing resistance as described in Article 5.7.5 of AASHTO LRFD (2014). The bearing capacity of the ledge can be obtained by:

$$V_n = 0.85 f_c' A_1 m \tag{2.19}$$

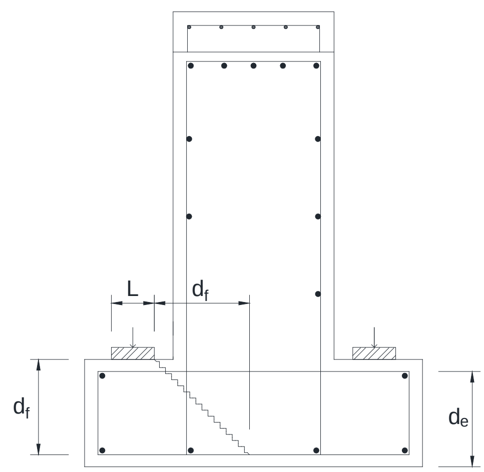
where A_1 = area under bearing device; and m = modification factor, which is taken as the lesser of:

$$m = 2;$$
$$m = \sqrt{\frac{A_2}{A_1}} \tag{2.20}$$

where A_2 = projected bearing area, as shown in Figure 2.8, which is described in Article 5.7.5 of AASHTO LRFD (2014).



(a) Plan view



(b) Cross-section

Figure 2.7. Punching Shear Failure Surface.

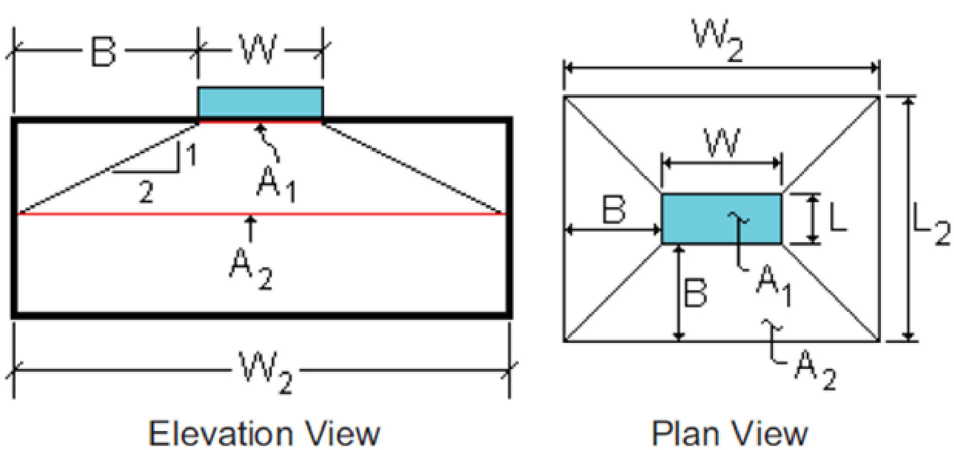


Figure 2.8. Truncated Pyramid for Bearing (TxDOT, 2010).

2.4 Bent Caps for Design Examples

To demonstrate implementation of the design procedures, two bent cap design scenarios are considered:

- A double-column bent that has hanger, ledge flexure, and punching shear deficiencies (Bent 13).
- A single-column bent that has critical deficiencies at the exterior girder locations (Bent 22).

Bent 13 is the typical asymmetric double-column bent analyzed by Hurlebaus et al. (2018a). Deficiencies are provided in Table 2.1. Both exterior and interior girder locations have hanger and punching shear deficiency, while ledge flexure deficiency is found only for the interior girder location with increased traffic loads.

Bent 22 is a typical single-column bent analyzed by Hurlebaus et al. (2018a). Deficiencies are provided in Table 2.2. Both exterior and interior girder locations have hanger deficiency. No other deficiencies are present.

The details of capacity calculations for each bent are presented in Appendix B. For Bent 13, all solutions except load-balancing PT (Solution 14) are designed to address the deficiencies, as described in Appendix C. All six solutions are designed to provide sufficient strength for the deficiencies in Bent 22, as presented in Appendix D.

Table 2.1. Load Summary of Bent 13.

Failure Mode	Girder No.	Capacity C (kips)	Demand D (kips)	Deficiency $D/\phi - C$ (kips)
Hanger	Ext. 1	198	247	76
	Ext. 2	198	247	76
	Int. 1			*
	Int. 2	229	287	90
	Int. 3	229	287	90
	Int. 4			*
	Int. 5	394	287	n/a
Ledge shear friction	Ext. 1	599	247	n/a
	Ext. 2	599	247	n/a
	Int. 1	643	287	n/a
	Int. 2	643	287	n/a
	Int. 3	643	287	n/a
	Int. 4	643	287	n/a
	Int. 5	643	287	n/a
Ledge flexure	Ext. 1	297	247	n/a
	Ext. 2	297	247	n/a
	Int. 1	299	287	20
	Int. 2	299	287	20
	Int. 3	299	287	20
	Int. 4	299	287	20
	Int. 5	299	287	20
Punching shear	Ext.	261	247	13
	Int.	345	287	n/a
Bearing	Ext.	934	247	n/a
	Int.	934	287	n/a

** Girder located over column; need for hanger reinforcement is bypassed.*

Table 2.2. Load Summary of Bent 22.

Failure Mode	Girder No.	Capacity C (kips)	Demand D (kips)	Deficiency $D/\phi - C$ (kips)
Hanger	Ext. 1	214	207	16
	Ext. 2	214	207	16
	Int. 1	227	235	34
	Int. 2	346	235	n/a
	Int. 3			*
	Int. 4	346	235	n/a
	Int. 5	227	235	34
Ledge shear friction	Ext. 1	575	207	n/a
	Ext. 2	575	207	n/a
	Int. 1	911	235	n/a
	Int. 2	1129	235	n/a
	Int. 3	1530	235	n/a
	Int. 4	1129	235	n/a
	Int. 5	911	235	n/a
Ledge flexure	Ext. 1	287	207	n/a
	Ext. 2	287	207	n/a
	Int. 1	480	235	n/a
	Int. 2	598	235	n/a
	Int. 3	625	235	n/a
	Int. 4	598	235	n/a
	Int. 5	480	235	n/a
Punching shear	Ext.	237	207	n/a
	Int. 1 &5	494	235	n/a
Bearing	Ext.	937	207	n/a
	Int.	937	235	n/a

** Girder located over column; need for hanger reinforcement is bypassed.*

CHAPTER 3: SELECTION OF RETROFIT SOLUTION

In this chapter, recommendations are provided to guide engineers in selecting a retrofit solution to use for strengthening in-service inverted-T bent caps. Three general selection criteria should be considered: deficiencies addressed, obstacles, and costs. Here, a general discussion is provided for selection of the six retrofit solutions addressed in this report: end-region stiffener (Solution 3), clamped threadbar with channel (Solution 8), load-balancing PT (Solution 14), concrete infill with partial- and full-depth FRP anchored by steel waling (Solutions 16 and 17), and large bearing pad (Solution 18). Full details on all selection criteria, application to 18 retrofit solutions, and demonstration of a method for ranking solutions are provided by Hurlebaus et al. (2018a).

3.1 Deficiency Type and Location

The key selection criterion for retrofit solutions is the ability to effectively strengthen all deficiencies. While it is necessary to check all failure modes at all girders, deficiencies are most likely to be found in the hanger capacity, ledge flexure capacity, and/or punching shear capacity. Table 3.1 indicates the failure mode and girder locations strengthened by each solution.

Table 3.1. Deficiencies Addressed by Retrofit Solution.

Solution No.	Description of Retrofit Solution	Failure Mode Strengthened			Girder Locations	
		Hanger	Ledge Flexure	Punching Shear	Ext.	Int.
3	End-region stiffener	X	X	X	X	
8	Clamped threadbar with channel	X	X	X	X	X
14	Load-balancing PT	X	X	X	X	X
16	Concrete infill with partial-depth FRP anchored by steel waling		X	X	X	X
17	Concrete infill with full-depth FRP anchored by steel waling	X	X	X	X	X
18	Large bearing pad			X	X	X

If deficiencies are restricted to the ledge (ledge shear friction, ledge flexure, and/or punching shear deficiency), recommended solutions are large bearing pad (Solution 18), partial-depth FRP (Solution 16), and load-balancing PT (Solution 14). Solution 18 only addresses punching shear deficiency.

If ledge and hanger deficiencies are present, all solutions except Solution 16 and 18 are viable. If the hanger deficiency is dominant, the recommended solutions are clamped threadbar (Solution 8), full-depth FRP (Solution 17), and end-region stiffener (Solution 3) since ledge strengthening is secondary for these solutions.

Although Solution 3 is only applicable for exterior girders, it may be useful in combination with other solutions. Most solutions considered address deficiencies at individual girder locations. If deficiencies are identified at many girders, Solution 14 may be an attractive option because it provides an alternative load path, effectively reducing the demand on the ledges at all locations.

3.2 Obstacles

Selection of a solution to use will ultimately by affected by obstacles encountered in implementing the solution and the potential obstacles introduced by the solution. Table 3.2 lists the obstacles that affect selecting retrofit solutions and their level for each subcategory.

Table 3.2. Obstacles to Consider in Selection of Retrofit Solution.

Solution No.	Accessibility	Web Clearance Needed	Reinforcement Risk	Reduced Clearance
3	Ends	No	High	No
8	Above & below	No	High	Yes
14	Ends & below	Yes	Low	No
16	Below	No	Low	No
17	Below	Yes	Low	No
18	Below	No	None	No

Installation of a solution requires access to the bent cap that may be affected by adjacent structural components and/or roadways. The second column of Table 3.2 indicates the access required to install retrofit components.

While all solutions require access from below, only Solution 8 requires top access that would require lane closure. Implementation of Solutions 3, 16, and 17 at the exterior girder would require access from the end, while accessibility to the end is required to implement the anchor plates for Solution 14 at any location. The third column of Table 3.2 indicates solutions that require a minimum clearance adjacent to the web. Bridges with diaphragms between girders may prohibit installation of full-depth FRP (Solution 17). Load-balancing PT (Solution 14) is not practical when the clearance between the web and the girder ends cannot accommodate the required amount of prestressing.

Many solutions require drilling holes to install anchors or threadbars; therefore, potential risk of encountering internal reinforcement must be considered. The fourth column of Table 3.2 indicates this risk level.

Finally, protrusion of retrofit components may reduce clearance below the bent cap. Of the six solutions considered here, clearance reduction is minor for all but Solution 8, which has a reduction equal to the channel depth.

3.3 Costs

Costs, both initial and life cycle, are another driving factor in selection of retrofit solution. Table 3.3 lists the obstacles that affect selecting retrofit solutions and their level for each subcategory.

Initial costs include the construction and lane closure costs. Although construction costs (materials, equipment, and labor) will be dependent on individual project characteristics, the cost estimates of preliminary designs (Hurlebaus et al., 2018a) are shown in the second column of Table 3.3 to provide guidance. In addition to construction costs, initial costs can be considered to include the indirect costs of lane closure. The third column of Table 3.3 indicates the need for no, minor, or major lane closures below and above, with major closures considered to be seven days or longer.

While difference in initial costs of some solutions may be significant, it is important to consider the full life-cycle costs of each solution. The fourth column of Table 3.3 indicates the lowest level inspection methods required to adequately assess the condition of each retrofit. The final column indicates the primary durability concern that may impact the service life of the retrofit.

Table 3.3. Costs to Consider in Selection of Retrofit Solution.

Solution No.	Construction Costs	Lane Closure	Ease of Monitoring	Durability
3	$10K	Below (minor)	Visual inspection using a lift	Corrosion/ debonding
8	$19K	Below (major) Above (major)	Borescope testing	Corrosion
14	$24K	Below (minor) Above (minor)	Visual inspection using a lift	Corrosion
16	$35K	Below (major)	Borescope testing	Debonding
17	$39K	Below (major) Above (minor)	Inspection using nondestructive testing	Debonding
18	$6K	Below (minor) Above (minor)	Visual inspection using a lift	None

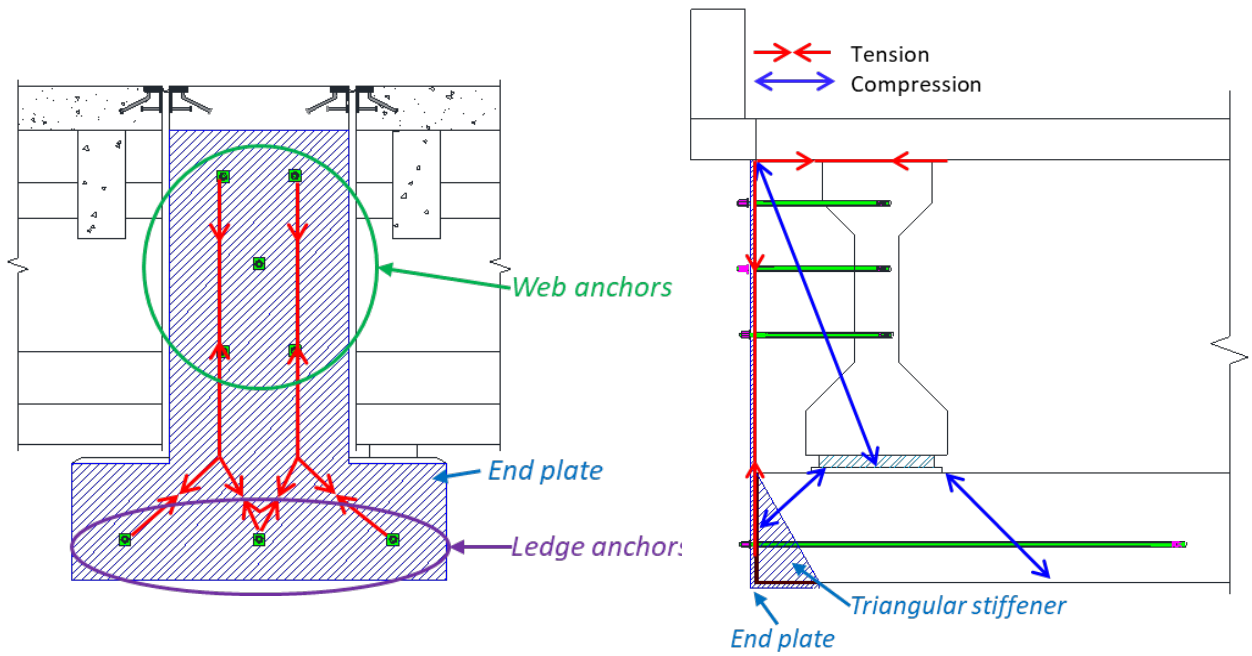
3.4 Closure

Retrofit selection should be based primarily on the deficiencies of in-service inverted-T bent caps, with additional consideration for obstacles to implementation, initial costs, and life-cycle costs. Ultimately, selection will depend on the unique characteristics of a project (bent configuration, location, and bridge purpose), typical practices of local jurisdictions, and importance assigned to each selection criterion. Expanded details on all selection criteria, application to a wider range of retrofit solutions, and an example of a methodology to select appropriate solutions are provided by Hurlebaus et al. (2018a).

CHAPTER 4: END-REGION STIFFENER (SOLUTION 3)

The end-region stiffener retrofit solution (Solution 3) is designed to increase hanger, ledge flexure, and punching shear capacity at the end region of the inverted-T bent cap. Figure 4.1 shows a schematic overview of the solution with load paths.

As described by Hurlebaus et al. (2018b), the end region of an inverted-T bent cap with this solution failed because of stress exceedance at the strut-to-node interface. This is because the end plate could not resist the concentrated node stress at the bottom tip of the plates. The proposed design procedure incorporates detailing recommendations to avoid such as failure.



(a) End face *(b) Side elevation*

Figure 4.1. Components and Load Path for End-Region Stiffener.

4.1 Design Procedures

The required design procedure for the end-region stiffener are detailed below. An in-depth explanation is presented in the following subsections:

- Step 1: Specify anchors (web and ledge).
- Step 2: Design steel plate.

4.1.1 Step 1: Specify Anchors

The required strengths for the anchors are obtained using the relevant strength reduction factor for concrete at the anchorage zone. Based on the required strength, the type of anchor can be selected. In general, adhesive anchors, such as epoxy anchors, provide relatively high shear strength with longer embedded depth compared to mechanical anchors. Thus, use of adhesive anchors is recommended to anchor the end plate.

As shown in Figure 4.1(a), the anchors are categorized in two groups (web and ledge) for design purposes. In the following subsection, the design procedure of each anchor group is described.

4.1.1.1 Web Anchors

Web anchors (green circle in Figure 4.1[a]) must provide shear capacity greater than or equal to the hanger deficiency. Anchors must be designed for minimum steel strength, concrete breakout strength, and concrete pryout strength based on ACI 318-14 (ACI Committee 318, 2014). Bond strength of the anchors is not considered when the anchors are under shear. Generally, the anchor shear strength will control the design strength, in which case the number and type of anchors should be selected based on:

$$V_n = 0.6 f_u A_b n_w > V_{h,req} \tag{4.1}$$

where f_u = ultimate strength of the anchor steel; A_b = net area of the anchor; n_w = number of web anchors; and $V_{h,req}$ = hanger deficiency.

The anchor layout is determined based on minimum spacing of anchors, minimum edge distance of anchors, and existing reinforcement in the bent cap as follows:

$$s_{min} = 6d_a \tag{4.2}$$

$$c_{min} = \max(1.5\,, 6d_a) \tag{4.3}$$

where s_{min} = minimum spacing of anchors; d_a = anchor diameter; and c_{min} = minimum edge distance.

4.1.1.2 Ledge Anchors

The ledge anchors (purple circle in Figure 4.1[a]) should be designed to resist deficient ledge capacity: ledge shear friction, ledge flexure, and/or punching shear. Since the anchors are under a combination of shear and tension, the design strength of the anchors is minimum steel strength, concrete breakout strength, bond strength (only for adhesive anchors), and concrete pryout strength based on ACI 318-14 (ACI Committee 318, 2014). To anchor efficiently, the number of anchors should be a minimum of three, and they should be anchored at the bottom kern point of the ledge.

To avoid the strut-and-tie node failures observed in laboratory tests (Hurlebaus et al., 2018b), ledge anchors should be anchored beyond the region of significant ledge damage beneath the bearing pad. The full embedded depth of the ledge anchors, d_e, as shown in Figure 4.2, is calculated as:

$$d_e \geq c + \frac{W}{2} + h + h_{ef} \tag{4.4}$$

where d_e = embedded depth of ledge anchor; c = distance from the center of the bearing pad to the end of the bent cap; W = width of the bearing pad; h = ledge height; and h_{ef} = minimum effective embedded depth specified by the manufacturer.

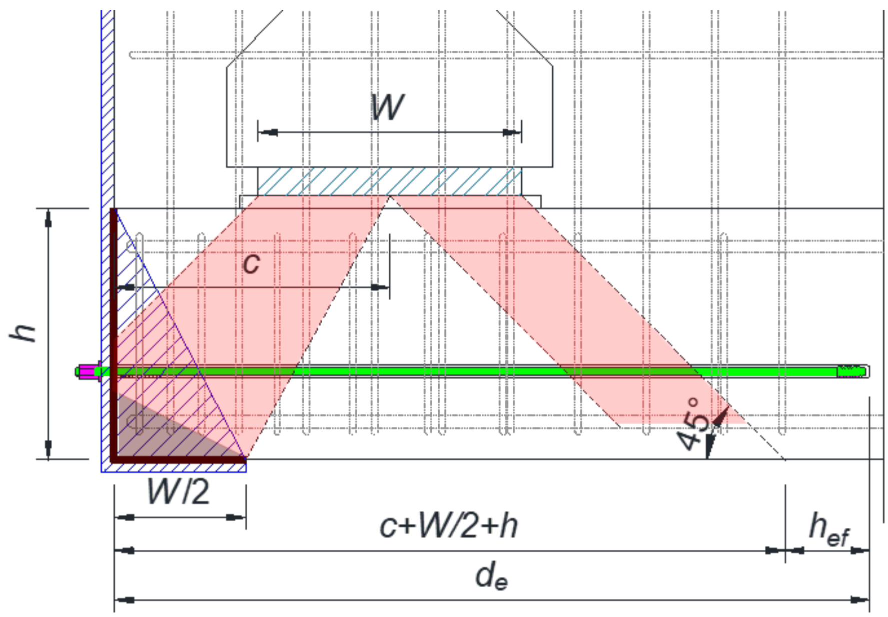


Figure 4.2. Details of Ledge for End-Region Stiffener.

For calculating the ledge anchor capacity, only the length h_{ef} beyond the region of significant damage is used. Similar to the web anchors, the shear strength of the steel anchors is less than other strengths, and the type of anchor should be chosen based on the shear strength of steel for the anchor. However, using anchors with a smaller diameter than the web anchors is recommended since an anchor hole for a ledge anchor is deeper than a hole for a web anchor.

Since the ledge anchors resist both shear and tension force, the capacity for interaction of shear and tension should be checked by:

$$\left(\frac{N_u}{\phi N_n}\right)^{\varsigma} + \left(\frac{V_u}{\phi V_n}\right)^{\varsigma} \leq 1.0 \tag{4.5}$$

where N_u and V_u = required tension and shear force, respectively; ϕN_n and ϕV_n = tension and shear design strength of anchor, respectively; and ς = 0.6.

4.1.2 Step 2: Design Steel Plate

As shown in Figure 4.1(b), the steel plate is fabricated in the shape of the cross-section. The bottom length of the steel plate, the extension underneath the bent, should have sufficient length, which must be longer than the projected one-half of the bearing length in 45 degrees (shaded area in Figure 4.2) to provide sufficient resistance to node stress.

A maximum thickness of 1 in. is recommended to permit bending at the bottom and to ensure the weight is manageable for installation. If the minimum required thickness is larger than 1 in., extra bearing plates are required on the anchors. The minimum required thickness of the plate is primarily controlled by shear and axial bearing force of the anchors (AISC, 2010):

$$t_{s,bearing} \geq \frac{V_u}{\phi 2.4 d_b f_u} \tag{4.6}$$

$$t_{a,bearing} \geq \frac{V_u}{\phi 2.0 d_b f_u} \tag{4.7}$$

where $t_{s,bearing}$ and $t_{a,bearing}$ = minimum thickness for shear and axial bearing strength, respectively; ϕ = strength reduction factor for LRFD; d_b = diameter of the bolt; and f_u = ultimate strength of the plate.

To avoid stress exceedance at the strut-to-node interface, triangular stiffeners (shown in Figure 4.1[a]) are recommended. The triangular stiffener can be designed using “Design Aid for Triangular Bracket Plates Using AISC Specifications” (Shakya and Vinnakota, 2008) to select the aspect ratio of the stiffener (ratio of the height, *a*, to the width, *b*), the required strength, and the minimum thickness of the plate. Shakya and Vinnakota (2008) provided tables to determine the minimum ratio of the thickness to the width based on steel yield strength, F_y. The triangular stiffeners should be attached by welds designed based on AISC (2010) specifications.

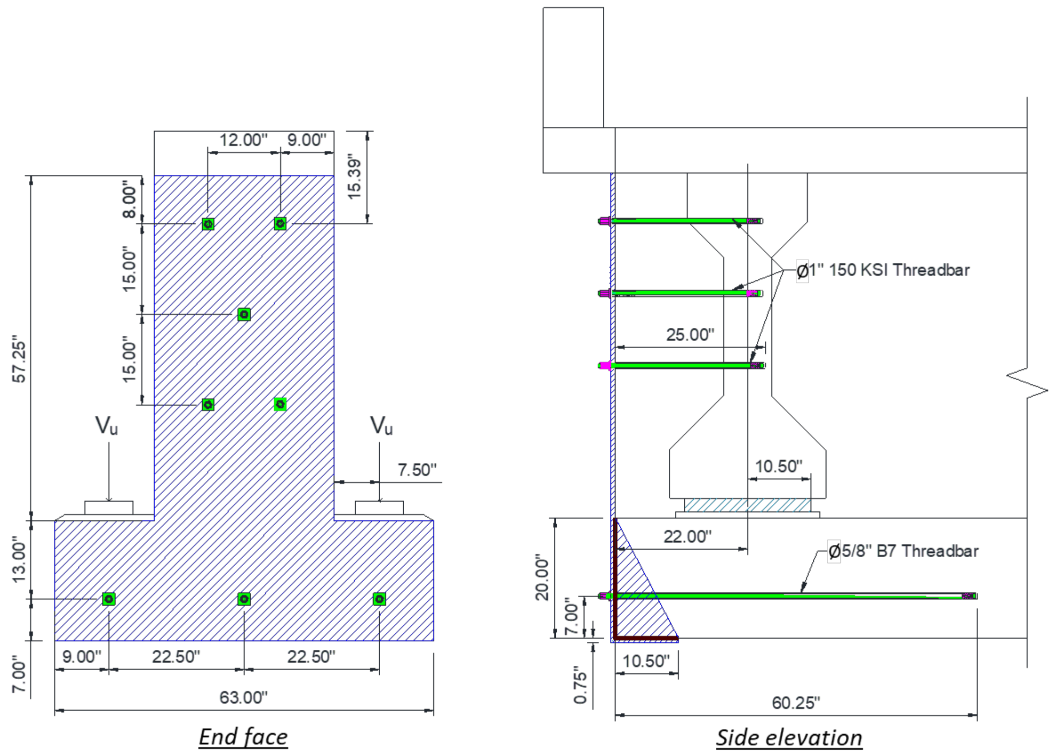
4.2 Discussion of Design Example

An end-region stiffener is an effective retrofit to increase hanger, ledge flexure, and punching shear capacities at an exterior girder seating region. Since both the double- and single-column bents described in Chapter 2 have deficiencies at the exterior girder, this solution is developed for both bent types; the design calculations are presented in Section B.1 and C.1 of the appendices, respectively. Figure 4.3 shows the details of the end-region stiffener for the example bents. Table 4.1 summarizes the increased capacity at the exterior girders.

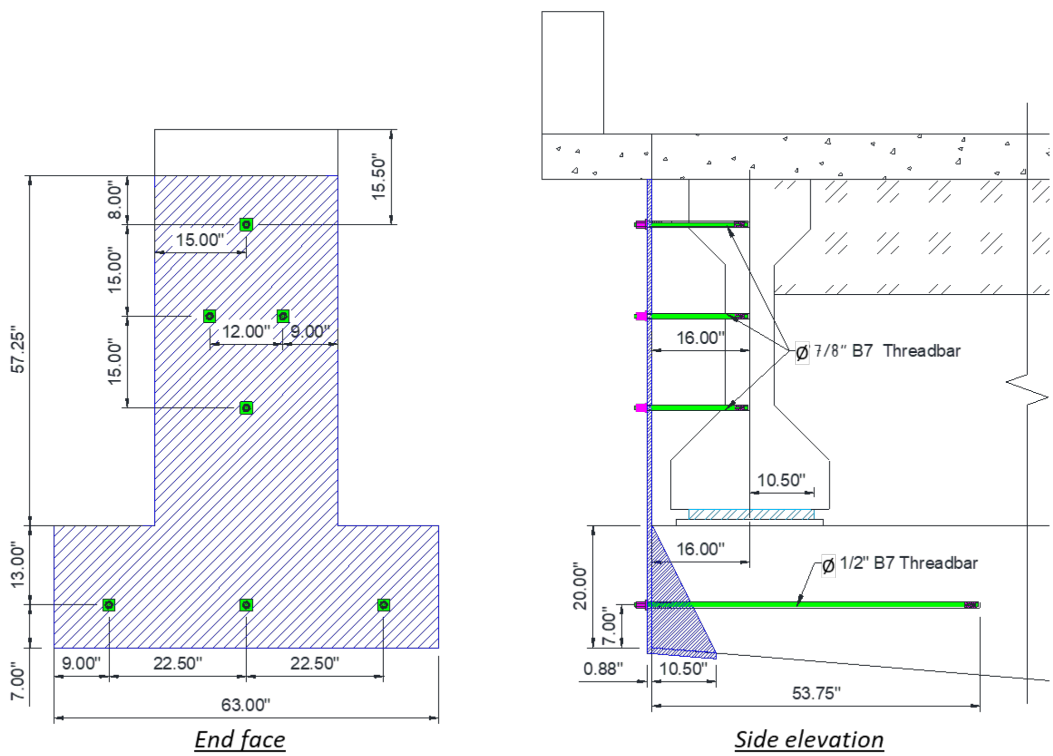
A sample Williams epoxy anchor, composed of an All-threadbar and Ultrabond adhesive, is used to develop the design example. The epoxy is designed for heavy anchoring with the maximum in-sevice temperature of 134°F and 1.64 ksi bond strength for a 14-day cure.

For the double-column bent, the hanger deficiency is the largest. However, ledge and punching shear deficiencies also exist. To provide the required increase in the hanger capacity, five 1 in. diameter 150 ksi threadbars are used. The anchor selection is controlled by the ledge flexure deficiency. The smallest diameter holes are desired to avoid conflict with internal reinforcement, leading to three 0.625 in. diameter B7 threadbars. Design of the plate thickness is controlled by shear capacity of the web anchors, requiring selection of a 0.75 in. thick plate. Use of a 10.5 in. bottom extension with 0.5 in. stiffener is determined by the geometry of the ledge and the location of the bearing pad. The solution designed for the double-column bent increases the capacity of the bent by 40 percent, with the overstrength factor increase from 0.72 to 1.12.

The single-column bent largest deficiency is hanger (smaller than the double-column bent), with a small punching shear deficiency also present at the anchorage zone. To provide the required increase in the hanger capacity, four 0.875 in. diameter B7 thread rods are used. Ledge anchor selection, controlled by the punching shear deficiency, is three 0.5 in diameter B7 thread rods. Design of the plate thickness is controlled by bearing capacity of the plate for the web anchors, leading to selection of a 1 in. thick plate. Use of a 10.5 in. bottom extension with 0.5 in. stiffener is determined by the geometry of the ledge and the location of the bearing pad. The solution provides an 11 percent increase in hanger capacity. The overstrength factor is increased from 0.93 to 1.04.



(a) For double-column bent



(b) For single-column bent

Figure 4.3. Details of End-Region Stiffener for Design Examples.

Table 4.1. Capacity Increase from End-Region Stiffener Retrofit.

Bent Type	Capacity ϕC (kip)		Demand D (kip)	Overstrength Factor $\phi C/D$
Double Column	Original	178	247	0.72
	Retrofitted	276		1.12
Single Column	Original	193	207	0.93
	Retrofitted	216		1.04

Note: ϕ = strength reduction factor, 0.9.

4.3 Limitations and Recommendations for Construction

Since the steel plate is attached on the end face of the bent caps, the end face must be accessible to drill the holes and to anchor the steel plate. Potential accessibility challenges are (a) adjacent structures such as those at the north end of the elevated lanes of IH 35 in Austin, Texas; and (b) adjacent traffic for which lane closure would have a serious impact on the traveling public. Additional details are documented by Hurlebaus et al. (2018a).

To place the end plate, the end surface of the bent needs to be cleared. In addition, covering the surface with a grout that allows application on a vertical surface to fill the gap between the plate and the end surface is recommended. The end plate needs to be placed immediately following application of the grout. Thus, the end plate should be lifted and ready to install before finishing grouting the end surface. The anchors must be fastened before the grout is completely hardened to anchor the plate effectively.

Since there are several layers of ledge reinforcement, using a rebar indicator prior to deciding anchor hole locations on the ledge to avoid the existing ledge reinforcement is recommended. This is because drilling through a rebar is time consuming and not efficient in both structural behavior and constructability. If there is no sufficient gap between the adjacent ledge reinforcement so that a hole for an anchor needs to be drilled through a ledge reinforcement, the engineers may need to decide the anchor locations that can minimize the damage on the ledge reinforcement.

Adhesive anchors require time to harden, as specified by the anchor manufacturer. Thus, the end plate needs to be implemented after the anchors are fully cured.

CHAPTER 5: CLAMPED THREADBAR WITH CHANNEL (SOLUTION 8)

Figure 5.1 shows the retrofit solution using long threadbars embedded in the web of the inverted-T bent cap that may be deficient in hanger capacity. If ledge deficiencies need to be addressed, steel channels can be used. Since threadbars within the web act as hanger reinforcement, this solution transfers the loads from the ledge into the web via a set of threadbars, while steel channels resist ledge shear friction and flexural forces that are generated by the girders. The threadbars are torqued to induce prestress so that the prestressing force should inhibit cracking in the web and ledges.

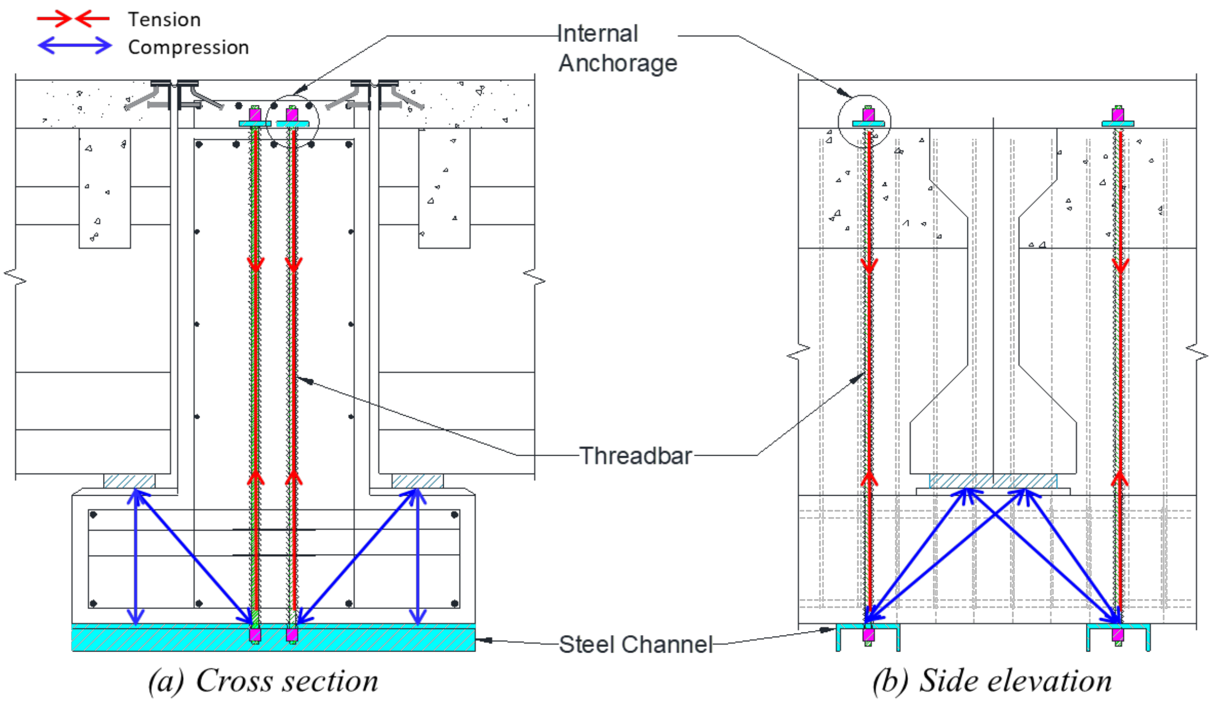


Figure 5.1. Overview of Clamped Threadbar with Steel Channel to Transfer Loads into Web.

5.1 Design Procedures

The required steps for designing a clamped threadbar with steel channel are detailed below. An in-depth explanation is presented in the following subsections:

- Step 1: Specify hanger threadbar.
- Step 2: Design bearing plate.
- Step 3: Specify steel channel (if necessary).

5.1.1 Step 1: Specify Hanger Threadbar

The threadbars anchored at the web must have sufficient strength to transfer the loads from the ledge into the web. Thus, the required strength of a hanger threadbar is the calculated hanger deficiency. Use of a high-strength threadbar with 150 ksi ultimate strength is recommended as the hanger threadbar because it allows for prestressing. With the required strength and known yield strength of the threadbar, F_y, the required area of the threadbar, A_{req}, can be obtained by:

$$A_{req} = \frac{V_{h,req}}{F_y} \tag{5.1}$$

where $V_{h,req}$ = hanger deficiency.

The minimum number of threadbars can be calculated from the known area of a chosen threadbar. While increasing the bar size, the number of bars can be reduced, but increased difficulty in boring holes results. If threadbars cannot be placed within the center of the web, an even number of threadbars is required to avoid an uneven contribution of bars on each side of the web.

To increase hanger capacity only, threadbars need to be placed as close to the center of the girder location as possible. If the solution is used with channels to increase punching shear capacity, placement of hangers must consider expected failure plane in the ledge.

5.1.2 Step 2: Design Bearing Plate

Since the threadbar is torqued when it is installed, the bearing plate must be able to resist bearing force due to prestressing. Use of a hex nut is recommended along with a washer and bearing plate. Dimensions for the washer and hex nut are generally provided by the threadbar manufacturer. The size of the bearing plate must be determined by the required bearing area and thickness. The required thickness of the bearing plate can be determined by Equation (4.7), with the required bearing area given by:

$$A_{b,req} \geq \frac{P_b}{\phi_c 0.85 f_c'} \tag{5.2}$$

where P_b = prestressing force induced at the installation in kip; ϕ_c = strength reduction factor for LRFD, 0.65; and f'_c = specified concrete strength in ksi.

While a square or circular plate is recommended, a rectangular plate may be used to avoid interference with existing reinforcement in the deck slab region (see Figure 5.1[a]).

5.1.3 Step 3: Specify Steel Channel

If ledge deficiencies are present, channels should be incorporated into the retrofit solution. To ensure the proper flow of forces (shown in Figure 5.1[b]), channels must be located at the location of the hanger threadbar.

The channels are placed to bend about the minor axis; thus, a compactness check is required (see AISC Specification Table B4.1b [2010]). Since the width-to-thickness ratio of MC type channels is less than the limit state of compact/noncompact, MC channels are designated as compact sections. For the channel with a compact section, yielding controls the capacity of the channel since no flange or web local buckling is expected. Thus, the channel will be primarily selected based on the web thickness, which must meet the thickness requirements obtained using Equation (4.7).

The channel should also have greater section modulus than the required section modulus. The required elastic and plastic section moduli about the minor axis are calculated based on the required flexural strength, which is the ledge flexure deficiency:

$$S_{y,req} \geq \frac{M_d/n_c}{1.6F_y} \tag{5.3}$$

$$Z_{y,req} \geq \frac{M_d/n_c}{F_y} \tag{5.4}$$

where M_d = ledge flexure deficiency; n_c = number of channels; F_y = yield strength of the steel channel in ksi; $Z_{y,req}$ = required plastic modulus of the channel section; and $S_{y,req}$ = required elastic modulus of the section.

The maximum of the required elastic or plastic section modulus governs the channel selection.

5.2 Discussion of Design Example

The retrofit solution is developed for the double- and single-column bent caps (Bent 13 and Bent 22) in Section B.2 and C.2 of the appendices, respectively. Table 5.1 provides the capacity increase of the bent caps for design examples under ultimate load. For hanger capacity increase, a high-strength threadbar is used, and a standard steel channel is used to increase ledge flexure capacity. Figure 5.2 shows the details of the clamped threadbar with channel for example bents.

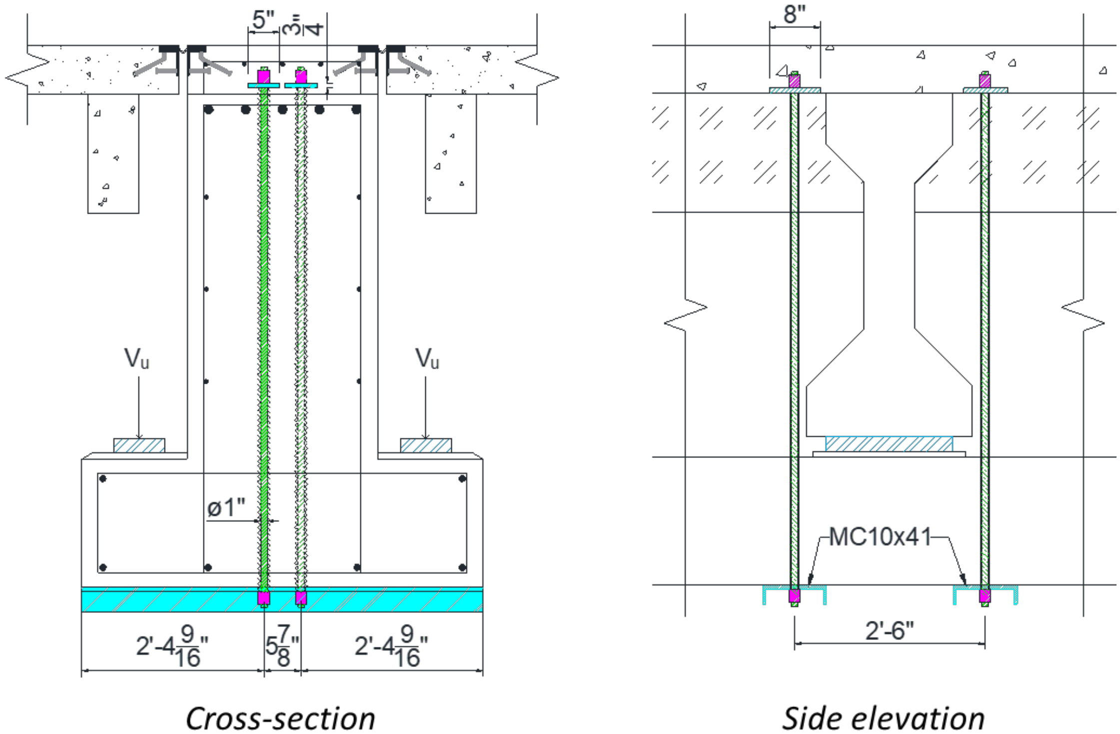
The double-column bent example maximum deficiency is the hanger capacity. To increase hanger capacity, two 1 in. 150 ksi high-strength threadbars are used for both interior and exterior. The alignment of the threadbars is staggered to avoid clashing with the longitudinal reinforcing bars. To increase ledge flexure capacity, two C 10 x 30 steel channels are used at interior girders, while no channels are needed for the exterior portion of the cap since there is no ledge flexure deficiency. The internal anchors at the top for the threadbars consist of a 5 in. x 8 in. x 0.75 in. rectangular plate with washer and hex nut as specified by the manufacturer.

For the single-column bent that only has hanger deficiencies at both interior and exterior girders, B7 threadbars are used. To avoid interference with the longitudinal reinforcement, an even number of threadbars is required. This results in use of different diameters for the threadbars at exterior and interior girders. Two 0.625 in. and two 0.75 in. threadbars are used without steel channels for exterior and interior girders, respectively. For the internal anchorage, a 4 in. x 4 in. x 0.375 in. square bearing plate with hex nut and washer is used as specified by the manufacturer. Since the single-column bent has the ledge with tapered section, the hillside washer must be used at the bottom of the bent to clamp the threadbars instead of a regular washer. Based on the overstrength factor, the solution for the single-column bent increases 8 percent and 13 percent of the hanger capacities for the exterior and interior girder locations of the bents, respectively.

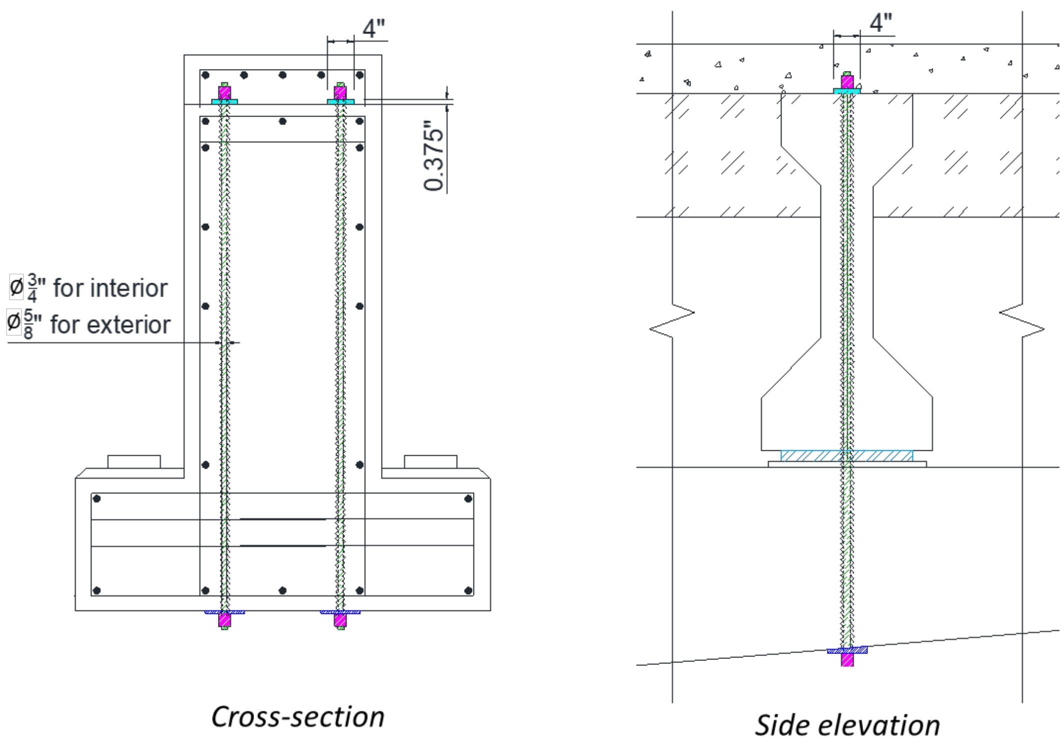
Table 5.1. Capacity Increase from Clamped Threadbar Retrofit.

Bent Type	**Girder Location**	**Capacity ϕC (kip)**		**Demand D (kip)**	**Overstrength Factor $\phi C/D$**
Double Column	**Exterior**	Original	178	247	0.72
		Retrofitted	270		1.09
	Interior	Original	206	287	0.72
		Retrofitted	298		1.04
Single Column	**Exterior**	Original	193	207	0.93
		Retrofitted	209		1.01
	Interior	Original	204	235	0.87
		Retrofitted	239		1.00

Note: ϕ = strength reduction factor, 0.9.



(a) For double-column bent



(b) For single-column bent

Figure 5.2. Details of Clamped Threadbar with Channel for Design Examples.

5.3 Limitations and Recommendations for Construction

To attach a steel channel at the bottom, the underneath of the bent cap should be accessible, while accessibility to the deck slab is required to drill the holes for the threadbars. This may require both above and below lane closure. In addition to lane closure, the clearance below the bent caps has a reduction equal to the channel depth, and the clearance adequacy should be checked prior to construction.

To avoid internal reinforcement, especially existing hanger reinforcement, use of a rebar detector is recommended to design the location of the threadbars. Since spacing of hanger reinforcement is generally larger than the hole size for the threadbar, it is possible to avoid hanger reinforcement while boring through the web.

The threadbars are placed by boring holes from the deck slab downward; this permits the use of water-lubricated diamond core bits, if preferred. Drilling a large diameter recess hole first (approximately 8 in. in diameter) through the deck slab, and then a small diameter hole sufficient to pass the threadbar, is recommended. The bar is placed, and the upper anchorage is essentially a nut, a washer, and a bearing plate. Once complete, the upper recess hole is filled with grout/concrete.

The channel (with predrilled hole) needs to be secured to the bottom surface, and putting in the grout before placing the channel is recommended. If grout is used, the threadbar needs to be fastened within the allowable working time for the grout. A curing time for the grout is required before fully reopening the bridge for all traffic.

CHAPTER 6: LOAD-BALANCING PT (SOLUTION 14)

The load-balancing prestressed solution (Solution 14) uses a PT system that strengthens the entire inverted-T bent at once. PT strands are installed as close as possible to the web and anchored at the end of the bent cap with an end-region anchor plate. The PT strands address the deficiencies of the bent cap by providing upward forces, lifting the cantilever parts (Figure 6.1[a]), and transferring the loads from the girder to the column through the concrete saddles. For double-column bents, the PT strands pass beneath the interior girders to increase capacities at the interior girder locations by transferring the interior girder loads to concrete saddles on both columns (Figure 6.1[b]). Concrete saddles lightly reinforced with the minimum steel are placed at the column to ensure the effective inclination angle of the PT bars.

6.1 Design Procedures

The required procedure for designing load-balancing PT are detailed below. An in-depth explanation is presented in the following subsections:

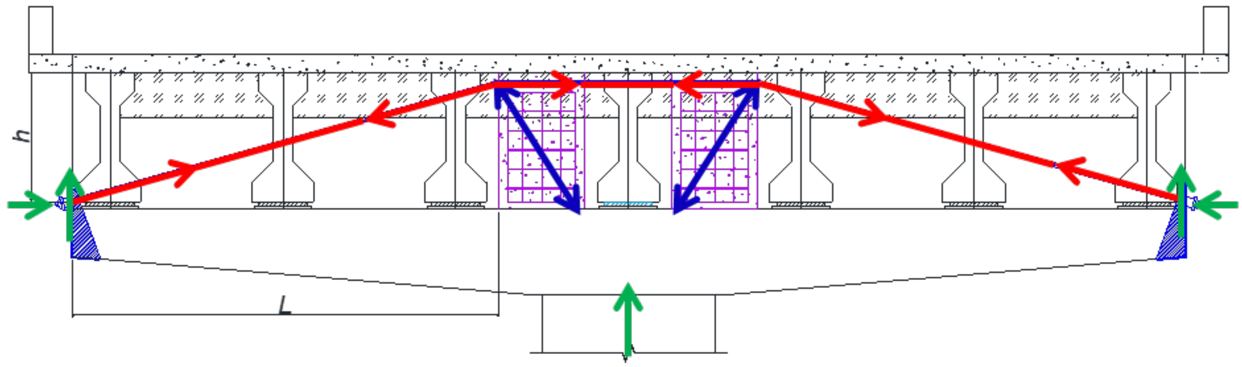
- Step 1: Specify strand.
- Step 2: Design end-region anchor system.
- Step 3: Design beveled plate.

6.1.1 Step 1: Specify Strand

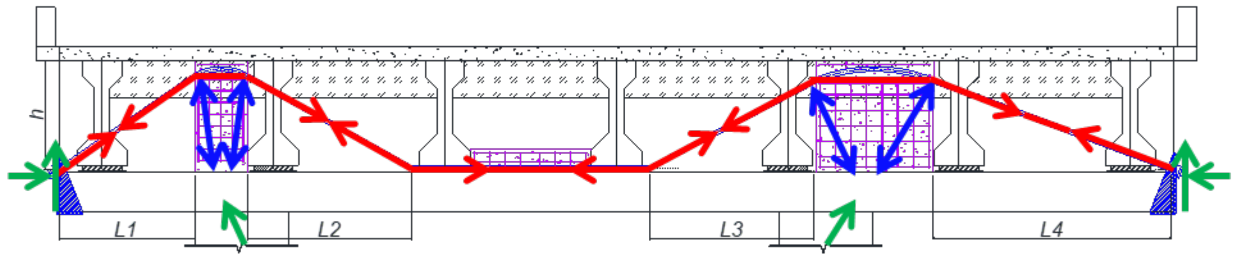
The strands should be designed to resist the maximum force demand at the exterior and interior parts. The required strength of the strand can be obtained from geometry:

$$F_{req} = \frac{V_d}{\sin(\alpha)} \tag{6.1}$$

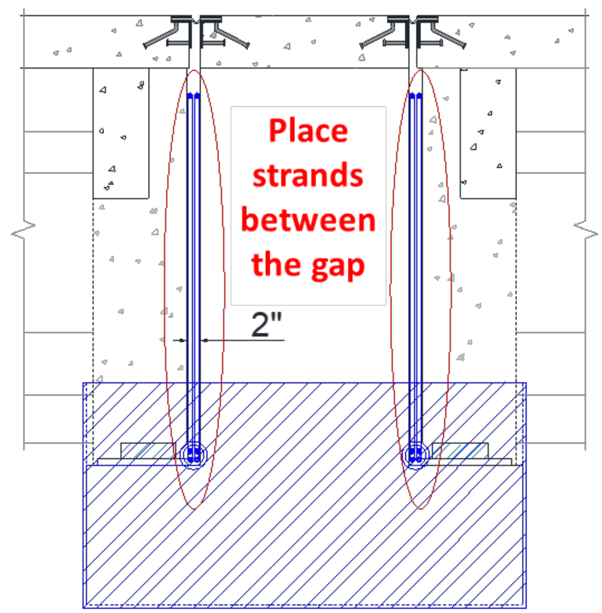
where F = required tension force of PT strands; V_d = required supplemental load capacity, which is the maximum deficiency; and $\alpha = \tan^{-1}(h/L)$, which is the angle of the PT bars, where h and L are described in Figure 6.1.



(a) Elevation of single-column bent



(b) Elevation of double-column bent



(c) Cross-section at the end of the bent

Figure 6.1. Load-Balancing PT System to Overcome Predominant Deficiency of the Bent.

The strength of the PT strand at the service limit state must be greater than the required strength of the strand, F_{req}. The strength of the PT strands at service limit state after losses can be obtained by:

$$P_{pe} = f_{pe} A_{pt} \geq F_{req} \tag{6.2}$$

where A_{pt} = cross-section area of PT strand; $f_{pe} = 0.8 f_{py}$ = stress at service limit state after losses (AASHTO, 2014); $f_{py} \leq 0.85 f_{pu}$ = yield stress of PT strand (AASHTO, 2014); and f_{pu} = ultimate stress of PT strand in ksi.

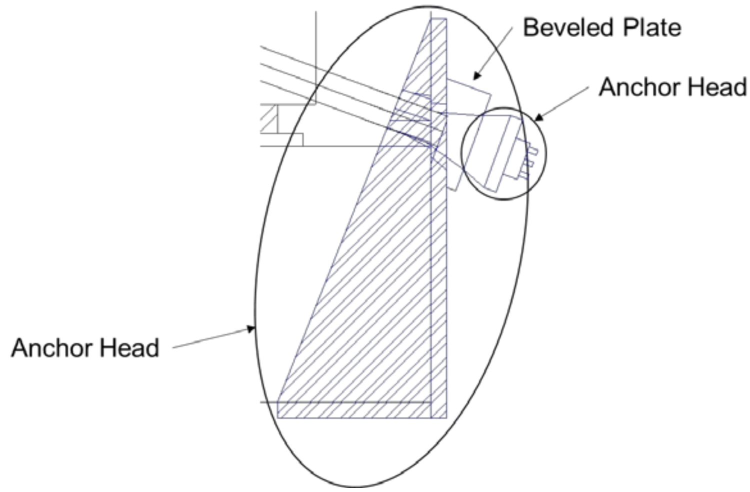
6.1.2 Step 2: Design End-Region Anchor System

To properly anchor the PT strands, an end-region anchorage system (anchor head, end-region anchor plate, and beveled plate) is required, as shown in Figure 6.2. Generally, an anchor head to anchor the strands is provided by the PT strand manufacturer. The end-region anchor plate and the beveled plate need to be customized and designed for the bearing forces and inclination angle.

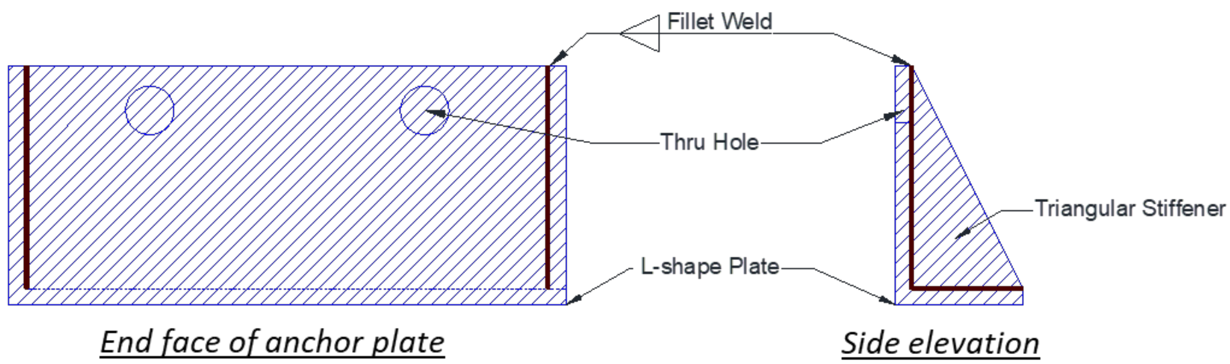
The end-region anchor plate needs to be an L-shape. This shape can be formed by either bending a plate or welding two plates. The anchor plate thickness and plate dimensions must meet the required bearing thickness calculated by Equations (4.6) and (4.7) as well as the bearing area obtained by Equation (5.2). To resist bending due to the prestressing force, triangular stiffeners must be attached to the anchor plate, as shown in Figure 6.2, and designed in accordance with the AISC (2010) specification along with welding details.

6.1.3 Step 3: Design Beveled Plate

The beveled plate to anchor the strands with an angle must be designed for the angle defined for PT strands in Step 1. The minimum thickness of the beveled plate must be larger than the required bearing thickness as determined by Equations (4.6) and (4.7). (Note: Square is easier to manufacture.)



(a) Components of anchor system



(b) Details of end-region anchor plate

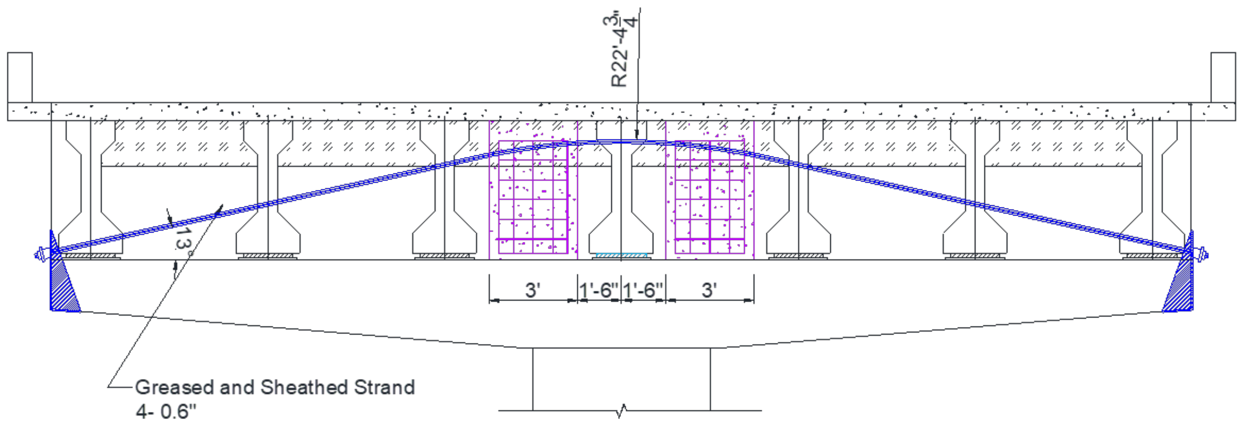
Figure 6.2. Details of Anchor System.

6.2 Discussion of Design Example

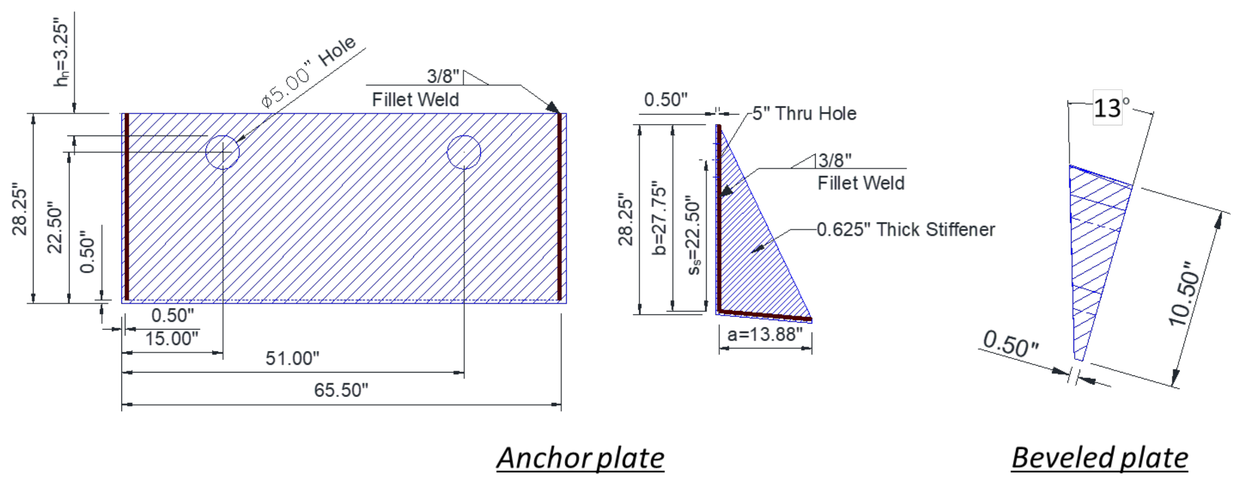
The design examples for double- and single-column bents are presented in Section B.3 and C.3 in the appendices, respectively. In Section B.3, a design example is provided for the double-column bent (Bent 13) to illustrate the design concept, however the solution is invalid for field implementation as the bent cap configuration cannot provide the minimum radius for PT strands. For the single-column bent, the solution is fully designed, and the example is provided in Section C.3. Figure 6.3 shows the design example for Bent 22.

Table 6.1 provides the capacity increase of the single-column bent with the developed design example under ultimate load. Since this solution is determined to be not applicable for Bent 13, the increased capacity with this solution is not provided.

For the single-column bent, four 0.6 in. diameter PT strands are used on each side of the bent cap with an inclination angle of 13 degrees. The greased and sheathed strand manufactured by VSL can be used for this case. Concrete saddles are located on both sides of the center girder, as shown in Figure 6.3(a). The radius of the strands is 22 ft 4-3/4 in., which is larger than the manufacturer's minimum radius of 9.8 ft (provided by VSL). Anchorage (or anchor head) can be chosen from among several types of anchorages for four of the 0.6 in. strands. A 0.5 in. thick anchor plate is used for each end, with a 13-degree angle for the beveled plates. The beveled plate is designed to have the minimum thickness of 0.5 in. and outer diameter of 10.5 in. With the solution designed for the single-column bent, the hanger capacity is increased by 16 percent and 40 percent for exterior and interior girders, respectively.



(a) Elevation view



(b) Anchor system

Figure 6.3. Design Example for Single-Column Bent.

Table 6.1. Capacity Increase from Load-Balancing PT Retrofit.

Girder Location		Capacity ϕC (kip)	Demand D (kip)	Overstrength Factor $\phi C/D$
Exterior	Original	193	207	0.93
	Retrofitted	226		1.09
Interior	Original	204	235	0.87
	Retrofitted	334		1.27

Note: ϕ = strength reduction factor, 0.9.

6.3 Limitations and Recommendations for Construction

To implement the PT solution, there must be a sufficient gap between the girder ends and the web of the bent caps since the solution is designed to place PT strands within that gap. If the gap is satisfactory for the PT strands, the adjacent structures can be the major concern because this solution requires accessibility to the end face to install the end-region anchor system.

First, to anchor the end-region anchor plate, the holes for mechanical anchors need to be drilled on the end surface of the bent, and the anchor plate is placed with grout. Then, the PT strands are properly placed with anchor heads and prestressed. These procedures require at least 5 ft space at the end region, especially to induce prestressing force using a jacking machine. However, as noted by Hurlebaus et al. (2018a), some bents for the main and thruway lanes are adjacent to each other and access to the end surface of the bents are not possible. The solution is not applicable for these bent caps.

For a double-column bent with deficiencies at the interior girder locations, the PT strands need to be placed beneath the interior girders to increase bent cap capacities. Since the PT strands must meet the minimum bend radius specified by the manufacturer to avoid damage on the strands, the bent cap and girder configuration must be able to provide proper angle of inclination, with a greater bend radius for the PT strands than the required radius.

The use of unbonded sheathed strands, which permit the PT to be applied within the confines of the restricted space, is recommended. For the end-region anchor plate, welding the anchor plate and beveled plate together before placing the PT strands is recommended to avoid kinks at the end region.

CHAPTER 7: CONCRETE INFILL WITH PARTIAL-DEPTH FRP ANCHORED BY STEEL WALING (SOLUTION 16)

The partial-depth FRP solution (Solution 16) utilizes an FRP to increase ledge and punching shear capacities. Infill concrete between the girders creates a rectangular cross-section to minimize FRP bends. Threadbars are used to connect the web and infill concrete and to provide a location for attachment of a waling used to hold the FRP in place. The solution is intended for an inverted-T bent cap with diaphragms between girders, which limits the height of infill concrete and FRP. Since partial-depth infill concrete and FRP is not able to transfer girder load back to the top tension chord, the solution is not applicable for hanger deficiency.

7.1 Design Procedures

The required steps for designing the partial-depth FRP retrofit are listed below, and an in-depth explanation is presented in the following subsections:

- Step 1: Design FRP composite.
- Step 2: Design threadbar and infill concrete reinforcement.
- Step 3: Design steel waling.

7.1.1 Step 1: Design FRP Composite

The FRP wrap is primarily designed for shear as specified by ACI 440.2R (ACI Committee 440, 2008). The code gives an estimation for the shear contribution of the FRP for a general case in which an even spacing of narrow FRP strips is used. For the inverted-T bent cap, continuous FRP strips are used on either side of the girder. Therefore, a modified equation is used to estimate the shear contribution of the FRP:

$$V_f = A_{fv} f_{fe} \tag{7.1}$$

where A_{fv} = area of FRP strip; and f_{fe} = tensile stress in the FRP strip.

The area of FRP, A_{fv}, is:

$$A_{fv} = n t_f w_f \tag{7.2}$$

where n = number of FRP layers; t_f = thickness of FRP composite; and w_f = width of FRP strip.

The tensile stress in the FRP, f_{fe}, is directly proportional to the level of strain that can be developed in the FRP strip at nominal strength:

$$f_{fe} = \varepsilon_{fe} E_f \tag{7.3}$$

where ε_{fe} = effective strain in FRP; and E_f = tensile modulus of FRP.

In the code equations, w_f is the width of a single FRP strip placed evenly throughout the beam. In the current case, it is defined as the effective width of the FRP that is engaged in transferring the girder load to the web. FRP strips are attached only between the girders; therefore, the effective width is calculated by subtracting from the distribution width the bottom width of the girder and the thickness of the debonding foam sheet between the infill concrete and the face of the girder. The effective strain of FRP, ε_{fe}, is taken as 0.004 since the ends of the FRP strips are anchored by the steel waling.

7.1.2 Step 2: Design Threadbar and Infill Concrete Reinforcement

Threadbars are provided to resist the required shear demand. The design shear strength of the threadbar is 60 percent of the ultimate strength.

Since the infill concrete is not loaded with significant force, minimum reinforcement based on ACI 318-14 (ACI Committee 318, 2014) is provided. Transverse and longitudinal reinforcements are provided to meet the requirement. Transverse reinforcement is arranged to avoid interference with the threadbars.

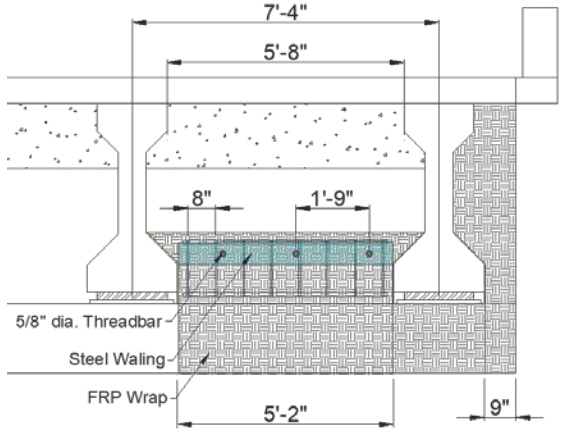
7.1.3 Step 3: Design Steel Waling

The size of the steel waling is determined to ensure sufficient bond strength can be developed between the waling and the FRP strip. It may vary based on the bond strength of the resin that is used for the FRP composite. The length of the steel waling should be equal to the length of the concrete infill. The thickness of the steel waling is designed based on shear bearing at the connection with the threadbars:

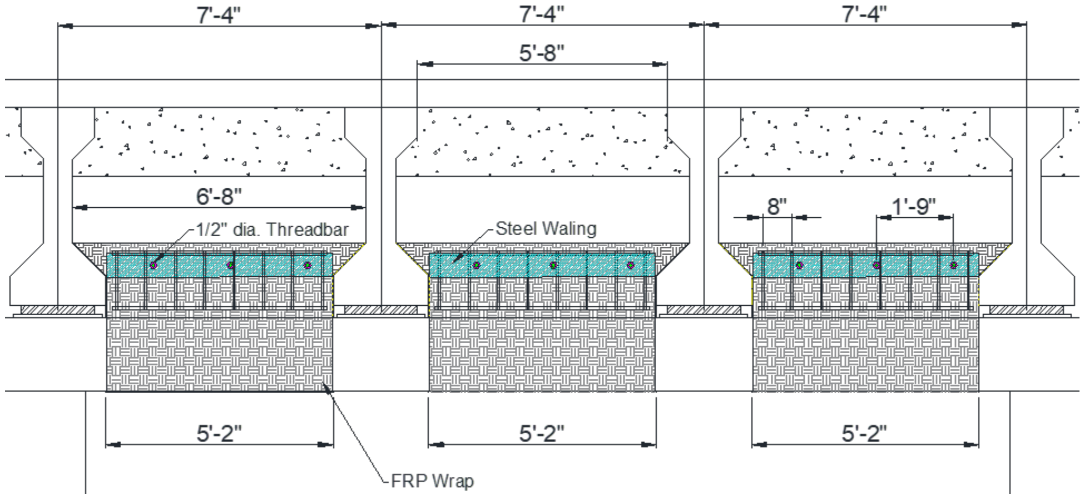
$$t_{bearing} \geq \frac{V_u}{\phi 1.8 d_b f_u} \tag{7.4}$$

7.2 Discussion of Design Example

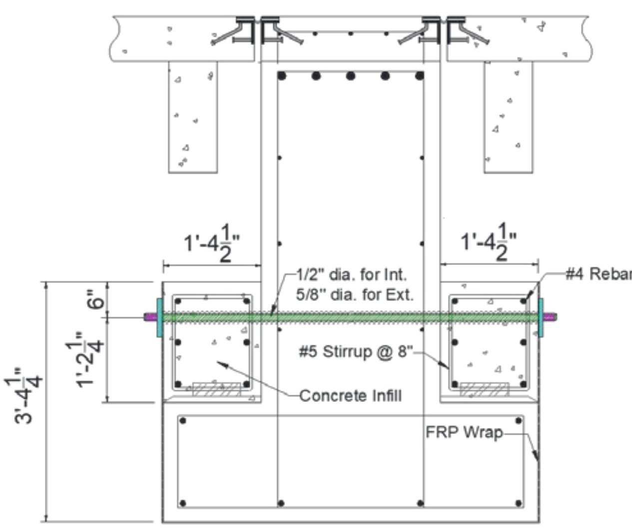
The retrofit is developed only for the typical twin-column bent cap; the solution is not applicable to the single-column bent, which is only deficient in hanger. Figure 7.1 shows the design details. Table 7.1 provides the capacity increase of the double-column bent cap with the developed design example under ultimate load demand. A sample FRP composite with tensile modulus of 33,000 ksi and thickness of 0.013 in. is used to develop the design example.



(a) Exterior elevation



(b) Interior elevation



(c) Cross-section

Figure 7.1. Partial-Depth FRP Design Example for Double-Column Bent.

For the exterior, a single layer of FRP is used with a 64 in. x 4 in. x 0.75 in. steel waling held by three 1 in. diameter B7 Grade threadbars (125 ksi). The solution increases the strength by 67 percent. The overstrength factor is increased to 1.21 from 0.07.

For the interior, a single layer of FRP is used with a 63 in. x 4 in. steel waling held by three 1 in. diameter threadbars. The interior is critical in punching shear and is improved by 45 percent with the developed retrofit. The overstrength factor is increased to 1.04 from 0.72. It should be noted that the interior portion of the double-column bent cap still has deficient hanger capacity.

Table 7.1. Capacity Increase from Partial-Depth FRP Retrofit for Double-Column Bent Cap.

Girder Location	Capacity ϕC (kip)		Demand D (kip)	Overstrength Factor $\phi C/D$
Exterior	Original	178	247	0.72
	Retrofitted	298		1.21
Interior	Original	206	287	0.72
	Retrofitted	299*		1.04

Note: ϕ = strength reduction factor, 0.9.
** Punching shear capacity after retrofit.*

7.3 Limitations and Recommendations for Construction

Holes for the threadbars must be drilled prior to casting infill concrete. To avoid internal reinforcement, especially the hanger reinforcement, use of a rebar detector is recommended to identify the location of the threadbars. The outside threadbars need to be sufficiently away from the girder to ensure a workable space for drilling.

To fill the gap between the girders with concrete, formworks and rebar cages for the infill concrete need to be constructed prior to the concrete pour. The dimensions need to be determined based on the actual geometry of the structure. The reinforcement for the infill concrete needs to be designed to avoid the threadbars. Determining the location of the holes on the formworks after placing the threadbars is recommended.

Pouring concrete could be a challenge due to limited space, and the lane below may need to be closed. Infill concrete does not need to be fully cured before the next operation step. Seven days curing time may be enough for the infill concrete to operate the surface treatment for the FRP.

Prior to placing the FRP, the substrate must be properly prepared to meet the requirement specified by the manufacturer. Sharp corners must be rounded to avoid stress concentration on the FRP strips. FRP composites generally consist of several components, with a specified duration between applications of each component. High temperature will reduce the working time substantially. Therefore, the working time for the FRP composite application should be arranged in advance.

CHAPTER 8: CONCRETE INFILL WITH FULL-DEPTH FRP ANCHORED BY STEEL WALING (SOLUTION 17)

The full-depth FRP solution (Solution 17) utilizes FRP to increase hanger, ledge, and punching shear capacities. Similar to Solution 16, infill concrete transforms the cross-section to a rectangular shape to minimize FRP bends. Through threadbars at the top and embedded threadbars at the bottom of the web are used to provide (a) continuity between the new and old concrete, and (b) a location for attachment of the walings that anchor the FRP.

8.1 Design Procedures

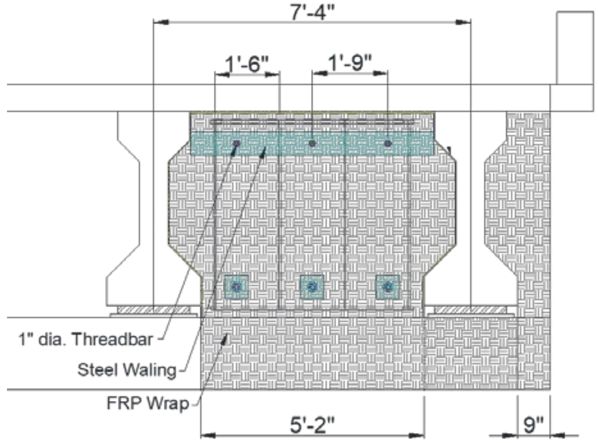
The required steps for designing the full-depth FRP retrofit are:

- Step 1: Design FRP composite.
- Step 2: Design threadbar and infill concrete reinforcement.
- Step 3: Design steel waling.

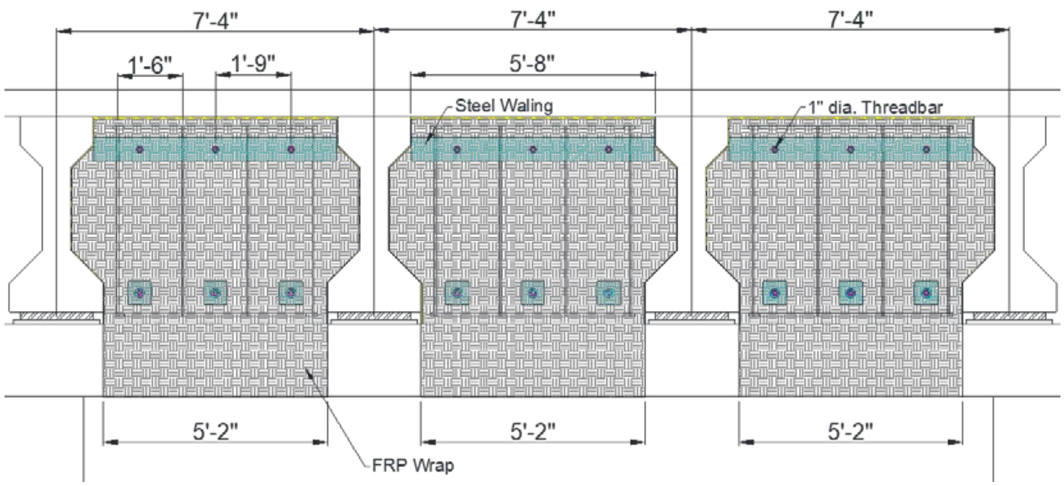
Since the design procedures are similar to Solution 16, details are not repeated in this section. The effective width of the FRP strips is determined based on the distribution width of each term of inverted-T bent cap capacities, and the effective strain is assumed as 0.004. The fully raised FRP strips also contribute strength to the hanger deficiency. The top-layer threadbars are designed to resist shear force at the connection with the steel walings. The same number of threadbars is used at the bottom layer to provide continuity between the concrete infill and the web. Minimum reinforcement is provided for the concrete infill. Steel walings are designed for the shear bearing at the connection with the threadbars and to have enough bond strength between the plate and the FRP strips.

8.2 Discussion of Design Example

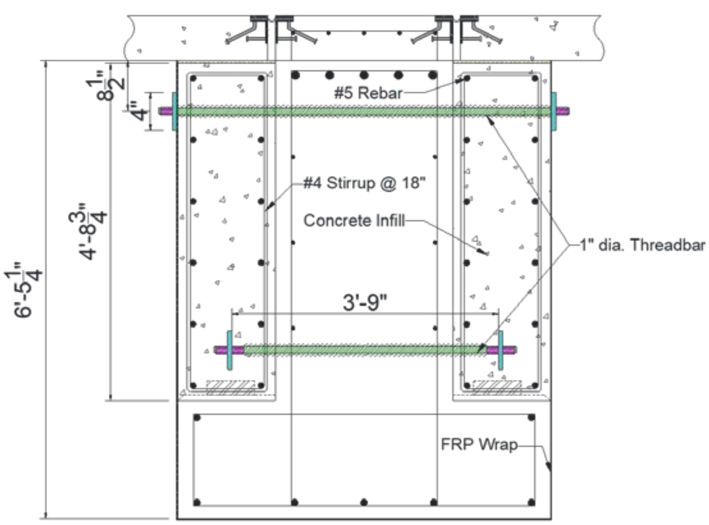
The solution is developed for both double- and single-column bent caps. Figure 8.1 and Figure 8.2 show the design details for the double- and single-column bents, respectively. Table 8.1 provides the capacity increase of the bent caps with the developed design examples under ultimate load demand. A sample FRP composite (BASF C160) with a tensile modulus of 33,400 ksi and thickness of 0.04 in. is used to develop the design examples.



(a) Exterior elevation

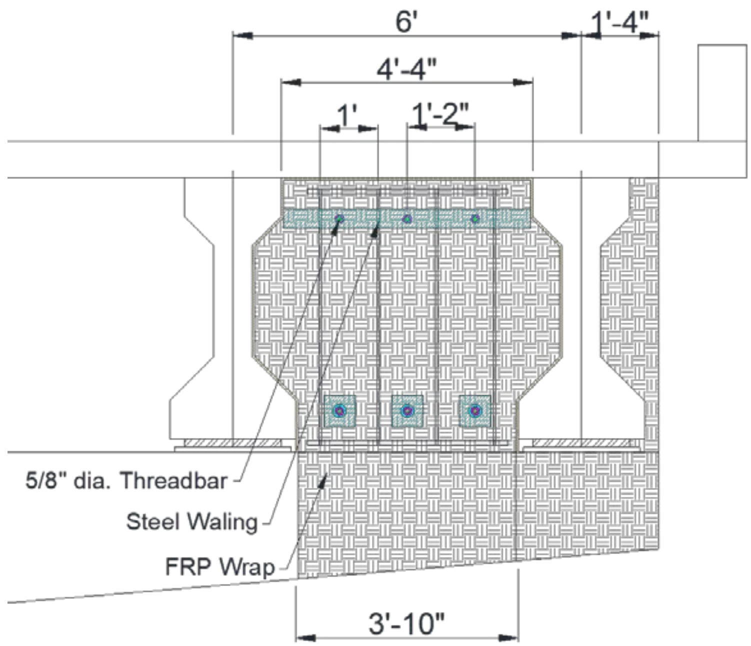


(b) Interior elevation

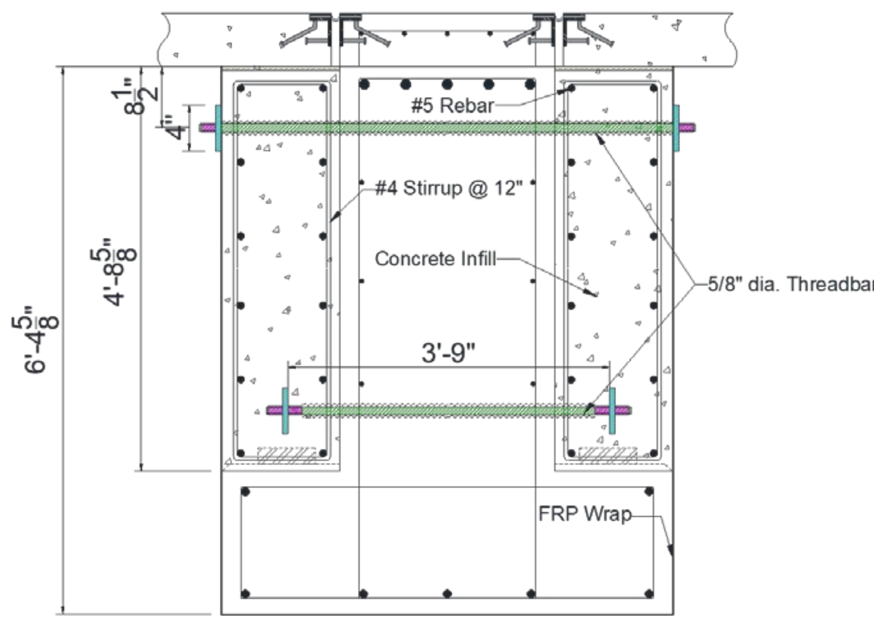


(c) Cross-section

Figure 8.1. Full-Depth FRP Design Example for Double-Column Bent.



(a) Elevation



(c) Cross-section

Figure 8.2. Full-Depth FRP Design Example for Single-Column Bent.

Six 5/8 in. and 1 in. diameter B7 Grade threadbars (125 ksi) are provided for the infill concrete of the single- and double-column bent cap, respectively. Threadbars are provided in two layers, with three in each layer. A single and two layers of FRP anchored by 64 in. x 14 in. x 0.75 in. steel waling are used for the exterior and interior of the double-column bent cap, respectively. A single layer of FRP with 64 in. x 14 in. x 0.75 in. steel waling is used for the single-column bent cap. The developed solution increases the strength of the exterior and interior of the double-column bent cap by 56 percent and 42 percent, respectively. For the single-column bent cap, the strength increases by 19 percent and 5 percent for the exterior and interior, respectively.

Table 8.1. Capacity Increase from Full-Depth FRP Retrofit.

Bent Type	Girder Location	Capacity ϕC (kip)		Demand D (kip)	Overstrength Factor $\phi C/D$
Double Column	**Exterior**	Original	178	247	0.72
		Retrofitted	277		1.12
	Interior	Original	206	287	0.72
		Retrofitted	292		1.02
Single Column	**Exterior**	Original	193	207	0.93
		Retrofitted	229		1.11
	Interior	Original	204	235	0.87
		Retrofitted	239		1.02

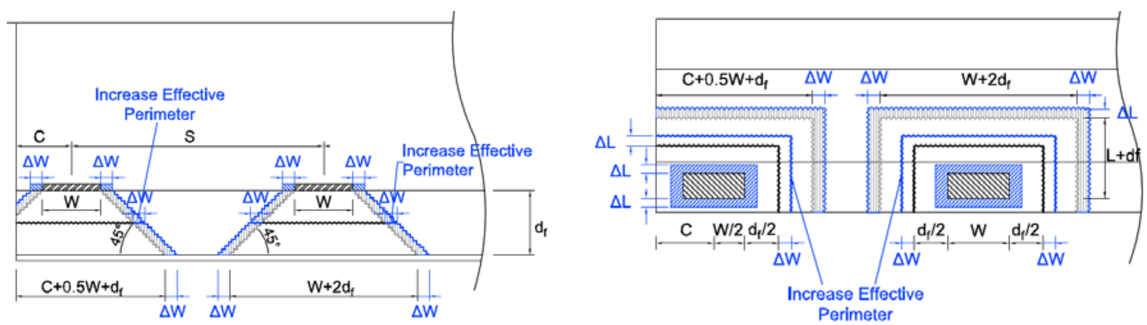
Note: ϕ = strength reduction factor, 0.9.

8.3 Limitations and Recommendations for Construction

The challenges for the partial-depth FRP retrofit (Solution 16) also apply to the full-depth FRP retrofit. The concrete infill for the full-depth FRP solution presents an additional challenge to implement the retrofit. Since the infill concrete extends to the top of the bent cap, the concrete must be poured from the deck slab down. A hole for pumping the concrete will need to be drilled from the deck at the proper location. This operation may require cutting an existing reinforcement in the deck. Traffic lanes above and below may need to be closed during the concrete pour.

CHAPTER 9: LARGE BEARING PAD (SOLUTION 18)

Figure 9.1 shows the concept of the retrofit solution to strengthen punching shear capacity by using larger bearing pads. A punching failure may occur if the girder reactions are sufficient to punch out a truncated pyramid beneath the bearing pad. Punching shear capacity depends on several parameters: girder spacing, edge distance, ledge depth, and bearing pad size. Girder spacing, edge distance, and ledge depth are fixed by geometry, but bearing pads can be replaced. The proposed solution, use of increased bearing pad size, is expected to enhance the punching shear performance by increasing the load distribution area.



(a) Side elevation

(a) Plan view

Figure 9.1. Solution for Punching Shear Failure by Increasing Bearing Pad Size.

9.1 Design Procedures

A rational modification for punching shear capacity equations given by AASHTO LRFD (2014) and TxDOT (2015) is proposed in Section 2.3.3. The nominal punching shear resistance of interior of inverted-T bent cap ledge can be calculated using Equation (2.17a). For exterior, the lessor of Equation (2.17a) or (2.18a) controls. In general, c is less than $\mathrm{W}/2 + L + d_f$, and Equation (2.18a) controls the punching shear capacity.

In the proposed punching shear capacity equations, the area of the concrete failure surface is approximated as the average of the perimeter of the bearing pad and the perimeter at depth, d_f, assuming 35-degree slopes multiplied by d_f. Therefore, the terms in parentheses in Equation (2.17a) and (2.18a) are the effective perimeter of the concrete failure surface. Increasing the bearing dimension (i.e., W and L) in the equations will increase the effective perimeter of the failure surface, and hence increase the punching shear capacity. In the developed design example, the term of the effective perimeter is defined as p, and the required increment of the effective perimeter (i.e., Δp) is worked out using the deficient capacity. The increment is then achieved by increasing W and L, with consideration for limits imposed by the geometry of the bent cap.

9.2 Discussion of Design Example

The retrofit with a large bearing pad is developed only for the exterior of the typical double-column bent cap (Bent 13). The interior of the double-column bent cap and the single-column bent cap (Bent 22) has sufficient punching shear capacity to resist the load demand. Table 9.1 provides the capacity improvement of the double-column bent cap with the developed design example under ultimate load demand.

Figure 9.2 shows the design details. A 23 in. x 11 in. size bearing pad is used to replace the original bearing pad at the exterior, which has a size of 21 in. x 8 in. The punching shear capacity is improved by 6 percent, and the overstrength factor is increased to 1.01 from 0.95.

9.3 Limitations and Recommendations for Construction

The girders may need to be lifted to replace the original bearing pad. A flat jack cylinder can be used to lift the girders. Dowel bars used to connect the girder to the bent cap ledge (Figure 9.3) may be an obstacle to implementation of the retrofit. The dowel bar typically extends 6 in. from the top of the bearing seat. The original bearing pad may need to be cut out to avoid the dowel bar.

Table 9.1. Capacity Increase from Large Bearing Pad.

Bent Type	Girder Location	Capacity ϕC (kip)		Demand D (kip)	Overstrength Factor $\phi C/D$
Double Column	Exterior	Original	235	247	0.95
		Retrofitted	250		1.01

Note: ϕ = strength reduction factor, 0.9.

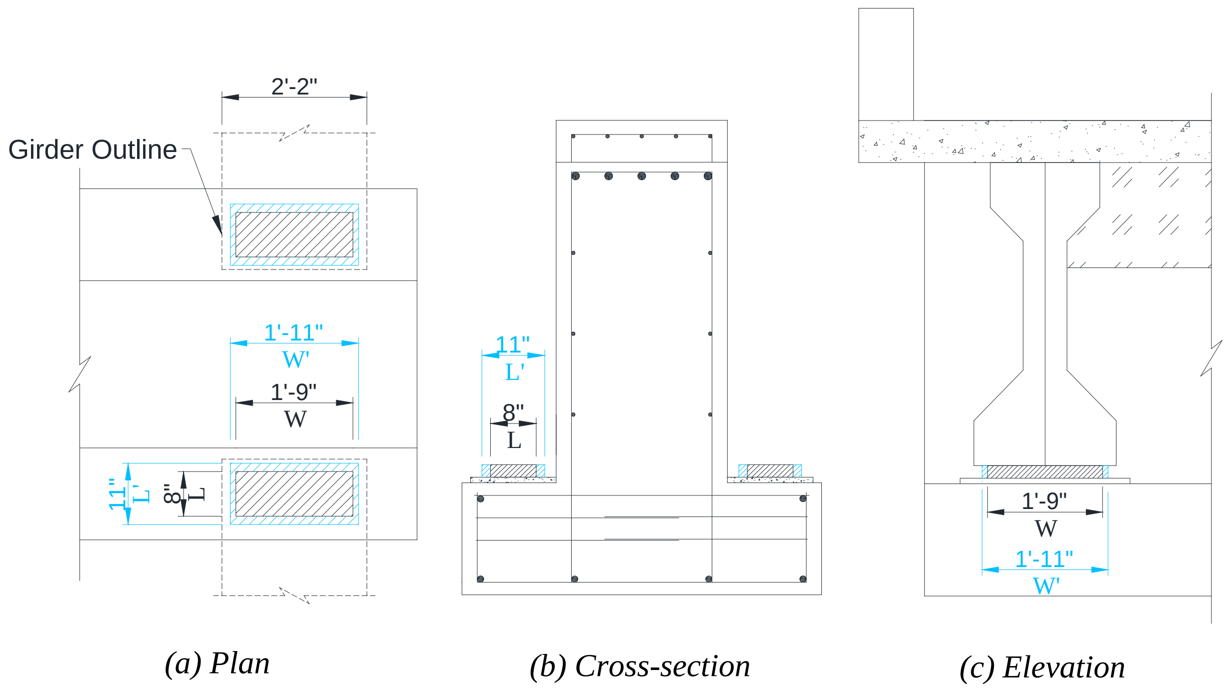


Figure 9.2. Large Bearing Pad Design Example for Exterior of Double-Column Bent.

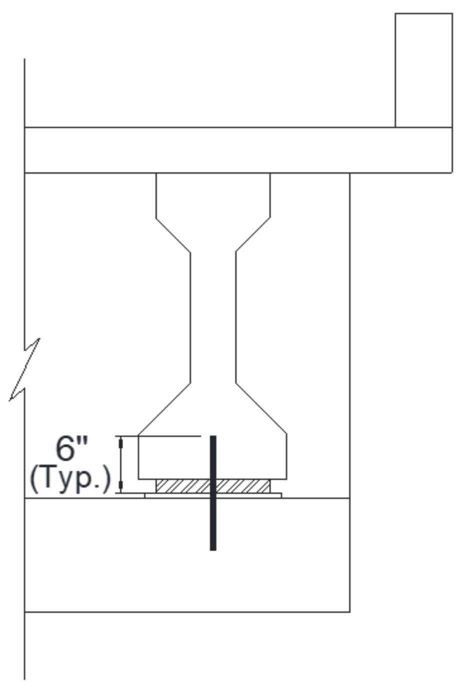


Figure 9.3. Dowel Bar Connecting Girder to Inverted-T Bent Cap.

CHAPTER 10: SUMMARY

For certain older in-service bridges, for example, the upper deck of IH-35 in Austin, the inverted-T bent caps may need to be strengthened. To evaluate the performance of an inverted-T bent cap with and without retrofit solutions, Thirty-three individual tests were conducted on eight half-scale specimens. Findings from the experimental program were used to develop these recommendations for evaluation of in-service inverted-T bent caps and design of selected retrofit solutions.

Two in-service inverted-T bent cap types, typical double- and single-column bent caps, were evaluated against AASHTO LRFD (2014) sectional methods. Some rational modifications for exterior distribution widths for ledge flexure, ledge shear friction, and the angle of the truncated pyramid for punching shear capacity were recommended by Hurlebaus et al. (2018b). The identified strength deficiencies were used to develop the proposed retrofit solutions.

Guidance for the selection of retrofit solutions was provided with three general criteria: deficiencies addressed, obstacles, and costs. Retrofit selection should be mostly based on the strength requirements, with additional consideration for obstacles to implementations as well as initial and life-cycle costs.

Finally, design recommendations for six retrofit solutions were developed based on the verified findings from the experimental results. The design recommendations were made for addressing critical failure modes of inverted-T bent caps. General design procedures were given in detail. The specific designs of each retrofit solution along with the limitations and recommendations for construction were discussed. The examples for designing the retrofit solutions are presented in Appendix B and C for double- and single-column bents, respectively.

The design examples for three retrofit solutions—end-region stiffener (Solution 3), clamped threadbar with channel (Solution 8), and concrete infill with full-depth FRP anchored by steel waling (Solution 17)—were developed for both double- and single-column bents; Solution 3 was only developed for exterior portions. The load-balancing PT (Solution 14) was developed for only single-column bent caps due to the limitation of the geometry of double-column bent caps. The concrete infill with partial-depth FRP anchored by steel waling (Solution 16) was developed for only the double-column bent cap since it is not able to enhance hanger capacity, which must be addressed for the single-column bent cap. The large bearing pad (Solution 18) was developed for only double-column bent caps since the single-column bent cap has sufficient punching shear capacity. All the solutions are able to address the deficient capacities of the bent caps.

REFERENCES

AASHTO. (2014). *LRFD Bridge Design Specifications*, Seventh Edition. American Association of State Highway and Transportation Officials, Washington DC.

ACI Committee 318. (2014). *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*. American Concrete Institute, Farmington Hills, MI.

ACI Committee 440. (2008). *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*. American Concrete Institute, Farmington Hills, MI.

AISC. (2010). *Specification for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, IL.

Furlong, R.W., and Mirza, S.A. (1974). *Strength and Serviceability of Inverted T-beam Bent-Caps Subject to Combined Flexure, Shear, and Torsion*. TxDOT Report #153-1F, Center for Highway Research, University of Texas at Austin, Austin, TX.

Hurlebaus, S., Mander, J.B., Birely, A.C., Terzioglu, T., Cui, J., and Park, S.H. (2018a). *Strengthening of Existing Inverted-T Bent Caps—Volume 1: Preliminary Design*. Rep. No. FHWA/TX-18/0-6893-R1-Vol1, Texas A&M Transportation Institute, The Texas A&M University System.

Hurlebaus, S., Mander, J.B., Birely, A.C., Terzioglu, T., Cui, J., and Park, S.H. (2018b). *Strengthening of Existing Inverted-T Bent Caps—Volume 2: Experimental Test Program*. Rep. No. FHWA/TX-18/0-6893-R1-Vol2, Texas A&M Transportation Institute, The Texas A&M University System.

Shakya, S., and Vinnakota, S. (2008). "Design Aid for Triangular Bracket Plates Using AISC Specifications." *Engineering Journal*, Third Quarter, pp. 187–196.

TxDOT (2010). *Inverted-T Bent Cap Design Example*, Bridge Division, Texas Department of Transportation, Austin, TX.

TxDOT. (2015). *Bridge Design Manual—LRFD*. Texas Department of Transportation, Center for Highway Research, University of Texas at Austin, Austin, TX.

APPENDIX A. BENT CAP ANALYSIS

A.1 DOUBLE-COLUMN BENT (BENT 13)

Dimension

Specimen dimension

$b_f := 63\ in$ — *Bottom Flange Width*

$b_{web} := 30\ in$ — *Web Width*

$b_{ledge} := 16.5\ in$ — *Single Ledge Width*

$d_{ledge} := 20\ in$ — *Ledge Height*

$d_e := 17.5\ in$ — *Distance from Top Layer of Ledge Reinforcement to Bottom of Ledge*

$d_f := 17\ in$ — *Distance from Top of Ledge to Centroid of Bottom Layer of Ledge Reinforcement*

$a_v := 7.5\ in$ — *Distance from Web Face to Center of Bearing Pad*

$cover := 2.5\ in$ — *Average Concrete Cover of Web*

$a_f := a_v + cover$ $\quad a_f = 10\ in$

$brgseat := 1\ in$ — *Bearing Seat Buildup, in the specimen there is no bearing seat buildup*

$h := d_{ledge} + brgseat$ $\quad h = 21\ in$

$S := 88\ in$ — *Girder Spacing*

$c := 22\ in$ — *Distance from the Center Line of the Exterior Girder to the Edge of the Cap measured along the Cap*

Bearing Pad Dimension

$W := 21\ in$ — *Bearing Pad Width*

$L := 8\ in$ — *Bearing Pad Length*

Material Properties

Concrete Strength

$f'_c := 3.6\ ksi$ — *Concrete Strength*

Steel

$f_y := 60\ ksi$ — *Yield Strength of Reinforcement*

$E_s := 29000\ ksi$ — *Young's Modulus*

Ledge Shear Friction

Exterior

Distribution Width

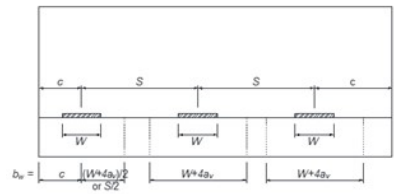
b_{s_ext} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

Modified to half of the distribution width (or half the girder spacing) and the distance to the edge of the cap

$$W+4\ a_v=51\ in$$

$$\frac{S}{2}+c=66\ in$$

$$\frac{1}{2}\left(W+4\ a_v\right)+c=47.5\ in$$



$b_{s\_ext}=47.5\ in$

Capacity

V_{ns_ext} = minimum of:

$0.2\ f'_c \cdot b_{s\_ext} \cdot d_e=598.5\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-1)*

$0.8\ ksi \cdot b_{s\_ext} \cdot d_e=665\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-2)*

$V_{ns\_ext}=598.5\ kip$

Interior

Interior

b_{s_int} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$$W+4\ a_v=51\ in$$

$$S=88\ in$$

$b_{s\_int}=51\ in$

Capacity

V_{ns_int} = minimum of:

$0.2\ f'_c \cdot b_{s\_int} \cdot d_e=642.6\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-1)*

$0.8\ ksi \cdot b_{s\_int} \cdot d_e=714\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-2)*

$V_{ns\_int}=642.6\ kip$

Ledge Flexure

$A_{s5} := 0.31\ in^2$ — *Area of #5 rebar*

Exterior

Distribution Width

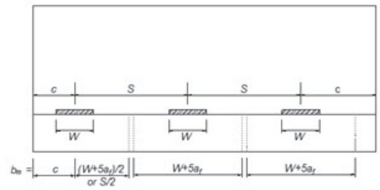
b_{m_ext} = minimum of: — *(AASHTO LRFD 5.13.2.5.2)*

Modified to half of the distribution width (or half the girder spacing) and the distance to the edge of the cap

$$W + 5\ a_f = 71\ in$$

$$\frac{S}{2} + c = 66\ in$$

$$\frac{1}{2}\left(W + 5\ a_f\right) + c = 57.5\ in$$



$b_{m\_ext} = 57.5\ in$

Capacity

$A_s := A_{s5} \cdot 8$ $\qquad A_s = 2.5\ in^2$ — *Primary Ledge Reinforcement within distribution width*

For combined axial tension and bending

$$T = N_u + \phi \cdot A_s \cdot f_y = \phi \cdot 0.85 \cdot f'_c \cdot a \cdot b$$

$V_u := 247\ kip$ — *Exterior Ledge Shear Factored Load*

$N_u := 0.2 \cdot V_u$ $\qquad N_u = 49.4\ kip$ — *Maximum Concurrent Axial Tension*

$\phi := 0.9$

$$a := \frac{\dfrac{N_u}{\phi} + A_s \cdot f_y}{0.85\ f'_c \cdot b_{m\_ext}} = 1.2\ in$$

$$M_{n\_ext} := A_s \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \qquad M_{n\_ext} = 209.8\ kip \cdot ft$$

$$M_n = V_n \cdot a_v + N_n \cdot \left(h - d_e\right)$$ *(AASHTO LRFD 5.13.2.4.1-1)*

$$V_{nf\_ext} := \frac{M_{n\_ext}}{a_v + 0.2 \cdot \left(h - d_e\right)} \qquad V_{nf\_ext} = 307.1\ kip$$

Interior

Distribution Width

b_{m_int} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$W+5\ a_f=71\ in$

$S=88\ in$

$b_{m\_int}:=71\ in$

Capacity

$A_s:=A_{s5}\cdot 8$, $A_s=2.48\ in^2$ *Primary Ledge Reinforcement within distribution width*

For combined axial tension and bending

$T=P_u+\phi\cdot A_s\cdot f_y=\phi\cdot 0.85\cdot f'_c\cdot a\cdot b$

$V_u:=287\ kip$ *Interior Ledge Shear Factored Load*

$N_u:=0.2\cdot V_u$ $N_u=57.4\ kip$ *Concurrent Axial Tension*

$\phi:=0.9$

$$a:=\frac{\dfrac{N_u}{\phi}+A_s\cdot f_y}{0.85\ f'_c\cdot b_{m\_int}}=1\ in$$ *(AASHTO LRFD 5.13.2.4.1-1)*

$M_{n\_int}:=A_s\cdot f_y\cdot\left(d_e-\frac{a}{2}\right)$ $M_{n\_int}=210.9\ kip\cdot ft$

$M_n=V_n\cdot a_v+N_n\cdot\left(h-d_e\right)$

$$V_{nf\_int}:=\frac{M_{n\_int}}{a_v+0.2\cdot\left(h-d_e\right)}$$ $V_{nf\_int}=308.7\ kip$

Hanger Reinforcement *AASHTO LRFD 5.13.2.5.5*

$A_{hr}:=2\ legs\cdot A_{s5}$ $A_{hr}=0.62\ in^2$ *Area of Hanger Reinforcement-2 legs*

$s:=6\ in$ *Hanger Reinforcement Spacing*

Exterior

- BDM-LRFD Ch.4, Sec. 5, Design Criteria - Modified to limit the distribution width to the edge of the cap (or half the girder spacing and the distance to the edge of the cap). This will prevent distribution widths from overlapping or extending over the edge of the cap.

For the Service Limit

V_{sh_ext} = minimum of:

$$\frac{1}{2}\ \frac{A_{hr}\cdot\left(\frac{2}{3}\cdot f_y\right)}{s}\cdot\left(\frac{W+3\ a_v}{2}+c\right)=90.4\ kip$$

TxDOT uses 2/3 f_y based on Furlong & Mirza Eq. 5.4 instead of 0.5 f_y from AASHTO LRFD Eq. 5.13.2.5.5-1

$$\frac{1}{2}\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s}\cdot\left(\frac{S}{2}+c\right)=136.4\ kip$$

$$V_{sh\_ext}=90.4\ kip$$

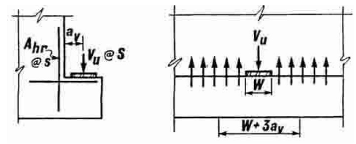


Figure 5.13.2.5.5-1—Single-Ledge Hanger Reinforcement

For the Strength Limit

V_{nh} = minimum of:

$$\frac{1}{2}\frac{A_{hr}\cdot f_y}{s}\cdot\left(\frac{S}{2}+c\right)=204.6\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-2) This equation accounts for hanger reinforcement within shear critical region

$$\frac{1}{2}\left(0.063\cdot\sqrt{f'_c\cdot ksi}\cdot b_f\cdot d_f+\frac{A_{hr}\cdot f_y}{s}\cdot\left(\frac{W+2\cdot d_f}{2}+c\right)\right)=217.5\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-3) This equation accounts for hanger reinforcement within flexural and shear critical region and concrete contribution

$$V_{nh\_ext}=204.6\ kip$$

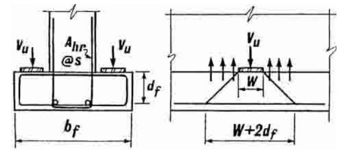


Figure 5.13.2.5.5-2—Inverted T-Beam Hanger Reinforcement

Interior

For the Service Limit

V_{sh_int}= minimum of:

TxDOT uses $2/3 f_y$ based on Furlong & Mirza Eq. 5.4 instead of $0.5 f_y$ from AASHTO LRFD Eq. 5.13.2.5.5-1

$$\frac{1}{2}\frac{A_{hr}\cdot\left(\frac{2}{3}\cdot f_y\right)}{s}\cdot\left(W+3\ a_v\right)=89.9\ kip$$

$$\frac{1}{2}\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s}\cdot S=181.9\ kip$$

$$V_{sh\_int}=89.9\ kip$$

For the Strength Limit

V_{nh} = minimum of:

$$\frac{1}{2}\frac{A_{hr}\cdot f_y}{s}\cdot S=272.8\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-2) This equation accounts for hanger reinforcement within shear critical region

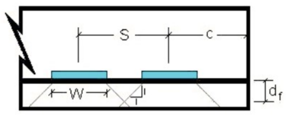
$$\frac{1}{2}\left(0.063\cdot\sqrt{f'_c\cdot ksi}\cdot b_f\cdot d_f+\frac{A_{hr}\cdot f_y}{s}\cdot\left(W+2\cdot d_f\right)\right)=234.5\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-3) This equation accounts for hanger reinforcement within flexural and shear critical region and concrete contribution

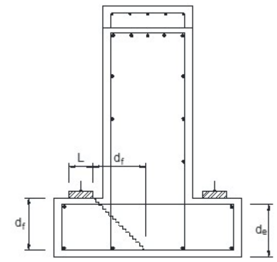
$$V_{nh\_int}=234.5\ kip$$

Punching Shear

Exterior



TxDOT uses d_f instead of d_e for Punching Shear (BDM-LRFD, Ch. 4, Sec. 5, Design Criteria)



V_{np_ext} = minimum of:

$$0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(W + 2 \cdot L + 2 \cdot d_f \cdot \cot(35\ °)\right) \cdot d_f = 345\ kip$$

$$0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(0.5 \cdot W + L + d_f \cdot \cot(35\ °) + c\right) \cdot d_f = 261.2\ kip$$

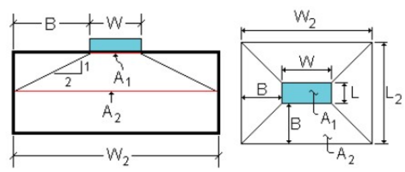
$$V_{np\_ext} = 261.2\ kip$$

Interior

$$V_{np\_int} := 0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(W + 2 \cdot L + 2 \cdot d_f \cdot \cot(35\ °)\right) \cdot d_f \qquad V_{np\_int} = 345\ kip$$

Bearing

A_1 is the loaded area (bearing pad area). A_2 is the area of the lowest rectangle contained entirely within the support (the inverted-T bent cap). A_2 must not overlap the truncated pyramid of another load in either direction, nor can it extend beyond the edges of the cap in any direction.



Elevation View

Plan View

(AASHTO LRFD 5.5.4.2.1)

$A_1 := W \cdot L$ $\qquad A_1 = 168\ in^2$ $\qquad$ *Area under Bearing Pad*

Exterior

B = minimum of:

Distance from the perimeter of A_1 to the perimeter of A_2, as shown in the above figures.

$$\left(b_{ledge} - a_v\right) - \frac{L}{2} = 5\ in$$

$$\left(a_v + \frac{b_{web}}{2}\right) - \frac{L}{2} = 18.5\ in$$

$$2 \cdot d_{ledge} = 40\ in$$

$$\frac{S}{2} - \frac{W}{2} = 33.5\ in$$

$$c - \frac{W}{2} = 11.5\ in$$

$$B := min\left(\left(b_{ledge} - a_v\right) - \frac{L}{2}, \left(a_v + \frac{b_{web}}{2}\right) - \frac{L}{2}, 2 \cdot d_{ledge}, \frac{S}{2} - \frac{W}{2}, c - \frac{W}{2}\right) \qquad B = 5\ in$$

$L_2 := L + 2 \cdot B$ $\qquad L_2 = 18\ in$

$W_2 := W + 2 \cdot B$ $\qquad W_2 = 31\ in$

$A_2 := L_2 \cdot W_2$ $\qquad A_2 = 558\ in^2$

$m =$ minimum of: *Modification Factor*

$\sqrt{\dfrac{A_2}{A_1}} = 1.8$ *(AASHTO LRFD Eq.5.7.5-3)*

2

$m := min\left(\sqrt{\dfrac{A_2}{A_1}}, 2\right)$ $m = 1.8$

$V_{nb\_ext} := 0.85 \cdot f'_c \cdot A_1 \cdot m$ $V_{nb\_ext} = 936.9\ kip$ *(AASHTO LRFD Eq.5.7.5-2)*

Interior

B = minimum of: *Distance from the perimeter of A_1 to the perimeter of A_2, as shown in the above figures.*

$\left(b_{ledge} - a_v\right) - \dfrac{L}{2} = 5\ in$

$\left(a_v + \dfrac{b_{web}}{2}\right) - \dfrac{L}{2} = 18.5\ in$

$2 \cdot d_{ledge} = 40\ in$

$\dfrac{S}{2} - \dfrac{W}{2} = 33.5\ in$

$B := min\left(\left(b_{ledge} - a_v\right) - \dfrac{L}{2}, \left(a_v + \dfrac{b_{web}}{2}\right) - \dfrac{L}{2}, 2 \cdot d_{ledge}, \dfrac{S}{2} - \dfrac{W}{2}\right)$ $B = 5\ in$

$L_2 := L + 2 \cdot B$ $L_2 = 18\ in$

$W_2 := W + 2 \cdot B$ $W_2 = 31\ in$

$A_2 := L_2 \cdot W_2$ $A_2 = 558\ in^2$

$m =$ minimum of: *Modification Factor*

$\sqrt{\dfrac{A_2}{A_1}} = 1.8$ *(AASHTO LRFD Eq.5.7.5-3)*

2

$m := min\left(\sqrt{\dfrac{A_2}{A_1}}, 2\right)$ $m = 1.8$

$V_{nb\_int} := 0.85 \cdot f'_c \cdot A_1 \cdot m$ $V_{nb\_int} = 936.9\ kip$ *(AASHTO LRFD Eq.5.7.5-2)*

Capacity of Bent 13

Capacity of Bent 13 is the minimum of V_{ns}, V_{nf}, V_{nh}, V_{np}, and V_{nb}

Exterior

V_{n_ext} = minimum of:

$V_{ns\_ext} = 598.5\ kip$ *Expected ledge shear-friction strength*

$V_{nf\_ext} = 307.1\ kip$ *Expected ledge flexure strength*

$V_{nh\_ext} = 204.6\ kip$ *Expected hanger strength*

$V_{np\_ext} = 261.2\ kip$ *Expected punching shear strength*

$V_{nb\_ext} = 936.9\ kip$ *Expected bearing strength*

$V_{n\_ext} := min\left(V_{ns\_ext}, V_{nf\_ext}, V_{nh\_ext}, V_{np\_ext}, V_{nb\_ext}\right) = 204.6\ kip$ Hanger strength control

Interior

V_{n_int} = minimum of:

$V_{ns\_int} = 642.6\ kip$ *Expected ledge shear-friction strength*

$V_{nf\_int} = 308.7\ kip$ *Expected ledge flexure strength*

$V_{nh\_int} = 234.5\ kip$ *Expected hanger strength*

$V_{np\_int} = 345\ kip$ *Expected punching shear strength*

$V_{nb\_int} = 936.9\ kip$ *Expected bearing strength*

$V_{n\_int} := min\left(V_{ns\_int}, V_{nf\_int}, V_{nh\_int}, V_{np\_int}, V_{nb\_int}\right) = 234.5\ kip$ Hanger strength control

A.2 SINGLE-COLUMN BENT (BENT 22)

Dimension

Specimen dimension

$b_f := 63\ in$ *Width of Bottom Flange*

$b_{web} := 30\ in$ *Web Width*

$b_{ledge} := 16.5\ in$ *Single Ledge Width*

$a_v := 7.5\ in$ *Distance from Web Face to Center of Bearing Pad*

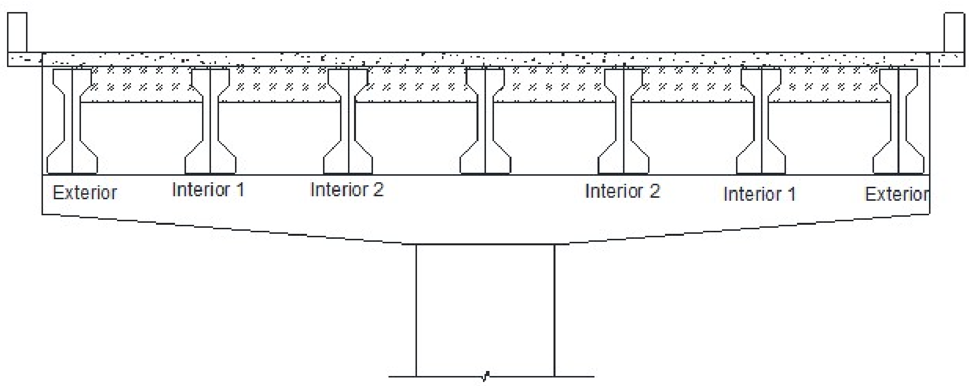
$cover := 2.5\ in$ *Average Concrete Cover of Web*

$a_f := a_v + cover$ $a_f = 10\ in$

$S := 72\ in$ *Girder Spacing*

$C := 16\ in$ *Distance from the Center Line of the Exterior Girder to the Edge of the Cap measured along the Cap*

$brgseat := 1\ in$ *Bearing Seat Buildup, in the specimen there is no bearing seat buildup*



d_{ledge} *Ledge Height*

$d_{ledge\_ext} := 21.75\ in$ *for exterior*

$d_{ledge\_int1} := 27.31\ in$ *for interior1*

$d_{ledge\_int2} := 33.25\ in$ *for interior2*

$h = d_{ledge} + brgseat$

$h_{ext} := d_{ledge\_ext} + brgseat = 22.8\ in$ *for exterior*

$h_{int1} := d_{ledge\_int1} + brgseat = 28.3\ in$ *for interior1*

$h_{int2} := d_{ledge\_int2} + brgseat = 34.3\ in$ *for interior2*

d_e *Distance from top of ledge to centroid of bottom layer of ledge reinforcement*

$d_{e\_ext} := 19.25\ in$ *for exterior*

$d_{e\_int1} := 24.81\ in$ *for interior1*

$d_{e\_int2} := 30.75\ in$ *for interior2*

d_f — *Distance from top layer of ledge reinforcement to bottom of ledge*

$d_{f\_ext} := 18.75\ in$ — *for exterior*

$d_{f\_int1} := 24.31\ in$ — *for interior1*

$d_{f\_int2} := 30.25\ in$ — *for interior2*

Bearing Pad Dimension

$W := 21\ in$ — *Bearing Pad Width*

$L := 8\ in$ — *Bearing Pad Length*

Material Properties

Concrete Strength

$f'_c := 3.6\ ksi$ — *Concrete Strength*

Steel Properties

$f_y := 60\ ksi$ — *Yield Strength of Reinforcement*

$E_s := 29000\ ksi$ — *Young's modulus*

Ledge Shear Friction

Exterior

Distribution Width

b_{s_ext} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$W + 4\ a_v = 51\ in$

$\frac{S}{2} + C = 52\ in$

$\frac{1}{2}\left(W + 4\ a_v\right) + C = 41.5\ in$

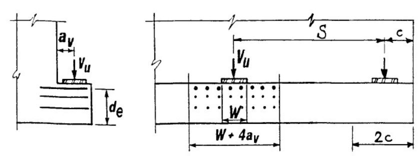


Figure 5.13.2.5.2-1—Design of Beam Ledges for Shear

$b_{s\_ext} := 41.5\ in$

Capacity

V_{ns_ext} = minimum of:

$0.2\ f'_c \cdot b_{s\_ext} \cdot d_{e\_ext} = 575.2\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-1)*

$0.8\ ksi \cdot b_{s\_ext} \cdot d_{e\_ext} = 639.1\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-2)*

$V_{ns\_ext} = 575.2\ kip$

Interior1

Interior1

b_{s_int1} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$W + 4\ a_v = 51\ in$

$S = 72\ in$

$b_{s\_int1} = 51\ in$

Capacity

V_{nc_int1} = minimum of:

$0.2\ f'_c \cdot b_{s\_int1} \cdot d_{e\_int1} = 911\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-1)*

$0.8\ ksi \cdot b_{s\_int1} \cdot d_{e\_int1} = 1012.2\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-2)*

$V_{ns\_int1} = 911\ kip$

Interior2

Interior2

b_{s_int2} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$W + 4\ a_v = 51\ in$

$S = 72\ in$

$b_{s\_int2} = 51\ in$

Capacity

V_{ns_int2} = minimum of:

$0.2\ f'_c \cdot b_{s\_int2} \cdot d_{e\_int2} = 1129.1\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-1)*

$0.8\ ksi \cdot b_{s\_int2} \cdot d_{e\_int2} = 1254.6\ kip$ *(AASHTO LRFD Eq.5.13.2.4.2-2)*

$V_{ns\_int2} = 1129.1\ kip$

Ledge Flexural

$s := 6\ in$ *Spacing of Ledge Reinforcement*

$A_{s5} := 0.31\ in^2$ *Area of #5 bar*

Exterior

Distribution Width

b_{m_ext} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$W + 5\ a_f = 71\ in$

$\frac{S}{2} + C = 52\ in$

$\frac{1}{2}\left(W + 5\ a_f\right) + C = 51.5\ in$

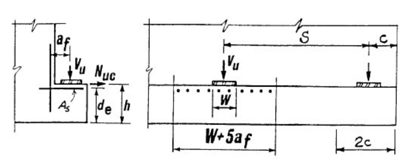


Figure 5.13.2.5.3-1—Design of Beam Ledges for Flexure and Horizontal Force

$b_{m\_ext} := 51.5\ in$

Capacity

$A_s := A_{s5} \cdot 7$ $\quad A_s = 2.17\ in^2$ *Primary Ledge Reinforcement within distribution width*

For combined axial tension and bending

$$T = N_u + \phi \cdot A_s \cdot f_y = \phi \cdot 0.85 \cdot f'_c \cdot a \cdot b$$

$V_u := 207\ kip$ *Exterior Ledge Shear Factored Load*

$N_u := 0.2 \cdot V_u$ $N_u = 41.4\ kip$ *Maximum Concurrent Axial Tension*

$\phi := 0.9$

$$a := \frac{\frac{N_u}{\phi} + A_s \cdot f_y}{0.85\ f'_c \cdot b_{m\_ext}} = 1.1\ in$$

$M_{n\_ext} := A_s \cdot f_y \cdot \left(d_{e\_ext} - \frac{a}{2}\right)$ $M_{n\_ext} = 202.8\ kip \cdot ft$

$M_n = V_n \cdot a_v + N_n \cdot (h - d_e)$ *(AASHTO LRFD 5.13.2.4.1-1)*

$V_{nf\_ext} := \frac{M_{n\_ext}}{a_v + 0.2 \cdot (h_{ext} - d_{e\_ext})}$ $V_{nf\_ext} = 296.8\ kip$

Interior1

Distribution Width

b_{m_int} = minimum of: *(AASHTO LRFD 5.13.2.5.2)*

$W + 5\ a_f = 71\ in$

$S = 72\ in$

$b_{m\_int} := 71\ in$

Capacity

$A_s := A_{s5} \cdot 9$ $A_s = 2.79\ in^2$ *Primary Ledge Reinforcement within distribution width*

For combined axial tension and bending

$T = N_u + \phi \cdot A_s \cdot f_y = \phi \cdot 0.85 \cdot f'_c \cdot a \cdot b$

$V_u := 235\ kip$ *Exterior Ledge Shear Factored Load*

$N_u := 0.2 \cdot V_u$ $N_u = 47\ kip$ *Maximum Concurrent Axial Tension*

$\phi := 0.9$

$$a := \frac{\frac{N_u}{\phi} + A_s \cdot f_y}{0.85\ f'_c \cdot b_{m\_int}} = 1\ in$$

$M_{n\_int1} := A_s \cdot f_y \cdot \left(d_{e\_int1} - \frac{a}{2}\right)$ $M_{n\_int1} = 339\ kip \cdot ft$

$M_n = V_n \cdot a_v + N_n \cdot (h - d_e)$ *(AASHTO LRFD 5.13.2.4.1-1)*

$V_{nf\_int1} := \frac{M_{n\_int1}}{a_v + 0.2 \cdot (h_{int1} - d_{e\_int1})}$ $V_{nf\_int1} = 496.2\ kip$

Interior2

$b_{m\_int} := 71\ in$

Capacity

$A_s := A_{s5} \cdot 9$ $\quad A_s = 2.8\ in^2$ *Primary Ledge Reinforcement within distribution width*

For combined axial tension and bending

$$T = N_u + \phi \cdot A_s \cdot f_y = \phi \cdot 0.85 \cdot f'_c \cdot a \cdot b$$

$V_u := 235\ kip$ *Exterior Ledge Shear Factored Load*

$N_u := 0.2 \cdot V_u$ $\quad N_u = 47\ kip$ *Maximum Concurrent Axial Tension*

$\phi := 0.9$

$$a := \frac{\dfrac{N_u}{\phi} + A_s \cdot f_y}{0.85\ f'_c \cdot b_{m\_int}} = 1\ in$$

$$M_{n\_int2} := A_s \cdot f_y \cdot \left(d_{e\_int2} - \frac{a}{2}\right) \qquad M_{n\_int2} = 421.9\ kip \cdot ft$$

$$M_n = V_n \cdot a_v + N_n \cdot (h - d_e)$$ *(AASHTO LRFD 5.13.2.4.1-1)*

$$V_{nf\_int2} := \frac{M_{n\_int2}}{a_v + 0.2 \cdot (h_{int2} - d_{e\_int2})} \qquad V_{nf\_int2} = 617.4\ kip$$

Hanger Reinforcement

$A_{hr} := 2\ ledgs \cdot A_{s5}$ $\quad A_{hr} := 0.6\ in^2$ *Area of Hanger Reinforcement-2 legs*

Exterior

$s_{ext} := 4.375\ in$ *Spacing of Hanger Reinforcement*

For the Service Limit

V_{sh_ext} = minimum of:

$$\frac{1}{2} \frac{A_{hr} \cdot \left(\dfrac{2}{3} \cdot f_y\right)}{s_{ext}} \cdot \left(\frac{W + 3\ a_v}{2} + C\right) = 103.5\ kip$$

$$\frac{1}{2} \frac{A_{hr} \cdot \left(\dfrac{2}{3}\ f_y\right)}{s_{ext}} \cdot \left(\frac{S}{2} + C\right) = 142.6\ kip$$

(BDM-LRFD Ch.4, Sec. 5, Design Criteria - Modified to limit the distribution width to the edge of the cap (or half the girder spacing and the distance to the edge of the cap). This will prevent distribution widths from overlapping or extending over the edge of the cap.)

$V_{sh\_ext} = 103.5\ kip$

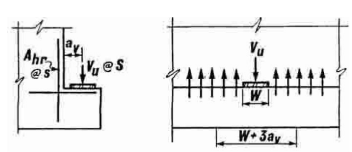


Figure 5.13.2.5.5-1—Single-Ledge Hanger Reinforcement

For the Strength Limit

V_{nh} = minimum of:

$$\frac{1}{2}\left(\frac{A_{hr} \cdot f_y}{s_{ext}} \cdot \left(\frac{S}{2} + C\right)\right) = 213.9\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-2) This equation accounts for hanger reinforcement within shear critical region

$$\frac{1}{2}\left(0.063 \cdot \sqrt{f'_c \cdot ksi} \cdot b_f \cdot d_{f\_ext} + \frac{A_{hr} \cdot f_y}{s_{ext}} \cdot \left(\frac{W + 2 \cdot d_{f\_ext}}{2} + C\right)\right) = 256.8\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-3) This equation accounts for hanger reinforcement within flexural and shear critical region and concrete contribution

$$V_{nh\_ext} = 213.9\ kip$$

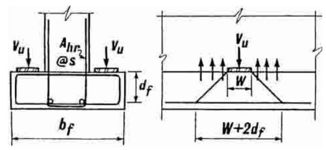


Figure 5.13.2.5.5-2—Inverted T-Beam Hanger Reinforcement

Interior1

$s_{int1} := 5.7\ in$ *Spacing of Hanger Reinforcement*

For the Service Limit

V_{sh_int1} = minimum of:

$$\frac{1}{2}\,\frac{A_{hr} \cdot \left(\frac{2}{3} \cdot f_y\right)}{s_{int1}} \cdot \left(W + 3\ a_v\right) = 91.6\ kip$$

$$\frac{1}{2}\,\frac{A_{hr} \cdot \left(\frac{2}{3}\, f_y\right)}{s_{int1}} \cdot S = 151.6\ kip$$

(BDM-LRFD Ch.4, Sec. 5, Design Criteria - Modified to limit the distribution width to the edge of the cap (or half the girder spacing and the distance to the edge of the cap). This will prevent distribution widths from overlapping or extending over the edge of the cap.)

$$V_{sh\_int1} = 91.6\ kip$$

For the Strength Limit

V_{nh} = minimum of:

$$\frac{1}{2}\left(\frac{A_{hr} \cdot f_y}{s_{int1}} \cdot S\right) = 227.4\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-2) This equation accounts for hanger reinforcement within shear critical region

$$\frac{1}{2}\left(0.063 \cdot \sqrt{f'_c \cdot ksi} \cdot b_f \cdot d_{f\_int1} + \frac{A_{hr} \cdot f_y}{s_{int1}} \cdot \left(W + 2 \cdot d_{f\_int1}\right)\right) = 311.4\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-3) This equation accounts for hanger reinforcement within flexural and shear critical region and concrete contribution

$$V_{nh\_int1} = 227.4\ kip$$

Interior2

$s_{int2} := 3.5\ in$ *Spacing of Hanger Reinforcement*

For the Service Limit

V_{sh_int2} = minimum of:

$$\frac{1}{2} \cdot \frac{A_{hr} \cdot \left(\frac{2}{3} \cdot f_y\right)}{s_{int2}} \cdot \left(W + 3\ a_v\right) = 149.1\ kip$$

$$\frac{1}{2} \cdot \frac{A_{hr} \cdot \left(\frac{2}{3}\, f_y\right)}{s_{int2}} \cdot S = 246.9\ kip$$

$$V_{sh\_int2} = 149.1\ kip$$

(BDM-LRFD Ch.4, Sec. 5, Design Criteria - Modified to limit the distribution width to the edge of the cap (or half the girder spacing and the distance to the edge of the cap). This will prevent distribution widths from overlapping or extending over the edge of the cap.)

For the Strength Limit

V_{nh} = minimum of:

$$\frac{1}{2}\left(\frac{A_{hr} \cdot f_y}{s_{int2}} \cdot S\right) = 370.3\ kip$$

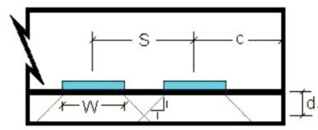
(AASHTO LRFD Eq.5.13.2.5.5-2) This equation accounts for hanger reinforcement within shear critical region

$$\frac{1}{2}\left(0.063 \cdot \sqrt{f'_c \cdot ksi} \cdot b_f \cdot d_{f\_int2} + \frac{A_{hr} \cdot f_y}{s_{int1}} \cdot \left(W + 2 \cdot d_{f\_int2}\right)\right) = 371.3\ kip$$

(AASHTO LRFD Eq.5.13.2.5.5-3) This equation accounts for hanger reinforcement within flexural and shear critical region and concrete contribution

$V_{nh\_int2} = 370.3\ kip$

Punching Shear



TxDOT uses d_f instead of d_e for Punching Shear (BDM-LRFD, Ch. 4, Sec. 5, Design Criteria)

Exterior

V_{np_ext} = minimum of:

$$0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(0.5 \cdot W + L + d_{f\_ext} \cdot \cot(35\ °) + C\right) \cdot d_{f\_ext} = 272.5\ kip$$

$$0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(W + 2 \cdot L + 2 \cdot d_{f\_ext} \cdot \cot(35\ °)\right) \cdot d_{f\_ext} = 402.7\ kip$$

$V_{np\_ext} = 272.5\ kip$

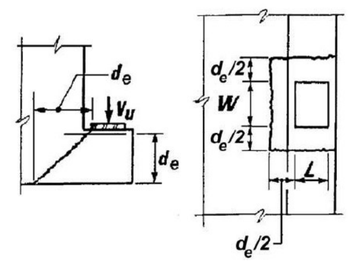


Figure 5.13.2.5.4-1—Design of Beam Ledges for Punching Shear

Interior1

$$V_{np\_int1} := 0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(W + 2 \cdot L + 2 \cdot d_{f\_int1} \cdot \cot(35\ °)\right) \cdot d_{f\_int1}$$

$V_{np\_int1} = 613.7\ kip$

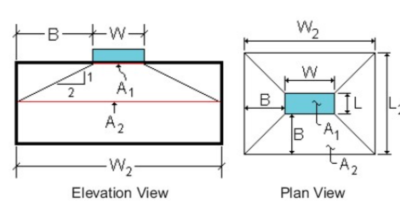
Interior2

$$V_{np\_int2} := 0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(W + 2 \cdot L + 2 \cdot d_{f\_int2} \cdot \cot(35\ °)\right) \cdot d_{f\_int2}$$

$V_{np\_int2} = 885.3\ kip$

Bearing

A_1 is the loaded area (bearing pad area). A_2 is the area of the lowest rectangle contained wholly within the support (the inverted T cap). A_2 must not overlap the truncated pyramid of another load in either direction, nor can it extend beyond the edges of the cap in any direction.



$A_1 := W \cdot L$ $\qquad A_1 = 168\ in^2$ $\qquad$ *Area under Bearing Pad*

Exterior

B = minimum of:

$$\left(b_{ledge} - a_v\right) - \frac{L}{2} = 5\ in$$

Distance from the perimeter of A_1 to the perimeter of A_2, as shown in the above figures.

$$\left(a_v+\frac{b_{web}}{2}\right)-\frac{L}{2}=18.5\ in$$

$$2\cdot d_{ledge\_ext}=43.5\ in$$

$$\frac{S}{2}-\frac{W}{2}=25.5\ in$$

$$C-\frac{W}{2}=5.5\ in$$

$$B:=min\left(\left(b_{ledge}-a_v\right)-\frac{L}{2},\left(a_v+\frac{b_{web}}{2}\right)-\frac{L}{2},2\cdot d_{ledge\_ext},\frac{S}{2}-\frac{W}{2},C-\frac{W}{2}\right)$$

$B=5\ in$

$L_2:=L+2\cdot B$ $\qquad L_2=18\ in$

$W_2:=W+2\cdot B$ $\qquad W_2=31\ in$

$A_2:=L_2\cdot W_2$ $\qquad A_2=558\ in^2$

$m=$ minimum of: *Modification Factor*

$$\sqrt{\frac{A_2}{A_1}}=1.8$$ *(AASHTO LRFD Eq.5.7.5-3)*

2

$m:=min\left(\sqrt{\frac{A_2}{A_1}},2\right)$ $\qquad m=1.8$

$V_{nb\_ext}:=0.85\cdot f'_c\cdot A_1\cdot m$ $\qquad V_{nb\_ext}=936.9\ kip$ *(AASHTO LRFD Eq.5.7.5-2)*

Interior1

B = minimum of: *Distance from the perimeter of A_1 to the perimeter of A_2, as shown in the above figures.*

$$\left(b_{ledge}-a_v\right)-\frac{L}{2}=5\ in$$

$$\left(a_v+\frac{b_{web}}{2}\right)-\frac{L}{2}=18.5\ in$$

$$2\cdot d_{ledge\_int1}=54.6\ in$$

$$\frac{S}{2}-\frac{W}{2}=25.5\ in$$

$$B:=min\left(\left(b_{ledge}-a_v\right)-\frac{L}{2},\left(a_v+\frac{b_{web}}{2}\right)-\frac{L}{2},2\cdot d_{ledge\_int1},\frac{S}{2}-\frac{W}{2}\right)$$ $B=5\ in$

$L_2:=L+2\cdot B$ $\qquad L_2=18\ in$

$W_2:=W+2\cdot B$ $\qquad W_2=31\ in$

$A_2 := L_2 \cdot W_2$ $\qquad A_2 = 558\ in^2$

$m =$ lesser of: *Modification Factor*

$\sqrt{\dfrac{A_2}{A_1}} = 1.8$ *(AASHTO LRFD Eq.5.7.5-3)*

2

$m := min\left(\sqrt{\dfrac{A_2}{A_1}}, 2\right)$ $\qquad m = 1.8$

$V_{nb\_int1} := 0.85 \cdot f'_c \cdot A_1 \cdot m$ $\qquad V_{nb\_int1} = 936.9\ kip$ *(AASHTO LRFD Eq.5.7.5-2)*

Interior2

B = minimum of: *Distance from the perimeter of A_1 to the perimeter of A_2, as shown in the above figures.*

$\left(b_{ledge} - a_v\right) - \dfrac{L}{2} = 5\ in$

$\left(a_v + \dfrac{b_{web}}{2}\right) - \dfrac{L}{2} = 18.5\ in$

$2 \cdot d_{ledge\_int2} = 66.5\ in$

$\dfrac{S}{2} - \dfrac{W}{2} = 25.5\ in$

$B := min\left(\left(b_{ledge} - a_v\right) - \dfrac{L}{2}, \left(a_v + \dfrac{b_{web}}{2}\right) - \dfrac{L}{2}, 2 \cdot d_{ledge\_int2}, \dfrac{S}{2} - \dfrac{W}{2}\right)$ $\qquad B = 5\ in$

$L_2 := L + 2 \cdot B$ $\qquad L_2 = 18\ in$

$W_2 := W + 2 \cdot B$ $\qquad W_2 = 31\ in$

$A_2 := L_2 \cdot W_2$ $\qquad A_2 = 558\ in^2$

$m =$ lesser of: *Modification Factor*

$\sqrt{\dfrac{A_2}{A_1}} = 1.8$ *(AASHTO LRFD Eq.5.7.5-3)*

2

$m := min\left(\sqrt{\dfrac{A_2}{A_1}}, 2\right)$ $\qquad m = 1.8$

$V_{nb\_int2} := 0.85 \cdot f'_c \cdot A_1 \cdot m$ $\qquad V_{nb\_int2} = 936.9\ kip$ *(AASHTO LRFD Eq.5.7.5-2)*

Capacity of Bent 22

Capacity of Bent 22 is the minimum of V_{ns}, V_{nf}, V_{nh}, V_{np}, and V_{nb}

Exterior

V_{n_ext} = minimum of:

$V_{ns\_ext} = 575.2\ kip$	*Expected ledge shear-friction strength*
$V_{nf\_ext} = 296.8\ kip$	*Expected ledge flexure strength*
$V_{nh\_ext} = 213.9\ kip$	*Expected hanger strength*
$V_{np\_ext} = 272.5\ kip$	*Expected punching shear strength*
$V_{nb\_ext} = 936.9\ kip$	*Expected bearing strength*
$V_{n\_ext} := min\left(V_{ns\_ext}, V_{nf\_ext}, V_{nh\_ext}, V_{np\_ext}, V_{nb\_ext}\right) = 213.9\ kip$	Hanger strength control

Interior1

V_{n_int1} = minimum of:

$V_{ns\_int1} = 911\ kip$	*Expected ledge shear-friction strength*
$V_{nf\_int1} = 496.2\ kip$	*Expected ledge flexure strength*
$V_{nh\_int1} = 227.4\ kip$	*Expected hanger strength*
$V_{np\_int1} = 613.7\ kip$	*Expected punching shear strength*
$V_{nb\_int1} = 936.9\ kip$	*Expected bearing strength*
$V_{n\_int1} := min\left(V_{ns\_int1}, V_{nf\_int1}, V_{nh\_int1}, V_{np\_int1}, V_{nb\_int1}\right) = 227.4\ kip$	Hanger strength control

Interior2

V_{n_int2} = minimum of:

$V_{ns\_int2} = 1129.1\ kip$	*Expected ledge shear-friction strength*
$V_{nf\_int2} = 617.4\ kip$	*Expected ledge flexure strength*
$V_{nh\_int2} = 370.3\ kip$	*Expected hanger strength*
$V_{np\_int2} = 885.3\ kip$	*Expected punching shear strength*
$V_{nb\_int2} = 936.9\ kip$	*Expected bearing strength*
$V_{n\_int2} := min\left(V_{ns\_int2}, V_{nf\_int2}, V_{nh\_int2}, V_{np\_int2}, V_{nb\_int2}\right) = 370.3\ kip$	Hanger strength control

APPENDIX B. DESIGN EXAMPLE FOR DOUBLE-COLUMN BENT

B.1 SOLUTION 3: END REGION STIFFENER

Design Problem:

Bent 13 has hanger, ledge flexure, and punching shear deficiencies at exterior girder location based on AASHTO LRFD (2014). End-region stiffener provides alternative load paths so that it can increase ledge and hanger capacities at the exterior girder location.

The required load demands on the exterior ledges are shown below:

$V_{u\_ext} := 247\ kip$ (for single ledge)

$V_{ut\_ext} := V_{u\_ext} \cdot 2 = 494\ kip$ (for both ledges)

$M_{u\_ext} := V_{u\_ext} \cdot a_v + 0.2\ V_{u\_ext} \cdot \left(h - d_e\right) = 168.78\ kip \cdot ft$ *Concurrent ledge moment on a single ledge*

Specify Web Anchor

1. Determine the required shear force for the web anchors(V_{h_req})

$\phi := 0.8$ *Strength reduction factor for normal weight concrete in anchorage zone (AASHTO 5.5.4.2)*

$V_{dh\_ext} := \dfrac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 110.75\ kip$ *Hanger deficiency*

$V_{h\_req} := 2 \cdot V_{dh\_ext} = 221.5\ kip$

Try 1 in. diameter epoxy anchor with high-strength (150 ksi) threadbar

2. Determine required number of anchors

- As an example, epoxy anchor with high strength threadbar manufactured by Williams Form Inc. may be used.

Properties of 1 in. diameter epoxy anchor

$d_a := 1\ in$ *Anchor diameter*

$A_a := 0.85\ in^2$

$f_{ya} := 120\ ksi$

$f_{ut} := 150\ ksi$

$T := A_a \cdot f_{ut} = 127.5\ kip$ *Design load = tensile strength (provided by manufacturer)*

$W_l := 31\ kip$ *Working load (provided by manufacturer)*

$h_{ef} := 25\ in$ *Embedded depth (provided by manufacturer)*

$V_{s3w} := 0.6 \cdot T = 76.5\ kip$ *Shear strength*

$d_h := d_a + \dfrac{1}{8}\ in = 1.13\ in$ *Hole size*

Required number of anchors on the web

$\phi_a := 0.65$ *Strength reduction factor for post-installed anchors with Category 2 (ACI 318-14 17.3.3)*

$n_{w3\_req} := \dfrac{V_{h\_req}}{\phi_a \cdot V_{s3w}} = 4.45$

Try 5 anchors on the web $n_{3w} := 5$

※ Anchors on the ledges may not contribute for hanger resistance. Thus, in this design example, anchors on the ledges are not accounted for in resisting hanger but accounted for in resisting shear and pullout tension force on the ledges.

3. Determine layouts of the anchors

$S_{a\_min} := 6 \cdot d_a = 6\ in$ *Minimum spacing*

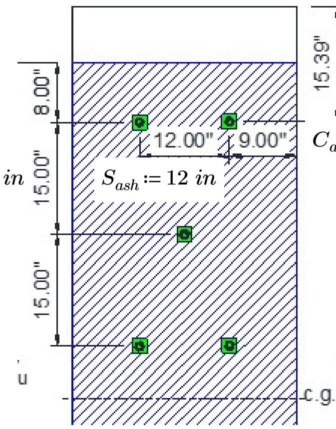
C_{a_min}= maximum of *Minimum edge distance*

$1.5\ in$

$6 \cdot d_a = 6\ in$

$C_{a\_min} := 6\ in$

Try layout shown in figure



$C_{asv} := 15.39\ in$

$C_{ash} := 9\ in$

$S_{asv} := 15\ in$

$S_{ash} := 12\ in$

4. Check that the shear capacity of web anchors is greater than the demand

- According to ACI 318-14, anchors in shear should be checked for steel strength, concrete break out strength, and concrete pryout strength.

Steel Strength of Anchor

$V_{sn3} := V_{s3w} \cdot n_{3w} = 382.5\ kip$

$\phi_a \cdot V_{sn3} = 248.63\ kip$ $>$ $V_{h\_req} = 221.5\ kip$ (O.K.)

Concrete Breakout Strength of Anchor

$S_{cr} := 3\ C_{ash} = 27\ in$ *Critical spacing (ACI 318-14 17.2.1.1)*

$min\left(S_{asv}, S_{ash}\right) \le S_{cr}$ *Group effect shall be considered*

$V_{cb3} = V_{cbg} \cdot 2$ *Concrete breakout strength for shear loading parallel to an edge*

$V_{cbg} = \dfrac{A_{Vc}}{A_{Vco}} \psi_{ec\_v} \cdot \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b$ *Concrete breakout strength for shear loading perpendicular to an edge on a group of anchors*

$A_{Vco} := 4.5 \cdot \left(C_{ash}\right)^2 = 364.5\ in^2$ *Projected area for single anchor in deep member in the direction perpendicular to the shear force*

$A_{Vc} := \left(S_{ash} + 2 \cdot C_{ash}\right)\left(C_{asv} + S_{asv} \cdot 3 + 1.5\ h_{ef}\right) = 2936.7\ in^2 >$ $n_{3w} \cdot A_{Vco} = 1822.5\ in^2$ *Projected area of the failure surface on the side of the concrete member at its edge for a group of anchors*

take $A_{Vc} := 1822.5\ in^2$

$\lambda_a := 1.0$ *Modification factor to reflect the reduced mechanical properties of lightweight concrete*

V_b = minimum of *Basic concrete breakout strength value for a single anchor*

$$V_{ba} := \left(7 \cdot \left(\frac{h_{ef}}{d_a}\right)^{0.2} \cdot \sqrt{d_a}\right) \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{ash}\right)^{1.5} = 682.65\ kip$$

$$V_{bb} := 9\ in^{\frac{1}{2}} \cdot \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{ash}\right)^{1.5} = 461.06\ kip$$

$V_b := min\left(V_{ba}, V_{bb}\right) = 461.06\ kip$

$\psi_{ec\_v} := 1.0$ *Modification factor for anchor groups loaded eccentrically*

$\psi_{ed\_v} := 1.0$ *Modification factor for edge effect*

$\psi_{c\_v} := 1.4$ *Modification factor for cracking effect at service*

$\psi_{h\_v} := 1.0$ *Modification factor for anchors located in narrow concrete member*

$$V_{cbg} := \frac{A_{Vc}}{A_{Vco}} \psi_{ec\_v} \cdot \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b = 3227.42\ kip$$

$\phi_a \cdot V_{cbg} = 2097.82\ kip$ $>$ $V_{h\_req} = 221.5\ kip$ (O.K.)

<u>Concrete Pryout Strength of Anchor</u>

$V_{cpg} = k_{cp} \cdot N_{cpg}$ *Concrete pryout strength*

$k_{cp} := 2.0$ *"k_{cp}" is 2.0 for effective embedded length of anchor larger than 2.5 in.*

$$N_{cpg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba}$$ *Lesser of bond strength of anchor and concrete breakout strength of anchor in tension*

$\tau_{uncr} := 1640\ psi$ *Characteristic bond stress in uncracked concrete*

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}} = 12.21\ in$$ *Rectilinear area that projects outward a distance*

$A_{Nco} := (2 \cdot c_{Na})^2 = 596.36\ in^2$ *Projected area for single anchor in deep member in the direction perpendicular to the shear force*

$A_{Nc} := (S_{ash} + 2 \cdot C_{ash})\ (2\ c_{Na} + 3 \cdot S_{asv}) = 2082.62\ in^2$ $<$ $n_{3w} \cdot A_{Nco} = 2981.82\ in^2$ *Projected area of the failure surface on the side of the concrete member at its edge for a group of anchors*

$A_{Nc} := 2082.62\ in^2$

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_h \cdot h_{ef} = 144.91\ kip$ *Basic bond strength of a single adhesive anchor in tension in cracked concrete*

$\psi_{ec\_Na} := 1.0$ *Modification factor for anchor groups loaded eccentrically*

$$\psi_{ed\_Na} := 0.7 + 0.3\ \frac{C_{ash}}{c_{Na}} = 0.92$$ *Modification factor for edge effect*

$$\psi_{cp\_Na} := \frac{C_{ash}}{c_{Na}} = 0.74$$ *Modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting*

$$N_{cpg} := \frac{A_{Nc}}{A_{Nco}} \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 343.57\ kip$$

$N_{cp3} := k_{cp} \cdot N_{cpg} = 687.15\ kip$

$\phi_a \cdot N_{cp3} = 446.65\ kip$ $>$ $V_{h\_req} = 221.5\ kip$ (O.K.)

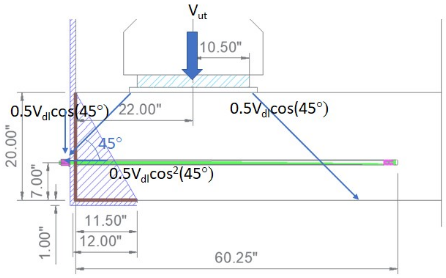
<u>Increased hanger capacity for single ledge</u>

$\phi V_{3h} := min\left(\phi_a \cdot V_{sn3}, \phi_a \cdot V_{cbg}, \phi_a \cdot N_{cp3}\right) = 248.63\ kip$

$$\phi V_{nh} := \frac{\phi V_{3h}}{2} + \phi \cdot V_{nh\_ext} = 282.71\ kip$$

Specify Ledge Anchor

1. Determine the required shear force for the web anchors(V_{l_req})



- Since the anchor hole for the ledge anchor is deeper than holes for the web anchor, try using anchors with smaller diameter.
- As an example, epoxy anchor with B7 threadbar manufactured by Williams Form Inc. may be used
- To ensure embedded depth is not affected by cracking on the ledges, the length of embedded depth for ledge anchors shall be taken as follows:

With the effective embedded depth $h_{ef} := 5.625\ in$, which is the minimum required embedded depth specified by manufacturer

$$h_e := c + \frac{W}{2} + h + h_{ef} = 59.13\ in$$

Properties of 5/8 in. diameter epoxy anchor

$d_a := \frac{5}{8}\ in$	*Anchor diameter*
$A_a := 0.23\ in^2$	
$f_{ya} := 105\ ksi$	
$f_{ut} := 125\ ksi$	
$T := A_a \cdot f_{ut} = 28.75\ kip$	*Design load = tensile strength (provided by manufacturer)*
$W_l := 10.25\ kip$	*Working load (provided by manufacturer)*
$h_{ef} = 5.63\ in$	*Embedded depth*
$V_{s3l} := 0.6 \cdot T = 17.25\ kip$	*Shear strength*
$d_h := d_a + \frac{1}{8}\ in = 0.75\ in$	*Hole size*

Required shear strength for a group of ledge anchors

$V_{nl\_ext} := min\left(V_{ns\_ext}, V_{nf\_ext}, V_{np\_ext}\right) = 261.18\ kip$	*Minimum capacity*
$V_{dl} := 0.5 \cdot \left(\frac{V_{u\_ext}}{\phi} - V_{nl\_ext}\right) \cdot \cos\left(45°\right)^2 = 11.89\ kip$	*Vertical component of ledge deficiency for a single ledge*
$N_{dl} := 0.5 \cdot \left(\frac{V_{u\_ext}}{\phi} - V_{np\_ext}\right) \cdot \cos\left(45°\right)^2 = 11.89\ kip$	*Horizontal component of ledge deficiency for a single ledge*

$$V_{3l\_req} := 2 \cdot V_{dl} = 23.78\ kip$$

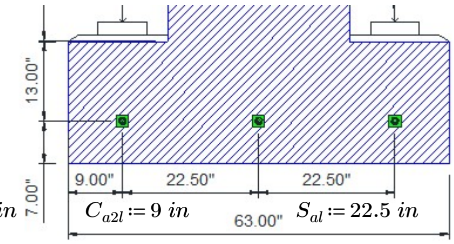
$$N_{3l\_req} := 2 \cdot N_{dl} = 23.78\ kip$$

2. Determine required number of anchors

<u>Required number of anchors on the ledge</u>

$$n_{s3\_req} := \frac{V_{3l\_req}}{\phi_a \cdot V_{s3l}} = 2.12$$

$$n_{s3\_req} := \frac{N_{3l\_req}}{\phi_a \cdot T} = 1.27$$



$C_{a1l} := 7\ in$ $\quad C_{a2l} := 9\ in$ $\quad S_{al} := 22.5\ in$

- Try 3 anchors on the ledge ($n_{3l} := 3$) with layouts shown in figure.
*Place anchors at the bottom kern point. ($C_{a1l} := 7\ in$)

3. Check that the shear and tension capacity of ledge anchors is greater than the demand

<u>Steel Strength of Anchor</u>

$N_{sn3h} := T \cdot n_{3l} = 86.25\ kip$

$\phi_a \cdot N_{sn3h} = 56.06\ kip$ $\quad > \quad$ $N_{3l\_req} = 23.78\ kip$ $\quad$ (O.K.)

$V_{sn3v} := V_{s3l} \cdot n_{3l} = 51.75\ kip$

$\phi_a \cdot V_{sn3v} = 33.64\ kip$ $\quad > \quad$ $V_{3l\_req} = 23.78\ kip$ $\quad$ (O.K.)

<u>Concrete Breakout Strength of Anchor</u>

$S_{cr} := 3\ h_{ef} = 16.88\ in$ — *Critical spacing (ACI 318-14 17.2.1.1)*

Since $S_{al} = 76.33\ in > S_{cr} = 16.88\ in$, group effect shall not be considered.

$N_{cb} = \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ — *Concrete breakout strength of anchor in tension for a group of anchors*

$V_{cb} = \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b$ — *Concrete breakout strength for shear loading perpendicular to an edge on a group of anchors*

$1.5\ h_{ef} = 8.44\ in$

$\lambda_a := 1.0$ — *Modification factor to reflect the reduced mechanical properties of light weight concrete*

$\kappa_c := 17$ — *" κ_c " is 17 for pot-installed anchors*

$$N_b := \kappa_c \cdot \lambda_a \cdot \sqrt{f'_c \cdot ksi} \cdot h_{ef}^{1.5}\ in^{\frac{1}{2}} = 430.31\ kip$$

V_b = minimum of — *Basic concrete breakout strength value for a single anchor*

$$V_{ba} := \left(7 \cdot \left(\frac{h_{ef}}{d_a}\right)^{0.2} \cdot \sqrt{d_a}\right) \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{a1l}\right)^{1.5} = 301.78\ kip$$

$$V_{bb} := 9\ in^{\frac{1}{2}} \cdot \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{a1l}\right)^{1.5} = 316.26\ kip$$

$V_b := min\left(V_{ba}, V_{bb}\right) = 301.78\ kip$

$\psi_{ed\_N} := 0.7 + 0.3 \dfrac{C_{a1l}}{1.5\ h_{ef}} = 0.95$ *Modification factor for edge effect*

$\psi_{ed\_v} := \psi_{ed\_N}$

$\psi_{c\_N} := 1.4 = \psi_{c\_v}$ *Modification factor for cracking effect at service*

$C_{ac} := 2\ h_{ef}$ *The critical edge distance (ACI 318-14 Sec.17.7.6)*

$\psi_{cp\_N} := \dfrac{C_{a1l}}{C_{ac}} = 0.62$

$\psi_{h\_v} := 1.0$ *Modification factor for anchors located in narrow concrete member*

$N_{cbg} := n_{3l} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 762.2\ kip$

$V_{cbg} := n_{3l} \cdot \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b = 1202.68\ kip$

$\phi_a \cdot N_{cbg} = 495.43\ kip$ $>$ $N_{3l\_req} = 23.78\ kip$ (O.K.)

$\phi_a \cdot V_{cbg} = 781.74\ kip$ $>$ $V_{3l\_req} = 23.78\ kip$ (O.K.)

Bond Strength of Anchor

$h_{ef} = 5.63\ in$ *Embedded depth*

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\dfrac{\tau_{uncr}}{1100\ psi}} = 7.63\ in$ *Rectilinear area that projects outward a distance*

$S_{cr} := 2\ c_{Na} = 15.26\ in$

Since $S_{al} = 22.5\ in > S_{cr} = 15.26\ in$, group effect shall not be considered.

$\tau_{uncr} := 1640\ psi$ *Characteristic bond stress in un-cracked concrete*

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_h \cdot h_{ef} = 21.74\ kip$ *Basic bond strength of a single adhesive anchor in tension in cracked concrete*

$\psi_{ed\_Na} := 0.7 + 0.3 \dfrac{C_{a1l}}{c_{Na}} = 0.98$ *Modification factor for edge effect*

$\psi_{cp\_Na} := \dfrac{C_{a1l}}{c_{Na}} = 0.92$ *Modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting*

$N_a := \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 19.44\ kip$

$N_{a3} := n_{3l} \cdot N_a = 58.33\ kip$

$\phi_a \cdot N_{a3} = 37.91\ kip$ $>$ $N_{3l\_req} = 23.78\ kip$ (O.K.)

Concrete Pryout Strength of Anchor

$V_{cpg} = k_{cp} \cdot N_{cpg}$ — *Concrete pryout strength*

$k_{cp} := 2.0$ — *"k_{cp}" is 2.0 for effective embedded length of anchor larger than 2.5 in.*

$\tau_{uncr} := 1640\ psi$ — *Characteristic bond stress in un-cracked concrete*

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\dfrac{\tau_{uncr}}{1100\ psi}} = 7.63\ in$ — *Rectilinear area that projects outward a distance*

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_h \cdot h_{ef} = 21.74\ kip$ — *Basic bond strength of a single adhesive anchor in tension in cracked concrete*

$\psi_{ed\_Na} := 0.7 + 0.3\ \dfrac{C_{a1l}}{c_{Na}} = 0.98$ — *Modification factor for edge effect*

$\psi_{cp\_Na} := \dfrac{C_{a1l}}{c_{Na}} = 0.92$ — *Modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting*

$N_{cpg} := n_{3l} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 58.33\ kip$

$N_{cp3} := k_{cp} \cdot N_{cpg} = 116.66\ kip$

$\phi_a \cdot N_{cp3} = 75.83\ kip$ $>$ $N_{3l\_req} = 23.78\ kip$ (O.K.)

Increased hanger capacity for single ledge

$\phi V_{3l} := min\left(\phi_a \cdot N_{sn3h}, \phi_a \cdot V_{sn3v}, \phi_a \cdot N_{cbg}, \phi_a \cdot V_{cbg}, \phi_a \cdot N_{a3}, \phi_a \cdot N_{cp3}\right) = 33.64\ kip$

$$\phi V_{nl} := \frac{\phi V_{3l}}{\cos(45°)^2} + \phi \cdot V_{nl\_ext} = 276.22\ kip$$

4. Check the interaction between the shear and tension of ledge anchors

$$\left(\frac{V_{3l\_req}}{V_{s3l} \cdot n_{3l}}\right)^{\frac{5}{3}} + \left(\frac{N_{3l\_req}}{T \cdot n_{3l}}\right)^{\frac{5}{3}} = 0.39 \quad < 1.0 \qquad \text{(O.K.)}$$

End Plate Design

1. Determine required thickness of end plate

Grade 50 steel end plate

$F_y := 50\ ksi$ — *Yield stress*

$F_u := 65\ ksi$ — *Ultimate stress*

$E := 29000\ ksi$ — *Young's modulus*

Required thickness for hanger deficiency

$V_{dh\_ext} = 110.75\ kip$

$V_{nh3\_ext} = b_{stem} \cdot t_d \cdot F_y$

$$t_{d\_req} := \frac{V_{dh\_ext}}{b_{stem} \cdot F_y} = 0.07\ in$$

Required thickness for axial bearing

$\phi_t := 0.75$ *Resistance factor (AISC Specification J3.10)*

$$t_{a\_req} := \frac{W_l}{\phi_t \cdot 2.4\ d_h \cdot F_u} = 0.12\ in$$

Required thickness for shear bearing

$$V_{s3} := \max\left(\frac{V_{h\_req}}{n_{3w}}, \frac{V_{3l\_req}}{n_{3l}}\right) = 44.3\ kip$$

$\phi_s := 0.65$ *Resistance factor (AISC Specification J3.10)*

$$t_{sb\_req} := \frac{V_{s3}}{\phi_s \cdot 2.0\ d_h \cdot F_u} = 0.7\ in$$

Required thickness for shear rupture

$\phi_r := 0.75$

$h_n := C_{a1l} - \frac{d_h}{2} = 6.63\ in$ *Distance from edge of the plate to the edge of the nearest hole*

$$t_{sr\_req} := \frac{V_{s3}}{\phi_r \cdot 0.6 \cdot h_n \cdot F_y} = 0.3\ in$$

$t_{req} := \max\left(t_{d\_req}, t_{a\_req}, t_{sb\_req}, t_{sr\_req}\right) = 0.7\ in$ *Minimum thickness of plate*

$t := 0.75\ in$

Design Triangular Stiffener

- Since the thickness of end region stiffener is limited to 1 in. for practical reason, if the minimum thickness is thicker than 1in. a triangular stiffener should be designed to resist bending of the plate. In this design example, the thickness of the plate is less than 1 in. Thus, this stiffener is only designed for shear and tension.

1. Determine required thickness for horizontal force

Grade 50 steel end plate

$F_y := 50\ ksi$ *Yield stress*

$F_u := 65\ ksi$ *Ultimate stress*

$E := 29000\ ksi$ *Young's modulus*

$b := 20\ in$

$a := 11.5\ in$

$\frac{a}{b} = 0.58$

$P_u := 0.5 \cdot V_{dl} \cdot \cos(45°)^2 = 2.97\ kip$

$P_n := \dfrac{P_u}{\phi} = 3.72\ kip$

$s_h := 10\ in$

$m_s := \dfrac{P_n \cdot s_h}{b^3 \cdot E} = 1.6 \cdot 10^{-7}$ *Dimensionless moment*

From the design aid table (Shakya and Vinnakota, 2008) and using interpolation,

$\dfrac{t_s}{b} = 7.38 \cdot 10^{-3}$

Thus, the plate thickness t_s is

$t_{s\_req} := b \cdot 7.38 \cdot 10^{-3} = 0.15\ in$

Try $t_s := 0.25\ in$

<u>Check minimum thickness</u>

$\dfrac{t}{b} > 0.0230$ for $F_y = 50\ ksi$ and $\dfrac{a}{b} = 0.58$

$t_{s\_min} := 0.0230 \cdot b = 0.46\ in$ > $t_s = 0.25\ in$ (N.G.)

Try $t_s := 0.5\ in$ > $t_{s\_min} = 0.46\ in$ (O.K.)

2. Determine required thickness for vertical force

$\dfrac{b}{a} = 1.74$

$P_u := 0.5 \cdot V_{dl} \cdot \cos(45°)^2 = 2.97\ kip$

$P_n := \dfrac{P_u}{\phi} = 3.72\ kip$

$s_v := \dfrac{a}{2} = 5.75\ in$

$m_s := \dfrac{P_n \cdot s_v}{a^3 \cdot E} = 4.84 \cdot 10^{-7}$

From the design aid table (Shakya and Vinnakota, 2008) and using interpolation,

$\dfrac{t_s}{a} = 8.58 \cdot 10^{-3}$

Thus, the plate thickness t_s is

$t_{s\_req} := a \cdot 8.58 \cdot 10^{-3} = 0.1\ in$

Check minimum thickness

$\frac{t}{a} > 0.0399$ for $F_y = 50\ ksi$ and $\frac{b}{a} = 1.74$

$t_{s\_min} := 0.0399 \cdot a = 0.46\ in$ < $t_s = 0.5\ in$ (O.K.)

3. Determine weld size a_w

$a_{w\_min} := \frac{5}{16}\ in = 0.31\ in$ *(AISC Specification Table J2.4)*

$a_{w\_max} := t - \frac{1}{16}\ in = 0.69\ in$ *(AISC Specification J2.2)*

With E70 electrodes,

$F_{EXX} := 70\ ksi$

$F_{nw} = 0.6\ F_{EXX}\left(1.0 + 0.5\ sin(\theta)^{1.5}\right)$ *(AISC Specification J2-4)*

$\phi := 0.75$ *(AISC Specification J2.4)*

$\phi F_{nw} \cdot A_{we} = \phi F_{nw} \cdot t_e \cdot L = \phi F_{nw} \cdot cos(45°) \cdot a_w \cdot L$ *Fillet weld strength*

For vertical weld

$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin(0°)^{1.5}\right) = 42\ ksi$

$L := b = 20\ in$

$a_{w\_req} := \frac{P_n}{\phi \cdot F_{nw} \cdot \cos(45°) \cdot L} = 0.01\ in$

For horizontal weld

$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin(0°)^{1.5}\right) = 42\ ksi$

$L := a = 11.5\ in$

$a_{w\_req} := \frac{P_n}{\phi \cdot F_{nw} \cdot \cos(45°) \cdot L} = 0.01\ in$

Use 0.5 in. fillet weld for both sides

B.2 SOLUTION 8: CLAMPED THREADBAR WITH CHANNEL

Design Problem:

Bent 13 has hanger, ledge flexure, and punching shear deficiencies based on AASHTO LRFD (2014). Clamped threadbar with channel will be designed to strengthen the bent cap.

The required loads on the ledges are shown below:

Exterior

$V_{u\_ext} := 247\ kip$ (for single ledge)

$V_{ut\_ext} := V_{u\_ext} \cdot 2 = 494\ kip$ (for both ledges)

$M_{u\_ext} := V_{u\_ext} \cdot a_v + 0.2\ V_{u\_ext} \cdot \left(h - d_e\right) = 168.78\ kip \cdot ft$

Interior

$V_{u\_int} := 287\ kip$ (for single ledge)

$V_{ut\_int} := V_{u\_int} \cdot 2 = 574\ kip$ (for both ledges)

$M_{u\_int} := V_{u\_int} \cdot a_v + 0.2\ V_{u\_int} \cdot \left(h - d_e\right) = 196.12\ kip \cdot ft$

Specify Threadbar

1. Determine the hanger deficiency for single ledge

$\phi := 0.9$ *Resistance factor (AASHTO LRFD 5.5.4.2)*

Exterior

$$V_{hd\_ext} := \frac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 76.44\ kip$$

Interior

$$V_{hd\_int} := \frac{V_{u\_int}}{\phi} - V_{nh\_int} = 89.88\ kip$$

2. Determine the required threadbar contribution

Exterior

$V_{ext\_req} := V_{hd\_ext} \cdot 2 = 152.89\ kip$

Interior

$V_{int\_req} := V_{hd\_int} \cdot 2 = 179.76\ kip$

3. Determine the required area of the threadbar

Threadbar properties:

Use high strength threadbar (150 ksi threadbar)

$f_y := 120\ ksi$ *Yield strength*

$f_u := 150\ ksi$ *Ultimate strength*

Exterior

$$A_{ext\_req} := \frac{V_{ext\_req}}{f_y} = 1.27\ in^2$$

Interior

$A_{int\_req} := \dfrac{V_{int\_req}}{f_y} = 1.5\ in^2$

4. Determine the number of threadbars

Try 1 in. diameter 150 ksi threadbar

$d_b := 1\ in$ *Bar diameter*

$A := 0.85\ in^2$ *Net area of the bar*

$f_y := 120\ ksi$ *Yield strength*

$f_u := 150\ ksi$ *Ultimate strength*

Exterior

$n_{t\_ext\_req} := \dfrac{A_{ext\_req}}{A} = 1.5$ $n_{ext} := 2$

Interior

$n_{t\_int\_req} := \dfrac{A_{int\_req}}{A} = 1.76$ $n_{int} := 2$

※ By increasing bar size, the number of bars can be reduced, but it should be aligned with the number of channels.

5. Check that the capacity is greater than the demand (service limit and strength limit)

$V_t := A \cdot f_y = 102\ kip$ *Tensile strength - contribution of a threadbar*

Exterior

$V_{sh\_ext} = 175\ kip$ *Exterior hanger capacity at service limit for single ledge*

$V_{slt\_ext} := 371\ kip$ *Exterior service load for both ledges*

$V_{s\_ext} := V_t \cdot n_{ext} + V_{sh\_ext} \cdot 2 = 554\ kip$ (service limit)

$\phi \cdot V_{s\_ext} = 498.6\ kip$ > $V_{slt\_ext} = 371\ kip$ (O.K.)

$V_{nr\_ext} := V_t \cdot n_{ext} + V_{nh\_ext} \cdot 2 = 600\ kip$ (strength limit)

$\phi \cdot V_{nr\_ext} = 540\ kip$ > $V_{ut\_ext} = 494\ kip$ (O.K.)

Interior

$V_{sh\_int} = 130.5\ kip$ *Interior hanger capacity at service limit for single ledge*

$V_{slt\_int} := 385.2\ kip$ *Interior service load for both ledges*

$V_{s\_int} := V_t \cdot n_{int} + V_{sh\_int} \cdot 2 = 465\ kip$ (service limit)

$\phi \cdot V_{s\_int} = 418.5\ kip$ > $V_{slt\_int} = 385.2\ kip$ (O.K.)

$V_{nr\_int} := \left(V_t \cdot n_{int} + V_{nh\_int} \cdot 2\right) = 662.02\ kip$ (strength limit)

$\phi \cdot V_{nr\_int} = 595.82\ kip$ > $V_{ut\_int} = 574\ kip$ (O.K.)

6. Determine spacing between the threadbars

$S_{t\_min} := 2.75 \cdot d_b = 2.75\ in$ *Minimum spacing (AISC Specification J6)*

<u>Exterior</u>

$b_{h\_ext} := min\left(\dfrac{W+3\ a_v}{2}+c, \dfrac{W+2\ d_f}{2}+c, \dfrac{S}{2}+c\right)$ *Hanger distribution width for exterior*

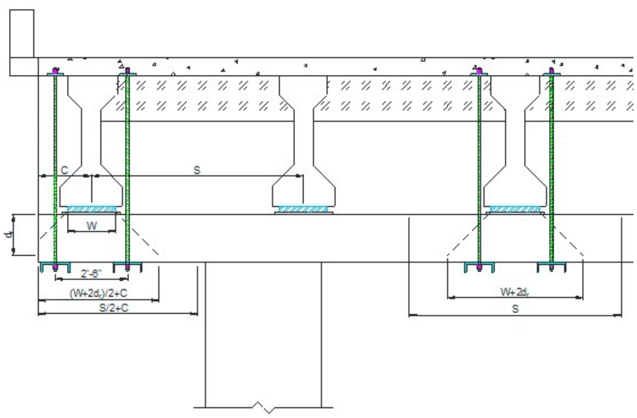
$S_{t\_max\_ext} := \dfrac{b_{h\_ext}}{n_{ext}-1} = 43.75\ in$ *Maximum spacing for exterior*

<u>Interior</u>

$b_{h\_int} := min\left(W+3\ a_v, W+2\ d_f, S\right)$ *Hanger distribution width for interior*

$S_{t\_max\_int} := \dfrac{b_{s\_int}}{n_{int}-1} = 51\ in$ *Maximum spacing for interior*

Try 30 in. spacing between the threadbars ($S := 30\ in$)



Specify Channel

1. Determine the required elastic section modulus and plastic section modulus for a channel

- The required nominal ledge flexure strength can be obtained by

$M_{n\_req} = \dfrac{M_u}{\phi} - M_n$

- typical channels have compact section, nominal strength of the channel is

$M_n = min\left(1.6\ F_y \cdot S_y, F_y \cdot Z_y\right)$ *(AISC Specification Eq. F6-1)*

<u>A36 steel channel properties</u>

$F_y := 36\ ksi$ *Yield stress of the channel*

$F_u := 65\ ksi$ *Ultimate stress of the channel*

$E := 29000\ ksi$ *Young's modulus of the channel*

<u>Exterior</u>

$M_{n\_ext\_req} := \dfrac{M_{u\_ext}}{\phi} - M_{n\_ext} = -15.68\ kip \cdot ft$

No deficiency for exterior

Interior

$M_{n\_int\_req} := \dfrac{M_{u\_int}}{\phi} - M_{n\_int} = 13.65\ kip \cdot ft$

$S_{int\_req} := \dfrac{M_{n\_int\_req}}{1.6\ F_y \cdot n_{int}} = 1.42\ in^3$ *Required elastic section modulus for a channel*

$Z_{int\_req} := \dfrac{M_{n\_int\_req}}{F_y \cdot n_{int}} = 2.27\ in^3$ *Required plastic section modulus for a channel*

2. Determine the required web thickness of the channel (t_{req})

$V_{pt} := 0.6 \cdot f_u \cdot A = 76.5\ kip$ *Design load for anchoring (Williams Form)*

$\phi_t := 0.75$ *Resistance factor (AISC Specification J3.10)*

Required thickness for axial bearing

$t_{min\_req} := \dfrac{V_{pt}}{\phi_t \cdot 2.4\ d_b \cdot F_u} = 0.65\ in$ *Minimum web thickness for bearing*

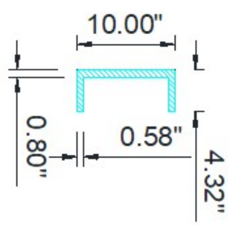
- A channel should be chosen based on required elastic modulus, plastic modulus, and web thickness of the channel.
  Try C10x30 Channel

3. Calculate contribution of a channel (M_c and V_c)

MC10x41 Channel properties

$t_w := 0.80\ in$ > $t_{min\_req} = 0.65\ in$ (O.K.)

$b_f := 4.32\ in$



$t_f := 0.58\ in$

$d := 10\ in$

$S_y := 1.65\ in^3$

$Z_y := 3.78\ in^3$

Contribution of a channel

M_c = minimum of

$1.6\ F_y \cdot S_y = 7.92\ kip \cdot ft$

$F_y \cdot Z_y = 11.34\ kip \cdot ft$

$M_c = 7.92\ kip \cdot ft$

$V_c := \dfrac{M_c}{a_v} = 12.67\ kip$

4. Check that the ledge flexure capacity is greater than the demand

Exterior

$M_{n\_ext} = 203.22\ kip \cdot ft$ *Ledge flexure capacity for single ledge*

$\phi \cdot M_{n\_ext} = 182.9\ kip \cdot ft$ > $M_{u\_ext} = 168.78\ kip \cdot ft$ (No channel needed)

Interior

$M_{n\_int} = 204.26\ kip \cdot ft$ *Ledge flexure capacity for single ledge*

$M_{nc\_int} := \left(M_c \cdot n_{int} + M_{n\_int}\right) = 220.1\ kip \cdot ft$

$\phi \cdot M_{nc\_int} = 198.09\ kip \cdot ft$ > $M_{u\_int} = 196.12\ kip \cdot ft$ (O.K.)

$V_{nc\_int} := V_c \cdot n_{int} + V_{nf\_int} = 324.26\ kip$

$\phi \cdot V_{nc\_int} = 291.84\ kip$ > $V_{u\_int} = 287\ kip$ (O.K.)

5. Check spacing between assembled Solution 8 (threadbar + channel)

Minimum spacing

$S_{c\_min} := d = 10\ in$ < $S = 30\ in$ (O.K.)

Maximum spacing

Interior

$b_{m\_ext} = 57.5\ in$ *Exterior distribution width for ledge flexure*

$S_{c\_max\_ext} := \dfrac{b_{m\_ext}}{n_{ext} - 1} - d = 47.5\ in$ > $S = 30\ in$ (O.K.)

Exterior

$b_{m\_int} = 71\ in$ *Interior distribution width for ledge flexure*

$S_{c\_max\_int} := \dfrac{b_{m\_int}}{n_{int} - 1} - d = 61\ in$ > $S = 30\ in$ (O.K.)

Bearing Plate Design

1. Determine required thickness of plate

$V_{pt} = 76.5\ kip$ *Design load for anchoring (Williams Form)*

$\phi_t := 0.75$ *Resistance factor (AISC Specification J3.10)*

A36 steel properties

$F_y := 36\ ksi$ *Yield stress of the channel*

$F_u := 65\ ksi$ *Ultimate stress of the channel*

$E := 29000\ ksi$ *Young's modulus of the channel*

<u>Required thickness for axial bearing</u>

$$t_{req} := \frac{V_{pt}}{\phi_t \cdot 2.4\ d_b \cdot F_u} = 0.65\ in$$

Use 0.75 in. thickness plate

2. Determine required bearing area

$\phi_c := 0.65$ *Resistance factor (AISC Specification J8)*

$$A_{req} := \frac{V_{pt}}{\phi_c \cdot 0.85 \cdot f'_c} = 38.46\ in^2$$

Use 5 in. x 8 in. rectangular plate with 0.75 in. thickness

B.3 SOLUTION 14: LOAD BALANCING POST-TENSIONING

Design Problem:

Bent 13 has hanger, ledge flexure, and punching shear deficiencies for both exterior and interior ledge based on AASHTO LRFD (2014). Load balancing post tensioning strand will provide alternative load paths so that it can improve overall bent capacities.

The required load demands on single ledge are shown below:

Exterior

$V_{u\_ext} := 247\ kip$ (for single ledge)

$M_{u\_ext} := V_{u\_ext} \cdot a_v + 0.2\ V_{u\_ext} \cdot \left(h - d_e\right) = 168.78\ kip \cdot ft$

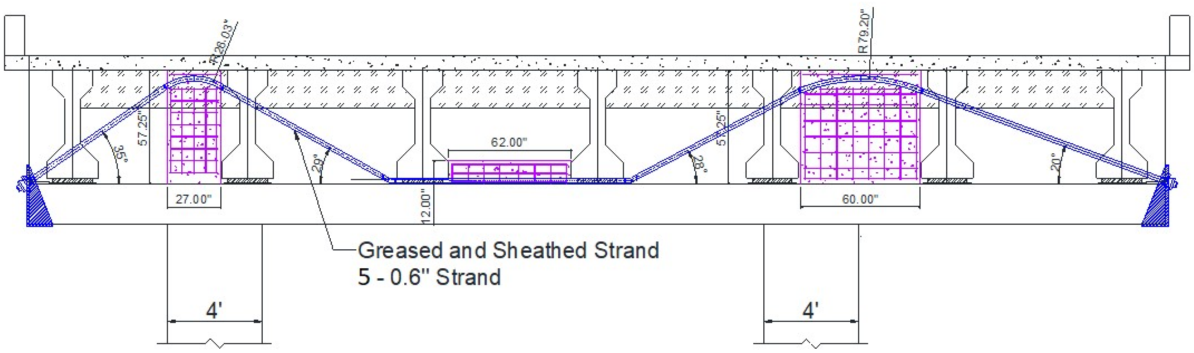
Interior1

$V_{u\_int} := 287\ kip$ (for single ledge)

$M_{u\_int} := V_{u\_int} \cdot a_v + 0.2\ V_{u\_int} \cdot \left(h - d_e\right) = 196.12\ kip \cdot ft$

Concrete infill block

- To transfer load from the strands to the column, the concrete infill block needs to be seated on the column, and center of gravity of the concrete block should be placed on the column. For this design example, the dimension of the concrete infill block is shown below. Only minimum reinforcement is needed for the concrete infill block.



Reinforcement of Infill Concrete Block

1.Determine the maximum deficiency

$\phi := 0.9$ *Resistance factor (AASHTO LRFD 5.5.4.2)*

Exterior

$$V_{d\_ext} := \frac{V_{u\_ext}}{\phi} - V_{n\_ext} = 69.84\ kip$$

Interior

$$V_{d\_int} := \frac{V_{u\_int}}{\phi} - V_{n\_int} = 84.38\ kip$$

Exterior

$$V_{14\_ext\_req} := \max\left(\frac{V_{d\_ext}}{\sin\left(35°\right)}, \frac{V_{d\_ext}}{\sin\left(20°\right)}\right) = 204.21\ kip$$

<u>Interior</u>

$$V_{14\_int\_req} := \max\left(\frac{V_{d\_int}}{\sin(29°)}, \frac{V_{d\_int}}{\sin(28°)}\right) = 179.73\ kip$$

2. Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5\ in$ — *Width of infill concrete block*

$h_c := 56.75\ in$ — *Height of infill concrete block*

$d_{e\_max} := 60\ in$ — *The maximum length of infill concrete block*

$f'_c := 3.6\ ksi$ — *Concrete strength (use same concrete as the in-service structure)*

$f_y := 60\ ksi$ — *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = the maximum of:

$$\frac{3\sqrt{f'_c \cdot psi}}{f_y} b_w \cdot d_{e\_max} = 2.97\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_{e\_max} = 3.3\ in^2$$

$A_{f\_min} := 3.3\ in^2$ — *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

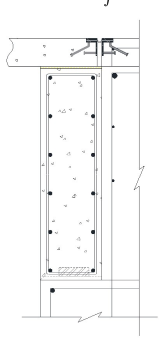
$s_{f\_max} := 12\ in$ — *(ACI 318-14)*

$n_{re} := \frac{h_c}{s_{f\_max}} + 1 = 5.73$ — *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 6$ — *Take integer*

$n_f := n_{re} \cdot 2 = 12$ — *Required number of longitudinal reinforcement*

$A_{fs\_re} := \frac{A_{f\_min}}{n_f} = 0.28\ in^2$ — *Required area of single longitudinal bar*



<u>Place #5 ($A_{fs} := 0.3\ in^2$) longitudinal bars at four corners of the concrete block, and evenly place four #5 longitudinal bars along the height of the concrete block on each side.</u>

3.Determine required shear reinforcement

- Use double leg #4 stirrup

$A_v := 2 \cdot 0.2\ in^2 = 0.4\ in^2$

- Concrete shear strength

$V_c := 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d_e = 107\ kip$

- Required spacing of stirrups

$V_{d\_max} := \max\left(V_{14\_ext\_req}, V_{14\_int\_req}\right)$

$$s_{v\_req} := \frac{A_v \cdot f_y \cdot d_e}{V_{d\_max} - V_c} = 2.06\ in$$

- Check maximum spacing of stirrups

s_{v_max} = the maximum of:

$$\frac{d_{e\_max}}{2} = 30\ in$$

$$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c \cdot psi}\ b_w} = 32.32\ in$$

$$\frac{A_v \cdot f_y}{50\ psi \cdot b_w} = 29.09\ in$$

$s_{v\_max} := 29.09\ in$

<u>Consider the geometry of the concrete infill block, evenly place four #4 double leg stirrups at a spacing of 25 in.</u>

<u>Specify PT Strand</u>

1. Determine required number of strands (try 0.6 in. strand)

<u>Properties of 0.6" strand</u>

$f_{pu} := 270\ ksi$	*Ultimate stress*
$A_{pt} := 0.217\ in^2$	*Net area of strand*
$P_{pu} := f_{pu} \cdot A_{pt} = 58.59\ kip$	*Ultimate strength of strand*
$P_{ps} := 0.7 \cdot P_{pu} = 41.01\ kip$	*Maximum force after transfer of prestressing force*
$f_{py} := 0.85 \cdot f_{pu} = 229.5\ ksi$	*Yield stress*
$f_{pe} := 0.8\ f_{py} = 183.6\ ksi$	*Stress at service limit state after losses*
$P_{pe} := f_{pe} \cdot A_{pt} = 39.84\ kip$	*Strength at service limit state after losses*

Required number of strands

Exterior

$$n_{14\_ext\_req} := \frac{V_{14\_ext\_req}}{P_{ps}} = 4.98$$

Interior

$$n_{14\_int\_req} := \frac{V_{14\_int\_req}}{P_{ps}} = 4.38$$

use 5 - 0.6" strands ($n_{14} := 5$)

4. Calculate contribution of a strand (V_{14})

Use smaller angle of left and right overhangs for exterior and interior

Exterior

$$V_{14\_ext} := P_{ps} \cdot \sin\left(20°\right) = 14.03\ kip$$

Interior

$$V_{14\_int} := P_{ps} \cdot \sin\left(28°\right) = 19.25\ kip$$

3. Check that the shear capacity is greater than the demand

Exterior

$$V_{n\_ext} := \left(V_{14\_ext} \cdot n_{14} + V_{n\_ext}\right) = 274.74\ kip$$

$\phi \cdot V_{n\_ext} = 247.26\ kip$ > $V_{u\_ext} = 247\ kip$ (O.K.)

Interior

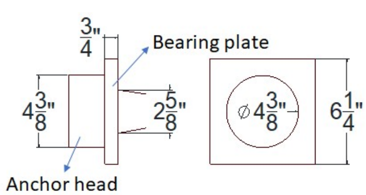
$$V_{n\_int} := \left(V_{14\_int} \cdot n_{14} + V_{n\_int}\right) = 330.78\ kip$$

$\phi \cdot V_{n\_int} = 297.7\ kip$ > $V_{u\_int} = 287\ kip$ (O.K.)

4. Determine Anchorage

With 4 - 0.6" strands, the anchorage will be chosen from manufacturer's multi strands anchorage catalog.

- For an example Type E 0.6 (unit 6-4) by VSL may be used, and its dimension is shown below:



Anchor System Design

1. Determine required thickness of end plate

$$P_{pt} := n_{14} \cdot P_{pe} = 199.21\ kip$$

A36 steel end plate and beveled properties

$F_y := 36\ ksi$ *Yield stress*

$F_u := 65\ ksi$ *Ultimate stress*

$E := 29000\ ksi$ *Young's modulus*

- Assume the diameter of the strand bundle is 4.65 in.

$d_s := 4.65\ in$

with 5 in. hole and beveled plate size, the height of vertical plate is determined

$$b := 20\ in + \frac{5}{2}\ in + \frac{17.5}{2}\ in = 31.25\ in$$

Required thickness for axial bearing

$\phi_b := 0.75$ *Resistance factor (AISC Specification J3.10)*

$$t_{a\_req} := \frac{P_{pt} \cdot \cos(20°)}{\phi_b \cdot 2.4\ d_s \cdot F_u} = 0.34\ in$$

Required thickness for shear bearing

$$t_{a\_req} := \frac{P_{pt} \cdot \sin(35°)}{\phi_b \cdot 2.0\ d_s \cdot F_u} = 0.25\ in$$

Required thickness for shear yielding

$\phi_y := 1$

$$t_{a\_req} := \frac{P_{pt} \cdot \sin(35°)}{\phi_y \cdot 0.6\ b \cdot F_y} = 0.17\ in$$

Required thickness for shear rupture

$\phi_r := 0.75$

$h_n := 5.5\ in$

$$t_{a\_req} := \frac{P_{pt} \cdot \sin(35°)}{\phi_r \cdot 0.6 \cdot h_n \cdot F_y} = 1.28\ in$$

Use 1.5 in thick anchor plates ($t_a := 1.5\ in$)

2. Determine dimension of beveled plate

$\phi_c := 0.65$ *Resistance factor (AISC Specification J8)*

$$A_{req} := \frac{P_{pt}}{\phi_c \cdot 0.85 \cdot f'_c} = 100.15\ in^2$$

$$d_{req} := \sqrt{\frac{A_{req} \cdot 4}{\pi}} = 11.29\ in$$

Use 12 in. outer diameter beveled plate for whole bundle of strands

3. Determine dimensions of the triangular stiffener

- Since the recommended plate aspect ratio, a/b, ranges from 0.5 to 3.0, try a/b=0.5

$a := b \cdot 0.5 = 15.63\ in$

$P_u := P_{pt} \cdot \cos(20°) = 187.19\ kip$

$P_n := \frac{P_u}{\phi} = 207.99\ kip$

$s_s := 22.5\ in$

$m_s := \frac{P_n \cdot s_s}{b^3 \cdot E} = 5.29 \cdot 10^{-6}$ *Dimensionless moment*

- From the design aid table (Shakya and Vinnakota, 2008) and using interpolation,

$\frac{t_s}{b} = 11.20 \cdot 10^{-3}$

Thus, the plate thickness t_s is

$t_s := b \cdot 11.20\ 10^{-3} = 0.35\ in$

Use 0.625 in. triangular plate ($t_s := 0.625\ in$)

Check minimum thickness

$\frac{t}{b} > 0.0188$ for $F_y = 36\ ksi$ and $\frac{a}{b} = 0.5$

$t_{s\_min} := 0.0188 \cdot b = 0.59\ in$ < $t_s = 0.63\ in$ (O.K.)

4. Determine weld size a

$a_{min} := \frac{1}{4}\ in = 0.25\ in$ *(AISC Specification Table J2.4)*

$a_{max} := t_a - \frac{1}{16}\ in = 1.44\ in$ *(AISC Specification J2.2)*

With E70 electrodes,

$F_{EXX} := 70\ ksi$

$F_{nw} = 0.6\ F_{EXX} \left(1.0 + 0.5\ sin(\theta)^{1.5}\right)$ *(AISC Specification J2-4)*

$\phi := 0.75$ *(AISC Specification J2.4)*

$\phi F_{nw} \cdot A_{we} = \phi F_{nw} \cdot t_e \cdot L = \phi F_{nw} \cdot cos(45°) \cdot a \cdot L$ *Fillet weld strength*

For vertical weld

$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin(16°)^{1.5}\right) = 45.04\ ksi$

$L := b = 31.25\ in$

$a_{req} := \frac{P_{pt}}{\phi \cdot F_{nw} \cdot \cos(45°) \cdot L} = 0.27\ in$

For horizontal weld

$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin(90°)^{1.5}\right) = 63\ ksi$

$L := a = 15.63\ in$

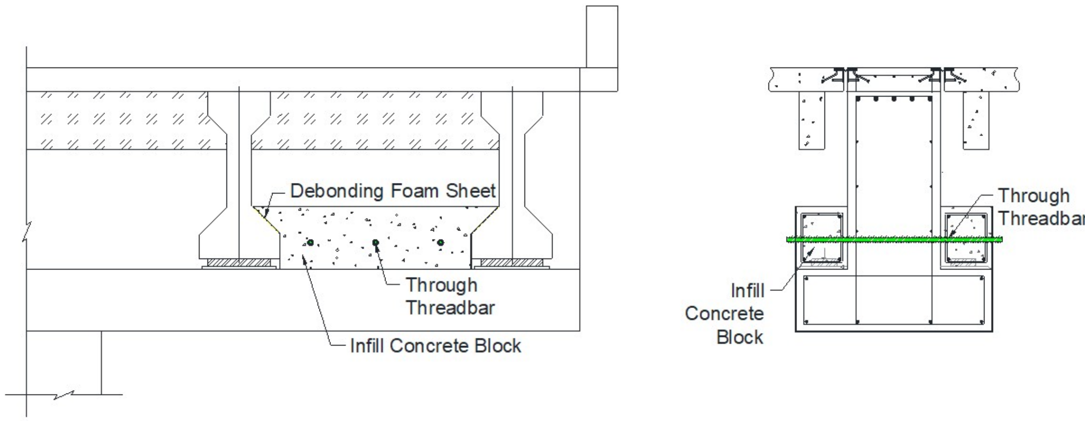
$a_{req} := \frac{P_{pt}}{\phi \cdot F_{nw} \cdot \cos(45°) \cdot L} = 0.38\ in$

Use 0.5 in. fillet weld for both sides

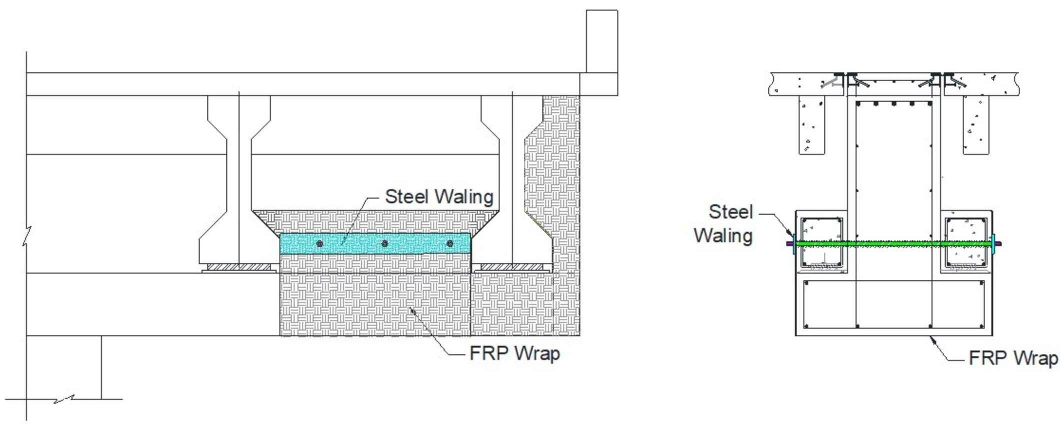
B.4 SOLUTION 16: CONCRETE INFILL WITH PARTIAL-DEPTH FRP ANCHORED BY STEEL WALING (EXTERIOR)

Design Problem:

Bent 13 has hanger and punching shear deficiency for exterior based on AASHTO LRFD (2014). FRP wraps with concrete infill block between the girders will be designed to strengthen the bent cap. Following figures show the steps of installation. The gaps between the girders will be infilled by concrete with through threadbar. FRP wraps will be attached on the surface between the girders. Steel walings will be installed at the termination region of the FRP wraps to provide anchorage.



(a) Infill concrete with through threadbar



(b) Attach FRP wrap with steel waling

The required load demand on the exterior ledge is

$V_{u\_ext} := 247\ kip$ (for single ledge)

FRP Wrap Design

1. Determine the deficiencies for single ledge

$\phi := 0.9$ *Strength reduction factor*

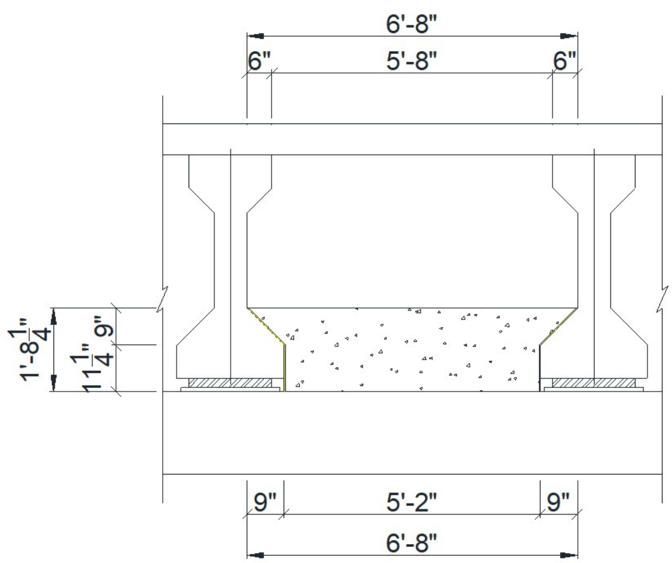
$V_{hd\_ext} := \dfrac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 76.44\ kip$ *Hanger deficiency*

$V_{pd\_ext} := \dfrac{V_{u\_ext}}{\phi} - V_{np\_ext} = 13.26\ kip$ *Punching shear deficiency*

2. Determine the factored self-weight of the infill concrete blocks

Through threadbars will be used to provide location for steel waling to hold FRP wraps. Assume 1 in. diameter threadbar will be used for the solution, then the height of the concrete block will need to be at least 12 in. to enable the threadbars to have sufficient edge distance which is defined as $6\ d$. Considering the constructability, the following geometry for the infill concrete block may be used for the solution.

Dimensions of infill concrete block



Half weight of an infill concrete block will be distributed to exterior girder.

$w_c := 0.015\ \dfrac{kip}{ft^3}$ *Unit self-weight of reinforced concrete*

$Vol_c := 12.8\ ft^3$ *Volume of single infill concrete block*

$W_c := 1.25 \cdot 0.5 \cdot w_c \cdot Vol_c = 0.12\ kip$ *Factored self-weight of infill concrete block*

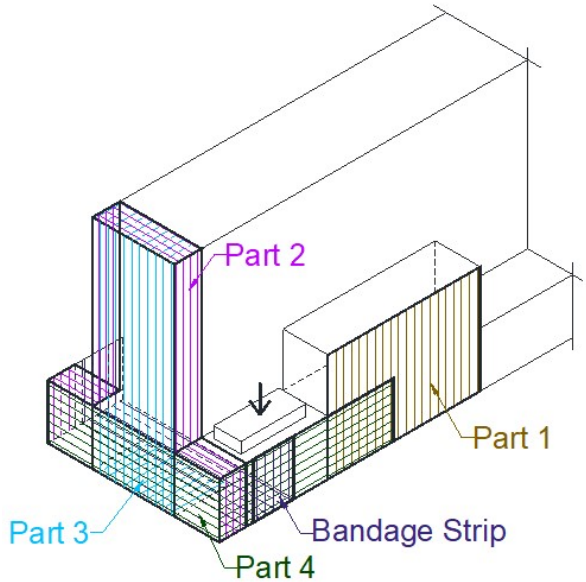
3. Re-calculate the deficiencies for single ledge

$V_{hd\_int} := V_{hd\_ext} + \dfrac{W_c}{\phi} = 76.58\ kip$ *Hanger deficiency*

$V_{pd\_int} := V_{pd\_ext} + \dfrac{W_c}{\phi} = 13.4\ kip$ *Punching shear deficiency*

4. Determine required FRP strength

The following FRP wrapping scheme for the exterior region is considered in this example.



Part 1 - U-wrap attached on the infill concrete block with steel waling. Contribute strength to hanger, ledge and punching shear.

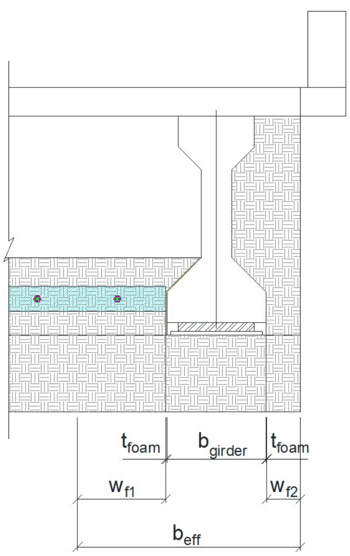
Part 2 - Attached to the end of the bent cap. Enclose entire inverted-T section. Contribute strength to hanger, ledge and punching shear.

Part 3 - Attached on the end surface of the bent cap. Vertically wrapping the web of the bent cap. Contribute strength to hanger. (End region anchorage is recommended. May use bandage strip or mechanical/FRP anchors)

Part 4 - Attached on the end surface of the bent cap. Horizontally wrapping the flange of the bent cap. Assume no strength contribution.

4.1 FRP Part 1 and Part 2

- Determine effective width of FRP wraps within the distribution width of each term of capacities



FRP wraps will be attached from the edge of the girders. Therefore, effective width of FRP wraps can be calculated by subtracting bottom width of the girder and the thickness of debonding foam sheet from the distribution width of each term of capacities.

Assume the thickness of the debonding foam sheet is 0.5 in.

$t_{foam} := 0.5\ in$ *Thickness of debonding foam sheet*

$b_{girder} := 26\ in$ *Bottom width of the girders*

Effective width of FRP can be calculated as

$$w_f := b_{eff} - b_{girder} - 2 \cdot t_{foam}$$

- Distribution width of each term of capacities

$C := 22\ in$ *Distance from center of bearing pad to end of bent cap*

$b_{h\_ext} := \dfrac{W + 2\ d_f}{2} + C = 49.5\ in$ *Distribution width of hanger*

$b_{p\_ext} := \frac{W + 2\ d_f}{2} + C = 49.5\ in$ *Distribution width of punching shear*

- Effective width of FRP wraps

$w_{fh} := b_{h\_ext} - b_{girder} - 2 \cdot t_{foam} = 22.5\ in$ *Effective FRP width within term of hanger*

$w_{fp} := b_{p\_ext} - b_{girder} - 2 \cdot t_{foam} = 22.5\ in$ *Effective FRP width within term of punching shear*

- Effective strain of FRP wraps

ACI 440.2R - 08 recommends a bond-reduction coefficient to calculate effective strain of FRP for the FRP systems that do not enclose the entire section. However, higher effective strain can be used for the system that mechanical anchorages used at termination region but should not exceed 0.004. For this solution, mechanical anchorages will be provided at termination point. Therefore, take FRP effective strain as

$\varepsilon_{fe\_1} := 0.004 \quad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.95$ *Reduction factor for U-wrap*

- Strength contribution of FRP wrap

$V_f := \psi_f \cdot w_f \cdot \varepsilon_{fe} \cdot E_f \cdot t_f$ *$E_f \cdot t_f$ is the tensile modulus per unit width of FRP*

$V_{fh\_1} := \psi_f \cdot w_{fh} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$ *Strength contribution of Part 1 and 2 to hanger capacity*

$V_{fp} := \psi_f \cdot w_{fp} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$ *Strength contribution of Part 1 and 2 to punching shear capacity*

4.2 FRP Part 3

- Effective width of FRP wrap

Use same width as the web of the bent cap

$w_{fh\_end} := 30\ in$

- Effective strain of FRP wraps

Assume end region anchorage (or bandage) will be provided to FRP Part 3

$\varepsilon_{fe\_2} := 0.004 \quad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.95$ *Reduction factor for U-wrap*

- Strength contribution of FRP wrap

$V_{fh\_2} := \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2} \cdot E_f \cdot t_f$

Strength contribution of Part 3 to hanger capacity

5. Determine required tensile modulus per unit width ($E_{unit} := E_f \cdot t_f$) of FRP wraps

- Required FRP tensile modulus (per unit)

$E_{unit} = E_f \cdot t_f$

- Hanger

$V_{fh} := V_{fh\_1} + V_{fh\_2} = \psi_f \cdot w_{fh} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f + \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2} \cdot E_f \cdot t_f$

Set $V_{fh} = V_{hd\_ext}$

$$E_{unit\_h} := \frac{V_{hd\_ext}}{\psi_f \cdot w_{fh} \cdot \varepsilon_{fe\_1} + \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2}} = 383.18 \ \frac{kip}{in}$$

- Punching shear

$V_{fp} := \psi_f \cdot w_{fp} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$

Set $V_{fp} = V_{pd\_ext}$

$$E_{unit\_p} := \frac{V_{pd\_ext}}{\psi_f \cdot w_{fp} \cdot \varepsilon_{fe}} = 155.13 \ \frac{kip}{in}$$

Maximum required E_{unit} is

$$E_{unit\_req} := \max\left(E_{unit\_h}, E_{unit\_p}\right) = 383.18 \ \frac{kip}{in}$$

- Select FRP products from the TxDOT provided pre-qualified FRP product list

<u>May use single layer of BASF C160.</u>

- Specified properties of FRP

$f_{fu} := 150 \ ksi$ — *Ultimate tensile strength*

$E_f := 10700 \ ksi$ — *Tensile modulus*

$t_f := 0.08 \ in$ — *Nominal thickness*

$\varepsilon_{fu} := 0.014$ — *Ultimate rupture strain*

- Check FRP strain limit

$0.75 \cdot \varepsilon_{fu} = 0.011$ > $\varepsilon_{fe} := 0.004$ (O.K.) *(ACI 440.2R-08 Eq.11-6(a))*

Reinforcement of Infill Concrete Block

1.Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5\ in$ *Width of infill concrete block*

$h_c := 20.25\ in$ *Height of infill concrete block*

$d_e := 17.25\ in$ *Bottom width of the girders*

$f'_c := 3.6\ ksi$ *Concrete strength (use same as the in-service structure)*

$f_y := 60\ ksi$ *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = maximum of:

$$\frac{3\sqrt{f'_c}}{f_y} b_w \cdot d_e = 0.85\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_e = 0.94\ in^2$$

$A_{f\_min} := 0.94\ in^2$ *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

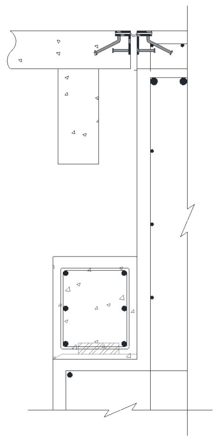
$s_{f\_max} := 12\ in$

$n_{re} := \dfrac{h_c}{s_{f\_max}} + 1 = 2.69$ *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 3$ *Take integer*

$n_f := n_{re} \cdot 2 = 6$ *Required number of longitudinal reinforcement*

$A_{fs\_re} := \dfrac{A_{f\_min}}{n_f} = 0.16\ in^2$ *Required area of single longitudinal bar*



Place #4 ($A_{fs} := 0.2\ in^2$) longitudinal bars at four corners of the concrete block, and place one #4 longitudinal bar at half depth of the concrete block on each side.

2. Determine required shear reinforcement

- Use double leg #5 stirrup

$$A_v := 2 \cdot 0.31\ in^2 = 0.62\ in^2$$

- Concrete shear strength

$$V_c := 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d_e = 34\ kip$$

- Required spacing of stirrups

$$s_{v\_req} := \frac{A_v \cdot f_y \cdot d_e}{V_{hd\_ext} - V_c} = 8.7\ in$$

- Check maximum spacing of stirrups

s_{v_max} = maximum of:

$$\frac{d_e}{2} = 8.63\ in$$

$$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c}\ b_w} = 50\ in$$

$$\frac{A_v \cdot f_y}{50\ b_w} = 45\ in$$

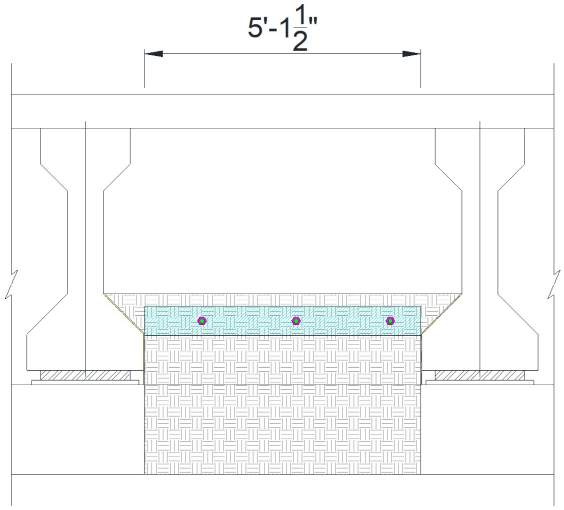
$$s_{v\_max} := 8.63\ in$$

Consider the geometry of the concrete infill block, evenly place four #4 double leg stirrups at a spacing of 8 in.

Threadbar and Waling Design

1. Threadbar Design

$V_{d\_ext} := \max\left(V_{hd\_ext}, V_{pd\_ext}\right) = 76.44\ kip$ *Maximum deficiency*



May need at least three threadbars to provide uniform fixture to a 68 in. wide FRP wrap.

Try to use three threadbars.

- Shear demand for single threadbar

$$V_{u\_single} := \frac{V_{d\_ext}}{3} = 25.48\ kip$$

- Shear capacity of single threadbar

$$V_{n\_single} := 0.6 \cdot f_u \cdot A_n$$

Use B7 Grade threadbar

$f_u := 125\ ksi$ *Tensile strength of B7 Grade threadbar*

$A_{n\_req} := \frac{V_{u\_single}}{0.6 \cdot f_u} = 0.34\ in^2$ *Required nominal area of single threadbar*

Use 1 in. diameter B7 threadbar. ($A_{n\_single} := 0.606\ in^2$)

- Minimum spacing and edge distance

$d_b := 1\ in$ *Diameter of threadbar*

$s_{t\_min} := 6\ d_b = 6\ in$ *Minimum spacing*

$s_{te\_min} := 6\ d_b = 6\ in$ *Minimum edge distance*

Use of 6 in. edge distance from the top of the concrete block. For the constructability, use 12 in. side edge distance.

Evenly space three 1 in. diameter B7 Grade threadbars at 21 in. with the edge distances of 12 in. and 8.5 in. from the side and top, respectively.

2. Waling Design

Use A36 Grade steel

$F_y := 36\ ksi$

- Required thickness for shear bearing

$\phi := 0.75$ *(AISC Specification J3.10)*

$$t_{req} := \frac{V_{u\_single}}{\phi \cdot 2.0 \cdot d_b \cdot F_y} = 0.47\ in$$

- Required bearing area

(AISC Specification J8)

$\phi_c := 0.65$

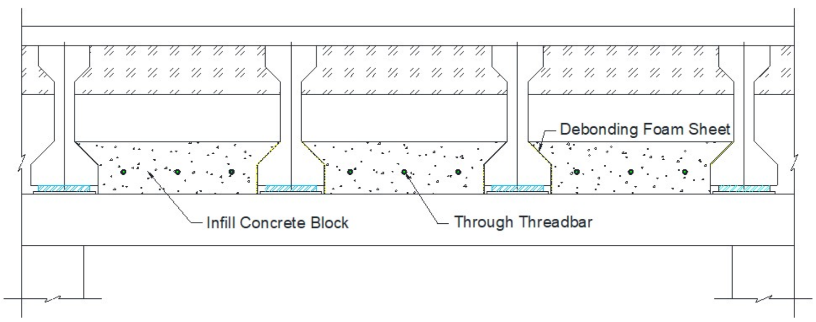
$$A_{b\_req} := \frac{V_{d\_ext}}{\phi_c \cdot 0.85 \cdot f'_c} = 54.8\ in^2$$

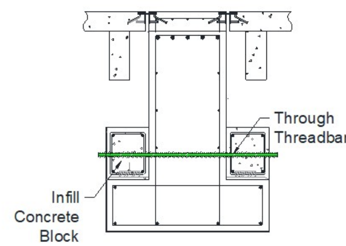
Use 4" x 64" ($A_b := 256\ in^2$) with 0.75 in. thickness continuous steel waling.

B.5 SOLUTION 16: CONCRETE INFILL WITH PARTIAL-DEPTH FRP ANCHORED BY STEEL WALING (INTERIOR)

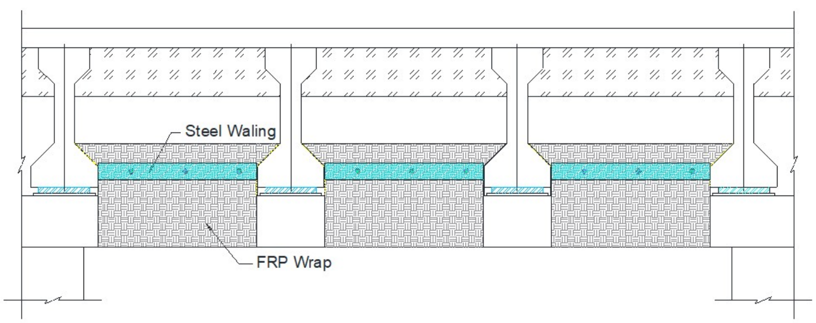
Design Problem:

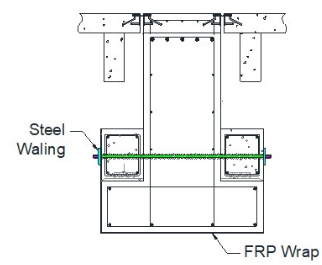
Bent 13 has ledge flexure and hanger deficiency for interior based on AASHTO LRFD (2014). FRP wraps with concrete infill block between the girders will be designed to strengthen the bent cap. Following figures show the steps of installation. The gaps between the girders will be infilled by concrete with through threadbar. FRP wraps will be attached on the surface between the girders. Steel walings will be installed at the termination region of the FRP wraps to provide anchorage.





(a) Infill concrete with through threadbar





(b) Attach FRP wrap with steel waling

The required load demand on the interior ledge is

$V_{u\_int} := 287\ kip$ (for single ledge)

FRP Wrap Design

1. Determine the deficiencies for single ledge

$\phi := 0.9$ *Strength reduction factor*

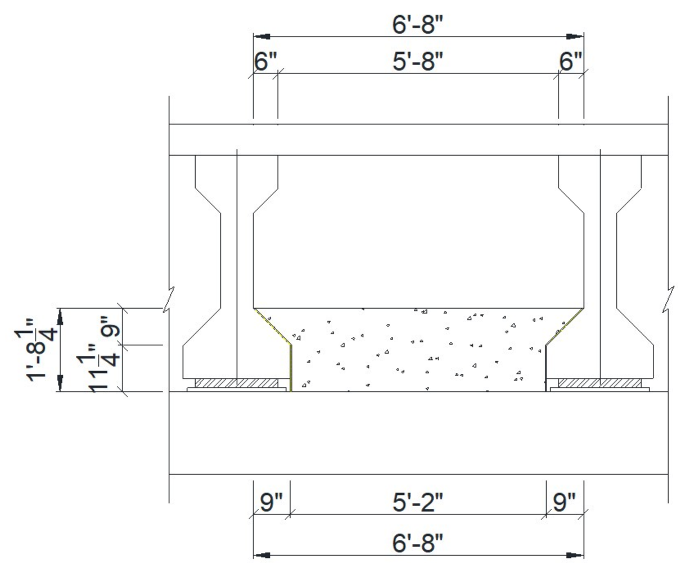
$V_{lfd\_int} := \dfrac{V_{u\_int}}{\phi} - V_{nf\_int} = 19.97\ kip$ *Ledge deficiency*

$V_{hd\_int} := \dfrac{V_{u\_int}}{\phi} - V_{nh\_int} = 89.88\ kip$ *Hanger deficiency*

There are two terms that may control the hanger capacity of Bent 13. One accounts for hanger reinforcement within the shear critical region, which considers a distribution width same as the girder spacing S. The other accounts for combined concrete and hanger reinforcement contribution within flexural and shear critical region. The distribution of the second term is defined by the width of the bearing pad and the effective depth of the ledge, which is smaller than the distribution of the first term. The first term governs the hanger capacity. However, both terms are not sufficient to resist the demand of Bent 13. The current solution is only able to contribute strength to the second term of the hanger capacity, but does not work for the first term of the hanger capacity. Bent 13 will still have hanger deficiency even if the solution strengthened the second term of the hanger. Therefore, this design example will work out the solution for the ledge flexure deficiency.

2. Determine the factored self weight of the infill concrete blocks

Through threadbars will be used to provide location to steel waling to hold FRP wraps. Assume 1 in. diameter threadbar will be used for the solution, then the height of the concrete block will need to be at least 12 in. to enable the threadbars to have sufficient edge distance which is defined as $6\ d$. Considering the constructability, the following geometry for the infill concrete block may be used for the solution.



Dimensions of infill concrete block

$$w_c \coloneqq 0.015\ \frac{kip}{ft^3}$$ *Unit self-weight of reinforced concrete*

$$Vol_c \coloneqq 12.8\ ft^3$$ *Volume of single infill concrete block*

$$W_c \coloneqq 1.25 \cdot w_c \cdot Vol_c = 0.24\ kip$$ *Factored self-weight of infill concrete block*

3. Re-calculate the deficiencies for single ledge

$$V_{lfd\_int} \coloneqq V_{lfd\_int} + \frac{W_c}{\phi} = 20.24\ kip$$ *Ledge deficiency*

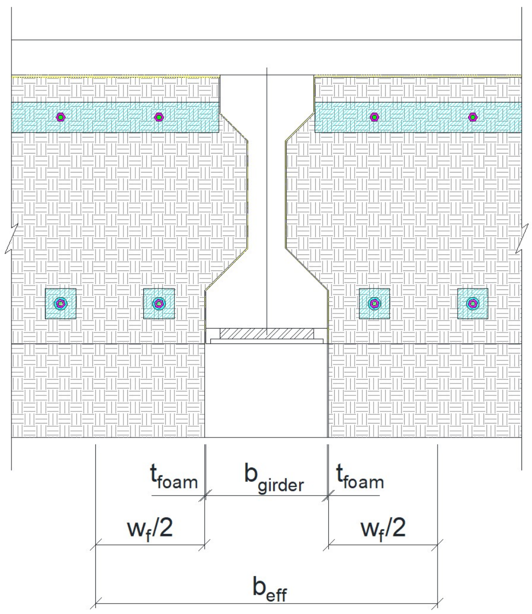
$$V_{hd\_int} \coloneqq V_{hd\_int} + \frac{W_c}{\phi} = 90.15\ kip$$ *Hanger deficiency*

4. Determine the effective width of FRP wraps within the distribution width of each term of capacities

- Distribution width of each term of capacities

$b_{lf\_int} := min\left(W + 5\ a_f, S\right) = 71\ in$ *Distribution width of ledge flexure*

$b_{p\_int} := W + 2\ d_f = 55\ in$ *Distribution width of punching shear*



FRP wraps will be attached from the edge of the girders. Therefore, effective width of FRP wraps can be calculated by subtracting bottom width of the girder and the thickness of debonding foam sheet from the distribution width of each term of capacities.

Assume the thickness of the debonding foam sheet is 0.5 in.

$t_{foam} := 0.5\ in$ *Thickness of debonding foam sheet*

$b_{girder} := 26\ in$ *Bottom width of the girders*

Effective width of FRP can be calculated as

$w_f := b_{eff} - b_{girder} - 2 \cdot t_{foam}$

- Effective width of FRP wraps

$w_{lf} := b_{lf\_int} - b_{girder} - 2 \cdot t_{foam} = 44\ in$ *Effective FRP width within term of ledge flexure*

5. Determine required tensile modulus per unit width ($E_{unit} := E_f \cdot t_f$) of the FRP wraps

- Effective strain of FRP wraps

ACI 440.2R - 08 recommends a bond-reduction coefficient to calculate effective strain of FRP for the FRP systems that do not enclose the entire section. However, higher effective strain can be used for the system that mechanical anchorages used at termination region but should not exceed 0.004. For this solution, mechanical anchorages will be provided at termination point. Therefore, take FRP effective strain as

$\varepsilon_{fe} := 0.004 \qquad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.95$ *Reduction factor for U-wrap*

- Strength contribution of FRP reinforcement

$$\phi V_f := \psi_f \cdot w_f \cdot \varepsilon_{fe} \cdot E_f \cdot t_f$$

- Required FRP tensile modulus (per unit length)

$$E_{unit} = E_f \cdot t_f = \frac{V_f}{\psi_f \cdot w_f \cdot \varepsilon_{fe}}$$

- Determine required FRP tensile modulus (per unit) for each term of capacities

$$E_{unit\_lf} := \frac{V_{lfd\_int}}{\psi_f \cdot w_{lf} \cdot \varepsilon_{fe}} = 121.03 \ \frac{kip}{in}$$

$$E_{unit\_req} := E_{unit\_lf} = 121.03 \ \frac{kip}{in}$$

- Select FRP products from the TxDOT provided pre-qualified FRP product list

<u>May use single layer of BASF C160.</u>

- Specified properties of FRP

$f_{fu} := 150 \ ksi$ *Ultimate tensile strength*

$E_f := 10700 \ ksi$ *Tensile modulus*

$t_f := 0.08 \ in$ *Nominal thickness*

$\varepsilon_{fu} := 0.014$ *Ultimate rupture strain*

- Check FRP strain limit

$0.75 \cdot \varepsilon_{fu} = 0.011 \quad > \quad \varepsilon_{fe} := 0.004$ (O.K.) *(ACI 440.2R-08 Eq.11-6(a))*

<u>Reinforcement of Infill Concrete Block</u>

1. Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5 \ in$ *Width of infill concrete block*

$h_c := 20.25 \ in$ *Height of infill concrete block*

$d_e := 17.25 \ in$ *Bottom width of the girders*

$f'_c := 3.6 \ ksi$ *Concrete strength (use same as the in-service structure)*

$f_y := 60 \ ksi$ *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = maximum of:

$$\frac{3\sqrt{f'_c}}{f_y} b_w \cdot d_e = 0.85\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_e = 0.94\ in^2$$

$A_{f\_min} := 0.94\ in^2$ *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

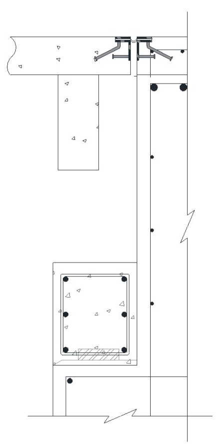
$s_{f\_max} := 12\ in$ *(ACI 318-14)*

$n_{re} := \frac{h_c}{s_{f\_max}} + 1 = 2.69$ *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 3$ *Take integer*

$n_f := n_{re} \cdot 2 = 6$ *Required number of longitudinal reinforcement*

$A_{fs\_re} := \frac{A_{f\_min}}{n_f} = 0.16\ in^2$ *Required area of single longitudinal bar*



Place #4 ($A_{fs} := 0.2\ in^2$) longitudinal bars at four corners of the concrete block, and place one #4 longitudinal bar at half depth of the concrete block on each side.

2. Determine required shear reinforcement

- Use double leg #5 stirrup

$A_v := 2 \cdot 0.31\ in^2 = 0.62\ in^2$

- Concrete shear strength

$V_c := 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d_e = 34\ kip$

- Required spacing of stirrups

$s_{v\_req} := \dfrac{A_v \cdot f_y \cdot d_e}{V_{lfd\_int} - V_c} = 8.7\ in$

- Check maximum spacing of stirrups

s_{v_max} = maximum of:

$\dfrac{d_e}{2} = 8.63\ in$

$\dfrac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c}\ b_w} = 50\ in$

$\dfrac{A_v \cdot f_y}{50\ b_w} = 45\ in$

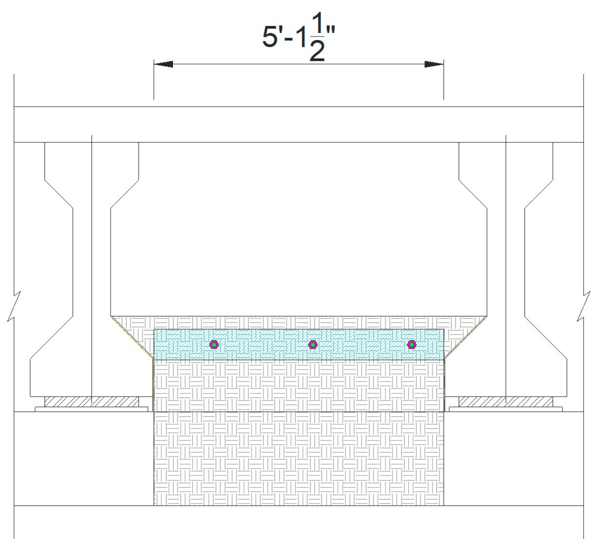
$s_{v\_max} := 8.63\ in$

<u>Consider the geometry of the concrete infill block, evenly place four #4 double leg stirrups at a spacing of 8 in.</u>

<u>Threadbar and Waling Design</u>

1. Threadbar Design

$V_{d\_int} := V_{lfd\_int} = 20.24\ kip$ *Maximum deficiency*



May need at least three threadbars to provide uniform fixture to a 62 in. wide FRP wrap.

Try to use three threadbars.

- Shear demand for single threadbar

$V_{u\_single} := \dfrac{V_{d\_int}}{3} = 6.75\ kip$

- Shear capacity of single threadbar

$V_{n\_single} := 0.6 \cdot f_u \cdot A_n$

Use B7 Grade threadbar

$f_u := 125\ ksi$ *Tensile strength of B7 Grade threadbar*

$A_{n\_req} := \dfrac{V_{u\_single}}{0.6 \cdot f_u} = 0.09\ in^2$ *Required nominal area of single threadbar*

<u>Use 1/2 in. diameter B7 grade threadbar</u> ($A_{n\_single} := 0.142\ in^2$).

- Minimum spacing and edge distance

$d_b := 0.5\ in$ *Diameter of threadbar*

$s_{t\_min} := 6\ d_b = 3\ in$ *Minimum spacing*

$s_{te\_min} := 6\ d_b = 3\ in$ *Minimum edge distance*

Use of 6 in. edge distance from the top of the concrete block. For the constructability, use 12 in. side edge distance.

<u>Evenly space three 1/2 in. diameter B7 Grade threadbars at 21 in. with the edge distances of 12 in. and 6 in. from the side and top, respectively.</u>

2. Waling Design

Use A36 Grade steel

$F_y := 36\ ksi$

- Required thickness for shear bearing

$\phi := 0.75$ *(AISC Specification J3.10)*

$t_{req} := \dfrac{V_{u\_single}}{\phi \cdot 2.0 \cdot d_b \cdot F_y} = 0.25\ in$

- Required bearing area

$\phi_c := 0.65$ *(AISC Specification J8)*

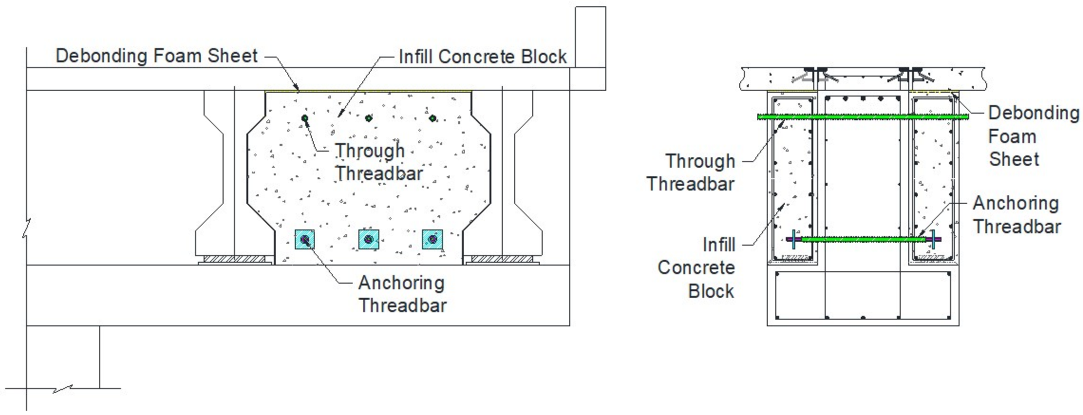
$A_{b\_req} := \dfrac{V_{d\_int}}{\phi_c \cdot 0.85 \cdot f'_c} = 54.8\ in^2$

<u>Use 4" x 61.5" ($A_b := 246\ in^2$) with 0.25 in. thickness continuous steel waling.</u>

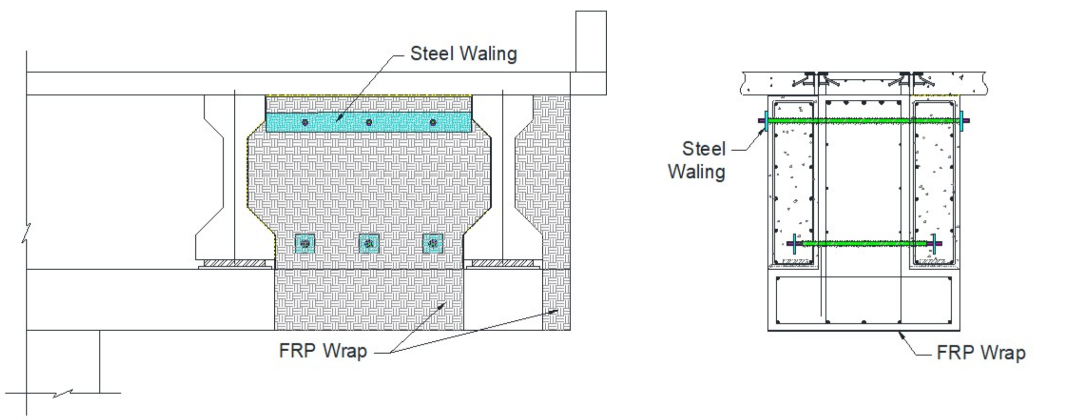
B.6 SOLUTION 17: CONCRETE INFILL WITH FULL-DEPTH FRP ANCHORED BY STEEL WALING (EXTERIOR)

Design Problem:

Bent 13 has hanger and punching shear deficiency for exterior based on AASHTO LRFD (2014). FRP wraps with concrete infill block between the girders will be designed to strengthen the bent cap. Following figures show the steps of installation. The gaps between the girders will be infilled by concrete with through threadbar. FRP wraps will be attached on the surface between the girders. Steel walings will be installed at the termination region of the FRP wraps to provide anchorage.



(a) Infill concrete with through threadbar



(b) Attach FRP wrap with steel waling

The required load demand on the exterior ledge is

$V_{u\_ext} := 247\ kip$ (for single ledge)

FRP Wrap Design

1. Determine the deficiencies for single ledge

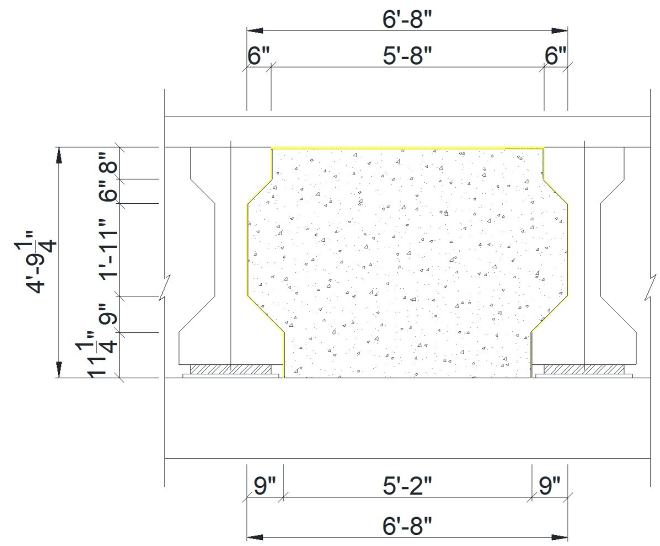
$\phi := 0.9$ *Strength reduction factor*

$$V_{hd\_ext} := \frac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 76.44\ kip$$ *Hanger deficiency*

$V_{pd\_ext} := \frac{V_{u\_ext}}{\phi} - V_{np\_ext} = 13.3\ kip$ *Punching shear deficiency*

2. Determine the factored self-weight of the infill concrete blocks

Dimensions of infill concrete block



Half weight of an infill concrete block will be distributed to exterior girder.

$w_c := 0.015\ \frac{kip}{ft^3}$ *Unit self-weight of reinforced concrete*

$Vol_c := 39.8\ ft^3$ *Volume of single infill concrete block*

$W_c := 1.25 \cdot 0.5 \cdot w_c \cdot Vol_c = 0.37\ kip$ *Factored self-weight of infill concrete block*

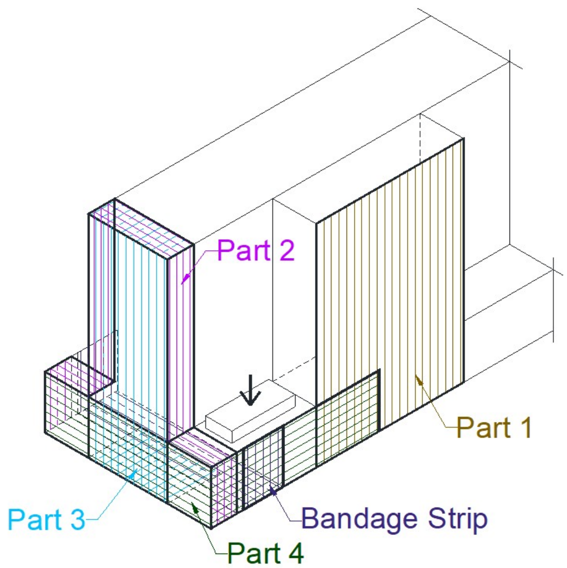
3. Re-calculate the deficiencies for single ledge

$V_{hd\_int} := V_{hd\_ext} + \frac{W_c}{\phi} = 76.86\ kip$ *Hanger deficiency*

$V_{pd\_int} := V_{pd\_ext} + \frac{W_c}{\phi} = 13.68\ kip$ *Punching shear deficiency*

4. Determine required FRP strength

The following FRP wrapping scheme for the exterior region is considered in this example.



Part 1 - U-wrap attached on the infill concrete block with steel waling. Contribute strength to hanger, ledge and punching shear.

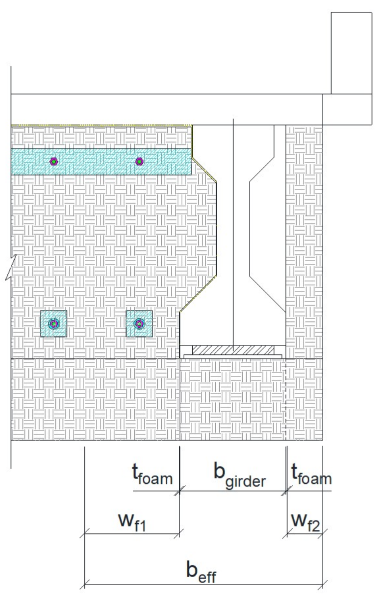
Part 2 - Attached to the end of the bent cap. Enclose entire inverted-T section. Contribute strength to hanger, ledge and punching shear.

Part 3 - Attached on the end surface of the bent cap. Vertically wrapping the web of the bent cap. Contribute strength to hanger. (End region anchorage is recommended. May use bandage strip or mechanical/FRP anchors)

Part 4 - Attached on the end surface of the bent cap. Horizontally wrapping the flange of the bent cap. Assume no strength contribution.

4.1 FRP Part 1 and Part 2

- Determine effective width of FRP wraps within the distribution width of each term of capacities



FRP wraps will be attached from the edge of the girders. Therefore, effective width of FRP wraps can be calculated by subtracting bottom width of the girder and the thickness of debonding foam sheet from the distribution width of each term of capacities.

Assume the thickness of the debonding foam sheet is 0.5 in.

$t_{foam} := 0.5\ in$ *Thickness of debonding foam sheet*

$b_{girder} := 26\ in$ *Bottom width of the girders*

Effective width of FRP can be calculated as

$$w_f := b_{eff} - b_{girder} - 2 \cdot t_{foam}$$

- Distribution width of each term of capacities

$C := 22\ in$ *Distance from center of bearing pad to end of bent cap*

$$b_{h\_ext} := \frac{W + 2\ d_f}{2} + C = 49.5\ in$$ *Distribution width of hanger*

$b_{p\_ext} := \frac{W + 2\ d_f}{2} + C = 49.5\ in$ *Distribution width of punching shear*

- Effective width of FRP wraps

$w_{fh} := b_{h\_ext} - b_{girder} - 2 \cdot t_{foam} = 22.5\ in$ *Effective FRP width within term of hanger*

$w_{fp} := b_{p\_ext} - b_{girder} - 2 \cdot t_{foam} = 22.5\ in$ *Effective FRP width within term of punching shear*

- Effective strain of FRP wraps

ACI 440.2R - 08 recommends a bond-reduction coefficient to calculate effective strain of FRP for the FRP systems that do not enclose the entire section. However, higher effective strain can be used for the system that mechanical anchorages used at termination region but should not exceed 0.004. For this solution, mechanical anchorages will be provided at termination point. Therefore, take FRP effective strain as

$\varepsilon_{fe\_1} := 0.004 \quad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.85$ *Reduction factor for U-wrap*

- Strength contribution of FRP wrap

$V_f := \psi_f \cdot w_f \cdot \varepsilon_{fe} \cdot E_f \cdot t_f$ *$E_f \cdot t_f$ is the tensile modulus per unit width of FRP*

$V_{fh\_1} := \psi_f \cdot w_{fh} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$ *Strength contribution of Part 1 and 2 to hanger capacity*

$V_{fp} := \psi_f \cdot w_{fp} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$ *Strength contribution of Part 1 and 2 to punching shear capacity*

4.2 FRP Part 3

- Effective width of FRP wrap

Use same width as the web of the bent cap

$w_{fh\_end} := 30\ in$

- Effective strain of FRP wraps

Assume end region anchorage (or bandage) will be provided to FRP Part 3

$\varepsilon_{fe\_2} := 0.004 \quad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.95$ *Reduction factor for U-wrap*

- Strength contribution of FRP wrap

$V_{fh\_2} := \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2} \cdot E_f \cdot t_f$ *Strength contribution of Part 3 to hanger capacity*

5. Determine required tensile modulus per unit width ($E_{unit} := E_f \cdot t_f$) of FRP wraps

- Required FRP tensile modulus (per unit)

$E_{unit} = E_f \cdot t_f$

- Hanger

$V_{fh} := V_{fh\_1} + V_{fh\_2} = \psi_f \cdot w_{fh} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f + \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2} \cdot E_f \cdot t_f$

Set $V_{fh} = V_{hd\_ext}$

$$E_{unit\_h} := \frac{V_{hd\_ext}}{\psi_f \cdot w_{fh} \cdot \varepsilon_{fe\_1} + \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2}} = 383.18\ \frac{kip}{in}$$

- Punching shear

$V_{fp} := \psi_f \cdot w_{fp} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$

Set $V_{fp} = V_{pd\_ext}$

$$E_{unit\_p} := \frac{V_{pd\_ext}}{\psi_f \cdot w_{fp} \cdot \varepsilon_{fe}} = 155.13\ \frac{kip}{in}$$

Maximum required E_{unit} is

$$E_{unit\_req} := \max\left(E_{unit\_h}, E_{unit\_p}\right) = 383.18\ \frac{kip}{in}$$

- Select FRP products from the TxDOT provided pre-qualified FRP product list

<u>May use single layer of BASF C160.</u>

- Specified properties of FRP

$f_{fu} := 150\ ksi$ *Ultimate tensile strength*

$E_f := 10700\ ksi$ *Tensile modulus*

$t_f := 0.08\ in$ *Nominal thickness*

$\varepsilon_{fu} := 0.014$ *Ultimate rupture strain*

- Check FRP strain limit

$0.75 \cdot \varepsilon_{fu} = 0.011 \quad > \quad \varepsilon_{fe} := 0.004$ (O.K.) *(ACI 440.2R-08 Eq.11-6(a))*

Reinforcement of Infill Concrete Block

1.Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5\ in$ *Width of infill concrete block*

$h_c := 56.75\ in$ *Height of infill concrete block*

$d_e := 54.25\ in$ *Bottom width of the girders*

$f'_c := 3.6\ ksi$ *Concrete strength (use same as the in-service structure)*

$f_y := 60\ ksi$ *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = maximum of:

$$\frac{3\sqrt{f'_c}}{f_y} b_w \cdot d_e = 2.69\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_e = 2.98\ in^2$$

$A_{f\_min} := 2.98\ in^2$ *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

$s_{f\_max} := 12\ in$

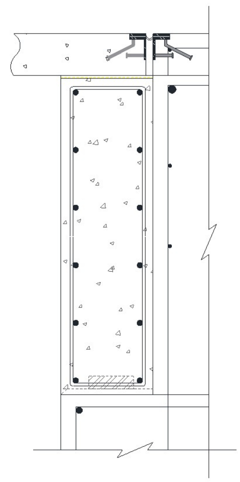
$n_{re} := \dfrac{h_c}{s_{f\_max}} + 1 = 5.73$ *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 6$ *Take integer*

$n_f := n_{re} \cdot 2 = 12$ *Required number of longitudinal reinforcement*

$A_{fs\_re} := \frac{A_{f\_min}}{n_f} = 0.25\ in^2$

Required area of single longitudinal bar



Place #5 ($A_{fs} := 0.3\ in^2$) longitudinal bar at four corners of the concrete block, and evenly place four #5 longitudinal bars along the height of the concrete block on each side.

2. Determine required shear reinforcement

- Use double leg #4 stirrup

$A_v := 2 \cdot 0.2\ in^2 = 0.4\ in^2$

- Concrete shear strength

$V_c := 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d_e = 107\ kip$

- Required spacing of stirrups

$s_{v\_req} := \frac{A_v \cdot f_y \cdot d_e}{V_{hd\_ext} - V_c} = 651\ in$

- Check maximum spacing of stirrups

s_{v_max} = maximum of:

$\frac{d_e}{2} = 27.13\ in$

$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c}\ b_w} = 32\ in$

$\frac{A_v \cdot f_y}{50\ b_w} = 29\ in$

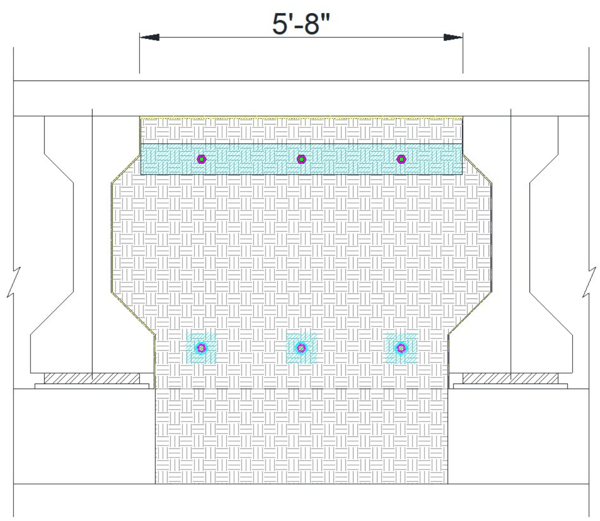
$s_{v\_max} := 27.13\ in$

Consider the geometry of the concrete infill block, evenly place four #4 double leg stirrups at a spacing of 18 in.

Threadbar and Waling Design

1. Threadbar Design

$V_{d\_ext} := \max\left(V_{hd\_ext}, V_{pd\_ext}\right) = 76.44\ kip$ *Maximum deficiency*



May need at least three threadbars to provide uniform fixture to a 68 in. wide FRP wrap.

Try to use three threadbars.

- Shear demand for single threadbar

$$V_{u\_single} := \frac{V_{d\_ext}}{3} = 25.48\ kip$$

- Shear capacity of single threadbar

$$V_{n\_single} := 0.6 \cdot f_u \cdot A_n$$

Use B7 Grade threadbar

$f_u := 125\ ksi$ *Tensile strength of B7 Grade threadbar*

$$A_{n\_req} := \frac{V_{u\_single}}{0.6 \cdot f_u} = 0.34\ in^2$$ *Required nominal area of single threadbar*

Use 1 in. diameter B7 threadbar. ($A_{n\_single} := 0.606\ in^2$)

- Minimum spacing and edge distance

$d_b := 1\ in$ *Diameter of threadbar*

$s_{t\_min} := 6\ d_b = 6\ in$ *Minimum spacing*

$s_{te\_min} := 6\ d_b = 6\ in$ *Minimum edge distance*

Use of 6 in. edge distance from the top of the concrete block will provide the most effective end anchorage to FRP wraps. However, to avoid the flexure reinforcement provided at the top of the stem, the through threadbars need to be at least 8 in. away from the top of the concrete block.

For the constructability, use 12 in. side edge distance.

Evenly space three 1 in. diameter B7 Grade threadbars at 21 in. with the edge distances of 12 in. and 8.5 in. from the side and top, respectively.

2. Waling Design

Use A36 Grade steel

$F_y := 36\ ksi$

- Required thickness for shear bearing

$\phi := 0.75$ *(AISC Specification J3.10)*

$$t_{req} := \frac{V_{u\_single}}{\phi \cdot 2.0 \cdot d_b \cdot F_y} = 0.47\ in$$

- Required bearing area

(AISC Specification J8)

$\phi_c := 0.65$

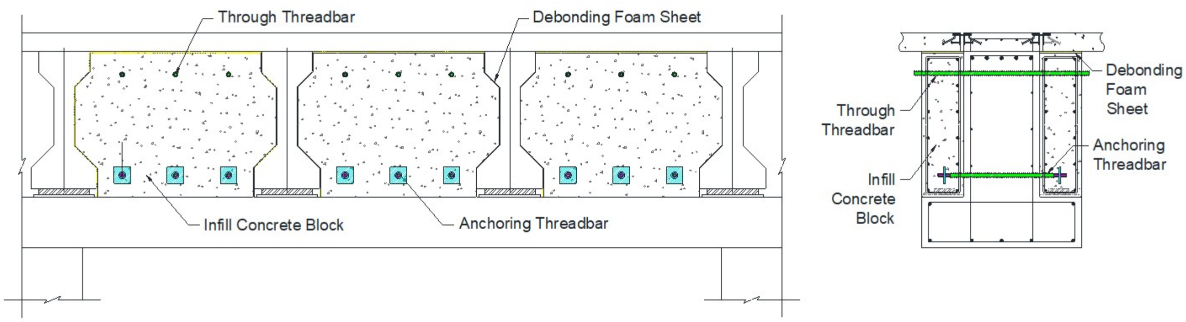
$$A_{b\_req} := \frac{V_{d\_ext}}{\phi_c \cdot 0.85 \cdot f'_c} = 54.8\ in^2$$

<u>Use 4" x 68" ($A_b := 272\ in^2$) with 0.75 in. thickness continuous steel waling.</u>

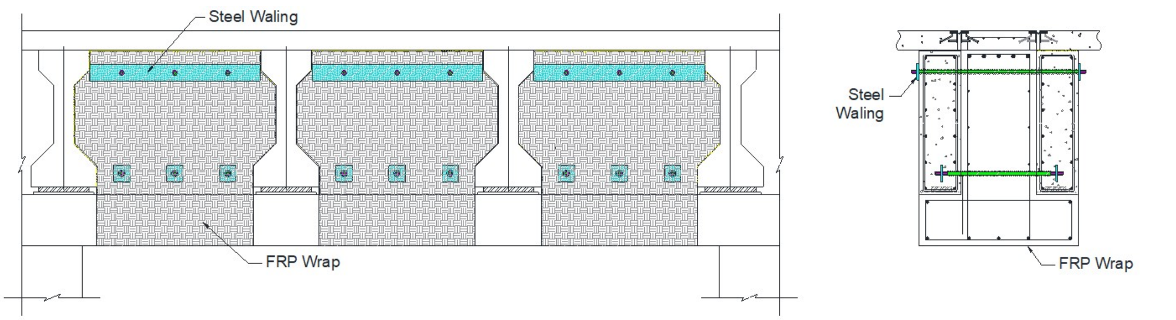
B.7 SOLUTION 17: CONCRETE INFILL WITH FULL-DEPTH FRP ANCHORED BY STEEL WALING (INTERIOR)

Design Problem:

Bent 13 has ledge flexure and hanger deficiency for interior based on AASHTO LRFD (2014). FRP wraps with concrete infill block between the girders will be designed to strengthen the bent cap. Following figures show the steps of installation. The gaps between the girders will be infilled by concrete with through threadbar. FRP wraps will be attached on the surface between the girders. Steel walings will be installed at the termination region of the FRP wraps to provide anchorage.



(a) Infill concrete with through threadbar



(b) Attach FRP wrap with steel waling

The required load demand on the interior ledge is

$V_{u\_int} := 287\ kip$ (for single ledge)

FRP Wrap Design

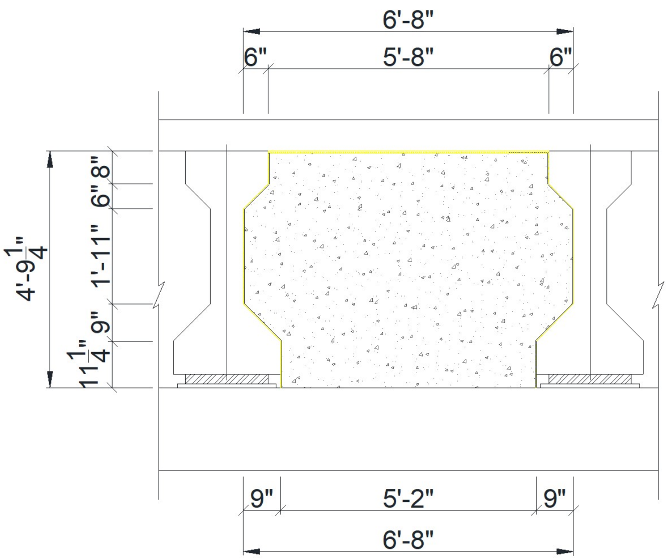
1. Determine the deficiencies for single ledge

$\phi := 0.9$ *Strength reduction factor*

$$V_{lfd\_int} := \frac{V_{u\_int}}{\phi} - V_{nf\_int} = 19.97\ kip$$ *Ledge deficiency*

$$V_{hd\_int} := \frac{V_{u\_int}}{\phi} - V_{nh\_int} = 89.88\ kip$$ *Hanger deficiency*

2. Determine the factored self-weight of the infill concrete blocks



Dimensions of infill concrete block

$w_c := 0.015 \dfrac{kip}{ft^3}$ *Unit self-weight of reinforced concrete*

$Vol_c := 39.8\ ft^3$ *Volume of single infill concrete block*

$W_c := 1.25 \cdot w_c \cdot Vol_c = 0.75\ kip$ *Factored self-weight of infill concrete block*

3. Re-calculate the deficiencies for single ledge

$V_{lfd\_int} := V_{lfd\_int} + \dfrac{W_c}{\phi} = 20.8\ kip$ *Ledge deficiency*

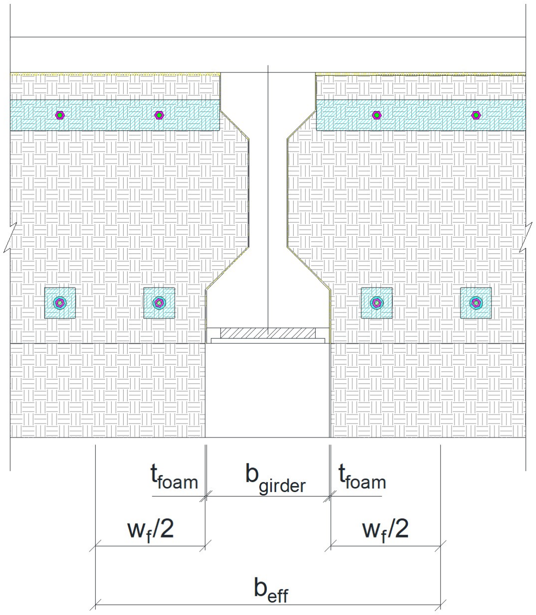
$V_{hd\_int} := V_{hd\_int} + \dfrac{W_c}{\phi} = 90.71\ kip$ *Hanger deficiency*

4. Determine the effective width of FRP wraps within the distribution width of each term of capacities

- Distribution width of each term of capacities

$b_{lf\_int} := min\left(W + 5\ a_f, S\right) = 71\ in$ *Distribution width of ledge flexure*

$b_{h\_int} := W + 2\ d_f = 55\ in$ *Distribution width of hanger*



FRP wraps will be attached from the edge of the girders. Therefore, effective width of FRP wraps can be calculated by subtracting bottom width of the girder and the thickness of debonding foam sheet from the distribution width of each term of capacities.

Assume the thickness of the debonding foam sheet is 0.5 in.

$t_{foam} := 0.5\ in$ *Thickness of debonding foam sheet*

$b_{girder} := 26\ in$ *Bottom width of the girders*

Effective width of FRP can be calculated as

$w_f := b_{eff} - b_{girder} - 2 \cdot t_{foam}$

- Effective width of FRP wraps

$w_{lf} := b_{lf\_int} - b_{girder} - 2 \cdot t_{foam} = 44\ in$ *Effective FRP width within term of ledge flexure*

$w_{hf} := b_{h\_int} - b_{girder} - 2 \cdot t_{foam} = 28\ in$ *Effective FRP width within term of hanger*

5. Determine required tensile modulus per unit width ($E_{unit} := E_f \cdot t_f$) of the FRP wraps

- Effective strain of FRP wraps

ACI 440.2R - 08 recommends a bond-reduction coefficient to calculate effective strain of FRP for the FRP systems that do not enclose the entire section. However, higher effective strain can be used for the system that mechanical anchorages used at termination region but should not exceed 0.004. For this solution, mechanical anchorages will be provided at termination point. Therefore, take FRP effective strain as

$\varepsilon_{fe} := 0.004$ $(<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.95$ *Reduction factor for U-wrap*

- Strength contribution of FRP reinforcement

$\phi V_f := \psi_f \cdot w_f \cdot \varepsilon_{fe} \cdot E_f \cdot t_f$

- Required FRP tensile modulus (per unit length)

$$E_{unit} = E_f \cdot t_f = \frac{V_f}{\psi_f \cdot w_f \cdot \varepsilon_{fe}}$$

- Determine required FRP tensile modulus (per unit) for each term of capacities

$$E_{unit\_lf} := \frac{V_{lfd\_int}}{\psi_f \cdot w_{lf} \cdot \varepsilon_{fe}} = 124.39\ \frac{kip}{in}$$

$$E_{unit\_h} := \frac{V_{hd\_int}}{\psi_f \cdot w_{hf} \cdot \varepsilon_{fe}} = 852.51\ \frac{kip}{in}$$

$$E_{unit\_req} := \max\left(E_{unit\_lf}, E_{unit\_h}\right) = 852.51\ \frac{kip}{in}$$

- Select FRP products from the TxDOT provided pre-qualified FRP product list

May use two layers of BASF C160.

- Properties of FRP Strip

$f_{fu} := 580\ ksi$ *Ultimate tensile strength*

$E_f := 33400\ ksi$ *Tensile modulus*

$t_f := 0.04\ in$ *Nominal thickness*

$\varepsilon_{fu} := 0.017$ *Ultimate rupture strain*

- Check FRP strain limit

$0.75 \cdot \varepsilon_{fu} = 0.01 \quad > \quad \varepsilon_{fe} := 0.004$ (O.K.) *(ACI 440.2R-08 Eq.11-6(a))*

Reinforcement of Infill Concrete Block

1. Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5\ in$ *Width of infill concrete block*

$h_c := 56.75\ in$ *Height of infill concrete block*

$d_e := 54.25\ in$ *Bottom width of the girders*

$f'_c := 3.6\ ksi$ *Concrete strength (use same as the in-service structure)*

$f_y := 60\ ksi$ *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = maximum of:

$$\frac{3\sqrt{f'_c}}{f_y} b_w \cdot d_e = 2.69\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_e = 2.98\ in^2$$

$A_{f\_min} := 2.98\ in^2$ *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

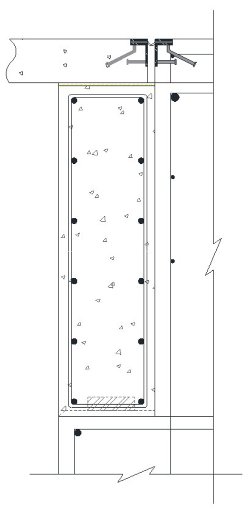
$s_{f\_max} := 12\ in$ *(ACI 318-14)*

$n_{re} := \frac{h_c}{s_{f\_max}} + 1 = 5.73$ *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 6$ *Take integer*

$n_f := n_{re} \cdot 2 = 12$ *Required number of longitudinal reinforcement*

$A_{fs\_re} := \frac{A_{f\_min}}{n_f} = 0.25\ in^2$ *Required area of single longitudinal bar*



Place #5 ($A_{fs} := 0.3\ in^2$) longitudinal bars at four corners of the concrete block, and evenly place four #5 longitudinal bars along the height of the concrete block on each side.

2.Determine required shear reinforcement

- Use double leg #4 stirrup

$A_v := 2 \cdot 0.2\ in^2 = 0.4\ in^2$

- Concrete shear strength

$V_c := 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d_e = 107\ kip$

- Required spacing of stirrups

$$s_{v\_req} := \frac{A_v \cdot f_y \cdot d_e}{V_{lfd\_int} - V_c} = 651\ in$$

- Check maximum spacing of stirrups

s_{v_max} = maximum of:

$$\frac{d_e}{2} = 27.13\ in$$

$$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c}\ b_w} = 32\ in$$

$$\frac{A_v \cdot f_y}{50\ b_w} = 29\ in$$

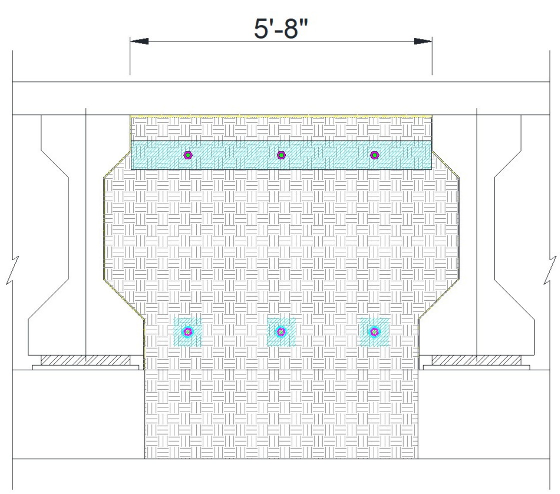
$s_{v\_max} := 27.13\ in$

Consider the geometry of the concrete infill block, evenly place four #4 double leg stirrups at a spacing of 18 in.

Threadbar and Waling Design

1. Threadbar Design

$V_{d\_int} := \max\left(V_{lfd\_int}, V_{hd\_int}\right) = 90.71\ kip$ *Maximum deficiency*



May need at least three threadbars to provide uniform fixture to a 68 in. wide FRP wrap.

Try to use three threadbars.

- Shear demand for single threadbar

$$V_{u\_single} := \frac{V_{d\_int}}{3} = 30.24\ kip$$

- Shear capacity of single threadbar

$V_{n\_single} := 0.6 \cdot f_u \cdot A_n$

Use B7 Grade threadbar

$f_u := 125\ ksi$ *Tensile strength of B7 Grade threadbar*

$A_{n\_req} := \dfrac{V_{u\_single}}{0.6 \cdot f_u} = 0.4\ in^2$ *Required nominal area of single threadbar*

Use 1 in. diameter B7 grade threadbar ($A_{n\_single} := 0.606\ in^2$).

- Minimum spacing and edge distance

$d_b := 1\ in$ *Diameter of threadbar*

$s_{t\_min} := 6\ d_b = 6\ in$ *Minimum spacing*

$s_{te\_min} := 6\ d_b = 6\ in$ *Minimum edge distance*

Use of 6 in. edge distance from the top of the concrete block will provide the most effective end anchorage to FRP wraps. However, to avoid the flexure reinforcement provided at the top of the stem, the through threadbars need to be at least 8 in. away from the top of the concrete block.

For the constructability, use 12 in. side edge distance.

Evenly space three 1 in. diameter B7 Grade threadbars at 21 in. with the edge distances of 12 in. and 8.5 in. from the side and top, respectively.

2. Waling Design

Use A36 Grade steel

$F_y := 36\ ksi$

- Required thickness for shear bearing

$\phi := 0.75$ *(AISC Specification J3.10)*

$t_{req} := \dfrac{V_{u\_single}}{\phi \cdot 2.0 \cdot d_b \cdot F_y} = 0.56\ in$

- Required bearing area

$\phi_c := 0.65$ *(AISC Specification J8)*

$A_{b\_req} := \dfrac{V_{d\_int}}{\phi_c \cdot 0.85 \cdot f'_c} = 54.8\ in^2$

Use 4" x 68" ($A_b := 272\ in^2$) with 0.75 in. thickness continuous steel waling.

B.8 SOLUTION 18: LARGE BEARING PAD

Design Problem:

Bent 13 has punching shear deficiency for exterior girders. The proposed solution, use of increased size bearing pad, is mainly expected to increase the punching shear capacity of the bent cap by increasing the load distribution area. The solution may also increase the ledge flexure and hanger capacity of the bent cap. But the strength increase will not be as effective as the punching shear. The solution will be designed for the punching shear deficiency.

Punching Shear Capacity Calculation

The proposed equations for punching shear capacity calculation are shown below.

Interior

$$V_{np\_int} := 0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \underline{(W + 2 \cdot L + 2 \cdot d_f \cdot \cot(35\,°))} \cdot d_f$$

Exterior

V_{np_ext} = minimum of:

$$0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \underline{(0.5 \cdot W + L + d_f \cdot cot(35\,°) + C)} \cdot d_f$$

$$0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \underline{(W + 2 \cdot L + 2 \cdot d_f \cdot cot(35\,°))} \cdot d_f$$

The underlined terms in the above equations are the effective perimeter of the concrete failure surface. Increase of bearing pad dimension (i.e. W and L) will increase the effective perimeter of the failure surface and hence, increase the punching shear capacity. In this design example, the terms of effective perimeter will be defined as p, and the increment of the effective perimeter will be defined as Δp.

Required Increment of Bearing Pad Dimension

1. Load demands on exterior single ledge

$V_{u\_ext} := 247\ kip$ (for single ledge)

2. Determine the deficiencies for exterior single ledge (V_{pd})

$\phi := 0.9$ *Resistance factor*

$V_{pd\_ext} := \dfrac{V_{u\_ext}}{\phi} - V_{np\_ext} = 13.3\ kip$ *Punching shear deficiency for exterior*

3. Determine required effective perimeter increment Δp

The relation between deficiency V_{pd} and Δp can be expressed as

$$V_{pd} := 0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \Delta p \cdot d_f$$

From the above equation, Δp can be calculated as

$$\Delta p := \frac{V_{pd}}{0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot d_f}$$

$$\Delta p_{ext\_req} := \frac{V_{pd\_ext}}{0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot d_f}$$

$\Delta p_{ext\_req} = 3.3\ in$ *Required effective perimeter increment for exterior*

4. Determine increment of bearing pad dimension

p_{ext} = lesser of:

$0.5 \cdot W + L + d_f \cdot \cot(35°) + C = 64.8\ in$

$W + 2 \cdot L + 2 \cdot d_f \cdot \cot(35°) = 85.6\ in$

$p_{ext} := 0.5 \cdot W + L + d_f \cdot \cot(35°) + C$ (d_f and C is constant)

Therefore,

$\Delta p_{ext} := 0.5 \cdot \Delta W + \Delta L$

Try increasing W by 1 in. and L by 1.5 in. on each side.

$\Delta W_{ext} := 2 \cdot 1\ in = 2\ in$ $\Delta L_{ext} := 2 \cdot 1.5\ in = 3\ in$

Increment of interior effective perimeter of the concrete failure surface

$\Delta p_{ext} := 0.5 \cdot \Delta W_{ext} + \Delta L_{ext} = 4\ in$

- Check exterior punching shear capacity with increased bearing pad dimension

$W'_{ext} := W + \Delta W_{ext} = 23\ in$

$L'_{ext} := L + \Delta L_{ext} = 11\ in$

$V'_{np\_ext} := 0.125 \cdot \sqrt{f'_c \cdot ksi} \cdot \left(0.5 \cdot W'_{ext} + L'_{ext} + d_f \cdot \cot(35°) + C\right) \cdot d_f = 277.3\ kip$

$\phi \cdot V'_{np\_ext} = 249.6\ kip$ $>$ $V_{u\_ext} = 247\ kip$ (O.K.)

APPENDIX C. DESIGN EXAMPLE FOR SINGLE-COLUMN BENT

C.1 SOLUTION 3: END REGION STIFFENER

Design Problem:

Bent 22 has hanger and punching shear deficiencies with anhorage zone reduction factor at exterior girder locations based on AASHTO LRFD (2014). End-region stiffener provides alternative load paths so that it can improve the exterior bent capacity.

The required load demands on the exterior ledges are shown below:

$V_{u\_ext} := 207\ kip$ (for single ledge)

$V_{ut\_ext} := V_{u\_ext} \cdot 2 = 414\ kip$ (for both ledges)

$M_{u\_ext} := V_{u\_ext} \cdot a_v + 0.2\ V_{u\_ext} \cdot \left(h_{ext} - d_{e\_ext}\right) = 141.45\ kip \cdot ft$ *Concurrent ledge moment on a single ledge*

Speficy Web Anchor

1. Determine the required shear force for the web anchors(V_{3_req})

$\phi := 0.8$ *Strength reduction factor for normal weight concrete in anchorage zone (AASHTO 5.5.4.2)*

$V_{dh\_ext} := \dfrac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 44.81\ kip$ *Hanger deficiency*

$V_{h\_req} := 2 \cdot V_{dh\_ext} = 89.61\ kip$

Try 1 in. diameter epoxy anchor with B7 threadbar

2. Determine required number of anchors

- As an example, epoxy anchor with B7 threadbar manufactured by Williams Form Inc. may be used.

Properties of 0.875 in. diameter epoxy anchor

$d_a := 0.875\ in$ *Anchor diameter*

$A_a := 0.464\ in^2$

$f_{ya} := 105\ ksi$

$f_{ut} := 125\ ksi$

$T := A_a \cdot f_{ut} = 58\ kip$ *Design load = tensile strength (Williams Form)*

$W_s := 31\ kip$ *Working load (Williams Form)*

$h_{ef} := 16\ in$ *Embedded depth (Williams Form)*

$V_{s3w} := 0.6 \cdot T = 34.8\ kip$ *Shear strength*

$d_h := d_a + \dfrac{1}{8}\ in = 1\ in$ *Hole size*

Required number of anchors on the web

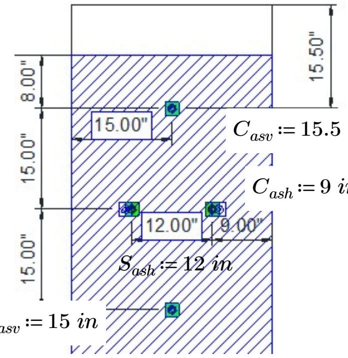
$\phi_a := 0.65$ *Strength reduction factor for post-installed anchors with Category 2 (ACI 318-14 17.3.3)*

$n_{3w\_req} := \dfrac{V_{h\_req}}{\phi_a \cdot V_{s3w}} = 3.96$

Try 4 anchors on the web $n_{3w} := 4$

※ Anchors on the ledges may not contribute for shear resistance but hold the end plate. Thus, in this design example, anchors on the ledges are not accounted for in resisting shear force but accounted for in resisting pullout tension force on the ledges.

3. Determine layouts of the anchors



$C_{asv} := 15.5\ in$

$C_{ash} := 9\ in$

$S_{ash} := 12\ in$

$S_{asv} := 15\ in$

$S_{a\_min} := 6 \cdot d_a = 5.25\ in$ *Minimum spacing*

C_{a_min} = maximum of *Minimum edge distance*

$1.5\ in$

$6 \cdot d_a$

$S_{a\_min} = 6 \cdot d_a$

$C_{a\_min} := 7.5\ in$

Try layouts shown in figure

4. Check that the shear capacity of web anchors is greater than the demand

- According to ACI 318-14, anchors in shear should be checked for steel strength, concrete breakout strength, and concrete pryout strength.

<u>Steel Strength of Anchor</u>

$V_{sn3} := V_{s3w} \cdot n_{3w} = 139.2\ kip$

$\phi_a \cdot V_{sn3} = 90.48\ kip \quad > \quad V_{h\_req} = 89.61\ kip$ (O.K.)

<u>Concrete Breakout Strength of Anchor</u>

$S_{cr} := 3\ C_{ash} = 27\ in$ *Critical spacing (ACI 318-14 17.2.1.1)*

$min\left(S_{asv}, S_{ash}\right) \le S_{cr}$ *Group effect shall be considered*

$V_{cb3} = V_{cbg} \cdot 2$ *Concrete breakout strength for shear loading parallel to an edge*

$V_{cbg} = \dfrac{A_{Vc}}{A_{Vco}} \psi_{ec\_v} \cdot \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b$ *Concrete breakout strength for shear loading perpendicular to an edge on a group of anchors*

$A_{Vco} := 4.5 \cdot \left(C_{ash}\right)^2 = 364.5\ in^2$ *Projected area for single anchor in deep member in the direction perpendicular to the shear force*

$A_{Vc} := \left(S_{ash} + 2 \cdot C_{ash}\right)\left(C_{asv} + S_{asv} \cdot 3 + 1.5\ h_{ef}\right) = 2535\ in^2 \quad > \quad n_{3w} \cdot A_{Vco} = 1458\ in^2$ *Projected area of the failure surface on the side of the concrete member at its edge for a group of anchors*

take $A_{Vc} := 1458\ in^2$

$\lambda_a := 1.0$ *Modification factor to reflect the reduced mechanical properties of light weight concrete*

V_b = minimum of *Basic concrete breakout strength value for a single anchor*

$$V_{ba} := \left(7 \cdot \left(\frac{h_{ef}}{d_a}\right)^{0.2} \cdot \sqrt{d_a}\right) \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{ash}\right)^{1.5} = 599.85\ kip$$

$$V_{bb} := 9\ in^{\frac{1}{2}} \cdot \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{ash}\right)^{1.5} = 461.06\ kip$$

$V_b := min\left(V_{ba}, V_{bb}\right) = 461.06\ kip$

$\psi_{ec\_v} := 1.0$ *Modification factor for anchor groups loaded eccentrically*

$\psi_{ed\_v} := 1.0$ *Modification factor for edge effect*

$\psi_{c\_v} := 1.4$ *Modification factor for cracking effect at service*

$\psi_{h\_v} := 1.0$ *Modification factor for anchors located in narrow concrete member*

$V_{cbg} := \frac{A_{Vc}}{A_{Vco}} \psi_{ec\_v} \cdot \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b = 2581.94\ kip$

$\phi_a \cdot V_{cbg} = 1678.26\ kip$ $>$ $V_{h\_req} = 89.61\ kip$ (O.K.)

Concrete Pryout Strength of Anchor

$V_{cpg} = k_{cp} \cdot N_{cpg}$ *Concrete pryout strength*

$k_{cp} := 2.0$ *"k_{cp}" is 2.0 for effective embedded length of anchor larger than 2.5 in.*

$N_{cpg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba}$ *Lesser of bond strength of anchor and concrete breakout strength of anchor in tension*

$\tau_{uncr} := 1640\ psi$ *Characteristic bond stress in un-cracked concrete*

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}} = 10.68\ in$ *Rectilinear area that projects outward a distance*

$A_{Nco} := \left(2 \cdot c_{Na}\right)^2 = 456.59\ in^2$ *Projected area for single anchor in deep member in the direction perpendicular to the shear force*

$A_{Nc} := \left(S_{ash} + 2 \cdot C_{ash}\right) \left(2\ c_{Na} + 3 \cdot S_{asv}\right) = 1991.04\ in^2$ $>$ $n_{3w} \cdot A_{Nco} = 1826.36\ in^2$ *Projected area of the failure surface on the side of the concrete member at its edge for a group of anchors*

$A_{Nc} := 1826.36\ in^2$

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_h \cdot h_{ef} = 82.44\ kip$ *Basic bond strength of a single adhesive anchor in tension in cracked concrete*

$\psi_{ec\_Na} := 1.0$ *Modification factor for anchor groups loaded eccentrically*

$\psi_{ed\_Na} := 0.7 + 0.3\ \frac{C_{ash}}{c_{Na}} = 0.95$ *Modification factor for edge effect*

$\psi_{cp\_Na} := \frac{C_{ash}}{c_{Na}} = 0.84$ *Modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting*

$N_{cpg} := \frac{A_{Nc}}{A_{Nco}} \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 264.63\ kip$

$N_{cp3} := k_{cp} \cdot N_{cpg} = 529.27\ kip$

$\phi_a \cdot N_{cp3} = 344.02\ kip$ $>$ $V_{h\_req} = 89.61\ kip$ (O.K.)

Increased hanger capacity for single ledge

$\phi V_{3h} := min\left(\phi_a \cdot V_{sn3}, \phi_a \cdot V_{cbg}, \phi_a \cdot N_{cp3}\right) = 90.48\ kip$

$\phi V_{nh} := \frac{\phi V_{3h}}{2} + \phi \cdot V_{nh\_ext} = 216.39\ kip$

Specify Ledge Anchor

1. Determine the required shear force for the webanchors(V_{3_req})

- Since the anchor hole for the ledge anchor is deeper than holes for the web anchor, use anchors with smaller diameter.
- As an example, 1/5 in. diameter epoxy anchor with B7 threadbar manufactured by Williams Form Inc. may be used
- To ensure embedded depth is not affected by cracking on the ledges, the length of embedded depth for ledge anchors shall be taken as follows:

With the effective embedded depth $h_{ef} := 4.5\ in$, which is the minimum required embedded depth specified by manufacturer

$$h_e := C + \frac{W}{2} + h_{ext} + h_{ef} = 53.75\ in$$

Properties of 1/5 in. diameter epoxy anchor

$d_a := \frac{1}{5}\ in$ *Anchor diameter*

$A_a := 0.144\ in^2$

$f_{ya} := 105\ ksi$

$f_{ut} := 125\ ksi$

$T := A_a \cdot f_{ut} = 18\ kip$ *Design load = tensile strength (Williams Form)*

$W_l := 7.25\ kip$ *Working load (Williams Form)*

$h_{ef} = 4.5\ in$ *Embedded depth (Williams Form)*

$V_{s3l} := 0.6 \cdot T = 10.8\ kip$ *Shear strength*

$d_h := d_a + \frac{1}{8}\ in = 0.33\ in$ *Hole size*

Required shear strength for a group of ledge anchors

$V_{nl\_ext} := min\left(V_{ns\_ext}, V_{nf\_ext}, V_{np\_ext}\right) = 236.8\ kip$ *Minimum ledge capacity*

$V_{dl} := 0.5 \cdot \left(\frac{V_{u\_ext}}{\phi} - V_{nl\_ext}\right) \cdot \cos\left(45°\right)^2 = 5.49\ kip$ *Vertical component of ledge deficiency for a single ledge*

$N_{dl} := 0.5 \cdot \left(\frac{V_{u\_ext}}{\phi} - V_{nl\_ext}\right) \cdot \cos\left(45°\right)^2 = 5.49\ kip$ *Horizontal component of ledge deficiency for a single ledge*

$V_{3l\_req} := 2 \cdot V_{dl} = 10.97\ kip$

$N_{3l\_req} := 2 \cdot N_{dl} = 10.97\ kip$

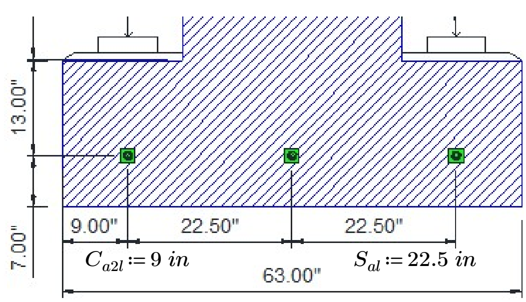
2. Determine required number of anchors

Required number of anchors on the web

$$n_{s3\_req} := \frac{V_{3l\_req}}{\phi_a \cdot V_{s3l}} = 1.56$$

$$n_{s3\_req} := \frac{N_{3l\_req}}{\phi_a \cdot T} = 0.94$$

$C_{a1l} := 7\ in$



$C_{a2l} := 9\ in$ $S_{al} := 22.5\ in$

- Try 3 anchors on the ledge (the minimum number of anchors) ($n_{3l} := 3$) with layouts shown in figure.
*Place anchors at the bottom kern point. ($C_{a1l} := 7\ in$)

3. Check that the shear and tension capacity of ledge anchors is greater than the demand

Steel Strength of Anchor

$N_{sn3h} := T \cdot n_{3l} = 54\ kip$

$\phi_a \cdot N_{sn3h} = 35.1\ kip$ > $N_{3l\_req} = 10.97\ kip$ (O.K.)

$V_{sn3v} := V_{s3l} \cdot n_{3l} = 32.4\ kip$

$\phi_a \cdot V_{sn3v} = 21.06\ kip$ > $V_{3l\_req} = 10.97\ kip$ (O.K.)

Concrete Breakout Strength of Anchor

$S_{cr} := 3\ h_{ef} = 13.5\ in$ *Critical spacing (ACI 318-14 17.2.1.1)*

Since $S_{al} = 76.33\ in > S_{cr} = 13.5\ in$, group effect shall not be considered.

$N_{cb} = \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b$ *Concrete breakout strength of anchor in tension for a single anchor*

$V_{cb} = \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b$ *Concrete breakout strength for shear loading perpendicular to an edge on a single anchor*

$\lambda_a := 1.0$ *Modification factor to reflect the reduced mechanical properties of light weight concrete*

$\kappa_c := 17$ *"κ_c" is 17 for pot-installed anchors*

$N_b := \kappa_c \cdot \lambda_a \cdot \sqrt{f'_c \cdot ksi} \cdot h_{ef}^{1.5}\ in^{\frac{1}{2}} = 307.91\ kip$

V_b = minimum of *Basic concrete breakout strength value for a single anchor*

$$V_{ba} := \left(7 \cdot \left(\frac{h_{ef}}{d_a}\right)^{0.2} \cdot \sqrt{d_a}\right) \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{a1l}\right)^{1.5} = 205.04\ kip$$

$$V_{bb} := 9\ in^{\frac{1}{2}} \cdot \lambda_a \cdot \sqrt{f'_c \cdot 1000\ psi} \cdot \left(C_{a1l}\right)^{1.5} = 316.26\ kip$$

$V_b := min\left(V_{ba}, V_{bb}\right) = 205.04\ kip$

$\psi_{ed\_N} := 0.7 + 0.3\ \dfrac{C_{a1l}}{1.5\ h_{ef}} = 1.01$ *Modification factor for edge effect*

$\psi_{ed\_v} := \psi_{ed\_N}$

$\psi_{c\_N} := 1.4 = \psi_{c\_v}$ *Modification factor for cracking effect at service*

$C_{ac} := 2\ h_{ef}$ *The critical edge distance (ACI 318-14 Sec.17.7.6)*

$\psi_{cp\_N} := \dfrac{C_{a1l}}{C_{ac}} = 0.78$

$\psi_{h\_v} := 1.0$ *Modification factor for anchors located in narrow concrete member*

$N_{cbg} := n_{3l} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 726.43\ kip$

$V_{cbg} := n_{3l} \cdot \psi_{ed\_v} \cdot \psi_{c\_v} \cdot \psi_{h\_v} \cdot V_b = 870.76\ kip$

$\phi_a \cdot N_{cbg} = 472.18\ kip$ > $N_{3l\_req} = 10.97\ kip$ (O.K.)

$\phi_a \cdot V_{cbg} = 565.99\ kip$ > $V_{3l\_req} = 10.97\ kip$ (O.K.)

<u>Bond Strength of Anchor</u>

$h_{ef} = 4.5\ in$ *Embedded depth*

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\dfrac{\tau_{uncr}}{1100\ psi}} = 2.44\ in$ *Rectilinear area that projects outward a distance*

$S_{cr} := 2\ c_{Na} = 4.88\ in$

Since $S_{al} = 22.5\ in > S_{cr} = 4.88\ in$, group effect shall not be considered.

$N_{cp} = \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba}$ *Concrete pryout strength*

$\tau_{uncr} := 1640\ psi$ *Characteristic bond stress in uncracked concrete*

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_h \cdot h_{ef} = 7.54\ kip$ *Basic bond strength of a single adhesive anchor in tension in cracked concrete*

Modification factor for anchor groups loaded eccentrically

$\psi_{ed\_Na} := 0.7 + 0.3\ \dfrac{C_{a1l}}{c_{Na}} = 1.56$ *Modification factor for edge effect*

$\psi_{cp\_Na} := \dfrac{C_{a1l}}{c_{Na}} = 2.87$ *Modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting*

$N_{a3} := n_{3l} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 101.08\ kip$

$\phi_a \cdot N_{a3} = 65.7\ kip$ > $N_{3l\_req} = 10.97\ kip$ (O.K.)

<u>Concrete Pryout Strength of Anchor</u>

$V_{cpg} = k_{cp} \cdot N_{cpg}$ *Concrete pryout strength*

$k_{cp} := 2.0$ *"k_{cp}" is 2.0 for effective embedded length of anchor larger than 2.5 in.*

$\tau_{uncr} := 1640\ psi$ *Characteristic bond stress in uncracked concrete*

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\dfrac{\tau_{uncr}}{1100\ psi}} = 2.44\ in$ *Rectilinear area that projects outward a distance*

$N_{ba} := \lambda_a \cdot \tau_{uncr} \cdot \pi \cdot d_h \cdot h_{ef} = 7.54\ kip$ *Basic bond strength of a single adhesive anchor in tension in cracked concrete*

$\psi_{ed\_Na} := 0.7 + 0.3\ \dfrac{C_{a1l}}{c_{Na}} = 1.56$ *Modification factor for edge effect*

$\psi_{cp\_Na} := \dfrac{C_{a1l}}{c_{Na}} = 2.87$ *Modification factor for adhesive anchors designed for uncracked concrete without supplementary reinforcement to control splitting*

$N_{cpg} := n_{3l} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 101.08\ kip$

$N_{cp3} := k_{cp} \cdot N_{cpg} = 202.16\ kip$

$\phi_a \cdot N_{cp3} = 131.4\ kip$ $>$ $N_{3l\_req} = 10.97\ kip$ (O.K.)

Increased hanger capacity for single ledge

$\phi V_{3l} := min\left(\phi_a \cdot N_{sn3h}, \phi_a \cdot V_{sn3v}, \phi_a \cdot N_{cbg}, \phi_a \cdot V_{cbg}, \phi_a \cdot N_{a3}, \phi_a \cdot N_{cp3}\right) = 21.06\ kip$

$$\phi V_{nl} := \frac{\phi V_{3l}}{\cos\left(45°\right)^2} + \phi \cdot V_{nl\_ext} = 231.56\ kip$$

4. Check the interaction between the shear and tension of ledge anchors

$$\left(\frac{V_{3l\_req}}{V_{s3l} \cdot n_{3l}}\right)^{\frac{5}{3}} + \left(\frac{N_{3l\_req}}{T \cdot n_{3l}}\right)^{\frac{5}{3}} = 0.23 \quad < 1.0 \qquad \text{(O.K.)}$$

End Plate Design

1. Determine required thickness of end plate

Grade 50 steel end plate

$F_y := 50\ ksi$ *Yield stress*

$F_u := 65\ ksi$ *Ultimate stress*

$E := 29000\ ksi$ *Young's modulus*

Required thickness for hanger deficiency

$V_{dh\_ext} = 44.81\ kip$

$V_{nh3\_ext} = b_{stem} \cdot t_d \cdot F_y$

$$t_{d\_req} := \frac{V_{dh\_ext}}{b_{stem} \cdot F_y} = 0.03\ in$$

Required thickness for axial bearing

$\phi_t := 0.75$ *Resistance factor (AISC Specification J3.10)*

$$t_{a\_req} := \frac{W_s}{\phi_t \cdot 2.4\ d_h \cdot F_u} = 0.82\ in$$

Required thickness for shear bearing

$$V_{s3} := \max\left(\frac{V_{h\_req}}{n_{3w}}, \frac{V_{3l\_req}}{n_{3l}}\right) = 22.4\ kip$$

$\phi_s := 0.65$ *Resistance factor (AISC Specification J3.10)*

$$t_{sb\_req} := \frac{V_{s3}}{\phi_s \cdot 2.0\ d_h \cdot F_u} = 0.82\ in$$

Required thickness for shear rupture

$\phi_r := 0.75$

$h_n := C_{a1l} - \frac{d_h}{2} = 6.84\ in$ *Distance from edge of the plate to the edge of the nearest hole*

$t_{sr\_req} := \frac{V_{s3}}{\phi_r \cdot 0.6 \cdot h_n \cdot F_y} = 0.15\ in$

$t_{req} := \max\left(t_{d\_req}, t_{a\_req}, t_{sb\_req}, t_{sr\_req}\right) = 0.82\ in$ *Minimum thickness of plate*

Try 0.875 in. thick plate ($t := 0.875\ in$)

Design Triangular Stiffener

1. Determine required thickness for horizontal force

Grade 50 steel end plate

$F_y := 50\ ksi$ *Yield stress*

$F_u := 65\ ksi$ *Ultimate stress*

$E := 29000\ ksi$ *Young's modulus*

$b := 20\ in$

$a := 10.5\ in$

$\frac{a}{b} = 0.53$

$P_u := 0.5 \cdot N_{dl} \cdot \cos\left(45°\right)^2 = 1.37\ kip$

$P_n := \frac{P_u}{\phi} = 1.71\ kip$

$s_h := 9.5\ in$

$m_s := \frac{P_n \cdot s_h}{b^3 \cdot E} = 7.02 \cdot 10^{-8}$ *Dimensionless moment*

From the design aid table (Shakya and Vinnakota, 2008) and using interpolation,

$\frac{t_s}{b} = 7.76 \cdot 10^{-3}$

Thus, the plate thickness t_s is

$t_s := b \cdot 7.76 \cdot 10^{-3} = 0.16\ in$

Try $t_s := 0.5\ in$

Check minimum thickness

$\frac{t}{b} > 0.0225$ for $F_y = 50\ ksi$ and $\frac{a}{b} = 0.53$

$t_{s\_min} := 0.0225 \cdot b = 0.45\ in$ $<$ $t_s = 0.5\ in$

2. Determine required thickness for vertical force

$$\frac{b}{a} = 1.9$$

$$P_u := 0.5 \cdot N_{dl} \cdot \cos\left(45°\right)^2 = 1.37\ kip$$

$$P_n := \frac{P_u}{\phi} = 1.71\ kip$$

$$s_v := \frac{a}{2} = 5.25\ in$$

$$m_s := \frac{P_n \cdot s_v}{a^3 \cdot E} = 2.68 \cdot 10^{-7}$$

From the design aid table (Shakya and Vinnakota, 2008) and using interpolation,

$$\frac{t_s}{a} = 6.62 \cdot 10^{-3}$$

Thus, the plate thickness t_s is

$$t_{s\_req} := a \cdot 6.62 \cdot 10^{-3} = 0.07\ in \qquad < \qquad t_s = 0.5\ in$$

<u>Check minimum thickness</u>

$\frac{t}{a} > 0.0427$ for $F_y = 50\ ksi$ and $\frac{b}{a} = 1.9$

$$t_{s\_min} := 0.0427 \cdot a = 0.45\ in \qquad < \qquad t_s = 0.5\ in$$

3. Determine weld size a_w

$a_{w\_min} := \frac{1}{4}\ in = 0.25\ in$ *(AISC Specification Table J2.4)*

$a_{w\_max} := t_s - \frac{1}{16}\ in = 0.44\ in$ *(AISC Specification J2.2)*

With E70 electrodes,

$$F_{EXX} := 70\ ksi$$

$F_{nw} = 0.6\ F_{EXX}\left(1.0 + 0.5\ sin\left(\theta\right)^{1.5}\right)$ *(AISC Specification J2-4)*

$\phi := 0.75$ *(AISC Specification J2.4)*

$\phi F_{nw} \cdot A_{we} = \phi F_{nw} \cdot t_e \cdot L = \phi F_{nw} \cdot cos\left(45°\right) \cdot a_w \cdot L$ *Fillet weld strength*

<u>For vertical weld</u>

$$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin\left(0°\right)^{1.5}\right) = 42\ ksi$$

$$L := b = 20\ in$$

$$a_{w\_req} := \frac{P_n}{\phi \cdot F_{nw} \cdot \cos\left(45°\right) \cdot L} = 0.004\ in$$

<u>For horizontal weld</u>

$$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5 \sin\left(0°\right)^{1.5}\right) = 42 \; ksi$$

$$L := a = 10.5 \; in$$

$$a_{w\_req} := \frac{P_n}{\phi \cdot F_{nw} \cdot \cos\left(45°\right) \cdot L} = 0.01 \; in$$

Use 0.25 in. fillet weld for both sides

C.2 SOLUTION 8: CLAMPED THREADBAR WITH CHANNEL

Design Problem:

Bent 22 has hanger deficiencies for exterior and interior based on AASHTO LRFD (2014). Clamped threadbar with channel will be designed to strengthen "Exterior" and "Interior 1" part .

The required loads on the ledges are shown below:

Exterior

$V_{u\_ext} := 207\ \text{kip}$ (for single ledge)

$V_{ut\_ext} := V_{u\_ext} \cdot 2 = 328.28\ \text{kip}$ (for both ledges)

$M_{u\_ext} := V_{u\_ext} \cdot a_v + 0.2\ V_{u\_ext} \cdot \left(h_{ext} - d_{e\_ext}\right) = 141.45\ \text{kip} \cdot \text{ft}$

Interior1

$V_{u\_int} := 235\ \text{kip}$ (for single ledge)

$V_{ut\_int} := V_{u\_int} \cdot 2 = 480.41\ \text{kip}$ (for both ledges)

$M_{u\_int} := V_{u\_int} \cdot a_v + 0.2\ V_{u\_int} \cdot \left(h_{int1} - d_{e\_int1}\right) = 160.58\ \text{kip} \cdot \text{ft}$

Specify Threadbar

1.Determine the hanger deficiency (V_{hd}) for single ledge

$\phi := 0.9$ *Resistance factor (AASHTO LRFD 5.5.4.2)*

Exterior

$$V_{hd\_ext} := \frac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 16.06\ \text{kip}$$

Interior

$$V_{hd\_int} := \frac{V_{u\_int}}{\phi} - V_{nh\_int1} = 33.74\ \text{kip}$$

2. Determine the required threadbar contribution

Exterior

$V_{ext\_req} := V_{hd\_ext} \cdot 2 = 32.11\ \text{kip}$

Interior

$V_{int\_req} := V_{hd\_int} \cdot 2 = 67.49\ \text{kip}$

3. Determine the required area of the threadbar

Threadbar properties:

Use Grade B7 Threadbar

$f_y := 105\ \text{ksi}$ *Yield stress of bar*

$f_u := 125\ \text{ksi}$ *Ultimate stress of bar*

Exterior

$$A_{ext\_req} := \frac{V_{ext\_req}}{f_y} = 0.31\ \text{in}^2$$

Interior

$A_{int\_req} := \dfrac{V_{int\_req}}{f_y} = 0.64 \text{ in}^2$

4. Determine the number of threadbars

Try 0.75 in. diameter high strength

$d_b := 0.75 \text{ in}$ *Bar diameter*

$A := 0.334 \text{ in}^2$ *Net area of bar*

$f_y := 105 \text{ ksi}$ *Yield stress of bar*

$f_u := 125 \text{ ksi}$ *Ultimate stress of bar*

Exterior

$n_{t\_ext\_req} := \dfrac{A_{ext\_req}}{A} = 0.92$

reduce bar size to 5/8 in. diameter to have even number of bars

$A_{est} := 0.232 \text{ in}^2$

$n_{t\_ext\_req} := \dfrac{A_{ext\_req}}{A_{est}} = 1.32$ $n_{ext} := 2$

Interior

$n_{t\_int\_req} := \dfrac{A_{int\_req}}{A} = 1.92$ $n_{int} := 2$

5. Check that the capacity is greater than the demand (service limit and strength limit)

$V_t := A \cdot f_y = 35.07 \text{ kip}$ *Tensile strength - contribution of a bar*

$V_{t\_ext} := A_{est} \cdot f_y = 24.36 \text{ kip}$

Exterior

$V_{sh\_ext} = 103.54 \text{ kip}$ *Exterior hanger capacity at service limit for single ledge*

$V_{slt\_ext} := V_{sh\_ext} \cdot 2 = 207.09 \text{ kip}$ *Exterior service load for both ledges*

$V_{s\_ext} := V_{t\_ext} \cdot n_{ext} + V_{sh\_ext} \cdot 2 = 255.81 \text{ kip}$ (service limit)

$\phi \cdot V_{s\_ext} = 230.23 \text{ kip} \quad > \quad V_{slt\_ext} = 207.09 \text{ kip}$ (O.K.)

$V_{nr\_ext} := V_t \cdot n_{ext} + V_{nh\_ext} \cdot 2 = 498.03 \text{ kip}$ (strength limit)

$\phi \cdot V_{nr\_ext} = 448.22 \text{ kip} \quad > \quad V_{ut\_ext} = 414 \text{ kip}$ (O.K.)

Interior1

$V_{sh\_int1} = 91.58 \text{ kip}$ *Interior hanger capacity at service limit for single ledge*

$V_{slt\_int} := 297.38 \text{ kip}$ *Interior service load for both ledges*

$V_{s\_int} := V_t \cdot n_{int} + V_{sh\_int1} \cdot 2 = 253.3 \text{ kip}$ (service limit)

$\phi \cdot V_{s\_int} = 227.97 \text{ kip} \quad > \quad V_{slt\_int} = 137.37 \text{ kip}$ (O.K.)

$V_{nr\_int} := \left(V_t \cdot n_{int} + V_{nh\_int1} \cdot 2\right) = 524.88 \text{ kip}$ (strength limit)

$\phi \cdot V_{nr\_int} = 472.39 \text{ kip} \quad > \quad V_{ut\_int} = 470 \text{ kip}$ (O.K.)

Channel Design

- There is no deficiency on the ledge, thus, channel is not needed.

Bearing Plate Design

1. Determine required thickness of plate

$V_{pt} := 0.6 \cdot f_u \cdot A = 25.05 \text{ kip}$ *Design Load for Anchoring (Williams Form)*

$\phi_t := 0.75$ *Resistance factor (AISC Specification J3.10)*

A36 steel properties

$F_y := 36 \text{ ksi}$ *Yield stress of the channel*

$F_u := 65 \text{ ksi}$ *Ultimate stress of the channel*

$E := 29000 \text{ ksi}$ *Young's modulus of the channel*

Required thickness for axial bearing

$$t_{req} := \frac{V_{pt}}{\phi_t \cdot 2.4\ d_b \cdot F_u} = 0.29 \text{ in}$$

Use 0.375 in. thickness plate

2. Determine required bearing area

$\phi_c := 0.65$ *Resistance factor (AISC Specification J8)*

$$A_{req} := \frac{V_{pt}}{\phi_c \cdot 0.85 \cdot f'_c} = 12.59 \text{ in}^2$$

Use 4 in. x 4 in. rectangular plate with 0.375 in. thickness

※ For fastening threadbars, hillside washers are needed.

C.3 SOLUTION 14: LOAD BALANCING POST TENSIONING

Design Problem:

Bent 22 has hanger deficiency for exterior and interior based on AASHTO LRFD (2014). Load balancing post-tensioning (PT) strands will be designed to provides alternative load paths so that it can improve the overall bent capacity.

The required load demands on single ledge are shown below:

Exterior

$V_{u\_ext} := 207\ kip$ (for single ledge)

$M_{u\_ext} := V_{u\_ext} \cdot a_v + 0.2\ V_{u\_ext} \cdot (h_{ext} - d_{e\_ext}) = 141.45\ kip \cdot ft$

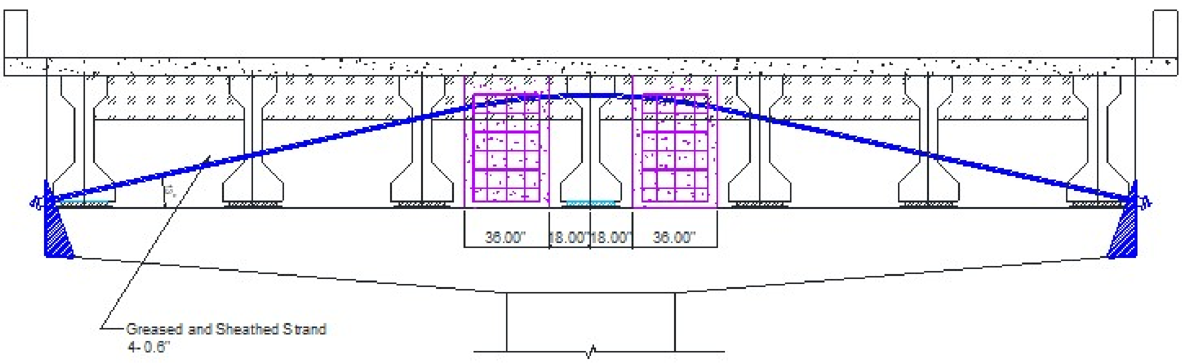
Interior1

$V_{u\_int} := 235\ kip$ (for single ledge)

$M_{u\_int} := V_{u\_int} \cdot a_v + 0.2\ V_{u\_int} \cdot (h_{int1} - d_{e\_int1}) = 160.58\ kip \cdot ft$

Concrete infill block

- To transfer load from the strands to the column, the concrete infill block needs to be seated on the column, and center of gravity of the concrete block should be placed on the column. For this design example, the dimension of the concrete infill block is shown below. Only minimum reinforcement is needed for the concrete infill block.



Reinforcement of Infill Concrete Block

1.Determine the maximum deficiency

$\phi := 0.9$ *Resistance factor (AASHTO LRFD 5.5.4.2)*

Exterior

$$V_{d\_ext} := \frac{V_{u\_ext}}{\phi} - V_{n\_ext} = 16.06\ kip$$

Interior1

$$V_{d\_int} := \frac{V_{u\_int}}{\phi} - V_{n\_int1} = 33.74\ kip$$

- Since Bent 22 is single column bent, regardless the portion of the bent, the maximum deficiency will be used to design this solution.

$V_d := \max(V_{d\_ext}, V_{d\_int}) = 33.74\ kip$

2. Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5\ in$ *Width of infill concrete block*

$h_c := 56.75\ in$ *Height of infill concrete block*

$d_e := 30\ in$ *Length of infill concrete block*

$f'_c := 3.6\ ksi$ *Concrete strength (use same concrete as the in-service structure)*

$f_y := 60\ ksi$ *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = the maximum of:

$$\frac{3\sqrt{f'_c \cdot psi}}{f_y} b_w \cdot d_e = 1.49\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_e = 1.65\ in^2$$

$A_{f\_min} := 1.49\ in^2$ *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

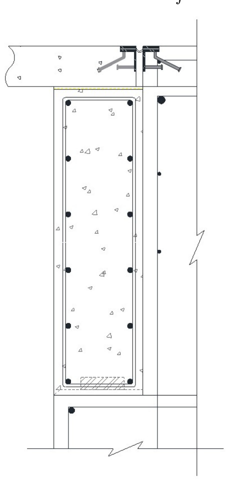
$s_{f\_max} := 12\ in$ *(ACI 318-14)*

$n_{re} := \frac{h_c}{s_{f\_max}} + 1 = 5.73$ *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 6$ *Take integer*

$n_f := n_{re} \cdot 2 = 12$ *Required number of longitudinal reinforcement*

$A_{fs\_re} := \frac{A_{f\_min}}{n_f} = 0.12\ in^2$ *Required area of single longitudinal bar*



Place #4 ($A_{fs} := 0.2\ in^2$) longitudinal bars at four corners of the concrete block, and evenly place four #4 longitudinal bars along the height of the concrete block on each side.

3.Determine required shear reinforcement

- Use double leg #3 stirrup

$A_v := 2 \cdot 0.11\ in^2 = 0.22\ in^2$

- Concrete shear strength

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_w \cdot d_e = 59.4\ kip$

- Required spacing of stirrups

$$s_{v\_req} := \frac{A_v \cdot f_y \cdot d_e}{V_d - V_c} = -15.43\ in$$

- Check maximum spacing of stirrups

s_{v_max} = the maximum of:

$$\frac{d_e}{2} = 15\ in$$

$$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c \cdot psi}\ b_w} = 17.78\ in$$

$$\frac{A_v \cdot f_y}{50\ psi\ b_w} = 16\ in$$

$s_{v\_max} := 17.78\ in$

Consider the geometry of the concrete infill block, evenly place four #3 double leg stirrups at a spacing of 15 in.

Specify PT Strand

1.Determine the required post-tensioning force (V_{14_req})

The required upward force for the strands is

$$V_{14\_req} := \frac{V_{d\_int}}{\sin(13°)} = 150\ kip$$

2. Determine required number of strands (try 0.6 in. strand)

Properties of 0.6" strand

$f_{pu} := 270\ ksi$	*Ultimate stress*
$A_{pt} := 0.217\ in^2$	*Net area of strand*
$P_{pu} := f_{pu} \cdot A_{pt} = 58.59\ kip$	*Ultimate strength of strand*
$P_{ps} := 0.7 \cdot P_{pu} = 41.01\ kip$	*Maximum force after transfer of prestressing force*
$f_{py} := 0.85 \cdot f_{pu} = 229.5\ ksi$	*Yield stress*
$f_{pe} := 0.8\ f_{py} = 183.6\ ksi$	*Stress at service limit state after losses*

$P_{pe} := f_{pe} \cdot A_{pt} = 39.84\ kip$ *Strength at service limit state after losses*

Required number of strands

$n_{14\_req} := \dfrac{V_{14\_req}}{P_{ps}} = 3.66$ use 4 - 0.6" strands ($n_{14} := 4$)

3. Check that the shear capacity is greater than the demand

$V_{14} := P_{ps} \cdot \sin(13°) = 9.23\ kip$

Exterior

$V_{n\_ext} := \left(V_{14} \cdot n_{14} + V_{n\_ext}\right) = 250.85\ kip$

$\phi \cdot V_{n\_ext} = 225.76\ kip$ > $V_{u\_ext} = 207\ kip$ (O.K.)

Interior

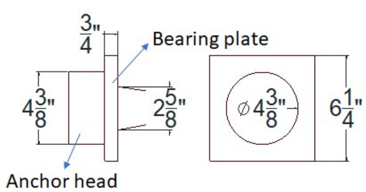
$V_{n\_int1} := \left(V_{14} \cdot n_{14} + V_{n\_int1}\right) = 264.27\ kip$

$\phi \cdot V_{n\_int1} = 237.84\ kip$ > $V_{u\_int} = 235\ kip$ (O.K.)

4. Determine Anchorage

With 4 - 0.6" strands, the anchorage will be chosen from manufacturer's multi strands anchorage catalog.

- For an example Type E 0.6 (unit 6-4) by VSL may be used, and its dimension is shown below:



Anchor System Design

1. Determine required thickness of end plate

$P_{pt} := n_{14} \cdot P_{pe} = 159.36\ kip$

A36 steel end plate and beveled properties

$F_y := 36\ ksi$ *Yield stress*

$F_u := 65\ ksi$ *Ultimate stress*

$E := 29000\ ksi$ *Young's modulus*

Assume the diameter of the strand bundle is 2.56 in. (same as ϕC of anchorage)

$d_s := 2.56\ in$

with 5 in. hole and beveled plate size, the height of vertical plate is determined

$b := 20\ in + \dfrac{5}{2}\ in + \dfrac{10.5}{2}\ in = 27.75\ in$

Required thickness for axial bearing

$\phi_b := 0.75$ *Resistance factor (AISC Specification J3.10)*

$$t_{a\_req} := \frac{P_{pt} \cdot \cos(16°)}{\phi_b \cdot 2.4\ d_s \cdot F_u} = 0.51\ in$$

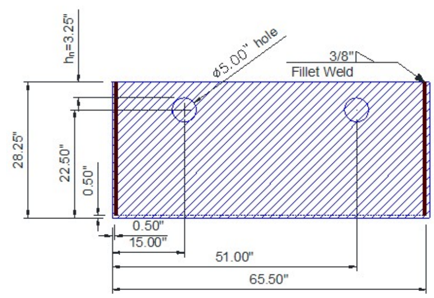
Required thickness for shear bearing

$$t_{a\_req} := \frac{P_{pt} \cdot \sin(16°)}{\phi_b \cdot 1.8\ d_s \cdot F_u} = 0.2\ in$$

Required thickness for shear yielding

$\phi_y := 1$

$$t_{a\_req} := \frac{P_{pt} \cdot \sin(16°)}{\phi_y \cdot 0.6\ b \cdot F_y} = 0.07\ in$$



Required thickness for shear rupture

$\phi_r := 0.75$

$h_n := 5.5\ in$

$$t_{a\_req} := \frac{P_{pt} \cdot \sin(16°)}{\phi_r \cdot 0.6 \cdot h_n \cdot F_y} = 0.49\ in$$

Use 0.5 in. thick anchor plates ($t_a := 0.5\ in$)

2. Determine dimension of beveled plate

$\phi_c := 0.65$ *Resistance factor (AISC Specification J8)*

$$A_{req} := \frac{P_{pt}}{\phi_c \cdot 0.85 \cdot f'_c} = 80.12\ in^2$$

$$d_{req} := \sqrt{\frac{A_{req} \cdot 4}{\pi}} = 10.1\ in$$

Use 10.5 in. outer diameter beveled plate for whole bundle of strands with the minimum thickness of 0.5 in.

3. Determine dimensions of the triangular stiffener *(AISC Design Manual)*

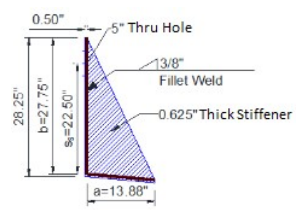
- Since the recommended plate aspect ratio, a/b, ranges from 0.5 to 3.0, try a/b=0.5

$a := b \cdot 0.5 = 13.88\ in$

$P_u := P_{pt} \cdot \cos(16°) = 153.19\ kip$

$$P_n := \frac{P_u}{\phi} = 170.21\ kip$$

$s_s := 22.5\ in$



$$m_s := \frac{P_n \cdot s_s}{b^3 \cdot E} = 6.18 \cdot 10^{-6}$$ *Dimensionless moment*

- From the design aid table (Shakya and Vinnakota, 2008) and using interpolation,

$$\frac{t_s}{b} = 11.95 \cdot 10^{-3}$$

Thus, the plate thickness t_s is

$$t_s := b \cdot 11.95\ 10^{-3} = 0.33\ in$$

Use 0.625 in. triangular plate ($t_s := 0.625\ in$)

<u>Check minimum thickness</u>

$\frac{t}{b} > 0.0188$ for $F_y = 36\ ksi$ and $\frac{a}{b} = 0.5$

$t_{s\_min} := 0.0188 \cdot b = 0.52\ in$ < $t_s = 0.63\ in$ (O.K.)

4. Determine weld size a

$a_{min} := \frac{1}{4}\ in = 0.25\ in$ *(AISC Specification Table J2.4)*

$a_{max} := t_a - \frac{1}{16}\ in = 0.44\ in$ *(AISC Specification J2.2)*

With E70 electrodes,

$F_{EXX} := 70\ ksi$

$F_{nw} = 0.6\ F_{EXX} \left(1.0 + 0.5\ sin(\theta)^{1.5}\right)$ *(AISC Specification J2-4)*

$\phi := 0.75$ *(AISC Specification J2.4)*

$\phi F_{nw} \cdot A_{we} = \phi F_{nw} \cdot t_e \cdot L = \phi F_{nw} \cdot cos(45°) \cdot a \cdot L$ *Fillet weld strength*

<u>For vertical weld</u>

$$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin(16°)^{1.5}\right) = 45.04\ ksi$$

$$L := b = 27.75\ in$$

$$a_{req} := \frac{P_{pt}}{\phi \cdot F_{nw} \cdot \cos(45°) \cdot L} = 0.24\ in$$

<u>For horizontal weld</u>

$$F_{nw} := 0.6 \cdot F_{EXX} \cdot \left(1.0 + 0.5\ \sin(90°)^{1.5}\right) = 63\ ksi$$

$$L := a = 13.88\ in$$

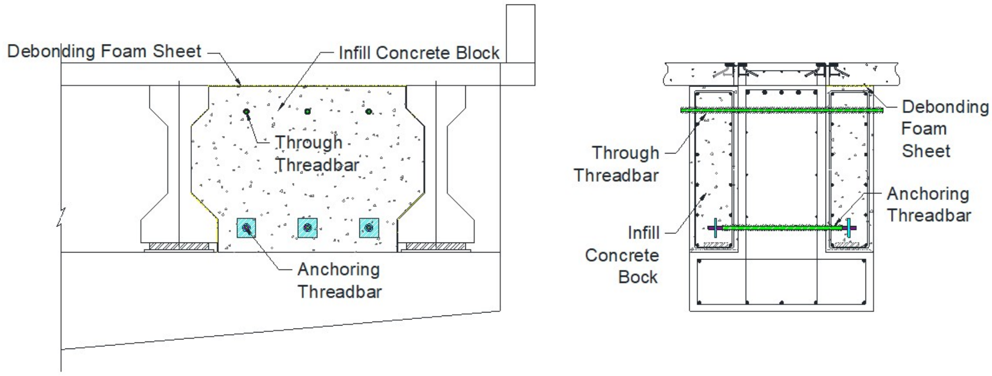
$$a_{req} := \frac{P_{pt}}{\phi \cdot F_{nw} \cdot \cos(45°) \cdot L} = 0.34\ in$$

Use 0.375 in. fillet weld for both sides

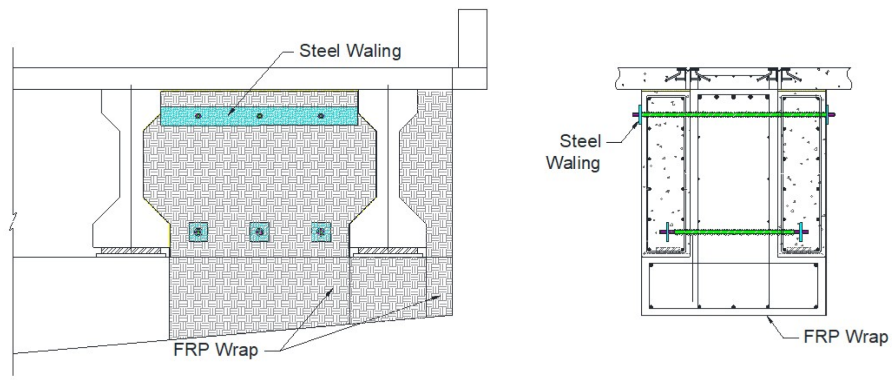
C.4 SOLUTION 17: CONCRETE INFILL WITH FULL-DEPTH FRP ANCHORED BY STEEL WALING

Design Problem:

Bent 22 only has hanger deficiency for exterior and the first interior girder based on AASHTO LRFD (2014). FRP wraps with concrete infill block between the girders will be designed to strengthen the bent cap. Following figures show the steps of installation. The gaps between the girders will be infilled by concrete with through threadbar. FRP wraps will be attached on the surface between the girders. Steel walings will be installed at the termination region of the FRP wraps to provide anchorage.



(a) Infill concrete with through threadbar



(b) Attach FRP wrap with steel waling

The required load demands on the exterior and interior ledges are

$V_{u\_ext} := 207\ kip$ (for single ledge)

$V_{u\_int} := 235\ kip$ (for single ledge)

FRP Wrap Design

1. Determine the deficiencies for single ledge

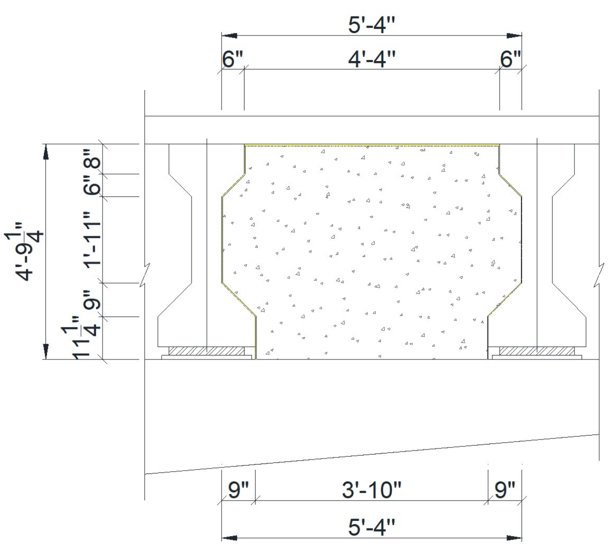
$\phi := 0.9$ *Strength reduction factor*

$V_{hd\_ext} := \dfrac{V_{u\_ext}}{\phi} - V_{nh\_ext} = 16.06\ kip$ *Hanger deficiency of exterior girder*

$V_{hd\_int} := \dfrac{V_{u\_int}}{\phi} - V_{nh\_int1} = 33.7\ kip$ *Hanger deficiency of first interior girder*

2. Determine the factored self-weight of the infill concrete blocks

Dimensions of infill concrete block



Half weight of an infill concrete block will be distributed to each girder.

$w_c := 0.015\ \dfrac{kip}{ft^3}$ *Unit self-weight of reinforced concrete*

$Vol_c := 39.8\ ft^3$ *Volume of single infill concrete block*

$W_c := 1.25 \cdot 0.5 \cdot w_c \cdot Vol_c = 0.37\ kip$ *Factored self-weight of infill concrete block*

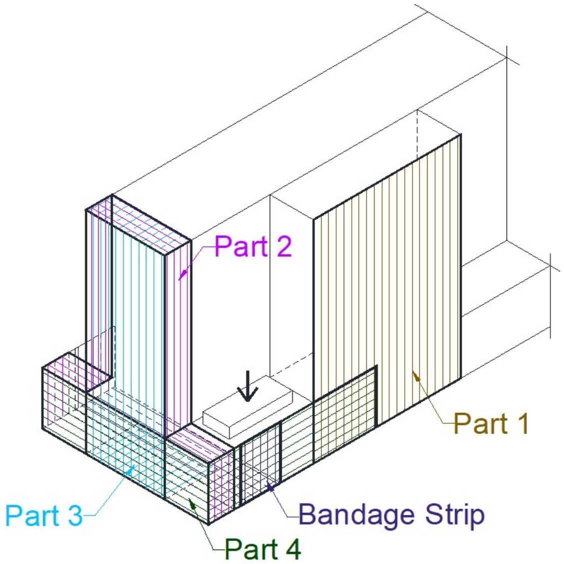
3. Re-calculate the deficiencies for single ledge

$V_{hd\_ext} := V_{hd\_ext} + \dfrac{W_c}{\phi} = 16.47\ kip$ *Hanger deficiency of exterior girder*

$V_{hd\_int} := V_{hd\_int} + \dfrac{W_c}{\phi} = 34.16\ kip$ *Hanger deficiency of out-most interior girder*

4. Determine required FRP strength

The following FRP wrapping scheme for the exterior region is considered in this example.



Part 1 - U-wrap attached on the infill concrete block with steel waling. Contribute strength to hanger, ledge and punching shear.

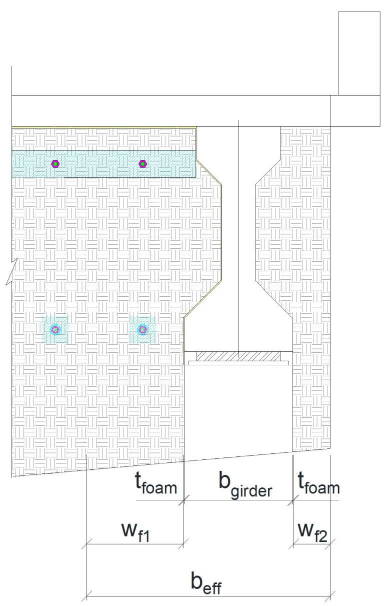
Part 2 - Attached to the end of the bent cap. Enclose entire inverted-T section. Contribute strength to hanger, ledge and punching shear.

Part 3 - Attached on the end surface of the bent cap. Vertically wrapping the web of the bent cap. Contribute strength to hanger. (End region anchorage is recommended. May use bandage strip or mechanical/FRP anchors)

Part 4 - Attached on the end surface of the bent cap. Horizontally wrapping the flange of the bent cap. Assume no strength contribution.

4.1 FRP Part 1 and Part 2

- Determine effective width of FRP wraps within the distribution width of each term of capacities



FRP wraps will be attached from the edge of the girders. Therefore, effective width of FRP wraps can be calculated by subtracting bottom width of the girder and the thickness of debonding foam sheet from the distribution width of each term of capacities.

Assume the thickness of the debonding foam sheet is 0.5 in.

$t_{foam} := 0.5\ in$ *Thickness of debonding foam sheet*

$b_{girder} := 26\ in$ *Bottom width of the girders*

Effective width of FRP can be calculated as

$w_f := b_{eff} - b_{girder} - 2 \cdot t_{foam}$

- Distribution width of each term of capacities

$C := 22\ in$ *Distance from center of bearing pad to end of bent cap*

$$b_{h\_ext} := \frac{W + 2\ d_{f\_ext}}{2} + C = 51.25\ in$$ *Distribution width of exterior girder for hanger*

$b_{h\_int} := S = 72\ in$ *Distribution width of first interior girder for hanger*

- Effective width of FRP wraps

$w_{fh\_ext} := b_{h\_ext} - b_{girder} - 2 \cdot t_{foam} = 24.25\ in$ *Effective FRP width for exterior girder*

$w_{fh\_int} := \dfrac{\left(b_{h\_int} - b_{girder} - 2 \cdot t_{foam}\right)}{2} = 22.5\ in$ *Effective FRP width for first interior girder*

- Effective strain of FRP wraps

ACI 440.2R - 08 recommends a bond-reduction coefficient to calculate effective strain of FRP for the FRP systems that do not enclose the entire section. However, higher effective strain can be used for the system that mechanical anchorages used at termination region but should not exceed 0.004. For this solution, mechanical anchorages will be provided at termination point. Therefore, take FRP effective strain as

$\varepsilon_{fe\_1} := 0.004 \quad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.85$ *Reduction factor for U-wrap*

- Strength contribution of FRP wrap

$V_f := \psi_f \cdot w_f \cdot \varepsilon_{fe} \cdot E_f \cdot t_f$ *$E_f \cdot t_f$ is the tensile modulus per unit width of FRP*

$V_{fh\_ext} := \psi_f \cdot w_{fh\_ext} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$ *Strength contribution of Part 1 and 2 to exterior hanger capacity*

$V_{fh\_int} := \psi_f \cdot w_{fh\_int} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$ *Strength contribution of Part 1 to out-most interior hanger capacity*

4.2 FRP Part 3

- Effective width of FRP wrap

Use same width as the web of the bent cap

$w_{fh\_end} := 30\ in$

- Effective strain of FRP wraps

Assume end region anchorage (or bandage) will be provided to FRP Part 3

$\varepsilon_{fe\_2} := 0.004 \quad (<= 0.75 \cdot \varepsilon_{fu})$ *(ACI 440.2R-08 Eq.11-6(a))*

- Reduction factor for FRP based on wrapping schemes

$\psi_f := 0.95$ *Reduction factor for U-wrap*

- Strength contribution of FRP wrap

$V_{fh\_2} := \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2} \cdot E_f \cdot t_f$ *Strength contribution of Part 3 to exterior hanger capacity*

5. Determine required tensile modulus per unit width ($E_{unit} := E_f \cdot t_f$) of FRP wraps

- Required FRP tensile modulus (per unit)

$E_{unit} = E_f \cdot t_f$

- Exterior hanger

$V_{fh} := V_{fh\_ext} + V_{fh\_2} = \psi_f \cdot w_{fh\_ext} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f + \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2} \cdot E_f \cdot t_f$

Set $V_{fh} = V_{hd\_ext}$

$$E_{unit\_h} := \frac{V_{hd\_ext}}{\psi_f \cdot w_{fh\_ext} \cdot \varepsilon_{fe\_1} + \psi_f \cdot w_{fh\_end} \cdot \varepsilon_{fe\_2}} = 79.9 \ \frac{kip}{in}$$

- First interior hanger

$V_{fh\_int} := \psi_f \cdot w_{fh\_int} \cdot \varepsilon_{fe\_1} \cdot E_f \cdot t_f$

Set $V_{fp} = V_{hd\_int}$

$$E_{unit\_p} := \frac{V_{hd\_int}}{\psi_f \cdot w_{fh\_int} \cdot \varepsilon_{fe}} = 399.5 \ \frac{kip}{in}$$

Maximum required E_{unit} is

$$E_{unit\_req} := \max\left(E_{unit\_h}, E_{unit\_p}\right) = 399.5 \ \frac{kip}{in}$$

- Select FRP products from the TxDOT provided pre-qualified FRP product list

<u>May use single layer of BASF C160.</u>

- Specified properties of FRP

$f_{fu} := 150 \ ksi$ *Ultimate tensile strength*

$E_f := 10700 \ ksi$ *Tensile modulus*

$t_f := 0.08 \ in$ *Nominal thickness*

$\varepsilon_{fu} := 0.014$ *Ultimate rupture strain*

- Check FRP strain limit

$0.75 \cdot \varepsilon_{fu} = 0.011 \quad > \quad \varepsilon_{fe} := 0.004$ (O.K.) *(ACI 440.2R-08 Eq.11-6(a))*

Reinforcement of Infill Concrete Block

1.Determine required longitudinal reinforcement

The concrete blocks not likely subject to bending moment. Therefore, this design example will provide minimum required longitudinal reinforcement

- Geometry of concrete block

$b_w := 16.5\ in$ *Width of infill concrete block*

$h_c := 56.75\ in$ *Height of infill concrete block*

$d_e := 54.25\ in$ *Bottom width of the girders*

$f'_c := 3.6\ ksi$ *Concrete strength (use same as the in-service structure)*

$f_y := 60\ ksi$ *Yield strength of reinforcement steel*

- Minimum flexure reinforcement

A_{f_min} = maximum of:

$$\frac{3\sqrt{f'_c}}{f_y} b_w \cdot d_e = 2.69\ in^2$$

$$\frac{200\ psi}{f_y} b_w \cdot d_e = 2.98\ in^2$$

$A_{f\_min} := 2.98\ in^2$ *Required minimum flexure reinforcement area*

- Maximum spacing of longitudinal reinforcement

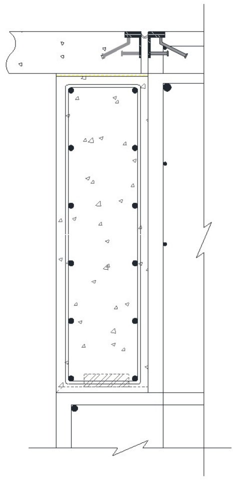
$s_{f\_max} := 12\ in$

$n_{re} := \frac{h_c}{s_{f\_max}} + 1 = 5.73$ *Required number of longitudinal bars on each side of infill concrete block*

$n_{re} := 6$ *Take integer*

$n_f := n_{re} \cdot 2 = 12$ *Required number of longitudinal reinforcement*

$A_{fs\_re} := \frac{A_{f\_min}}{n_f} = 0.25\ in^2$ *Required area of single longitudinal bar*



Place #5 ($A_{fs} := 0.3\ in^2$) longitudinal bar at four corners of the concrete block, and evenly place four #5 longitudinal bars along the height of the concrete block on each side.

2. Determine required shear reinforcement

- Use double leg #4 stirrup

$A_v := 2 \cdot 0.2\ in^2 = 0.4\ in^2$

- Concrete shear strength

$V_c := 2 \cdot \sqrt{f'_c} \cdot b_w \cdot d_e = 107\ kip$

- Required spacing of stirrups

$$s_{v\_req} := \frac{A_v \cdot f_y \cdot d_e}{V_{hd\_ext} - V_c} = 651\ in$$

- Check maximum spacing of stirrups

s_{v_max} = maximum of:

$$\frac{d_e}{2} = 27.13\ in$$

$$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt{f'_c}\ b_w} = 32\ in$$

$$\frac{A_v \cdot f_y}{50\ b_w} = 29\ in$$

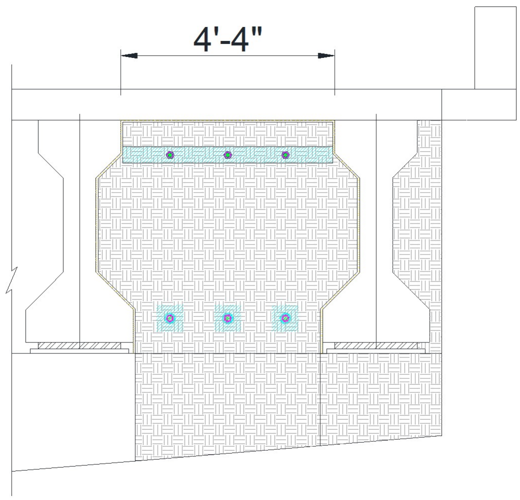
$s_{v\_max} := 27.13\ in$

Consider the geometry of the concrete infill block, evenly place four #4 double leg stirrups at a spacing of 12 in.

Threadbar and Waling Design

1. Threadbar Design

$V_{d\_ext} := \max\left(V_{hd\_ext}, V_{hd\_int}\right) = 34.16\ kip$ *Maximum deficiency*



May need at least three threadbars to provide uniform fixture to a 68 in. wide FRP wrap.

Try to use three threadbars.

- Shear demand for single threadbar

$$V_{u\_single} := \frac{V_{d\_ext}}{3} = 11.39\ kip$$

- Shear capacity of single threadbar

$$V_{n\_single} := 0.6 \cdot f_u \cdot A_n$$

Use B7 Grade threadbar

$f_u := 125\ ksi$ *Tensile strength of B7 Grade threadbar*

$A_{n\_req} := \dfrac{V_{u\_single}}{0.6 \cdot f_u} = 0.15\ in^2$ *Required nominal area of single threadbar*

<u>Use 5/8 in. diameter B7 threadbar. ($A_{n\_single} := 0.226\ in^2$)</u>

- Minimum spacing and edge distance

$d_b := \dfrac{5}{8}\ in$ *Diameter of threadbar*

$s_{t\_min} := 6\ d_b = 3.75\ in$ *Minimum spacing*

$s_{te\_min} := 6\ d_b = 3.75\ in$ *Minimum edge distance*

Use of 6 in. edge distance from the top of the concrete block will provide the most effective end anchorage to FRP wraps. However, to avoid the flexure reinforcement provided at the top of the stem, the through threadbars need to be at least 8 in. away from the top of the concrete block.

For the constructability, use 12 in. side edge distance.

<u>Evenly space three 5/8 in. diameter B7 Grade threadbars at 14 in. with the edge distances of 12 in. and 8.5 in. from the side and top, respectively.</u>

2. Waling Design

Use A36 Grade steel

$F_y := 36\ ksi$

- Required thickness for shear bearing

$\phi := 0.75$ *(AISC Specification J3.10)*

$$t_{req} := \frac{V_{u\_single}}{\phi \cdot 2.0 \cdot d_b \cdot F_y} = 0.34\ in$$

- Required bearing area

(AISC Specification J8)

$\phi_c := 0.65$

$$A_{b\_req} := \frac{V_{d\_ext}}{\phi_c \cdot 0.85 \cdot f'_c} = 54.8\ in^2$$

<u>Use 4" x 68" ($A_b := 272\ in^2$) with 0.375 in. thickness continuous steel waling.</u>