Development of Design Procedures for Concrete Adhesive Anchors

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16. Abstract

A research program was initiated to provide simplified design guidance for adhesive anchor use on WisDOT projects, commensurate with the current WisDOT approved products. The research program consisted of a literature review, written survey of state DOTs, laboratory testing of wingwalls simulating an upper wingwall replacement, review of WisDOT policy, and recommendations based on findings. The recommendations developed from the research program are WisDOT should allow adhesive anchors in parapets with strict adherence to manufacturer installation instructions and design procedures following AASHTO and ACI with AASHTO designed anchor capacity limited by anchor spacing, allow adhesive anchor use in sustained tensile loading applications, allow adhesive anchors in overhead or upwardly inclined installation applications, allow alternative design approaches to utilize high bond strength products, change installation procedure requirements in the WisDOT Standard Specification to follow the manufacturers published installation instructions, allow adhesive anchors in abutment wingwall replacement, and allow adhesive anchors for concrete parapet replacement. Further research is recommended to investigate adhesive anchor performance in structural members compared to code design equations and the effect of reinforcement bar coatings on the bond strength of adhesives.

17. Key Words
Adhesive anchor, abutment, sustained load, bond strength, concrete breakout, creep, wingwall, upwardly inclined, overhead, concrete cracking, critical edge distance

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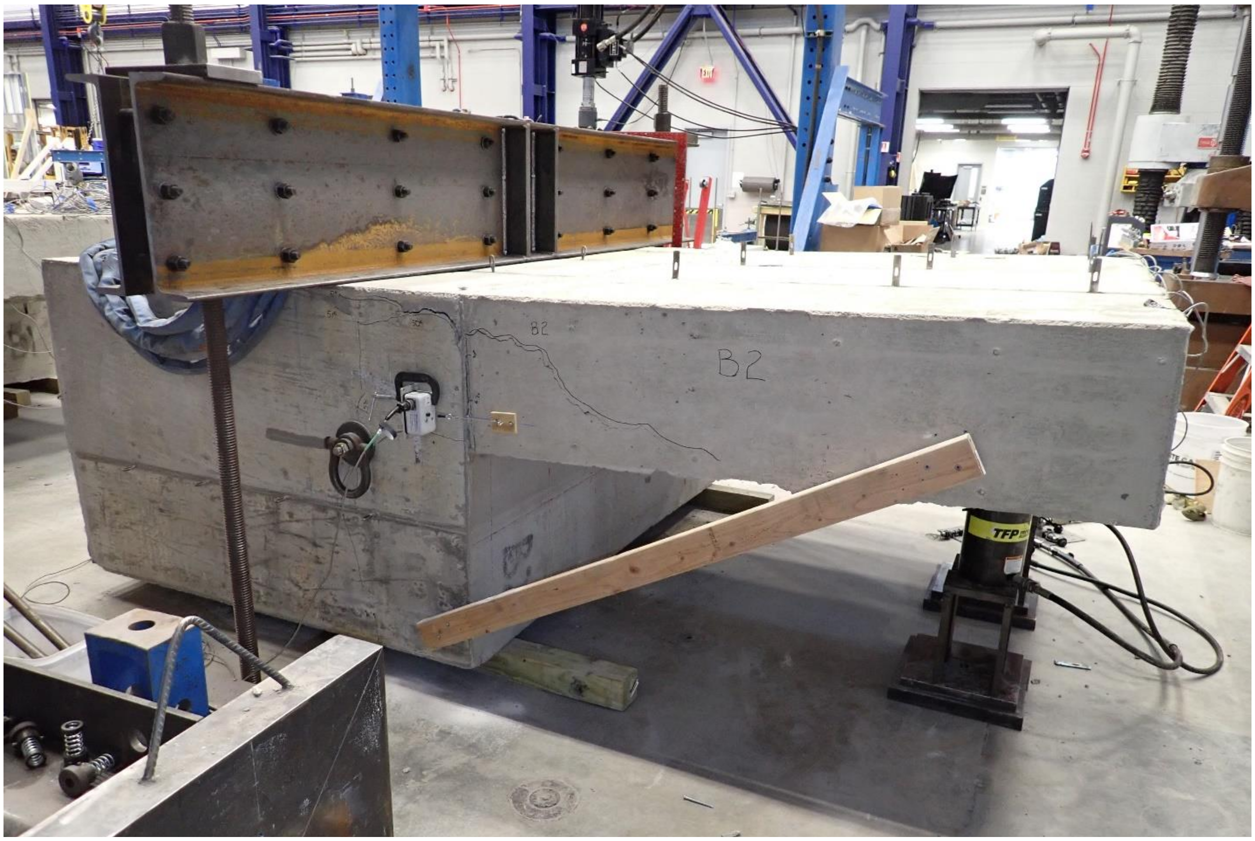

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EXECUTIVE SUMMARY

Concrete adhesive anchors are used in new and retrofitted transportation structures by Wisconsin and other state departments of transportation (DOT). Typical applications include pedestrian railing or fencing attachment and abutment and pier extension and replacement. The anchors can be reinforcing bars or threaded rods installed in holes drilled in concrete and anchored with a polymeric adhesive. The use of adhesive anchors for sustained loads in overhead applications has been prohibited by many state DOTs, including WisDOT, after the collapse of concrete panels from the ceiling of the Boston Tunnel (the Big Dig Tunnel) in 2006 and subsequent moratoria by the National Transportation Safety Board (NTSB) [1] and the Federal Highway Administration (FHWA) [2]. The NTSB and FHWA investigations on the panel collapse in the Boston Tunnel concluded that the main cause was insufficient creep resistance, i.e., the strength of adhesive anchors under sustained loads [1] [2]. Another possible cause was due to poor installation of the anchors, partially due to the lack of stringent quality assurance and control requirements for adhesive anchors at the time [3].

This research program was initiated to provide simplified design guidance for adhesive anchor use on WisDOT projects, commensurate with the current WisDOT approved products. Currently, such applications include backwall, paving block and wing replacements (upper and lower sections), as well as abutment and pier extensions. The research program also resulted in providing design guidance and examples for adhesive anchors used for concrete parapet replacement on WisDOT projects, which is currently not allowed for wingwall replacement and for abutment extension.

The research program consisted of a literature review, written survey of state DOTs, design examples for adhesive anchors in three different applications, laboratory testing of wingwalls simulating an upper wingwall replacement, review of WisDOT policy, and recommendations based on findings.

The main findings of the literature review are focused on code design procedures for adhesive anchors, effect of impact loadings on adhesive anchors, effect of corrosion protection coating (epoxy) on adhesive anchors, and state DOT policies on characteristic bond stresses.

Design of adhesive anchors is covered in Chapter 17 of ACI 318-19 and described in AASHTO LRFD 9th Edition (2020), Section 5.13. This AASHTO section specifies that adhesive anchors are to meet the criteria of ACI 355.4 (2011) and be designed, detailed, and installed using the provisions of ACI 318-14, Chapter 17, except for two modifications regarding adhesive anchors under impact loading and sustained tension.

ACI 318, both the 2014 version referenced by AASHTO LRFD and the current 2019 version, states that the design procedure does not apply for impact-load conditions. However, AASHTO LRFD states that the design procedure can apply for evaluating strength under impact loading provided that the anchors have an impact strength equal to or greater than their static strength, as shown by either testing or a combination of testing and analysis. Sustained tensile loading is addressed in both ACI 318 and AASHTO by including a sustained load factor. State DOTs specify if and when adhesive anchors can be used in sustained load applications.

Few adhesive manufacturers have information regarding bond strength of coated reinforcing bars. Those manufacturers that do have test information recommend a 15 percent reduction in bond strength. Other manufacturers recommend using development length factors for epoxy coated bars listed in ACI 318.

The characteristic bond stress, τ, of adhesive anchoring material is used to calculate the basic bond strength, N_{ba}, and the critical distance, c_{Na}. The characteristic bond strength is a property of the adhesive product that represents both the inherent material and the installation and use conditions. A variety of factors affect the characteristic bond strength, including presence of concrete cracking, anchor size, drilling method, degree of concrete saturation at the time of hole drilling and anchor installation, concrete temperature and age at time of installation, peak concrete temperatures, chemical exposure from the environment during anchor service life, and the type and duration of loading [4].

ACI 318 provides lower-bound default values for anchors meeting the qualification requirements of ACI 355.4 (see ACI 318 Table 17.6.5.2.5) where the product-specific characteristic bond stress is not known. These default values are much smaller than those of products in WisDOT approved product list, and thus, using them would generally result in designs that are too conservative.

Testing was conducted to simulate an upper wingwall replacement on top of an existing lower wingwall to determine performance of epoxy coated reinforcing bars adhesively anchored into the lower wingwall and cast into the upper wingwall. A total of two test samples were fabricated to represent an upper wingwall replacement on a lower wingwall. The performance of both walls was similar in that the ultimate load values and cold-joint opening displacement and load-reinforcing bar strain characteristics were similar. The maximum load achieved for Wall B1 was 49,700 lbf and for Wall B2 was 50,610 lbf. The failure mode for both wingwall samples was yielding of the tension reinforcing steel (back face reinforcing) and concrete cracking/crushing. Calculations were made based on ACI design equations to determine the anticipated failure mode of the adhesively anchored reinforcement in the lower wingwall. The failure modes considered were concrete breakout, reinforcing steel yield and fracture, and adhesive bond failure. The controlling calculated design strength failure mode was concrete breakout. Based on a calculated design concrete breakout failure, the anticipated test load was 23,700-lbf at a loading distance of 48-in from the wingwall joint. The ACI design equations for concrete breakout are a lower bound based on a 5 percent fractile of a normal distribution of large database set of test results. The ultimate concrete breakout capacity was determined to be approximately 34,500 lbf. The capacity of a conventionally reinforced upper wingwall cast against a lower wingwall using concrete strengths achieved for the test walls and the steel yield strength reported on the mill certificate was determined. A test load of 33,600 lbf would be expected based on the flexural yield capacity of a conventionally reinforced wingwall. A test load of approximately 52,100 lbf would be expected for ultimate strength of steel. The tests of the wingwalls indicated the ACI design procedure results in a design that is conservative for this application.

Based on the findings of the research program, the following recommendations are made:

- Allow adhesive anchors in parapets with strict adherence to manufacturer printed installation instructions and design procedures following AASHTO and ACI with AASHTO designed anchor capacity limited by anchor spacing. Dynamic increase factors (DIF) may be considered when determining strength of adhesive anchors under impact loadings, but further research is needed to determine DIF values for anchor bond strengths.
- Allow use of adhesive anchors in sustained tensile loading applications in accordance with AASHTO LRFD-9 for potential cost-saving applications.

- Allow adhesive anchors in overhead or upwardly inclined installations with the requirement that the installation be performed by an ACI certified adhesive anchor installer, continuous inspection of installation is performed, and proof load a percentage of installed anchors.
- Consider alternative design approaches to utilize products with high bond strengths. In lieu of an alternative design approach, it is recommended that the WisDOT current minimum characteristic bond stress table (Table 40.16-1) be accompanied by a list of assumptions used to compile it including strength reduction factors, temperature range and type of inspection (periodic or continuous).
- Remove the WisDOT Standard Specification Section 502.3.12 that states the drilled holes are to be cleaned by flushing with water followed with air blow until the hole is dry and dust-free. This section should be replaced with a statement that adhesive anchor installation shall strictly follow the manufacturer's printed installation instructions (MPII).
- Allow adhesive anchors in abutment wingwall replacement.
- Allow adhesive anchors for replacement of concrete parapet that is to be connected to an existing concrete bridge deck.

Based on the findings of the research program, the following are suggestions for further research:

- Validate the design procedures proposed in the current research for concrete parapet replacement with full scale testing. The tests performed for this research program indicated that current ACI design procedures for adhesive anchors can be applied for wing wall replacement with reasonable conservatism. However, the level of conservatism may or may not be the same for other applications such as concrete parapet replacement.
- Further investigate the use of adhesive anchors for abutment and/or pier extension. The design example provided in this research indicated that the design strengths of abutment sections using adhesive anchors are much lower than the original design strength assuming fully developed reinforcement. However, this does not necessarily preclude potential values of adhesive anchors in this application for several reasons. First, the design equations may be over-conservative. Second, the required strength of the abutment section, which is not well understood, could be significantly lower than the original design strength and may be potentially met by the strength of the rehabilitated abutment section. Improved understanding of the design loads and required strength of abutment would be beneficial in evaluating potential value of using adhesive anchors.
- Investigate the effect of reinforcement bar coatings on the bond strength of adhesives. Bond strength factors for reinforcement coatings are currently not addressed in code approval testing or code design equations.
- Investigate effect of impact loadings on anchor bond strengths. While literature indicates that a DIF can be applied to calculating anchor strengths associated with concrete and steel failures under impact/dynamic loads, there is little information on effect of impact/dynamic loads on anchor bond strengths.

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CHAPTER 1. INTRODUCTION

Concrete adhesive anchors are used in new and retrofitted transportation structures by Wisconsin and other state departments of transportation (DOT). Typical applications include pedestrian railing or fencing attachment, and abutment and pier extension and replacement. The anchors can be reinforcing bars or threaded rods installed in holes drilled in concrete and anchored with a polymeric adhesive. The use of adhesive anchors for sustained loads in overhead applications has been prohibited by many state DOTs, including WisDOT, after the collapse of concrete panels from the ceiling of the Boston Tunnel (the Big Dig Tunnel) in 2006 and subsequent moratoria by the National Transportation Safety Board (NTSB) [1] and the Federal Highway Administration (FHWA) [2]. The NTSB and FHWA investigations on the panel collapse in the Boston Tunnel concluded that the main cause was insufficient creep resistance, i.e., the strength of adhesive anchors under sustained loads [1] [2]. Another possible cause was due to poor installation of the anchors, partially due to the lack of stringent quality assurance and control requirements for adhesive anchors at the time [3].

Some state DOTs further prohibited adhesive anchors in non-sustained loading applications where failures of the anchors may endanger the public. Currently, WisDOT does not permit the use of adhesive anchors for crashworthy bridge parapet/railing (WisDOT Bridge Manual [5]). The use of mechanical anchors is currently restricted by WisDOT due to a variety of issues including anchor installation, design requirements that are more restrictive than adhesive anchors, and the collection of corrosive elements within the anchor hole.

Without the option of using post-installed anchors, casting of new concrete would be required with a significant portion of the edge of deck or slab to be removed to obtain sufficient development for both the existing deck and new parapet reinforcing bars. This would result in significantly higher costs. For design of adhesive anchors, WisDOT adopted the procedure in ACI 318-14 Chapter 17 [4] which is also referenced by a recently added section 5.13 of the AASHTO Specifications [6]. This procedure allows determination of tensile and shear strengths of anchors.

Since the Boston Tunnel collapse, extensive research has been conducted on performance of adhesive anchors resulting in significant improvements in reliability, quality control, testing protocols, installation procedures, and design and installation training. Design of adhesive anchors has been developed and was first incorporated into ACI 318 code in 2011. As indicated above, the AASHTO Bridge Design code has incorporated ACI 318-14 (Chapter 17), and many state DOTs have adopted the ACI design procedure. Many state DOTs also require that adhesive anchors be installed by or under supervision of an ACI certified installer, and field proof tests be conducted in accordance with ASTM International (ASTM) E3121.

The research for this report was conducted to provide simplified design guidance for adhesive anchor use on WisDOT projects, commensurate with the current WisDOT approved products, that include wingwall replacement, abutment extension, and temporary barrier installation. The research also provides design guidance for adhesive anchor use for concrete parapet replacement on WisDOT projects, which is currently not allowed. The design guidance is based on literature review, survey of state DOTs, assessment of WisDOT policies, and laboratory testing. Further research is suggested based on test results and current design equation comparisons.

CHAPTER 2. SUMMARY OF LITERATURE REVIEW AND DOT SURVEY

2.1. Summary of Literature Review

The main findings from the literature review are summarized in this section. The topics are listed followed by a discussion of the findings. Detailed results of the literature review are provided in Appendix B.

- Design procedure for adhesive anchors
- Effect of impact loadings on adhesive anchors
- Traffic parapets with adhesive anchors
- Effect of epoxy coating on adhesive anchor performance
- State DOT policies on characteristic bond stresses

2.1.1. Design Procedures for Adhesive Anchors

2.1.1.1. ACI Design Procedure

The design of adhesive anchors is covered in Chapter 17 of ACI 318-19. Six primary failure modes are to be considered, for which the design strength requirements are shown in Table 2.1. These failure modes include steel in tension, concrete breakout in tension, bond in tension, steel in shear, concrete breakout in shear and concrete pryout in shear. For adhesive anchors subject to sustained tension, an additional requirement is that the sustained tension shall not exceed 0.55 times the design bond strength of anchor. Splitting failure of the concrete is addressed by meeting minimum geometrical requirements for anchor edge distances, anchor spacing, and concrete member thickness. Supplementary reinforcement may be used to control splitting. If the anchors are subject to both tension and shear, interaction effects may need to be considered depending on the ratios between the factored loads and the governing strengths, $N_{ua}/\Phi N_n$ for tension and $V_{ua}/\Phi V_n$ for shear. If the ratio exceeds 20 percent for both shear and tensile loading, then the sum of the ratios must not exceed 1.2, per ACI 318 Section 17.8.

Table 2.1. Design strength requirements of adhesive anchors per ACI 318 Table 17.5.2 and Section 17.5.2.2 [4]

Failure Mode	**Single Anchor**	**Anchor Group**[1]	
		Individual Anchor in a Group	**Anchors as a Group**
Steel strength in tension	$\Phi N_{sa} \geq N_{ua}$	$\Phi N_{sa} \geq N_{ua,i}$	--
Concrete breakout strength in tension	$\Phi N_{cb} \geq N_{ua}$	--	$\Phi N_{cbg} \geq N_{ua,g}$
Bond strength of adhesive anchor in tension	$\Phi N_{ag} \geq N_{ua}$	--	$\Phi N_{ag} \geq N_{ua,g}$
Bond strength of adhesive anchor in sustained tension	$0.55\Phi N_{ba} \geq N_{ua,s}$	$0.55\Phi N_{ba} \geq N_{ua,s,i}$	--
Steel strength in shear	$\Phi V_{sa} \geq V_{ua}$	$\Phi V_{sa} \geq V_{ua,i}$	--
Concrete breakout strength in shear	$\Phi V_{cb} \geq V_{ua}$	--	$\Phi V_{cbg} \geq V_{ua,g}$
Concrete pryout strength in shear	$\Phi V_{cp} \geq V_{ua}$	--	$\Phi V_{cpg} \geq V_{ua,g}$

Notes: [1]*Design strengths for steel and sustained tension failure modes shall be calculated for the most highly stressed anchor in the group.*

The strength reduction factor Φ used in each failure mode is defined by Tables 17.5.3(a), (b), and (c) in ACI 318.

2.1.1.2. AASHTO LRFD Amendments to ACI 318 Design Procedure For Adhesive Anchors

Design of adhesive anchors is described in AASHTO LRFD 9th Edition (2020) (AASHTO-20), Section 5.13. This section specifies that adhesive anchors are to meet the criteria of ACI 355.4 (2011) (test program performed by an independent and accredited test agency), and be designed, detailed, and installed using the provisions of ACI 318-14, Chapter 17, except for two modifications regarding adhesive anchors under impact loading and sustained tension as discussed below.

1. **Impact Loading.**

ACI 318-14, the version referenced by AASHTO-20, and ACI 318-19, states that the ACI design procedures do not apply to impact load conditions. However, AASHTO LRFD 2014 states that the exclusion for impact loads need not apply to post-installed anchors provided that the anchors have an impact strength equal to or greater than their static strength, as shown by either testing or a combination of testing and analysis. This deviation from ACI is based on several references including ACI 349-13 - *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, and two studies by Dickey et al. [7] and Braimah et al. [8]. AASHTO-20, Section 5.13.1 further states that the use of any documented impact resistance is permitted to be applied at the Owner's (person or agency having jurisdiction over the bridge) option but is not required.

2. **Sustained Tension.**

Per ACI 318, sustained tensile load shall not exceed 0.55 times the factored bond strength ($0.55\phi N_{ba}$) as mentioned above and shown in Table 2.1. The 0.55-factor is based on the requirements of ACI 355.4, which evaluates product behavior under sustained tensile loading conditions designed to represent a 50-year duration at a standard temperature of 70°F and a 10-year duration at an elevated temperature of 110°F. AASHTO-20 modifies the 0.55-factor by specifying a 0.50-factor based on recommendations by Cook et al. (2013) for structures with a 100-year life at 70°F or a 20-year life at an elevated temperature of 110°F.

In addition, AASHTO-20 only requires strength under sustained tensile loading to be assessed (i.e., the 0.50 factor to be applied) when "significant sustained tensile loads" are present. AASHTO-20 defines a "significant" sustained tensile load as with an unfactored magnitude exceeding 10 percent of the ultimate capacity of the anchor or anchor group. In contrast, ACI 318-19 requires the 0.55 strength-reduction factor to be considered regardless of the sustained tension load magnitude.

2.1.2. Effect of impact loadings

As mentioned above, ACI 318-19 does not allow using the design procedure in Chapter 17 for impact loadings while AASHTO LRFD-9 removed this exclusion provided that the anchors have an impact strength equal to or greater than their static strength. Since this topic is important for applications such as traffic parapets on a bridge deck, further discussion is provided in this section.

The literature indicated that strength of adhesive anchors under impact loading can meet or exceed that under static loading. Solomos and Berra [9] performed testing of three types of anchors (adhesive anchor Hilti HVZ, undercut anchor HAD, and cast-in-place headed stud) under dynamic and static tensile loadings in uncracked concrete and found that in all cases, concrete breakout strength of the anchor under dynamic loading was higher than that under static loading for the same condition. The researchers attributed this dynamic effect to the increased strength of concrete and suggested a Dynamic Increase Factor (DIF) of 1.25 for all of the three types of anchors.

Braimah et al. [8] studied the dynamic behavior of threaded rod anchors embedded in concrete with an epoxy-based adhesive under impulse type loading. The research found that the DIF varied from 1.2 to 3.2 depending on the anchor diameter, embedment depth, embedment angle, and concrete strength. The DIF was higher for shallower embedment lengths where concrete breakout was observed compared to anchors with deeper embedment where steel failure was dominant. The difference in DIF was attributed to the greater increase in concrete strength versus smaller increases in steel strength under impact loading. The researchers proposed a DIF of 1.2 for design purposes.

In a study sponsored by WisDOT in 2012, Dickey et al. [7] performed a literature review on the effect of impact loadings and attempted to determine a DIF for adhesive anchors. The researchers proposed a DIF of 1.18 for steel strength in tension and shear, 1.88 for concrete breakout strength in tension and shear, and 1.40 for bond strength in tension; however, because these values were based on comparing the tested anchor strengths under dynamic loads with the analytical strengths instead of comparing the dynamic strength with tested strength under static loadings, the research did not accurately capture the effect of dynamic loading. The report indicated that comparing the tested anchor strengths under dynamic loads versus those under static loadings was not achievable due to difficulties in controlling and measuring strain rates and in controlling the failure modes. Dickey also noted that in a study sponsored by Michigan DOT in 2001, the bond stress at the concrete-epoxy interface for impact loading was 150 percent greater than that of static loading and that cold winter temperatures did not impact the dynamic bond strength of the anchors tested.

In ACI 349-13, *Code Requirements for Nuclear Safety-Related Concrete Structures*, various DIF values are specified for yield strength of steel and compressive strength of concrete depending on the grade of steel/concrete, strength type (e.g., shear or compression), and strain rate. The maximum DIF permitted by ACI 349-13 (corresponding to a strain rate of about 0.01 to 0.03 in./in./sec.) is 1.10 for a yield strength of Grade 60 reinforcing steel, 1.25 for concrete compressive strength (f'_c = 4 to 6 ksi) used for axial and flexural compression, and 1.10 for f'_c used for determining shear strength. No DIF for f'_c is specified for concrete in tension. Although strain rates in concrete and steel reinforcement are typically not measured during crash testing of traffic barriers, a previous report indicated that significant damage to the parapets occurred in very short periods of time, for example, within 0.1 second [10]. Thus, for design of traffic parapets under vehicular collision loads, the DIF values specified in ACI 349-13 noted above appear more conservative than the other reported results and, therefore, more applicable.

A thorough review of manufacturers product technical data for adhesive anchors was not performed; however, a brief review of Hilti Product Technical Guide (2011) found that the listed bond strengths for the HIT RE-500 SD adhesive may be increased by 40% for short-term loads including wind or seismic. For other Hilti adhesive products including those in the WisDOT approved product lists (HIT-RE 500 V3, HIT-HY 100, HIT-HY 200), the Hilti Product Technical Guide (2019) does not provide DIFs for bond strength, suggesting that the effect of dynamic loadings varies among products, even for those from the same manufacturer.

In addition to reviewing literature, the authors sent informal email inquiries to four adhesive anchor manufacturers (DeWALT, Simpson, ITW, and Hilti) requesting potential test data for their products under impact loadings. No manufacturer data appear to be available.

2.1.3. Use of Adhesive Anchors in Traffic Parapets

While ACI 318-19 states that the design procedure for adhesive anchors in the Code does not apply for impact loadings (AASHTO-20 does not have this restriction), many state DOTs permit the use of adhesive anchors in concrete parapets to resist vehicular impact loadings, and some have standard designs for parapet retrofit with adhesive anchors. This section discusses several examples of these designs and notable research related to performance of concrete parapets with adhesive anchors.

A study sponsored by Texas DOT in 2007 [11] evaluated a retrofit design of the TxDOT T501 (Figure 2.1) continuous concrete railing using epoxy adhesive anchors for static and dynamic loadings (using a bogie). The #5 U-shaped bars used in original railing-deck connection were replaced with a single line of #6 S-shaped bars placed near the traffic face of the railing using Hilti RE 500 Adhesive with an embedment depth of 5-1/4 inches on an 8-inch-thick concrete deck. For the interior rail segment with anchor spacing of 16 inches on center, the strengths of two retrofit railing specimens from dynamic testing were 66 to 70 kips, comparable with those of the original railing (66 and 75 kips) and exceeding the design load of 54 kips. For the end rail segment with 16-inch anchor spacing, the rail strength from dynamic testing was 46 kips, which was comparable to the strength of the original railing but was still lower than the 54-kip design load. The end rail segment design was modified with 8-inch anchor spacing and tested using static loading. The static strength of the modified design was 50 kips, slightly improved but still lower than the design load. Closer anchor spacing was recommended for the end rail segment to improve strength. In addition, the researchers recommended using adhesive anchors with a minimum embedment depth of 5-1/4 inches and a minimum deck thickness of 8 inches.

TxDOT Bridge Railing Manual (2020) has standard details for retrofitting different concrete railing types, including T221, T222, C221, T402, C402, and SSTR, using adhesive anchors. Examples for railing types SSTR and T222 (both evaluated and approved for MASH TL-4) are shown in Figure 2.2. The adhesive anchors on the front face (traffic side) of these railings are similar to those used in the study on railing TxDOT T501 discussed above, but a row of secondary adhesive anchors was added to the back face (non-traffic side). For the SSTR Rail, the adhesive anchors on the front face (#2 in Figure 2.2 for SSTR) are No. 6 bars spaced at 16 inches on center for the locations away from the end of the railing and 8 inches on center within 4 feet at each end of the rail. The adhesive anchors on the back face (#11 in Figure 2 for SSTR) are No. 4 bars spaced at a maximum of 4 feet. For the T222 Rail, the adhesive anchors on the front face (#2 in Figure 2.2 for T221, T222, and C221 in Figure 2.2) are No. 6 bars spaced at 10-1/2 inches on center for the locations away from the end of the railing and 8 inches on center within 4 feet at each end of the rail. The adhesive anchors on the back face (numbered 11 in Figure 2.2 for T222) are No. 4 bars spaced at a maximum spacing of 4 feet, the same as for SSTR Rail. In these designs, the No. 6 front face bar (numbered 2 in Figure 2.2 for T222) is embedded 5-1/4 inches into the concrete deck when the minimum deck thickness is 7 inches and is required to achieve a basic bond strength in tension, N_{ba}, of 20 kips; the No. 4 bar is embedded 4 inches and is required to achieve a basic bond strength in tension, N_{ba}, of 10 kips. It is unclear if these strengths are for cracked or for uncracked concrete but could be conservatively taken as the cracked concrete capacity. TxDOT also has standard details for temporary precast concrete barriers connected to a concrete deck with adhesive anchors in a space at the center of the barrier (Figure 2.3). The adhesive anchor is No. 8 bar embedded 5-1/4 inches into the concrete deck and spaced at 6 feet on center and is required to achieve a basic bond strength in tension, N_{ba}, of 26 kips. Stainless bars are required when the bars are to remain embedded in the bridge deck. The temporary precast barriers are 30-ft. long and pin connected to each other with two threaded rods dropped into exposed u-bars at each end.

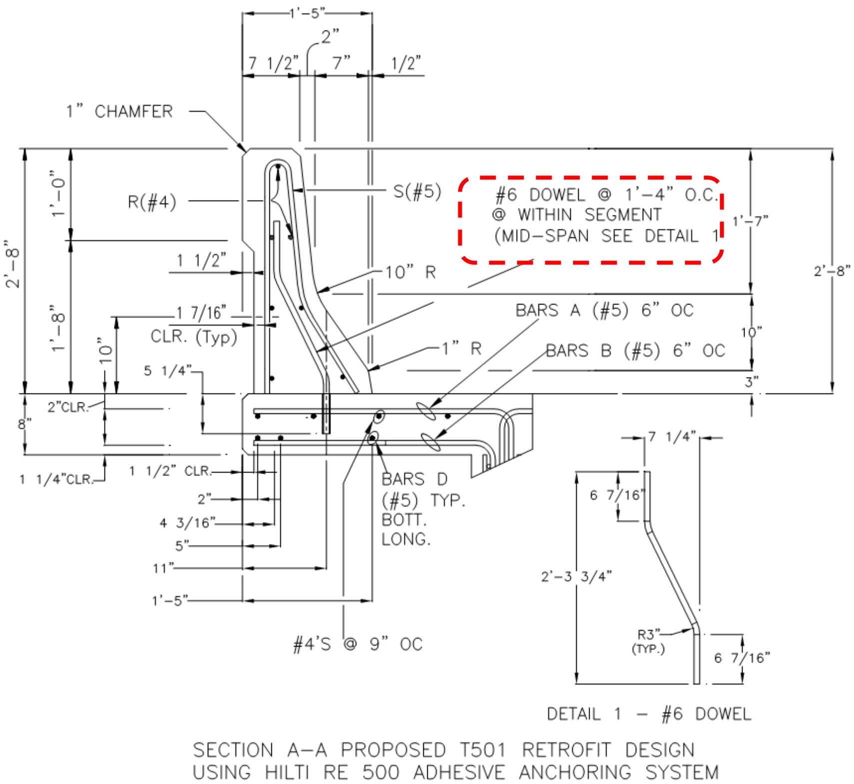

Figure 2.1 Concrete rail retrofit using adhesive anchors [7]

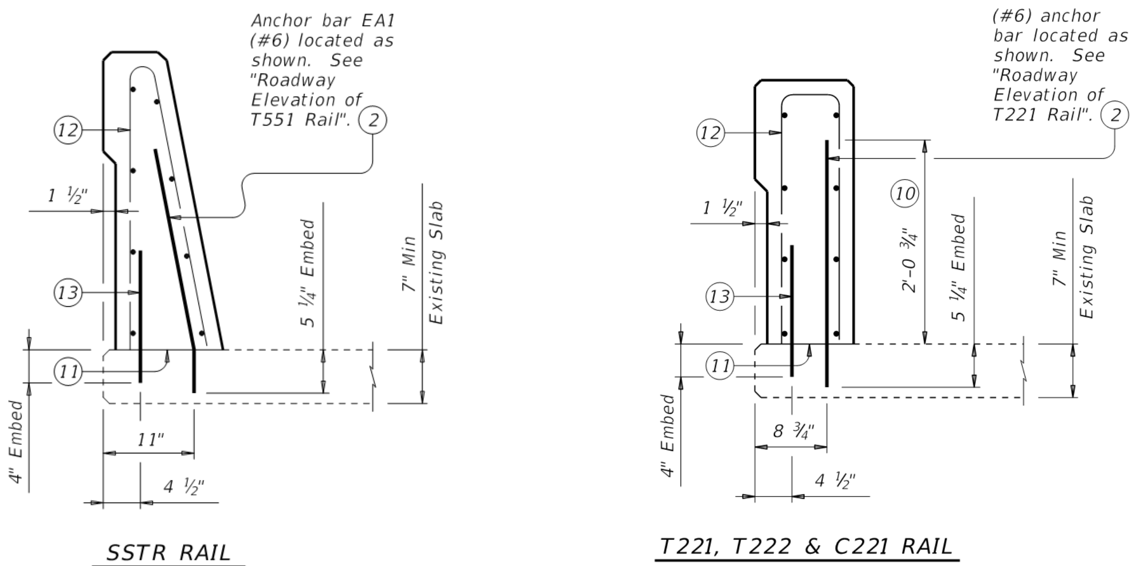

Figure 2.2. Examples of concrete railing retrofit design with adhesive anchors. TxDOT railing types SSTR and T222 are approved for MASH TL-4, 36" high.

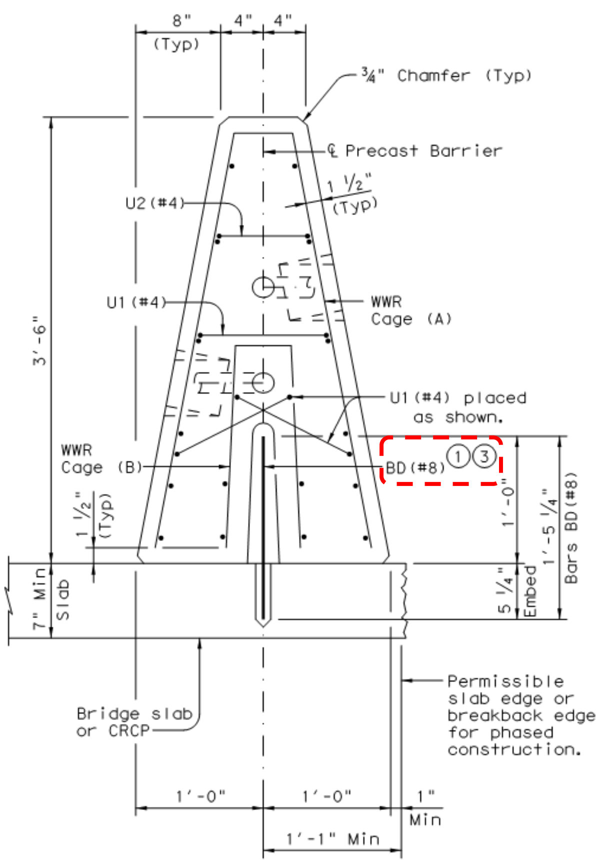

Figure 2.3. Example of temporary concrete barrier with adhesive anchors, approved for MASH TL-4 by TxDOT.

Hawaii DOT has one standard detail for 42-inch-tall aesthetic concrete bridge rail connected to a bridge deck with adhesive anchors, which is approved for MASH TL-3 (Figure 2.4). The adhesive anchors are No. 5 epoxy coated reinforcing bars spaced 6 inches on center on traffic (impact) side and 12 inches on center on the other(back) side. The embedment depth into the deck slab is 8 inches for the anchors on both sides of the barrier.

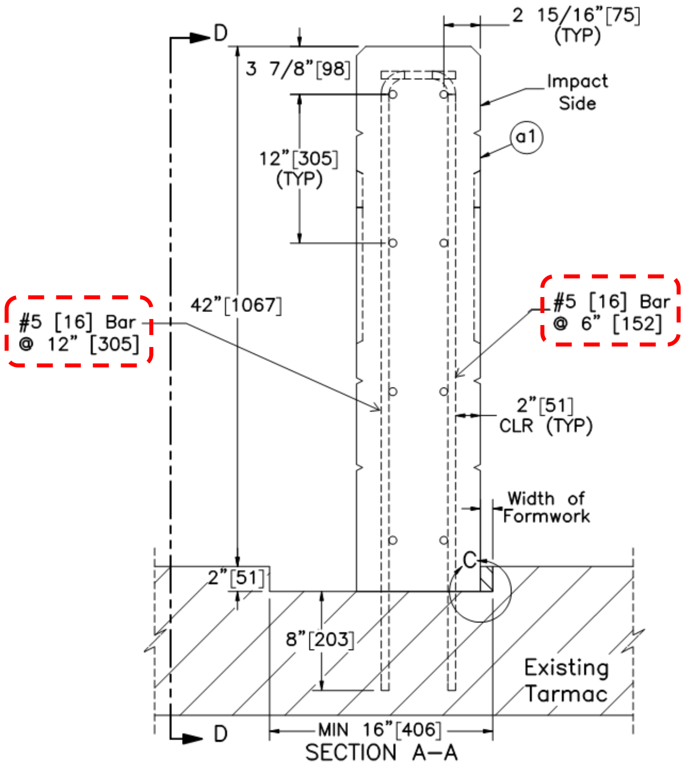

Figure 2.4. Hawaii DOT concrete rail retrofit with adhesive anchors, MASH TL-3 rating.

Caltrans has standard details for a concrete barrier retrofit. An example of concrete barrier Type 842 (42-in. tall), which is approved for MASH TL-4, is shown in Figure 2.5. The adhesive anchors on both traffic (front) side and the back side are No. 5 bars spaced at 6 inches on center and embedded 5 inches into the concrete deck. The minimum depth of concrete deck below the drilled hole is 1-1/2 inches.

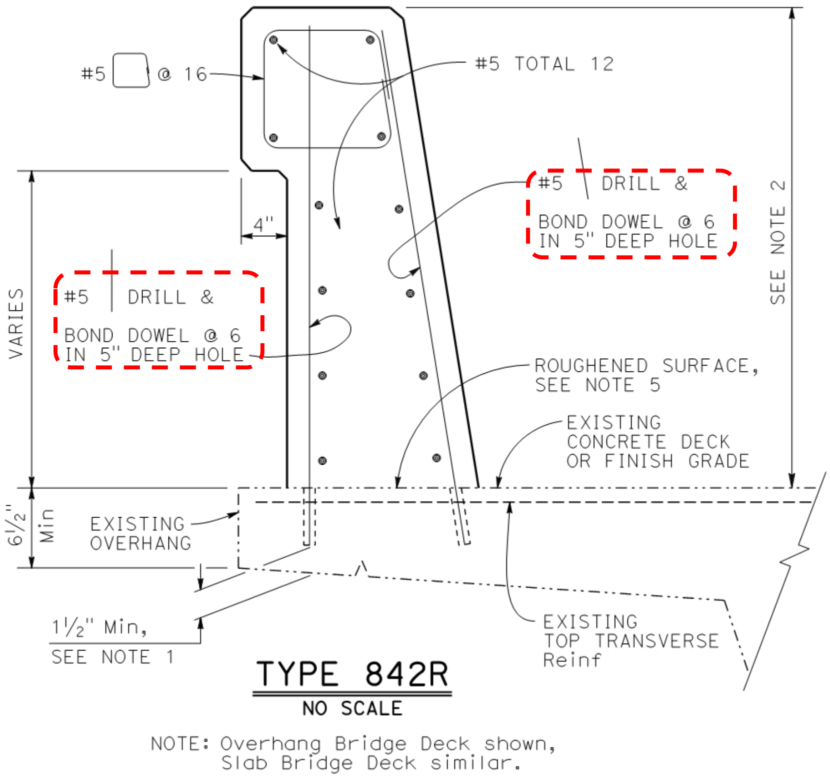

Figure 2.5. Caltrans Concrete Barrier Type 842 Retrofit, 42" high. MASH TL-4 Rating.

WisDOT has standard details for interior concrete parapet and temporary concrete barrier with adhesive anchors. In the interior parapet (Figure 2.6), the adhesive anchors on both the traffic (front) side and the other (back) side are No. 6 bars spaced at 16 inches on center for the interior portion of the parapet and 8 inches on center for a length of 4 feet at each end of the parapet (i.e., adjacent to parapet joints at abutments, expansion joints and construction joints). The embedment depth is 5-1/4 inches for all the anchors. This interior parapet design is currently used only in conjunction with crashworthy adjacent exterior parapets. In the temporary concrete barrier in Figure 2.7, which is 12 ft. 6 in. long and approved for MASH TL-3, three 1-1/8-inch diameter ASTM A307 threaded rods are adhesively anchored and embedded 5-1/4 inches into the concrete deck on the traffic side to resist vehicular impact loads. A minimum bond strength of 1,800 psi is required for the adhesive.

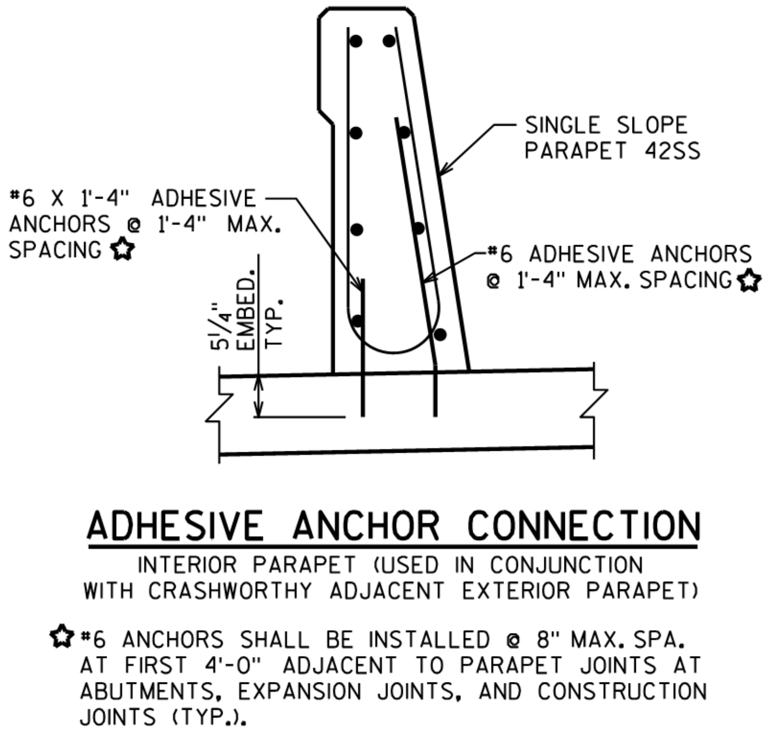

Figure 2.6. WisDOT interior concrete parapet (currently not permitted for use as crash-worthy parapet).

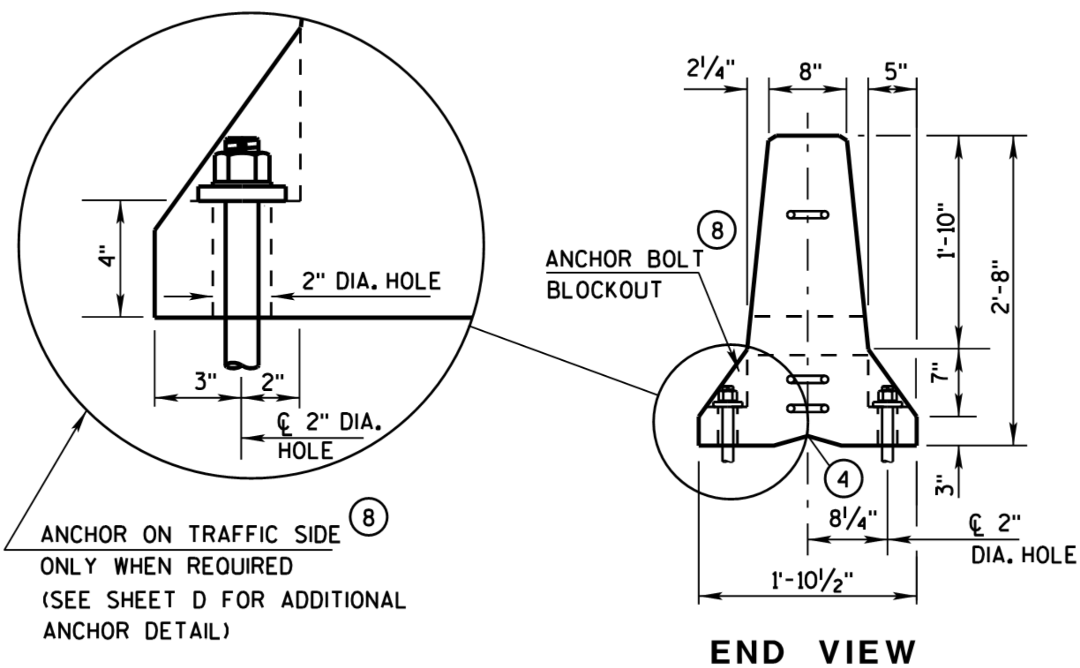

Figure 2.7. WisDOT temporary precast concrete barrier with adhesive anchors, MASH TL-3 Rating.

2.1.4. Effect of coating on adhesive anchors

Research has shown that epoxy coating on reinforcing bar adhesive anchors may reduce bond strength of the adhesive anchors (as compared with uncoated reinforcing bar), but the effect varies with adhesive products. A study performed by the Minnesota DOT [10] investigated the effect of epoxy coating reinforcing bars on anchor strengths using four adhesive products (Powers AC100+ Gold, Red Head A7+, Hilti HIT-RE 500 V3, and ATC Ultrabond 365CC) and found that the epoxy coating slightly reduced the tensile bond strength of the anchor for adhesive products from some manufacturers (up to 6%), but not for others. The researchers proposed using a reduction factor of 0.9 for bond strength of epoxy coated reinforcing bar anchor. A study sponsored by Caltrans [17] found that tested tensile strengths of epoxy coated reinforcing bar anchors were lower than manufacture-provided strengths for uncoated reinforcing bars for one of the three adhesives used. It was noted that the failure occurred at the interface between the adhesive and epoxy coating. The researchers recommended using a reduction factor for bond strength of adhesive anchors but did not indicate the value of such factor. In a WisDOT-sponsored study [7], the researchers proposed using a factor of 0.9 for bond strength of epoxy coated reinforcing bar anchor.

Several DOTs have limitations on the use of epoxy coated reinforcing bars as adhesive anchors. MnDOT issued a memo in 2017 which specified limitations on adhesive anchors with epoxy coated reinforcing bars. Epoxy coated reinforcing bars are only allowed when they resist little or no tension, e.g., reinforcing bars near the back face of a concrete barrier. For applications where the reinforcing bar anchors are to resist tension, stainless steel or uncoated reinforcing bars are required, depending on exposure condition and whether the concrete elements have uncoated or epoxy coated reinforcing bars, with a high level of testing required to confirm adequate tensile resistance. For example, stainless steel must be used for reinforcing bar anchors on the front face of a concrete barrier on a concrete deck that has epoxy coated reinforcing bars. TxDOT retrofit guide for concrete rails (2019) specifies that adhesive reinforcing bar anchors may not have any epoxy coating within the embedded length of the anchor.

A review of ICC Evaluation Services (ICC-ES) Evaluation Service Reports (ESR) for adhesive anchor products in WisDOT's approved product list indicated that coating (except for zinc) is typically not permitted within the embedded portion of reinforcing bar anchors. This is because the manufacturer's published bond stresses are determined based on testing of uncoated reinforcing bars.

In addition to reviewing published literature, the authors sent informal email inquiries to four adhesive anchor manufacturers (DeWALT, Simpson Strong-Tie, ITW, and Hilti) asking for potential test data on their products regarding effect of epoxy coating. Simpson Strong-Tie indicated that epoxy coating may reduce the bond strength by 10 to 15% for their adhesive products. This result is based on test data, provided in a letter, stating that four adhesives including SET-3G, SET-XP, AT-XP and ET-HP may be used with epoxy coated reinforcing bars when a factor of 0.85 is applied to the characteristic bond strength for uncoated reinforcing bars published in an ESR. DeWALT did not provide any test data but recommended using an adjustment factor similar to the modification factor for epoxy coating used to calculate the development length of reinforcing bars in accordance with relevant codes. Hilti and ITW did not provide any data or comments.

2.1.5. State DOT policies on applications of adhesive anchors

In addition to conducting a DOT survey, the research team reviewed manuals and specifications of twelve select DOTs to obtain more detailed information on their policies on design and application of adhesive anchors. The state DOTs included in the review and their acronyms are presented in Table 2.2. A discussion on current policies on the general use of adhesive anchors, based on review of manuals and specifications of the twelve states is provided in Appendix B. This section summarizes the state DOTs' policies on the use of adhesive anchors to support sustained tension and in overhead applications and on characteristic bond stress.

Table 2.2. State DOTs Whose Manuals, Specifications, and Other Literature were Reviewed

State DOT	Acronym
Wisconsin DOT	WisDOT
California DOT	Caltrans
Florida DOT	FDOT
Illinois DOT	IDOT
Indiana DOT	INDOT
Iowa DOT	IowaDOT
Michigan DOT	MDOT
Minnesota DOT	MnDOT
Nebraska DOT	NDOT
New York State DOT	NYSDOT
South Dakota DOT	SDDOT
Texas DOT	TxDOT

Notes: [1]*The Bridge Office Policies and Procedures was published by the Nebraska Department of Roads (NDOR) instead of the Nebraska Department of Transportation (NDOT). As such this report sometimes refers to NDOR.*

2.1.5.1. State DOT policies on adhesive anchors to support sustained tensile loads and in overhead applications

Many DOTs prohibit or severely (or significantly) limit the use of adhesive anchors to support sustained tensile loads and in overhead applications. Policies of several agencies regarding the use of adhesive anchors to support sustained tensile loads and/or in overhead applications are summarized as follows.

- Caltrans generally prohibits the use of adhesive anchors in sustained tensile load applications and defines "sustained tensile load" as a constant, unfactored tensile load greater than 10% of the nominal bond strength in tension of the anchor or anchor group.
- FDOT prohibits the use of adhesive anchors in overhead or upwardly-inclined positions, when the loading is predominately sustained tension, when there is a lack of structural redundancy, and/or when any of the service limit states result in tension loading of the adhesive. A "predominately sustained tension load" scenario is defined as when the permanent factored tension load is greater than 30% of the factored tensile resistance.
- IDOT typically does not allow adhesive anchors in overhead applications or sustained tension loading conditions.
- IowaDOT does not allow adhesive anchors in sustained tensile load overhead applications in highway projects.

- MDOT prohibits the use of adhesive anchors for overhead installation and has placed a moratorium on all adhesive anchors under sustained tensile loads due to the possibility of creep failure.
- For MnDOT, adhesive anchors are generally prohibited in sustained tension loading applications if their failure poses a direct threat to the safety of the travelling public, the installation is over or directly supporting traffic, or the concrete is structurally unsound. Similar to Caltrans, MnDOT only considers sustained tension loads that are at least 10% of the factored nominal tensile capacity of the anchor; note Caltrans uses the nominal capacity, not the capacity with a phi-factor. If the sustained tension load is less than 10% of the factored nominal tension capacity, then sustained tension loading does not need to be considered in the design procedure.
- NDOT states that resin adhesives on the Approved Product List should not be used in sustained tensile load applications.
- NYSDOT prohibits the use of adhesive anchors in all horizontal, overhead, and upwardly-inclined positions and any permanent applications subject to sustained tensile load, including cantilever applications.
- SDDOT states that adhesive anchors are not allowed in sustained tensile load applications in concrete members.
- VADOT does not allow the use of adhesive anchors for applications in which the anchors are subject to tension (axial or flexural loads) due to sustained, cyclical or fatigue loadings. In addition, the use of adhesive anchors is not permitted for attaching permanent parapets and barriers to the concrete bridge deck or superstructure.

2.1.5.2. State DOT Policies on Characteristic Bond Stress

The characteristic bond stress, τ, of the adhesive is used to calculate the basic bond strength, N_{ba}, and the critical distance, c_{Na}. The characteristic bond strength is a property of the adhesive product that represents both the inherent material and the installation and use conditions. A variety of factors affect the characteristic bond strength, including presence of concrete cracking, anchor diameter, anchor embedment depth, drilling method, degree of concrete saturation at the time of hole drilling and anchor installation, concrete temperature, age of concrete at time of installation, peak concrete temperature and duration of peak temperature during service life, and chemical exposure from the environment during anchor service life, and the type and duration of loading [4].

ACI 318 provides lower-bound default values for anchors meeting the qualification requirements of ACI 355.4 (see ACI 318 Table 17.6.5.2.5) where the product-specific characteristic bond stress is not known. These default values are much smaller than those of products in WisDOT approved product list, and thus, using these code tabulated values would generally result in designs that are very conservative.

WisDOT provides design values for characteristic bond stress in Table 40.16-1 in the Bridge Design Manual [3]. The table is shown in Figure 2.8. These are the minimum values compiled from the data of products in the Approved Product List for different anchor diameters and embedment depths, concrete cracking, and moisture conditions. For each of the tabulated conditions, the characteristic bond stress value is the lowest value of all the approved products that is standardized to allow the use of a single strength reduction factor of 0.65, which corresponds to Anchor Category 1 per ACI 318-19, for all the adhesive products. For example, for water-saturated, uncracked concrete, No. 4 reinforcing bars, the minimum bond stress for cracked concrete in the table is 370 psi, when combined with a strength reduction factor of 0.65, results in a factored bond stress, $\phi\tau_{uncr}$, of 241 psi. The 370 psi was obtained from the manufacturer evaluation report (for Simpson AT-XP) which indicates a characteristic bond stress of 990 psi to be used with two strength reduction factors of 0.45 and 0.54, resulting in the same $\phi\tau_{uncr}$ of 241 psi. For dry concrete, the characteristic uncracked bond stress varies from 770 to 990 psi and from 460 to 490 psi for cracked concrete depending on the anchor size. For water-saturated concrete, the characteristic bond stresses are substantially reduced, varying from 370 to 600 psi for uncracked concrete and from 280 to 410 psi for cracked concrete. The WisDOT approach is conservative and allows the designer to determine the minimum anchor strength without the need to specify the adhesive product to be used, and also allows the contractor to use any adhesive in the approved product list. The downside of this design approach is it does not allow using products with high-bond strengths in applications where the high-bond strengths are essential, such as anchoring into a structure with limited depth, like a bridge deck.

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	460	370	280
#5 or 5/8"	970	460	510	390
#6 or 3/4"	950	490	500	410
#7 or 7/8"	930	490	490	340
#8 or 1"	770	490	600	340

Table 40.16-1
Tension Design Table for Concrete Anchors

Figure 2.8. Table provided by WisDOT that specifies the minimum characteristic bond strengths to be assumed in design of adhesive anchors [5].

MnDOT maintains Approved/Qualified Products Lists for adhesive anchors. All adhesive anchors using reinforcing bars must demonstrate an uncracked characteristic bond strength of at least 1,000 psi and a cracked characteristic bond strength of at least 500 psi [20]. These stress values are higher than those used by WisDOT and not dependent on the anchor size. There are two classifications of threaded rods of which the lower-strength class has the same requirements as reinforcing bars while the higher-strength class must have an uncracked characteristic bond strength of at least 1,500 psi and a cracked characteristic bond strength of at least 750 psi [21]. The prequalification process specified by MnDOT additionally considers the effects of damp holes and corrosion protection methods, such as the use of epoxy-coated, galvanized, or stainless steel [22]. To address the effects of moist installation holes, MnDOT requires that the adhesive anchor must have a strength reduction factor in wet concrete corresponding to that of a Category 2 anchor without supplementary reinforcement (0.55 per ACI 318 Table 17.5.3). Additionally, MnDOT requires testing per AC308 Section 3.4 demonstrating that the adhesive will meet the specified strength requirements for hot-dipped galvanized ASTM F1554 threaded rods, stainless steel threaded rods, epoxy-coated reinforcing, and stainless-steel reinforcing.

Caltrans specifies minimum factored characteristic bond stresses for pre-approved adhesive anchors [18]. Chemical adhesives are required to have a factored characteristic bond stress $\Phi^*\tau_{cr}$ of at least 540 psi if threaded rods are used and 490 psi if reinforcing bars are used for the following conditions (the list below is not complete):

- Cracked concrete
- Water saturated concrete
- Periodic inspection
- Long term peak in-service concrete temperature ≥ 110°F
- Short term peak in-service concrete temperature ≥ 165°F

To compare the 490-psi value with those used by WisDOT, this value is divided by a strength reduction factor 0.65 used by WisDOT. This results in an unfactored characteristic bond stress of 754 psi, which is higher than the values used by WisDOT for the same condition. These WisDOT values vary from 280 to 410 psi depending on the anchor size.

FDOT has a different approach to specifying characteristic bond stress in which the adhesives are categorized into two classes: Type HV and Type HSHV, the latter of which is a higher strength adhesive anchor. Type HV anchors are intended for structural applications and Type HSHV anchors are intended for use in traffic railing retrofit applications where mechanical anchors are not practical and the predominant loading is from vehicle impact. Type HSHV anchors are not intended for sustained tension loads. Standard FDOT Specifications [18], Section 937, provides minimum characteristic bond strength requirements for adhesive anchors in a variety of scenarios, as shown in Figure 2.9. The characteristic bond strength of the anchors is determined in accordance with FM 5-568, Florida Method of Test for Anchor System Tests for Adhesive-Bonded Anchors and Dowels [8]. According to this test method, a 5/8-inch threaded rod anchor with 4-inch embedment is tested per ASTM E488, and the specified bond strength for the adhesive is calculated as μ - 2σ, where μ is the average bond stress and σ is the standard deviation. The required characteristic bond stresses vary from 1080 to 2290 psi for Type HV anchors and from 1830 to 3060 psi for Type HSHV anchors depending on various factors including loading, moisture, temperature, orientation, and curing conditions. These stress values are notably higher than those used by WisDOT.

	Type HV	Type HSHV
Confined Tension	2,290 psi	3,060 psi
Damp-Hole Installation	1,680 psi	1,830 psi
Elevated Temperature	2,290 psi	3,060 psi
Horizontal Orientation	2,060 psi	2,060 psi
Short Term Cure	1,710 psi	1,710 psi
Specified Bond Strength	1,080 psi	1,830 psi
Maximum Coefficient of Variation for Uniform Bond Stress: 20%.		

Figure 2.9. FDOT Specification Table 937-1 for minimum characteristic bond strengths for adhesive anchors in different applications [9].

2.2. Summary of DOT Survey

A survey was performed to evaluate the overall practices of state transportation agencies regarding design and use of adhesive anchors in bridges. The research team sent online survey invites to 50 state DOTs and received 26 responses. The survey results were divided into the following topics:

- Applications of adhesive anchors in bridges
- Design and detailing of adhesive anchors
- Construction, quality control, and challenges to the use of adhesive anchors.

This section summarizes the main results. The survey questionnaire and full results of the survey are provided in Appendix A.

2.2.1. Applications of adhesive anchors in bridges

The main findings are as follows:

- Fifteen states (54%) allow the use of adhesive anchors in permanent replacements of crash-worthy traffic railings attached to a bridge deck.
- Eleven states (42%) allow the use of adhesive anchors in abutment wingwall and backwall replacement and extension.
- Seventeen states (65%) do not allow the use of adhesive anchors in applications with sustained tensioned loads. Nine states (35%) allow these applications with sustained load limits per ACI 318 or state-specified load limits.
- Sixteen states (62%) do not allow the use of adhesive anchors installed in an overhead or upwardly inclined position. Ten states (39%) allow these applications with restrictions, for example, adhesive anchors are avoided in structures over traffic.

2.2.2. Design and detailing of adhesive anchors

The main findings are as follows:

- Most of the states indicated that they follow AASHTO LRFD and/or ACI 318 design procedures, although eight states (31%) mentioned that their specifications have deviations from AASHTO/ACI. These deviations are discussed in Appendix B.
- Regarding determination of bond strength:
  - Most of the states mentioned that they follow ACI 318 procedures on bond strength of adhesive anchors.
  - Most of the states do not specify minimum characteristic bond stress values and rely on data from the manufacturers of specific products used in the projects.
  - Concrete moisture condition during installation is generally required to be considered. Some states also consider concrete moisture condition in service and concrete temperature during installation and/or in service.

  - Most of the states do not specify effect of anchor coating while some specify limitations on the use of epoxy coated bars as adhesive anchors. For example, TxDOT does allow presence of epoxy coating within the embedded length, and MnDOT only allows epoxy coated bars when the adhesive anchors are not to resist tension.
- Twenty states (77%) indicated they do not have standard details for concrete adhesive anchors.

2.2.3. Construction, quality control, challenges

The main findings are as follows:

- About half of the states that responded require proof load testing of adhesive anchors. Typical testing load is 80% yield stress of anchor.
- Main challenges to the use of adhesive anchors include:
  - Lack of design guidelines for specific applications.
  - Products having different characteristic bond strengths, embedment depths, and requirements, making it hard to develop standard plan notes and specifications.
  - Concerns about quality control of anchor installation including lack of quality control and acceptance procedures, and/or that the procedures are difficult to enforce.
  - Lack of ACI certified installers.
  - Concerns about performance of adhesive anchors and high costs of products certified to meet ACI requirements.

CHAPTER 3. ASSESSMENT, RECOMMENDATIONS AND GUIDELINES

In this section, key WisDOT policies on adhesive anchors are summarized and discussed in relation to practices by other states and to the design procedures in ACI 318 and AASHTO LRFD. Based on this assessment, changes to WisDOT current policies are recommended.

3.1. Assessment of WisDOT policies

Assessment of WisDOT policies and recommendations are provided in the following areas:

1. Applications of adhesive anchors in bridges including:
   - Traffic railing replacement
   - Sustained tension loads (abutment walls, pier extension)
   - Overhead and upwardly inclined installation orientations
2. Design of adhesive anchors, including:
   - Bond strength
   - Anchor group effects
   - Strength reduction factors (ϕ factors)
3. Installation of adhesive anchors

3.1.1. Applications of adhesive anchors

3.1.1.1. Traffic railing replacement

3.1.1.1.1 Discussion - WisDOT - Bridge Manual Chapter 30 (WisDOT Manual) currently does not allow the use of adhesive anchors for crash-worthy concrete parapets. The WisDOT Manual limits construction options for parapet replacement. Consequently, there is the potential for significantly higher costs in situations where the edge of the deck needs to be removed far enough to provide the develop length of the existing deck bars and new parapet bars in the replacement work.

The current policy is somewhat consistent with ACI 318-19 which states that the design procedure in Chapter 17 of the code does not apply for impact load conditions. AASHTO LRFD, however, states that the design procedure in ACI 318 can be used for evaluating strength under impact loading provided that the anchors have an impact strength equal to or greater than their static strength. This condition has to be shown either by testing or a combination of testing and analysis.

More than half of the states responding to our survey indicate that adhesive anchors are allowed in construction of crash-worthy traffic railings. Some of these state DOTs have standard details for concrete parapet retrofit using adhesive anchors that are approved for MASH test levels. Previous reported research also indicated that traffic railings retrofitted using adhesive anchors could achieve equivalent strengths to cast-in-place railings under static and dynamic loadings. The strengths of concrete and steel tend to increase for the high-load rates occurring during vehicular impact. There is no indication in the literature that bond strength is negatively affected by high-load rates; while the opposite is true, that is, a reduction, in bond strength occurs when sustained tension loads exist due to creep of the adhesive.

3.1.1.1.2 Recommendation - Therefore, it is recommended that WisDOT consider permitting the use of adhesive anchors for concrete parapet replacement. Dynamic increase factors (DIF) may be considered when determining strength of adhesive anchors under impact loadings. Per ACI 349-13, a maximum DIF of 1.10 may be included for yield strength of steel and for compressive strength of concrete when calculating concrete shear strength. Literature reviewed by the authors does not appear sufficient to justify a DIF for concrete breakout strength and bond strength in tension.

3.1.1.2. Sustained tension loads

3.1.1.2.1 Discussion - Per current WisDOT Bridge Manual Chapter 40, adhesive anchors under sustained tension loads are not permitted. (Note: an equation for checking adhesive anchors subjected to sustained tension force is given in the WisDOT Manual, but it is unclear when it is applicable). The current policy significantly limits the use of adhesive anchors since, as written, it restricts adhesive anchors even in applications with small, sustained loads. Most state DOTs responding to our survey also have a similar restriction, which was likely originated from safety concerns raised by NTSB and FHWA moratoria after the failure at the Boston Tunnel in 2006. Some DOTs responding permit the use of adhesive anchors that are subjected to a sustained tension load limited to a small fraction of the static capacity.

As mentioned in Chapter 2, significant research has been conducted and improved the understanding of performance of adhesive anchors under sustained tension loading. In addition, manufacturers have developed new formulations that have gone through independent code approval testing that includes analysis of performance and allows development of sustained load limits to be more rationally established. FHWA published a new technical advisory in 2018, superseding its 2007 technical advisory, which no longer discourages the use of adhesive anchors in applications with sustained tension loading. The use of adhesive anchors to resist sustained tension is currently permitted by both ACI 318 and AASHTO LRFD. ACI 318-19 equation 17.5.2.2 applies to adhesive anchors under sustained tension loading, which includes an additional strength reduction factor of 0.55 to recognize the lower bond strength compared with non-sustained loading. AASHTO LRFD-9 Section 5.13 further reduces the factor to 0.50.

3.1.1.2.2 **–** Recommendation - To allow for potential cost-saving applications of adhesive anchors, it is recommended that WisDOT consider permitting the use of adhesive anchors in sustained tensile loading applications in accordance with AASHTO LRFD-9.

3.1.1.3. Overhead and upwardly inclined applications

3.1.1.3.1. Dicussion

Per current WisDOT Bridge Manual Chapter 40, adhesive anchors installed in the overhead or upwardly inclined position are not permitted. Most of the states responding to our survey have a similar restriction. The main reasons for this restriction include difficulties in quality control of anchor adhesive installation in overhead and upwardly inclined applications and concerns with performance of those type anchors.

ACI 318 has specific requirements for adhesive anchors to be installed in a horizontal or upwardly inclined orientation. The anchors must be qualified in accordance with ACI 355.4 requirements for sensitivity to installation direction, and if used to resist sustained tensile loads, must be installed by an ACI certified installer. Adhesive products qualified for upwardly inclined installation are available.

3.1.1.3.2. Recommendation

If WisDOT has a need for using adhesive anchors in an overhead or upwardly inclined installation orientation, it is recommended that WisDOT allow this application and require that the installation be performed by a certified installer in addition to proof testing a percentage of installed anchors.

3.1.2. Design of adhesive anchors

3.1.2.1. Bond strength

Calculation of the bond strength of an anchor using a particular adhesive requires determination of the adhesive characteristic bond stress which is dependent on concrete cracking, reinforcing and environmental conditions of the concrete, surface characteristics of the anchor, and the type of loading.

3.1.2.1.1. Characteristic bond strength

As discussed in Chapter 2, the current WisDOT Bridge Manual specifies minimum characteristic bond stress values in Table 40.16-1 based on the lowest bond strength of all the products in the Approved Product List for "Concrete Adhesive Anchors". While this approach is conservative and simplifies the design and approval efforts, it does not allow for utilization of products with high bond strengths, which could be essential for retrofit work in existing structures with limited depths such as concrete bridge decks. Two alternative approaches are recommended for consideration to allow for utilization of adhesives with high bond strengths.

Alternative approach No. 1

The first approach, which is similar to that employed by TxDOT, is to allow an option for the designer to specify the required characteristic bond stress for a given concrete cracking, moisture, and temperature condition in addition to specifying the anchor size, embedment depth and spacing on the drawings. Below is an example illustrating how specifying the required characteristic bond stress would affect the selection of adhesive product.

- The required factored characteristic bond stress, $\phi\tau_{cr}$, is 570 psi for No. 6 reinforcing bar in cracked, water-saturated concrete in temperature range A (maximum short-term temperature = 130°F, maximum long-term temperature = 110°F).
- Two adhesives are considered for this condition. Adhesive #1 (Hilti HIT HY-100) has a characteristic bond stress of 775 psi with a strength reduction factor of 0.65 for water-saturated concrete. Adhesive #2 (Hilti HIT-HY-200-R) has a characteristic bond stress of 1090 psi with a strength reduction factor of 0.55 for water-saturated concrete. The factored characteristic bond stresses are 504 and 600 psi for Adhesives #1 and #2, respectively, meaning only Adhesive #2 meets the requirement.

Alternative approach No. 2

The second approach, which is employed by FDOT, is to classify the adhesive products. The adhesive products may be classified into two types with different characteristic bond strength requirements and intended uses; below is an example:

- Type 1 adhesives. This type may be used for all applications permitted by WisDOT and consists of all products in the current Approved Product List.

- Type 2 adhesives. This type is intended for use in applications with limited embedment depths such as replacement of railings on a bridge deck and consists of products that have greater characteristic bond strengths than the minimum values in the current table. A new table of characteristic bond stress values will need to be developed for Type 2 adhesives.

In addition to the alternative approaches above, it is recommended that WisDOT current minimum characteristic bond stress table (Table 40.16-1) be accompanied by a list of assumptions used to compile it including temperature range and type of inspection (periodic or continuous).

3.1.2.1.2. Epoxy coating

Effect of anchor coating (i.e., epoxy coating or galvanizing) on the anchor bond strength is not specifically considered in WisDOT Bridge Manual. Most of the states reviewed do not consider the effect of coatings. TxDOT does not permit epoxy coating within the embedded length and MnDOT restricts the use of epoxy coated reinforcing bars as adhesive anchors subjected to tensile loading. Manufacturers' evaluation reports typically state that coating within the embedded length of reinforcing bar anchor is not permitted. Literature reviewed by the authors indicates that up to a 15% reduction in the anchor bond strength may occur due to epoxy coating and that the reduction varies among different adhesive products.

Thus, it seems reasonable to include a reduction factor when determining the anchor bond strength; however, further research is needed to make recommendations on the reduction factor to be used for design. An alternative approach is to require that the epoxy coating within the embedment length of the anchor be removed, and the anchor be cleaned before installation. It is acknowledged there are other considerations regarding reinforcement durability associated with removing the epoxy coating.

3.1.2.1.3. Effect of concrete cracking condition

Per current WisDOT Bridge Manual, characteristic bond strength values are selected based on anticipating that concrete will be in the cracked condition at service load levels. This is consistent with the manner within the ACI 318 design methodology. If analysis indicates concrete cracking at service load levels, the characteristic bond stress of adhesive anchor in cracked concrete, τ_{cr}, is used and adhesive anchors must be qualified for use in cracked concrete in accordance with ICC-ES AC308/ACI 355.4. For adhesive anchors located in regions of a concrete member where analysis indicates no concrete cracking at service load levels, τ_{uncr}, is permitted to be used in place of τ_{cr}.

No change to this policy is recommended.

3.1.2.2. Group effect

The design procedure in WisDOT Bridge Manual indirectly considers the group effect and does not specifically follow ACI 318 procedures for calculating strength of an anchor group. It may be desirable to modify the Manual to be consistent with ACI 318.

3.1.3. Anchor installation

Adhesive anchor technology has advanced, and code improvements have been made since the Boston Tunnel project. Adhesive anchor manufacturers have improved adhesive materials and installation methods and building code committees have developed rigorous test criteria and requirements to address a variety of installation and application issues, which include conditions occurring at the Boston

Tunnel project and other incidents not as high profile. Adhesive anchors are subjected to testing in accordance with ACI 355.4 by an independent, accredited test laboratory to determine adhesive anchor characteristic bond strengths as well as strength reduction factors to be applied to characteristic bond strengths for anchor design. ACI has developed and implemented an adhesive anchor installer course to educate on the proper installation procedures and consequences with improper installation.

Adhesive anchor installation and quality control are addressed in WisDOT Standard Specification Section 502.3.14 and states the drilled holes are to be cleaned by flushing with water followed with air blow until the hole is dry and dust-free. In WJE experience, it is often difficult and time-consuming to completely dry the holes that were flushed with water, and the residue moisture in the concrete may interfere with the adhesive curing and impair the bond strength. The use of water to clean anchor holes is not recommended; instead, anchor installation, including hole cleaning, should strictly follow the manufacturer's printed installation instructions (MPII). Typical hole cleaning for adhesive anchor installation consists of the blow-brush-blow method. The process is drilling the specified diameter hole, using specified air pressure to blow the concrete cuttings from the bottom of the drilled hole, using a specific diameter bottle brush to clean the sides of the drilled hole, and using air pressure to blow the concrete cuttings removed during the bottle brush use. Certain manufacturers have developed a hollow drill bit and vacuum dust extraction system that is an alternative to the blow-brush-blow cleaning method. The dust extraction system consists of a vacuum connected to a special hollow drill bit. The vacuum removes the concrete cuttings during the drilling process through the hollow portion of the drill bit. Using a hollow drill bit and vacuum dust extraction system eliminates the need for the blow-brush-blow method.

CHAPTER 4. DESIGN EXAMPLES FOR ADHESIVE ANCHORS

Three design examples were performed to illustrate the design procedure for adhesive anchors in accordance with ACI 318-19, AASHTO LRFD 9th Edition, and 2020 WisDOT Bridge Manual for three different applications, including:

1. Wingwall replacement
2. Abutment extension
3. Traffic parapet replacement

4.1. Design Example 1 - Wingwall Replacement

This example illustrates the design for replacement of an upper wingwall that is to be connected to an existing lower wingwall with adhesive anchors. Flexural and shear resistances of the wall are checked in accordance with WisDOT Manual and AASHTO LRFD. Contribution of the axial force in the wall due to its self-weight to increasing the wall flexural resistance is small and was conservatively disregarded in the calculations. Tensile resistance of the adhesive anchors was calculated in accordance with ACI 318 Chapter 17 and AASHTO LRFD Section 5.13. Interface shear resistance (at the interface between the new upper wing and existing lower wing) is calculated in accordance with AASHTO LRFD Section 5.7.4.3.

4.1.1. Design Requirements for Abutment Wingwalls

General design requirements for abutments are specified in Chapter 12 of WisDOT Bridge Manual. The following applies for design of wingwalls:

- Design loads include lateral earth pressure and live load surcharge. Load factors are presented in Table 4.1.
- Railing loads are not required to be applied to the wingwalls.
- Passive earth pressure resistance is generally not utilized.
- The resistance of the wing pile to horizontal forces should not be included in the calculations for the wing capacity.
- Wingwalls are designed as cantilevers extending from the abutment body.
- The primary force in wingwalls without special footings that are poured monolithically with the abutment body is the bending moment. Torsion is usually neglected.

Table 4.1. Load Factors for Wingwall Design (WisDOT Manual 12.8.2)

Load	Strength I	Service I
Lateral earth pressure, active, EH	1.50	1.00
Live load surcharge, LS	1.75	1.00

4.1.2. Design Procedure for Adhesive Anchors in Wingwall Replacement

The design procedure for adhesive anchors in wingwall replacement presented in the example is as follows:

1. Determine lateral earth pressure and live load surcharge acting on the upper wingwall per WisDOT Manual and AASHTO LRFD.
2. Calculate bending moment and shear force for a section at bottom of the upper wingwall.
3. Calculate tension in back face adhesive anchors due to bending moment assuming linear distribution of compressive stress in concrete and disregarding tensile stress in concrete for two load cases: 1) total tension due to both lateral earth pressure and live load surcharge, and 2) sustained tension due to lateral earth pressure. Contribution of front face adhesive anchors is disregarded.
4. Calculate shear force in adhesive anchors.
5. Calculate tensile and shear resistances of adhesive anchors in accordance with ACI 318-19 Sections 17.6 and 17.7.
6. Calculate interface shear resistance in accordance with AASHTO LRFD, Section 5.7.4.3
7. Check demand vs capacity for anchors in tension due to all applicable loads.
8. Check demand vs capacity for anchors due to sustained tension in accordance with AASHTO LRFD Section 5.13.2.2 by using ACI 318-19 Equation 17.5.2.2, but with a factor of 0.5 in place of 0.55.
9. Check if the interface shear resistance is sufficient to resist the design lateral loads.
10. If the interface shear resistance is sufficient, shear force in the anchors does not need to be checked. If the interface shear is not sufficient, shear resistance and tension-shear interaction in the anchors needs to be checked in accordance with ACI 318, Chapter 17. Note that in the example provided, shear resistance and tension-shear interaction are checked regardless of the interface shear resistance to illustrate the procedure.

4.1.3. Wingwall Design Example

In part 1 of the example, detailed calculations are provided for one set of wing wall parameters using MathCAD. In part 2, expanded calculations for different wing wall geometries are generated using Excel spreadsheets.

4.1.3.1. Part 1 - Detailed Calculations

The wingwall elevation and section used in the example are presented in Figure 4.1 and Figure 4.2. The main design parameters are as follows:

- Height of upper wingwall H_U = 5 ft.
- Length of upper wingwall L_U = 14 ft.
- Length of lower wingwall L_L = 7.5 ft.
- Thickness of upper wingwall B_U = 15 in.
- Thickness of lower wingwall B_L = 36 in.
- Adhesive anchor design is as follows:
  - (6)-#6 reinforcing bars, Grade 60.
  - Embedment length = 15 in.
  - Anchor spacing = 15 in.

Design assumptions and other design parameters are provided in the calculations in Appendix C1.

A summary of the demand capacity ratio (D/C) results is as follows (D/C ≤ 1 is considered sufficient):

- For the load case considering all loads, tensile resistance of the anchors is sufficient, and the governing failure mode is concrete breakout with D/C = 0.82. For the load case considering sustained tension, bond strength of the anchors is sufficient with D/C = 0.55.
- Shear interface resistance is sufficient with D/C = 0.16, and thus, shear in the adhesive anchors need not to be considered.
- If shear interface resistance is disregarded, shear resistance of the adhesive anchors is not sufficient with D/C = 1.18.

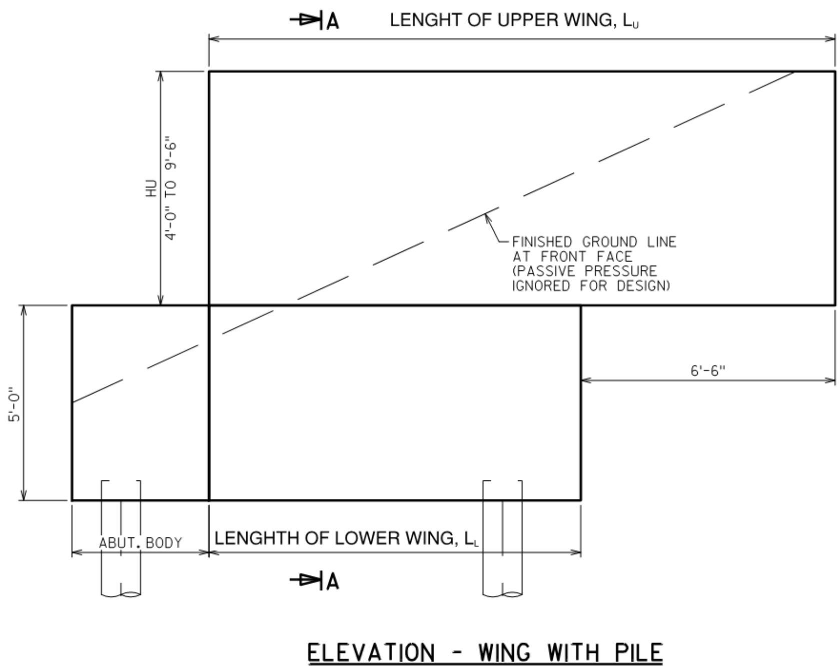

Figure 4.1. Wingwall elevation showing the replaced upper wingwall on an existing lower wingwall

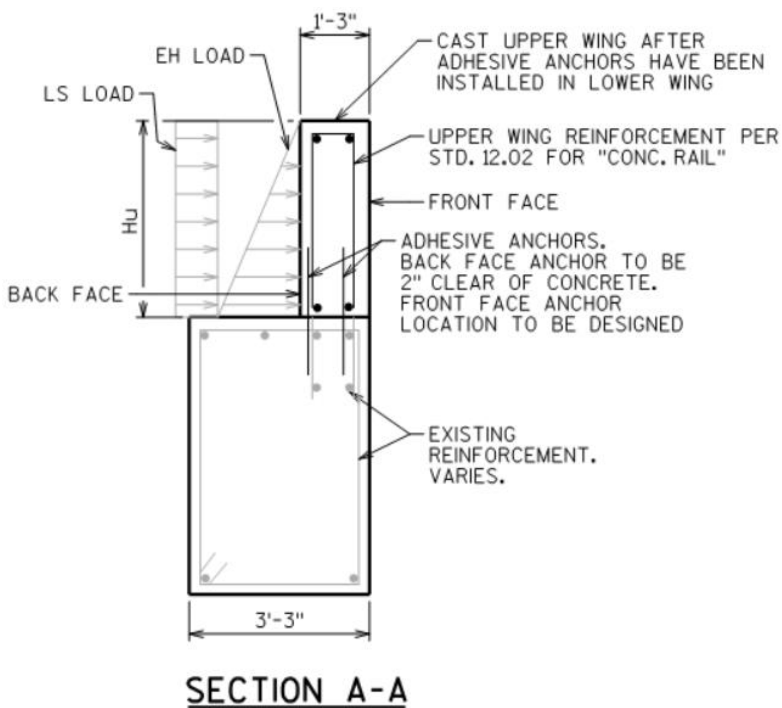

Figure 4.2. Wing wall section

1.1.1.1. Part 2 - Expanded Calculations

In part 2, calculations are generated using Excel spreadsheets (provided in Appendix C1) for upper wing wall heights ranging from 4-ft 0-in to 9-ft 6-in, upper wing lengths ranging from 10-ft 0-in to 12-ft 0-in for walls without a cantilever and 14-ft 0-in to 24-ft 0-in for walls with a 6-ft 6-in cantilever. The cantilever length is based on WisDOT wing wall standard designs. In the calculations, No. 6 adhesive anchors with 15-inch spacing and 15-inch embedment depth were used. Characteristic bond stresses were selected such that the calculated anchor bond strength is equal or greater than the concrete breakout strength in tension. The main findings from the calculations are summarized as follows.

- Reducing anchor spacing, i.e., increasing the number of anchors, does not increase flexural resistance of the upper wing wall in a meaningful way since the tensile strength of the anchor group is limited by concrete breakout, which is limited by the geometries of the wall.
- The 15-inch standard wall is adequate for $H_U \leq$5-ft-6-in with a 6-ft-6-in cantilever and $H_U \leq$7-ft-0-in without a cantilever. For other wall geometries, the thickness of the wall needs to be increased to meet the flexural demand.

Table 4.2 summarizes upper wing wall thicknesses that meet the flexural demands for different, typical wall geometries. The intent of this table is to provide an example illustrating an approach to simplify the design of adhesive anchors for wing wall replacement, and not to cover all geometries or show the most optimal designs.

Table 4.2. Summary of Upper Wing Wall Thicknesses (B_U) for Different Wing Wall Geometries

L_U	**10-ft 0-in to 12-ft 0-in**	**14-ft 0-in to 16-ft 0-in**	**18-ft 0-in to 24-ft 0-in**
L_L	**= L_U**	**= L_U - (6-ft 6-in)**	**= L_U - (6-ft 6-in)**
Wall Cantilever	**0-ft 0-in**	**6-ft 6-in**	**6-ft 6-in**
$H_U \leq$ 5-ft 6-in	15-in	15-in	15-in
5-ft 6-in $\leq H_U \leq$ 7-ft 0-in	15-in	24-in	24-in
7-ft 0-in $\leq H_U \leq$ 8-ft 6-in	24-in	-	30-in
8-ft 6-in $\leq H_U \leq$ 9-ft 6-in	24-in	-	-

Notes:

In all cases, No. 6 adhesive anchors @ 15-in spacing, 15-in embed., τ_{cr} = 450 psi (τ_{uncr} = 1,350 psi).

See Section 4.1.3.1 for definitions of the notations.

"-" indicates adhesive anchor design is not applicable.

4.2. Design Example 2 - Abutment Extension

This example illustrates the calculations of the flexural resistance of an abutment section in repaired conditions in which the new abutment is connected to an existing abutment using several alternate designs of adhesive anchors. The flexural resistances in repaired conditions were compared with each other and with that in the original construction, i.e., with cast-in-place fully developed reinforcement. The abutment section in the original condition is shown in Figure 4.3 with (6)-#6 reinforcing bars on the tension side (Case 1). Four alternate designs of adhesive anchors include:

- Case 2 - (6)-#6 reinforcing bars;
- Case 3A - (6)-#8 reinforcing bars;
- Case 3B - (9)-#6 reinforcing bars; and
- Case 3C - Two rows of (6)-#6 reinforcing bars (12-#6 reinforcing bars in total)

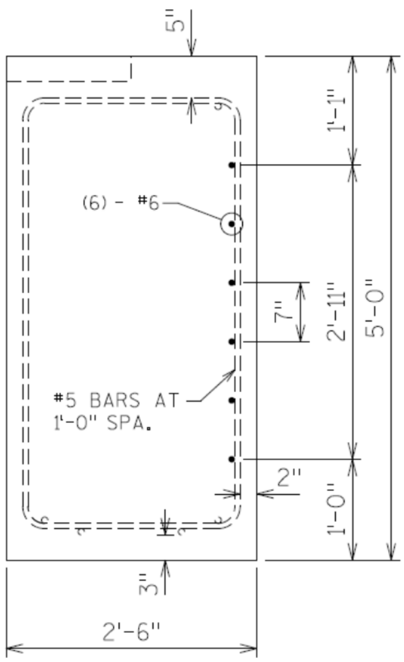

Figure 4.3. Abutment cross section in the original condition with (6)-#6 reinforcing bars on the tension side.

The calculations are provided in Appendix C2. A summary of the results is as follows:

- In all four designs of adhesive anchors, the factored flexural resistance of the section was calculated to be only about 17% of the resistance of the original section.
- The governing failure mode in every alternate adhesive anchor design was concrete breakout in tension, which is limited by the edge distance and anchor spacing. This is consistent with the commentary in ACI 318-19, Section R17.6.5.1. For a #6 anchor with 15-in. embedment depth as in the example, the concrete breakout strength of the group of 6 anchors was smaller than that of a single anchor that has an edge distance greater than $1.5h_{ef}$. (h_{ef} is the effective embedment depth). Increasing the anchor size and/or the number of anchors (including adding an additional row of anchors) does not improve the concrete breakout strength in a meaningful way since the concrete

breakout strength of the anchor group is limited by the projected concrete failure area of the anchor group, A_{Nc}, which is limited by the edge distance and anchor spacing.

- The use of lower resistance factors (strength reduction factors) for adhesive anchors, as compared with the resistance factor for flexure of a tension-controlled section in accordance with ACI 318 also contributed to the reduced flexural resistance for the repaired sections.

4.3. Design Example 3 - Concrete Parapet

This example illustrates the design for replacement of concrete parapet that is to be connected to an existing concrete bridge deck with adhesive anchors. Resistance of the parapet to lateral vehicular impact loads is calculated in accordance with AASHTO LRFD Chapter 13. Axial force in the parapet was not considered. Tensile resistance of the adhesive anchors was calculated in accordance with ACI 318 Chapter 17 and AASHTO LRFD Section 5.13. Interface shear resistance (at the interface between the new parapet and existing concrete deck) is calculated in accordance with AASHTO LRFD Section 5.7.4.3.

4.3.1. Design Requirements for Traffic Concrete Railings

Design requirements for traffic concrete railings/parapets are specified in AASHTO LRFD 9th Edition, Chapter 13. Crash testing is required for approval of a railing system and its connection to the deck for the desired test level. The railing specimen for crash testing may be designed to resist the transverse force corresponding to the required test level in accordance with Appendix A13 of AASHTO LRFD.

$$R_w \geq F_t \quad \text{(AASHTO Eq. A13.2-2)}$$

Where,

- F_t is the transverse design force per AASHTO Table A13.2-1, reproduced in Figure 4.4, depending on the required test level.
- R_w = railing resistance to transverse load, determined using a yield line approach per AASHTO Section A13.3.1. Design parameters in the yield line analysis for impacts within a wall segment and at the end of a wall segment are illustrated in Figure 4.5 and Figure 4.6, respectively.

For impacts within a wall segment:

$$R_w = \left(\frac{2}{2L_c - L_t}\right)\left(8M_b + 8M_w + \frac{M_c L_c^2}{H}\right)$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_w)}{M_c}}$$

For impacts at end of a wall segment or at a joint:

$$R_w = \left(\frac{2}{2L_c - L_t}\right)\left(M_b + M_w + \frac{M_c L_c^2}{H}\right)$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H\left(\frac{M_b + M_w}{M_c}\right)}$$

Where,

- H = height of parapet wall (ft)
- L_c = critical length of yield line failure pattern (ft)
- L_t = longitudinal length of distribution of impact force F_t (ft);
- M_b = additional flexural resistance of beam in addition to M_w, if any, at top of wall (kip-ft); M_b = 0 for typical concrete parapets.
- M_c = flexural resistance of cantilevered walls about an axis parallel to the longitudinal axis of the bridge (kip-ft/ft).
- M_w = flexural resistance of the wall about its vertical axis (kip-ft).

When M_c and M_w varies with height (e.g., the width of the concrete railing varies along the height), each resistance is taken as the average of its value along the height of the railing.

Design assumptions for the yield line analysis per AASHTO LRFD include the following:

- The positive and negative wall resisting moments are equal
- M_c and M_w do not vary significantly over the height of the wall
- Yielding of horizontal reinforcement and vertical reinforcement

It is noted that a shear design procedure for concrete parapet is not defined in AASHTO LRFD.

	Railing Test Levels					
Design Forces and Designations	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (kips)	13.5	27.0	54.0	54.0	124.0	175.0
F_L Longitudinal (kips)	4.5	9.0	18.0	18.0	41.0	58.0
F_v Vertical (kips) Down	4.5	4.5	4.5	18.0	80.0	80.0
L_t and L_L (ft)	4.0	4.0	4.0	3.5	8.0	8.0
L_v (ft)	18.0	18.0	18.0	18.0	40.0	40.0
H_e (min) (in.)	18.0	20.0	24.0	32.0	42.0	56.0
Minimum H Height of Rail (in.)	27.0	27.0	27.0	32.0	42.0	90.0

Figure 4.4. Design forces for traffic railings (AASHTO Table A13.2-1)

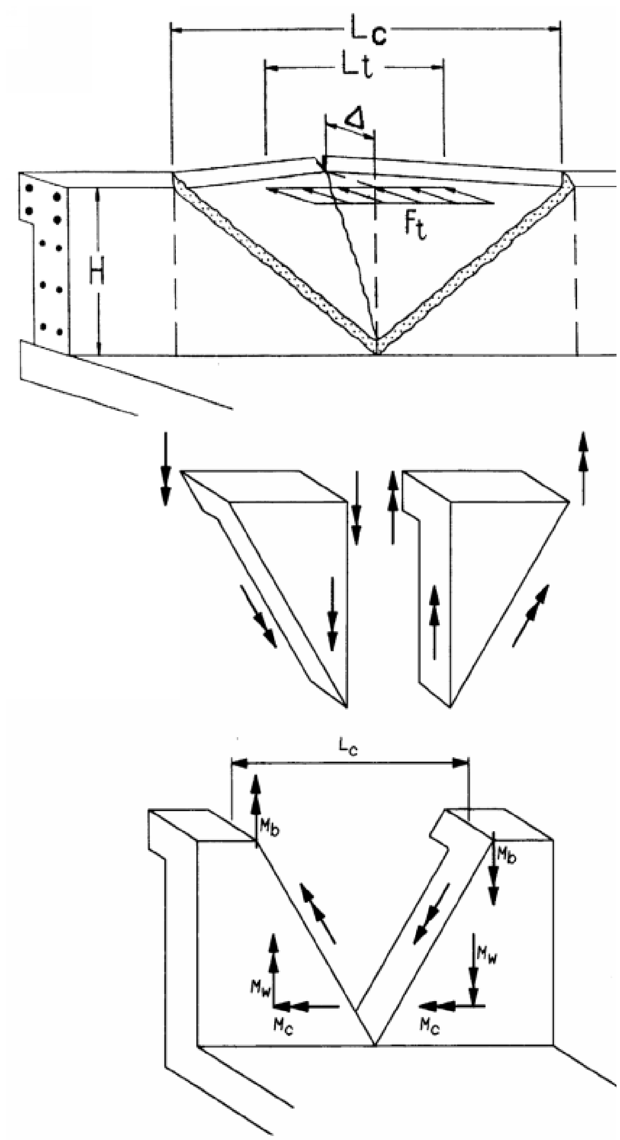

Figure 4.5. Illustration of design parameters using yield line analysis of concrete parapet walls for impact within wall segment (AASHTO LRFD Figure CA13.3.1-1)

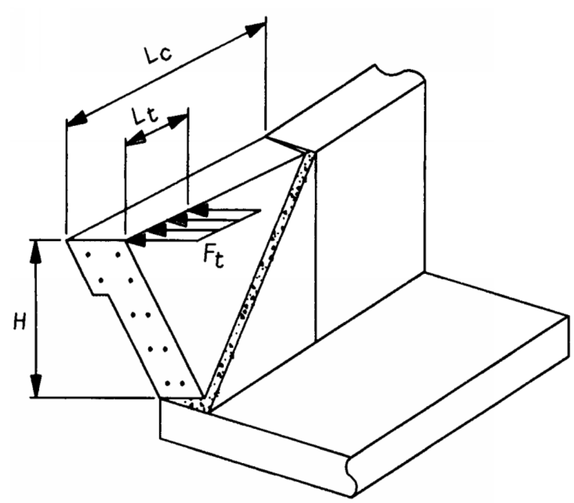

Figure 4.6. Illustration of design parameters using yield line analysis of concrete parapet walls for impact near end of wall segment (AASHTO LRFD Figure CA13.3.1-2)

4.3.2. Design of Adhesive Anchors in Traffic Concrete Railings

4.3.2.1. Design Requirements for Adhesive Anchors in Traffic Concrete Parapets

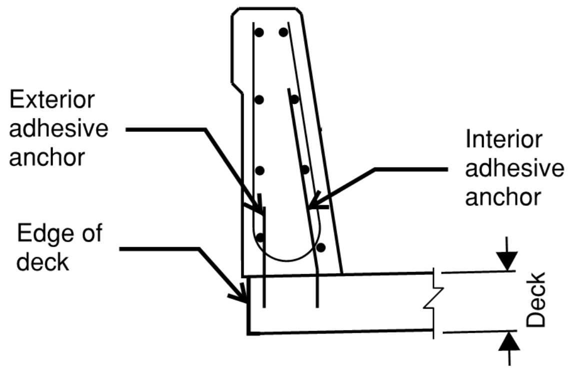

Figure 4.7. Example of adhesive anchors used in a concrete parapet on a bridge deck.

Adhesive anchors connect the concrete parapet to the concrete deck. An example of adhesive anchors used in a concrete parapet on a bridge deck is shown in Figure 4.7.

Below are the requirements for design of adhesive anchors in concrete parapets to resist the transverse impact force:

- The design transverse force F_t is calculated based on AASHTO Table A13.2-1 (Figure 4.4)
- The parapet resistance to transverse force R_w is calculated based on a yield line analysis per AASHTO Section A13.3.1.
  - Two rows of anchors are provided on the interior face (traffic side) and exterior face of the parapet.
  - Anchors in the interior row are designed such that the anchor yields at the nominal tensile strength before concrete breakout and bond failures occur. Yield strength of the anchors is used to calculate flexural resistance of the parapet about its longitudinal axis M_c (See Section 4.3.1), which is used to calculate the parapet resistance R_w
  - Anchors in the exterior row are provided as secondary anchors. Contribution of the exterior anchors may be conservatively disregarded.
- The shear strength of the anchors, interface shear resistance (at parapet-deck interface), and shear strength of the parapet in bending about its vertical axis contribute to resisting shear force in the parapet resulting from the transverse vehicular impact load. If the total of interface shear resistance and shear strength of the parapet is sufficient to resist the shear in the parapet due to the transverse load, which is typically the case as demonstrated in the example, shear force in the adhesive anchors does not need to be considered.

4.3.2.2. Proposed Design Procedure for Concrete Parapet with Adhesive Anchors

Based on the design requirements discussed above, the authors propose the following procedure for design of concrete parapets connected to a concrete deck using adhesive anchors:

1. Determine the design transverse force F_t and distribution length L_t based on the required test level (Figure 4.4).

2. Determine tensile strength of the adhesive anchors in accordance with ACI 318 Chapter 17 disregarding tension-shear interaction.
3. Check if the anchor nominal tensile strength is equal or greater than its yield strength. If so, proceed to Step 4. If not, try using smaller anchors and/or longer anchor spacing and repeat Step 3.
4. Determine M_c, averaged over the height of the parapet, in which M_c of the section at the parapet-deck interface is determined based on tensile strength of the interior adhesive anchors.
5. Determine M_w
6. Determine L_c and R_w
7. Check parapet resistance:

$$R_w \geq F_t$$

8. Calculate parapet-deck interface shear resistance, ϕV_{ni} per LRFD Section 5.7.4.3 over the length L_c.
9. Calculate shear resistance of the parapet in bending about its vertical axis, $\phi V_{w,}$ per LRFD Section 5.7.3.
10. Calculate the total shear resistance based on interface shear resistance and shear resistance of the concrete parapet in bending about its vertical axis
    - For impacts within a wall segment:

$$V_n = \Phi_v (2V_w + V_{ni})$$

    - For impacts at end of wall:

$$V_n = \Phi_v (2V_w + V_{ni})$$

11. Verify that $\Phi_v V_n \geq F_t$. If so, shear force in adhesive anchors needs not to be considered.

4.3.3. Concrete Parapet Design Example

4.3.3.1. Design input

The parapet design used in the example is based on WisDOT 42SS parapet as shown in Figure 4.8 and Figure 4.9. The initial design input is as follows.

<u>Design force and distribution length:</u>

- For WisDOT 42SS Parapet, the test level is TL-4, resulting in F_t = 54 kips and L_t = 3.5 ft.

<u>Bridge deck:</u>

- Deck thickness = 8 in.
- Concrete deck compressive strength f'_c = 4 ksi

<u>Parapet materials</u>

- Concrete parapet compressive strength f'_c = 4 ksi
- Steel reinforcement yield strength f_y = 60 ksi
- Steel anchor yield strength f_{ya} = 60 ksi

- Steel anchor ultimate strength f_{ua} = 80 ksi

Parapet reinforcement:

- Horizontal reinforcement:
  - Within-wall segment: #4 as shown
  - End-of-wall segment: #8 as shown. The larger reinforcing bars are selected to account for the higher demand at the end of the parapet to resist impact loadings.
- Cast-in-place vertical reinforcement: #5 spaced at 12 in.
- Adhesive anchors: #6 spaced at 15 in.; embedment length = 5-1/2 in.

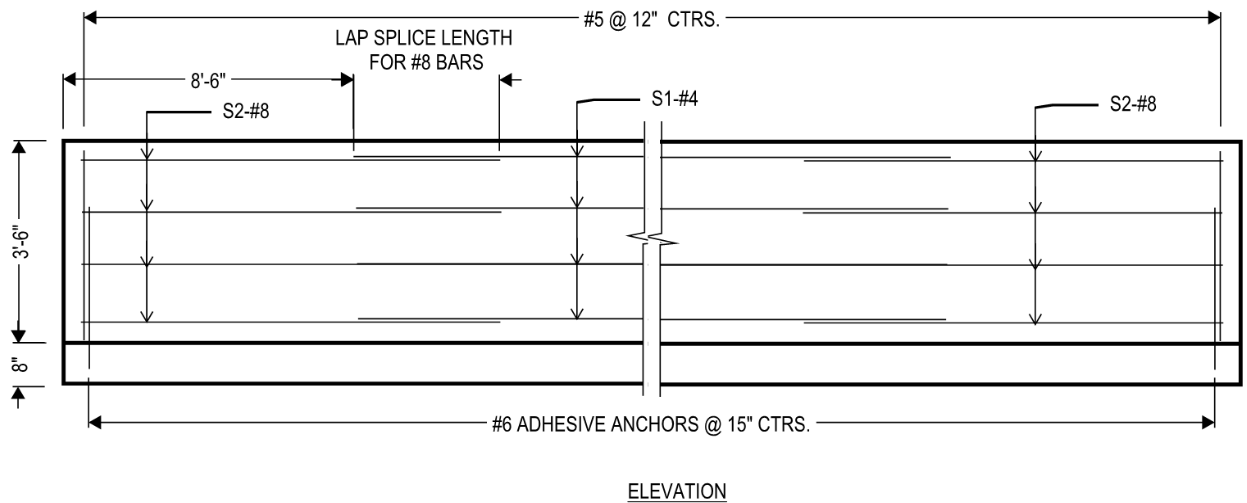

Figure 4.8. Parapet Elevation

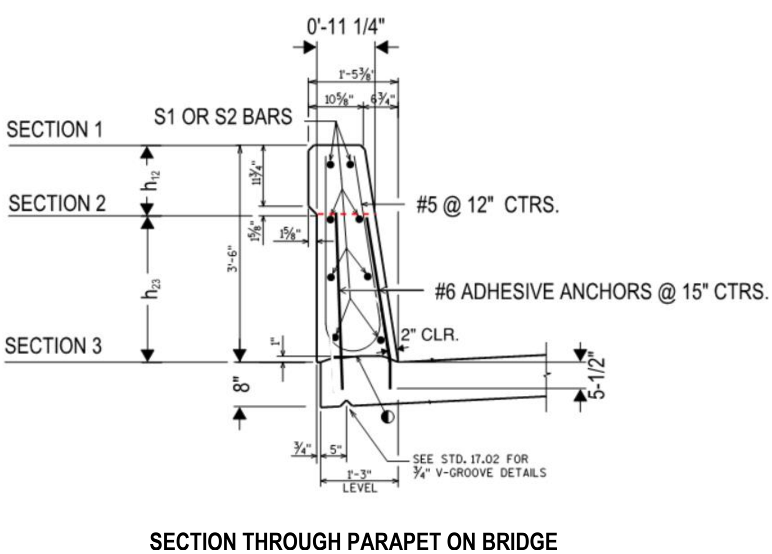

Figure 4.9. Parapet cross-section

4.3.3.2. Calculations and results

Calculations are provided in Appendix C3. A summary of the results and comments are as follows:

- In the initial adhesive anchor design presented in 4.3.3.1, the tensile strength of the #6 adhesive anchor (in interior row) is governed by concrete breakout, and the nominal strength is smaller than yield strength of the anchor. This does not meet an assumption of the design method based on yield line analysis which assumes yielding of the steel reinforcement; thus, the anchor design needs to be revised in order to use this design method.
- To obtain a revised adhesive anchor design to meet the above assumption, several trial calculations with varying anchor size, anchor spacing, and embedment depth were performed and indicated that a smaller anchor size is needed for the nominal tensile strength of anchor to equal its yield strength. In this example, #4 adhesive anchors are used in the revised design with the same anchor spacing and embedment depth as in the initial design. The results show that the tensile strength of the #4 adhesive anchor is governed by concrete breakout, and the nominal strength is greater than yield strength of the anchor. An adhesive with characteristic bond stress of 1,500 psi (for cracked concrete) is required to avoid bond failure. The resistance of revised parapet exceeds the transverse force for both within-wall segment and end-of-wall segment.

CHAPTER 5. LABORATORY INVESTIGATIONS

In this section, samples prepared for testing are described and test results are presented. WisDOT policies on adhesive anchors were used in preparing the test samples. Adhesive anchor performance is compared to current design code capacity equations as well as capacities based on the material strength. Appendix D contains additional information and figures regarding the test samples, test configuration, and test results. Comments regarding WisDOT current design manual are made based on the test results.

5.1. Test Sample Description

Testing was conducted to simulate an upper wingwall replacement cast on top of an existing lower wingwall to evaluate lateral load resistance of the upper wingwall. The upper wingwall was connected to the lower wingwall using epoxy coated reinforcing bars adhesively anchored into the lower wingwall and cast into the upper wingwall. Two test samples were fabricated to represent an upper wingwall replacement on a lower wingwall.

The lower wingwall measured 40-in deep by 80-in wide by 60-in tall (see Figure D.1. in Appendix D for a sketch of the wingwall specimen, including lower and upper wingwalls and reinforcement). A total of sixteen (16) No. 6 reinforcing bars were positioned within the perimeter of the stirrup reinforcement and had a 12-in spacing and approximately 2-in clear cover. Stirrups fabricated from No. 5 reinforcing bar were spaced at 12-in centers along the 80-in length. The lower wing wall concrete had an average compressive strength of 5110 psi at the time of wall testing (78 days old).

The upper wingwall measured 15-in deep by 80-in wide by 60-in tall. A total of eight (8) No. 4 reinforcing bars spaced at 8 inches were positioned longitudinally at the back face and five (5) No. 4 reinforcing bars spaced at 16 inches were positioned longitudinally at the front face. A total of nine (9) No. 5 U-shaped stirrups spaced at 9 inches were positioned along the upper wingwall. The upper wingwall concrete had an average compressive strength of 2020 psi at the time of wall testing (34 days old), which was unexpectedly lower than the target strength of 3,500 psi. This, however, does not affect performance of the adhesive anchors because concrete breakout strengths of the anchors are determined by the concrete strength of the lower wingwall.

The upper wingwall was anchored to the lower wingwall with a total of ten (10) No. 6 epoxy coated reinforcing bars with five (5) along the back face and five (5) along the front face. The actual yield strength (f_{ya} = 68.7 ksi) and ultimate strength (f_{ua} = 106.6 ksi) of the reinforcing bars used in the test samples was provided by mill certificates from the steel supplier. The No. 6 reinforcing bars were adhesively anchored 15-in deep into the lower wingwall and spaced at 16-in centers with a clear cover of 3 inches on the back face and 4 inches at the front face of the upper wingwall. The embedment depth of 15 inches is twenty (20) times the nominal diameter of a No. 6 reinforcing bar, which is the stated embedment depth requirement in Section 40.16.1.1 of the WisDOT Bridge Manual.

The No. 6 reinforcing bar were anchored 15-in deep using Simpson Strong-Tie AT-XP adhesive. This adhesive was selected because it is listed on the approved WisDOT list with the lowest characteristic bond strength for uncracked concrete among the adhesive products listed. Installation of the reinforcing bar, including hole drilling and cleaning, adhesive injection, and bar insertion, was performed vertically downward and followed the manufacturer's printed installation instructions (MPII). The concrete surface of the lower wingwall was roughened to an approximate amplitude of ¼-in. in accordance with ACI 318-19, Table 22.9.4.2 prior to casting the upper wingwall.

5.2. Test Configuration

Once the upper wingwall concrete cured for 28 days the samples were positioned horizontally from the concrete casting direction for testing purposes. Although the testing position of the test samples was different than an actual upper and lower wingwall, the testing configuration did not affect the performance. The gravity self-weight of the test sample upper wingwall was not acting on the lower wingwall as it would be in the actual construction. The actual wingwall construction orientation would have increased the friction coefficient at the upper and lower wingwall joint. The test configuration models the lateral load on the wingwall accurately but was conservative compared to the actual interface shear between the lower and upper wingwall construction.

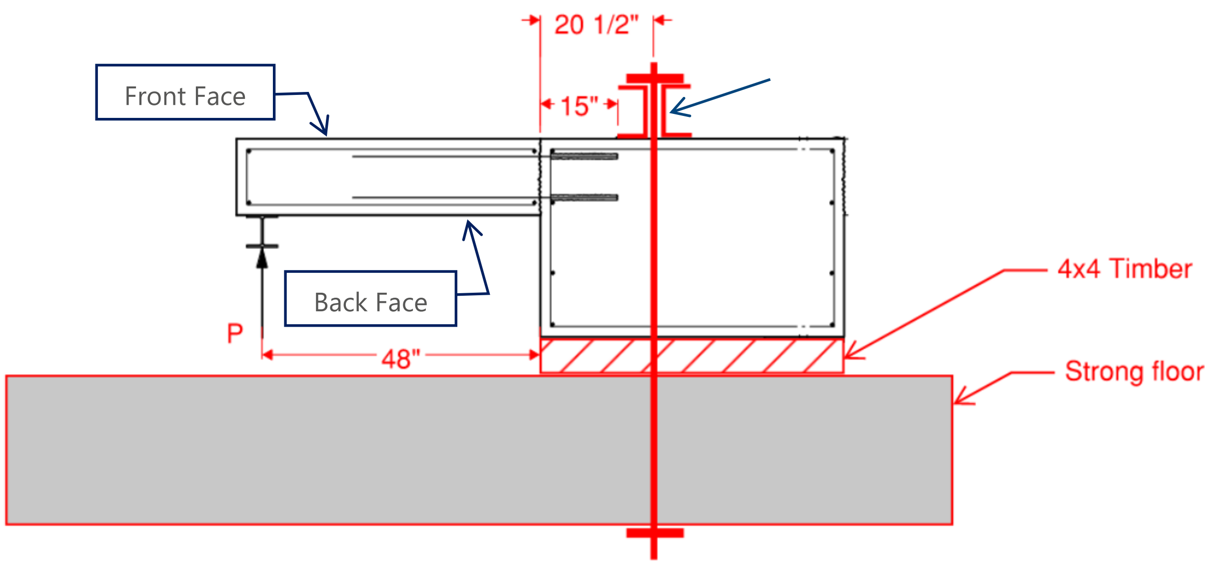

Figure 5.1. Schematic of test setup. Note: the 15-inch dimension indicates distance from the tip of the restraining beam's leg to the joint between upper and lower wingwalls

Hydraulic rams were used to apply load (P) to the upper wingwall while the lower wingwall was restrained using a beam and threaded rods anchored to the laboratory strong floor. The hydraulic rams were positioned 48 inches from the joint between the upper and lower wingwalls and were positioned symmetrical about the center line of the 60-in dimension of the walls.

The beam restraining the lower wingwall was positioned beyond the 15 in. embedment depth of the adhesive anchor so as not to confine concrete in the area of the adhesively anchored reinforcing bars.

Instrumentation was installed to collect data during testing and consisted of a pressure transducer, cable extension transducers (CET), and strain gages. The pressure transducer was in line with a hydraulic pump used for the hydraulic rams. The applied load was calculated by multiplying the magnitude of the pressure by the effective area of the hydraulic rams. The CETs were positioned across the wingwall joint and at the elevation of the back face adhesively anchored reinforcement. Strain gages were installed on the tension side of the adhesively anchored reinforcement approximately 2 inches above the concrete surface of the lower wingwall (Figure 5.1). The instrumentation was connected to a computer-controlled data acquisition system that continuously recorded and displayed their output.

5.3. Test Performance

Testing was conducted on two samples designated as B1 and B2. Load application consisted of using an electric hydraulic pump and two rams positioned 48 inches from the wingwall joint and approximately 26½ inches apart. Load was applied monotonically, load-displacement and strain data were monitored, visual observations were made periodically, and notable events were documented for each test. Load application was discontinued when an increase in load could not be achieved.

5.4. Test Results

The test orientation of the upper wingwall required that its self-weight be subtracted from the measured maximum load to determine the bending moment and shear force at the joint between the upper and lower wingwalls. The performance of both test walls was similar in that the applied ultimate load values, and load-displacement and load-reinforcing bar strain characteristics were similar. The maximum load achieved (applied load minus self-weight) for Wall B1 was 49,700 lbf and for Wall B2 was 50,610 lbf. The failure mode for both wingwall samples was first yielding of the tension reinforcing steel (back face reinforcing) and then concrete cracking/crushing of the compression zone of the bending moment on the lower wingwall.

The load-displacement plots show three distinct regions of behavior (Figure 5.2 and Figure 5.3). The first region is between zero and approximately 13,800 lbf. This region showed linear behavior with little to no displacement and no observed distress in the concrete. The second region is between 13,800 lbf and approximately 48,000 lbf for Wall B1 and 52,000 lbf for wall B2. This region is where the wingwall joint gradually opened to 0.065-in for Wall B1 and 0.070-in for Wall B2 and eventually the concrete began to crack in the compression zone of the bending moment. The third region is where an increase in wingwall joint opening occurs with little to no increase in applied load. This region is where concrete cracking continued, and the reinforcing bar stress approached the ultimate stress of the reinforcing and where there is possible bond failure of the adhesively anchored reinforcing bars.

The reinforcement strain data for the back face reinforcement for both wall samples were in tension throughout the test. The front face reinforcement started in compression until the wingwall joint opening became so wide and propagated to the depth of the front side reinforcement placing this reinforcement in tension. The tension reinforcement stress increased above yield as the maximum applied load was reached.

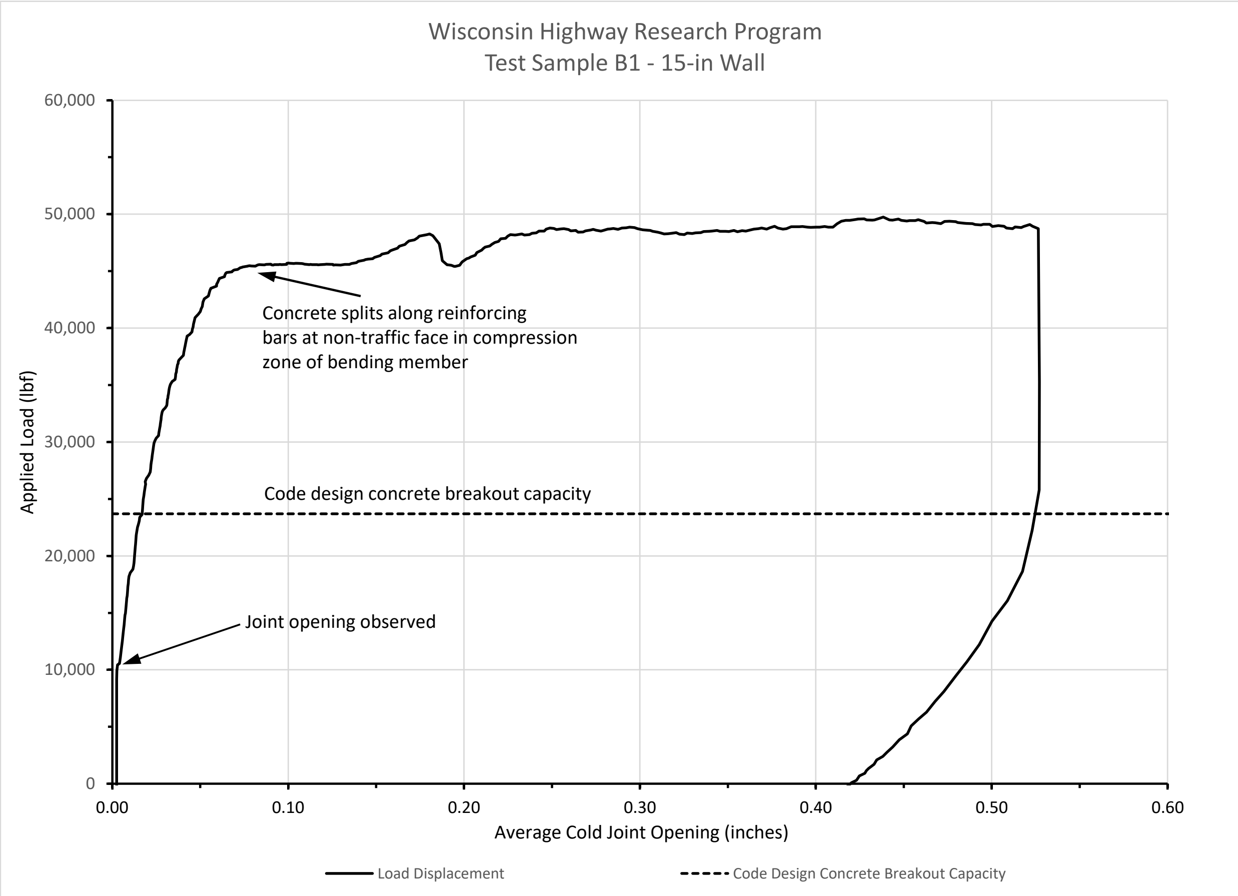

Figure 5.2. Load-displacement plot for Wall B1

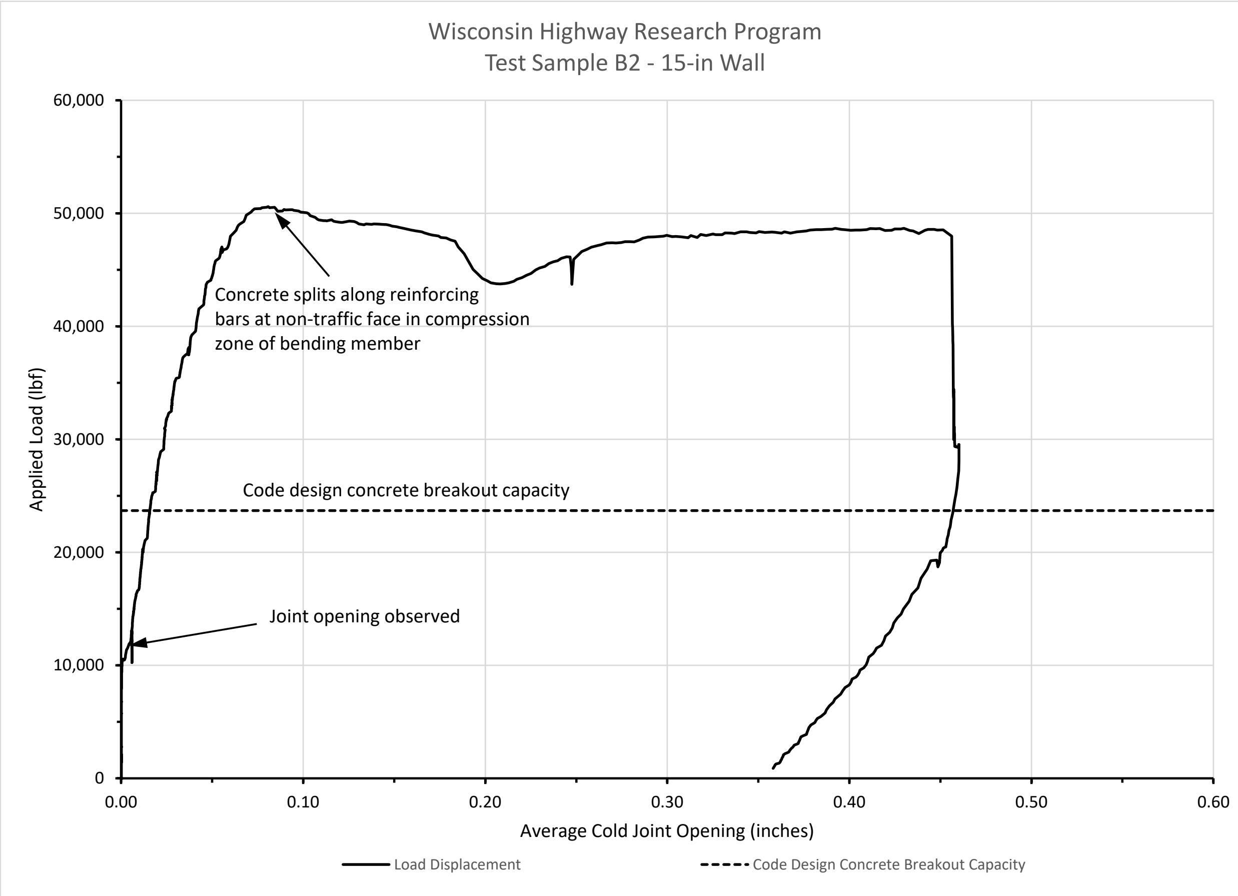

Figure 5.3. Load-displacement plot for Wall B2

5.5. Wall Capacity Calculations

Calculations of the wall capacity were made based on ACI design equations without using strength reduction factors (ϕ factors) to determine code design capacity of the wall for different failure modes of the adhesively anchored reinforcement. The failure modes considered were concrete breakout in tension, reinforcing steel yield and fracture, and bond failure. The controlling calculated design strength failure mode for the adhesively anchored reinforcing bars was a tension concrete breakout. The calculations for these failure modes are included in Appendix D. The calculated code design capacity strengths are included on the load-displacement plots in Figure 5.2 and Figure 5.3.

Based on the calculated code design concrete breakout failure mode, the ACI design equations resulted in an anticipated maximum test load of 23,700-lbf at a loading distance of 48-in from the wingwall joint, which results in an anticipated maximum test moment of 94,800 lbf-ft.

5.6. Observations and Discussion

The following are observations made from the wingwall testing and a discussion of the ACI design equations.

- The embedment depth for the adhesively anchored No. 6 reinforcing bar was 20 times the anchor diameter (15-in) per Chapter 40 of the Wisconsin Bridge Design Manual. This embedment depth is less than the calculated code development length for a No. 6 epoxy coated reinforcing bar. This implies the bond strength of the adhesive used (Simpson AT-XP, which had the lowest listed bond strength in the WisDOT APL) can develop the yield strength of a No. 6 epoxy coated reinforcing bar at 15-in embedment when installed in dry, uncracked concrete.
- The capacity of a conventionally reinforced upper wingwall cast against a lower wingwall using the concrete strengths achieved for the test walls and the mill report reinforcement bar yield strength (f_y = 68.7 ksi) was also determined. The capacity for a conventionally reinforced wingwall calculated to be 33,580-lbf. The moment capacity calculated to be 134,300 lbf-ft.
- The failure mode observed during the 15-in wingwall tests is the same as would be expected when designing cast-in-place reinforced concrete. The maximum load achieved for each test sample was approximately 150 percent greater than the calculated flexural yield capacity of a similar conventionally reinforced wingwall configuration.
- The minimum code requirements for reinforced concrete design use a concept to keep probability of failure low by keeping the load resistance greater than the load demand. The variation that could occur in loads is managed using load factors. The variation that could occur in capacity is managed using strength reduction factors and selection of a characteristic design value that is less than the average resistance capacity. The goal is to minimize the overlap of the potential load demand and the capacity (Figure 5.4). Part of the ACI design equation strength reduction factors is the coefficient, k_c, used in determining the basic concrete breakout strength. The ACI design equation for concrete breakout is based on research and a large data set of test results. From this data set the coefficient, k_c, was determined for uncracked concrete to be 35 based on ultimate load values. The design concept for concrete anchorage that the capacity exceeds the load demand is based on the 5 percent fractile for k_c, which is 24, such that there is a 90 percent confidence that 95 percent of the actual strength will be exceeded. This results in the ACI design equations being conservative as observed from the test results.

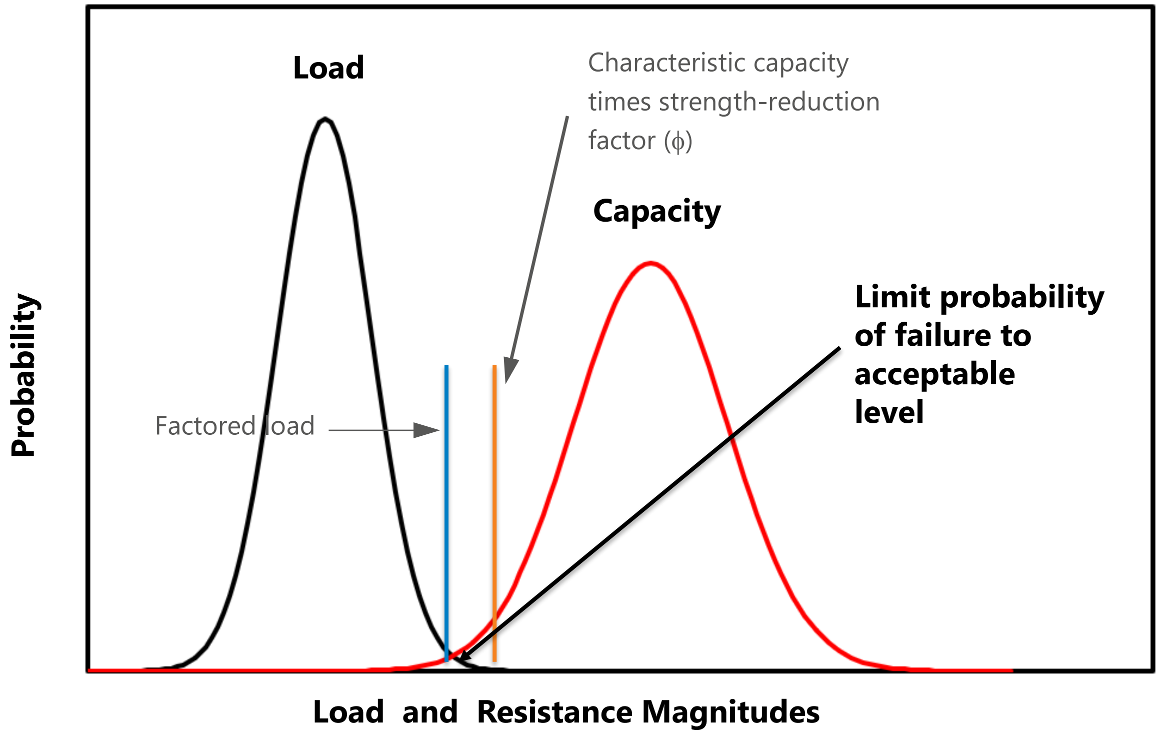

Figure 5.4. Concrete design philosophy of capacity exceeding load demand

- Although the test samples experienced steel yielding before concrete crushing, concrete breakout was the calculated controlling failure mode based on ACI 318-19 design equations. The maximum load achieved was approximately 210 percent of the calculated design concrete breakout capacity using a k_c coefficient of 24, as prescribed by ACI 318-19. The maximum test loads were approximately 145 percent of the concrete breakout calculation using a k_c coefficient of 35. This is shown in Figure 5.5 and Figure 5.6 as Concrete Breakout Capacity. Also shown in Figure 5.5 and Figure 5.6 are loads that correspond to reinforcing bar yield and ultimate capacity.

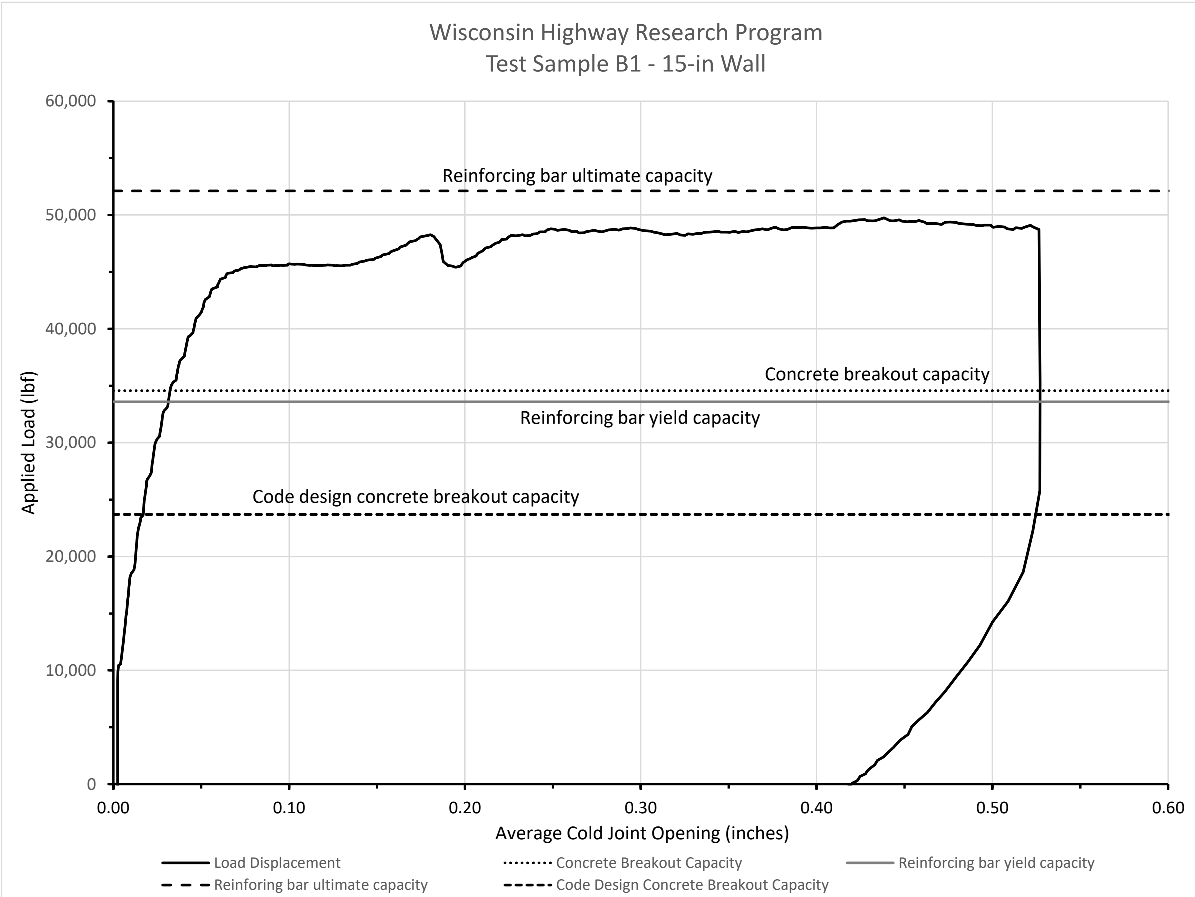

Figure 5.5. Load-displacement data for Test Wall B1 with capacity for code design concrete breakout capacity, concrete breakout capacity, reinforcing bar yield and ultimate capacity.

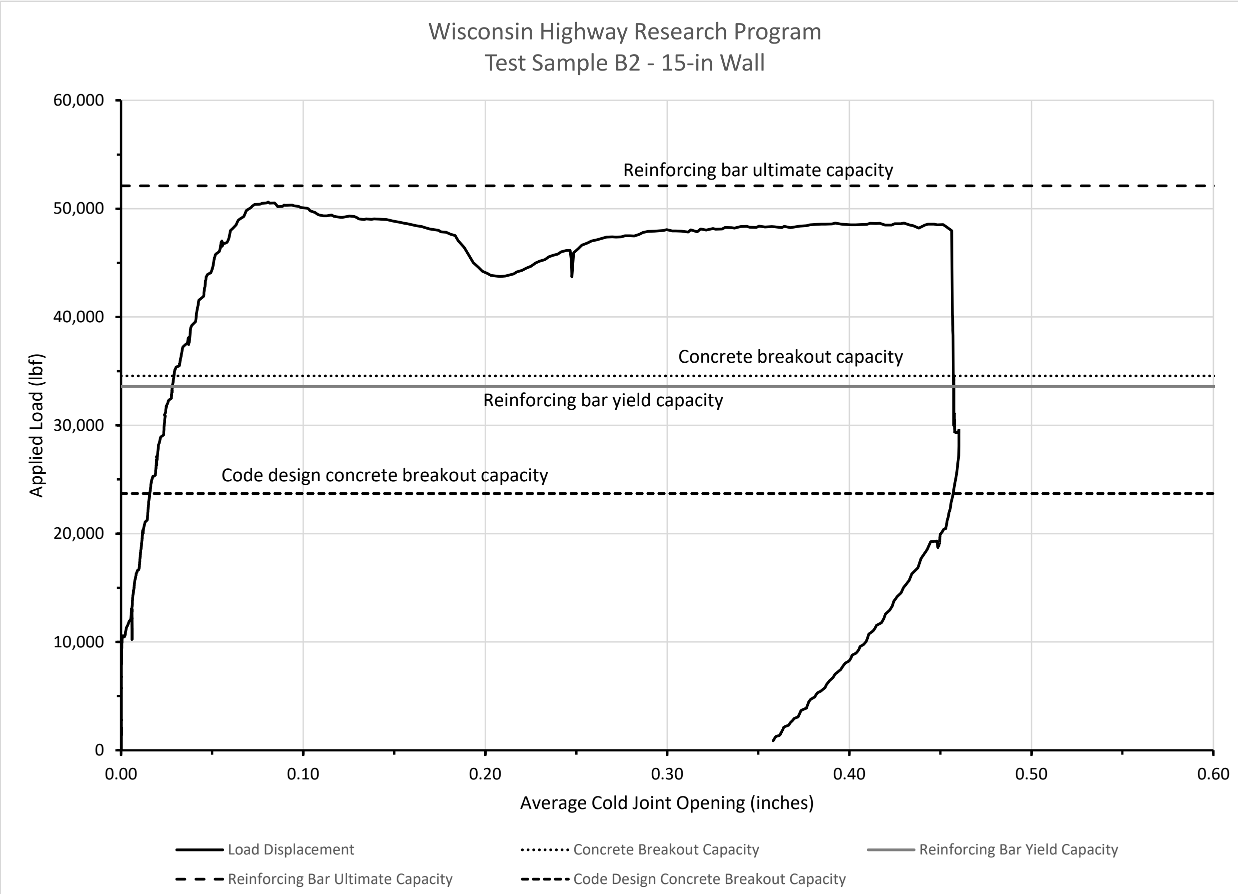

Figure 5.6. Load-displacement data for Test Wall B2 with capacity for code design concrete breakout capacity, concrete breakout capacity, reinforcing bar yield and ultimate capacity.

- The database used to develop the ACI design equations is based on tensile testing of anchors. The testing for this research program created a case where the tension anchors are influenced by the compression force developed by the bending moment. The authors believe this compression force has some contribution to the test samples exceeding the code design concrete breakout value and that the ultimate applied load exceeds both the steel yield capacity and the concrete breakout capacity. Research has been performed and published on the influence of bending compression force [10].
- The ACI design equations also use a concrete breakout area based on anchor embedment depth and an assumed projected failure cone of 35 degrees. To account for a group of anchors, a ratio is determined from the concrete area of the anchor group to the concrete area considering the spacing and edge conditions of the anchor group to that of a single anchor without influence of edges. This ratio value (> 1) is applied to the basic concrete breakout equation for a single anchor. This approach is applicable to embedment depths up to approximately $h_{ef} = 7d_a$. Unfortunately, this calculation approach has not been calibrated for deeper embedments experienced in these tests and calculation results in conservative estimations of concrete breakout capacity. The test results for this research program demonstrate that the design equations are conservative for concrete breakout capacity with deep embedments.

CHAPTER 6. SUMMARY AND CONCLUSIONS

A research program was initiated to provide simplified design guidance for adhesive anchor use on WisDOT projects, commensurate with the current WisDOT approved products. The research program consisted of a literature review, written survey of state DOTs, laboratory testing of wingwalls simulating an upper wingwall replacement, review of WisDOT policy, and recommendations based on findings. Below are recommendations.

- Allow adhesive anchors in parapets with strict adherence to manufacturer installation instructions and design procedures following AASHTO and ACI with AASHTO designed anchor capacity limited by anchor spacing. Dynamic increase factors (DIF) may be considered when determining strength of adhesive anchors under impact loadings, but further research is needed to determine DIF values for anchor bond strengths.
- Allow use of adhesive anchors in sustained tensile loading applications in accordance with AASHTO LRFD-9 for potential cost-saving applications.
- Allow adhesive anchors in overhead or upwardly inclined installations with the requirement that the installation be performed by an ACI certified adhesive anchor installer, continuous inspection of installation is performed, and proof load a percentage of installed anchors.
- Consider alternative design approaches to utilize products with high bond strengths. In lieu of an alternative design approach, it is recommended that the WisDOT current minimum characteristic bond stress table (Table 40.16-1) be accompanied by a list of assumptions used to compile it including strength reduction factors, temperature range and type of inspection (periodic or continuous).
- Remove the WisDOT Standard Specification Section 502.3.12 that states the drilled holes are to be cleaned by flushing with water followed with air blow until the hole is dry and dust-free. This section should be replaced with a statement that adhesive anchor installation shall strictly follow the manufacturer's printed installation instructions (MPII).
- Allow adhesive anchors in abutment wingwall replacement.
- Allow adhesive anchors for replacement of concrete parapet that is to be connected to an existing concrete bridge deck.

Based on the findings of the research program, the following are suggestions for further research:

- Validate the design procedures proposed in the current research for concrete parapet replacement with full scale testing. The tests performed for this research program indicated that current ACI design procedures for adhesive anchors can be applied for wing wall replacement with reasonable conservatism. However, the level of conservatism may or may not be the same for other applications such as concrete parapet replacement.
- Further investigate the use of adhesive anchors for abutment and/or pier extension. The design example provided in this research indicated that the design strengths of abutment sections using adhesive anchors are much lower than the original design strength assuming fully developed reinforcement. However, this does not necessarily preclude potential values of adhesive anchors in this application for several reasons. First, the design equations may be over-conservative. Second, the

required strength of the abutment section, which is not well understood, could be significantly lower than the original design strength and may be potentially met by the strength of the rehabilitated abutment section. Improved understanding of the design loads and required strength of abutment would be beneficial in evaluating potential values of using adhesive anchors.

- Investigate the effect of reinforcement bar coatings on the bond strength of adhesives. Bond strength factors for reinforcement coatings are currently not addressed in code approval testing or code design equations.
- Investigate effect of impact loadings on anchor bond strengths. While literature indicates that a DIF can be applied to calculating anchor strengths associated with concrete and steel failures under impact/dynamic loads, there is little information on effect of impact/dynamic loads on anchor bond strengths.

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APPENDIX A. DOT SURVEY

Appendix A1 – DOT Survey Questionnaire

Appendix A2 – Summary of DOT Survey Results

Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, Illinois 60062
847.272.7400 tel
www.wje.com

Wisconsin Highway Research Program Survey
Development of Design Procedures for Concrete Adhesive Anchors

This survey is part of an ongoing research project funded by the Wisconsin Highway Research Program (WHRP), "Development of Design Procedures for Concrete Adhesive Anchors" (0092-21-01). The survey is expected to take approximately 15 minutes. We appreciate your time and feedback in completing the following form. Please email the completed form to the following contact:

Le Pham
Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, IL 60062
Email: LPham@wje.com
Phone: (847) 753-6449

1. Please select applications in which concrete adhesive anchors are allowed by your agency.

☐ Crashworthy traffic railing/parapet/barrier (permanent)

☐ Crashworthy traffic railing/parapet/barrier (temporary)

☐ Non-crashworthy traffic railing/parapet/barrier (permanent)

☐ Non-crashworthy traffic railing/parapet/barrier (temporary)

☐ Abutment wingwall extension/replacement

☐ Abutment backwall extension/replacement

☐ Bent/pier cap extension

☐ Others. Please describe: ____

____

2. Does your agency allow the use of adhesive anchors in applications with sustained tension loads?

○ No.

○ Yes, regardless of loading level.

○ Yes, if the sustained load does not exceed a certain limit. Please indicate the load limit:

____

○ Other. Please describe: ____

3. Does your agency allow the use of adhesive anchors installed in an overhead or upwardly inclined position?

- ○ No.
- ○ Yes, with no restrictions.
- ○ Yes, with restrictions. Please describe the restrictions: ____
____
- ○ Other. Please describe: ____
____

4. Does your agency allow the use of adhesive anchors for permanent replacement of crashworthy traffic railing/barrier attached to a bridge deck?

- ○ Yes.
- ○ No, but we are considering using them in the future.
- ○ No, and we are not considering using them in the future.
- ○ Other. Please describe: ____

5. Does your agency have design guidelines for concrete adhesive anchors that differ from the guidelines given in AASHTO LRFD-9 Section 5.13 and ACI 318-19 Chapter 17?

- ○ No.
- ○ Yes. Please list the document: ____

6. Please select the option that best describes your agency's practice of specifying the characteristic bond stress for the design of adhesive anchors.

- ○ Only characteristic bond stresses specified in design guidelines are used.
- ○ Only characteristic bond stresses provided by the manufacturers are used.
- ○ Characteristic bond stresses specified in design guidelines or provided by the manufacturers can be used.
- ○ Other. Please describe: ____

7. Please select parameters required to be considered in your design guidelines in order to determine the bond strength of adhesive anchors:

☐ Cracked concrete condition

☐ Concrete moisture condition during installation

☐ Concrete moisture condition in service

☐ Concrete temperature during installation

☐ Concrete temperature in service

☐ Anchor diameter

☐ Anchor spacing

☐ Anchor distance to concrete edge

☐ Anchor group action

☐ Anchor loading condition (e.g. sustained loading, seismic loading)

☐ Anchor coating (e.g. rebar epoxy coating and galvanized coating)

☐ Anchor hole drilling method (e.g. rotary impact drilled vs core-drilled holes)

☐ Chemical exposure

☐ Other. Please describe: ____

8. Does your agency have standard details for concrete adhesive anchors?

○ No.

○ Yes. Please list the document: ____

9. Does your agency require field proof testing of concrete adhesive anchors?

○ No.

○ Yes. Please indicate in what applications and list key testing requirements (e.g proof load, test frequency, etc): ____

____

10. Has your agency conducted or funded any research on concrete adhesive anchors in the last 10 years?

- ○ No.
- ○ Yes. If reports are available, please provide links: ______________________________

11. In your experience, what are the main challenges to using adhesive anchors in bridge structures? (Please provide as much detail as possible; e.g. high costs, lack of quality control procedures, lack of design guidelines for specific applications, etc.)

Please provide your name, agency and department below.

Name: ______________________________

Agency: ______________________________

Department: ______________________________

If you are interested in additional email correspondence and/or a follow-up phone conversation to further assist the researchers in this project, please provide your contact information.

Email: ______________________________

Phone: ______________________________

Thank you very much for your time and input!

Appendix A2. DOT Survey Results

1. Please select applications in which concrete adhesive anchors are allowed by your agency.

Results of Question 1 are provided in Table A2.1, Figure A2.1and Figure A2.2.

Table A2.1. Results of Survey Question #1

Applications	No. of States	Percent of States Responded
Crashworthy traffic railing/parapet/barrier (permanent)	14	54%
Crashworthy traffic railing/parapet/barrier (temporary)	16	62%
Non-crashworthy traffic railing/parapet/barrier (permanent)	11	42%
Non-crashworthy traffic railing/parapet/barrier (temporary)	12	46%
Abutment wingwall extension/replacement	11	42%
Abutment backwall extension/replacement	11	42%
Bent/pier cap extension	10	38%
Other[1]	13	50%

26 states responded to this question.

[1] Responses:

- *Michigan: Crashworthy Traffic Railing etc. on Non-NHS routes only. Bridge sign connections. Pavement lane ties.*
- *Iowa: In the past, we have allowed adhesive anchors for bridge mounted sign supports. Current usage is very limited due to the lack of quality control and the availability of certified installers.*
- *Illinois: We use adhesive anchors on piers and other substructure components for connections where minimal tension is anticipated.*
- *Minnesota: We allow adhesive anchors for infill walls, crash struts, and limited sustained tension applications. We do not allow adhesive anchors as primary reinforcement in pier cap overhangs or as primary reinforcement in wingwall extensions.*
- *Caltrans: Concrete adhesive anchors are allowed when designed in accordance with ACI 318-14 except that we prohibit sustained tension.*
- *South Carolina: Sidewalk reinforcing attachment, extension of culverts, widening of bridge decks, utility attachments, fencing to barriers.*
- *New Hampshire: NHDOT's Bureau of Bridge Design does not specify these and Bureau of Bridge Maintenance only uses mechanical anchors.*
- *Georgia: Utility retrofits; aesthetic retrofits.*
- *Florida: Replacement of damaged extension bars (lap splice) during for deck widening.*
- *Alaska: Seismic retrofits.*
- *West Virginia: Bearing anchor bolts.*
- *New York: I assume by adhesive anchors you are referring to chemical anchors and not cementitious. We allow chemical anchors in temporary applications as part of an approved PE design. See NYSDOT Specification 586.*

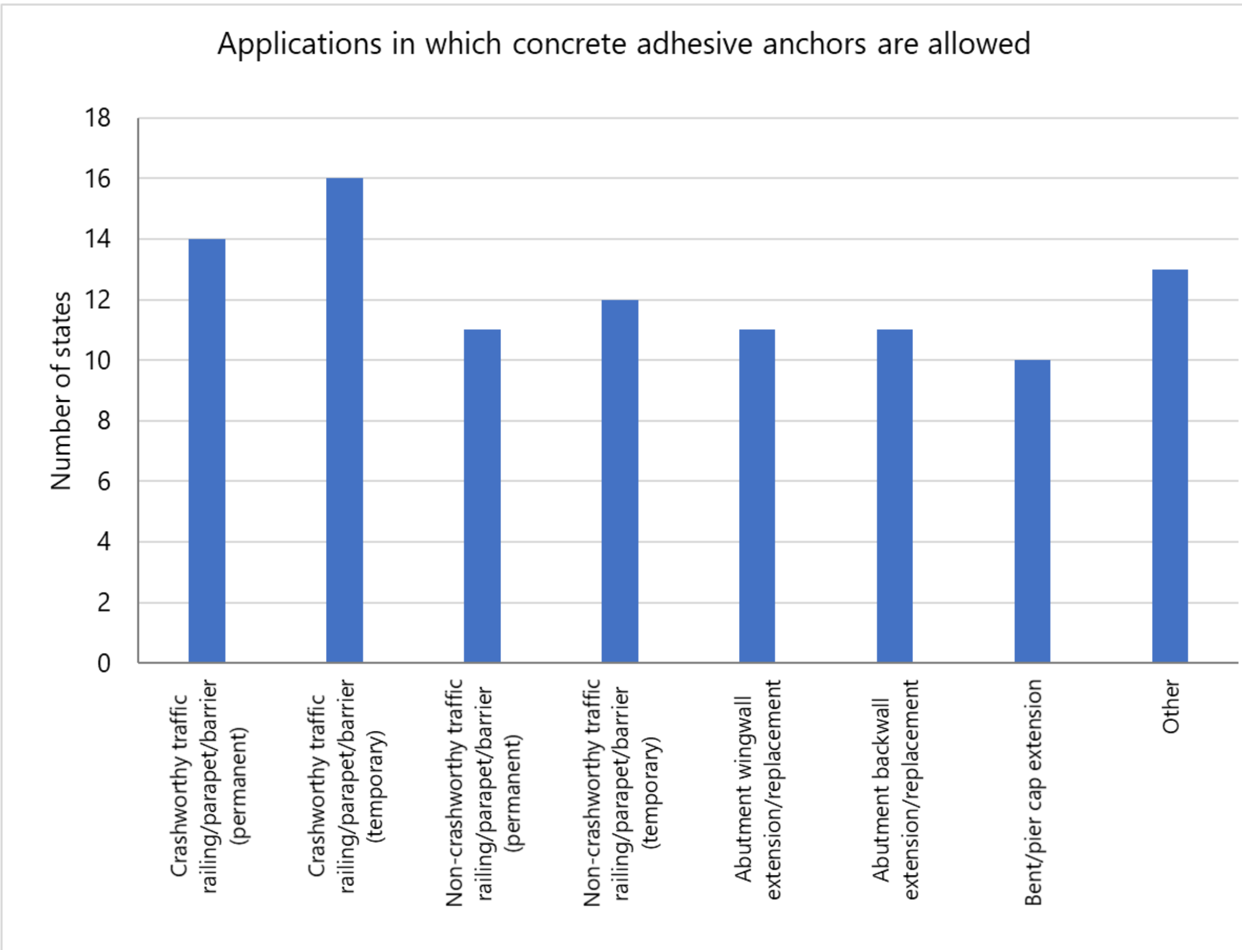


Figure A2.1. Results of Survey Question #1 (By Number of States)

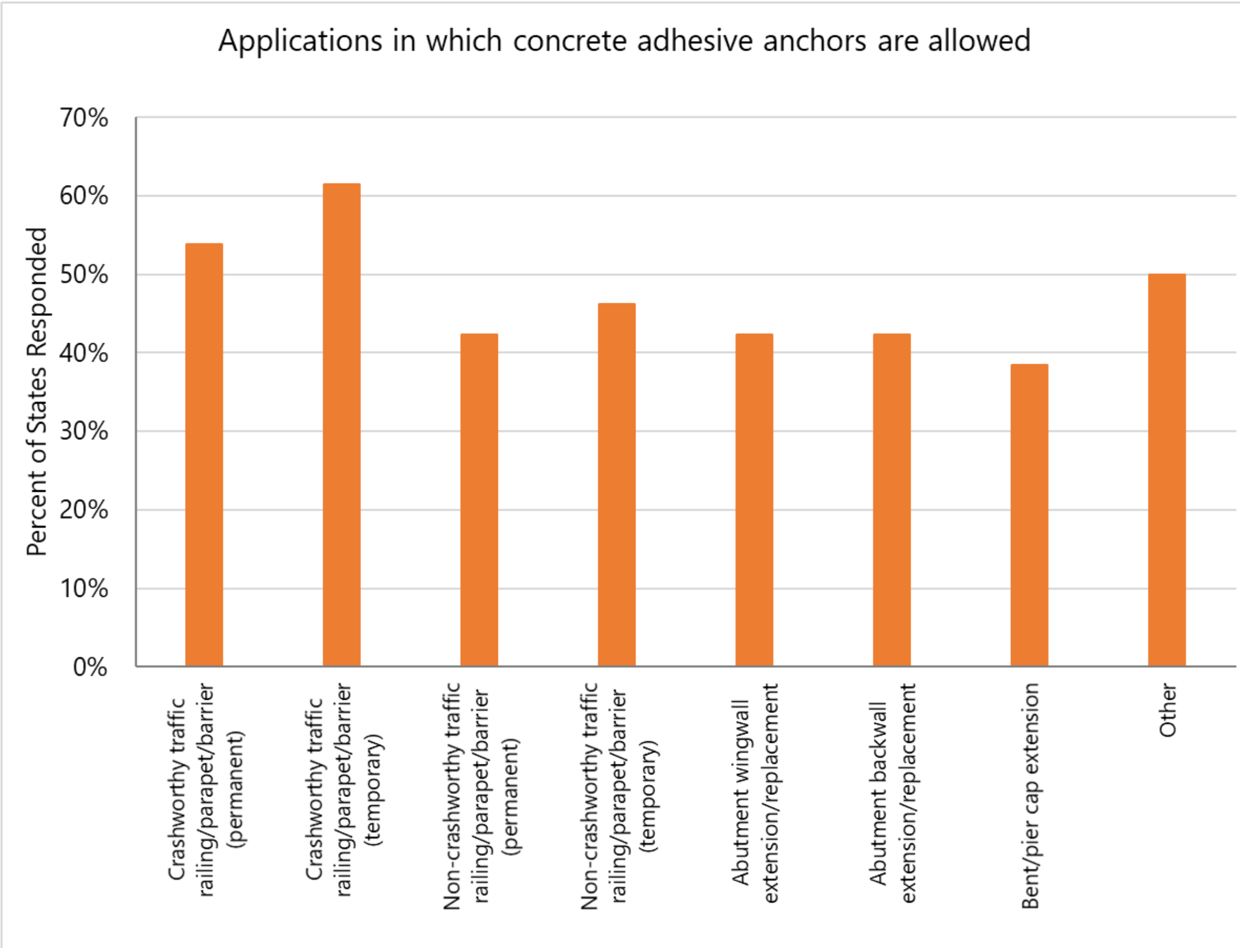


Figure A2.2. Results of Survey Question #1 (By Percent of States Responded)

2. Does your agency allow the use of adhesive anchors in applications with sustained tension loads?

Results of Question 2 are provided in Table A2.2, Figure A2.3 and Figure A2.4.

Other applications mentioned in the responses include:

- When allowed, strength design of anchors shall comply with ACI 318.
- This application has been discouraged in the past but is not currently prohibited.

Table A2.2. Results of Survey Question #2

Response	No. of States	Percent of States Responded
Yes, regardless of loading level (except per ACI 318)	4	15%
Yes, if the sustained load does not exceed a certain limit (other than per ACI 318) [1]	2	8%
No	17	65%
Other [2]	3	12%

26 states responded to this question.

[1] Minnesota: The sustained tension cannot exceed half the total factored anchor bond resistance.

Florida: Thirty percent of Factored Load Resistance in some cases.

Tennessee: It varies, and we test to manufacturers recommendations, then use accordingly.

[2] Responses:

- *Iowa: When allowed, strength design of anchors shall comply with ACI 318.*
- *Indiana: This application has been discouraged in the past but is not currently prohibited.*
- *Oregon: We design adhesive anchors mostly according to ACI 318 Ch 17. Design of anchors subjected to sustained tension loads will include significant strength reduction factors, therefore the anchors will see small, factored sustained tension loads.*
- *New York: Temporary applications only.*

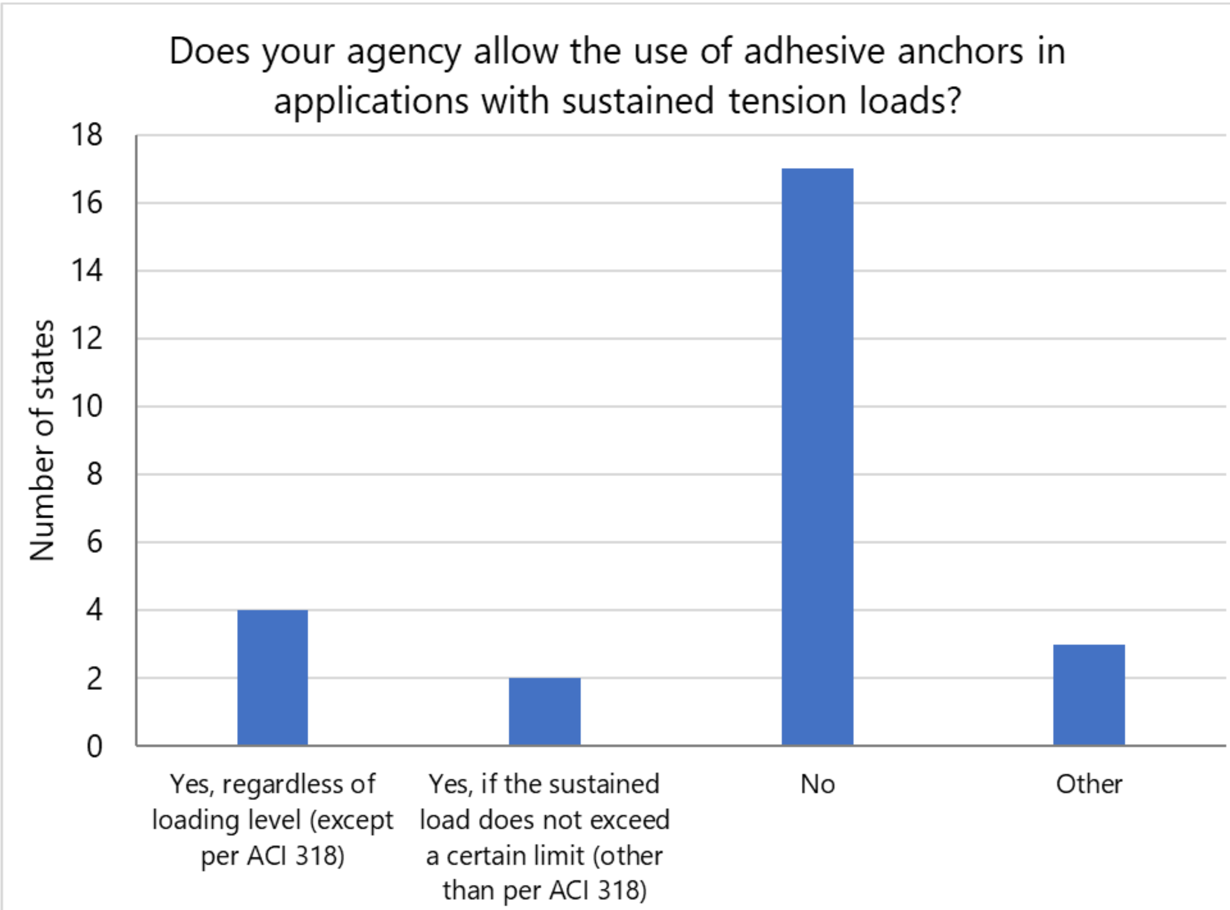


Figure A2.3. Results of Survey Question #2 (By Number of States)

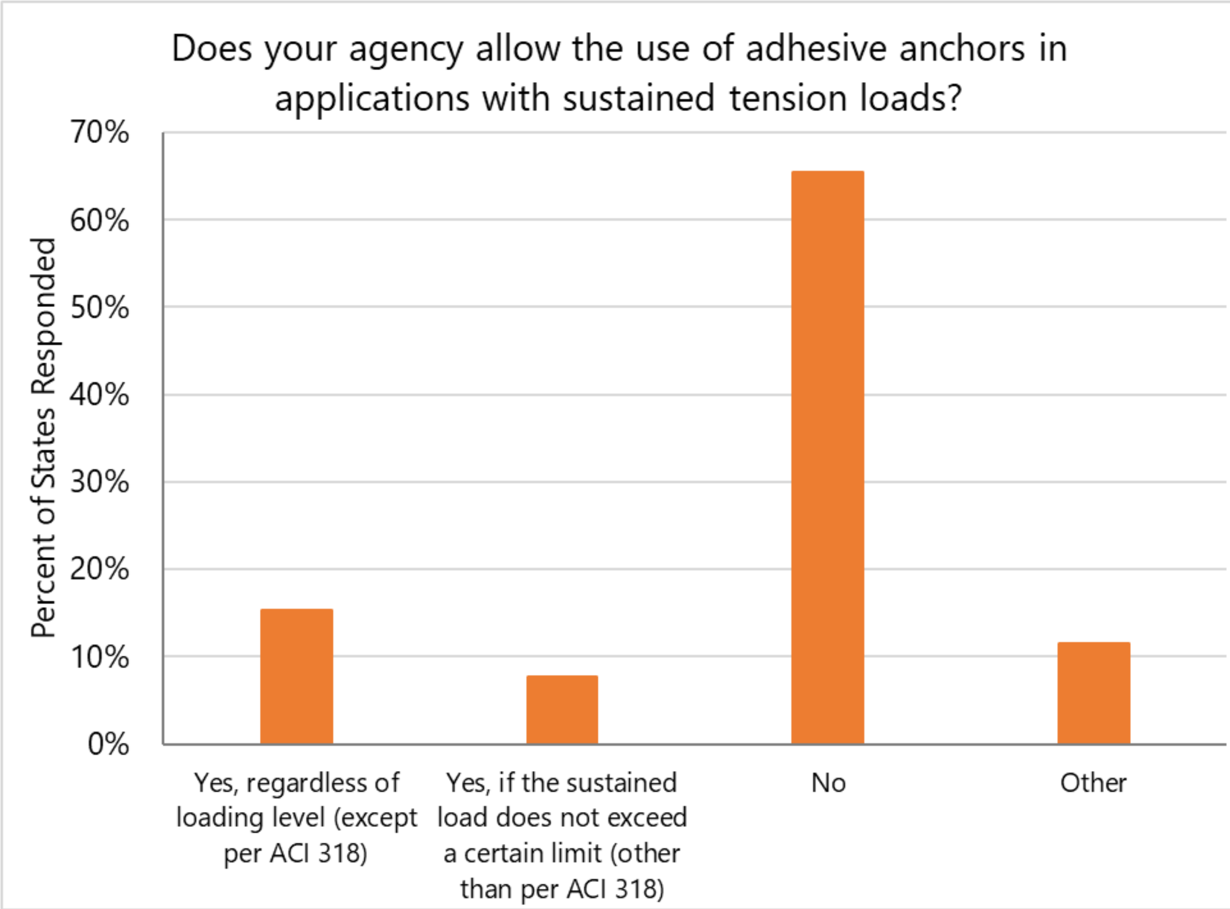


Figure A2.4. Results of Survey Question #2 (By Percent of States Responded)

3. Does your agency allow the use of adhesive anchors installed in an overhead or upwardly inclined position?

Results of Question 3 are provided in Table A2.3, Figure A2.5, and Figure A2.6.

Other applications mentioned in the responses include:

- When allowed, strength design of anchors shall comply with ACI 318.
- This application has been discouraged in the past but is not currently prohibited.

Table A2.3. Results of Survey Question #3

Response	No. of States	Percent of States Responded
Yes, with no restrictions	1	4%
Yes, with restrictions [1]	7	27%
No	16	62%
Other [2]	2	8%

26 states responded to this question.

[1] Responses:

- *Michigan: The anchors are used for overhead structures; however, they can only be used in holes that are horizontal or angled upward.*
- *Minnesota: We do not allow adhesive anchors in pier cap retrofits, support, or repairs. Abutments paving brackets that support approach slabs over voided abutments; etc.*
- *Texas: Avoid use over traffic.*
- *Georgia: Utility attachments with cross-members below the supported utility to prevent falling below if the adhesive fails.*
- *Tennessee: If used load restrictions based on design and according to manufacturer's recommendations*
- *Oregon: ACI/CRSI certified installers are required for installation of these anchors.*
- *Nevada: No sustained tension.*

[2] Indiana: This application has been discouraged in the past but is not currently prohibited.

Caltrans: Allowed in accordance AASHTO LRFD BDS 8th Edition, ACI 318-14, and ACI 355.4 and Caltrans policy

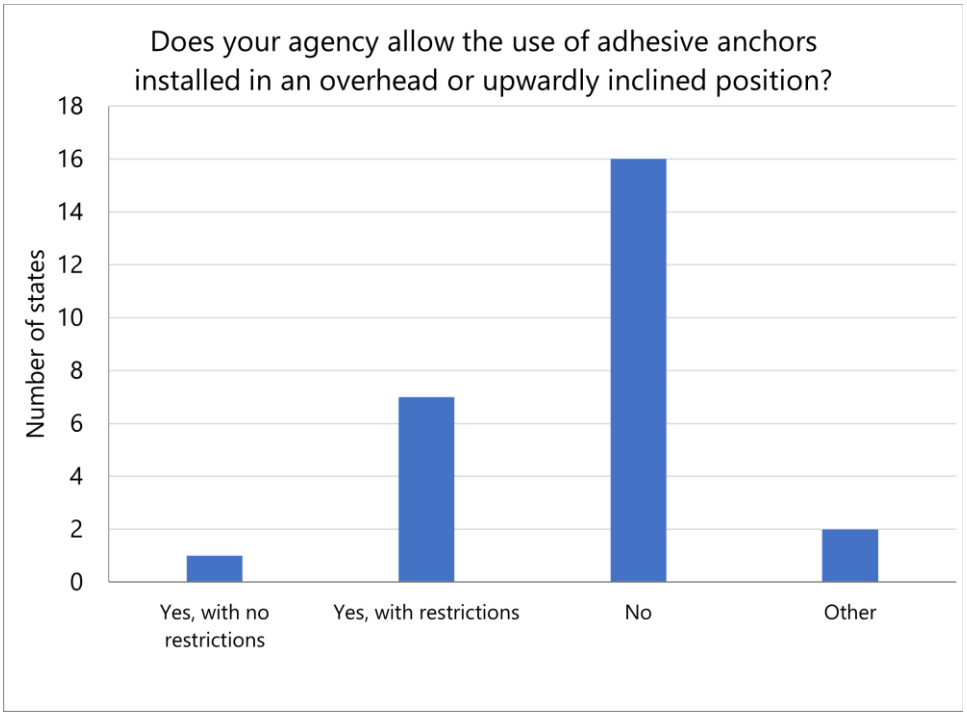


Figure A2.5. Results of Survey Question #3 (By Number of States)

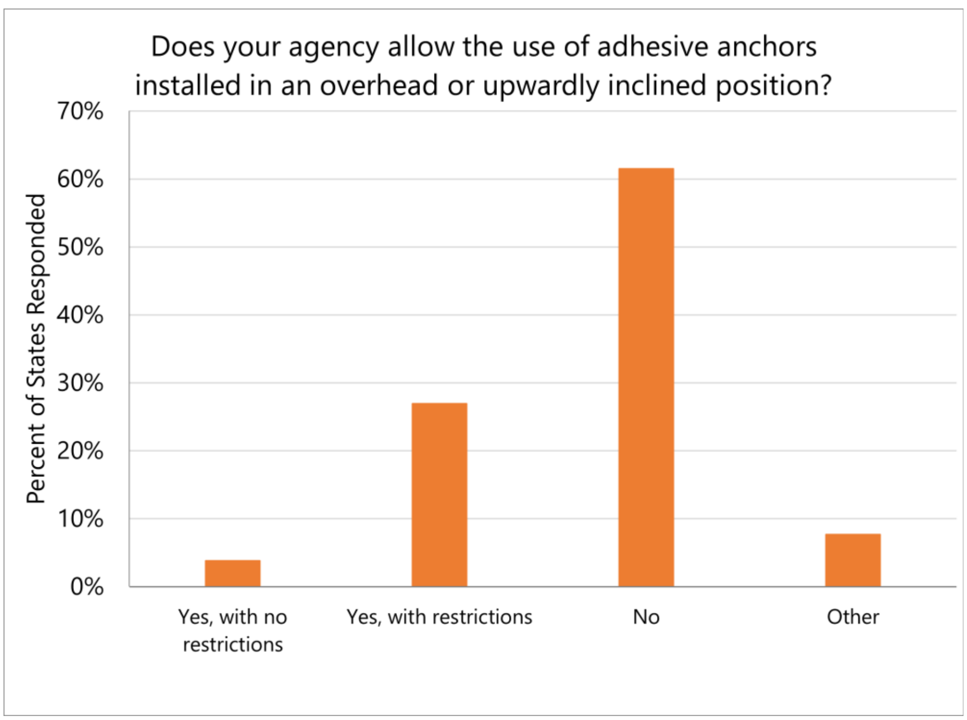


Figure A2.6. Results of Survey Question #3 (By Percent of States Responded)

4. Does your agency allow the use of adhesive anchors for permanent replacement of crashworthy traffic railing/barrier attached to a bridge deck?

Table A2.4. Results of Survey Question #4

Applications	No. of States	Percent of States Responded
Yes	15	58%
No, but we are considering using them in the future.	1	4%
No, and we are not considering using them in the future.	8	31%
Other [1]	2	8%

26 states responded to this question.

[1] Michigan: Non-NHS routes only.

Caltrans: Allowed in accordance AASHTO LRFD BDS 8th Edition, ACI 318-14, and ACI 355.4 and Caltrans policy

Oregon: Yes, but in most cases, since existing deck overhang is usually thinner than modern designed bridge deck overhang, we ended up using thru bolts.

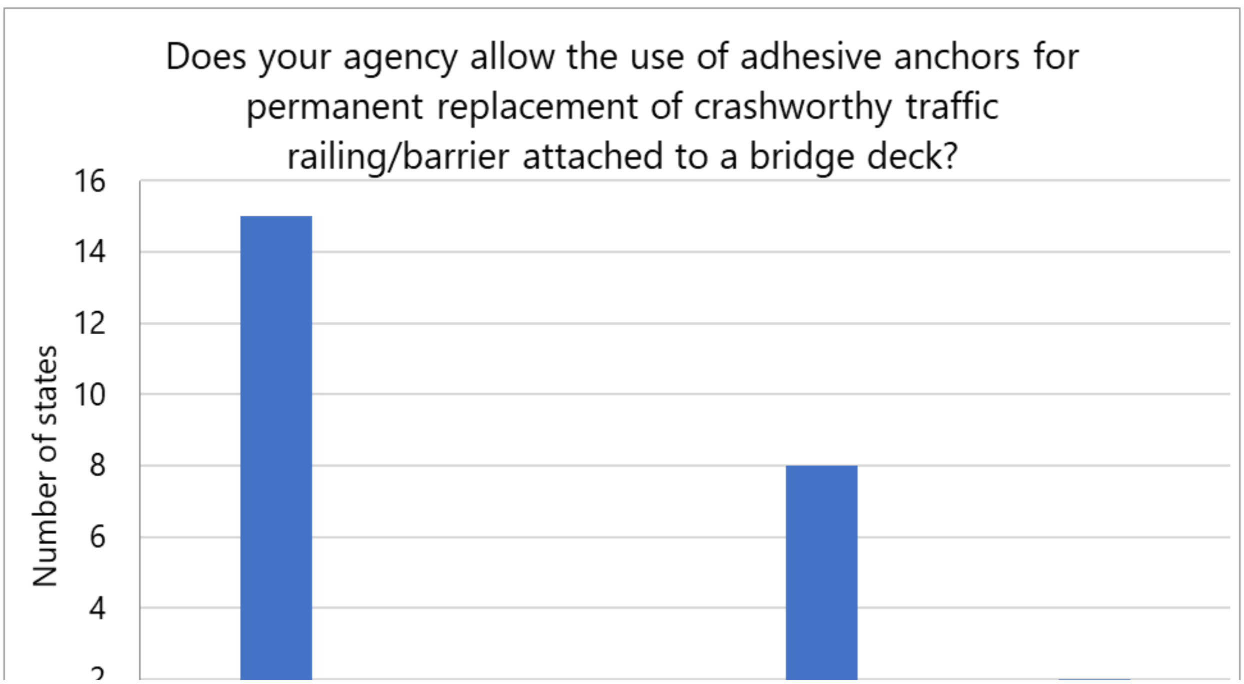


Figure A2.7. Results of Survey Question #4 (By Number of States)

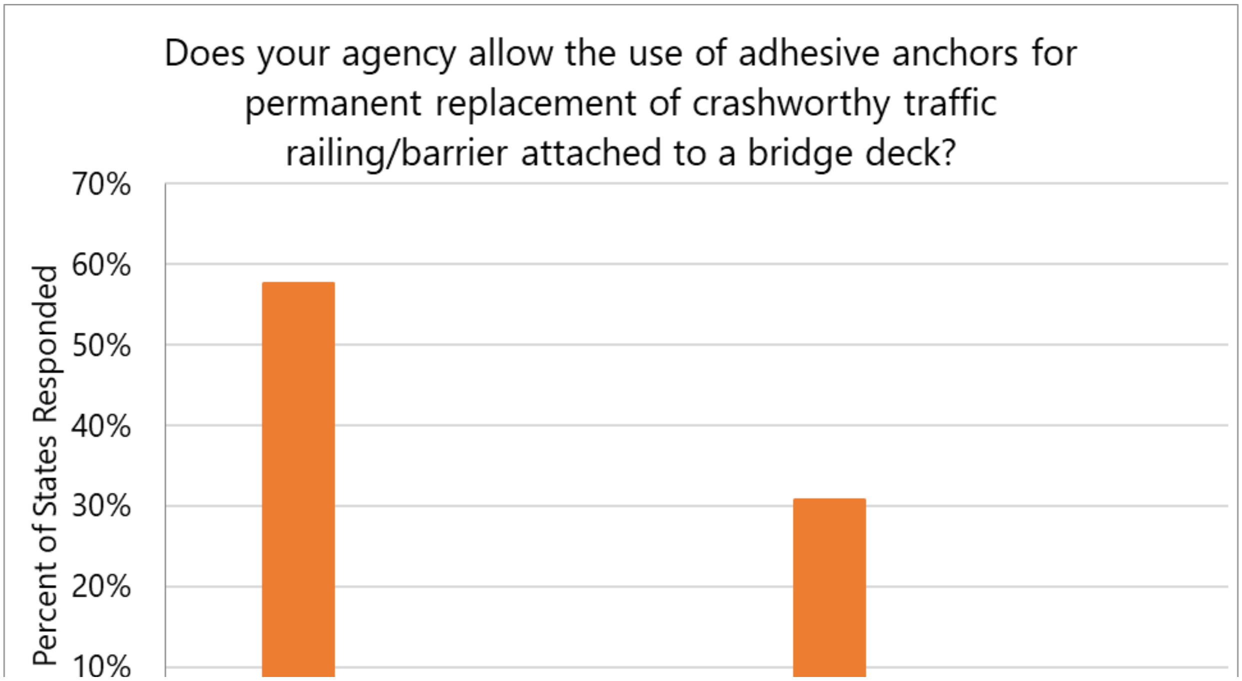


Figure A2.8. Results of Survey Question #4 (By Percent of States Responded)

5. Does your agency have design guidelines for concrete adhesive anchors that differ from the guidelines given in AASHTO LRFD-9 Section 5.13 and ACI 318-19 Chapter 17?

Table A2.5. Results of Survey Question #5

Applications	No. of States	Percent of States Responded
Yes[1]	8	31%
No	17	65%
N/A	1	4%

26 states responded to this question.

[1] Responses:

- *Indiana: Chapter 412 of the Indiana Design Manual*
- *Michigan: We do not have design guidelines that differ necessarily; however, we specify installation depth.*
- *Minnesota: MnDOT LRFD Bridge Design Manual, MnDOT Technical Memorandum 18-11-B-01, and SB provisions list the deviations.*
- *Illinois: We follow AASHTO but set anchors per 509.06 and 1027.01 of our standard specifications. We also have a Qualified Product list for chemical adhesives on our website*
- *Caltrans: Structure Technical Policy 5.50: https://dot.ca.gov/-/media/dot-media/programs/engineering/documents/structure-technical-policy/section-5/202007-stp0550postinstalledadhesiveanchorsinconcrete-a11y.pdf*
- *South Carolina: Bridge Desgin Memorandum DM0408 - Adhesively Bonded Anchors and Dowels*
- *Florida: Structures Manual, Volume 1 - Structures Design Guidelines, Section 1.6.2. https://www.fdot.gov/structures/structuresmanual/currentrelease/structuresmanual.shtm*

- *Oregon: We refer to both codes, but modified some parameters, i.e. characteristic bond strength (based on adhesive products on our QPL), resistance factor for Extreme Event II (not available in ACI 318 nor AASHTO LRFD).*

6. Please select the option that best describes your agency's practice of specifying the characteristic bond stress for the design of adhesive anchors.

Table A2.6. Results of Survey Question #6

Applications	No. of States	Percent of States Responded
Characteristic bond stresses specified in design guidelines or provided by the manufacturers can be used.	4	15%
Only characteristic bond stresses provided by the manufacturers are used.	11	42%
Only characteristic bond stresses specified in design guidelines are used.	5	19%
Other[1]	6	23%

26 states responded to this question.

[1] Minnesota: Typically, we use specified design strengths and vet adhesives to ensure they meet those guidelines. We do, in rare cases, allow the manufacturers strength values to be used if the design cannot be satisfied by the strengths we require.

Caltrans: See authorized materials requirements https://dot.ca.gov/-/media/dot-media/programs/engineering/documents/mets/chem-adhesives-criteria-a11y.pdf

New Hampshire: We don't use these.

Alaska: New procedures still in development

Oregon: We use a QPL system, which has about 12 products. We don't know which product that contractor will choose for construction, therefore we analyzed what would be an appropriate number based on the products' ICC-ES report.

New York: Chemical Adhesive products must go through preapproval testing by NYSDOT to be acceptable. Once approved, manufacturers' design tables are typically used.

7. Please select parameters required to be considered in your design guidelines in order to determine the bond strength of adhesive anchors:

Table A2.7. Results of Survey Question #7

Applications	No. of States	Percent of States Responded
Cracked concrete condition	9	35%
Concrete moisture condition during installation	7	27%
Concrete moisture condition in service	1	4%
Concrete temperature during installation	5	19%
Concrete temperature in service	2	8%
Anchor diameter	14	54%
Anchor spacing	15	58%
Anchor distance to concrete edge	16	62%
Anchor group action	9	35%
Anchor loading condition (e.g., sustained loading, seismic loading)	11	42%
Anchor coating (e.g. rebar epoxy coating and galvanized coating)	8	31%
Anchor hole drilling method (e.g. rotary impact drilled vs core-drilled holes)	7	27%
Chemical exposure	1	4%
Other [1]	11	42%

26 states responded to this question.

[1] Responses:

- *Indiana: Manufacturer recommendations must be followed.*
- *Ohio: ODOT does not have design guidelines for adhesive anchors.*
- *Minnesota: Note that the other conditions do affect the bond strength. We treat every anchor as if it is installed in a saturated concrete. We also do not allow epoxy coatings on rebar that is to resist tension loads.*
- *Iowa: Design shall comply with ACI 318.*
- *Illinois: Please see link to our QPL and testing: https://idot.illinois.gov/Assets/uploads/files/Doing-Business/Specialty-Lists/Highways/Materials/Materials-&-Physical-Research/Metals/chemicaladhesives.pdf*
- *Caltrans: All the parameters listed and any others required by the design specifications*
- *South Dakota: N/A - No set guidelines in South Dakota*
- *New Hampshire: We don't use these.*
- *Georgia: Rely on requirements/guidelines of manufacturer.*
- *Alaska: Still in development*
- *Alabama: Not addressed in design guidelines.*

8. Does your agency have standard details for concrete adhesive anchors?

Table A2.8. Results of Survey Question #8

Applications	No. of States	Percent of States Responded
Yes[1]	4	15%
No	20	77%
N/A	2	8%

26 states responded to this question.

[1] Responses:

- *Indiana: Indiana Design Manual Fig. 412-3B lists design data for anchor systems.*
- *Ohio: Standard Bridge Drawing VPF-1-90*
- *South Carolina: SCDOT Bridge Drawings 700-04 (General Notes and Details for Flat Slabs) & 700-05 (General Details)*
- *Florida: Some Standard Plans have predesign Adhesive Anchors such as Index 102-110 (Page 2); 515-052; 515-062; 515-070 & 515-080: https://www.fdot.gov/design/standardplans/current/default.shtm*

9. Does your agency require field proof testing of concrete adhesive anchors?

Table A2.9. Results of Survey Question #9

Applications	No. of States	Percent of States Responded
Yes[1]	11	42%
No	12	46%
N/A	3	12%

26 states responded to this question.

[1] Responses:

- *Wisconsin: Pullout tests are only required if the field engineer suspects improper installation. 3 tests per bar size, with up to 5% of additional bars if necessary. Tests are to 80% of bar yield stress. https://wisconsindot.gov/rdwy/stndspec/ss-05-02.pdf*
- *North Dakota: Anchor bolts for rail systems are load tested. The first 4 anchors installed are tested and then 10 percent of the remainder are tested.*
- *Michigan: We require proof testing prior to any adhesive anchoring on a project. We conduct proof testing on a per contractor, per adhesive system, per project basis. We then conduct field testing on a random selection of anchors placed during production. This would be in for all structural applications.*
- *Minnesota: The proof load is to the design strength of the anchor excluding group effects. We do not proof beyond 80% of the anchor rod capacity. We test 10% of anchorages for threaded rods and 2% of anchorages for rebar applications. Sustained tension anchorages are tested at 15% and we are adding provisions for continuous inspection.*
- *Texas: Proof load, 5% test frequency*
- *South Carolina: Test a minimum of 1 anchorage but not less than 10% of all anchors in the LOT to the test load shown on the Plans. If less than 60 anchorages are to be installed: Install and test the minimum required*

number of anchorages prior to installing the remaining anchorages. After installing the remaining anchorages, test a minimum of 2 of these anchorages at random locations selected by the RCE. If more than 60 anchorages are to be installed: Test the first 6 anchorages prior to installing the remaining anchorages. Then test, at random locations selected by the RCE, 10% of the number in excess of 60 anchorages. For every failed field test, perform two additional field tests on adjacent untested anchors or dowels within the LOT. Continue additional field tests until no more test failures occur, or until all anchors and dowels within the LOT are tested.

- *Florida: For Traffic Railing Installations only. Proof Testing at 4% frequency on LOT basis. Other applications at discretion of Engineer. See Standard Spec 416-6 for more details: https://www.fdot.gov/programmanagement/Implemented/SpecBooks/default.shtm*
- *Virginia: Min. pull-out of 32,000 lbs for anchor system for temporary traffic barrier.*
- *Tennessee: Pull test on steel railings*
- *Oregon: We require 2 tests during construction depending on how significance of the anchor application, i.e. Demonstration test - confined test to failure (specified min. pullout strength), 3 tests/lot. Production test - confined test to 50% of min. pullout strength, hold for 10 seconds, 1 test/50anchors/shift*
- *New York: See the latest NYSDOT Specification 586. https://www.dot.ny.gov/main/business-center/engineering/specifications/updated-standard-specifications-us*

10. Has your agency conducted or funded any research on concrete adhesive anchors in the last 10 years?

Table A2.10. Results of Survey Question #10

Applications	No. of States	Percent of States Responded
Yes[1]	3	12%
No	19	73%
N/A	4	15%

26 states responded to this question.

[1] Florida: BDV28 977-06: Confinement Effect of Metal Railing Narrow Baseplates on Adhesive Anchor Breakout Resistance. https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/research/reports/fdot-bdv28-977-06-rpt.pdf

11. In your experience, what are the main challenges to using adhesive anchors in bridge structures?

Table A2.11. Results of Survey Question #11

Agency	Responses	Type of Challenges			
		Design	Construction/ Quality Control	Performance	Cost
WisDOT	ACI isn't really geared towards bridge applications.	x			
Indiana DOT	Lack of design guidelines for specific applications has been a challenge. For example, there is sufficient guidance in AASHTO/ACI for development of new reinforcing into existing concrete, but we often use that reinforcement to transfer tensile stresses that are carried by existing reinforcement. It doesn't appear the this "splice" application is clearly covered by the codes.	x			
North Dakota DOT	Locations that are difficult to load test.		x		
Ohio DOT	The number of different products with all of the different strengths (especially characteristic bond strength) and individual requirements to obtain those strengths are overwhelming and makes specifying acceptable products difficult. Typically, ODOT accepts only material that has been evaluated by the ICC-ES. ODOT also does not have standardized construction and material specifications for adhesive anchors for applications other than fence anchors in drawing VPF-1-90. This requires special plan notes for every other application.	x			
Michigan DOT	We struggle mostly with construction oversight with our adhesive systems. The systems used for structural applications are required to be proof tested prior to installing production anchors, and this gets missed at times. The production anchors are randomly selected for field testing, which also gets missed sometimes. We also have trouble with standardizing our process, i.e. lane ties are anchored with structural adhesive, but have a different testing requirement than those used in bridge. Also we have adhesives used for	x	x		

Agency	Responses	Type of Challenges			
		Design	Construction/ Quality Control	Performance	Cost
	dowels, which are a completely different section of our materials guides, and there is sometimes confusion regarding which list should be used to select products.				
MnDOT (Minnesota)	The biggest challenge is quality control. We require the installer to be ACI certified, however, any failure we have had is usually linked to cleaning of the hole prior to installation or water filled holes not being cleared out prior to installation. You might also say we sometimes have a QA issue. Our field inspectors are not always familiar with the need for thorough hole cleaning. Rarely have we had a failure due to design oversites. In general, we have not had issues with these anchorages. Field issues are infrequent.		x		
Texas DOT	Control of installation procedures, maintaining quality.		x		
Iowa DOT	Our usage of adhesive anchors has been primarily for special cases. The main challenges are the lack of quality control procedures and the availability of ACI certified installers. Needing qualified inspectors during installation can also be challenging. We are also lacking in design guidelines and design examples.	x	x		
Illinois DOT	Variable embedment depths for each supplier for the same anchorage size	x			
Caltrans	None.				
South Dakota DOT	N/A				
WSDOT (Washington)	Performance and cost.			x	x
SCDOT (South Carolina)	Lack of quality control, Lack of consistent design guidelines for specific applications.	x	x		
NHDOT (New Hamsphire)	It was once pointed out to me that these anchors are often used in locations where their performance is very critical. And the installation procedure, including hole prep, is vital to the performance of these anchors. And, they are typically installed		x		

Agency	Responses	Type of Challenges			
		Design	Construction/ Quality Control	Performance	Cost
	by the least experienced construction worker.				
Georgia DOT	Main challenge is past experience with failure of adhesive anchors supporting decorative fence over the interstate and Big Dig issues with overhead application. It has made GDOT very cautious about when to allow the use of anchors. It seems recent guidance to allow anchors in direct tension and overhead applications place a lot of requirements and burden on the engineer and installer that don't seem to be worth the hassle to implement.		x	x	
Florida DOT	Good contractor installation quality control. Difficulty in predesigning generic systems under the ACI 318 Chapter 17 methodology - every application seems to become a Design-Build type situation under that method.	x	x		
Alaska DOT&PF	Using ACI criteria is a burden for AASHTO users (I.e. would rather the specifications were not referenced but converted to AASHTO language), technically installers and inspectors need ACI certification to comply with ACI/AASHTO, ACI does not exactly address rebar which is most of our applications	x	x		
Virginia DOT	The use of adhesive anchors is limited to applications in which the anchors (bolts) are subject only to shear. They are not to be used for applications in which the anchors are subject to tension (axial loads or flexure loads) due to sustained, cyclical or fatigue loadings. The term adhesive (anchors) includes, but is not limited to, epoxies and grouts (including non-shrink grouts).		x		
ALDOT (Alabama)	We typically do not use adhesive anchors in bridge structures.				
Tennessee DOT	lack of design guidelines, lack of quality control procedures	x	x		
West Virginia DOT	Lack of quality control procedures.		x		

Agency	Responses	Type of Challenges			
		Design	Construction/ Quality Control	Performance	Cost
Oregon DOT	In Oregon, I think design guidelines and construction specification are available. 2-3 years ago, designers hesitated using adhesive anchors. Recently, I could see that designers feel more comfortable with specifying them.				
Arkansas DOT	NA				
Delaware DOT	Lack of design guidelines for specific applications	x			
Nevada DOT	Quality control.		x		
New York State DOT	Consistent design calculations for manufacture's developed tables. High cost of having anchorage materials certified to ACI requirements.	x			x
No. of States		**12**	**14**	**2**	**2**
Percent of States		**46%**	**54%**	**8%**	**8%**

APPENDIX B. LITERATURE REVIEW

Background

General Use of Adhesive Anchors

Post-installed anchors are primarily used for repair or rehabilitation projects although they are occasionally used in new construction. They may be classified by two categories: adhesive anchors and mechanical anchors.

Adhesive anchors are also referred to as "chemical anchors." Per the American Concrete Institute (ACI), an adhesive anchor is defined as "a post-installed anchor, inserted into hardened concrete with an anchor hole diameter not greater than 1.5 times the anchor diameter, that transfers loads to the concrete by bond between the anchor and the adhesive, and bond between the adhesive and the concrete" and an adhesive is defined as "chemical components formulated from organic polymers, or a combination of organic polymers and inorganic materials that cure if blended together" [4]. The precise materials used can vary widely. Two-component epoxies are the most common adhesive used but the adhesive may be an epoxy, methacrylate, or urethane-methacrylate and may or may not contain fine aggregate or other inert fillers [7]. The Michigan DOT [7] additionally considers anchors using cementitious grouts to be adhesive anchors since load is still transferred through the bond between the grout and the substrate instead of by friction or bearing, as in a mechanical anchor; however, cementitious grouts do not classify as an "adhesive" per ACI 318. In contrast, the New York State DOT does not treat anchors held in place by cementitious grouts as adhesive anchors [8] and both ACI 318-19 and AASHTO LRFD-9 state that the design procedures for post-installed anchors do not apply to grouted anchors, defined as bonded anchors with a hole diameter greater than 1.5 times the anchor diameter [4, 5]. The anchor itself may be a threaded rod, deformed reinforcing bar, internally threaded steel sleeve with external deformations, or dowel. The type of steel element used typically affects the required embedment depth but does not otherwise change the design procedure or qualification process of the product.

Mechanical anchors may be further categorized as expansion, screw, and undercut anchors. Expansion anchors transfer loads by a combination of direct bearing and/or friction, screw anchors transfer load by engaging the hardened threads of the screw with the grooves cut into the hole walls during installation, and undercut anchors transfer load via the mechanical interlock between the anchor and the concrete at the embedded end of the anchor.

State DOTs often place restrictions on when adhesive anchors can be used due to concerns regarding their performance under specific loading conditions. The use of adhesive anchors for sustained tensile loads in overhead applications was prohibited by many state DOTs including WisDOT after the collapse of concrete panels from the ceiling of the Boston Tunnel (the Big Dig Tunnel) in 2006, which resulted in one fatality. The National Transportation Safety Board (NTSB) and Federal Highway Administration (FHWA) published subsequent moratoria and conducted investigations which concluded that the main cause of the panel collapse was insufficient creep resistance [1, 2]. According to Morrison et al. [6], another possible cause was poor installation of the anchors, partially due to the lack of stringent quality assurance and control requirements for adhesive anchors at the time. As a result, many state DOTs have placed widespread restrictions on the use of adhesive anchors, for example prohibiting their use in any applications with sustained tensile loading; any applications in horizontal, overhead, or upwardly-inclined positions; or in any applications where anchor failure would pose a direct threat to the travelling public, regardless of the load scenario.

However, mechanical anchors also have limitations. WisDOT has placed a moratorium on mechanical anchors for the following reasons [3]:

- Mechanical anchors are challenging to install;
- Design requirements are more restrictive for mechanical anchors than adhesive anchors;
- There is a greater potential for anchor corrosion due to collection of salt water in the hole due to application of deicing salts;
- There are a variety of anchor types to select from; and
- There are concerns with the ability to remove and reuse railings and fences when using mechanical anchors.

Michigan DOT [7] also generally prefers adhesive anchors over mechanical anchors and notes that mechanical anchors are more sensitive to installation procedures, especially the dimensions of the pre-drilled holes, and typically do not perform as well as adhesive anchors in service. Furthermore, the Illinois DOT [9] does not permit mechanical anchors to be used if the anchor will be subjected to vibration since mechanical anchors are known to be sensitive to vibration [7].

Previous Studies on Applications of Adhesive Anchors

Because of the disadvantages of mechanical anchors, a large number of studies on the mechanics, design, behavior, and performance of adhesive anchors have been completed with the goal of broadening the permissible applications for adhesive anchors. Table B.1 summarizes a few of these studies and their conclusions or recommendations [10, 11, 12, 13, 14, 15, 16]. The extensive work completed has resulted in the development of national-level standards for prequalification testing, quality assurance and quality control procedures for installation and design of adhesive anchors. Standard design of adhesive anchors was first incorporated in ACI 318, *Building Code Requirements for Structural Concrete*, in 2011 and later by the American Association of State Highway and Transportation Officials (AASHTO) to AASHTO LRFD, *Bridge Design Specifications*, in 2017. Adhesive anchors designed per ACI 318 are required to be qualified in accordance with ACI 355.4-11, *Qualification of Post-Installed Adhesive Anchors in Concrete under Sustained Loading Conditions* [17] and ASTM E488, *Standard Test Methods for Strength of Anchors in Concrete Elements*, provides standardized test methods for prequalification testing as well [18]. Installers or the supervising personnel are often required to be certified through the Adhesive Anchor Installer Certification program established by the ACI and Concrete Reinforcing Steel Institute (CRSI), and ASTM E3121, *Standard Test Methods for Field Testing of Anchors in Concrete or Masonry* provides a standardized procedure for field proof testing of adhesive anchors [19]. As a result of these new resources, the FHWA published a new technical advisory in 2018 superseding its 2007 technical advisory [18]. The FHWA currently recommends that adhesive anchors be designed and qualified per ACI 318 and ACI 355.4, and no longer discourages the use of adhesive anchors in applications with sustained tension loading as long as the anchors have been qualified for sustained tension loading conditions per ACI 355.4, or a rigorous and regular inspection program of the anchors is in place [18].

Many state DOTs have sponsored research on the use of adhesive anchors for specific applications, such as connection of bridge traffic railing with a concrete deck. In 2001, the Michigan DOT investigated the effectiveness of using adhesive anchors to retrofit concrete bridge railing connections to bridge decks [10]. They concluded that adhesive anchors can be used to connect concrete railings to bridge decks without changes in the railing reinforcement provided that an embedment depth is 12 times the bar diameter. The study recommended using #4 bars spaced at 8 inches on center with an embedment depth of 6 inches instead of #5 bars spaced at 12 inches on center with an embedment depth of 7 1/2 inches. The use of smaller bars with shallower embedment was to minimize the problem of punching through the deck when drilling the hole for the adhesive anchor.

A study sponsored by Texas DOT [11] evaluated a retrofit design of the TxDOT T501 continuous concrete railing using epoxy adhesive anchors. The #5 U-shaped bars used in original railing-deck connection were replaced with a single line of #6 S-shaped bars placed near the traffic face of the railing with an embedment depth of 5 1/4 inches. Strengths of the retrofit railing from both static and dynamic testing were comparable with those of the original railing.

A study sponsored by Iowa DOT [13] investigated the use of epoxy adhesive anchors for attachment of the steel posts of a bridge railing system to the concrete barrier. Through dynamic testing, three designs using adhesive anchors were found to have higher strengths than a traditional cast-in-place anchorage design. Iowa DOT has developed specifications for adhesive-bonded anchors and dowels for traffic railings, which specifies materials, installation procedures, and acceptance testing for the anchors [19]. Anchors shall be proof-loaded to 85% of the bond strength specified in the plans in field tests in accordance with ASTM E 488.

A study sponsored by the Minnesota DOT [14] investigated the effect of reinforcing bar epoxy coating on anchor strengths and found that the epoxy coating slightly reduced the tensile bond strength of the anchor for adhesive products from some manufacturers (up to 6%), but not for others. That study also found that the bond strengths calculated based on test results were significantly higher than the manufacturer published values for uncracked concrete, which were higher than the minimum characteristic bond strength of 1,000 psi required by Minnesota DOT. It should be noted that the design values used by Minnesota DOT are higher than those specified by WisDOT Bridge Manual as well as those specified by ACI 318-14.

In a recent WisDOT-sponsored study [12], dynamic and static load tests were performed on epoxy coated bars installed into drilled holes in a concrete slab using Hilti epoxy adhesives, showing that the anchor tensile and shear strengths determined using ACI 318-11 Appendix D (now ACI 318-14 Chapter 17) were conservative.

Table B.1. Summary of Select Research Studies on Adhesive Anchors

Sponsoring Agency	Year	Topic	Key Conclusions or Recommendations
Michigan DOT [10]	2001	Use of adhesive anchors to retrofit concrete bridge railing connections to bridge decks	A greater number of smaller bars with shallower embedment should be used to minimize the problem of punching through the deck when drilling the hole for the adhesive anchor.
Texas DOT [11]	2007	Use of epoxy adhesive anchors in retrofit design of a continuous concrete railing	Strengths of the retrofit railing were comparable to strengths of the original railing in both static and dynamic conditions.
NCHRP[1] [15]	2009	Test method for determining the ability of adhesive anchors to resist sustained tensile loads	A standardized test was developed, and later adopted as AASHTO TP-84-11, *Evaluation of Adhesive Anchors in Concrete under Sustained Loading Conditions*.
Wisconsin DOT [12]	2012	Design of cast-in-place parapets using adhesive anchors with epoxy-coated bars	A design methodology combining the ACI 318 procedure to determine anchor strengths and AASHTO LRFD yield line analysis to determine anchor load demand was proposed.
NCHRP[1] [16]	2013	Factors affecting long-term performance of adhesive anchors	Service temperatures and manufacturer's cure time are the primary factors that affect sustained load performance of adhesive anchors.
Iowa DOT [13]	2015	Use of epoxy adhesive anchors for attachment of steel posts to concrete barriers in a bridge railing system	Three designs using adhesive anchors demonstrated higher strengths than traditional cast-in-place anchor design; specifications for use and installation of adhesive anchors for traffic railings were developed as a result of this study.
Minnesota DOT [14]	2019	Effect of epoxy coatings on reinforcing bar on anchor strengths	The epoxy coating reduced tensile bond strength for some adhesive products up to 6 percent and did not affect bond strength for other products. Calculated bond strengths remained greater than the manufacturer-published values.

Notes: [1]National Cooperative Highway Research Program

Design of Concrete Adhesive Anchors

ACI 318-19 Chapter 17 provides a design procedure for adhesive anchors. An overview of the procedure and the variables and parameters used is described in the following subsection. AASHTO LRFD Section 5.13 typically refers to ACI 318 for design of adhesive anchors, but has some differences, which are identified in the second subsection. Differences between these national standards and state DOT design policies are discussed later in Section 2.4, Practices by State DOTs.

ACI 318-19 Chapter 17

When designing adhesive anchors in accordance with ACI 318-19 Chapter 17 the designer first needs to select several parameters, as described below:

Anchor diameter, d_a. The ACI design procedure is considered applicable to anchors with a diameter of up to 4 inches due to limited test data for anchors with larger diameters.

Embedment depth, h_{ef}. The ACI design procedure is considered applicable for adhesive anchors with embedment depths between $4d_a$ and $20d_a$.

Distance from concrete edge(s), c_a. Edge distance can affect the projected failure area and therefore decrease the strength of the anchor in some cases. Per ACI Table 17.9.2, the minimum edge distance permitted is the specified reinforcement cover, twice the maximum aggregate size, or the minimum edge distance determined by testing according to ACI 355.4, whichever is greatest. In the absence of product-specific data, a minimum edge distance of $6d_a$ is assumed in place of the ACI 355.4 result.

Number of anchors, n. The designer may choose to use a single anchor, multiple anchors, or an anchor group.

Spacing between anchors, s. When multiple anchors are used, the spacing determines whether each anchor functions as a single anchor, or if the anchors function as a group due to overlapping projected concrete breakout failure areas. A minimum anchor spacing of $6d_a$ is required per Table 17.9.2(a) unless supplemental reinforcement is provided to prevent splitting failure.

Additionally, the designer must identify the following parameters based on the existing conditions and available materials:

Concrete compressive strength, f'_c. The ACI design procedure does not permit values greater than 8000 psi to be used unless testing verifying acceptable performance has been completed.

Tensile strength of the steel anchor. The ultimate tensile strength of the anchor, f_{uta}, is considered in strength calculations. The upper limit of the f_{uta} for adhesive anchor design is 125 ksi or 1.9 times the yield strength of the anchor, f_{ya}.

Per ACI 318, the design of adhesive anchors considers six primary failure modes, for which the design strength requirements are shown in Table B.2. These modes include steel and concrete strength in tension, steel and concrete strength in shear, and bond strength of adhesive anchors in tension. ACI 318 additionally has a special strength requirement for adhesive anchors subject to sustained tension, included in Table B.2 as well. Splitting failure of the concrete is a potential failure mode, but ACI provides minimum requirements for anchor edge distances, anchor spacing, and concrete element thickness such that splitting failure does not need to be calculated. Alternatively, supplementary reinforcement may be used to control splitting.

If the anchors are subject to both tension and shear, then interaction effects may need to be considered depending on the ratios between the factored loads and the governing strengths, $N_{ua}/\Phi N_n$ for tension and $V_{ua}/\Phi V_n$ for shear. If the ratio exceeds 20 percent for both shear and tensile loading, then the sum of the ratios must not exceed 1.2, per ACI 318 Section 17.8.

Table B.2. Design strength requirements of adhesive anchors per ACI 318 Table 17.5.2 and Section 17.5.2.2 [4]

Failure Mode	Single Anchor	Anchor Group[1]	
		Individual Anchor in a Group	Anchors as a Group
Steel strength in tension	$\Phi N_{sa} \geq N_{ua}$	$\Phi N_{sa} \geq N_{ua,i}$	--
Concrete breakout strength in tension	$\Phi N_{cb} \geq N_{ua}$	--	$\Phi N_{cbg} \geq N_{ua,g}$
Bond strength of adhesive anchor in tension	$\Phi N_{ag} \geq N_{ua}$	--	$\Phi N_{ag} \geq N_{ua,g}$
Bond strength of adhesive anchor in sustained tension	$0.55\Phi N_{ba} \geq N_{ua,s}$	$0.55\Phi N_{ba} \geq N_{ua,s,i}$	--
Steel strength in shear	$\Phi V_{sa} \geq V_{ua}$	$\Phi V_{sa} \geq V_{ua,i}$	--
Concrete breakout strength in shear	$\Phi V_{cb} \geq V_{ua}$	--	$\Phi V_{cbg} \geq V_{ua,g}$
Concrete pryout strength in shear	$\Phi V_{cp} \geq V_{ua}$	--	$\Phi V_{cpg} \geq V_{ua,g}$

Notes: [1]*Design strengths for steel and sustained tension failure modes shall be calculated for the most highly stressed anchor in the group.*

The strength reduction factor Φ used in each failure mode is defined by Tables 17.5.3(a), (b), and (c) in ACI 318. The relevant factors for adhesive anchors are reproduced here in Table B.3 and

Table B.4. For failure modes associated with concrete, the factor depends on the sensitivity of the adhesive anchor to installation conditions and procedures. The sensitivity is assessed per ACI 355.4, which evaluates the influences of, (a) adhesive mixing, and (b), hole cleaning in dry, saturated, and water-filled/underwater scenarios.

Table B.3. Strength reduction factor Φ when adhesive anchor strength is governed by steel per Table 17.5.3(a) [4]

Type of Steel Element	Tension (Steel)	Shear (Steel)
Ductile	0.75	0.65
Brittle	0.65	0.60

Table B.4. Strength reduction factor Φ when adhesive anchor strength is governed by concrete breakout or bond per Table 17.5.3(b) and Table 17.5.3(c) [4]

Supplementary Reinforcement	Anchor Category from ACI 355.4[1]	Tension (concrete breakout or bond strength)	Shear (concrete breakout)
Present	1	0.75	0.75
	2	0.65	
	3	0.55	
Not present	1[2]	0.65	0.70
	2[2]	0.55	
	3[2]	0.45	

Notes: [1]*Anchor Category 1 indicates low sensitivity to installation and high reliability; Anchor Category 2 indicates medium sensitivity and medium reliability; and Anchor Category 3 indicates high sensitivity and lower reliability.*

[2]*The factors identified in these scenarios also apply when adhesive anchor strength is governed by concrete pryout strength. The presence of supplementary reinforcement is not relevant to pryout strength.*

Once the factored load is calculated, the governing tensile and shear strengths are determined by calculating the strength of the adhesive anchor or adhesive anchor group in each failure mode. The equations provided by ACI 318 for strength evaluation are shown in Table B.5 in order to highlight the variables that affect anchor strength.

Calculations of strength in steel failure modes are relatively straightforward as products of area and steel strength. Calculations of concrete or bond strength are generally based on calculation of the basic strength of a single anchor, which are summarized in Table B.6. The basic strength is then multiplied by modification factors (ψ_i) to account for effects due to load eccentricity, nearby edges of the element, concrete cracking, and potential for concrete splitting. The strength is also decreased if nearby concrete edges or anchors cause the projected concrete failure area of the anchor or anchor group to be smaller than it would be if these features were far away. The decrease is proportional to the ratio between the affected area and the unaffected area (A/A_o).

As can be seen in Table B.6, the basic strengths of adhesive anchors rely on a number of parameters, listed in Table B.7. Some, such as the embedment depth and anchor diameter, are selected by the designer while others, such as the concrete compressive strength, are determined by the existing conditions of the element. However, the values for the experimental constant k_c, the modification factor for lightweight concrete λ_a, and the characteristic bond stress τ_{cr} are defined by the ACI 318 code. ACI 318 Section 17.6.2.2.1 clearly states k_c is 17 for all post-installed anchors, including adhesive anchors. ACI 318 Table 17.2.4.1 clearly identifies λ_a as 0.8λ for concrete failures of adhesive anchors (i.e., concrete breakout strength in tension) and 0.6λ for bond failures of adhesive anchors, where λ is determined based on Chapter 19, which is not unique to anchor design.

Guidance for selecting τ_{cr} is addressed in ACI 318 Section 17.6.5.2. Per ACI, the characteristic bond stresses should be based on the 5 percent fractile of results of tests performed and evaluated per ACI 355.4. If there is evidence that the adhesive anchors will be located in a region with no cracking of the concrete at service loads, then the uncracked characteristic stress τ_{uncr} may be used instead, also defined as the 5 percent fractile of test results per ACI 355.4. If product-specific information is not available, the values of ACI 318 Table 17.6.5.2.5, shown in Table B.8 of this report, may be used as lower-bound default values instead provided that the following conditions are met:

- Anchors meet the requirements of ACI 355.4.
- Anchors are installed in holes drilled with a rotary impact drill or rock drill.
- Concrete compressive strength at time of anchor installation is at least 2500 psi.
- Concrete age at time of anchor installation is at least 21 days.
- Concrete temperature at time of anchor installation is at least 50°F.

Table B.5. Strength equations for potential failure modes of adhesive anchors per ACI 318, Chapter 17 [4]

Failure Mode	Strength of a Single Anchor	Strength of an Individual Anchor in a Group	Strength of an Anchor Group
Steel strength in tension	$N_{sa} = A_{se,N} f_{uta}$	$N_{sa,i} = A_{se,N} f_{uta}$	--
Concrete breakout strength in tension	$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	--	$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$
Bond strength of adhesive anchor in tension	$N_a = \frac{A_{Na}}{A_{Nao}} \psi_{ed,Na} \psi_{cp,Na} N_{ba}$	--	$N_{ag} = \frac{A_{Na}}{A_{Nao}} \psi_{ec,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba}$
Steel strength in shear	$V_{sa} = 0.6 A_{se,V} f_{uta}$	$V_{sa,i} = 0.6 A_{se,V} f_{uta}$	--
Concrete breakout strength in shear	$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b$	--	$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b$
Concrete pryout strength in shear	$V_{cp} = k_{cp} N_{cp}$	--	$V_{cpg} = k_{cpg} N_{cpg}$

Table B.6. Equations for basic strength of a single adhesive anchor in concrete or bond failure modes per ACI 318, Chapter 17 [4]

Failure Mode	Basic Strength of a Single Anchor
Concrete breakout strength in tension	$N_b = k_c \lambda_a \sqrt{f_c'} h_{ef}{}^{1.5}$
Bond strength of adhesive anchor in tension	$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$
Concrete breakout strength in shear	$V_b = min\left\{\left(7\left(\frac{l_e}{d_a}\right)^{0.2}\right)\sqrt{d_a}\lambda_a\sqrt{f_c'}(c_{a1})^{1.5} ; 9\lambda_a\sqrt{f_c'}(c_{a1})^{1.5}\right\}$
Concrete pryout strength in shear	$N_{cp} = min\{N_a ;\ N_{cb}\}$

Table B.7. Parameters needed to calculate basic anchor strengths and how they are determined

Symbol	Parameter	Source
k_c	Experimental constant	Defined in ACI 318 Section 17.6.2.2.1
λ_a	Modification factor for lightweight concrete	Defined in ACI 318 Table 17.2.4.1
f'_c	Concrete compressive strength	Determined by existing concrete
h_{ef}	Embedment depth	Selected by designer
τ_{cr}	Characteristic bond strength	Defined in ACI 318 Section 17.6.5.2 and Table 17.6.5.2.5
d_a	Anchor diameter	Selected by designer
l_e	Load-bearing length of the anchor for shear	Selected by designer
$c_{a,1}$	Critical distance between the axis of the critical anchor row and the element edge	Selected by designer; limited by geometry

Table B.8. Minimum characteristic bond stresses, per ACI 318 Table 17.6.5.2.5[1,2] [4]

Installation and Service Environment	Moisture Content of Concrete at Time of Anchor Installation	Peak In-Service Temperature of Concrete (°F)	τ_{cr} (psi)	τ_{uncr} (psi)
Outdoor	Dry to fully saturated	175	200	650
Indoor	Dry	110	300	1000

Notes: [1]*If anchor design includes sustained tension, multiply values of τ_{cr} and τ_{uncr} by 0.4.*

[2]*If anchor design includes earthquake-induced forces for structures assigned to SDC C, D, E, or F, multiply values of τ_{cr} by 0.8 and τ_{uncr} by 0.4.*

AASHTO LRFD-9 Section 5.13

Design of adhesive anchors is described in AASHTO LRFD Section 5.13. This section specifies that adhesive anchors are to meet the criteria of ACI 355.4 (2011) and be designed, detailed, and installed using the provisions of ACI 318-14, Chapter 17, except as modified within Section 5.13. Two amendments to the ACI design procedure for adhesive anchors are specified:

1. **Impact Loading.**

ACI 318, both the 2014 version referenced by AASHTO LRFD and the current 2019 version, clearly states that the design procedure does not apply for impact load conditions. However, AASHTO LRFD states that the design procedure can apply for evaluating strength under impact loading provided that the anchors have an impact strength equal to or greater than their static strength, as shown by either testing or a combination of testing and analysis. A Dynamic Increase Factor (DIF) is permitted to be applied in the analysis but is not required. AASHTO LRFD does not provide DIFs for any anchors, but references Dickey et al. [12] and Braimah et al. [20] for their work evaluating the impact behavior of adhesive anchors and acknowledges the DIFs recommended by Dickey et al. [12].

2. **Sustained Tension.**

AASHTO LRFD specifies a smaller strength reduction factor under sustained tensile load conditions than ACI 318. As shown in Table B.2, ACI 318 specifies an additional strength reduction factor of 0.55 when evaluating bond strength under sustained tensile loads. The 0.55-factor is based on the requirements of

ACI 355.4, which evaluates product behavior under sustained tensile loading conditions designed to represent a 50-year duration at a standard temperature of 70°F and a 10-year duration at an elevated temperature of 110°F.

Because many structures are designed for a 100-year service life, AASHTO LRFD specifies a strength reduction factor of 0.50 instead, as recommended by Cook et al. (2013) for structures with a 100-year life at 70°F or a 20-year life at an elevated temperature of 110°F. Furthermore, AASHTO LRFD only requires strength under sustained tensile loading to be assessed (i.e., the 0.50 strength reduction factor to be applied) when "significant sustained tensile loads" are present. AASHTO LRFD defines a "significant" sustained tensile load as at least 10 percent of the factored bond strength of the adhesive anchor, ΦN_{ba}. In contrast, ACI 318 requires the 0.55 strength reduction factor to be considered regardless of the magnitude of the sustained tensile load.

Practices by State DOTs

In addition to conducting the DOT survey, the research team reviewed manuals and specifications of twelve select DOTs to obtain more detailed information on their policies on design and application of adhesive anchors. The state DOTs included in the review and their acronyms are presented in Table B.9. This section summarizes the findings.

Table B.9. State DOTs Whose Manuals, Specifications, and Other Literature were Reviewed

State DOT	Acronym
Wisconsin DOT	WisDOT
California DOT	Caltrans
Florida DOT	FDOT
Illinois DOT	IDOT
Indiana DOT	INDOT
Iowa DOT	IowaDOT
Michigan DOT	MDOT
Minnesota DOT	MnDOT
Nebraska DOT	NDOT
New York State DOT	NYSDOT
South Dakota DOT	SDDOT
Texas DOT	TxDOT

Notes: [1]*The Bridge Office Policies and Procedures was published by the Nebraska Department of Roads (NDOR) instead of the Nebraska Department of Transportation (NDOT). As such this report sometimes refers to NDOR.*

Applications of Adhesive Anchors

Of the policies, manuals and specifications of the twelve states reviewed, the majority limit the use of adhesive anchors, as identified in Table B.10. As a basis for comparison, WisDOT generally uses adhesive anchors for bridge rehabilitation projects such as widening of abutments and piers and to attach pedestrian railings or fencing in new construction. WisDOT additionally states that adhesive anchors are used for interior traffic parapets only when the adjacent, exterior parapet is crash-test approved (i.e. cast-in-place anchors are used in the exterior parapet). However, adhesive anchors are prohibited from use in sustained tension load applications and overhead or upwardly inclined positions. Adhesive anchors are also restricted when extending pier caps of multi-columned piers or hammerhead piers without any new column support [3].

TxDOT is the only state DOT that did not provide discussion on permissible and prohibited applications for adhesive anchors. According to the Standard Specifications and their Bridge Railing Manual, TxDOT does use adhesive anchors for bridge railings and adhesive anchors permit certain railing types to be constructed more rapidly using slip forming [21, 22].

The sources reviewed from INDOT held little discussion on the application of adhesive anchors. INDOT does not permit an anchor system to be used between two concrete elements if moment must be transferred across the connection and instead requires exposing and splicing the reinforcement in the existing concrete member; however, this requirement applies to adhesive and mechanical anchors [23]. Like TxDOT, INDOT does not place any limitations on the use of adhesive anchors. Instead, INDOT warns that the FHWA strongly discourages using adhesive anchors for overhead applications or permanent sustained tensile loading but permits these applications as long as the anchors comply with the design requirements of FHWA Technical Advisory T5140.34 [24].

Caltrans, IDOT, IowaDOT, MDOT, NDOT, NYSDOT, and SDDOT have similar regulations prohibiting the use of adhesive anchors to support sustained tensile loads and/or in overhead applications. While the general intent is the same, there is some variation between the policies:

Caltrans generally prohibits the use of adhesive anchors in sustained tensile load applications and defines "sustained tensile load" as a constant, unfactored tensile load greater than 10% of the nominal bond strength in tension of the anchor or anchor group. Adhesive anchors are additionally not permitted in plastic hinge regions or under certain seismic loading conditions [25]. Two standard configurations for bridge-mounted signs that use adhesive anchors are provided; any alternative configurations require review by the DES Signs and Overhead Structure Specialist prior to use [26].

IDOT does not discuss the application of adhesive anchors in its Standard Specifications or Bridge Design Manual; however, a contract specification from 2019 addresses the issue and the policies stated in the contract are assumed to reflect the typical policies of IDOT [9]. Adhesive anchors are not permitted in overhead applications or sustained tension loading conditions. Both types of post-installed anchors (adhesive and mechanical) are only allowed where specifically indicated on Drawings or when approved for use by the Engineer. Adhesive anchors are to be used if the anchor is subjected to vibration and may be used under buried or submerged conditions.

IowaDOT maintains four separate lists of adhesive materials, referred to as Appendix A through Appendix D [27]. Products listed in Appendix D are explicitly defined as "approved adhesives used as chemical anchors," and these products are prohibited from being used in sustained tensile load overhead applications in highway projects. However, this restriction does not apply to the products listed in Appendices A through C, defined as "pourable polymer grouts intended for vertical installations or angled installations less than 45 degrees from vertical," "viscous polymer grouts intended for horizontal installations," and "polymer grouts for dowel bar installation," respectively. Products from Appendix B are used to connect structural steel traffic railing dowels and anchors to concrete barriers. Specific applications for products from Appendix A are not identified and dowel bar installation is not of interest in this review.

MDOT prohibits the use of adhesive anchors for overhead installation and has placed a moratorium on adhesive anchors under sustained tensile loads due to the possibility of creep failure [7]. However, MDOT does regularly use adhesive anchors for select railings on non-National Highway System (NHS) routes and bridge substructure repairs [28].

NDOT states that resin adhesives on the Approved Product List should not be used in sustained tensile load applications. Adhesive anchors are most commonly used to connect W-beam or thrie-beam guardrails to concrete members. Regarding concrete bridge rails, the threaded inserts should be cast in the new or reconstructed rail [30].

NYSDOT prohibits the use of adhesive anchors in all horizontal, overhead, and upwardly-inclined positions and any permanent applications subject to sustained tensile load, including cantilever applications [8]. Adhesive anchors are commonly used to anchor bridge railings, decorative railings, pedestrian fences, and screening since these elements do not have sustained tensile loads [31]. It should be noted that these restrictions only apply to materials listed as "Anchoring Materials - Chemically Curing." If sustained tensile loads exist and the application is permanent, a "Concrete Grouting Material" must be used [8].

SDDOT states that adhesive anchors are not allowed in sustained tensile load applications in concrete members; no further discussion is provided [32].

FDOT provides a relatively extensive discussion regarding the use of adhesive anchors. FDOT prohibits the use of adhesive anchors in overhead or upwardly-inclined positions, when the loading is predominately sustained tension, when there is a lack of structural redundancy, and/or when any of the service limit states result in tension loading of the adhesive. A "predominately sustained tension load" scenario is defined as when the permanent factored tension load is greater than 30% of the factored tensile resistance. FDOT specifically states that adhesive anchors are not to be used on traffic railing anchorages on new construction since they fall under at least one of the prohibited scenarios. Adhesive anchors are also prohibited when splicing with existing reinforcement or for signal or lighting support structures. However, FDOT does permit adhesive anchors to be used for traffic railing retrofit applications for existing concrete bridge decks and approach slabs, especially if through-bolting, undercut anchors, and threaded inserts are not practical and the predominant loading is very short-term, i.e., impact [33].

MnDOT published a detailed technical memorandum on the application of adhesive anchors, particularly under sustained tensile loading conditions [34]. The exact scenarios are listed in Table B.10. To summarize, adhesive anchors are generally prohibited in sustained tension loading applications if their failure poses a direct threat to the safety of the travelling public, the installation is over or directly supporting traffic, or the concrete is structurally unsound. Similar to Caltrans, MnDOT only considers sustained tension loads that are at least 10% of the factored nominal tensile capacity of the anchor; note Caltrans uses the nominal capacity, not the factored capacity. If the sustained tension load is less than 10% of the factored nominal tension capacity, then sustained tension loading does not need to be considered in the design procedure. MnDOT generally uses adhesive anchors for reconstruction of expansion joints and paving brackets, deck repairs and abutment or wingwall retrofits not expressly prohibited, and attachment of secondary structural elements to concrete, such as metal rails to concrete bases [35]. Attachment of concrete railings to bridge decks is not discussed.

The limitations identified by ACI 318-19 and AASHTO LRFD-9 on the application of their respective design procedures for adhesive anchors are not included in Table B.10 because they are more pertinent to the design procedures, discussed previously. However, the standards discuss the anticipated in-service conditions and loading scenarios of the adhesive anchors, which is more relevant to application and is therefore presented here. ACI 318-19 states that the procedures are for connected structural elements and safety-related attachments and structural elements. The procedures address tension, shear, and combined tension and shear loading and do not address high-cycle fatigue loads or impact loading. AASHTO LRFD-9 adopts the ACI 318-14 design procedures with several modifications, as described in a Section 0, **AASHTO LRFD-9 Section 5.13**, and therefore identifies similar limitations of the procedure. However, AASHTO LRFD-9 notes that the design procedure is valid for impact loads as long as the post-installed anchors have demonstrated an impact strength at least equal to their static strength via testing or a combination of testing and analysis. Both standards have several additional limitations regarding anchor diameter, concrete compressive strength, and embedment depth due to test data available to date.

Table B.10. Permitted and Prohibited Applications for Adhesive Anchors (per Reviewed Manuals and Specifications from Twelve State DOTs)

Organization	Recommended/Permitted Applications	Prohibited Applications
California DOT	Bridge-mounted signs (two approved configurations provided; any alternative configurations require review by the DES Signs and Overhead Structure Specialist)	Sustained tension load applications, wherein the constant unfactored tensile load exceeds 10% of the nominal tensile capacity Applications wherein the tensile or shear component of the earthquake load exceeds 20% of the total EXTREME EVENT I Limit State tensile or shear load In plastic hinge regions
Florida DOT	Horizontal, vertical downward, or downwardly inclined positions Traffic railing retrofit applications wherein through bolting, undercut anchors, or threaded inserts are not practical and the predominant loading is from very short-term loading Installation of traffic railing reinforcement and anchor bolts into existing concrete bridge decks and approach slabs	Overhead or upwardly-inclined positions Applications with predominately sustained tension loading and/or a lack of structural redundancy, such as traffic railing anchorages in new construction Applications wherein any of the Service Limit States result in tension loading of the adhesive Signal or lighting support structures Splicing with existing reinforcement in reinforced or prestressed concrete, except where specifically permitted by the Structures Manual or Standard Plans, or if application is validated by testing
Illinois DOT	Only when specifically indicated on Drawings or approved for use by Engineer Anchors subjected to vibration Where buried or submerged	Overhead applications Sustained tension loading conditions
Indiana DOT	Not discussed.	When moment transfer is required across the connection between the concrete members
Iowa DOT	Installation of structural steel traffic railing dowels and anchors to concrete barriers	Sustained tensile load overhead applications in highway projects
Michigan DOT	Bridge substructure repair Type 6 and 7 modified railings (non-NHS routes only)	Overhead installation Sustained tension loading scenarios (moratorium in place)
Minnesota DOT	Attachment of secondary structural members to new concrete or primary structural members to existing concrete	Installation in delaminated or structurally unsound concrete Generally when:

Organization	Recommended/Permitted Applications	Prohibited Applications
	Attachment of metal rails to concrete bases Reconstruction of expansion joints and paving brackets Limited applications under sustained tensile loading: Any application in abutment and wingwall retrofits that is not expressly prohibited Any application in deck repairs that is not expressly prohibited Attachment of bridge-mounted signs All cases wherein sustained tension load exceeds 10% of the factored nominal tensile capacity, as long as the application is not expressly prohibited and with the approval of the State Bridge Design Engineer All applications when sustained tensile loading does not exceed 10% of the factored nominal tensile capacity: Paving bracket reconstruction when the approach slab rests on grade End post retrofits cantilevered off the back of the abutment End blocks on parapet-type abutments Attachment of shear brackets to the back of retaining walls Attachment of baseplates with threaded rod anchors where sustained tension load is only because of the tightening of the nuts Support of pier struts retaining soil	Anchor failure poses a direct threat to the safety of the travelling public Installation over traffic, except as expressly permitted Directly supporting traffic, except as expressly permitted Sustained tensile loads under the following conditions: Pier cap retrofits, support or repairs Paving brackets supporting approach slabs that span voids behind the abutment Primary reinforcing for deck overhang repairs or replacement Primary reinforcing for abutment stems and wingwall widenings and retrofits Corbels supporting any elements that carry directly applied traffic loads (excluding paving brackets supporting approach slabs on grade) Supports for overhead cantilever signs, utilities and drainage systems, and catwalks
Nebraska DOT	W-beam and thrie-beam guardrail installations Attachment of concrete guardrails to concrete members	Sustained tension load applications
New York State DOT	Bridge railings, decorative railings, pedestrian fences, and screening	Permanent applications with sustained tensile loads Cantilever applications with sustained tensile loads All horizontal, overhead, or upwardly-inclined positions
South Dakota DOT	Not discussed.	Sustained tension load applications in concrete members
Texas DOT	Bridge railings	Not discussed.
Wisconsin DOT	Pedestrian railings/fencing in new construction Bridge rehabilitation, such as abutment and pier widening Parapets at interior traffic railing locations when the adjacent exterior parapet is crash-test approved	Pier cap extensions for multi-columned piers without any additional column support Crash-worthy traffic railings

Organization	Recommended/Permitted Applications	Prohibited Applications
		Extension of hammerhead piers without any new columns (requires further review) Overhead or upwardly-inclined positions Sustained tension loading

Basis for Design of Concrete Adhesive Anchors

The design basis for adhesive anchors and the supporting information and resources used or provided by the state DOTs included in this review are summarized in Table B.11. The majority of the state DOTs rely on the procedures provided by ACI 318 or AASHTO LRFD. Some state DOTs (Caltrans, MDOT, SDDOT, and TxDOT) reference the ACI or AASHTO code in general for design and do not specifically call out the design procedure to be used for adhesive anchors. Other state DOTs (IDOT, IowaDOT, MnDOT, NDOT, and NYSDOT) explicitly call out the design procedure specified per ACI 318 or AASHTO LRFD for adhesive anchor design. Many state DOTs provide additional commentary pertaining to the selection of anchor embedment depths and spacing or modification of the calculated strengths and strength reduction factors. Of the state DOTs reviewed, FDOT, INDOT, and WisDOT provide their own procedures for adhesive anchor design, of which the FDOT and INDOT procedures differ significantly from the ACI and AASHTO procedures. Additionally, FDOT, INDOT, and MnDOT provide detailed design examples or high-level tools to aid designers.

The design practices of WisDOT, FDOT, INDOT, MDOT, MnDOT, NDOT, and IowaDOT are discussed in more detail in the following subsections. These state DOTs are of interest because they provide detailed procedures, deviate from the ACI or AASHTO methods, or have useful design examples or tools. Their design procedures specifically tend to deviate from the ACI and AASHTO codes in at least one of the following topics:

1. Governing failure modes;
2. Strength reduction factors;
3. Sustained tension analysis;
4. Specified pullout capacities; and
5. Requirements for embedment depth and anchor spacing.

These select state DOTs also demonstrate variable practice pertaining to analysis of bond strength of adhesive anchors. Because of the complexity of this topic, it is addressed separately in Section 0, **Characteristic Bond Strength**.

Table B.11. Resources Provided by State DOTs for Adhesive Anchor Design

State DOT	National Code(s) Referenced	Procedures	Commentary	Design Examples or Tools
Caltrans	AASHTO LRFD-8 generally referenced	--	List of amendments to code given; no amendments for design of adhesive anchors.	--
Florida DOT	AASHTO LRFD is generally referenced; however, an alternate procedure for anchors is provided. Reader is referred to ACI 318 for conservative check of concrete breakout strength.	Procedures independent of national codes provided.	Commentary provided with procedures.	Design examples, and a downloadable MathCAD file for design calculations
Illinois DOT	ACI 318	--	--	--
Indiana DOT	AASHTO LRFD is generally referenced; however, an alternate procedure for anchors is provided.	Procedures independent of national codes provided.	Commentary provided with procedures.	A table of standard design strengths
Iowa DOT	ACI 318	--	Commentary on the permissible capacities is provided.	--
Michigan DOT	AASHTO LRFD is generally referenced	Calculations using LFD methodology instead of LRFD are also provided.	Commentary on selection of parameters is provided separately in other documents by MDOT.	--
Minnesota DOT	ACI 318 & AASHTO LRFD	--	Commentary provided with design example.	Detailed design example of ornamental railing post using adhesive anchors
Nebraska DOT	ACI 318-11, Appendix D	--	Commentary on strength requirements and assumptions is provided.	--
New York State DOT	ACI 318, current ed.	--	--	--
South Dakota DOT	AASHTO LRFD, latest ed. Is generally referenced	--	--	--
Texas DOT	AASHTO LRFD is generally referenced	--	--	--

State DOT	National Code(s) Referenced	Procedures	Commentary	Design Examples or Tools
Wisconsin DOT	Reader is referred to ACI 318 for more refined analysis. Text indicates AASHTO LRFD will be used in future updates.	Procedures in general accordance with ACI 318-14 are provided; some slight differences are present.	Commentary provided with procedures.	--

Design per the Wisconsin DOT

WisDOT provides a detailed design procedure for adhesive anchors in the WisDOT Bridge Manual, Chapter 40 - Bridge Rehabilitation [3]. The procedure is based on ACI 318-14 and generally agrees with the ACI code except in a few instances. The WisDOT design procedure deviates from the ACI code in the following ways:

1. **Governing Failure Mode.**

   As shown in Table B.2, ACI 318 considers each failure mode independently and does not specify a governing failure mode. However, WisDOT specifies the following hierarchies for tensile strength resistance N_r and shear strength resistance V_r, respectively:

$$N_r = \Phi_{ts} N_{sa} \leq \Phi_{tc} N_{cb} \leq \Phi_{tc} N_a$$

$$V_r = \Phi_{vs} V_{sa} \leq \Phi_{vc} V_{cb} \leq \Phi_{vp} V_{cp}$$

   This equation indicates that failure of the steel anchor is required to govern over concrete breakout, which must govern over bond strength in tension and concrete pryout in shear. It should be noted that concrete pryout strength for adhesive anchors depends on the smaller of the bond strength and concrete breakout in tension, as shown by the basic strength equation in Table B.6.

Strength Reduction Factors.

The strength reduction factors specified in the WisDOT design procedure are the same as those specified by ACI 318 in Table 17.5.3 for anchors in Category 1, low sensitivity to installation and high reliability. Strength reduction factors corresponding to anchors in Categories 2 or 3 are not considered in the WisDOT procedure.

Sustained Tension.

For sustained tension load conditions, the WisDOT design procedure specifies an additional strength reduction factor of 0.50, which agrees with the AASHTO LRFD modification to the ACI 318 procedure, which uses a factor of 0.55.

Specified Pullout Capacities.

Like ACI 318, WisDOT acknowledges that anchor pullout applies to mechanical anchors. Per the commentary provided by WisDOT, pullout capacities are only specified for mechanical anchors and minimum bond stresses are required instead for adhesive anchors.

Embedment Depth and Anchor Spacing.

The maximum embedment depth permitted by WisDOT is $20d_a$ and the minimum anchor spacing is $6d_a$, which agrees with ACI 318.

Design per the Florida DOT

FDOT provides a design procedure for adhesive anchors in the Structures Design Guidelines Volume 1 - General Requirements, Section 1.6, Post-Installed Anchor Systems [33]. The procedure deviates significantly from the ACI and AASHTO codes. The failure modes considered are steel anchor strength in tension, tensile strength of adhesive anchor bond, steel anchor strength in shear, and concrete breakout in shear. Calculations are not given for concrete breakout strength in tension or concrete pryout strength in shear. Concrete breakout under tension is generally assumed not to govern, although the commentary provided by FDOT warns that "use of higher bond strengths with close anchor spacing can potentially result in concrete breakout failure under tensile loading that may not be accounted for in the current equations." In these instances, FDOT directs the reader to ACI 318 Appendix D for a conservative check of concrete breakout strength. Other ways in which the FDOT procedure differs from the ACI and AASHTO codes are described below:

1. **Governing Failure Mode.**
   For adhesive anchor systems, FDOT generally requires a ductile failure. According to the commentary, a ductile failure may not be necessary depending on the resulting amount of over-strength resistance of the other failure modes; the load path and amount of redundancy in the anchorage system; the need for an advance warning of impending failure; and the dominant failure mode. If a ductile failure is not necessary, then the governing failure mode is to be the adhesive bond strength.

Strength Reduction Factors.

FDOT specifies its own set of strength reduction factors, Φ, for adhesive anchors. The capacity reduction factor for an adhesive anchor controlled by concrete embedment (i.e., bond strength in tension and concrete breakout in shear) is 0.85, or 1.0 in an extreme event load case. This is relatively high compared to the strength reduction factors recommended by ACI 318 Table 17.5.3, which do not exceed 0.75 for concrete failures as shown in

2. Table B.4. The strength reduction factor for adhesive anchors controlled by a steel failure mode (i.e., steel anchor in tension or shear) is 0.90, which is greater than 0.75 and 0.65, the factors recommended by ACI 318 Table 17.5.3 for ductile steel failure. In summary, FDOT uses less conservative factors than ACI.
3. **Sustained Tension.**
   Unlike ACI and AASHTO, FDOT prohibits the use of adhesive anchors when "predominantly sustained tension loads" are present. As a result, no calculation or additional strength reduction factor for sustained tension loading conditions is considered. However, the threshold for "predominantly" or "significant" sustained tension loads differs between FDOT and AASHTO. The threshold per FDOT is a load combination wherein the permanent component of the factored tensile load exceeds 30 percent of the factored tensile resistance for Type HV adhesives, i.e., a lower bound for the bond strength of products approved by FDOT. The threshold per AASHTO LRFD is 10 percent of the factored bond strength. Therefore, in some scenarios, FDOT may permit greater sustained tension loads than AASHTO LRFD without consideration of sustained tension in the analysis, and in others, FDOT may conservatively preclude the use of adhesive anchors compared to practice per AASHTO LRFD.
4. **Specified Pullout Capacities.**
   FDOT does not characterize adhesive anchor strength by pullout capacity.

5. **Embedment Depth and Anchor Spacing.**
   FDOT requirements for embedment depth and spacing differ from ACI 318. Regarding embedment depth, FDOT states that the embedment length must be large enough to achieve a steel anchor tensile strength of 1.25 times the yield strength or 1.0 times the tensile strength. If the anchors are in shear, then an embedment depth equal to 70 percent of the embedment depth determined for anchors in tension may be assumed. FDOT additionally requires an embedment depth of at least $6d_a$ for anchors in shear. In comparison, the minimum and maximum embedment depths specified by ACI 318, $4d_a$ and $20d_a$ respectively, are based on the theoretical limits of the bond model used in the analysis rather than the need to fully develop the steel anchor.

   Regarding anchor spacing, ACI 318 specifies a spacing of at least $6d_a$. FDOT specifies a minimum spacing of $12d_a$ for relatively high-strength adhesives (i.e., Type HSHV adhesives) and does not specify a global minimum spacing for all adhesive anchors. Additionally, ACI 318 and FDOT provide the critical anchor spacings listed in Table B.12, below which anchor group effects need to be considered. As shown, FDOT and ACI 318 both consider 3 times the edge distance to be the critical spacing for concrete breakout in shear. However, ACI 318 defines the critical spacing for bond strength as 2 times the critical distance c_{Na}, which is a function of anchor diameter d_a and uncracked characteristic bond strength τ_{uncr} while FDOT defines the critical spacing as $16d_a$.

Table B.12. Comparison between critical anchor spacings defined by ACI 318 [4] and FDOT [33].

Failure Mode	Critical Anchor Spacing per ACI 318[1]	Critical Anchor Spacing per FDOT
Concrete breakout in tension	$3h_{ef}$	n/a
Bond strength in tension	$2c_{Na}$	$16d_a$
Concrete breakout in shear	$3c_{a1}$	$3c_{a1}$

Notes: [1]Based on Table 17.5.1.3.1.

Design per the Indiana DOT

INDOT generally references AASHTO LRFD for bridge design; however, adhesive anchors are specified per INDOT's 2013 Design Manual using a unique approach [23]. To aid bridge designers, INDOT provides Figure 412-3B Design Data for Anchor Systems in 2013 Design Manual, Chapter 412 - Bridge Preservation. This figure contains a table, reproduced in Figure B.1, that provides general guidelines for hole diameter and embedment depth and reasonable strengths for Grade 60 reinforcing bars sized from No. 4 to No. 9. The footnotes of the table provide guidance for modifying the strengths in the table based on edge distance and anchor spacing.

The designer specifies the minimum pullout strength of the adhesive anchor based on Figure 412-3B. Whereas the designer selects the embedment depth in the ACI and AASHTO procedures, INDOT states that the embedment depth is to be per the manufacturer's requirements and literature. Furthermore, adjustments to the hole depth or diameter and reinforcement length required to meet the minimum pullout value specified by the designer is the responsibility of the contractor.

The differences between INDOT's procedures and the ACI and AASHTO codes are further discussed below:

1. **Governing Failure Mode.**
   The failure modes incorporated into Figure 412-3B are not fully transparent. The tension ultimate

bond strengths listed indicate that under tensile conditions, failure of the steel anchor should govern. The origin of the shear strengths provided in the figure is not identified.

2. **Strength Reduction Factors.**
   The strengths presented in Figure 412-3B do not appear to have any strength reduction factors applied. Furthermore, the text indicates that the values in Figure 412-3B are to be specified as minimum pullout values without modification.

3. **Sustained Tension.**
   While INDOT permits the use of adhesive anchors under sustained tension loading, INDOT does not provide a specific calculation for strength under sustained tension loading or modification to Figure 412-3B to account for sustained tension. However, INDOT does specify that adhesive anchors subject to permanent sustained tension or overhead applications be designed in compliance with the FHWA Technical Advisory T5140.34 issued on January 16, 2018 [24].

4. **Specified Pullout Capacities.**
   As discussed earlier, INDOT requires the designer to specify a minimum pullout capacity. The specification of a "pullout" strength is practical for communicating with the contractor, but is a misnomer in design. Per ACI 318, pullout strength is unique to mechanical anchors and the equivalent characteristic unique to adhesive anchors is bond strength. Therefore the minimum pullout capacity specified in fact represents the minimum bond strength N_{ba}. While several state DOTs specify a pullout strength, it is more common to specify the characteristic bond strength of the adhesive anchor, τ, as discussed in Section 0, **Characteristic Bond Strength**.

5. **Embedment Depth and Anchor Spacing.**
   Unlike ACI 318, INDOT does not place limitations on the embedment depth of the anchor. The minimum anchor spacing specified by INDOT is a function of the anchor diameter and embedment depth and varies from $8d_a$ to $12d_a$ for embedment depths greater than $8d_a$ and depths less than $6d_a$, respectively [24]. This meets the minimum anchor spacing $6d_a$ specified by ACI 318. However, INDOT generally assumes a critical anchor spacing of h_{ef} according to Figure 412-3B. This differs significantly from the critical anchor spacings provided by ACI 318 for various failure modes, shown in Table B.12.

Bar Size	Hole Diameter (in.)	Embedment (in.)	Tension Ultimate Bond Strength (kip)		Shear Strength (kip)
			100% f_y	125% f_y	
#4	5/8	4½	12.0	15.0	2.6
#5	3/4	5¾	18.6	23.2	4.8
#6	1	6¾	26.4	33.0	7.4
#7	1 1/8	8	36.0	45.0	10.6
#8	1 1/4	9	47.4	59.2	14.4
#9	1 3/8	10½	60.0	75.0	18.7

Notes:

1. Values are based on the use of 60-ksi reinforcement.
2. Hole diameter and embedment depth shall be per the manufacturer's requirement values shown are general guidelines.
3. Anchors are considered 100% effective if the edge distance is equivalent to, or g than, the standard embedment depth. The edge distance may be reduced to h standard embedment depth if the strength is reduced linearly to 70%.
4. Anchors are considered 100% effective if the spacing is equivalent to, or greate the standard embedment depth. Spacing may be reduced to half the st embedment depth if the strength is reduced linearly to 50%.

Figure B.1. Figure 412-3B Design Data for Anchor Systems, from INDOT's 2013 Design Manual [23].

Design per the Michigan DOT

MDOT generally adheres to the design procedures specified by AASHTO LRFD and does not specify any major modifications. However, several alternate equations are provided [28]. In these scenarios, the allowable tensile load uses a safety factor of 4 and the allowable shear load has a safety factor of 0.30, as shown in the following equations:

$$Allowable\ Tensile\ Load = \frac{(125\%) * f_y * A_t}{4}$$

$$Allowable\ Shear\ Load = 0.30 * f_y * A_t$$

The area A_t is used in both equations and represents the tensile stress area, assumed to be equivalent to the net section through the threads of a threaded anchor, or the nominal area of reinforcing steel.

Based on the commentary provided by MDOT, MDOT practice for adhesive anchors varies from the ACI and AASHTO codes in the following ways:

1. **Governing Failure Mode.**
   Like several other state DOTs, MDOT designs such that yielding of the steel anchor governs over concrete breakout or adhesive bond failure.
2. **Strength Reduction Factors.**
   MDOT does not modify the strength reduction factors provided by ACI 318.
3. **Sustained Tension.**
   MDOT currently has a moratorium on adhesive anchors in sustained tensile applications and therefore does not discuss design of adhesive anchors in sustained tension load conditions.
4. **Specified Pullout Capacities.**
   MDOT briefly discusses the role of using pull-out tests to assess the installation quality of anchors and lane ties, including adhesive anchors [36]. However, no further discussion on their specification is provided.
5. **Embedment Depth and Anchor Spacing.**
   MDOT specifies a minimum embedment depth of $9d_a$ for threaded bolts and $12d_a$ for reinforcing steel. These embedment depths are much larger than the minimum of $4d_a$ specified by ACI 318 and MDOT acknowledges that many manufacturers permit shallower embedment depths. However, MDOT chose their specified minima based on extensive testing of the products on the Qualified Products List and maintains that these relatively large embedment depths ensure the steel can fully develop 125% of its yield strength [7].

Design per the Minnesota DOT

In lieu of design procedures, MnDOT presents a detailed example for the design of adhesive anchors [35]. The example closely follows AASHTO LRFD Section 6.13.2 to evaluate the steel shear and tensile capacities and AASHTO LRFD Section 5.13 and ACI 318 Chapter 17 to evaluate the concrete shear and tensile capacities and the adhesive bond strength. Regarding the following topics:

1. **Governing Failure Mode.**
   MnDOT does not specify a governing failure mode.
2. **Strength Reduction Factors.**
   MnDOT does not modify the strength reduction factors specified by the ACI and AASHTO codes.
3. **Sustained Tension.**
   MnDOT permits adhesive anchors to be used in applications with sustained tension loading. A sustained tension load check is not shown in the design example, which shows the design of an ornamental railing post mounted on a concrete curb, but Technical Memorandum No. 18-11-B-01 [34] requires a sustained tension load check per ACI 318 Section 17.3.1.2 to be carried out.
4. **Specified Pullout Capacities.**
   In the provided design example, MnDOT describes how to specify a proof load for quality testing of adhesive anchors in the field. The proof load is the smaller of 80% of the anchor yield stress and the factored capacity of a single anchor in tension, i.e. the smaller of the factored concrete breakout

strength and the factored bond strength. For other state DOTs such as INDOT and MDOT, the term "proof load" is used in the context of qualification testing of the anchor in the laboratory than quality testing in the field.

5. **Embedment Depth and Anchor Spacing.**
   MnDOT places the same limits on embedment depth as ACI 318. However, the commentary in the design example warns designers that increasing the embedment depth does not necessarily increase anchor strength since an increase in embedment depth also increases the critical edge distance and critical anchor spacing. In some scenarios, increasing these critical parameters may cause the corresponding modification factors and subsequently the concrete strength to decrease.

Design per the Nebraska DOT or DOR

NDOR explicitly states that anchors are to be designed per Appendix D of ACI 318-11 [37]. The commentary additionally provides the following information:

1. **Governing Failure Mode.**
   NDOR does not specify a specific governing failure mode.
2. **Strength Reduction Factors.**
   NDOR does not use alternative strength reduction factors.
3. **Sustained Tension.**
   NDOR does not use adhesive anchors in sustained tensile load applications and design under sustained tension is not discussed.
4. **Specified Pullout Capacities.**
   The engineer is to specify a pullout capacity on the plans. NDOR requires steel anchors to be embedded to a sufficient depth such that the full tensile resistance of the reinforcement is developed and provides a table of the ultimate tensile force for Grade 60 reinforcement for rebar sizes No. 3 to No. 6, reproduced in Table B.13 below. The commentary implies that these values may be used as specified pull-out capacities.
5. **Embedment Depth and Anchor Spacing.**
   The engineer specifies the embedment depth of the anchor on the plans in addition to the pullout capacity. However, embedment depths may be adjusted in the field to meet the required pullout capacity. If the embedment depth is increased, then the engineer is to be informed.

Table B.13. Table of ultimate tensile force for Grade 60 reinforcement provided by NDOR [37].

Bar Size	Ultimate Tensile Force (lb)
No. 3	7,425
No. 4	13,500
No. 5	20,925
No. 6	29,700

Design per the Iowa DOT

IowaDOT explicitly states that strength design of anchors is to comply with ACI 318 and that manufacturers are to provide design parameters or allowable loads for their products [27]. The commentary provides the following information:

1. **Governing Failure Mode.**
   IowaDOT does not specify a governing failure mode for adhesive anchors.

Strength Reduction Factors.
IowaDOT requires that allowable loads recommended by the manufacturer not exceed 25% of the ultimate loads. This functionally decreases the strength reduction factors provided by ACI 318 in Table 17.5.3 (reported in Table B.3 and

2. Table B.4 in this report).
3. **Sustained Tension.**
   IowaDOT does not use adhesive anchors in sustained tensile load conditions.
4. **Specified Pullout Capacities.**
   IowaDOT does not specify pullout capacities for adhesive anchors.
5. **Embedment Depth and Anchor Spacing.**
   IowaDOT does not discuss these parameters.

Characteristic Bond Strength

The calculation for basic bond strength of an adhesive anchor, N_{ba}, was identified in Table B.6 as the characteristic bond strength τ of the adhesive multiplied by the embedded surface area $\pi * d_a * h_{ef}$ of the anchor [4]. This calculation is based on the uniform bond stress model. According to the commentary provided by ACI 318, the model applies regardless of whether the failure occurs at the interface between the concrete and the adhesive or the interface between the anchor and the adhesive. The anchor diameter d_a and embedment depth h_{ef} are selected by the designer, but the characteristic bond strength is a property of the adhesive that represents both the inherent material and the installation and use conditions. A variety of factors affect the characteristic bond strength, including the type and duration of loading, presence of concrete cracking, anchor size, drilling method, degree of concrete saturation at the time of hole drilling and anchor installation, concrete temperature and age at time of installation, and peak concrete temperatures and chemical exposure from the environment during anchor service life [4].

This means that every product has a unique characteristic bond stress that varies depending on the installation and service conditions and the appropriate characteristic bond stress for design should be determined based on testing, which is typically conducted per ACI 355.4. As such, the characteristic bond stress may not always be known during design, either because a specific product has not yet been selected or testing of the product and the effects of the anticipated installation and service conditions is not yet complete.

To address scenarios where the product-specific characteristic bond stress is not known, ACI 318 provides lower-bound default values in Table 17.6.5.2.5 (see Table B.8) for designers to use. These values are the minimum values that must be met for the anchor to meet the qualification requirements of ACI 355.4, and are therefore safe to assume provided the anchor installation and service environment is well-understood and characterized correctly. However, the commentary acknowledges that these minimum τ are conservative, stating that certain anchors in a dry, indoor environment in uncracked conditions can demonstrate a characteristic bond stress between 2000 and 2500 psi, instead of 1000 psi as listed in Table 17.6.5.2.5. The high level of conservatism makes use of the default values undesirable.

Additionally, Table 17.6.5.2.5 has limited applications and its interpretation can be challenging. Regarding limitations, the commentary clarifies that Table 17.6.5.2.5 only applies for holes made with a rotary impact drill or rock drill. Compared to these methods, core-drilled holes provide a relatively smooth surface, resulting in a decreased characteristic bond strength., and if a core-drilled hole is used, then the characteristic bond strength must be determined from testing per ACI 355.4.

Regarding interpretation, the commentary warns that “indoor” and “outdoor” are not meant to be taken literally. For example, for anchors installed on the interior of a structure but prior to construction of the building envelope, an “outdoor” environment should be assumed because the anchors may be subject to rainfall or other precipitation, causing the concrete to be saturated during anchor installation. The commentary provides the following definitions [4]:

> *“Indoor conditions represent anchors installed in dry concrete…and subjected to limited concrete temperature variations over the service life of the anchor. Outdoor conditions are assumed to occur if, at the time of installation, the concrete is exposed to weather that might leave the concrete wet. Anchors installed in outdoor conditions are also assumed to be subject to greater concrete temperature variations such as might be associated with freezing and thawing or elevated temperatures resulting from direct sun exposure.”*

As a result, characterization of the exposure conditions during installation relies on the designer’s judgment. Because the presence of moisture decreases the characteristic bond stress by a large amount, clearer direction for the characterization of exposure conditions is highly desirable.

Several state DOTs, including WisDOT, FDOT, and MnDOT, provide their own guidance for selecting a lower-bound characteristic bond strength. The alternate strengths provided by these state DOTs are based on their own qualifying criteria for adhesive anchors and pre-approved product lists. Other state DOTs, including MDOT and INDOT, sidestep the need to define a characteristic bond strength by requiring adhesive anchors to fail by yielding of the steel rather than bond strength. In these cases, adhesive anchors typically become pre-approved via pullout testing, wherein the adhesive anchor must demonstrate the ability to develop the ultimate or yield strength of the steel anchor. While a check for adhesive bond strength should still be conducted, the characteristic bond stress used in design becomes less critical. Further details on the approaches of select state DOTs are discussed in the following subsections.

Bond Strength per the Wisconsin DOT

WisDOT takes a similar approach to ACI 318 and provides minimum values for characteristic bond strength in Table 40.16-1, Tension Design Table for Concrete Anchors, of the DOT's Bridge Design Manual [3]. The table is shown in Figure B.2. Different minimum strengths are specified based on the moisture condition of the concrete at the time of installation (dry or water-saturated), the presence of cracking, and anchor diameter. The values are based on testing of the products on WisDOT's Approved Products List (APL) and represents the 5% fractile of the results of testing performed and evaluated according to ICC-ES AC308 [38]or ACI 355.4 [17].

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	460	370	280
#5 or 5/8"	970	460	510	390
#6 or 3/4"	950	490	500	410
#7 or 7/8"	930	490	490	340
#8 or 1"	770	490	600	340

Table 40.16-1
Tension Design Table for Concrete Anchors

Figure B.2. Table provided by WisDOT that specifies the minimum characteristic bond strengths to be assumed in design of adhesive anchors [3].

Bond Strength per the Florida DOT

FDOT refers designers to Section 937, Post-Installed Anchor Systems for Structural Applications in Concrete Elements, of the FDOT Standard Specifications [39]. Table 937-1, Uniform Bond Stress, within this section provides minimum characteristic bond strength requirements for adhesive anchors in a variety of scenarios, as shown in Figure B.3. The characteristic bond strength of the anchors is determined in accordance with FM 5-568, Florida Method of Test for Anchor System Tests for Adhesive-Bonded Anchors and Dowels [40]. Static tension tests of single anchors are conducted per ASTM E488, Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements, and ASTM E1512, Standard Test Methods for Testing Bond Performance of Adhesive-Bonded Anchors, as applicable. The installation and service conditions evaluated include:

Confined Tension. FDOT defines this as a situation wherein the reaction force from a static tension load is sufficiently close to the anchor to preclude concrete failure, but allow bond failure.

Damp-Hole Installation. A definition is not given in FM 5-568 but commentary accompanying the design procedure indicates this testing represents saturated-surface-dry conditions during anchor installation.

Elevated Temperature. The minimum temperature during testing is 108°F (42°C).

Horizontal Orientation. The longitudinal axis of the anchor is horizontal during installation and curing.

Short-Term Cure. The longitudinal axis of the anchor is horizontal during installation and curing and test loads are applied no later than 24 hours after installation.

Unconfined Tension. FDOT defines this as a situation wherein the reaction force from a static tension load is a sufficient distance from the anchor such that concrete failure or bond failure may occur.

Testing is typically conducted using anchors with a diameter of 5/8 inches (16 mm) and an embedment depth of 4 inches (102 mm), except in tests evaluating unconfined tension which evaluate three d_a-to-h_{ef} aspect ratios. The tension failure load is measured and used to calculate the uniform bond stress according to the uniform bond stress model implemented by ACI 318. The coefficient of variation is also evaluated from the Unconfined Tension tests. The specified bond strength for the adhesive product is calculated using the uniform bond stress (characteristic bond strength) from the unconfined tension testing τ_u and coefficient of variation from the unconfined tension testing COV_u as follows:

$$\tau = \tau_u * (1 - 2COV_u)$$

This is the origin for the Specified Bond Strengths in FDOT's Table 937-1 [39]. FDOT maintains two classes of adhesive anchors: Type HV and Type HSHV, the latter of which is a higher strength anchor. Type HV anchors are intended for structural applications and Type HSHV anchors are intended for use in traffic railing retrofit applications where mechanical anchors are not practical and the predominant loading is from vehicle impact. Type HSHV anchors are not intended for sustained tension loads, and it should be noted that FM 5-568 includes Long-Term Load (Creep) testing to assess the effect of creep on the characteristic bond strength in the Confined Tension scenario [40].

In the design commentary, FDOT notes that adhesive anchors are installed in clean, dry holes drilled in hardened concrete and that installation in holes that are in saturated-surface-dry condition is not pre-approved or recommended. However, installation under saturated-surface-dry conditions may be approved on a case-by-case basis. FDOT provides general guidance that the damp-hole strength of products on its APL is approximately 75% that of dry-hole strength [33].

Table 937-1
Uniform Bond Stress

	Type HV	Type HSHV
Confined Tension	2,290 psi	3,060 psi
Damp-Hole Installation	1,680 psi	1,830 psi
Elevated Temperature	2,290 psi	3,060 psi
Horizontal Orientation	2,060 psi	2,060 psi
Short Term Cure	1,710 psi	1,710 psi
Specified Bond Strength	1,080 psi	1,830 psi

Maximum Coefficient of Variation for Uniform Bond Stress: 20%.

Figure B.3. Table provided by FDOT that specifies the minimum characteristic bond strengths for adhesive anchors [39].

Bond Strength per the Minnesota DOT

MnDOT maintains detailed Approved/Qualified Products Lists for adhesive anchors. These lists are shown in Figure B.4 and Figure B.5. Pre-approval for use in sustained tension applications, with various sizes of anchors, with hammer-drilled holes, and with core-drilled holes is identified. All adhesive anchors using reinforcing bars must demonstrate an uncracked characteristic bond strength of at least 1,000 psi and a cracked characteristic bond strength of at least 500 psi [41]. There are two classifications of threaded rods, for which the lower-strength class has the same requirements as reinforcing bars. Adhesive anchors using threaded rods within the higher-strength class must have an uncracked characteristic bond strength of at least 1,500 psi and a cracked characteristic bond strength of at least 750 psi [42]. In the adhesive anchor design example, MnDOT selected a characteristic bond strength of 1,500 psi based on the design scenario (uncracked concrete and threaded rod anchor) and the specified minimum. Due to the framework of the A/QPL, no adjustments or other considerations needed to be made based on anchor diameter or hole type.

The prequalification process specified by MnDOT additionally considers the effects of damp holes and corrosion protection methods, such as the use of epoxy-coated, galvanized, or stainless steel [43]. To address the effects of moist installation holes, MnDOT requires that the adhesive anchor must have a strength reduction factor in wet concrete corresponding to that of a Category 2 anchor without supplementary reinforcement (0.55 per ACI 318 Table 17.5.3). Additionally, MnDOT requires testing per AC308 Section 3.4 demonstrating that the adhesive will meet the specified strength requirements for hot-dipped galvanized ASTM F1554 threaded rods, stainless steel threaded rods, epoxy-coated reinforcing, and stainless steel reinforcing.

For use in sustained tension applications

Product	Manufacturer	Approval expiration	Rebar Designations	Hole type: Hammer-drilled	Hole type drilled
A7+	ITW CCNA	6/25/2023	Approved for Rebar Designations #4 through #10	Approved	Not appro
AC200+	DEWALT	3/15/2024	Approved for Rebar Designations #4 through #10	Approved	Not appro
HILTI HIT-HY 100	Hilti	10/22/2021	Approved for Rebar Designations #4 through #10	Approved	Not appro
RE 500v3	Hilti	6/25/2023	Approved for Rebar Designations #4 through #10	Approved	Not appro
Red Head C6+	ITW Red Head	10/31/2020	Approved for Rebar Designations #4 through #10	Approved	Approved
ULTRABOND HS-1CC	Adhesive Technology Corp	10/22/2021	Approved for Rebar Designaton #4; not approved for Rebar Designations #5 through #10	Approved	Not appro

Prohibited from use in sustained tension applications

Product	Manufacturer	Approval expiration	Rebar Designations	Hole type: Hammer-drilled	Hole type: C drilled
AC100+ Gold	DEWALT	3/15/2024	Approved for Rebar Designations #4 through #9	Approved	Not approve
Red Head G5+	ITW Red Head	10/31/2020	Approved for Rebar Designations #4 through #10	Approved	Not approve

Figure B.4. MnDOT's Approved/Qualified Products List for adhesive anchors using reinforcing bars [41].

Min. uncracked characteristic bond strength of 1 KSI and min. cracked characteristic bond strength of 0.5

For use in sustained tension applications

Product	Manufacturer	Approval expiration	Threaded rod diameters	Hole type: Hammer-drilled	Hold type drilled
HILTI HIT-HY 100	Hilti	10/22/2021	Approved for 1/2", 5/8", 3/4", 7/8", 1"; not approved for 1 1/4"	Approved	Not appro
Red Head A7+	ITW CCNA	6/25/2023	Approved for 1/2", 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Not appro
Red Head C6+	ITW CCNA	10/31/2020	Approved for 1/2", 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Approved
Ultrabond 365CC	Adhesive Technology Corp	4/26/2020	Approved for 1/2", 5/8", 3/4"; not approved for 7/8", 1", 1 1/4"	Approved	Not appro

Prohibited from use in sustained tension applications

Product	Manufacturer	Approval expiration	Threaded rod diameters	Hole type: Hammer-drilled	Hold type: (
AC100+ Gold	DEWALT	3/15/2024	Approved for 1/2", 5/8", 3/4", 7/8", 1"; not approved for 1 1/4"	Approved	Not approve
Red Head G5+	ITW CCNA	10/31/2020	Approved for 1/2", 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Not approve

Min. uncracked characteristic bond strength of 1.5 KSI and min. cracked characteristic bond strength of 0.75

For use in sustained tension applications

Product	Manufacturer	Approval expiration	Threaded rod diameters	Hole type: Hammer-drilled	Hold type: Core-drilled
AC200+	DEWALT	3/15/2024	Approved for 1/2", 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Not approved
HILTI HIT-HY 100	Hilti	10/22/2021	Approved for 5/8", 3/4"; not approved for 1/2", 7/8", 1", 1 1/4"	Approved	Not approved
RE-500v3	Hilti	6/25/2023	Approved for 1/2", 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Not approved
Red Head A7+	ITW CCNA	6/25/2023	Approved for 1/2", 5/8", 3/4"; not approved for 7/8", 1", 1 1/4"	Approved	Not approved
Red Head C6+	ITW CNNA	10/31/2020	Approved for 1/2", 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Approved only for 1/2" an diameter rods
ULTRABOND HS-1CC	Adhesive Technology Corp	10/22/2021	Approved for 1/2"; not approved for 5/8", 3/4", 7/8", 1", 1 1/4"	Approved	Not approved

Prohibited from use in sustained tension applications

Product	Manufacturer	Approval expiration	Threaded rod diameters	Hole type: Hammer-drilled	Hold type: (
Red Head G5+	ITW CCNA	10/31/2020	Approved for 1/2", 5/8", 3/4"; not approved for 7/8", 1", 1 1/4"	Approved	Not approve

Figure B.5. MnDOT's Approved/Qualified Products List for adhesive anchors using threaded rods [42].

Bond Strength per the California DOT

Caltrans does not offer guidance for selecting a characteristic bond strength in design but does specify a minimum characteristic bond strength for pre-approved adhesive anchors [44]. While both two-part polymers or polymer mortars and resin capsules classify as adhesives used in adhesive anchors, Caltrans maintains separate Authorized Materials Lists (AMLs) for Chemical Adhesives (Drill and Bond Dowel) and Resin Capsule Anchors.

Chemical adhesives are required to have a factored bond strength $\Phi^*\tau_{cr}$ of at least 540 psi if threaded rods are used and 490 psi if reinforcing bars are used. Caltrans specifies that the strength reduction factor Φ must be consistent with the following conditions [44]:

- Exterior exposure or damp environments;
- Cracked concrete;
- Water-saturated concrete;
- Periodic inspection;
- No sustained tension;
- Horizontal and overhead installations;
- Concrete compressive strength of 4000 psi;
- Long term peak in-service concrete temperature greater than or equal to 110°F;
- Short term peak in-service concrete temperature greater than or equal to 165°F; and
- Non-seismic anchor.
- No further guidance on the strength reduction factor and the values to assume for the listed conditions is provided.
- The prequalification requirements for resin capsule anchors are currently under revision and therefore not available at this time.

State DOTs that Specify Pullout Capacity

INDOT, MDOT, and NYSDOT require that adhesive anchors demonstrate a minimum tensile pullout capacity instead of a minimum characteristic bond strength during prequalification testing. INDOT specifies that adhesive anchors must be capable of withstanding a tensile load equal to the yield strength of a No. 7, Grade 60, epoxy-coated, deformed steel rebar [45]. As such, all chemical adhesives on INDOT's QML are pre-approved for pullout loads that do not exceed the above specified yield strength and with reinforcing steel not exceeding No. 7 in size. INDOT requires project-specific testing or documentation for larger rebar [24].

Per MDOT, products must demonstrate the ability to develop 125% of the yield strength of the anchor in tension, and 100% of the yield strength of the anchor in shear. The adhesive must meet these requirements for ASTM A307 bolts 0.375 to 0.875 inches in diameter at a maximum embedment depth of $9d_a$, and for Grade 60 reinforcing steel sizes No. 4 to No. 8 at a maximum embedment depth of $12d_a$. As part of the prequalification process, MDOT runs three pullout tests using No. 6 rebar in concrete with a compressive strength of 4000 psi [36].

The NYSDOT prequalification process contains a screening step before the department will accept a product for internal testing by its Materials Bureau. Manufacturers must first submit data demonstrating that their product will meet the appropriate minimum pullout value as identified in Table B.14. For this testing, a 1-inch diameter threaded rod is used with an embedment depth of 10 inches. If the product meets this requirement, then NYSDOT will run a second set of tests using 5/8-inch diameter threaded rods and an embedment depth of 4 inches. The product must meet the corresponding minimum pullout value identified in Table B.14. Testing is typically conducted using concrete with a compressive strength of 4000 psi, but minimum pullout loads for tests using alternative concrete strengths are also provided [46].

Table B.14. Pullout loads specified by NYSDOT for prequalification of chemical adhesives.

Required Pullout Loads				
Screening Step (d_a = 1 inch, h_{ef} = 10 inches, threaded rod anchor)				
Concrete Strength (psi)	≤ 4000	4500	5000	5500
Minimum Pullout Load (lb)	51,120	54,225	57,150	59,940
NYSDOT Testing Step (d_a = 0.625 inches, h_{ef} = 4 inches, threaded rod anchor)				
Concrete Strength (psi)	≤ 4000	4500	5000	5500
Minimum Pullout Load (lb)	8,593	9,113	9,630	10,080

Source: Standard Specifications, Volume 4, Section 701-07, Anchoring Materials - Chemically Curing [46]

APPENDIX C. ADHESIVE ANCHOR DESIGN EXAMPLES

Appendix C1 – Design Example 1 – Wing Wall Replacement

Appendix C2 – Design Example 2 – Abutment Extension

Appendix C3 – Design Example 3 – Concrete Parapet replacement

DESIGN EXAMPLE 1 - WING WALL REPLACEMENT
PART 1 - DETAILED CALCULATIONS

General Information

In this example, flexrual resistance of a replaced concrete parapet connected to the existing bridge deck with adhesive anchors is calculated in accordance with AASHTO LRFD 9th Edition. Strength of adhesive anchors is calculated in accordance with ACI 318-19.

References

- AASHTO LRFD 9th Edition
- ACI 318-19 Building Code Requirements for Structural Concrete and Commentary
- WisDOT Bridge Manual 2020

Note: All sectional references in the calculations refer to the WisDOT Bridge Manual 2020 unless otherwise noted.

General Assumptions

- Concrete of the existing abutment is cracked for the purpose of calculating anchor resistances.
- Reinforcement in the existing abutment can function as supplementary reinforcement for or the purpose of calculating anchor resistances.

<u>Design Parameters</u>

Soil Properties

$\phi := 30deg$ — Angle of internal friction

$\gamma_s := 120pcf$ — Soil unit weight

$c := 0$ — Soil cohesion

$\delta := 0.67 \cdot \phi = 20 \cdot deg$ — Friction angle between backfill and wall, LRFD C3.11.5.3

Reinforced Concrete Parameters

$w_c := 0.15$ — Unit weight of concrete (kcf)

$f'_c := 3.5ksi$ — Concrete compressive strength

$f_y := 60ksi$ — Yield strength of reinforcing bars (anchors)

$E_s := 29000ksi$ — Modulus of elasticity of steel reinforcment

$E_c := 33000 \cdot (w_c)^{1.5} \cdot \sqrt{\frac{f'_c}{ksi}} ksi = 3587 \cdot ksi$ Modulus of elasticity of concrete, ACI 318 19.2.2.1a

$c_c := 2in$ Concrete cover per ACI 318 20.5.1.3.1 for No. 6 bars exposed to weather or in contact with ground

$d_{agg} := 1.5in$ Assumed max aggregate size

Resistance Factors, LRFD 5.5.4.2

$\phi_v := 0.9$ Reinforced concrete in shear

Geometry

$H_U := 5.5ft$ Height of upper wing

$H_L := 5ft$ Height of lower wing

$L_U := 14ft$ Length of upper wing wall

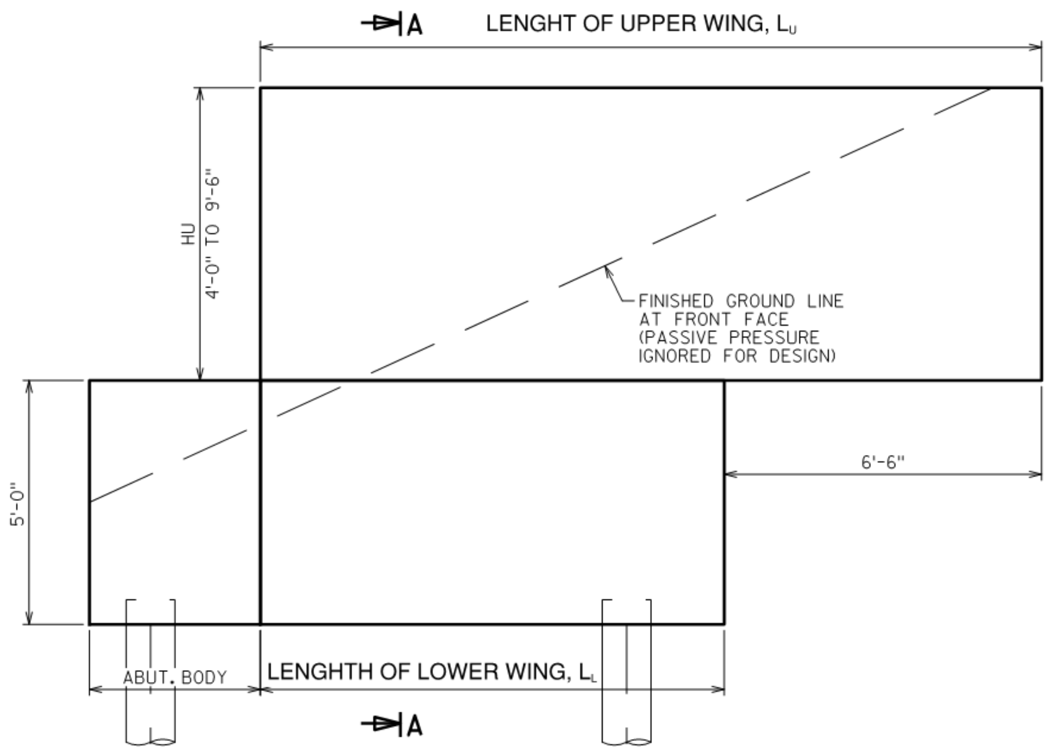
$L_L := 7.5ft$ Length of lower wing wall

$B_U := 15in$ Thickness of upper wing

$B_L := 39in$ Thickness of lower wing

$\beta := 0deg$ Angle of fill to horizontal

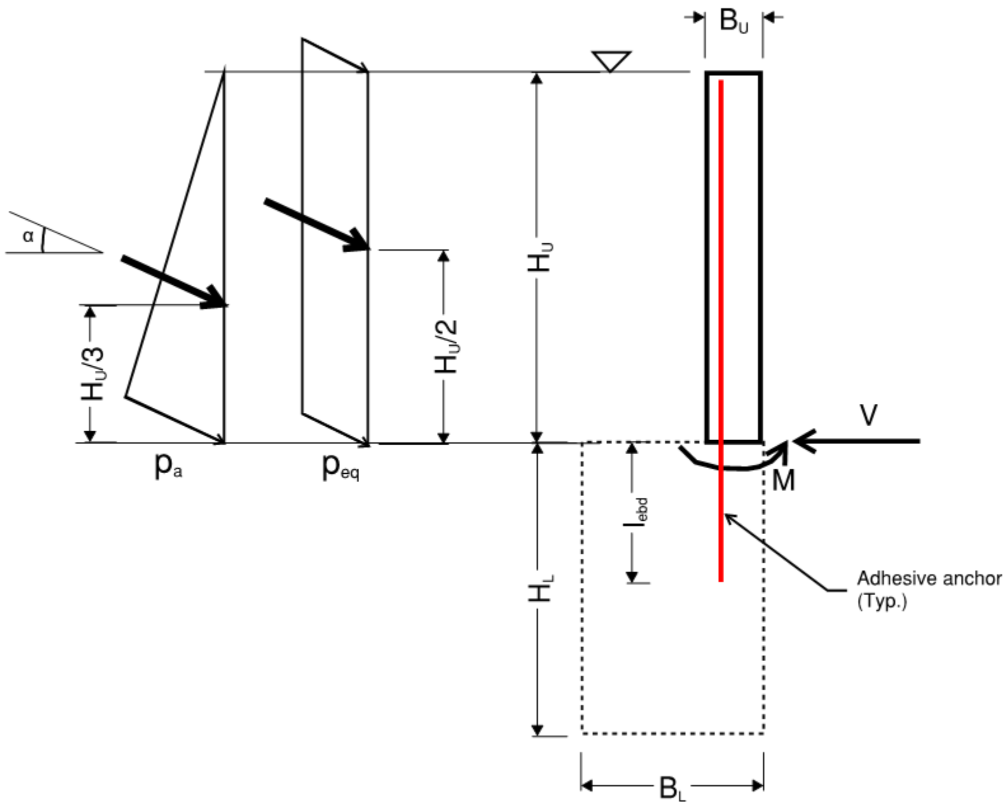
$\theta := 90deg$ Angle of back face of wall to the horizontal



ELEVATION - WING WITH PILE

Note: calculations of forces and resistances are for 1 foot long of wall.

Loads



WALL SECTION

Active Earth Lateral Pressure

Compute the coefficient of active earth pressure per LRFD 3.11.5.3

$\phi = 30 \cdot deg$

$\delta = 20.1 \cdot deg$

$\beta = 0 \cdot deg$

$\theta = 90 \cdot deg$

$\alpha := 90deg - \theta + \delta = 20.1 \cdot deg$

$$\Gamma := \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2 = 2.687$$

LRFD Eq. 3.11.5.3-2

$$k_a := \frac{\sin(\theta + \phi)^2}{\Gamma \cdot \sin(\theta)^2 \cdot \sin(\theta - \delta)}$$

$k_a = 0.297$

$p_a := k_a \cdot \gamma_s \cdot H_U = 196.2 \cdot psf$

Lateral active earth pressure (LRFD 3.11.5.1).
Note: The equivalent fluid unit weight of soil for

estimating lateral earth pressure per LRFD 3.11.5.5 is only applicable where Rankine earth pressure theory (as discussed in LRFD C3.11.5.3) is applicable

Live load surchage

$h_{eq} := 2ft$

Equivalent height of soil for surcharge live load on walls parallel to traffic (12.8.3).

$p_{eq} := h_{eq} \cdot \gamma_s \cdot k_a = 71.3 \cdot psf$

Load combinations and load factors

Load	Strength I	Service I
Lateral earth pressure, active, EH	1.50	1.00
Live load surcharge, LS	1.75	1.00

Caculations of Forces in the Wall

Calculate shear and bending moments at the bottom of the upper wing under lateral earth pressure and lateral pressure from live load surcharge. Self weight of the wall has only minimal effect on its flexural resistance and thus is disregarded in this example (consistent with ACI 318 11.5.2.2 for nonbearing walls).

$$\eta := \frac{L_U}{L_L} = 1.87$$

When the upper wing wall is longer than the lower wing wall (e.g. for wing with pile and length of upper wing exceeds 12 ft), the design forces at the bottom of the upper wall are multiplied with an amplification factor η.

$$F_{EH} := \frac{\eta \cdot p_a \cdot H_U \cdot 1ft}{2} = 1.01\,kip$$

Resultant force of horizontal active earth pressure

$$F_{EH\_X} := F_{EH} \cdot \cos(\alpha) = 0.95\,kip$$

Horizontal component of the resultant force of horizontal active earth pressure

$$F_{LS} := \eta \cdot p_{eq} \cdot H_U \cdot 1ft = 0.73 \cdot kip$$

Resultant force of surcharge pressure

$$F_{LS\_X} := F_{LS} \cdot \cos(\alpha) = 0.69 \cdot kip$$

Horizontal component of resultant force of surcharge pressure

Strength I

Load factors (12.8.2):

$\gamma_{EH1} := 1.5$

$\gamma_{LS1} := 1.75$

$\gamma_{DC\_S1} := 0.9$

$V_{u1} := \gamma_{EH1} \cdot F_{EH\_X} + \gamma_{LS1} \cdot F_{LS\_X} = 2.62 \cdot kip$

Shear force per foot length of wall (Strength I)

$M_{u1} := \gamma_{EH1} \cdot F_{EH\_X} \cdot \frac{H_U}{3} + \gamma_{LS1} \cdot F_{LS\_X} \cdot \frac{H_U}{2} = 5.91 \cdot kip \cdot ft$

Total bending moment per foot length of wall (Strength I)

$M_{u1\_s} := \gamma_{EH1} \cdot F_{EH\_X} \cdot \frac{H_U}{3} = 2.6 \, ft \cdot kip$

Bending moment due to earth pressure (sustained loads) per foot length of wall (Strength I)

Service I

Load factors (12.8.2):

$\gamma_{EH2} := 1.0$

$\gamma_{LS2} := 1.0$

$V_{u2} := \gamma_{EH2} \cdot F_{EH\_X} + \gamma_{LS2} \cdot F_{LS\_X} = 1.6 \, kip$

$M_{u2} := \gamma_{EH2} \cdot F_{EH\_X} \cdot \frac{H_U}{3} + \frac{\gamma_{LS2} \cdot F_{LS\_X} \cdot H_U}{2} = 3.63 \cdot kip \cdot ft$

Design of Adhesive Anchors

Concrete and Steel Anchor Properties

$f'_c = 3500 \, psi$

Concrete compressive strength

$f_{ua} := 80ksi$

Steel anchor tensile strength, ASTM A615 Grade 60

$f_{ya} := 60ksi$

Steel anchor yield strength, ASTM A615 Grade 60

Characteristic Bond Stress

$\tau_{cr} := 450psi$

Characteristic bond stress for cracked concrete, minimum value to avoid bond as the governing failure mode

$\tau_{uncr} := 3 \cdot \tau_{cr} = 1350 \, psi$

Characteristic bond stress for uncracked concrete, assumed to be three times the characteristic bond stress for cracked concrete. For cracked concrete, τ_{cr} is used to calculate the basic bond strength N_{ba}, but τ_{uncr} is used to calculate c_{Na}. For a given τ_{cr}, higher

τ_{uncr} actually decreases the bond strength N_a. Thus, to be conservative a maximum τ_{uncr}/τ_{cr} is chosen. For products in the WisDOT approved product list, τ_{uncr}/τ_{cr} ranges from 1.1 to 3.0.

Resistance Factors, LRFD 5.5.4.2

$\phi_{ts} := 0.75$

Strength reduction factor for steel in tension for ductile steel, 40.16.2 and & ACI 318 17.5.3. Rebars are considered ductile.

$\phi_{tc} := 0.75$

Strength reduction factor for concrete breakout and bond in tension for Anchor Category 1 with supplementary reinforcement, 40.16.3 & ACI 318 17.5.3. It is assumed that the existing vertical reinforcement in the lower wing wall functions as supplementary reinforcement for adhesive anchors as described in ACI 318 R17.5.3.

$\phi_{vs} := 0.65$

Strength reduction factor for steel in shear for ductile steel, 40.16.2 & ACI 318 17.5.3. Rebars are considered ductile.

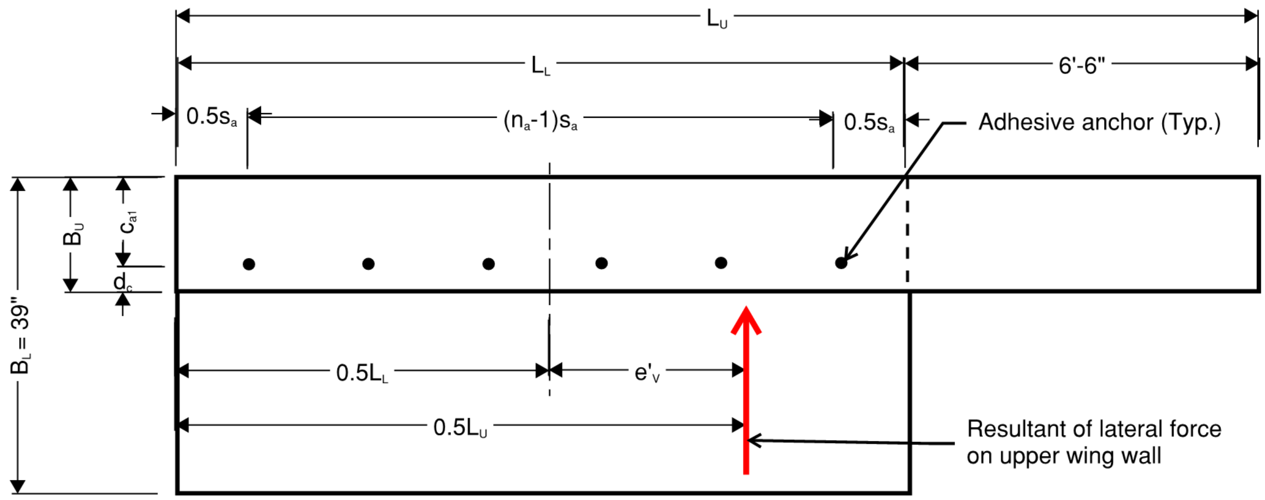
$\phi_{vc} := 0.75$

Strength reduction factor for concrete breakout in shear with supplementary reinforcement, 40.16.4 & ACI 318 17.5.3.

$\phi_{vp} := 0.70$

Strength reduction factor for concrete pryout in shear with supplementary reinforcement per ACI 318 17.5.3.

Geometries



n_a = number of anchors

PLAN VIEW

$H_L = 5 \cdot ft$	Height of lower wing wall
$L_L = 7.5 \cdot ft$	Length of lower wing wall
$h_a := H_L = 60\,in$	Concrete thickness
$d_a := 0.75in$	Diameter of anchor, #6 rebar
$n_a := 6$	Number of adhesive anchors
$s_a := \dfrac{L_L}{n_a} = 15\,in$	Anchor spacing longitudinally
$d_c := c_c + \dfrac{d_a}{2} = 2.4\,in$	Cover from center of anchors to back face of upper wing
$l_{ebd} := 15in$	Actual embedment depth
$c_{a1} := B_U - d_c = 12.6\,in$	Edge distance parallel to shear force
$c_{a2} := \dfrac{s_a}{2} = 7.5\,in$	Edge distance perpendicular to shear force
$c_{a3} := B_L - c_{a1} = 26.4\,in$	
$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15\,in$	Effective embedded length. ACI 318 17.3.3

$4 \cdot d_a \le h_{ef} \le 20 d_a = 1$ OK Embedment depth limit, ACI 318 17.3.3

$$A_{se\_N} := \frac{\pi \cdot d_a^{\,2}}{4} = 0.44\ in^2$$ Anchor cross-sectional area in tension

$$A_{se\_V} := \frac{\pi \cdot d_a^{\,2}}{4} = 0.44\ in^2$$ Anchor cross-sectional area in shear

$c_{a\_min} := \min\left(c_{a1}, c_{a2}, c_{a3}\right) = 7.5\ in$ Min edge distance

Check edge distances, spacings

$s_{a\_min} := 6 \cdot d_a = 4.5\ in$ Minimum spacing required, ACI 318 17.9.2

$c_{edge\_min} := \max\left(c_c, 2 \cdot d_{agg}, 6 \cdot d_a\right) = 4.5\ in$ Minimum edge distance required, ACI 318 17.9.2

$s_a \ge s_{a\_min} = 1$ OK

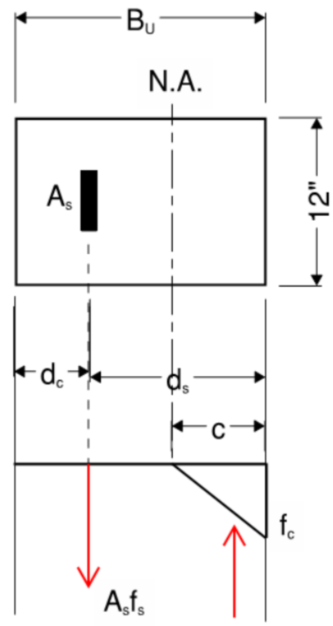
$c_{a\_min} \ge c_{edge\_min} = 1$ OK

Check Adhesive Anchors in Tension

Tensile Force in Anchor

Calculate tensile stress in anchors due to bending moment (Strength I) assuming linear compressive stress distribution in concrete and no tensile stress in concrete. Effect of self weight of the upper wall on flexural resistance of the wall is disregarded (conservative).

$A_s := \frac{A_{se\_N}}{s_a} \cdot 12in = 0.35\,in^2$ Area of reinforcement per foot length of wall



$d_s := B_U - d_c = 12.6\,in$

$\rho := \frac{A_s}{d_s \cdot 12in} = 0.0023$

$n := \frac{E_s}{E_c} = 8.1$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n = 0.18$

$j := 1 - \frac{k}{3} = 0.94$

$M_{u1} = 5.91\,ft \cdot kip$ Bending moment per foot length of wall due to factored loads

$d_s = 12.6\,in$

$f_{s\_ua} := \frac{M_{u1}}{A_s \cdot j \cdot d_s} = 16.9 \cdot ksi$

$T_{ua} := A_s \cdot f_{s\_ua} = 5.97\,kip$ Tensile force in anchor per foot length of wall due to factored loads

$N_{uag} := T_{ua} \cdot \frac{L_L}{1ft} = 44.8\,kip$ Total tensile force in anchor group due to factored loads

$N_{ua} := \frac{N_{uag}}{n_a} = 7.5\,kip$ Tensile force in one anchor due to factored loads

$N_{ua\_s} := N_{ua} \cdot \frac{M_{u1\_s}}{M_{u1}} = 3.3\,kip$ Tensile force in one anchor due to factored sustained load

Anchor tensile strength

The design in this example has two rows of anchors. It is anticipated that the interior row, which is further from the compression zone in the upper wall, functions as the main tension reinforcement resisting bending moment in the upper wall. Depending on the location of the neutral axis in the upper wall cross section, the exterior row may be in tension or compression. For simplicity, the contribution of the exterior row of anchors to the flexural resistance of the wall is conservatively disregarded in this example.

Tensile strength - steel

$f_{uta} := \min\left(f_{ua}, 1.9 \cdot f_{ya}, 125\text{ksi}\right) = 80 \cdot \text{ksi}$ Specified tensile strength of anchor steel, 40.16.3

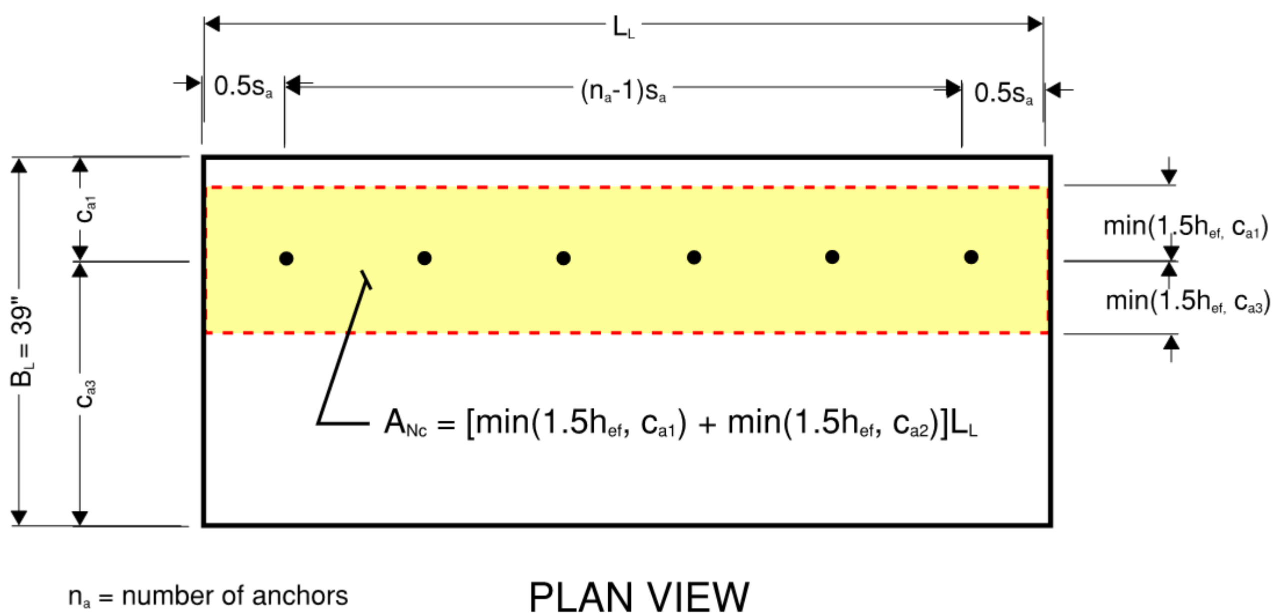
$N_{sa} := A_{se\_N} \cdot f_{uta} = 35.3\,\text{kip}$

$\phi_{ts} \cdot N_{sa} = 26.5\,\text{kip}$

Yield strength of steel anchor

$N_{ya} := A_{se\_N} \cdot f_{ya} = 26.5\,\text{kip}$

Tensile strength - Concrete breakout



$c_{a1} = 12.6\,\text{in}$ $c_{a2} = 7.5\,\text{in}$ $c_{a3} = 26.4\,\text{in}$

$$check_1 := \begin{cases} 1 & \text{if } c_{a1} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_2 := \begin{cases} 1 & \text{if } c_{a2} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_3 := \begin{cases} 1 & \text{if } c_{a3} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}$$

$c_{a\_max} := \max\left(c_{a1} \cdot check_1, c_{a2} \cdot check_2, c_{a3} \cdot check_3\right) = 12.6\ in$ — Max edge distance that does not exceed 1.5hef

$h_{ef} = 15\ in$ $\quad s_a = 15\ in$

$$h'_{ef} := \begin{cases} h_{ef} & \text{if } \left[\left(c_{a1} \ge 1.5 \cdot h_{ef}\right) \land \left(c_{a3} \ge 1.5 \cdot h_{ef}\right)\right] \lor \left(c_{a2} \ge 1.5 \cdot h_{ef}\right) \\ \max\left(\dfrac{c_{a\_max}}{1.5}, \dfrac{s_a}{3}\right) & \text{otherwise} \end{cases}$$

If anchors are located less than 1.5hef from three or more edges, hef for concrete breakout is corrected per ACI 17.6.2.1.2

$h'_{ef} = 8.4\ in$ — Corrected hef for concrete breakout

$A_{Nco} := 9 \cdot h'^{2}_{ef} = 637.6\ in^2$ — 40.16.3 and ACI 318 17.6.2.1.4

$n_a \cdot A_{Nco} = 3825.4\ in^2$

$A_{Nc1} := \left(\min\left(1.5 \cdot h'_{ef}, c_{a1}\right) + \min\left(1.5h'_{ef}, c_{a3}\right)\right) \cdot L_L = 2272.5\ in^2$

$A_{Nc} := \min\left(A_{Nc1}, n_a \cdot A_{Nco}\right) = 2272.5\ in^2$ — ACI 17.6.2.1.1

$\psi_{ec\_N} := 1$ — The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered. ACI 17.6.2.3

$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \ge 1.5 \cdot h'_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{1.5 \cdot h'_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$\psi_{ed\_N} = 0.88$

$\psi_{c\_N} := 1.0$ — For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$\psi_{cp\_N} := 1$ — Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$ — kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$$N_b := k_c \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h'_{ef}}{in}\right)^{1.5} lbf = 24.6 \cdot kip$$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 76.87 \cdot kip$$

Nominal concrete breakout strength of anchor group in tension (ACI 17.6.2.1b)

$$\phi_{tc} \cdot N_{cbg} = 57.66 \cdot kip$$

Anchor bond strength

$$\tau_{cr} = 450\,psi$$

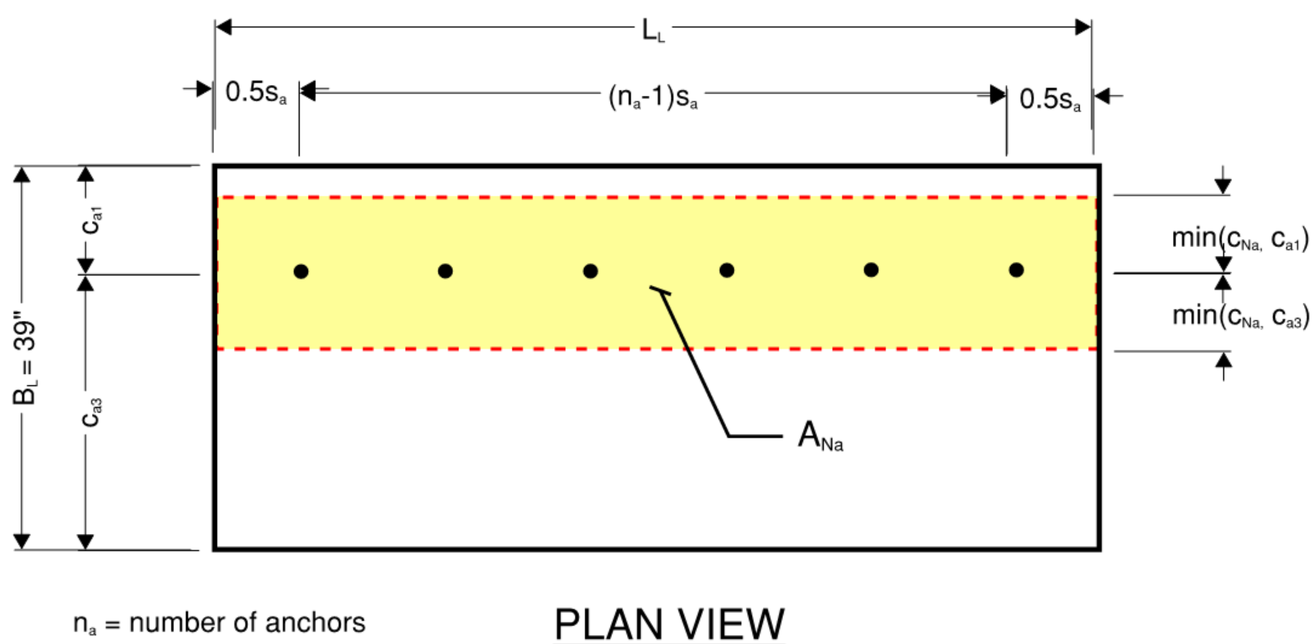
$$\tau_{uncr} = 1350\,psi$$

$$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 15.9\,kip$$

Basic bond strength, assuming cracked concrete given uncertain condition of the existing lower wing wall. 40.16.3 and ACI 318 17.6.5.2.1

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 8.31\,in$$

ACI 318 17.6.5.1.2b



$$A_{Nao} := 4 \cdot c_{Na}^{\,2} = 276.1\,in^2$$

ACI 318 Eq. 17.6.5.1.2a

$n_a \cdot A_{Nao} = 1656.8\ in^2$

$A_{Na1} := \left(\min\left(c_{Na}, c_{a1}\right) + \min\left(c_{Na}, c_{a3}\right)\right) \cdot L_L = 1495.6\ in^2$

$A_{Na} := \min\left(A_{Na1}, n_a \cdot A_{Nao}\right) = 1495.6\ in^2$

$\psi_{ec\_Na} := 1$

The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered.

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \ge c_{Na} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 0.97$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the lower wing wall functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 83.6\ kip$$

Nominal bond strength of anchor group, ACI 318 17.6.5.1

$\phi_{tc} \cdot N_{ag} = 62.7\ kip$

<u>Check anchors in tension (ACI 17.5.2)</u>

$n_a \cdot \left(\phi_{ts} \cdot N_{sa}\right) = 159\ kip$

$\phi_{tc} \cdot N_{cbg} = 57.7\ kip$

$\phi_{tc} \cdot N_{ag} = 62.7\ kip$

$$DC_{sN} := \frac{N_{ua}}{\left(\phi_{ts} \cdot N_{sa}\right)} = 0.28$$

Demand-capacity ratio steel in tension, individual anchor in a group

$$DC_{bN} := \frac{N_{uag}}{\left(\phi_{tc} \cdot N_{cbg}\right)} = 0.78$$

Demand-capacity ratio for concrete breakout in tension, a group of anchor

$$DC_{aN} := \frac{N_{uag}}{\left(\phi_{tc} \cdot N_{ag}\right)} = 0.71$$

Demand-capacity ratio for bond in tension, a group of anchor

$DC_N := \max\left(DC_{sN}, DC_{bN}, DC_{aN}\right) = 0.78$ D/C <= 1: OK

Governing demand-capacity ratio for anchor group in tension

Governing failure mode in tension

$$FailureMode\_N := \begin{cases} \text{"Steel Failure"} & \text{if } DC_N = DC_{sN} \\ \text{"Concrete Breakout"} & \text{if } DC_N = DC_{bN} \\ \text{"Bond"} & \text{otherwise} \end{cases}$$

$FailureMode\_N = \text{"Concrete Breakout"}$ Governing failure mode in tension

Additional check for anchors resisting sustained tension (LRFD 5.13.2.2 and ACI 318 17.5.2.2)

$N_{ua\_s} = 3.3\,kip$ Tensile force in one anchor due to factored sustained load

$0.5\phi_{tc} \cdot N_{ba} = 6.0\,kip$ Limit on factored bond strength of anchors under sustained tension load per LRFD 5.13.2.2 (ACI 318 Eq. 17.5.2.2, but with a factor of 0.50 in place of 0.55)

$$DC_{aNs} := \frac{N_{ua\_s}}{0.5\phi_{tc} \cdot N_{ba}} = 0.55$$ D/C <= 1: OK

Check Reinforcement for Crack Control , LRFD 5.6.7

$Class := 2$ Exposure condition

$$\gamma_e := \begin{cases} 1 & \text{if } Class = 1 \\ 0.75 & \text{if } Class = 2 \end{cases}$$ Exposure factor

$\gamma_e = 0.75$

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (B_U - d_c)} = 1.3$$ LRFD Eq. 5.6.7-2

Calculate stress in flexural reinforcement under service loads

$$\rho := \frac{A_s}{d_s \cdot 12in} = 0.0023$$

$$n := \frac{E_s}{E_c} = 8.1$$

$$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n = 0.18$$

$$j := 1 - \frac{k}{3} = 0.94$$

$$f_{ss} := \frac{M_{u2}}{A_s \cdot j \cdot d_s} = 10.4 \cdot ksi$$

Check if fss > 0.6fy

$$\frac{f_{ss}}{0.6f_y} = 0.3$$ < 1: OK

$$s_{a\_max} := \frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{f_{ss}}{ksi}} in - 2 \cdot d_c = 35.2\,in$$ LRFD Eq. 5.6.7-1

Check max spacing of reinforcing bars for crack control:

$$\frac{s_a}{s_{a\_max}} = 0.43$$ < 1: OK

Check maximum spacing of reinforcing bars, LRFD 5.10.3.2

$$s_{max} := \min\left(18in, 1.5 \cdot B_U\right) = 18\,in$$

$$\frac{s_a}{s_{max}} = 0.83$$ < 1: OK

Check minimum spacing of reinforcing bars, LRFD 5.10.3.1

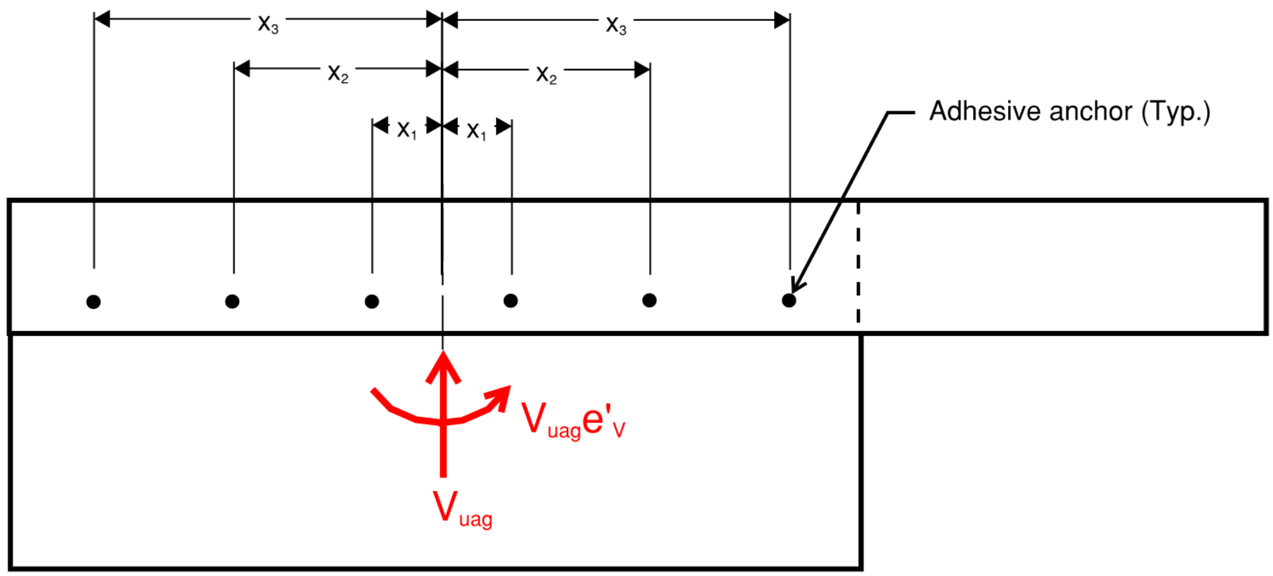
$d_{agg} = 1.5\,in$ Asummed maximum aggregate size

$$s_{min} := \max\left(1.5in, 1.5 \cdot d_a, 1.5d_{agg}\right) = 2.3\,in$$

$$\frac{s_a}{s_{min}} = 6.7$$ > 1: OK

Check Adhesive Anchors in Shear

Shear Force in Anchor Due to Factored Loads



PLAN VIEW

$n_a = 6$

$x_1 := \frac{s_a}{2} = 7.5\,in$

$x_2 := 1.5s_a = 22.5\,in$

$x_3 := 2.5 \cdot s_a = 37.5\,in$

$e'_V := 0.5 \cdot L_U - 0.5L_L = 39\,in$ Eccentricity for anchor group in shear, see wingwall plan

$V_{u1} = 2.6\,kip$ Lateral force per foot length of wall due to factored loads

$V_{uag} := V_{u1} \cdot \frac{L_L}{1ft} = 19.7\,kip$ Total lateral force in anchor group due to factored loads

$V_{ua\_max} := \frac{V_{uag}}{n_a} + \frac{V_{uag} \cdot e'_V \cdot x_3}{2 \cdot \left(x_1^2 + x_2^2 + x_3^2\right)} = 10.6\,kip$ Max shear force in one anchor in the group due to factored loads

Shear Strength of Anchors

<u>Steel shear strength</u>

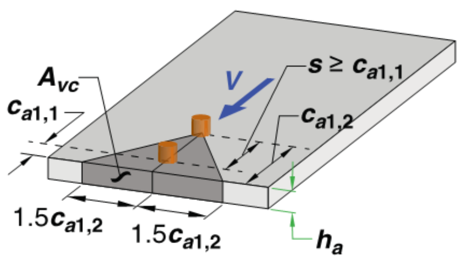
$V_{sa} := 0.6 \cdot A_{se\_V} \cdot f_{uta} = 21.2\ kip$ 40.16.4

$\phi_{vs} \cdot V_{sa} = 13.8\ kip$

Shear strength - Concrete breakout

For an anchor group consisting of two rows of anchors in the direction of shear force, ACI 318 R17.7.2.1 presents three cases as shown in Fig. R17.7.2.1b. When anchors are welded to a common plate, only Case 2 in which 100% of shear force is resisted by the interior row of anchors needs to be considered. In this example, the two rows of anchors are connected by the reinforced concrete wing wall and behave similarly to anchors welded to a common steel plate. Thus, only Case 2 needs to be considered and only shear resistance of anchors in the interior row needs to be checked.

If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1,2})h_a$



Case 2: Another assumption of the distribution of forces indicates that the total shear force would be critical on the rear anchor and its projected area. Only this assumption needs to be considered when anchors are welded to a common plate independent of s. For the calculation of concrete breakout, c_{a1} is taken as $c_{a1,2}$.

Note: For $s \geq c_{a1,1}$, both Case 1 and Case 2 should be evaluated to determine which controls for design except as noted for anchors welded to a common plate

(From ACI 318-19 Fig. R17.7.2.1b)

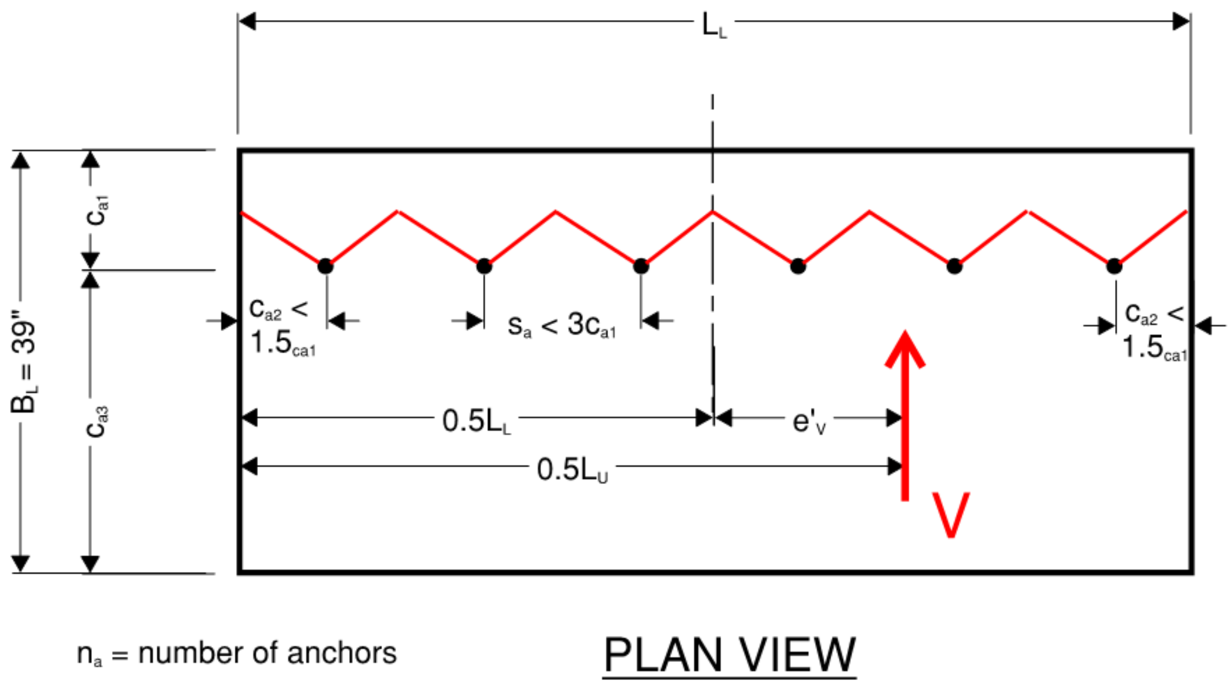
$H := \min(1.5c_{a1}, h_a) = 18.9\ in$

$A_{Vco} := 4.5c_{a1}^{\ 2} = 717.3\ in^2$ ACI 318 17.7.2.1.3

$n_a \cdot A_{Vco} = 4303.5\ in^2$

$A_{Vc1} := L_L \cdot H = 1704.4\ in^2$

$A_{Vc} := \min(A_{Vc1}, n_a \cdot A_{Vco}) = 1704.4\ in^2$ ACI 318 17.7.2.1.1



$e'_V := 0.5 \cdot L_U - 0.5L_L = 39\ in$

Eccentricity for anchor group in shear, see wingwall plan

$$\psi_{ec\_V} := \min\left(\frac{1}{1 + \frac{e'_V}{1.5 \cdot c_{a1}}}, 1\right) = 0.33$$

The eccentricity factor for a group of anchors.

$\psi_{ed\_V} := 1.0$

The modifcation factor for edge effect for a single anchor or group of anchors loaded in shear (ACI 318 17.7.2.4). Perpendicular shear with $c_{a2} >= 1.5c_{a1}$

$\psi_{c\_V} := 1.2$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of at least a No. 4 bar or greater between the anchor and the edge (40.16.4 and ACI 318 17.7.2.5). Reinforcment in the lower wing wall will likely meet this condition.

$$\psi_{h\_V} := \max\left(1, \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}}\right) = 1$$

The modifcation factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$ (40.16.4 and ACI 318 17.7.2.6.1)

$\psi_{p\_V} := 1$

Shear perpendicular to edge (40.16.4)

$l_e := \min(h_{ef}, 8 \cdot d_a) = 6\ in$

40.16.4 and ACI 318 17.7.2.2.1

$$V_{b1} := 7 \cdot \left[\left(\frac{l_e}{d_a} \right)^{0.2} \cdot \sqrt{\frac{d_a}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in} \right)^{1.5} \right] lbf = 24.4 \cdot kip$$

40.16.4 and ACI 318 17.7.2.2.1.a

$$V_{b2} := 9 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left[\left(\frac{c_{a1}}{in} \right)^{1.5} \right] lbf = 23.9 \cdot kip$$

40.16.4 and ACI 318 17.7.2.2.1.b

$$V_b := \min\left(V_{b1}, V_{b2}\right) = 23.9 \cdot kip$$

Basic breakout shear strength

$$V_{cbg} := \frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ec\_V} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot \psi_{h\_V} \cdot V_b = 22.3 \cdot kip$$

Nominal concrete breakout strength in shear perpendicular to the edge, for an anchor group (ACI 318 17.7.2.1b)

$$\phi_{vc} \cdot V_{cbg} = 16.7\, kip$$

<u>Shear strength - Concrete pryout</u>

$$N_{cpg} := \min\left(N_{ag}, N_{cbg}\right) = 76.9\, kip$$

Anchor tensile strength

$$h_{ef} = 15\, in$$

$$k_{cp} := \begin{cases} 1.0 & \text{if } h_{ef} < 2.5in \\ 2.0 & \text{otherwise} \end{cases}$$

ACI 318 17.7.3.1

Note: 40.16.4 requires that h_{ef} >= 2.5 in

$$k_{cp} = 2.0$$

$$V_{cpg} := k_{cp} \cdot N_{cpg}$$

Nominal concrete pryout strength in shear for an anchor group (ACI 318 17.7.3.1b)

$$\phi_{vp} \cdot V_{cpg} = 107.6\, kip$$

<u>Check anchor in shear (ACI 17.5.2)</u>

$$\phi_{vc} \cdot V_{cbg} = 16.7\, kip$$

$$\phi_{vp} \cdot V_{cpg} = 107.6\, kip$$

$$DC_{sV} := \frac{V_{ua\_max}}{\left(\phi_{vs} \cdot V_{sa}\right)} = 0.77$$

Demand-capacity ratio steel in shear, individual anchor in a group

$$DC_{bV} := \frac{V_{uag}}{\left(\phi_{vc} \cdot V_{cbg}\right)} = 1.18$$

Demand-capacity ratio for concrete breakout in shear, a group of anchor

$$DC_{aV} := \frac{V_{uag}}{\left(\phi_{vp} \cdot V_{cpg}\right)} = 0.18$$

Demand-capacity ratio for concrete pryout in shear, a group of anchor

$DC_V := \max(DC_{sV}, DC_{bV}, DC_{aV}) = 1.18$

Governing demand-capacity ratio for anchor group in shear

D/C >1: NG. Without considering shear interface between the upper and lower wing walls, shear resistance of the anchors is not sufficient to resist the lateral load.

Governing failure mode in shear

$$FailureMode\_V := \begin{cases} \text{"Steel Failure"} & \text{if } DC_V = DC_{sV} \\ \text{"Concrete Breakout"} & \text{if } DC_V = DC_{bV} \\ \text{"Concrete pryout"} & \text{otherwise} \end{cases}$$

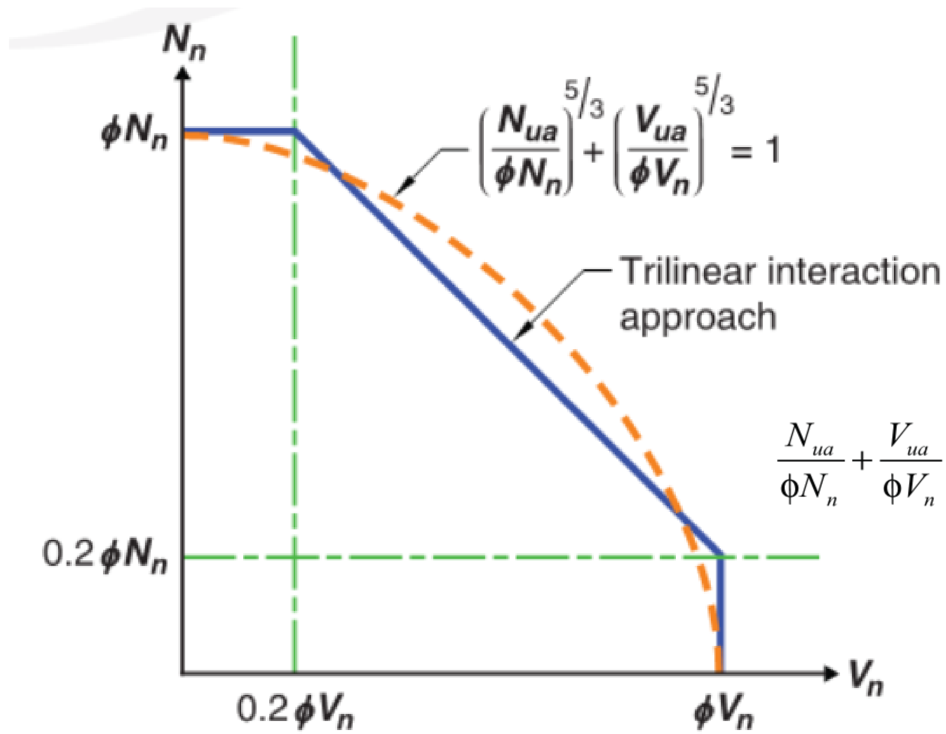
$FailureMode\_V = \text{"Concrete Breakout"}$

Governing shear failure mode

Check anchors for tension and shear interaction (ACI Section 17.8)

The interface shear resistance between the upper and lower wing sections contributes to resiting horizontal shear at the bottom of the upper wing per LRFD 5.7.4.3. If the shear force does not exceed interface shear resistance, the entire shear force can be resisted by the interface shear and shear in the anchors does not have to be checked. See Section Interface Shear Resistance for detail.

In case interface shear resistance is disregarded, the anchors are loaded in both tension and shear. The section below checks the anchors in for tension and shear interaction per ACI 17.8.



$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \qquad (17.8.3)$$

Fig. R17.8—Shear and tensile load interaction equation.

<u>Tension-shear Interaction for Concrete Breakout</u>

$DC_{bN} = 0.78$ $\qquad$ $DC_{bV} = 1.18$

$$NV\_Interaction := \begin{cases} \text{"Yes"} & \text{if } (DC_{bN} > 0.2) \wedge (DC_{bV} > 0.2) \\ \text{"No"} & \text{otherwise} \end{cases}$$

Check if tension and shear interaction needs to be considered (ACI 17.8.2)

$NV\_Interaction = \text{"Yes"}$

$DC_{bN} + DC_{bV} = 1.95$

$$Check := \begin{cases} \text{"OK"} & \text{if } (DC_{bN} + DC_{bV}) \leq 1.2 \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$Check = \text{"N.G."}$

Without considering shear interface between the upper and lower wing walls, resistance of the anchors considering tension-shear interaction is not sufficient to resist the lateral load.

Check interface shear resistance between the upper and lower walls

<u>Interface shear resistance , LRFD 5.7.4.3</u>

Horizontal shear at the bottom of the upper wing is resisted by the interface shear resistance between the upper and lower wing sections.

$b_{vi} := B_U = 15\,in$	Concrete interface width considered to be engaged in shear transfer
$L_{vi} := L_L = 90\,in$	Concrete interface length considered to be engaged in shear transfer
$A_{cv} := b_{vi} \cdot L_{vi} = 1350\,in^2$	Concrete interface area considered to be engaged in shear transfer
$A_{vf} := n_a \cdot A_{se\_V} = 2.65\,in^2$	Cross-sectional area of anchor group resisting shear

Cohesion and friction factors, LRFD 5.7.4.4:

For calculations in this example, it was assumed concrete placed against a clean concrete surface, free of laitance, with surface not intentionally roughened. In practice, it is advisable that the concrete surface be intentionally roughened to an amplitude of 1/4 inch to improve the interface shear resistance.

$c := 0.075ksi$	Cohesion factor
$\mu := 0.6$	Friction factor
$K_1 := 0.2$	Fraction of concrete strength available to resist interface shear
$K_2 := 0.8ksi$	Limiting interface shear resistance (ksi)

Interface shear resistances

LRFD Equation 5.7.4.3-3 for shear resistance of the interface plane is based on the assumption that the interface reinforcement is stressed to its design yield stress, f_y. Adhesive anchors may not be stressed to its design yield stress at the nominal resistance; thus, the tensile force used to determine interface shear resistance is the lesser of:

- Yield strength of the anchor and
- Governing anchor tensile strength

$T_{ri} := \min(\phi_v \cdot A_{vf} \cdot f_y, \phi_{tc} \cdot N_{cbg}, \phi_{tc} \cdot N_{ag}) = 57.7\,kip$	Factored tensile force in adhesive anchor used to determine interface shear resistance
$P_c := 0kip$	Permanent net compressive force normal to the shear plane. Conservatively disregard compression force by self-weight of the upper wing at the interface.
$V_{ri1} := \phi_v \cdot c \cdot A_{cv} + \mu \cdot (T_{ri} + \phi_v \cdot P_c) = 125.7\,kip$	LRFD Equation 5.7.4.3-3, modifed for adhesive anchors
$V_{ri2} := \phi_v \cdot K_1 \cdot f'_c \cdot A_{cv} = 850.5\,kip$	LRFD Equation 5.7.4.3-4
$V_{ri3} := \phi_v \cdot K_2 \cdot A_{cv} = 972\,kip$	LRFD Equation 5.7.4.3-5

$$V_{ri} := \min(V_{ri1}, V_{ri2}, V_{ri3}) = 125.7\,kip$$

$$V_{ui} := V_{u1} \cdot \frac{L_L}{1\,ft} = 19.67 \cdot kip$$

Total lateral force in anchor group due to factored loads

$$\frac{V_{ui}}{V_{ri}} = 0.16$$

D/C < 1: OK

The interface shear resistance is sufficient to resist the design lateral load.

Introduction

These calculations expand the calculations in Design Example 1 - Wing Wall Replacement for different, typical wall geometries. See Design Example 1 for more details.

Input

Soil Porperties

Angle of internal friction	ϕ =	30	deg
Soil unit weight	γ_s =	120	pcf
Soil cohesion	c =	0	pcf
Friction angle between backfill and wall	δ =	20	deg
Equivalent fluid unit weight of soil, active pressure	γ_{eq} =	35	pcf

Reinforced Concrete Parameters

Concrete unit weight	w_c =	0.15	kcf
Concrete compressive strength	f'_c =	3.5	ksi
Concrete compressive strength	f'_c =	3500	psi
Modulus of elasticity of steel	Es =	29000	ksi
Modulus of elasticity of concrete	Ec =	3587	ksi
Assumed max aggregate size	d_{agg} =	1.5	in
Clear concrete cover	c_c =	2.0	in

Anchor Material Properties

Steel anchor tensile strength	f_{ua} =	80	ksi
Steel anchor yield strength	f_{ya} =	60	ksi

Resistance Factors, LRFD 5.5.4.2

Strength reduction factor for shear	ϕ_v =	0.9	

Geometries

Angle of fill to the horizontal	β =	0	deg
Angle of back face of wall to the horizontal	θ =	90	deg

Wall and Anchor Designs

Wall Design No.	Cantilever Length (ft)	Wall Parameters						
		H_U (ft)	H_L (ft)	L_U (ft)	L_L (ft)	B_U (in)	B_L (in)	$\eta = L_U/L_L$
1	6.5	4.0	5.0	14.0	7.5	15	39	1.87
2	0	5.5	5.0	12.0	12.0	15	39	1.00
3	6.5	5.5	5.0	14.0	7.5	15	39	1.87
4	0	7.0	5.0	12.0	12.0	15	39	1.00
5	6.5	7.0	5.0	14.0	7.5	24	39	1.87
6	0	8.5	5.0	12.0	12.0	18	39	1.00
7	6.5	8.5	5.0	18.0	11.5	30	39	1.57
8	0	9.5	5.0	12.0	12.0	24	39	1.00
9	6.5	9.5	5.0	24.0	17.5	30	39	1.37

Wall Design No.	Anchor Parameters						
	d_a (in)	n_a	s_a (in)	l_ebd (in)	d_c (in)	τ_{uncr} (psi)	τ_{cr} (psi)
1	0.75	6	15.0	15	2.4	1350	450
2	0.75	10	14.4	15	2.4	1350	450
3	0.75	6	15.0	15	2.4	1350	450
4	0.75	10	14.4	15	2.4	1350	450
5	0.75	6	15.0	15	2.4	1350	450
6	0.75	10	14.4	15	2.4	1350	450
7	0.75	9	15.3	15	2.4	1350	450
8	0.75	10	14.4	15	2.4	1350	450
9	0.75	14	15.0	15	2.4	1650	550

Load Factors

	STRENGTH	SERVICE I
EH	1.50	1.00
LS	1.75	1.00

Calculations of Horizontal Earth Loads and Live Load Surcharge

(Horizontal earth load is based on equivalent fluid method)

	Γ =	2.687
Coefficient of active earth pressure	k_a =	0.297
	α =	20.1 deg
Equivalent height of soil for surcharge live load	h_{eq} =	2.0 ft

							STRENGTH I			SERVICE I	
Wall Design No.	p_a (psf)	F_{EH} (kip)	F_{EHX} (kip)	p_{eq} (psf)	F_{LS} (kip)	F_{LSX} (kip)	V_{u1} (kip)	M_{u1} (ft*kip)	M_{u1_s} (ft*kip)	V_{u2} (kip)	M_{u2} (ft*kip)
1	143	0.53	0.50	71.3	0.53	0.50	1.63	2.75	1.0	1.00	1.67
2	196	0.54	0.51	71.3	0.39	0.37	1.40	3.17	1.4	0.88	1.94
3	196	1.01	0.95	71.3	0.73	0.69	2.62	5.91	2.6	1.63	3.63
4	250	0.87	0.82	71.3	0.50	0.47	2.05	5.75	2.9	1.29	3.56
5	250	1.63	1.53	71.3	0.93	0.88	3.83	10.72	5.4	2.41	6.64
6	303	1.29	1.21	71.3	0.61	0.57	2.81	9.38	5.1	1.78	5.85
7	303	2.02	1.89	71.3	0.95	0.89	4.40	14.68	8.1	2.79	9.15
8	339	1.61	1.51	71.3	0.68	0.64	3.38	12.47	7.2	2.15	7.81
9	339	2.21	2.07	71.3	0.93	0.87	4.64	17.10	9.8	2.95	10.71

Anchor Strength Calculations

Geometric Calculations and Checks

Find c_{a_max} = the largest of the influencing edge distances not exceeding $1.5h_{ef}$ (ACI 318 17.6.2.1.2)

Wall Design No.	c_{a1} (in)	c_{a2} (in)	c_{a3} (in)	c_c (in)	Embed. depth limit check	h_{ef} (in)	A_{se} (in^2)	c_{a_min} (in)
1	12.6	7.5	26.4	2.0	OK	15	0.44	7.5
2	12.6	7.2	26.4	2.0	OK	15	0.44	7.2
3	12.6	7.5	26.4	2.0	OK	15	0.44	7.5
4	12.6	7.2	26.4	2.0	OK	15	0.44	7.2
5	21.6	7.5	17.4	2.0	OK	15	0.44	7.5
6	15.6	7.2	23.4	2.0	OK	15	0.44	7.2
7	27.6	7.7	11.4	2.0	OK	15	0.44	7.7
8	21.6	7.2	17.4	2.0	OK	15	0.44	7.2
9	27.6	7.5	11.4	2.0	OK	15	0.44	7.5

Check 1	Check 2	Check 3	c_{a_max} (in)
1	1	0	12.6
1	1	0	12.6
1	1	0	12.6
1	1	0	12.6
1	1	1	21.6
1	1	0	15.6
0	1	1	11.4
1	1	1	21.6
0	1	1	11.4

Check edge distances, spacing, and thickness (ACI 318 17.9)

Wall Design No.	s_{a_min} (in)	c_{edge_min} (in)	$s_a \geq s_{a\_min}$	$c_{a\_min} \geq c_{edge\_min}$
1	4.5	4.5	OK	OK
2	4.5	4.5	OK	OK
3	4.5	4.5	OK	OK
4	4.5	4.5	OK	OK
5	4.5	4.5	OK	OK
6	4.5	4.5	OK	OK
7	4.5	4.5	OK	OK
8	4.5	4.5	OK	OK
9	4.5	4.5	OK	OK

Tensile Force in Anchor

Modular Ratio n = 8.1

Wall Design No.	A_s (in^2)	d_s (in)	ρ	k	j	f_{s_ua} (ksi)	T_{ua} (kip)	N_{uag} (kip)	N_{ua} (kip)	N_{ua_s} (kip)
1	0.35	12.6	0.0023	0.18	0.94	7.9	2.78	20.8	3.5	1.3
2	0.37	12.6	0.0024	0.18	0.94	8.7	3.20	38.4	3.8	1.7
3	0.35	12.6	0.0023	0.18	0.94	16.9	5.97	44.8	7.5	3.3
4	0.37	12.6	0.0024	0.18	0.94	15.8	5.81	69.7	7.0	3.5
5	0.35	21.6	0.0014	0.14	0.95	17.6	6.24	46.8	7.8	3.9
6	0.37	15.6	0.0020	0.16	0.95	20.7	7.62	91.4	9.1	5.0
7	0.35	27.6	0.0010	0.12	0.96	19.2	6.65	76.4	8.5	4.7
8	0.37	21.6	0.0014	0.14	0.95	19.7	7.26	87.1	8.7	5.0
9	0.35	27.6	0.0011	0.12	0.96	21.9	7.75	135.6	9.7	5.6

Anchor Strength Calculations

Tensile Strength

Tensile Strength - Steel

Strength reduction factor for ductile tensile steel (ACI 318 17.5.3a)	ϕ_{ts} =	0.75	
Steel anchor tensile strength	f_{uta} =	80	ksi

Wall Design No.	N_{sa} (kip)	ϕ_{ts}*N_{sa} (kip)	N_{ya} = A_{se}*f_{ya} (kip)
1	35.3	26.5	26.5
2	35.3	26.5	26.5
3	35.3	26.5	26.5
4	35.3	26.5	26.5
5	35.3	26.5	26.5
6	35.3	26.5	26.5
7	35.3	26.5	26.5
8	35.3	26.5	26.5
9	35.3	26.5	26.5

Tensile Strength - Concrete Breakout

Breakout eccentricity factor	ψ_{ec_N} =	1
Breakout cracking factor	ψ_{c_N} =	1.0
Breakout splitting factor (ACI 17.6.2.6)	ψ_{cp_N} =	1
Coefficient for basic concrete breakout strength in tension (ACI 318 17.6.2.2.	k_c =	17
Strength reduction factor for concrete breakout (ACI 17.5.3b)	ϕ_{tc} =	0.75

Wall Design No.	h_{ef} (in)	$h`_{ef}$ (in)	A_{Nco} (in^2)	n_a*A_{Nco} (in^2)	A_{Nc1} (in^2)	A_{Nc} (in^2)	ψ_{ed_N}	N_b (kip)	N_{cbg} (kip)	ϕ_{tc}*N_{cbg} (kip)
1	15	8.4	638	3825	2273	2273	0.88	24.6	76.9	57.7
2	15	8.4	638	6376	3636	3636	0.87	24.6	122.0	91.5
3	15	8.4	638	3825	2273	2273	0.88	24.6	76.9	57.7
4	15	8.4	638	6376	3636	3636	0.87	24.6	122.0	91.5
5	15	14.4	1871	11223	3510	3510	0.80	55.1	83.1	62.3
6	15	10.4	977	9766	4500	4500	0.84	33.8	130.6	98.0
7	15	7.6	518	4658	3140	3140	0.90	21.0	114.9	86.2
8	15	14.4	1871	18706	5616	5616	0.80	55.1	132.2	99.2
9	15	7.6	518	7246	4778	4778	0.90	21.0	174.1	130.5

Tensile Strength - Anchor Bond

Bond eccentricity factor	ψ_{ec_Na} =	1

Wall Design No.	τ_{uncr} (psi)	τ_{cr} (psi)	c_{Na} (in)	A_{Nao} (in^2)	n_a*A_{Nao} (in^2)	A_{Na1} (in^2)	A_{Na} (in^2)	ψ_{cp_Na}	ψ_{ed_Na}	N_{ba} (kip)	N_{ag} (kip)	ϕ_{tc}*N_{ag} (kip)
1	1350	450	8.31	276.1	1657	1496	1496	1.00	0.97	15.90	83.62	62.7
2	1350	450	8.31	276.1	2761	2393	2393	1.00	0.96	15.90	132.30	99.2
3	1350	450	8.31	276.1	1657	1496	1496	1.00	0.97	15.90	83.62	62.7
4	1350	450	8.31	276.1	2761	2393	2393	1.00	0.96	15.90	132.30	99.2
5	1350	450	8.31	276.1	1657	1496	1496	1.00	0.97	15.90	83.62	62.7
6	1350	450	8.31	276.1	2761	2393	2393	1.00	0.96	15.90	132.30	99.2
7	1350	450	8.31	276.1	2485	2293	2293	1.00	0.98	15.90	129.02	96.8
8	1350	450	8.31	276.1	2761	2393	2393	1.00	0.96	15.90	132.30	99.2
9	1650	550	9.19	337.5	4725	3858	3858	1.00	0.94	19.44	209.97	157.5

Tensile Strength Checks

	Required Tension		Factored Resistances			Demand-Capacity Ratios					
Wall Design No.	Individual anchor, N_{ua} (kip)	Anchor group, N_{uag} (kip)	Steel, ϕ_{ts}*N_{sa} (kip)	Breakout, ϕ_{tc}*N_{cbg} (kip)	Bond, ϕ_{tc}*N_{ag} (kip)	Steel, DC_{sN}	Concrete B.O., DC_{bN}	Bond, DC_{aN}	Max DC_N	Governing Failure Mode	Check
1	3.5	20.8	26.5	57.7	62.7	0.13	0.36	0.33	**0.36**	**Concrete B.O**	**OK**
2	3.8	38.4	26.5	91.5	99.2	0.14	0.42	0.39	**0.42**	**Concrete B.O**	**OK**
3	7.5	44.8	26.5	57.7	62.7	0.28	0.78	0.71	**0.78**	**Concrete B.O**	**OK**
4	7.0	69.7	26.5	91.5	99.2	0.26	0.76	0.70	**0.76**	**Concrete B.O**	**OK**
5	7.8	46.8	26.5	62.3	62.7	0.29	0.75	0.75	**0.75**	**Concrete B.O**	**OK**
6	9.1	91.4	26.5	98.0	99.2	0.34	0.93	0.92	**0.93**	**Concrete B.O**	**OK**
7	8.5	76.4	26.5	86.2	96.8	0.32	0.89	0.79	**0.89**	**Concrete B.O**	**OK**
8	8.7	87.1	26.5	99.2	99.2	0.33	0.88	0.88	**0.88**	**Concrete B.O**	**OK**
9	9.7	135.6	26.5	130.5	157.5	0.37	1.04	0.86	**1.04**	**Concrete B.O**	NOT OK

Additional check for bond strength under sustained tension

Wall Design No.	Sustained tension, N_{ua_s} (kip)	$0.5\phi_{tc}$*N_{ba} (kip)	DC_{aNs}	Check
1	1.3	6.0	0.21	**OK**
2	1.7	6.0	0.28	**OK**
3	3.3	6.0	0.55	**OK**
4	3.5	6.0	0.58	**OK**
5	3.9	6.0	0.65	**OK**
6	5.0	6.0	0.84	**OK**
7	4.7	6.0	0.78	**OK**
8	5.0	6.0	0.84	**OK**
9	5.6	7.3	0.76	**OK**

Check Reinforcement for Crack Control and Bar Spacing, LRFD 5.6.7, 5.10.3.2, and 5.10.3.1

Exposure condition class = 2

Wall Design No.	γ_e	β_s	ρ	k	j	f_{ss} (ksi)	f_{ss} > $0.6f_y$ Check	s_a (in)	s_{a_max} (in)	Spacing for crack control	s_{max} (in)	Max spacing check	s_{min} (in)	Min spacing check
1	0.75	1.27	0.0023	0.18	0.94	4.8	OK	15.0	82.1	OK	18	OK	2.3	OK
2	0.75	1.27	0.0024	0.18	0.94	5.3	OK	14.4	72.8	OK	18	OK	2.3	OK
3	0.75	1.27	0.0023	0.18	0.94	10.4	OK	15.0	35.2	OK	18	OK	2.3	OK
4	0.75	1.27	0.0024	0.18	0.94	9.8	OK	14.4	37.6	OK	18	OK	2.3	OK
5	0.75	1.16	0.0014	0.14	0.95	10.9	OK	15.0	36.8	OK	18	OK	2.3	OK
6	0.75	1.22	0.0020	0.16	0.95	12.9	OK	14.4	28.7	OK	18	OK	2.3	OK
7	0.75	1.12	0.0010	0.12	0.96	12.0	OK	15.3	34.3	OK	18	OK	2.3	OK
8	0.75	1.16	0.0014	0.14	0.95	12.4	OK	14.4	32.0	OK	18	OK	2.3	OK
9	0.75	1.12	0.0011	0.12	0.96	13.7	OK	15.0	29.3	OK	18	OK	2.3	OK

Interface Shear Resistance, LRFD 5.7.4.3

Cohesion factor c = 0.075 ksi
Friction factor μ = 0.6
Fraction of concrete strength available to resist interface shear K_1 = 0.2
Limiting interface shear resistance (ksi) K_2 = 0.8 ksi
Permanent net compressive force normal to the shear plane P_c = 0 kip

Shear Strengths for Proposed Anchor Designs

Anchor Design No.	b_{vi} (in)	L_{vi} (in)	A_{cv} (in^2)	A_{vf} (in^2)	T_{ri} (kip)	V_{ri1} (kip)	V_{ri2} (kip)	V_{ri3} (kip)	Governing V_{ri} (kip)
1	15	90	1350	2.65	57.7	125.7	850.5	972.0	125.7
2	15	144	2160	4.42	91.5	200.7	1360.8	1555.2	200.7
3	15	90	1350	2.65	57.7	125.7	850.5	972.0	125.7
4	15	144	2160	4.42	91.5	200.7	1360.8	1555.2	200.7
5	24	90	2160	2.65	62.3	183.2	1360.8	1555.2	183.2
6	18	144	2592	4.42	98.0	233.7	1633.0	1866.2	233.7
7	30	138	4140	3.98	86.2	331.2	2608.2	2980.8	331.2
8	24	144	3456	4.42	99.2	292.8	2177.3	2488.3	292.8
9	30	210	6300	6.19	130.5	503.6	3969.0	4536.0	503.6

Shear Checks

Anchor Design No.	V_{ui} (kip)	V_{ri} (kip)	V_{u1}/V_{ri}	Check
1	12.19	125.7	0.10	OK
2	16.86	200.7	0.08	OK
3	19.67	125.7	0.16	OK
4	24.62	200.7	0.12	OK
5	28.73	183.2	0.16	OK
6	33.74	233.7	0.14	OK
7	50.61	331.2	0.15	OK
8	40.58	292.8	0.14	OK
9	81.15	503.6	0.16	OK

Summary

- Reducing anchor spacing, i.e. increasing the number of anchors, does not increase flexural resistance of the upper wing wall in a meaningful way since the tensile strength of the anchor group is limited by concrete breakout, which is limited by the geometries of the wall.
- The 15” standard wall is adequate for H_U ≤5’-6” with 6'-6" cantilever and H_U ≤7’-0” without cantilever. For other wall geometries, the thickness of the wall needs to be increased to meet flexural demand. The table below summarizes upper wing wall thicknesses that meet the flexural demands for different, typical wall geometries. The intent of this table is to provide an example illustrating an approach to simplify the design of adhesive anchors for wing wall replacement, and not to cover all geometries or show the most optimal designs.

Table for Upper Wing Wall Thickness B_U

L_U	10'-0" to 12'-0"	14'-0" to 16'-0"	18'-0" to 24'-0"
L_L	= L_U	= L_U - (6'-6")	= L_U - (6'-6")
Wall Cantilever	0'-0"	6'-6"	6'-6"
H_U ≤ 5'-6"	15"	15"	15"
5'-6" ≤ H_U ≤ 7'-0"	15"	24"	24"
7'-0" ≤ H_U ≤ 8'-6"	24"	-	30"
8'-6" ≤ H_U ≤ 9'-6"	24"	-	-

Note: All using #6 adhesive anchors @ 15” spacing, 15” embed., τ_{cr} = 450 psi (τ_{uncr} = 1,350 psi)

Notations

Total height of wing	H
Height of upper wing	H_U
Width of upper wing	B_U
Height of lower wing wall	H_L
Length of upper wing wall	L_U
Length of lower wing wall	L_L
Thickness of lower wing wall	B_L
Amplification factor for $L_U > L_L$	$\eta = L_U/L_L$
Diameter of anchor	d_a
Number of adhesive anchors	n_a
Adhesive anchor spacing	s_a
Actual embedment depth	l_{ebd}
Effective embedment depth	h_{ef}
Corrected effective embedment depth for concrete breakout strength in tension	$h`_{ef}$
Cover from center of anchors to back face of upper wing	d_c
Characteristic bond stress of adhesive anchor in uncracked concrete	τ_{uncr}
Characteristic bond stress of adhesive anchor in cracked concrete	τ_{cr}
Horizontal active earth pressure based on equivalent fluid unit weight of soil	p_a
Resultant force of horizontal active earth pressure	F_{EH}
Horizontal component of resultant force of lateral earth pressure	F_{EHX}
Equivalent earth pressure	p_{eq}
Resultant force of live load surcharge	F_{LS}
Horizontal component of resultant force of live load surcharge	F_{LSX}
Shear force per foot length of wall (Strength I)	V_{u1}
Total bending moment per foot length of wall (Strength I)	M_{u1}
Bending moment due to earth pressure (sustained loads) per foot length of wall (Strength I)	M_{u1_s}
Shear force per foot length of wall (Service I)	V_{u2}
Total bending moment per foot length of wall (Service I)	M_{u2}
Edge distance parallel to shear force	c_{a1}
Edge distance perpendicular to shear force on the front side	c_{a2}
Edge distance perpendicular to shear force on the back side	c_{a3}
Concrete cover per ACI 318 20.5.1.3.1	c_c
Effective embedded length (ACI 318 17.3.3)	h_{ef}
Anchor cross-sectional area	A_{se}
Minimum edge distance	c_{a_min}
Minimum spacing required (ACI 318 17.9.2)	s_{a_min}
Minimum edge distance required (ACI318 17.9.2)	c_{edge_min}
Area of reinforcement per foot length of wall	A_s
Effective depth of section in bending	d_s
Ratio of A_s to bd	ρ
Tensile force in anchor per foot length of wall due to factored loads	T_{ua}
Total tensile force in anchor group due to factored loads	N_{uag}
Tensile force in one anchor due to factored loads	N_{ua}

Tensile force in one anchor due to factored sustained load	N_{ua_s}
Nominal strength of a single steel anchor	N_{sa}
Factored ultimate strength of steel anchor	$\phi_{ts}*N_{sa}$
Yield strength of steel anchor	$N_{ya} = A_{se\_N}*f_{ya}$
Projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing	A_{Nco}
Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension	A_{Nc}
Breakout edge effect factor	Ψ_{ed_N}
Basic breakout strength of a single bolt in tension in cracked concrete (ACI 318-14 17.4.2.2a)	N_b
Nominal concrete breakout strength in tension of anchor group	N_{cbg}
Factored concrete breakout strength in tension	$\phi_{tc}*N_{cbg}$
Projected distance from center of an anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor	c_{Na}
Projected influence area of a single adhesive anchor, for calculation of bond strength in tension if not limited by edge distance or spacing	A_{Nao}
Projected influence area of a single adhesive anchor or group of anchors, for calculation of bond strength in tension	A_{Na}
Bond splitting factor	Ψ_{cp_Na}
Breakout edge effect factor used to modify tensile strength of adhesive anchors based on proximity to edges of concrete member	Ψ_{ed_Na}
Basic bond strength in tension of a single adhesive anchor	N_{ba}
Nominal bond strength in tension of adhesive anchor group	N_{ag}
Factored anchor group bond strength in tension	$\phi_{tc}*N_{ag}$
Demand-capacity ratio steel in tension, individual anchor in a group	DC_{sN}
Demand-capacity ratio for concrete breakout in tension, a group of anchors	DC_{bN}
Demand-capacity ratio for bond in tension, a group of anchors	DC_{aN}
Governing demand-capacity ratio for anchor group in tension	DC_N
Limit on factored bond strength of anchors under sustained tension load per LRFD 5.13.2.2 (ACI 318 Eq. 17.5.2.2, but with a factor of 0.50 in place of 0.55)	$0.5\phi_{tc}*N_{ba}$
Demand-capacity ratio for a single steel anchor under sustained tension	DC_{aNs}
Exposure factor	γ_e
Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face (LRFD 5.6.7)	β_s
Stress in flexural reinforcement under service loads	f_{ss}
Maximum spacing allowed for crack control (LRFD 5.6.7)	s_{a_max}
Maximum spacing of reinforcing bars (LRFD 5.10.3.2)	s_{max}
Minimum spacing of reinforcing bars (LRFD 5.10.3.1)	s_{min}
Interface width considered to be engaged in shear transfer	b_{vi}
Concrete interface length considered to be engaged in shear transfer	Lvi
Reinforcement area crossing shear interface	A_{cv}
Interface area considered to be engaged in shear transfer	A_{vf}
Factored tensile force in adhesive anchor (averaged per foot length of wall) for shear interface	T_{ri}
Nominal shear resistance of the interface plane (LRFD Equation 5.7.4.3-3, modifed for adhesive anchors)	Vri1
Nominal shear resistance limit 1 (LRFD Equation 5.7.4.3-4)	Vri2

Nominal shear resistance limit 2 (LRFD Equation 5.7.4.3-5)	Vri3
Factored interface shear resistance (averaged per foot length of wall)	V_{ri}
Total lateral force in anchor group due to factored loads	V_{ui}

DESIGN EXAMPLE 2 - ABUTMENT EXTENSION USING ADHESIVE ANCHORS

General Information

In this example, flexrual resistance of a replaced concrete parapet connected to the existing bridge deck with adhesive anchors is calculated in accordance with AASHTO LRFD 9th Edition. Strength of adhesive anchors is calculated in accordance with ACI 318-19.

References

- AASHTO LRFD 9th Edition
- ACI 318-19 Building Code Requirements for Structural Concrete and Commentary
- WisDOT Bridge Manual 2020

Note: All sectional references in the calculations refer to the WisDOT Bridge Manual 2020 unless otherwise noted.

General Assumptions

- Concrete of the existing abutment is cracked for the purpose of calculating anchor resistances.
- Reinforcement in the existing abutment can function as supplementary reinforcement for or the purpose of calculating anchor resistances.

Design Parameters

Reinforced Concrete Parameters

$w_c := 0.15$ Unit weight of concrete (kcf)

$f'_c := 3.5\text{ksi}$ Concrete compressive strength

$\lambda_a := 1$ Assumed normal weight concrete

$f_y := 60\text{ksi}$ Yield strength of reinforcing bars (anchors) ASTM A615 Grade 60

$f_u := 80\text{ksi}$ Tensile strength of reinforcing bars (anchors) ASTM A615 Grade 60

$E_s := 29000\text{ksi}$ Modulus of elasticity of steel reinforcment

$E_c := 33000 \cdot \left(w_c\right)^{1.5} \cdot \sqrt{\frac{f'_c}{\text{ksi}}}\,\text{ksi} = 3587 \cdot \text{ksi}$ Modulus of elasticity of concrete, ACI 318 19.2.2.1a

Geometry

$h := 2.5\text{ft}$ Width of abutment

$b := 5\text{ft}$ Height of abutment

Characteristic Bond Stress

$\tau_{cr} := 500\text{psi}$ Characteristic bond stress for cracked concrete, minimum value to avoid bond as the governing failure mode

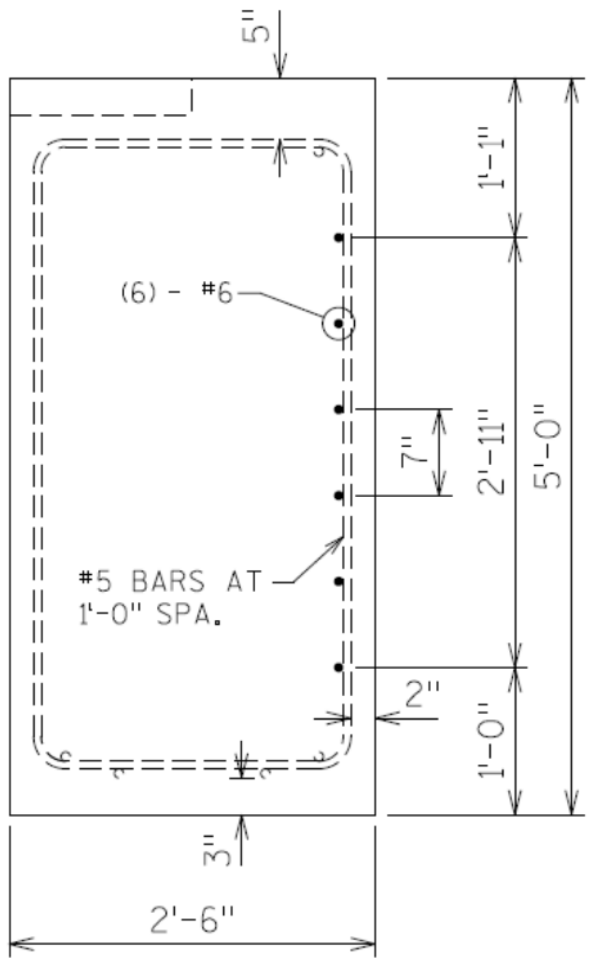
$\tau_{uncr} := 3 \cdot \tau_{cr} = 1500\,\text{psi}$ Characteristic bond stress for uncracked concrete, assumed to be three times the characteristic bond stress for cracked concrete.

For cracked concrete, τ_{cr} is used to calculate the basic bond strength N_{ba}, but τ_{uncr} is used to calculate c_{Na}. For a given τ_{cr}, higher τ_{uncr} increases c_{Na} and decreases the bond strength N_a. Thus, to be conservative a maximum τ_{uncr}/τ_{cr} is chosen. For products in the WisDOT approved product list, τ_{uncr}/τ_{cr} ranges from 1.1 to 3.0.

See equations in ACI 318-14, Section 17.4.5.1 for A_{Nao} and c_{Na} for further explanation of how τ_{uncr} influences calculated bond strength

Case 1: Original Construction (6)-#6 rebars

Calculate flexural resistance of the abutment section with original cast-in-place reinforcement per AASHTO LRFD. Assume 6-#6 on the back face are fully developed.



$c_c := 2in$ Concrete clear cover to stirrups

$d_{agg} := 1.5in$ Assumed max aggregate size

$d_a := 0.75 \cdot in$ Diameter of #6 reinforcing bars (anchors)

$d_{str} := 0.625 \cdot in$ Diameter of #5 stirrup

$A_{se} := 0.44 \cdot in^2$ Cross-sectional area of reinforcing bar (anchor)

$n_a := 6$ Number of reinforcing bars (anchors)

$A_s := n_a \cdot A_{se} = 2.64 \cdot in^2$ Total area of steel reinforcement

$d_{c1} := c_c + d_{str} + \frac{d_a}{2} = 3\ in$ Distance to center of longitudinal bars from near concrete edge

$d := h - d_{c1} = 27 \cdot in$ Depth of steel reinforcement

$T := A_s \cdot f_y = 158.4 \cdot kip$ Tensile force from steel reinforcement

$C = 0.85 * f'c * a * b$ Compressive force from concrete

$T = C$ Equilibrium

$b = 60\ in$

$a := \frac{T}{0.85 \cdot f'_c \cdot b} = 0.89 \cdot in$ Thickness of Whitney Block

$M_{n1} := \left(d - \frac{a}{2}\right) \cdot T = 350.54 \cdot ft \cdot kip$ Nominal flexural resistance of original section

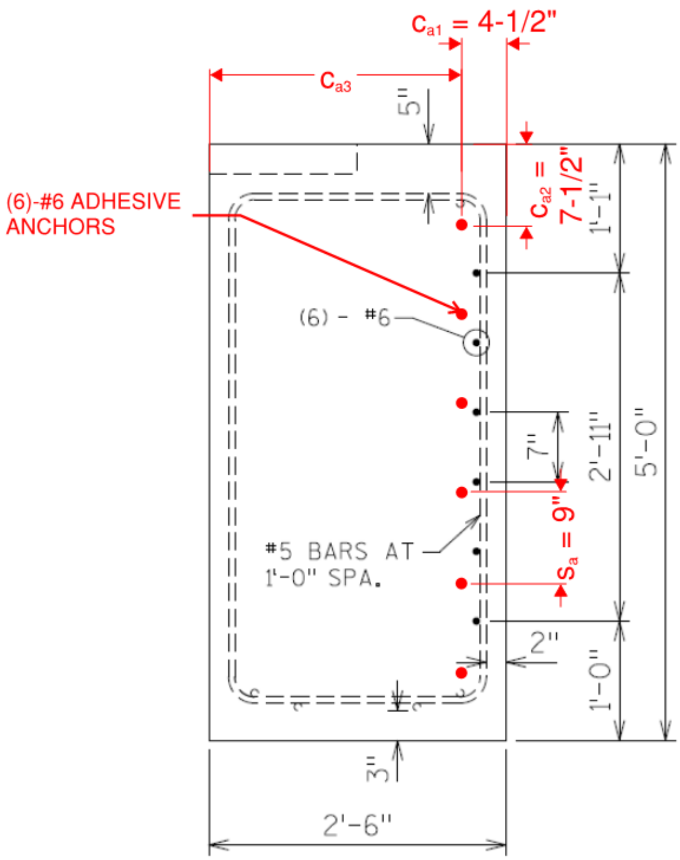
$\phi_1 := 0.90$ AASHTO 5.5.4.2

$\phi_1 \cdot M_{n1} = 315.49 \cdot ft \cdot kip$ Factored flexural resistance of original section

Case 2: (6)-#6 Adhesive Anchors

Calculate flexural resistance of the abutment section with 6-#6 rebar adhesive anchors. Tensile strength of the anchors is calculated in accordance with ACI 318-19. Flexural resistance is calculated assuming linear stress distribution for concrete in compression. Ignore tensile stress in concrete. Assume the original reinforcement is cut off and does not contribute to the flexural resistance of the extended abutment section.

CASE 2



$d_a = 0.75\ in$

$l_{ebd} := 15in$ — Actual embedment depth

$d_{c2} := 4.5in$

$d := h - d_{c2} = 25.5 \cdot in$ — depth of steel reinforcment

$h_{ef} := \min\left(l_{ebd}, 20 \cdot d_a\right) = 15 \cdot in$ — Effective embedded length, 40.16.1.1 and ACI 318 17.3.3

$s_a := 9in$ — Anchor spacing longitudinally

$c_{a1} := d_{c2} = 4.5 \cdot in$ — Edge distance from center of anchor to vertical edge of concrete, tension side

$c_{a2} := 7.5in$ — Edge distance from center of anchor to horizontal edge of concrete, tension side

$c_{a3} := d = 25.5 \cdot in$ — Edge distance from center of anchor to vertical edge of concrete, compression side

$c_{a\_min} := \min(c_{a1}, c_{a2}) = 4.5 \cdot in$ — Min edge distance

Check edge distances, spacings

$s_{a\_min} := 6 \cdot d_a = 4.5\,in$ — Minimum spacing required, 40.16.1.1 and ACI 318 17.9.2

$c_{edge\_min} := \max(c_c, 2 \cdot d_{agg}, 6 \cdot d_a) = 4.5\,in$ — Minimum edge distance required, ACI 318 17.9.2

$s_a \geq s_{a\_min} = 1$ — OK

$c_{a\_min} \geq c_{edge\_min} = 1$ — OK

Resistance Factors, LRFD 5.5.4.2

$\phi_{ts} := 0.75$ — Strength reduction factor for steel in tension for ductile steel, 40.16.2 and ACI 318 17.5.3. Rebars are considered ductile.

$\phi_{tc} := 0.75$ — Strength reduction factor for concrete breakout and bond in tension for Anchor Category 1 with supplementary reinforcement, 40.16.3 & ACI 318 17.5.3. It is assumed that the existing reinforcement in the abutment functions as supplementary reinforcement for adhesive anchors as described in ACI 318 R17.5.3.

Tensile Strength of Anchors

<u>Steel Strength of (6)-#6 anchors</u>

$f_{uta} := \min(f_u, 1.9 \cdot f_y, 125ksi) = 80 \cdot ksi$ — ACI 17.6.1.2

$N_{sa} := A_{se} \cdot f_{uta} = 35.2 \cdot kip$ — ACI Eq. 17.6.1.2

$N_{sa2} := N_{sa} = 35.2\,kip$

$\phi_{ts} \cdot n_a \cdot N_{sa} = 158.4 \cdot kip$ — Factored tensile strength of steel anchors

Concrete Breakout Strength of (6)-#6 anchor group

$c_{a1} = 4.5\,in$ $\quad c_{a2} = 7.5\,in$ $\quad c_{a3} = 25.5\,in$

$$check_1 := \begin{cases} 1 & \text{if } c_{a1} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_2 := \begin{cases} 1 & \text{if } c_{a2} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_3 := \begin{cases} 1 & \text{if } c_{a3} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}$$

$c_{a\_max} := \max(c_{a1} \cdot check_1, c_{a2} \cdot check_2, c_{a3} \cdot check_3) = 7.5\,in$ Max edge distance that is less than 1.5hef

$h_{ef} = 15\,in$ $\quad s_a = 9\,in$

$$h'_{ef} := \begin{cases} h_{ef} & \text{if } \left[\left(c_{a1} \ge 1.5 \cdot h_{ef}\right) \wedge \left(c_{a3} \ge 1.5 \cdot h_{ef}\right)\right] \vee \left(c_{a2} \ge 1.5 \cdot h_{ef}\right) \\ \max\left(\dfrac{c_{a\_max}}{1.5}, \dfrac{s_a}{3}\right) & \text{otherwise} \end{cases}$$

If anchors are located less than 1.5hef from three or more edges, hef for concrete breakout is corrected per ACI 17.6.2.1.2

$h'_{ef} = 5\,in$ Corrected hef for concrete breakout

$A_{Nco} := 9 \cdot h'^{2}_{ef} = 225\,in^2$ 40.16.3 and ACI 318 17.6.2.1.4

$A_{Nc1} := \left(\min\left(1.5 \cdot h'_{ef}, c_{a1}\right) + \min\left(1.5h'_{ef}, c_{a3}\right)\right) \cdot b = 720\,in^2$

$A_{Nc} := \min\left(A_{Nc1}, n_a \cdot A_{Nco}\right) = 720\,in^2$ ACI 17.6.2.1.1

$\psi_{ec\_N} := 1$ The eccentricity factor for a group of anchors does not apply beacause only one row of anchors is considered. ACI 17.6.2.3

$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \ge 1.5 \cdot h'_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{1.5 \cdot h'_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$\psi_{ed\_N} = 0.88$

$\psi_{c\_N} := 1.0$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$\psi_{cp\_N} := 1$

Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$

kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$$N_b := k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h'_{ef}}{in}\right)^{1.5} lbf = 11.24 \cdot kip$$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 31.66 \cdot kip$$

Nominal concrete breakout strength of anchor group in tension (ACI 17.6.2.1b)

$N_{cbg2} := N_{cbg} = 31.66\ kip$

$\phi_{tc} \cdot N_{cbg} = 23.75 \cdot kip$

<u>Anchor Bond Strength</u>

$\tau_{cr} = 500\ psi$

$\tau_{uncr} = 1500\ psi$

$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 17.67\ kip$

Basic bond strength, assuming cracked concrete given uncertain condition of the existing abutment, 40.16.3 and ACI 318 17.6.5.2.1, which is conservative

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 8.76\ in$$

ACI 318 17.6.5.1.2b

$A_{Nao} := 4 \cdot c_{Na}^{\ 2} = 306.82\ in^2$

ACI 318 Eq. 17.6.5.1.2a

$n_a \cdot A_{Nao} = 1840.91\ in^2$

$A_{Na1} := \left(\min\left(c_{Na}, c_{a1}\right) + \min\left(c_{Na}, c_{a3}\right)\right) \cdot b = 795.49\ in^2$

$A_{Na} := \min\left(A_{Na1}, n_a \cdot A_{Nao}\right) = 795.49\ in^2$

$\psi_{ec\_Na} := 1$

The eccentricity factor for a group of anchors does not apply beacause only one row of anchors is considered.

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \geq c_{Na} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$$\psi_{ed\_Na} = 0.85$$

$$\psi_{cp\_Na} := 1$$

Bond splitting factor assuming reinforcement in the existing abutment functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 39.13 \cdot \text{kip}$$

Nominal bond strength of anchor group, ACI 318 17.6.5.1

$$N_{ag2} := N_{ag} = 39.13 \text{ kip}$$

$$\phi_{tc} \cdot N_{ag} = 29.35 \cdot \text{kip}$$

Determine governing tensile strength of anchor group:

$$\phi_{ts} \cdot n_a \cdot N_{sa} = 158.4 \cdot \text{kip}$$ Steel Strength

$$\phi_{tc} \cdot N_{cbg} = 23.7 \cdot \text{kip}$$ Concrete Breakout Strength

$$\phi_{tc} \cdot N_{ag} = 29.4 \cdot \text{kip}$$ Anchor Bond Strength

$$\phi N_n := \min\left(\phi_{ts} \cdot n_a \cdot N_{sa}, \phi_{tc} \cdot N_{cbg}, \phi_{tc} \cdot N_{ag}\right) = 23.7 \cdot \text{kip}$$

Concrete breakout controls

Determine nominal tensile strength corresponding to the governing failure mode:

$$\phi_2 := \begin{cases} \phi_{ts} & \text{if } \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{cbg} \wedge \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{ag} \\ \phi_{tc} & \text{otherwise} \end{cases} = 0.75$$

$$N_n := \frac{\phi N_n}{\phi_2} = 31.66 \cdot \text{kip}$$

Nominal tensile strength (without strength reduction factor) corresponding to the governing failure mode

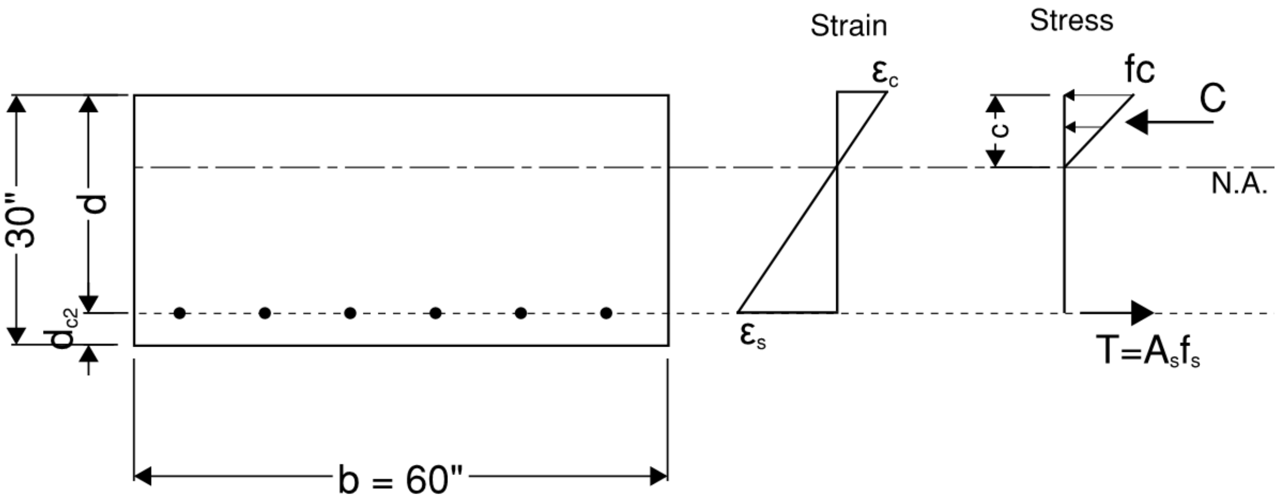
$$N_{n2} := N_n = 31.66 \text{ kip}$$

Governing failure mode in tension

$$\text{FailureMode\_N} := \begin{cases} \text{"Steel Failure"} & \text{if } \phi N_n = \phi_{ts} \cdot n_a \cdot N_{sa} \\ \text{"Concrete Breakout"} & \text{if } \phi N_n = \left(\phi_{tc} \cdot N_{cbg}\right) \\ \text{"Bond"} & \text{otherwise} \end{cases}$$

$FailureMode\_N$ = "Concrete Breakout"   Governing failure mode in tension

Flexural Resistance of Abutment Section



$N_n = 31.66\,kip$

$f_s := \frac{N_n}{A_s} = 11.99 \cdot ksi$ Tensile stress in anchor at nominal capacity

$n := \frac{E_s}{E_c} = 8.09$

Using equilibrium and strain compatibility to find the neutral axis (solve for c)

$A_s \cdot f_s = \frac{b \cdot c \cdot f_c}{2}$ Force equilibrium

$\frac{n \cdot f_c}{f_s} = \frac{c}{d - c}$ Strain compatability

$f_c := \frac{f_s}{n} \cdot \frac{c}{d - c}$

$A_s \cdot f_s := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c}\right)$

$c := 1in$ Guess value of c

$$f(c) := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c} \right) - A_s \cdot f_s$$

$c := root(f(c), c) = 3.92 \text{ in}$ Solve for c

Check equilibrium:

$$f_c := \frac{f_s}{n} \frac{c}{d - c} = 269.35 \text{ psi}$$

Compressive stress in concrete at the extreme compressive fiber

$$A_s \cdot f_s = \frac{b \cdot c}{2} \cdot f_c = 1$$

Equilibrium OK

$$M_{n2} := N_n \cdot \left(d - \frac{c}{3} \right) = 63.84 \cdot ft \cdot kip$$

$$\phi_2 \cdot M_{n2} = 47.88 \cdot ft \cdot kip$$

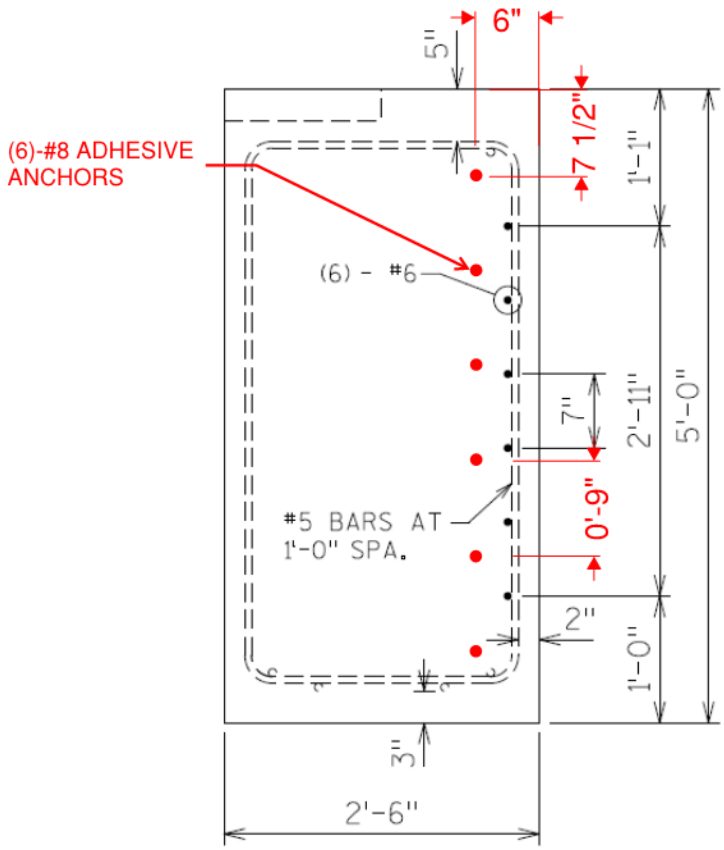
Flexural resistance is much smaller than in Case 1 because the tensile strength of anchors is smaller than yield strength and the resistance factor ϕ for adhesive anchor is smaller than that for a tension-controlled section in the original construction

Case 3: Alternative Designs of Adhesive Anchors

Three alternatives to the design in Case 2 (6-#6) are considered:
3A - Increase anchor size to (6)-#8
3B - Increase the number of anchors to (9)-#6
3C - Add another row of (6)-#6 anchors

3A) Increase Bar Size - 6 #8

CASE 3A



$d_a := 1in$ $n_a := 6$

$A_{se} := 0.79in^2$ $A_s := n_a \cdot A_{se} = 4.74\, in^2$

$l_{ebd} := 15in$ Actual embedment depth

$d_{c2} := 6in$ Edge distance increased to meet ACI 318 min edge distance requirements

$d := h - d_{c2} = 24 \cdot in$ Depth of steel reinforcment

$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15 \cdot in$ Effective embedded length. ACI 318 17.3.3

$s_a := 9in$ Anchor spacing longitudinally

$c_{a1} := d_{c2} = 6 \cdot in$

$c_{a2} := 7.5in$

$c_{a3} := d = 24 \cdot in$

$c_{a\_min} := \min(c_{a1}, c_{a2}) = 6 \cdot in$ Min edge distance

Check edge distances, spacings

$s_{a\_min} := 6 \cdot d_a = 6\ in$ Minimum spacing required, ACI 318 17.9.2

$c_{edge\_min} := \max(c_c, 2 \cdot d_{agg}, 6 \cdot d_a) = 6\ in$ Minimum edge distance required, ACI 318 17.9.2

$s_a \geq s_{a\_min} = 1$ OK

$c_{a\_min} \geq c_{edge\_min} = 1$ OK

Tensile Strength of Anchors

Steel Strength

$f_{uta} := \min(f_u, 1.9 \cdot f_y, 125ksi) = 80 \cdot ksi$ ACI 17.6.1.2

$N_{sa} := A_{se} \cdot f_{uta} = 63.2 \cdot kip$ ACI Eq. 17.6.1.2

$N_{sa3a} := N_{sa} = 63.2\ kip$

$\phi_{ts} \cdot n_a \cdot N_{sa} = 284.4 \cdot kip$

Concrete Breakout Strength

$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15 \cdot in$ Effective embedded length. ACI 318 17.3.3

$c_{a1} = 6\ in$ $c_{a2} = 7.5\ in$ $c_{a3} = 24\ in$

$$check_1 := \begin{cases} 1 & \text{if } c_{a1} \leq 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_2 := \begin{cases} 1 & \text{if } c_{a2} \leq 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_3 := \begin{cases} 1 & \text{if } c_{a3} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}$$

$c_{a\_max} := \max(c_{a1} \cdot check_1, c_{a2} \cdot check_2, c_{a3} \cdot check_3) = 7.5\ in$ — Max edge distance that is less than 1.5hef

$h_{ef} = 15\ in$ $s_a = 9\ in$

$$h'_{ef} := \begin{cases} h_{ef} & \text{if } [(c_{a1} \ge 1.5 \cdot h_{ef}) \wedge (c_{a3} \ge 1.5 \cdot h_{ef})] \vee (c_{a2} \ge 1.5 \cdot h_{ef}) \\ \max\left(\frac{c_{a\_max}}{1.5}, \frac{s_a}{3}\right) & \text{otherwise} \end{cases}$$

If anchors are located less than 1.5hef from three or more edges, hef for concrete breakout is corrected per ACI 17.6.2.1.2

$h'_{ef} = 5\ in$ — Corrected hef for concrete breakout

$A_{Nco} := 9 \cdot h'^{2}_{ef} = 225\ in^2$ — 40.16.3 and ACI 318 17.6.2.1.4

$A_{Nc1} := (\min(1.5 \cdot h'_{ef}, c_{a1}) + \min(1.5h'_{ef}, c_{a3})) \cdot b = 810\ in^2$

$A_{Nc} := \min(A_{Nc1}, n_a \cdot A_{Nco}) = 810\ in^2$ — ACI 17.6.2.1.1

$\psi_{ec\_N} := 1$ — The eccentricity factor for a group of anchors does not apply beacause only one row of anchors is considered. ACI 17.6.2.3

$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \ge 1.5 \cdot h'_{ef} \\ 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h'_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$\psi_{ed\_N} = 0.94$

$\psi_{c\_N} := 1.0$ — For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$\psi_{cp\_N} := 1$ — Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$ — kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$$N_b := k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h'_{ef}}{in}\right)^{1.5} lbf = 11.24 \cdot kip$$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 38.05 \cdot kip$

Nominal concrete breakout strength of anchor group in tension (ACI 17.6.2.1b)

$N_{cbg3a} := N_{cbg} = 38.05\ kip$

$\phi_{tc} \cdot N_{cbg} = 28.54 \cdot kip$

Concrete breakout strength in tension is greater than in Case 2

Anchor Bond Strength

$\tau_{cr} = 1.11 \cdot 450psi$

$\tau_{uncr} = 1500\ psi$

$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 23.56\ kip$

Basic bond strength, assuming cracked concrete given uncertain condition of the existing abutment, 40.16.3 and ACI 318 17.6.5.2.1

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 11.68\ in$

ACI 318 17.6.5.1.2b

$A_{Nao} := 4 \cdot c_{Na}^{\ 2} = 545.45\ in^2$

ACI 318 Eq. 17.6.5.1.2a

$n_a \cdot A_{Nao} = 3272.73\ in^2$

$A_{Na1} := \left(\min\left(c_{Na}, c_{a1}\right) + \min\left(c_{Na}, c_{a3}\right)\right) \cdot b = 1060.65\ in^2$

$A_{Na} := \min\left(A_{Na1}, n_a \cdot A_{Nao}\right) = 1060.65\ in^2$

$\psi_{ec\_Na} := 1$

The eccentricity factor for a group of anchors does not apply beacause only one row of anchors is considered.

$$\psi_{ed\_Na} := \left| \begin{array}{ll} 1 & \text{if } c_{a\_min} \geq c_{Na} \\ 0.7 + 0.3 \cdot \frac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{array} \right.$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 0.85$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the existing abutment functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 39.13 \cdot kip$ — Nominal bond strength of anchor group, ACI 318 17.6.5.1

$N_{ag3a} := N_{ag} = 39.13\ kip$

$\phi_{tc} \cdot N_{ag} = 29.35 \cdot kip$ — Bond strength in tension is the same as in Case 2

Determine governing tensile strength of anchor group:

$\phi_{ts} \cdot n_a \cdot N_{sa} = 284.4 \cdot kip$ — Steel Strength

$\phi_{tc} \cdot N_{cbg} = 28.54 \cdot kip$ — Concrete Breakout Strength

$\phi_{tc} \cdot N_{ag} = 29.35 \cdot kip$ — Anchor Bond Strength

$\phi N_n := \min\left(\phi_{ts} \cdot n_a \cdot N_{sa}, \phi_{tc} \cdot N_{cbg}, \phi_{tc} \cdot N_{ag}\right) = 28.54 \cdot kip$ — Concrete Breakout controls

Determine nominal tensile strength corresponding to the governing failure mode:

$$\phi_3 := \begin{cases} \phi_{ts} & \text{if } \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{cbg} \wedge \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{ag} \\ \phi_{tc} & \text{otherwise} \end{cases} = 0.75$$

$N_n := \frac{\phi N_n}{\phi_3} = 38.05 \cdot kip$ — Nominal tensile strength (without strength reduction factor) corresponding to the governing failure mode

$N_{n3a} := N_n = 38.05\ kip$

Moment Capacity

$f_s := \frac{N_n}{A_s} = 8.03 \cdot ksi$ — Tensile stress in anchor at nominal capacity

$n := \frac{E_s}{E_c} = 8.09$

$A_s \cdot f_s = \frac{b \cdot c \cdot f_c}{2}$ — Force equilibrium

$\frac{n \cdot f_c}{f_s} = \frac{c}{d - c}$ — Strain compatability

$f_c := \frac{f_s}{n} \cdot \frac{c}{d - c}$

$$A_s \cdot f_s := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c}\right)$$

$c := 1in$ Guess value of c

$$f(c) := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c}\right) - A_s \cdot f_s$$

$c := root(f(c), c) = 4.94\ in$ Solve for c

$$M_{n3a} := N_n \cdot \left(d - \frac{c}{3}\right) = 70.89 \cdot ft \cdot kip$$

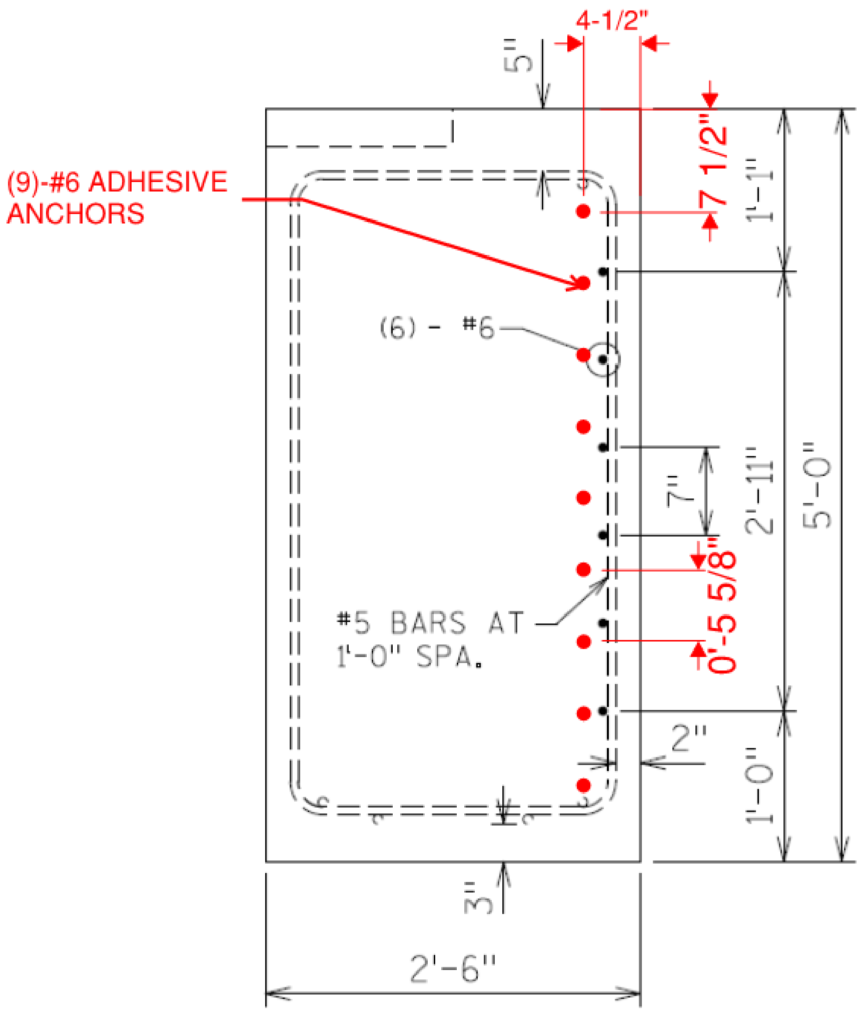
$$\phi_3 \cdot M_{n3a} = 53.16 \cdot ft \cdot kip$$

Flexural resistance is almost the same in Case 2

Conccrete breakout strength is greater than in Case 2 because of the increased edge distance required to meet ACI 318 requirement; however the increased edge distance also reduces the moment arm of the tensile force in the anchors.

3B) Increase Bar Quantity - 9 #6

CASE 3B



a

$A := 0.44in$ a $A_s := n_a \cdot A_{se} = 3.96\ in^2$

l_{ebd} Actual embedment depth

d_{c2} Same as in Case 2

$d := h - d_{c2} = 25.5 \cdot in$ Depth of steel reinforcment

$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15 \cdot in$ Effective embedded length. ACI 318 17.3.3

$s_a := 5.625\,in$ Anchor spacing longitudinally

$c_{a1} := d_{c2} = 4.5 \cdot in$

$c_{a2} := 7.5in$

$c_{a3} := d = 25.5 \cdot in$

$c_{a\_min} := \min(c_{a1}, c_{a2}) = 4.5 \cdot in$ Min edge distance

Check edge distances, spacings

$s_{a\_min} := 6 \cdot d_a = 4.5\,in$ Minimum spacing required, ACI 318 17.9.2

$c_{edge\_min} := \max(c_c, 2 \cdot d_{agg}, 6 \cdot d_a) = 4.5\,in$ Minimum edge distance required, ACI 318 17.9.2

$s_a \geq s_{a\_min} = 1$ OK

$c_{a\_min} \geq c_{edge\_min} = 1$ OK

Tensile Strength of Anchors

Steel Strength

$f_{uta} := \min(f_u, 1.9 \cdot f_y, 125ksi) = 80 \cdot ksi$ ACI 17.6.1.2

$N_{sa} := A_{se} \cdot f_{uta} = 35.2 \cdot kip$ ACI Eq. 17.6.1.2

$N_{sa3b} := N_{sa} = 35.2\,kip$

$\phi_{ts} \cdot n_a \cdot N_{sa} = 237.6 \cdot kip$

Concrete Breakout Strength

$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15 \cdot in$ Effective embedded length. ACI 318 17.3.3

$c_{a1} = 4.5\,in$ $c_{a2} = 7.5\,in$ $c_{a3} = 25.5\,in$

$$check_1 := \begin{cases} 1 & \text{if } c_{a1} \leq 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_2 := \begin{cases} 1 & \text{if } c_{a2} \leq 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_3 := \begin{cases} 1 & \text{if } c_{a3} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}$$

$c_{a\_max} := \max\left(c_{a1} \cdot check_1, c_{a2} \cdot check_2, c_{a3} \cdot check_3\right) = 7.5\,in$ Max edge distance that is less than 1.5hef

$h_{ef} = 15\,in$ $s_a = 5.63\,in$

$$h'_{ef} := \begin{cases} h_{ef} & \text{if } \left[\left(c_{a1} \ge 1.5 \cdot h_{ef}\right) \wedge \left(c_{a3} \ge 1.5 \cdot h_{ef}\right)\right] \vee \left(c_{a2} \ge 1.5 \cdot h_{ef}\right) \\ \max\left(\frac{c_{a\_max}}{1.5}, \frac{s_a}{3}\right) & \text{otherwise} \end{cases}$$

If anchors are located less than 1.5hef from three or more edges, hef for concrete breakout is corrected per ACI 17.6.2.1.2

$h'_{ef} = 5\,in$ Corrected hef for concrete breakout

$A_{Nco} := 9 \cdot h'^{2}_{ef} = 225\,in^2$ 40.16.3 and ACI 318 17.6.2.1.4

$A_{Nc1} := \left(\min\left(1.5 \cdot h'_{ef}, c_{a1}\right) + \min\left(1.5h'_{ef}, c_{a3}\right)\right) \cdot b = 720\,in^2$

$A_{Nc} := \min\left(A_{Nc1}, n_a \cdot A_{Nco}\right) = 720\,in^2$ ACI 17.6.2.1.1

$\psi_{ec\_N} := 1$ The eccentricity factor for a group of anchors does not apply beacause only one row of anchors is considered. ACI 17.6.2.3

$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \ge 1.5 \cdot h'_{ef} \\ 0.7 + 0.3 \cdot \frac{c_{a\_min}}{1.5 \cdot h'_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$\psi_{ed\_N} = 0.88$

$\psi_{c\_N} := 1.0$ For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$\psi_{cp\_N} := 1$ Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$ kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$N_b := k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h'_{ef}}{in}\right)^{1.5} lbf = 11.24 \cdot kip$ Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 31.66 \cdot kip$

Nominal concrete breakout strength of anchor group in tension (ACI 17.6.2.1b)

$N_{cbg3b} := N_{cbg} = 31.66\ kip$

$\phi_{tc} \cdot N_{cbg} = 23.75 \cdot kip$

Concrete breakout strength in tension is the same as in Case 2

Anchor Bond Strength

$\tau_{cr} = 1.11 \cdot 450psi$

$\tau_{uncr} = 1500\ psi$

$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 17.67\ kip$

Basic bond strength, assuming cracked concrete given uncertain condition of the existing abutment. 40.16.3 and ACI 318 17.6.5.2.1

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 8.76\ in$

ACI 318 17.6.5.1.2b

$A_{Nao} := 4 \cdot c_{Na}^2 = 306.82\ in^2$

ACI 318 Eq. 17.6.5.1.2a

$n_a \cdot A_{Nao} = 2761.36\ in^2$

$A_{Na1} := \left(\min\left(c_{Na}, c_{a1}\right) + \min\left(c_{Na}, c_{a3}\right)\right) \cdot b = 795.49\ in^2$

$A_{Na} := \min\left(A_{Na1}, n_a \cdot A_{Nao}\right) = 795.49\ in^2$

$\psi_{ec\_Na} := 1$

The eccentricity factor for a group of anchors does not apply beacause only one row of anchors is considered.

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \geq c_{Na} \\ 0.7 + 0.3 \cdot \frac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 0.85$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the lexisting abutment functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$N_{ag} := \dfrac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 39.13 \cdot kip$

Nominal bond strength of anchor group, ACI 318 17.6.5.1

$N_{ag3b} := N_{ag} = 39.13\ kip$

$\phi_{tc} \cdot N_{ag} = 29.35 \cdot kip$

Bond strength in tension is the same as in Case 2

Determine governing tensile strength of anchor group:

$\phi_{ts} \cdot n_a \cdot N_{sa} = 237.6 \cdot kip$ — Steel Strength

$\phi_{tc} \cdot N_{cbg} = 23.75 \cdot kip$ — Concrete Breakout Strength

$\phi_{tc} \cdot N_{ag} = 29.35 \cdot kip$ — Anchor Bond Strength

$\phi N_n := \min\left(\phi_{ts} \cdot n_a \cdot N_{sa}, \phi_{tc} \cdot N_{cbg}, \phi_{tc} \cdot N_{ag}\right) = 23.75 \cdot kip$

Concrete Breakout controls

Determine nominal tensile strength corresponding to the governing failure mode:

$$\phi_3 := \begin{array}{|l} \phi_{ts} \text{ if } \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{cbg} \wedge \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{ag} \\ \phi_{tc} \text{ otherwise} \end{array} = 0.75$$

$N_{n3b} := N_n = 38.05\ kip$

$N_n := \dfrac{\phi N_n}{\phi_3} = 31.66 \cdot kip$

Nominal tensile strength (without strength reduction factor) corresponding to the governing failure mode

Moment Capacity

$f_s := \dfrac{N_n}{A_s} = 8 \cdot ksi$

Tensile stress in anchor at nominal capacity

$n := \dfrac{E_s}{E_c} = 8.09$

$A_s \cdot f_s = \dfrac{b \cdot c \cdot f_c}{2}$

Force equilibrium

$\dfrac{n \cdot f_c}{f_s} = \dfrac{c}{d - c}$

Strain compatability

$$f_c := \frac{f_s}{n} \cdot \frac{c}{d - c}$$

$$A_s \cdot f_s := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c} \right)$$

$c := 1\text{in}$ Guess value of c

$$f(c) := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c} \right) - A_s \cdot f_s$$

$c := \text{root}(f(c), c) = 4.71\ \text{in}$ Solve for c

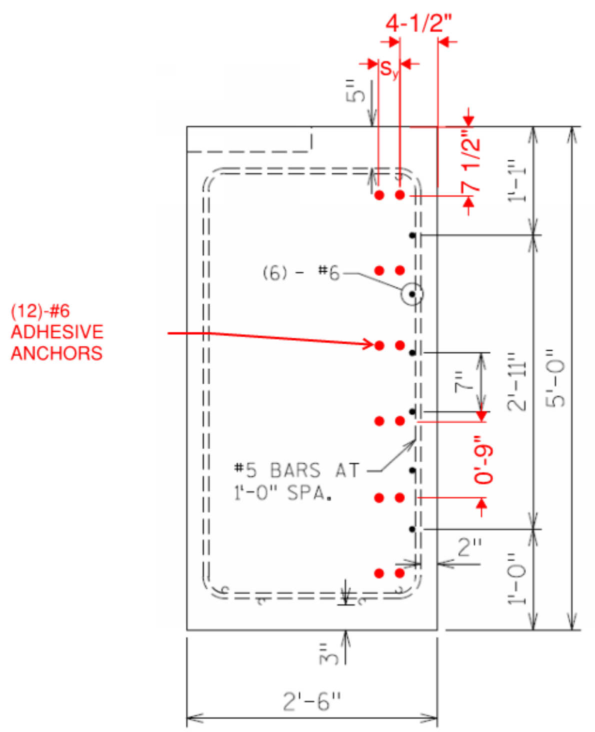
$$M_{n3b} := N_n \cdot \left(d - \frac{c}{3} \right) = 63.14 \cdot \text{ft} \cdot \text{kip}$$

$$\phi_3 \cdot M_{n3b} = 47.36 \cdot \text{ft} \cdot \text{kip}$$

Flexural resistance is slightly smaller than in Case 2. While the tensile strength of the anchor group is the same as in Case 2, the stress distribution in the concrete slightly changes due to the greater area of reinforcement (i.e. c is greater than in Case 2), resulting in the change in the flexural resistance.

3C) Add a Row of Bars - 2 rows of (6)-#6

CASE 3C



$d_a := 0.75in$

$n_a := 12$

$A_{se} := 0.44in^2$

$A_s := n_a \cdot A_{se} = 5.28\, in^2$

$A_{s1} := \frac{A_s}{2} = 2.64\, in^2$

$A_{s2} := \frac{A_s}{2} = 2.64\, in^2$

$s_y := 2in$ Center-to-center spacing of anchor rows (minimum rebar spacing permitted per ACI 318-19 25.2.1)

$l_{ebd} := 15in$ Actual embedment depth

$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15 \cdot in$ Effective embedded length. ACI 318 17.3.3

$d_{e2} := 4.5in$

$d_1 := h - d_{c2} = 25.5 \cdot in$ depth of steel reinforcment

$d_2 := h - d_{c2} - s_y = 23.5\,in$

$s_a := 9in$ Anchor spacing longitudinally

$c_{a1} := d_{c2} = 4.5 \cdot in$

$c_{a2} := 7.5in$

$c_{a3} := d_2 = 23.5 \cdot in$

$c_{a\_min} := \min\left(c_{a1}, c_{a2}, c_{a3}\right) = 4.5 \cdot in$ Min edge distance

Check edge distances, spacings

$s_{a\_min} := 6 \cdot d_a = 4.5\,in$ Minimum spacing required, ACI 318 17.9.2

$c_{edge\_min} := \max\left(c_c, 2 \cdot d_{agg}, 6 \cdot d_a\right) = 4.5\,in$ Minimum edge distance required, ACI 318 17.9.2

$s_a \geq s_{a\_min} = 1$ OK

$c_{a\_min} \geq c_{edge\_min} = 1$ OK

Tensile Strength of Anchors

<u>Steel Strength</u>

$f_{uta} := \min\left(f_u, 1.9 \cdot f_y, 125ksi\right) = 80 \cdot ksi$ ACI 17.6.1.2

$N_{sa} := A_{se} \cdot f_{uta} = 35.2 \cdot kip$ ACI Eq. 17.6.1.2

$N_{sa3c} := N_{sa} = 35.2\,kip$

$\phi_{ts} \cdot n_a \cdot N_{sa} = 316.8 \cdot kip$

<u>Concrete Breakout Strength</u>

$c_{a1} = 4.5\,in$ $c_{a2} = 7.5\,in$ $c_{a3} = 23.5\,in$

$$check_1 := \begin{cases} 1 & \text{if } c_{a1} \leq 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_2 := \begin{cases} 1 & \text{if } c_{a2} \leq 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check_3 := \begin{cases} 1 & \text{if } c_{a3} \le 1.5 \cdot h_{ef} \\ 0 & \text{otherwise} \end{cases}$$

$$check = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}$$

$$c_{a\_max} := \max\left(c_{a1} \cdot check_1, c_{a2} \cdot check_2, c_{a3} \cdot check_3\right) = 7.5\,in$$

Max edge distance that is less than 1.5hef

$h_{ef} = 15\,in$ $s_a = 9\,in$

$$h'_{ef} := \begin{cases} h_{ef} & \text{if } \left[\left(c_{a1} \ge 1.5 \cdot h_{ef}\right) \wedge \left(c_{a3} \ge 1.5 \cdot h_{ef}\right)\right] \vee \left(c_{a2} \ge 1.5 \cdot h_{ef}\right) \\ \max\left(\dfrac{c_{a\_max}}{1.5}, \dfrac{s_a}{3}\right) & \text{otherwise} \end{cases}$$

If anchors are located less than 1.5hef from three or more edges, hef for concrete breakout is corrected per ACI 17.6.2.1.2

$h'_{ef} = 5\,in$

Corrected hef for concrete breakout

$$A_{Nco} := 9 \cdot h'^{\,2}_{ef} = 225\,in^2$$

40.16.3 and ACI 318 17.6.2.1.4

$$A_{Nc1} := \left(\min\left(1.5 \cdot h'_{ef}, c_{a1}\right) + s_y + \min\left(1.5h'_{ef}, c_{a3}\right)\right) \cdot b = 840\,in^2$$

$$A_{Nc} := \min\left(A_{Nc1}, n_a \cdot A_{Nco}\right) = 840\,in^2$$

ACI 17.6.2.1.1

Eccentricity adjustment factor

Tensile forces of the two rows of anchors can be different, thus modification factor for an anchor group loaded eccentrically needs to be considered per ACI 318 17.6.2.3. The calculations here first assumes no eccentricity, then determine stress distribution in the section and assess effect of ecentricity.

$e'_N := 0in$

Assume no eccentricity in tension

$$\psi_{ec\_N} := \begin{cases} 1 & \text{if } \dfrac{1}{1 + \dfrac{e'_N}{1.5 \cdot h'_{ef}}} > 1 \\ \dfrac{1}{1 + \dfrac{e'_N}{1.5 \cdot h'_{ef}}} & \text{otherwise} \end{cases}$$

ACI Eq. 17.6.2.3.1

$\psi_{ec\_N} = 1$

$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \geq 1.5 \cdot h'_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{1.5 \cdot h'_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$\psi_{ed\_N} = 0.88$

$\psi_{c\_N} := 1.0$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$\psi_{cp\_N} := 1$

Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$

kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$$N_b := k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h'_{ef}}{in}\right)^{1.5} lbf = 11.24 \cdot kip$$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 36.9 \cdot kip$$

Nominal concrete breakout strength of anchor group in tension (ACI 17.6.2.1b)

$N_{cbg3c} := N_{cbg} = 36.94\ kip$

$\phi_{tc} \cdot N_{cbg} = 27.71 \cdot kip$

Concrete breakout strength in tension is slightly greater than in Case 2

Anchor Bond Strength

$\tau_{cr} = 500\ psi$

$\tau_{uncr} = 1500\ psi$

$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 17.67\ kip$

Basic bond strength, assuming cracked concrete given uncertain condition of the existing abutment. 40.16.3 and ACI 318 17.6.5.2.1

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 8.76\ in$$

ACI 318 17.6.5.1.2b

$A_{Nao} := 4 \cdot c_{Na}^{\ 2} = 306.82\ in^2$

ACI 318 Eq. 17.6.5.1.2a

$n_a \cdot A_{Nao} = 3681.82\ in^2$

$A_{Na1} := \left(\min\left(c_{Na}, c_{a1}\right) + s_y + \min\left(c_{Na}, c_{a3}\right)\right) \cdot b = 915.49\ in^2$

$A_{Na} := \min\left(A_{Na1}, n_a \cdot A_{Nao}\right) = 915.49\ in^2$

$$\psi_{ec\_Na} := \begin{cases} 1 & \text{if } \dfrac{1}{1 + \dfrac{e'_N}{c_{Na}}} > 1 \\ \dfrac{1}{1 + \dfrac{e'_N}{c_{Na}}} & \text{otherwise} \end{cases}$$

ACI Eq. 17.6.5.3.1

$\psi_{ec\_Na} = 1$

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \geq c_{Na} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 0.85$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the existing abutment functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 45.04 \cdot kip$$

Nominal bond strength of anchor group, ACI 318 17.6.5.1

$N_{ag3c} := N_{ag} = 45.04\ kip$

$\phi_{tc} \cdot N_{ag} = 33.78 \cdot kip$

Bond strength in tension is greater than in Case 2

Determine governing tensile strength of anchor group:

$\phi_{ts} \cdot n_a \cdot N_{sa} = 316.8 \cdot kip$ Steel Strength

$\phi_{tc} \cdot N_{cbg} = 27.71 \cdot kip$ Concrete Breakout Strength

$\phi_{tc} \cdot N_{ag} = 33.78 \cdot kip$ Anchor Bond Strength

$\phi N_n := \min\left(\phi_{ts} \cdot n_a \cdot N_{sa}, \phi_{tc} \cdot N_{cbg}, \phi_{tc} \cdot N_{ag}\right) = 27.71 \cdot kip$

Concrete breakout controls

Determine nominal tensile strength corresponding to the governing failure mode:

$$\phi_3 := \begin{cases} \phi_{ts} & \text{if } \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{cbg} \wedge \phi_{ts} \cdot n_a \cdot N_{sa} < \phi_{tc} \cdot N_{ag} \\ \phi_{tc} & \text{otherwise} \end{cases} = 0.75$$

$$N_n := \frac{\phi N_n}{\phi_3} = 36.94 \cdot kip$$

Nominal tensile strength (without strength reduction factor) corresponding to the governing failure mode

Moment Capacity

$$f_s := \frac{N_n}{A_s} = 7 \cdot ksi$$

Average tensile stress in anchor at nominal capacity

$$n := \frac{E_s}{E_c} = 8.09$$

$$d_g := d - \frac{s_y}{2} + e'_N = 24.5\,in \qquad d_g := \frac{d_1 + d_2}{2} = 24.5\,in$$

$$A_s \cdot f_s = \frac{b \cdot c \cdot f_c}{2}$$

Force equilibrium

$$\frac{n \cdot f_c}{f_s} = \frac{c}{d_g - c}$$

Strain compatability

$$f_c := \frac{f_s}{n} \cdot \frac{c}{d - c}$$

$$A_s \cdot f_s := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d_g - c} \right)$$

$$c := 1in$$

Guess value of c

$$f(c) := \frac{b \cdot c}{2} \cdot \left(\frac{f_s}{n} \cdot \frac{c}{d - c} \right) - A_s \cdot f_s$$

$$c := root(f(c), c) = 5.35\,in$$

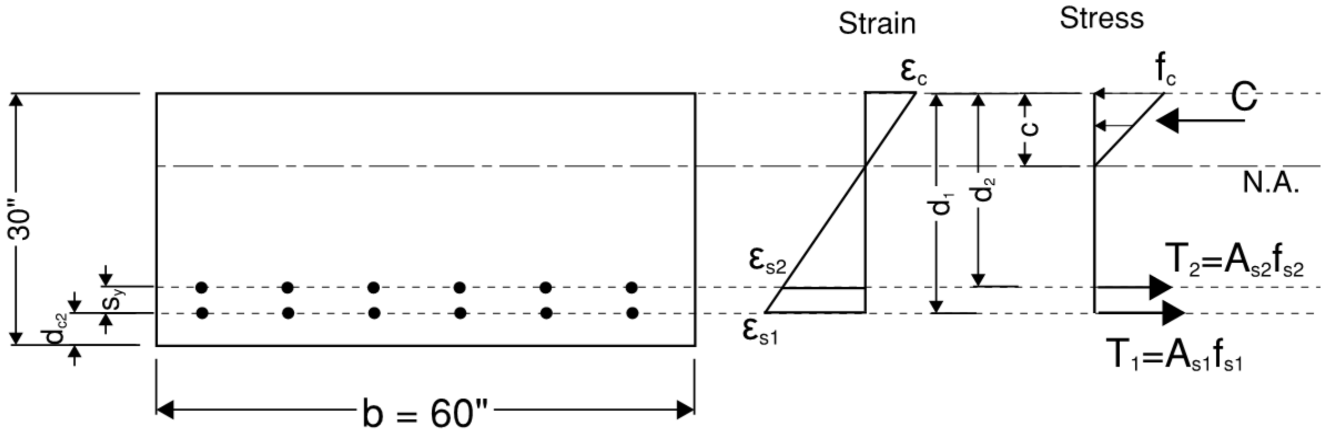
Solve for c

$$M_{n3c} := A_s \cdot f_s \cdot \left(d_g - \frac{c}{3} \right) = 69.93 \cdot ft \cdot kip$$

$$\phi_3 \cdot M_{n3c} = 52.45 \cdot ft \cdot kip$$

Without considering eccentricity factors for concrete breakout and bond strength, the flexural resistance is slightly greater than in Case 2.

Estimate eccentricity factors and recalculate anchor tensile strength



$$f_c := \frac{f_s}{n} \cdot \frac{c}{d_g - c} = 241.99\ \text{psi}$$

Compressive stress in concrete at extreme fiber

$$f_{s1} := n \cdot f_c \cdot \frac{d_1 - c}{c} = 7.36 \cdot \text{ksi}$$

Tensile stress in first row of anchors

$$f_{s2} := n \cdot f_c \cdot \frac{d_2 - c}{c} = 6.63 \cdot \text{ksi}$$

Tensile stress in second row of anchors

$$T_1 := A_{s1} \cdot f_{s1} = 19.44\ \text{kip}$$

$$T_2 := A_{s2} \cdot f_{s2} = 17.51\ \text{kip}$$

$$e'_N := \frac{\left(T_1 \cdot \frac{s_y}{2}\right) - \left(T_2 \cdot \frac{s_y}{2}\right)}{T_1 + T_2} = 0.05\ \text{in}$$

Eccentricity

$$\psi_{ec\_N} := \begin{cases} 1 & \text{if } \dfrac{1}{1 + \dfrac{e'_N}{1.5 \cdot h'_{ef}}} > 1 \\ \dfrac{1}{1 + \dfrac{e'_N}{1.5 \cdot h'_{ef}}} & \text{otherwise} \end{cases}$$

ACI Eq. 17.6.2.3.1

$$\psi_{ec\_N} = 0.993$$

Eccentricity factor for concrete breakout strength in tension

$$\psi_{ec\_Na} := \begin{cases} 1 & \text{if } \dfrac{1}{1 + \dfrac{e'_N}{c_{Na}}} > 1 \\ \dfrac{1}{1 + \dfrac{e'_N}{c_{Na}}} & \text{otherwise} \end{cases}$$

ACI Eq. 17.6.5.3.1

Eccentricity factor for bond strength

$\psi_{ec\_Na} = 0.994$

Effect of eccentricity on concrete breakout and bond strengths is negligible. No need to recalculate flexural resistance.

Additional calculations for increased spacing between the two rows of anchors, s_y, were performed (not shown here) and indicated that the tensile strength of the anchor group was slightly increased, but it was offset by the reduced effective depth of the reinforcement and thus, resulting in about the same the flexural resistance.

Summary of flexural resistances for different cases

Case 1 - Original construction (6)-#6

$M_{n1} = 350.54\ ft\cdot kip$

$\phi_1 \cdot M_{n1} = 315.49\ ft\cdot kip$

Case 2 - Adhesive anchors (6)-#6

$M_{n2} = 63.84\ ft\cdot kip$

$\phi_2 \cdot M_{n2} = 47.88\cdot ft\cdot kip$

$$\frac{\phi_2 \cdot M_{n2}}{\phi_1 \cdot M_{n1}} = 0.15$$

Case 3a - Adhesive anchors (6)-#8

$M_{n3a} = 70.89\ ft\cdot kip$

$\phi_3 \cdot M_{n3a} = 53.16\ ft\cdot kip$

$$\frac{\phi_3 \cdot M_{n3a}}{\phi_1 \cdot M_{n1}} = 0.17$$

Case 3b - Adhesive anchors (9)-#6

$M_{n3b} = 63.14\ ft \cdot kip$

$\phi_3 \cdot M_{n3b} = 47.36\ ft \cdot kip$

$\frac{\phi_3 \cdot M_{n3b}}{\phi_1 \cdot M_{n1}} = 0.15$

Case 3c - Adhesive anchors, two rows of (6)-#6

$M_{n3c} = 69.93\ ft \cdot kip$

$\phi_3 \cdot M_{n3c} = 52.45\ ft \cdot kip$

$\frac{\phi_3 \cdot M_{n3c}}{\phi_1 \cdot M_{n1}} = 0.17$

Concluding comments:

In all four designs of adhesive anchors, the factored flexural resistance of the section was calculated to be only about 17%, or less, of the resistance of the original section.

The governing failure mode in every alternate adhesive anchor design was concrete breakout in tension, which is limited by the edge distance and anchor spacing. This is consistent with the commentary in ACI 318-19, Section R17.6.5.1. For a #6 anchor with 15-in. embedment depth as in the example, the concrete breakout strength of the group of 6 anchors was smaller than that of a single anchor that has an edge distance greater than $1.5h_{ef}$. (h_{ef} is the effective embedment depth). Increasing the edge distance of the anchors (from the tensile surface) only slightly increases the flexural resistance since the increased in concrete breakout resistance thanks to larger edge distance is offset by the reduced moment arm (from the tension force to the center of the compression zone). Similarly, increasing the the number of anchors (including adding an additional row of anchors) does not improve the concrete breakout strength in a meaningful way since the concrete breakout strength of the anchor group is limited by the projected concrete failure area of the anchor group, A_{Nc}, which is limited by the edge distance and anchor spacing.

The use of lower resistance factors (strength reduction factors) for adhesive anchors, as compared with the resistance factor for flexure of a tension-controlled section, in accordance with ACI 318 also contributed to the reduced flexural resistance for the repaired sections.

DESIGN EXAMPLE 3 - CONCRETE PARRAPET REPLACEMENT USING ADHESIVE ANCHORS

General Information

In this example, lateral resistance to vehicular impact load of a replaced concrete parapet on a bridge deck is calculated in accordance with AASHTO LRFD 9th Edition. Tensile strength of adhesive anchors connecting the parapet to the deck is calculated in accordance with ACI 318-19.

References

- ACI 318-19 Building Code Requirements for Structural Concrete and Commentary
- WisDOT Bridge Manual 2020
- *AASHTO LRFD 9th Edition*

Note: All sectional references in the calculations refer to the WisDOT Bridge Manual 2020 unless otherwise noted.

General Assumptions

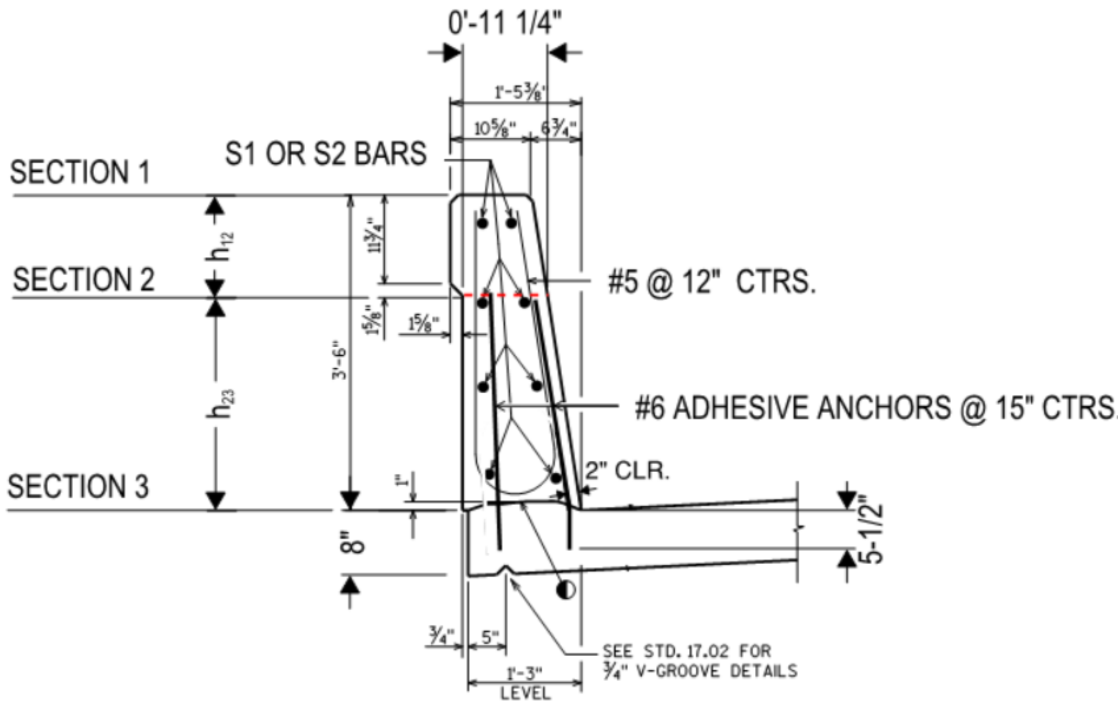
- *Concrete of the existing deck is cracked for the purpose of calculating anchor resistances.*
- *Reinforcement in the deck functions as supplementary reinforcement for adhesive anchors.*

Legend:

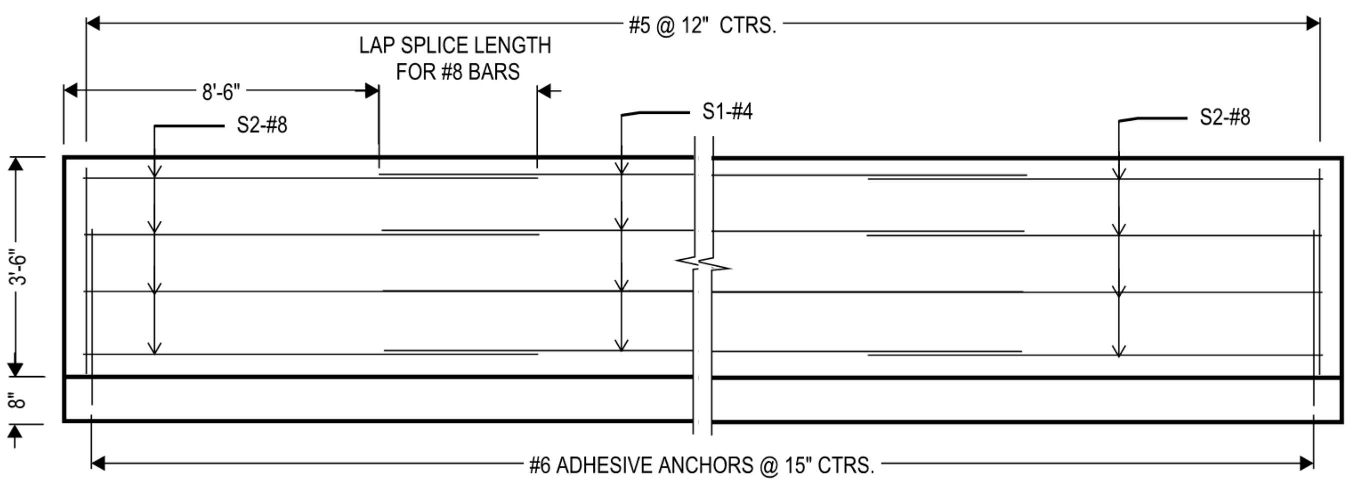
Input

Design Results/Check

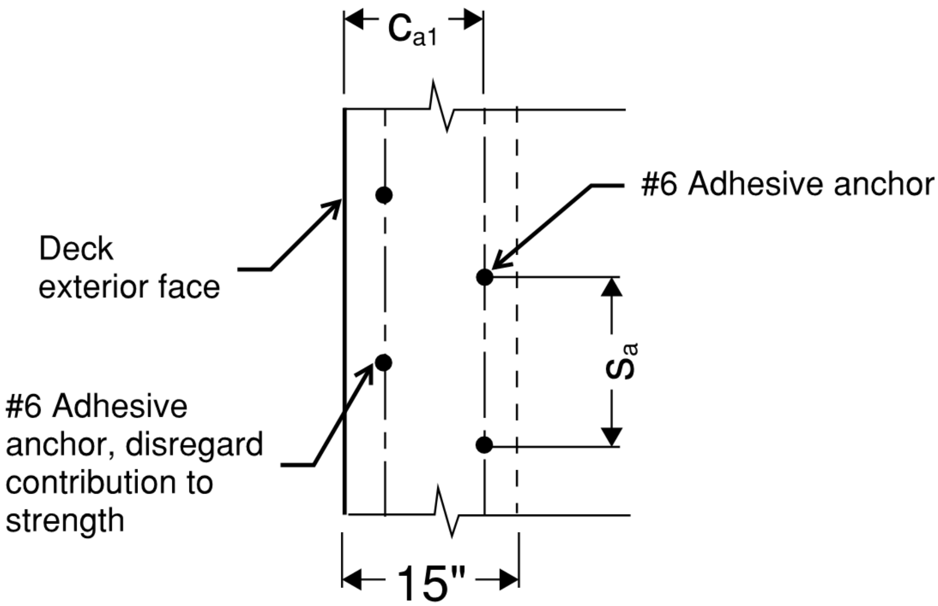
42SS Concrete Parapet with Adhesive Anchors



SECTION THROUGH PARAPET ON BRIGE



ELEVATION



PLAN VIEW

Materials and Geometries

$f'_c := 4000psi$	*Concrete compressive strength*
$f_{ua} := 80ksi$	*Steel anchor tensile strength*
$f_{ya} := 60ksi$	*Steel anchor yield strength*
$d_{agg} := 1.5in$	*Assumed max aggregate size*
$h_a := 8in$	*Concrete deck thickness*
$c_c := 2in$	*Clear cover from anchor to parapet interior face, concrete cover per ACI 318 20.5.1.3.1 for No. 6 bars exposed to weather or in contact with ground*

ADHESIVE ANCHOR STRENGTH (ACI 318-19 CHAPTER 17)

Tensile strength of #6 adhesive anchors

$d_a := 0.75in$ *Diameter of #6 anchor*

$s_a := 30in$ *Anchor spacing longitudinally*

$l_{ebd} := 5.5in$ *Actual embedment depth*

$4 \cdot d_a \le l_{ebd} \le 20d_a = 1$ *OK* *Embedment depth limit, ACI 318 17.3.3*

$h_{ef} := \min\left(l_{ebd}, 20 \cdot d_a\right) = 5.5\,in$ *Effective embedded length. ACI 318 17.3.3*

$A_{se\_N} := \dfrac{\pi \cdot d_a^{\,2}}{4} = 0.44\,in^2$ *Anchor cross-sectional area*

$A_{se\_V} := \dfrac{\pi \cdot d_a^{\,2}}{4} = 0.44\,in^2$

$c_{a1} := 15in - c_c - \dfrac{d_a}{2} = 12.6\,in$ *Edge distance from center or anchor to edge of deck*

$c_{a\_min} := c_{a1} = 12.6\,in$ *Min edge distance*

Check edge distances, spacings, and thicknesses

$s_{a\_min} := 6 \cdot d_a = 4.5\,in$ *Minimum spacing required, ACI 318 17.9.2*

$c_{edge\_min} := \max\left(c_c, 2 \cdot d_{agg}, 6 \cdot d_a\right) = 4.5\,in$ *Minimum edge distance required, ACI 318 17.9.2*

$s_a \ge s_{a\_min} = 1$ *OK*

$c_{a\_min} \ge c_{edge\_min} = 1$ *OK*

Adhesive anchor tensile strength

Tensile strength - steel

$\phi_{ts} := 0.75$ *For ductile steel, ACI 17.5.3*

$f_{uta} := \min\left(f_{ua}, 1.9 \cdot f_{ya}, 125ksi\right) = 80 \cdot ksi$

$N_{sa} := A_{se\_N} \cdot f_{uta} = 35.3\,kip$

$\phi_{ts} \cdot N_{sa} = 26.5\,kip$ *Factored tensile strength of steel anchor:*

Yield strength of steel anchor:

$N_{ya} := A_{se\_N} \cdot f_{ya} = 26.5\ \text{kip}$ — *Unfactored yield strength of steel anchor:*

$\phi_{ts} \cdot \left(N_{ya}\right) = 19.9\ \text{kip}$ — *Factored yield strength of steel anchor:*

<u>*Tensile strength - Concrete breakout*</u>

$s_{cr\_cb} := 3 \cdot h_{ef} = 16.5\ \text{in}$ — *Critical spacing for concrete breakout in tension, ACI 318 17.5.1.3.1*

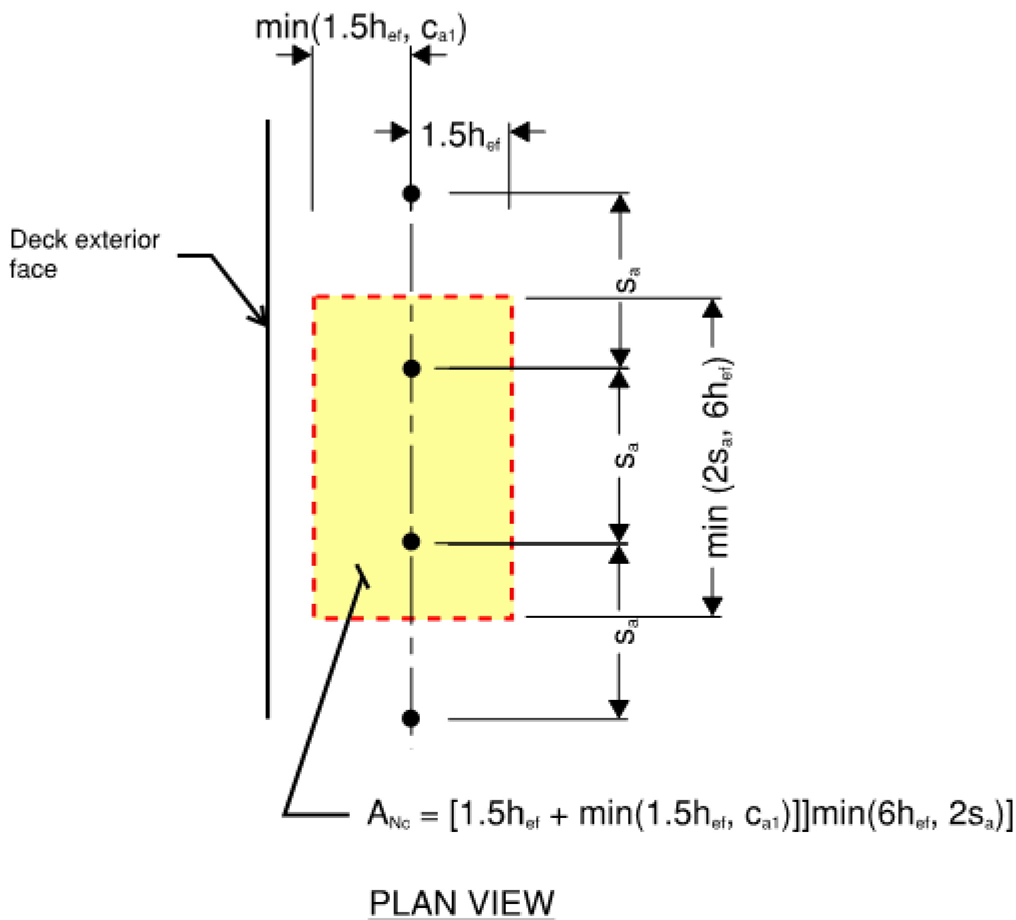
$s_a < s_{cr\_cb} = 0$ — *Anchor spacing is less than critical spacing, anchor group must be considered.*

Since the anchors are uniformly spaced, anchor group effect can be accounted for by considering a group of two adjacent anchors:

$n := 2$ — *Number of anchors in the group*

$A_{Nco} := 9 \cdot h_{ef}^{\ 2} = 272.2\ \text{in}^2$ — *40.16.3 and ACI 318 17.6.2.1.4*

$A_{Nc} := \left(\min\left(c_{a1}, 1.5 \cdot h_{ef}\right) + 1.5 h_{ef}\right) \cdot \min\left(2 s_a, 6 h_{ef}\right) = 544.5\ \text{in}^2$ — *ACI 17.6.2.1.1*



$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \geq 1.5 \cdot h_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{1.5 \cdot h_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$$\psi_{ed\_N} = 1$$

$$\psi_{ec\_N} := 1$$

The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered. ACI 17.6.2.3

$$\psi_{c\_N} := 1.0$$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$$\psi_{cp\_N} := 1$$

Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$

kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$N_b := k_c \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} lbf = 13.9 \cdot kip$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 27.7\, kip$

Concrete breakout strength of a single bolt in tension (ACI 318-14 17.4.2.1a)

$N_{cb} := \frac{N_{cbg}}{n} = 13.9\, kip$

Average concrete breakout strength of an anchor in a group

$\phi_{tc} := 0.75$

For concrete breakout or bond strength in tension with supplementary reinforcement, Anchor Category 1, ACI 318 17.5.3. Assuming deck reinforcement functions as supplementary reinforcement for adhesive anchors.

$\phi_{tc} \cdot N_{cb} = 10.4 \cdot kip$

<u>Anchor bond strength</u>

$\tau_{uncr} := 1800psi$

Characteristic bond stress for uncracked concrete

$\tau_{cr} := 1300psi$

Characteristic bond stress for uncracked concrete

Characteristic bond stresses were selected to avoid bond failure.

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 9.59\, in$

ACI 318 17.6.5.1.2b

$s_{cr\_a} := 2 \cdot c_{Na} = 19.2\, in$

Critical spacing for bond strength in tension, ACI 318 17.5.1.3.1

$s_a < s_{cr\_a} = 0$

Anchor spacing is less than critical spacing, anchor group must be considered.

Consider a group of two adjacent anchors:

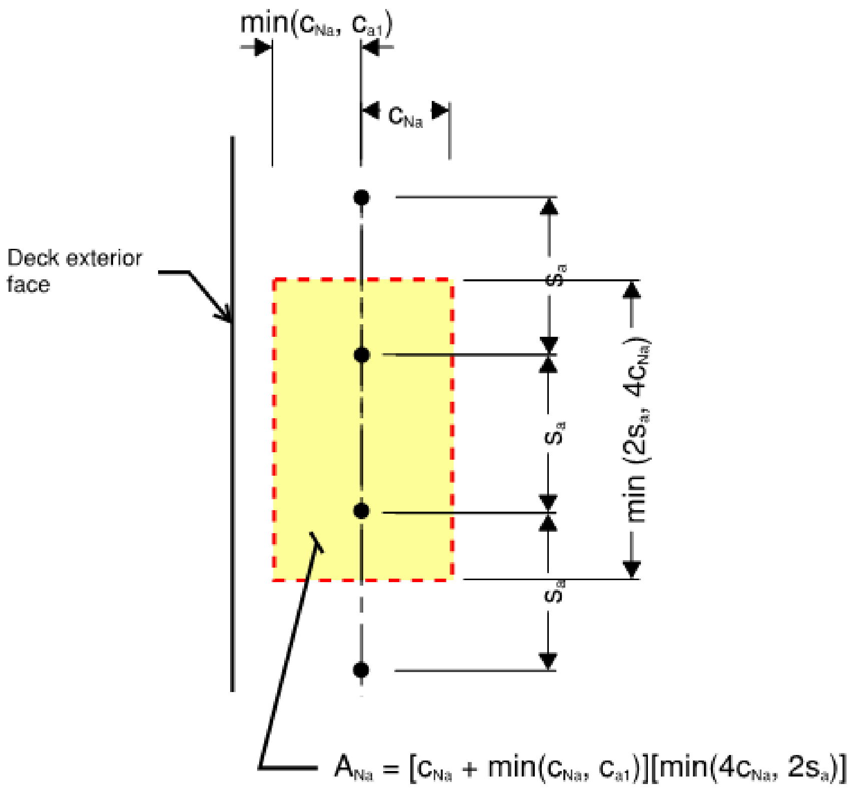
$n := 2$

Number of anchors in the group

$A_{Nao} := 4 \cdot c_{Na}^{\ 2} = 368.2\, in^2$

ACI 318 Eq. 17.6.5.1.2a

$A_{Na} := \left(c_{Na} + \min\left(c_{Na}, c_{a1}\right)\right) \cdot \min\left(4 \cdot c_{Na}, 2s_a\right) = 736.4\, in^2$



PLAN VIEW

$\psi_{ec\_Na} := 1$

The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered.

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \ge c_{Na} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 1$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the deck functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 16.85\ \text{kip}$

Basic bond strength, assuming cracked concrete deck. 40.16.3 and ACI 318 17.6.5.2.1

$$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 33.7\,\text{kip}$$

Nominal bond strength of anchor group, ACI 318 17.6.5.1

$$N_a := \frac{N_{ag}}{n} = 16.8\,\text{kip}$$

Average nominal bond strength of an anchor in group

Nominal strength of anchor in tension

$N_{sa} = 35.3\,\text{kip}$ — *Nominal steel tensile strength*

$N_{ya} = 26.5\,\text{kip}$ — *Nominal steel yield strength*

$N_{cb} = 13.9\,\text{kip}$ — *Nominal concrete breakout tensile strength*

$N_a = 16.8\,\text{kip}$ — *Nominal bond strength*

$N_n := \min(N_{sa}, N_{cb}, N_a) = 13.9\,\text{kip}$ — *Governing nominal strength of an adhesive anchor in group*

Factored strength of anchor in tension

$\phi_{ts} \cdot N_{sa} = 26.5\,\text{kip}$

$\phi_{tc} \cdot N_{cb} = 10.4\,\text{kip}$

$\phi_{tc} \cdot N_a = 12.6\,\text{kip}$

$N_r := \min(\phi_{ts} \cdot N_{sa}, \phi_{tc} \cdot N_{cb}, \phi_{tc} \cdot N_a) = 10.4\,\text{kip}$ — *Average factored strength of an adhesive anchor in group*

<u>Governing failure mode in tension</u>

$$\text{FailureMode\_N} := \begin{cases} \text{"Steel Failure"} & \text{if } N_r = \phi_{ts} \cdot N_{sa} \\ \text{"Concrete Breakout"} & \text{if } N_r = \phi_{tc} \cdot N_{cb} \\ \text{"Bond"} & \text{otherwise} \end{cases}$$

FailureMode_N = "Concrete Breakout" — *Governing failure mode in tension*

Check if anchor yields at nominal tensile strength:

$$\frac{N_n}{N_{ya}} = 0.52$$

< 1: Anchor does not yield at nominal tensile strength

Try smaller anchors

Tensile strength of #4 adhesive anchors

$d_a := 0.5in$ *Diameter of anchor*

$s_a := 15in$ *Anchor spacing longitudinally*

$l_{ebd} := 5.5in$ *Actual embedment depth*

$4 \cdot d_a \le l_{ebd} \le 20d_a = 1$ *OK* *Embedment depth limit, ACI 318 17.3.3*

$h_{ef} := \min\left(l_{ebd}, 20 \cdot d_a\right) = 5.5\,in$ *Effective embedded length. ACI 318 17.3.3*

$A_{se\_N} := \dfrac{\pi \cdot d_a^{\,2}}{4} = 0.2\,in^2$ *Anchor cross-sectional area*

$A_{se\_V} := \dfrac{\pi \cdot d_a^{\,2}}{4} = 0.2\,in^2$

$c_{a1} := 15in - c_c - \dfrac{d_a}{2} = 12.8\,in$ *Edge distance from center or anchor to edge of deck*

$c_{a\_min} := c_{a1} = 12.8\,in$ *Min edge distance*

Check edge distances, spacings, and thicknesses

$s_{a\_min} := 6 \cdot d_a = 3\,in$ *Minimum spacing required, ACI 318 17.9.2*

$c_{edge\_min} := \max\left(c_c, 2 \cdot d_{agg}, 6 \cdot d_a\right) = 3\,in$ *Minimum edge distance required, ACI 318 17.9.2*

$s_a \ge s_{a\_min} = 1$ *OK*

$c_{a\_min} \ge c_{edge\_min} = 1$ *OK*

Adhesive anchor tensile strength

Tensile strength - steel

$\phi_{ts} := 0.75$ *For ductile steel, ACI 17.5.3*

$f_{uta} := \min\left(f_{ua}, 1.9 \cdot f_{ya}, 125ksi\right) = 80 \cdot ksi$

$N_{sa} := A_{se\_N} \cdot f_{uta} = 15.7\,kip$

$\phi_{ts} \cdot N_{sa} = 11.8\ \text{kip}$ *Factored tensile strength of steel anchor:*

Yield strength of steel anchor:

$N_{ya} := A_{se\_N} \cdot f_{ya} = 11.8\ \text{kip}$ *Unfactored yield strength of steel anchor:*

$\phi_{ts} \cdot \left(N_{ya}\right) = 8.8\ \text{kip}$ *Factored yield strength of steel anchor:*

Tensile strength - Concrete breakout

$s_{cr\_cb} := 3 \cdot h_{ef} = 16.5\ \text{in}$ *Critical spacing for concrete breakout in tension, ACI 318 17.5.1.3.1*

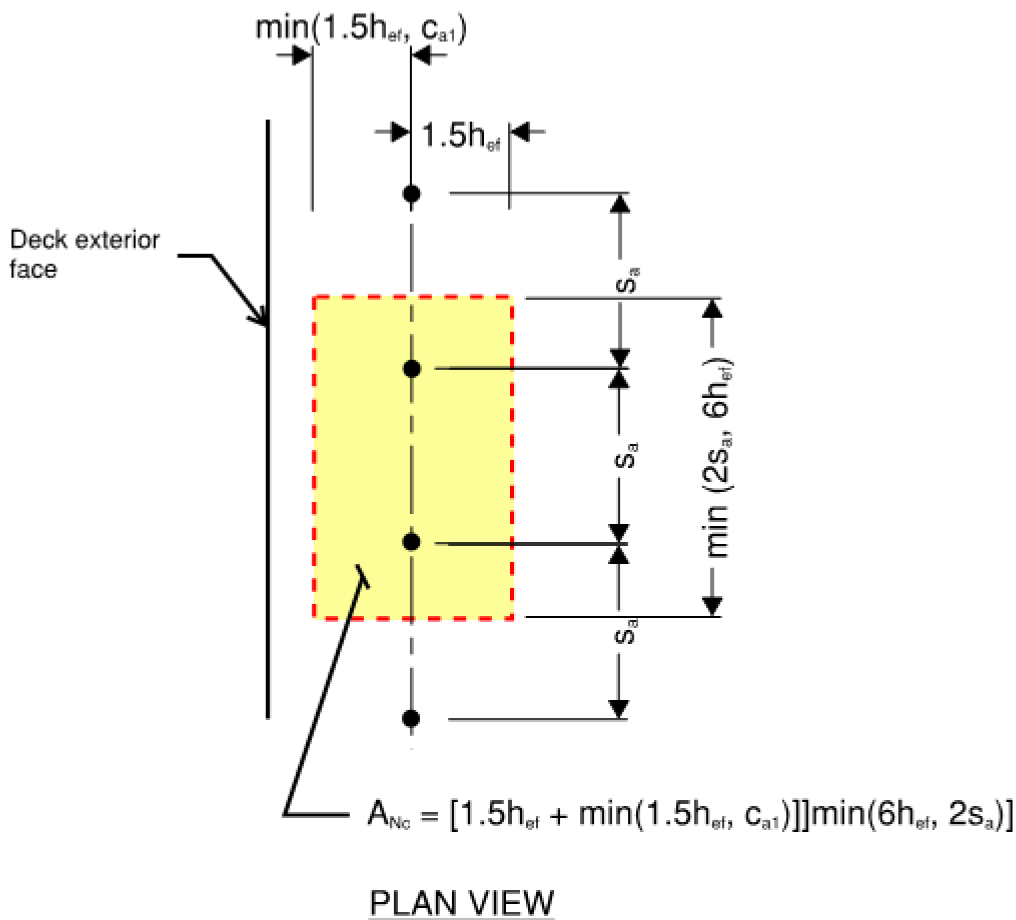
$s_a < s_{cr\_cb} = 1$ *Anchor spacing is less than critical spacing, anchor group must be considered.*

Since the anchors are uniformly spaced, anchor group effect can be accounted for by considering a group of two adjacent anchors:

$n := 2$ *Number of anchors in the group*

$A_{Nco} := 9 \cdot h_{ef}^{2} = 272.2\ \text{in}^2$ *40.16.3 and ACI 318 17.6.2.1.4*

$A_{Nc} := \left(\min\left(c_{a1}, 1.5 \cdot h_{ef}\right) + 1.5 h_{ef}\right) \cdot \min\left(2 s_a, 6 h_{ef}\right) = 495\ \text{in}^2$ *ACI 17.6.2.1.1*



$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \geq 1.5 \cdot h_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{1.5 \cdot h_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects, 40.16.3 and ACI 318 17.6.2.4

$\psi_{ed\_N} = 1$

$\psi_{ec\_N} := 1$

The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered. ACI 17.6.2.3

$\psi_{c\_N} := 1.0$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$\psi_{cp\_N} := 1$

Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6.2)

$k_c := 17$

kc = 17 for post-installed anchor, ACI 318 17.6.2.2.1

$N_b := k_c \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} lbf = 13.9 \cdot kip$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318 17.6.2.2.1)

$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 25.2\,kip$

Concrete breakout strength of a single bolt in tension (ACI 318-14 17.4.2.1a)

$N_{cb} := \frac{N_{cbg}}{n} = 12.6\,kip$

Average concrete breakout strength of an anchor in a group

$\phi_{tc} := 0.75$

For concrete breakout or bond strength in tension with supplementary reinforcement, Anchor Category 1, ACI 318 17.5.3. Assuming deck reinforcement functions as supplementary reinforcement for adhesive anchors.

$\phi_{tc} \cdot N_{cb} = 9.46 \cdot kip$

<u>Anchor bond strength</u>

$\tau_{uncr} := 1800psi$

Characteristic bond stress for uncracked concrete

$\tau_{cr} := 1500psi$

Characteristic bond stress for uncracked concrete

Characteristic bond stresses were increased to avoid bond failure

$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 6.4\,in$

ACI 318 17.6.5.1.2b

$s_{cr\_a} := 2 \cdot c_{Na} = 12.8\,in$

Critical spacing for bond strength in tension, ACI 318 17.5.1.3.1

$s_a < s_{cr\_a} = 0$

Anchor spacing is less than critical spacing, anchor group must be considered.

Consider a group of two adjacent anchors:

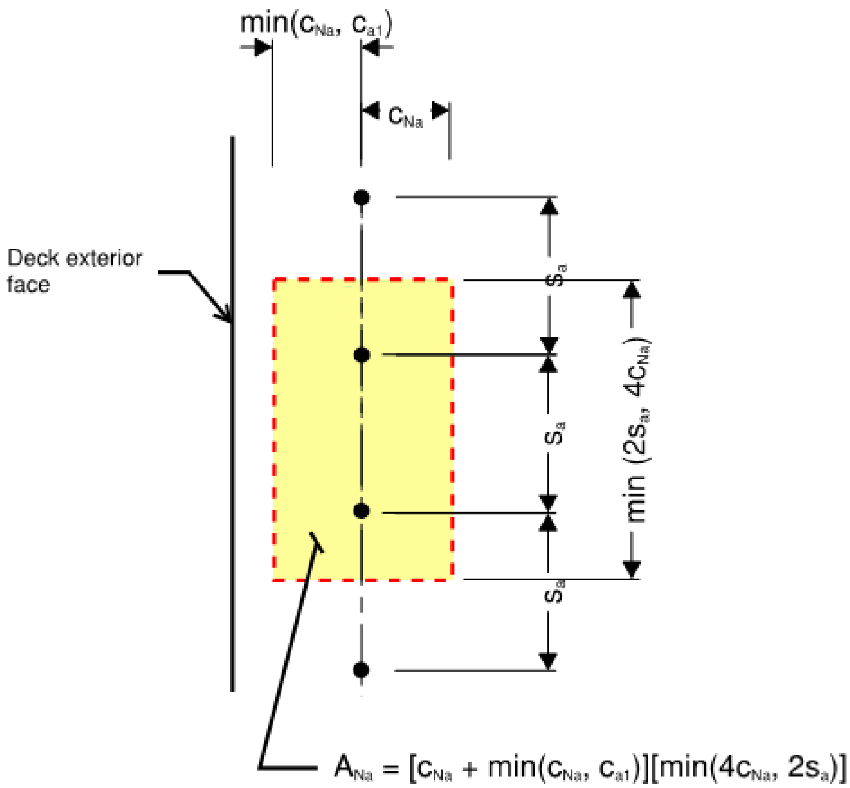
$n := 2$

Number of anchors in the group

$A_{Nao} := 4 \cdot c_{Na}^{\ 2} = 163.6\,in^2$

ACI 318 Eq. 17.6.5.1.2a

$A_{Na} := \left(c_{Na} + \min\left(c_{Na}, c_{a1}\right)\right) \cdot \min\left(4 \cdot c_{Na}, 2s_a\right) = 327.3\,in^2$



PLAN VIEW

$\psi_{ec\_Na} := 1$

The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered.

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \geq c_{Na} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 1$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the deck functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$N_{ba} := \tau_{cr} \cdot \pi \cdot d_a \cdot h_{ef} = 12.96\ \text{kip}$

Basic bond strength, assuming cracked concrete deck. 40.16.3 and ACI 318 17.6.5.2.1

$$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ec\_Na} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 25.9\,\text{kip}$$

Nominal bond strength of anchor group, ACI 318 17.6.5.1

$$N_a := \frac{N_{ag}}{n} = 13\,\text{kip}$$

Average nominal bond strength of an anchor in group

Nominal strength of anchor in tension

$$N_{sa} = 15.7\,\text{kip}$$
$$N_{cb} = 12.6\,\text{kip}$$
$$N_a = 13.0\,\text{kip}$$

$$N_n := \min\left(N_{sa}, N_{cb}, N_a\right) = 12.6\,\text{kip}$$

Average nominal strength of an adhesive anchor in group

Factored strength of anchor in tension

$$\phi_{ts} \cdot N_{sa} = 11.8\,\text{kip}$$
$$\phi_{tc} \cdot N_{cb} = 9.5\,\text{kip}$$
$$\phi_{tc} \cdot N_a = 9.7\,\text{kip}$$

$$N_r := \min\left(\phi_{ts} \cdot N_{sa}, \phi_{tc} \cdot N_{cb}, \phi_{tc} \cdot N_a\right) = 9.5\,\text{kip}$$

Average factored strength of an adhesive anchor in group

<u>Governing failure mode in tension</u>

$$\text{FailureMode\_N} := \left| \begin{array}{ll} \text{"Steel Failure"} & \text{if } N_r = \phi_{ts} \cdot N_{sa} \\ \text{"Concrete Breakout"} & \text{if } N_r = \phi_{tc} \cdot N_{cb} \\ \text{"Bond"} & \text{otherwise} \end{array} \right.$$

FailureMode_N = "Concrete Breakout"

Governing failure mode in tension

Check if anchor yields at nominal tensile strength:

$$\frac{N_n}{N_{ya}} = 1.07$$

> 1: Anchor yields at nominal tensile strength

DESIGN OF REINFORCED CONCRETE PARAPET USING YIELD LINE ANALYSIS (AASHTO LRFD CHAPTER 13)

<u>Loads</u>

42SS parapet has Test Level TL-4.

$F_t := 54\text{kip}$ — *Transverse design force, AASHTO Table A13.2-1*

$L_t := 3.5\text{ft}$ — *Transverse force distribution length, AASHTO Table A13.2-1*

$\phi := 1$ — *Resistance factor for Extreme Event Limit State, AASHTO 1.3.2.1*

$\phi_v := 1$

Geometry

$H := 42\text{in}$ — *Total height of parapet*

$h_{12} := 11.75\text{in} + 1.625\text{in} = 13.4\text{ in}$ — *Height of parapet between sections 1 and 2*

$h_{23} := H - h_{12} = 28.6\text{ in}$ — *Height of parapet between sections 2 and 3*

$w_1 := 10.625\text{in}$ — *Parapet width - Section 1*

$w_2 := 11.25\text{in}$ — *Parapet width - Section 2*

$w_3 := 15.75\text{in}$ — *Parapet width - Section 3*

$c_c = 2\text{ in}$ — *Clear cover to vertical bar*

Adhesive Anchor

Adhesive anchors at typical parapet section and at end of parapet

$d_a = 0.5\text{ in}$ — *Anchor diameter*

$f_y := f_{ya} = 60\cdot\text{ksi}$ — *Yield strength of steel reinforcment*

$T_n := N_n \cdot \frac{1\text{ft}}{s_a} = 10.1\text{ kip}$ — *Average nominal tensile strength of adhesive anchor per foot length of parapet*

$T_r := N_r \cdot \frac{1\text{ft}}{s_a} = 7.6\text{ kip}$ — *Average factored tensile strength of adhesive anchors per foot length of parapet*

Parapet Reinforcement

Reinforcement at a typical parapet section

$d_{bv\_typical} := 0.625\text{in}$ — *Vertical rebar diameter*

$s_{bv\_typical} := 12\text{in}$ — *Vertical rebar spacing*

$d_{bh\_typical} := 0.5in$ *Horizontal rebar diameter*

Reinforcement at end region of parapet

$d_{bv\_end} := 0.625in$ *Vertical rebar diameter*

$s_{bv\_end} := 12in$ *Vertical rebar spacing*

$d_{bh\_end} := 1.0in$ *Horizontal rebar diameter*

Parapet Resistance to Transverse Force - For impacts within a wall segment:

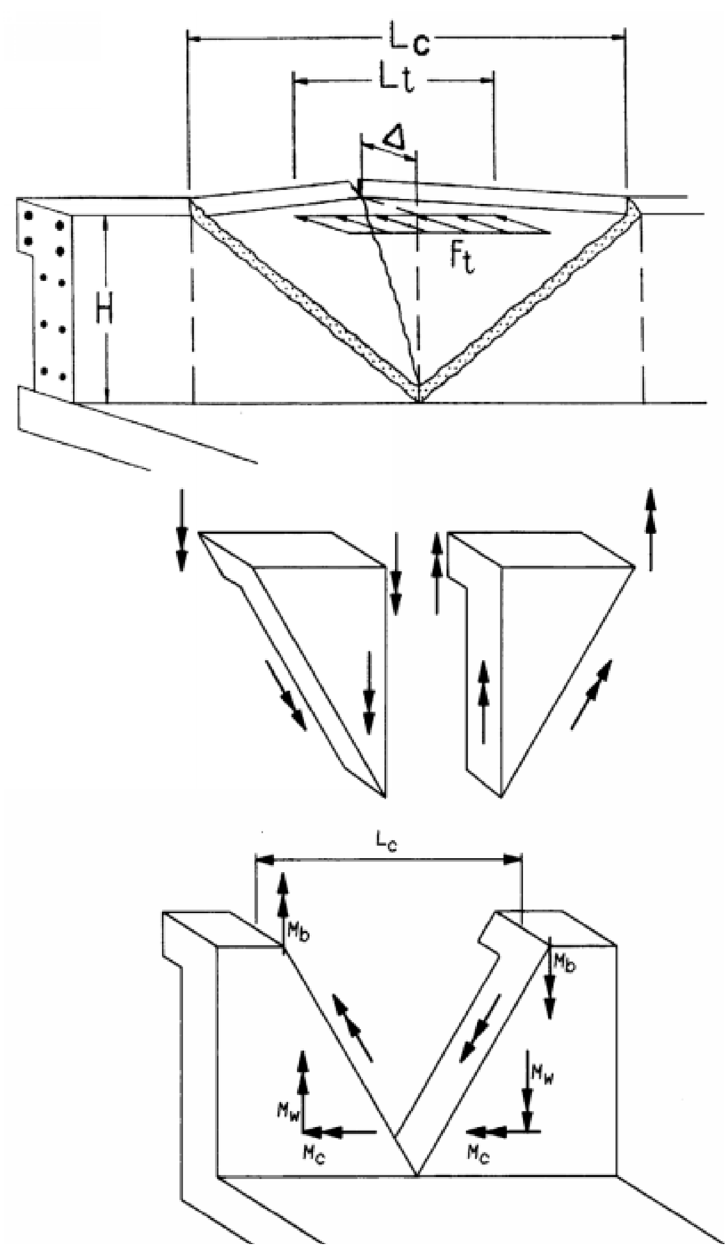


Figure CA13.3.1-1—Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment

Parapet Resistance to Lateral Impact Load - Within Wall Segment (LRFD A13.2.1)

Parapet CIP vertical bars

$d_{bv} := d_{bv\_typical} = 0.625\ in$ *Vertical bar diameter*

$s_{bv} := s_{bv\_typical} = 12\ in$ *Vertical bar spacing*

$A_{bv} := \dfrac{\pi\, d_{bv}^{\ 2}}{4} = 0.31\ in^2$ *Vertical bar area*

$A_{sv} := \dfrac{A_{bv} \cdot 12in}{s_{bv}} = 0.31 \cdot in^2$ *Vertical bar area per linear foot of parapet*

1. Determine M_C : flexural resistance of cantilevered parapet about an axis parallel to the longitudinal axis of the bridge. Flexural moment resistance is based on the vertical reinforcement in the barrier.

Calculation for 1 ft length of parapet:

$b := 12in$

At section 1:

$$a_1 := \frac{A_{sv} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.45\ in$$

$$d_1 := w_1 - c_c - \frac{d_{bv}}{2} = 8.31\ in$$

$$M_{c1} := \phi \cdot A_{sv} \cdot f_y \cdot \left(d_1 - \frac{a_1}{2}\right) = 12.4\ ft \cdot kip$$

At section 2:

$$a_2 := \frac{A_{sv} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.45\ in$$

$$d_2 := w_2 - \left(c_c - \frac{d_{bv}}{2}\right) = 9.6\ in$$

$$M_{c2} := \phi \cdot A_{sv} \cdot f_y \cdot \left(d_2 - \frac{a_2}{2}\right) = 14.3\ \mathrm{ft \cdot kip}$$

At section 3:

Flexural resistance of the parapet about its longitudinal axis at this section is determined based on tensile resistance of the adhesive anchor.

$$a_3 := \frac{T_n}{0.85 \cdot f'_c \cdot b} = 0.25\ \mathrm{in}$$

a_3 is calculated using nominal tensile resistance of adhesive anchors

$$d_3 := w_3 - \left(c_c - \frac{d_a}{2}\right) = 14\ \mathrm{in}$$

$$M_{c3} := T_r \cdot \left(d_3 - \frac{a_3}{2}\right) = 8.75\ \mathrm{ft \cdot kip}$$

M_{c3} is calculated using factored tensile resistance of adhesive anchors

Calculate average Mc

$$M_c := \frac{\left(\frac{M_{c1} + M_{c2}}{2}\right) \cdot h_{12} + \left(\frac{M_{c2} + M_{c3}}{2}\right) \cdot h_{23}}{H} = 12.12\ \mathrm{ft \cdot kip}$$

2. Determine M_w : flexural resistance of the parapet about its vertical axis.

$$d_{bh} := d_{bh\_typical} = 0.5\ \mathrm{in}$$

Parapet horizontal bar diameter

$$A_{bh} := \frac{\pi \cdot d_{bh}^2}{4} = 0.2\ \mathrm{in}^2$$

Parapet horizontal bar area

Upper portion between sections 1 and 2:

$$A_{sh} := 1 \cdot A_{bh} = 0.2\ \mathrm{in}^2$$

$$a_{12} := \frac{A_{sh} \cdot f_y}{0.85 \cdot f'_c \cdot h_{12}} = 0.3\ \mathrm{in}$$

$$w_{12} := \frac{w_1 + w_2}{2} = 10.9\ \mathrm{in}$$

Average width of parapet

$$d_{12} := w_{12} - c_c - d_{bv} - \frac{d_{bh}}{2} = 8.1\,\text{in}$$

$$M_{w12} := \phi \cdot A_{sh} \cdot f_y \cdot \left(d_{12} - \frac{a_{12}}{2}\right) = 7.8\,\text{ft}\cdot\text{kip}$$

Lower portion between sections 2 and 3:

$$A_s := 3 \cdot A_{bh} = 0.6\,\text{in}^2$$

$$a_{23} := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot h_{23}} = 0.4\,\text{in}$$

$$w_{23} := \frac{w_2 + w_3}{2} = 13.5\,\text{in}$$

Average width of parapet

$$d_{23} := w_{23} - c_c - d_a - \frac{d_{bh}}{2} = 10.7\,\text{in}$$

$$M_{w23} := \phi \cdot A_s \cdot f_y \cdot \left(d_{23} - \frac{a_{23}}{2}\right) = 31.1\,\text{ft}\cdot\text{kip}$$

Total Mw

$$M_w := M_{w12} + M_{w23} = 38.9\,\text{ft}\cdot\text{kip}$$

3. Rail resistance within a wall segment.

$$M_b := 0\,\text{ft}\cdot\text{kip}$$

There is no beam on top of parapet

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H \cdot (M_b + M_w)}{\frac{M_c}{1\,\text{ft}}}} = 11.4 \cdot \text{ft}$$

Critical length of yield line failure pattern, AASHTO A13.3.1-2

$$R_w := \left(\frac{2}{2 \cdot L_c - L_t}\right) \cdot \left(8 \cdot M_b + 8 \cdot M_w + \frac{M_c \cdot L_c^{\,2}}{H \cdot 1\,\text{ft}}\right) = 78.9\,\text{kip}$$

Railing resistance to transverse load, AASHTO A13.3.1-1

Check railing demand/capacity

$\frac{F_t}{R_w} = 0.68$

D/C <=1: OK

Shear Resistance

It is demonstrated in this section that the lateral impact load can be resisted by the interface shear resistance between the parapet and bridge deck and by the shear strentgh of the parapet in bending about the vertical axis. Thus, the adhesive anchors are not to be loaded in shear, and reduction in tensile strength of adhesive anchors due to tension-shear interaction per ACI 318, 17.8, does not need to be considered.

Interface shear resistance , LRFD 5.7.4.3

Calculate interface shear resistance between the parapet and bridge deck

$b_{vi} := 15in$ — *Interface width considered to be engaged in shear transfer*

$L_{vi} := L_c = 11.4 \cdot ft$ — *Interface length considered to be engaged in shear transfer*

$A_{cv} := b_{vi} \cdot L_{vi} = 2050.7\, in^2$ — *Interface area considered to be engaged in shear transfer*

$A_{vf} := A_{se\_N} \cdot \frac{L_c}{s_a} = 1.79\, in^2$ — *Cross-sectional area of steel reinforcement within the interface area*

Cohesion and friction factors, LRFD 5.7.4.4:

For calculations in this example, it was assumed concrete placed against a clean concrete surface, free of laitance, with surface not intentionally roughened. In practice, it is advisable that the concrete surface be intentionally roughened to an amplitude of 1/4 inch to improve the interface shear resistance.

$c := 0.075ksi$ — *Cohesion factor*

$\mu := 0.6$ — *Friction factor*

$K_1 := 0.2$ — *Fraction of concrete strength available to resist interface shear*

$K_2 := 0.8ksi$ — *Limiting interface shear resistance (ksi)*

Interface shear resistances

$P_c := 0\text{kip}$

Permanent net compressive force normal to the shear plane. Conservatively disregard compression force by self-weight of the upper wing at the interface.

$V_{ni1} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 218.2\ \text{kip}$

LRFD Equation 5.7.4.3-3

$V_{ni2} := K_1 \cdot f'_c \cdot A_{cv} = 1640.6\ \text{kip}$

LRFD Equation 5.7.4.3-4

$V_{ni3} := K_2 \cdot A_{cv} = 1640.6\ \text{kip}$

LRFD Equation 5.7.4.3-5

$V_{ni} := \min\left(V_{ni1}, V_{ni2}, V_{ni3}\right) = 218.2\ \text{kip}$

Nominal interface shear resistance over the length Lc

$V_{ri} := \phi_v \cdot V_{ni} = 218.2\ \text{kip}$

Factored interface shear resistance over the length Lc

<u>Parapet concrete shear strength in bending about its vertical axis</u>

Section 1

Upper portion between sections 1 and 2:

$$V_{w12} := 2 \cdot \left(\sqrt{\frac{f'_c}{psi}}\,psi\right) \cdot h_{12} \cdot d_{12} = 13.6\ \text{kip}$$

Shear strength of concrete (LRFD Eq. 5.7.3-3)

Lower portion between sections 2 and 3:

$$V_{w23} := 2 \cdot \left(\sqrt{\frac{f'_c}{psi}}\,psi\right) \cdot h_{23} \cdot d_{23} = 38.92\ \text{kip}$$

Shear strength of concrete (LRFD Eq. 5.7.3-3)

Total concrete shear resistance

$V_w := V_{w12} + V_{w23} = 52.6\ \text{kip}$

$\phi_v \cdot V_w = 52.6\ \text{kip}$

<u>Total shear resistance of parapet</u>

$V_r := V_{ri} + 2 \cdot \left(\phi_v \cdot V_w\right) = 323.4\ \text{kip}$

$V_u := F_t = 54 \cdot \text{kip}$

$$\frac{V_u}{V_r} = 0.17$$

D/C < 1: OK

Interface shear resistance and shear strentgh of the parapet are sufficient to resist the lateral load.

Parapet Resistance to Transverse Force For Impact At End of Wall or At Joint

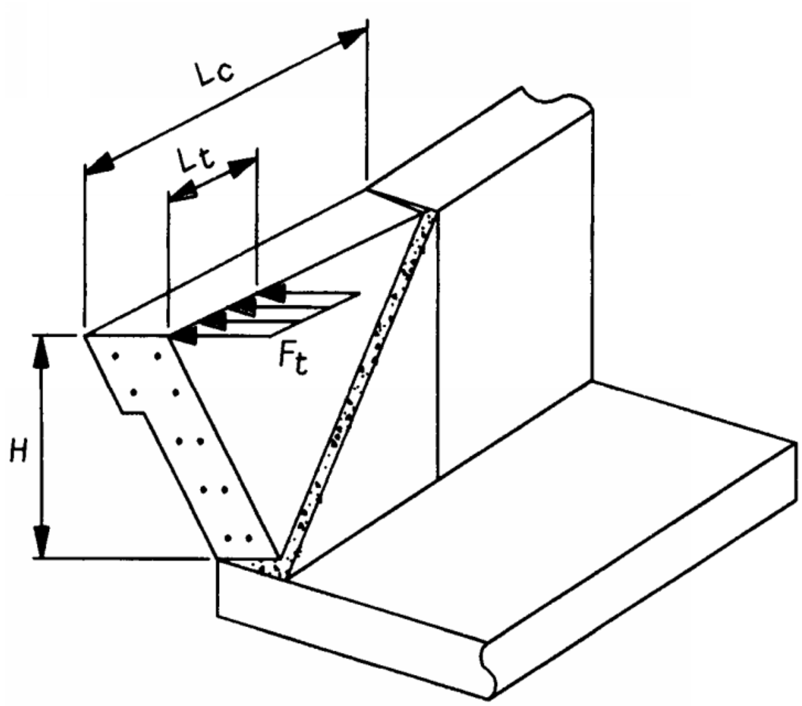


Figure CA13.3.1-2—Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

Parapet Resistance to Lateral Impact Load - End-of-Wall Segment (LRFD A13.3.1)

Parapet CIP vertical bars

$d_{bv} := d_{bv\_end} = 0.625\ \text{in}$ *Vertical bar diameter*

$s_{bv} := s_{bv\_end} = 12\ \text{in}$ *Vertical bar spacing*

$$A_{bv} := \frac{\pi\, d_{bv}^{\ 2}}{4} = 0.31\ \text{in}^2$$ *Vertical bar area*

$$A_{sv} := \frac{A_{bv} \cdot 12\text{in}}{s_{bv}} = 0.31 \cdot \text{in}^2$$ *Vertical bar area per linear foot of parapet*

1. Determine M_C : flexural resistance of cantilevered parapet about an axis parallel to the longitudinal axis of the bridge. Flexural moment resistance is based on the vertical reinforcement in the barrier.

Calculation for 1 ft length of parapet:

$b := 12in$

At section 1:

$$a_1 := \frac{A_{sv} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.45\ in$$

$$d_1 := w_1 - c_c - \frac{d_{bv}}{2} = 8.31\ in$$

$$M_{c1} := \phi \cdot A_{sv} \cdot f_y \cdot \left(d_1 - \frac{a_1}{2} \right) = 12.4\ ft \cdot kip$$

At section 2:

$$a_2 := \frac{A_{sv} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.45\ in$$

$$d_2 := w_2 - \left(c_c - \frac{d_{bv}}{2} \right) = 9.6\ in$$

$$M_{c2} := \phi \cdot A_{sv} \cdot f_y \cdot \left(d_2 - \frac{a_2}{2} \right) = 14.3\ ft \cdot kip$$

At section 3:

Flexural resistance of the parapet about its longitudinal axis at this section is determined based on tensile resistance of the adhesive anchor

$$a_3 := \frac{T_n}{0.85 \cdot f'_c \cdot b} = 0.25\ in$$

$$d_3 := w_3 - \left(c_c - \frac{d_a}{2} \right) = 14\ in$$

$$M_{c3} := T_r \cdot \left(d_3 - \frac{a_3}{2} \right) = 8.75\ ft \cdot kip$$

Calculate average Mc

$$M_c := \frac{\left(\frac{M_{c1} + M_{c2}}{2} \right) \cdot h_{12} + \left(\frac{M_{c2} + M_{c3}}{2} \right) \cdot h_{23}}{H} = 12.12\ ft \cdot kip$$

2. Determine M_w : flexural resistance of the parapet about its vertical axis.

$d_{bh} := d_{bh\_end} = 1\ in$ *Parapet horizontal bar diameter*

$A_{bh} := \frac{\pi \cdot d_{bh}^2}{4} = 0.8\ in^2$ *Parapet horizontal bar area*

Upper portion between sections 1 and 2:

$A_{sh} := 1 \cdot A_{bh} = 0.8\ in^2$

$a_{12} := \frac{A_{sh} \cdot f_y}{0.85 \cdot f'_c \cdot h_{12}} = 1\ in$

$w_{12} := \frac{w_1 + w_2}{2} = 10.9\ in$ *Average width of parapet*

$d_{12} := w_{12} - c_c - d_{bv} - \frac{d_{bh}}{2} = 7.8\ in$

$M_{w12} := \phi \cdot A_{sh} \cdot f_y \cdot \left(d_{12} - \frac{a_{12}}{2}\right) = 28.6\ ft \cdot kip$

Lower portion between sections 2 and 3:

$A_s := 3 \cdot A_{bh} = 2.4\ in^2$

$a_{23} := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot h_{23}} = 1.5\ in$

$w_{23} := \frac{w_2 + w_3}{2} = 13.5\ in$ *Average width of parapet*

$d_{23} := w_{23} - c_c - d_a - \frac{d_{bh}}{2} = 10.5\ in$

$M_{w23} := \phi \cdot A_s \cdot f_y \cdot \left(d_{23} - \frac{a_{23}}{2}\right) = 115.1\ ft \cdot kip$

Total Mw

$M_w := M_{w12} + M_{w23} = 143.8\ ft \cdot kip$

3. Rail resistance at end of wall segment.

$M_b := 0 ft \cdot kip$ *There is no beam on top of parapet*

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{H\cdot(M_b + M_w)}{\frac{M_c}{1ft}}} = 8.43\cdot ft$$

Critical length of yield line failure pattern, LRFD Eq. A13.3.1-4

$$R_w := \left(\frac{2}{2\cdot L_c - L_t}\right)\cdot\left[M_b + M_w + \frac{\left(\frac{M_c}{1ft}\right)L_c^{\ 2}}{H}\right] = 58.36\ kip$$

Railing resistance to transverse load, LRFD Eq. A13.3.1-3

Check railing demand/capacity

$$\frac{F_t}{R_w} = 0.93$$

D/C <1: OK

Shear Resistance

It is demonstrated in this section that the lateral impact load can be resisted by the interface shear resistance between the parapet and bridge deck and by the shear strentgh of the parapet in bending about the vertical axis. Thus, the adhesive anchors are not to be loaded in shear, and reduction in tensile strength of adhesive anchors due to tension-shear interaction per ACI 318, 17.8, does not need to be considered.

Interface shear resistance , LRFD 5.7.4.3

Calculate interface shear resistance between the parapet and bridge deck

$b_{vi} := 15in$

Interface width considered to be engaged in shear transfer

$L_{vi} := L_c = 8.4\cdot ft$

Interface length considered to be engaged in shear transfer

$A_{cv} := b_{vi}\cdot L_{vi} = 1517\ in^2$

Interface area considered to be engaged in shear transfer

$A_{vf} := A_{se\_N}\cdot\frac{L_c}{s_a} = 1.32\ in^2$

Cross-sectional area of steel reinforcement within the interface area

Cohesion and friction factors, LRFD 5.7.4.4:

For calculations in this example, it was assumed concrete placed against a clean concrete surface, free of laitance, with surface not intentionally roughened. In practice, it is advisable that the concrete surface be intentionally roughened to an amplitude of 1/4 inch to improve the interface shear resistance.

$c := 0.075ksi$

Cohesion factor

$\mu := 0.6$ *Friction factor*

$K_1 := 0.2$ *Fraction of concrete strength available to resist interface shear*

$K_2 := 0.8\text{ksi}$ *Limiting interface shear resistance (ksi)*

Interface shear resistances

$P_c := 0\text{kip}$ *Permanent net compressive force normal to the shear plane. Conservatively disregard compression force by self-weight of the upper wing at the interface.*

$V_{ni1} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 161.4\,\text{kip}$ *LRFD Equation 5.7.4.3-3*

$V_{ni2} := K_1 \cdot f'_c \cdot A_{cv} = 1213.6\,\text{kip}$ *LRFD Equation 5.7.4.3-4*

$V_{ni3} := K_2 \cdot A_{cv} = 1213.6\,\text{kip}$ *LRFD Equation 5.7.4.3-5*

$V_{ni} := \min\left(V_{ni1}, V_{ni2}, V_{ni3}\right) = 161.4\,\text{kip}$ *Nominal interface shear resistance over the length Lc*

$V_{ri} := \phi_v \cdot V_{ni} = 161.4\,\text{kip}$ *Factored interface shear resistance over the length Lc*

Parapet concrete shear strength in bending about its vertical axis

Section 1

Upper portion between sections 1 and 2:

$$V_{w12} := 2 \cdot \left(\sqrt{\frac{f'_c}{psi}}\,psi\right) \cdot h_{12} \cdot d_{12} = 13.2\,\text{kip}$$

Lower portion between sections 2 and 3:

$$V_{w23} := 2 \cdot \left(\sqrt{\frac{f'_c}{psi}}\,psi\right) \cdot h_{23} \cdot d_{23} = 38\,\text{kip}$$

Total concrete shear strength

$V_w := V_{w12} + V_{w23} = 51.2\,\text{kip}$

$\phi_v \cdot V_w = 51.2\,\text{kip}$

Total shear strength of parapet

$V_r := V_{ri} + (\phi_v \cdot V_w) = 212.7\ \text{kip}$

$V_u := F_t = 54 \cdot \text{kip}$

$\frac{V_u}{V_r} = 0.25$ *D/C < 1: OK*

Interface shear resistance and shear strentgh of the parapet are sufficient to resist the lateral load.

APPENDIX D. LABORATORY TESTING

Background

Wingwall test samples were prepared to understand the performance of adhesive anchors installed in a structure and exposed to a simulated load as would be experienced in an actual wingwall. WisDOT policies on adhesive anchors were used in preparing the test samples. Adhesive anchor performance is compared to current design code equations.

Test Sample Description

Testing was conducted to simulate an upper wingwall replacement to an existing lower wingwall to evaluate performance of epoxy coated reinforcing bars adhesively anchored into the lower wingwall and cast into the upper wingwall. Two test samples were fabricated to represent an upper wingwall replacement on a lower wingwall.

The lower wingwall measured 40-in deep by 80-in wide by 60-in tall. A total of sixteen (16) No. 6 rebars were positioned within the perimeter with a 12-in spacing and approximately 2-in clear cover. Stirrups fabricated from No. 5 reinforcing bar were spaced at 12-in centers along the 80-in length. The concrete had an average compressive strength of 5110 psi at the time of testing.

The upper wingwall measured 15-in deep by 80-in wide by 60-in tall. A total of eight (8) No. 4 reinforcing bars spaced at 8 inches were positioned longitudinally at the back face and five (5) No. 4 reinforcing bars spaced at 16 inches were positioned longitudinally at the front face. A total of nine (9) No. 5 U-shaped stirrups spaced at 9 inches were positioned along the upper wingwall.

The upper wingwall was anchored to the lower wingwall with a total of ten (10) No. 6 epoxy coated reinforcing bars with five (5) along the back face and five (5) along the front face. The No. 6 reinforcing bar were adhesively anchored 15-in deep into the lower wingwall and spaced at 16-in centers with a clear cover of 3 inches on the back face and 4 inches at the front face of the upper wingwall. The embedment depth of 15 inches is twenty (20) times the nominal diameter of a No. 6 reinforcing bar, which is the stated embedment depth requirement in Section 40.16.1.1 of the WisDOT Bridge Manual. The upper wingwalls had average compressive strength of 2025 psi at the time of testing. Drawings of a test wall are shown in Figure D.1.

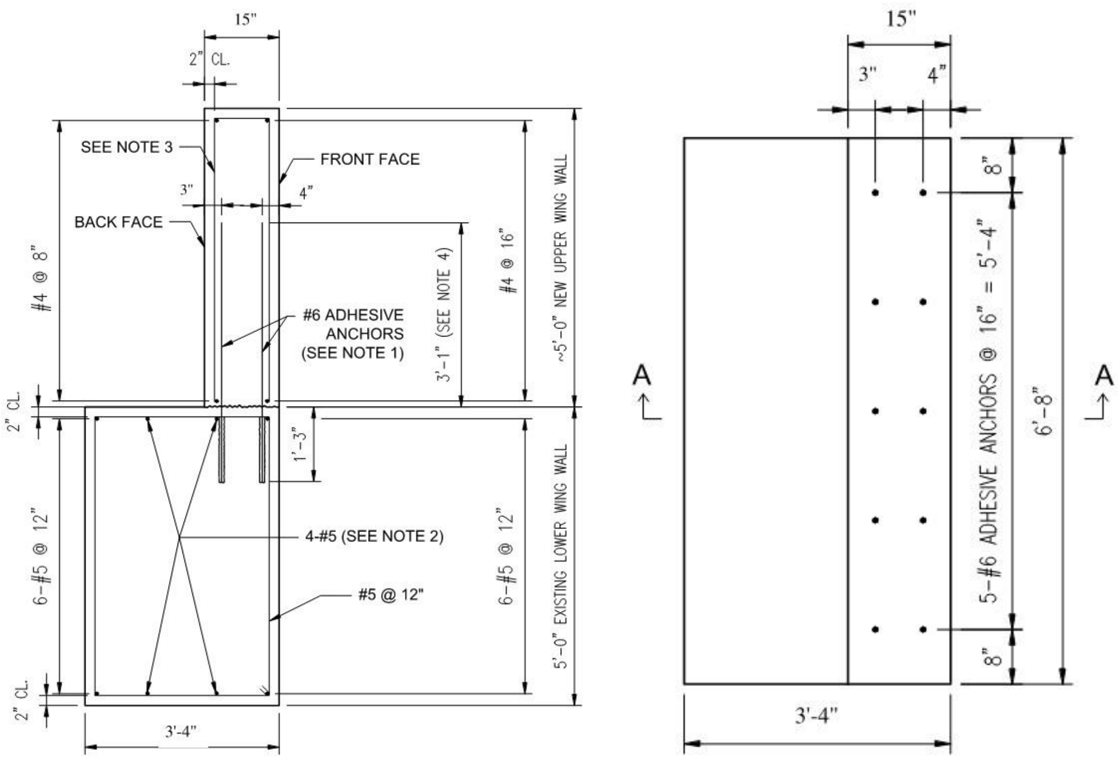


Figure D.1. Elevation (left drawing) and plan view (right drawing) of wingwall test sample.

The No. 6 reinforcing bar were anchored 15-in deep using Simpson Strong-Tie AT-XP adhesive. This adhesive was selected because it is listed with the lowest characteristic bond strength for uncracked concrete among those adhesives on the approved WisDOT list. Installation of the reinforcing bar followed the manufacturer's printed installation instructions, which consisted of blowing the drill cuttings from the bottom of the hole, brushing the hole with the specified Simpson Strong-Tie brush, and blowing the drill cuttings from the bottom of the hole brushed from the drill hole wall. The concrete surface of the lower wingwall was roughened to an approximate amplitude of ¼-in. in accordance with ACI 318-19, Table 22.9.4.2 prior to casting the upper wingwall.

Test Configuration

Once the upper wingwall concrete cured for a minimum of 28 days the samples were positioned for testing such that the upper wingwall was horizontal. A schematic drawing of the test configuration is shown in Figure D.2. Wood bracing was used to support the upper wingwall while moving the test sample into position for testing. The wood bracing was disengaged before testing began.

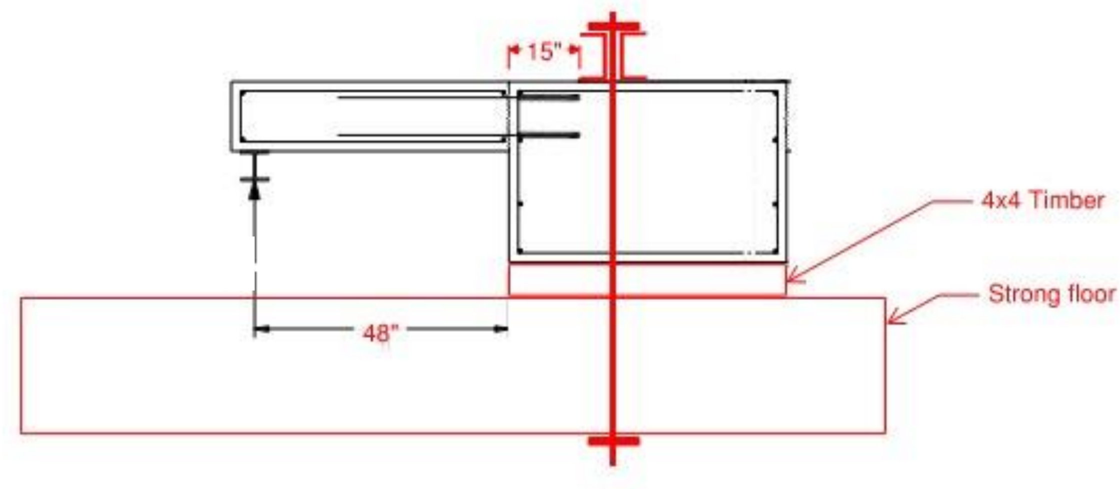


Figure D.2. Schematic drawing of test setup

Hydraulic rams were used to apply load to the upper wingwall while the lower wingwall was restrained using a beam and threaded rods anchored to the laboratory strong floor. The hydraulic rams were positioned 48 inches from the joint between the upper and lower wingwalls. The beam restraining the lower wingwall was positioned beyond the 15 in. embedment depth of the adhesive anchor so as not to confine concrete in the area of the adhesively anchored reinforcing bars.

Instrumentation was installed to collect data during testing and consisted of a pressure transducer, cable extension transducers (CET), and strain gages. The pressure transducer was in line with a hydraulic pump used for the hydraulic rams and was used to measure the applied load based on the ram area. The CETs were positioned across the wingwall joint and at the elevation of the back face adhesively anchored reinforcement (Figure D.3Figure D.3). Strain gages were installed on the tension side of the adhesively anchored reinforcement (Figure D.4) approximately 2 inches above the concrete surface of the lower wingwall (Figure D.5) The instrumentation was connected to a computer-controlled data acquisition system that continuously recorded and displayed their output (Figure D.6 and Figure D.7).

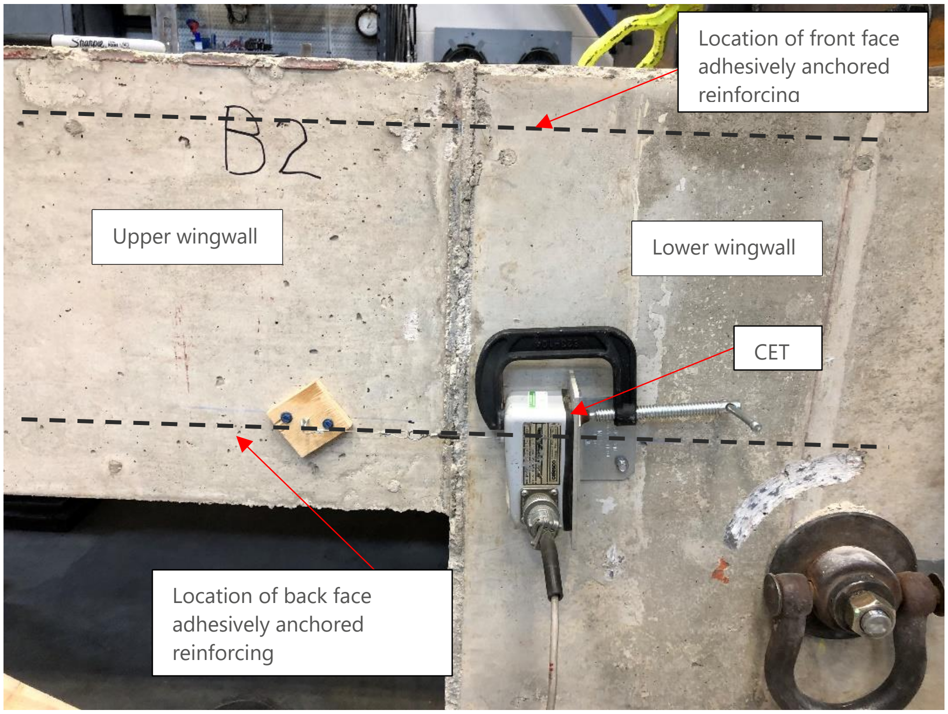


Figure D.3. CET positioned across wingwall joint (dashed lines indicate location of adhesively anchored reinforcing steel)

Figure D.4. Strain gage installed on reinforcement adhesively anchored into lower wingwall.

Figure D.5. Photo showing location of strain gages (red arrows) on reinforcement anchored into lower wingwall prior to casting of upper wingwall concrete.

Figure D.6. Test setup

Figure D.7. Test setup

Test Performance

Testing was conducted on two samples designated as B1 and B2. Load application consisted of using an electric hydraulic pump and two rams positioned 48 inches from the wingwall joint and approximately 26½ inches apart. Load was applied monotonically, load-displacement and strain data were monitored, visual observations were made periodically, and notable events were documented for each test. Load application was discontinued when an increase in load could not be achieved.

Test Results

The test orientation of the upper wingwall required that its gravity self-weight be subtracted from the measured maximum applied load to determine the bending moment and shear force at the joint between the upper and lower wingwalls. The performance of both test walls was similar in that the ultimate load values, and load-displacement and load-reinforcing bar strain characteristics were similar. The maximum load achieved for Wall B1 was 49,700 lbf and for Wall B2 was 50,610 lbf. The failure mode for both wingwall samples was yielding of the tension reinforcing steel (back face reinforcing) and concrete cracking/crushing. The condition of the wingwalls at approximate maximum applied load for Test Samples B1 and B2 is shown in Figure D.8 and Figure D.9, respectively.

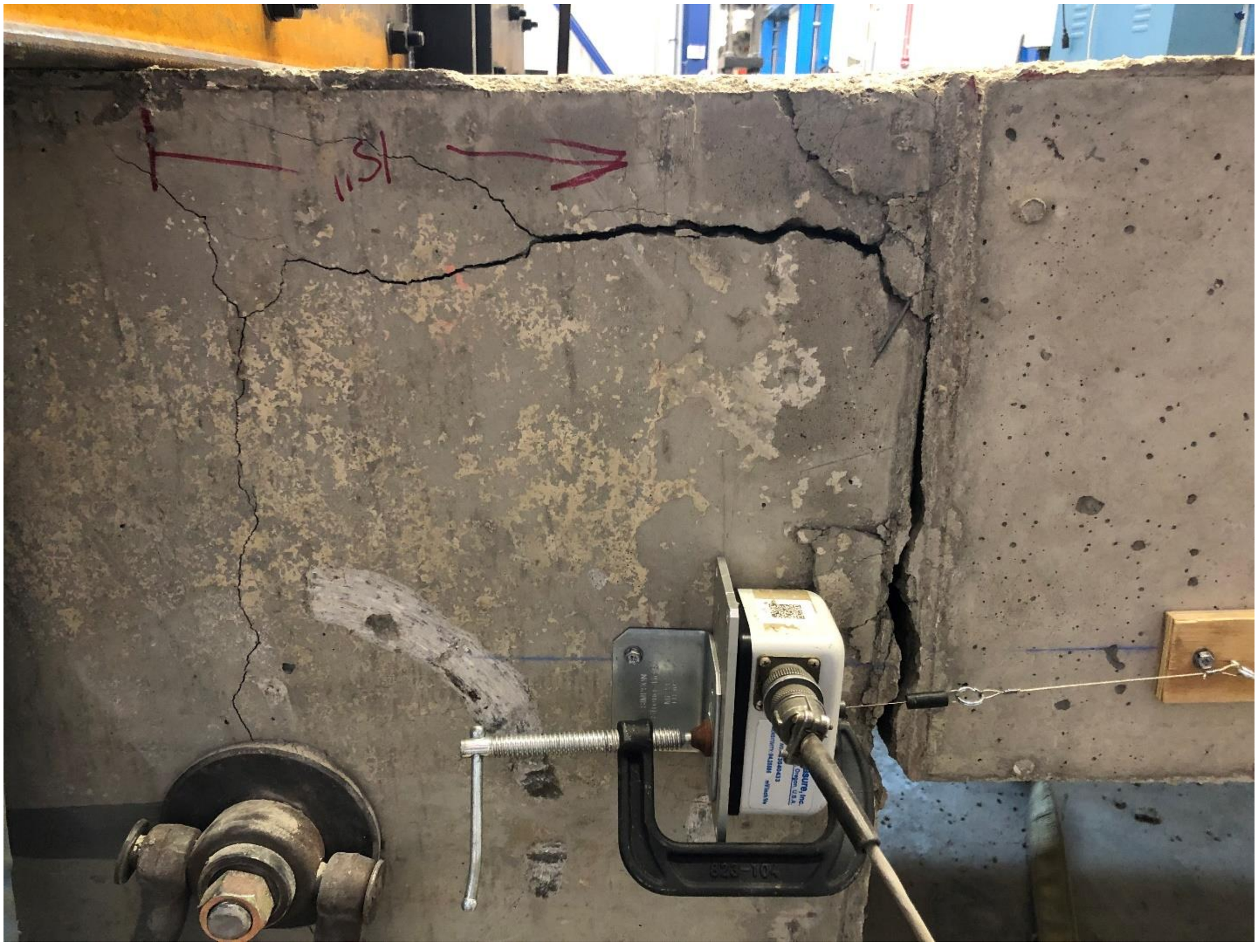


Figure D.8. Test Sample B1

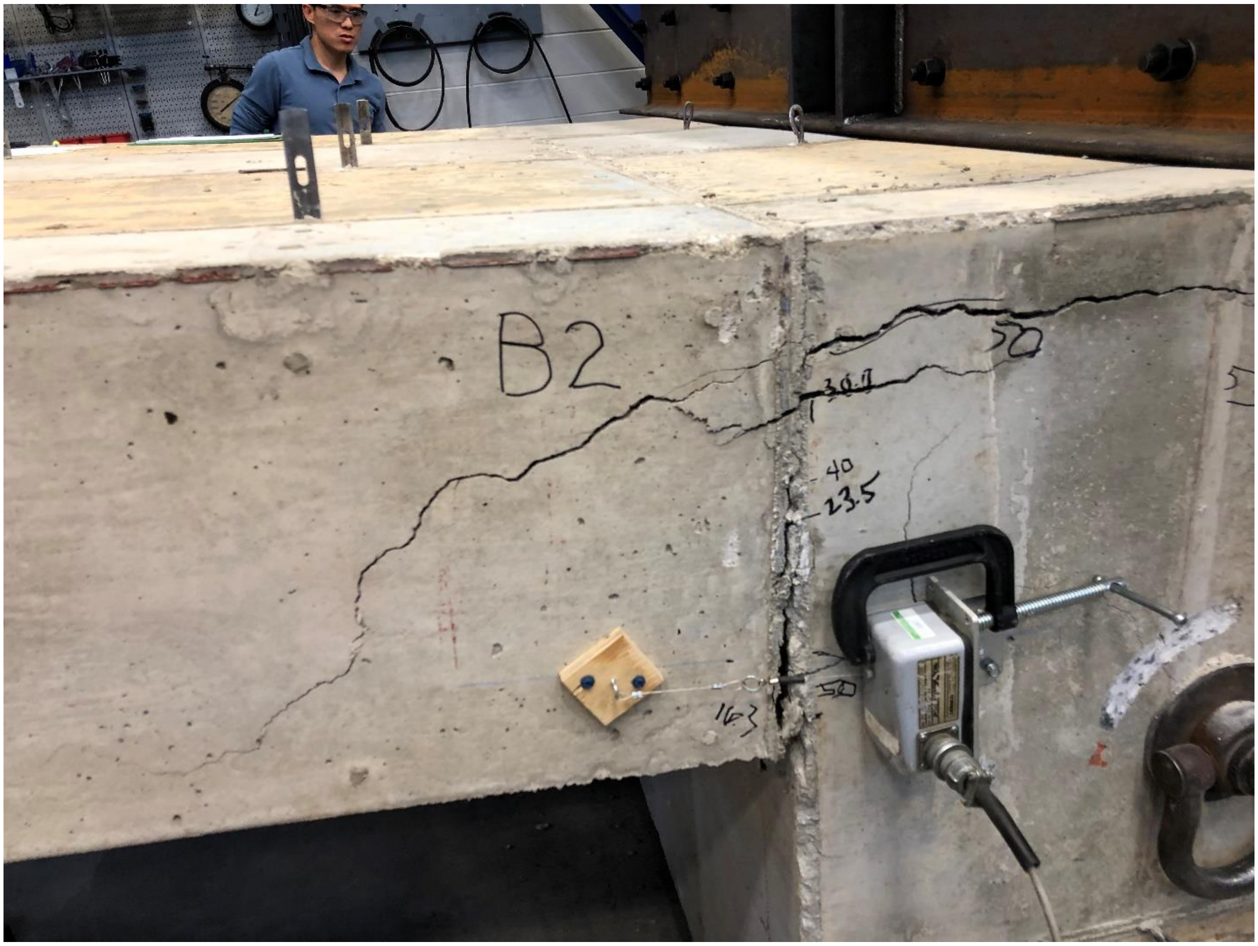


Figure D.9. Test Sample B2

The load-displacement plots show three distinct regions of behavior (Figure D.10 and Figure D.11). The first region is between zero and approximately 13,800 lbf. This region showed linear behavior with little to no displacement and no observed distress in the concrete. The second region is between 13,800 lbf and approximately 48,000 lbf for Wall B1 and 52,000 lbf for wall B2. This region is where concrete cracking began and wingwall joint opened to 0.065-in for Wall B1 and 0.070-in for Wall B2. The third region is where an increase is displacement occurs with little to no increase in load occurred. This region is where concrete cracking continued and reinforcing bar yielding was occurring.

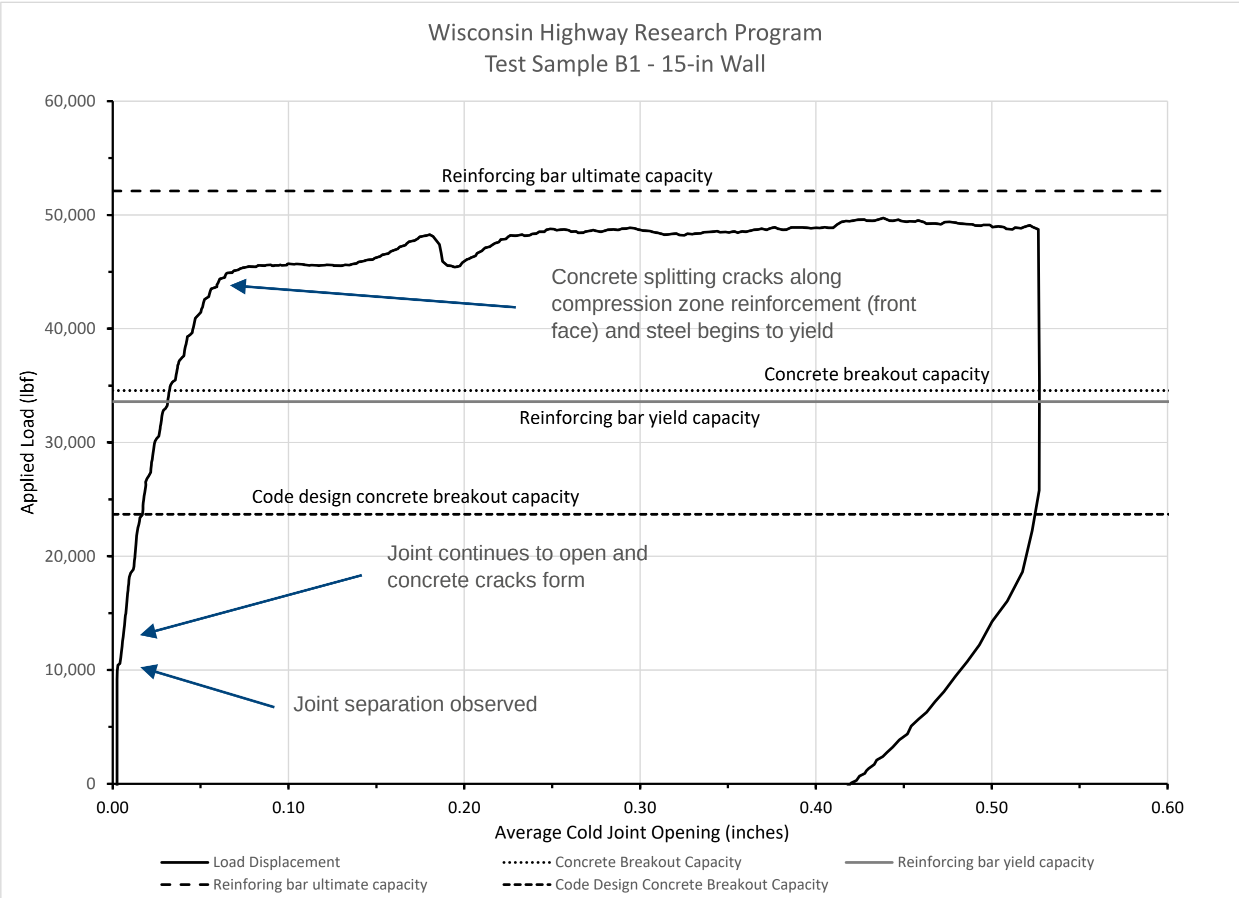


Figure D.10. Load-displacement plot for Wall B1

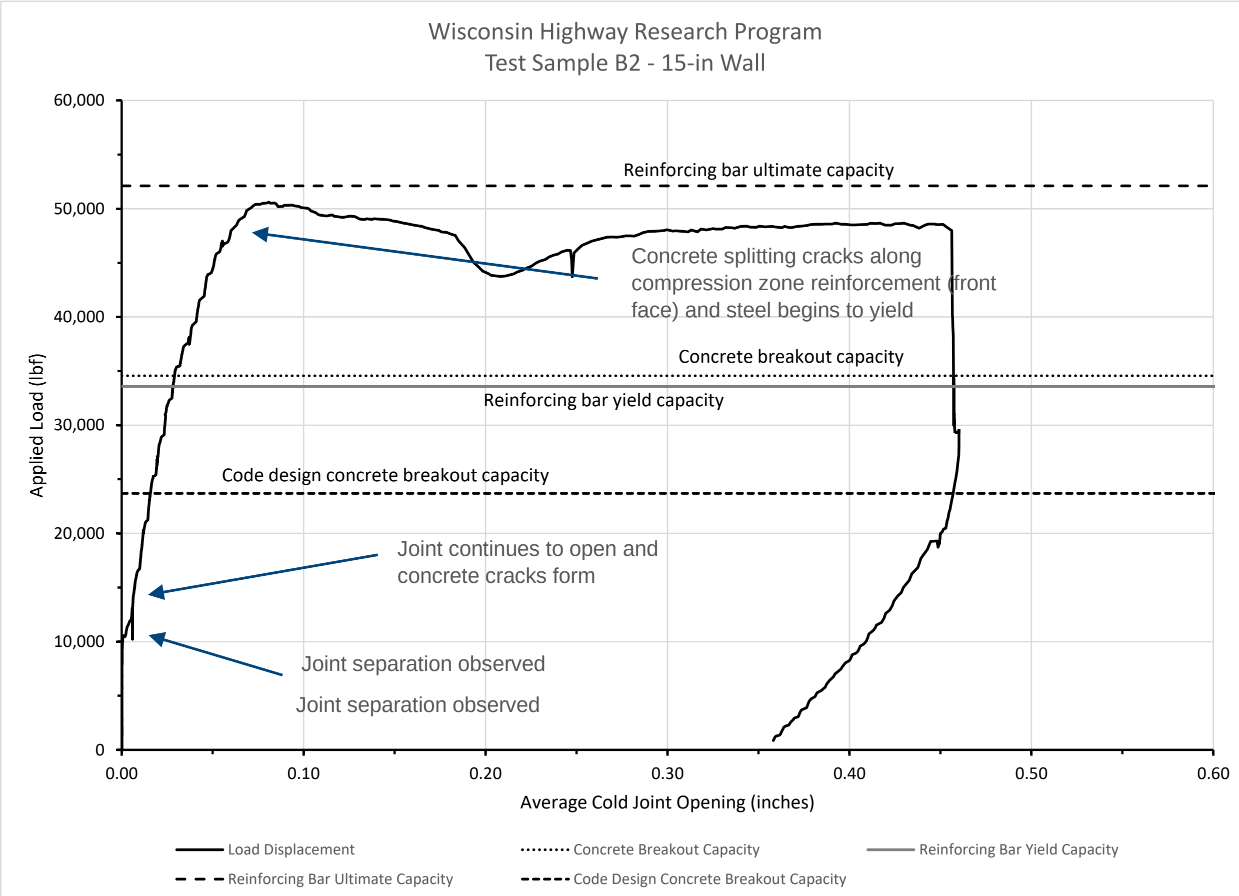


Figure D.11. Load-displacement plot for Wall B2

The reinforcement strain data plot in Figure D.11 indicates performance as anticipated. The back face reinforcement for both wall samples were in tension throughout the test. The front face reinforcement was in compression until the wingwall joint crack became so large as to place this reinforcement in tension. Both the back face and front face reinforcement yielded as the load was increased and the maximum applied load was approached. The load-strain relationship for each wall sample is shown in Figure D.11 through Figure D.14. It is noted that at different load levels some strain gages stopped functioning because they debonded from the steel or the wire connections failed prior to or during yielding.

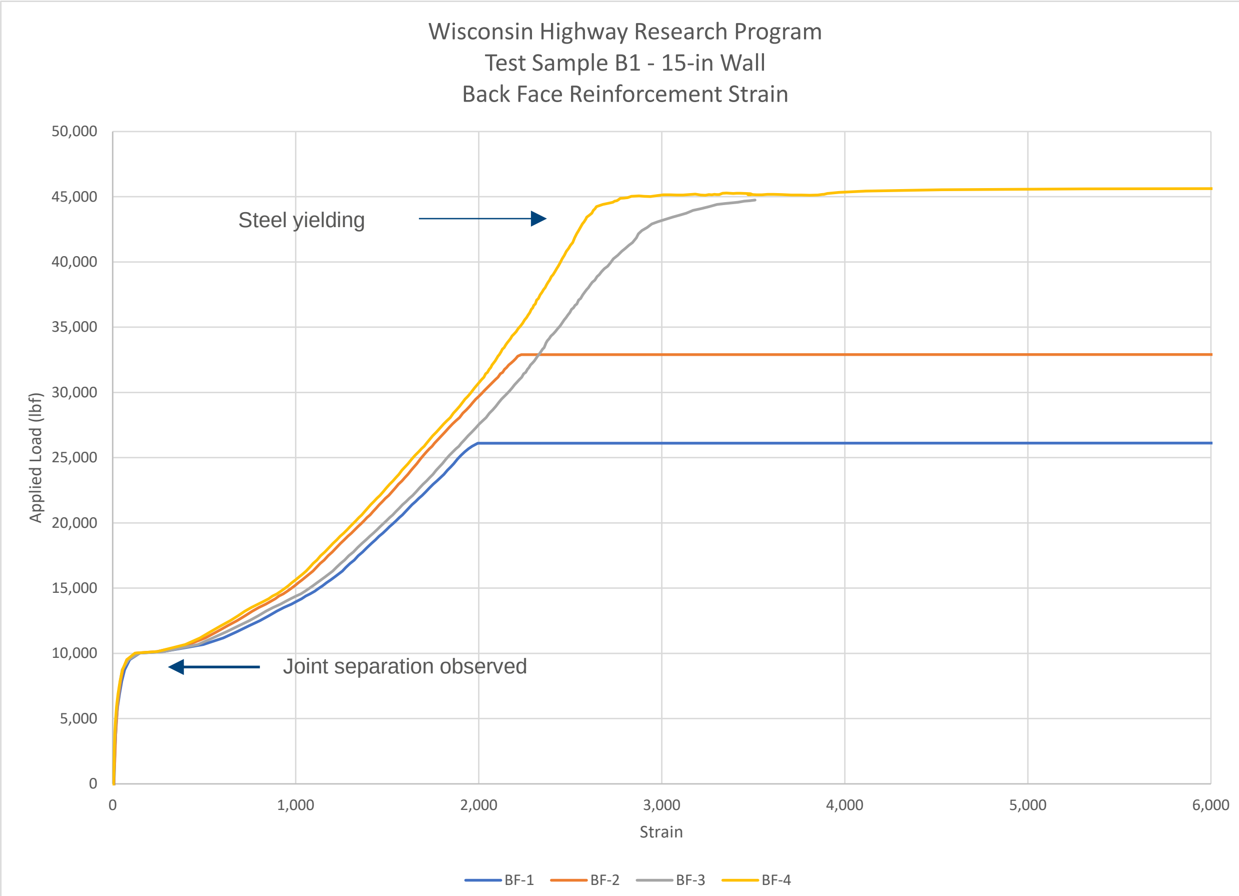


Figure D.12. Wall B1 load-strain plot for back face reinforcement

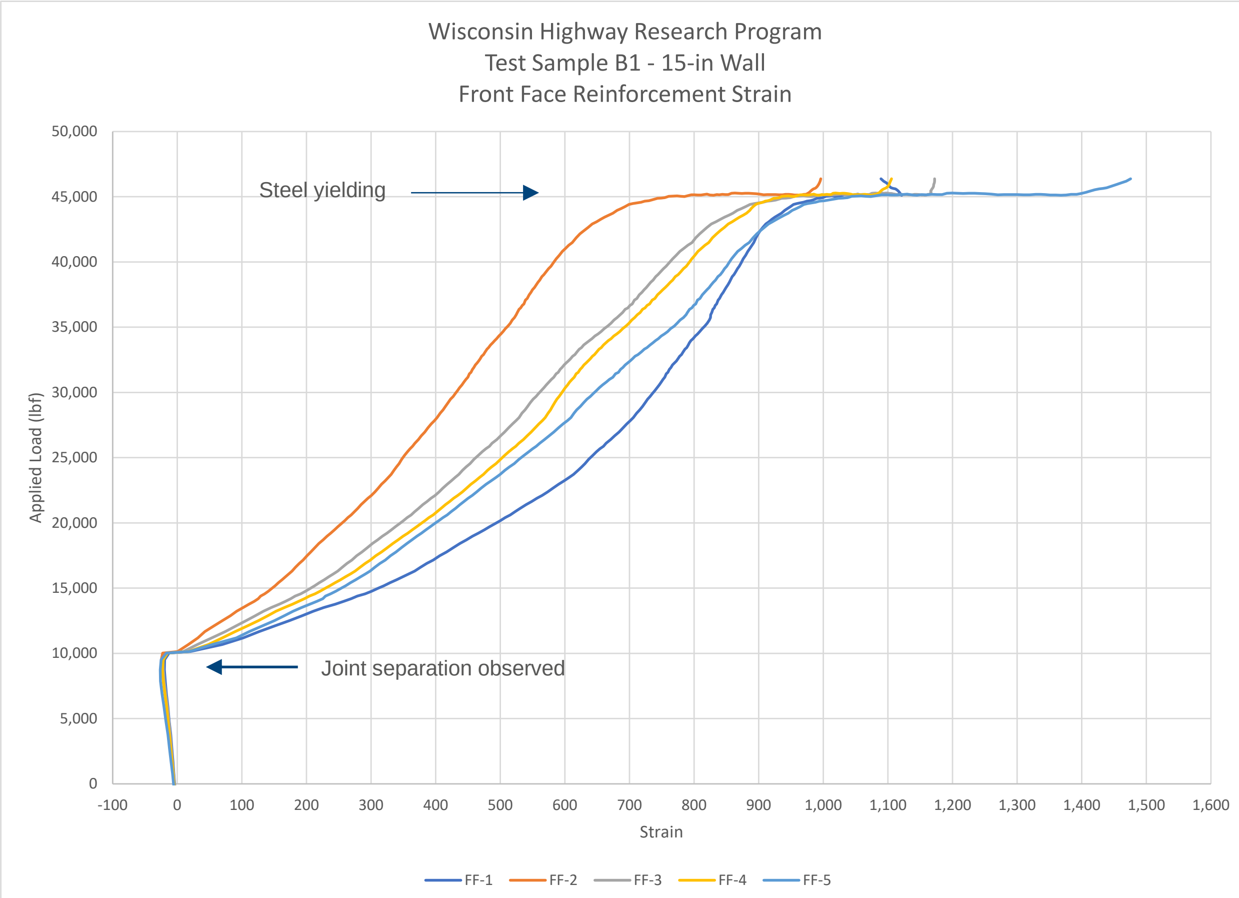


Figure D.13. Wall B1 load-strain plot for front face reinforcement

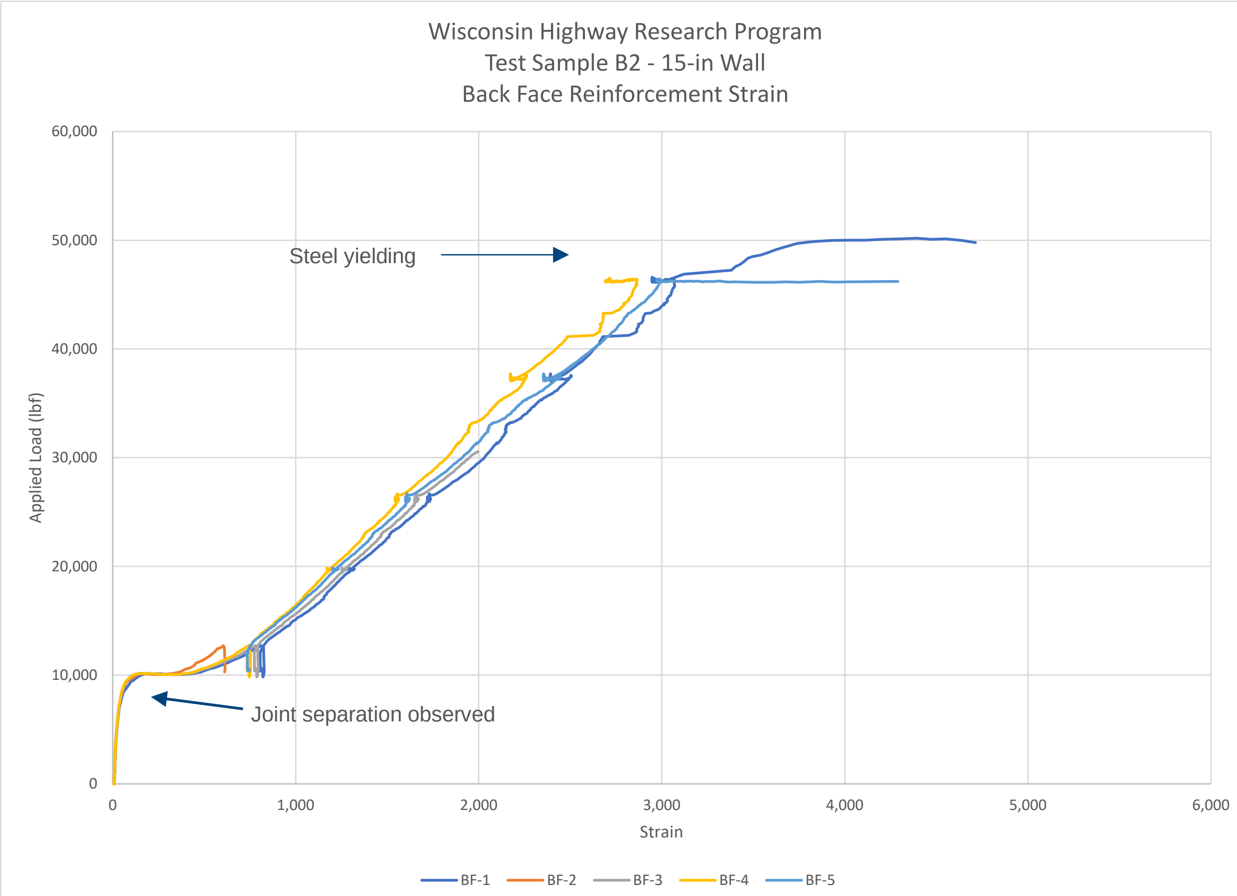


Figure D.14. Wall B2 load-strain plot for back face reinforcement

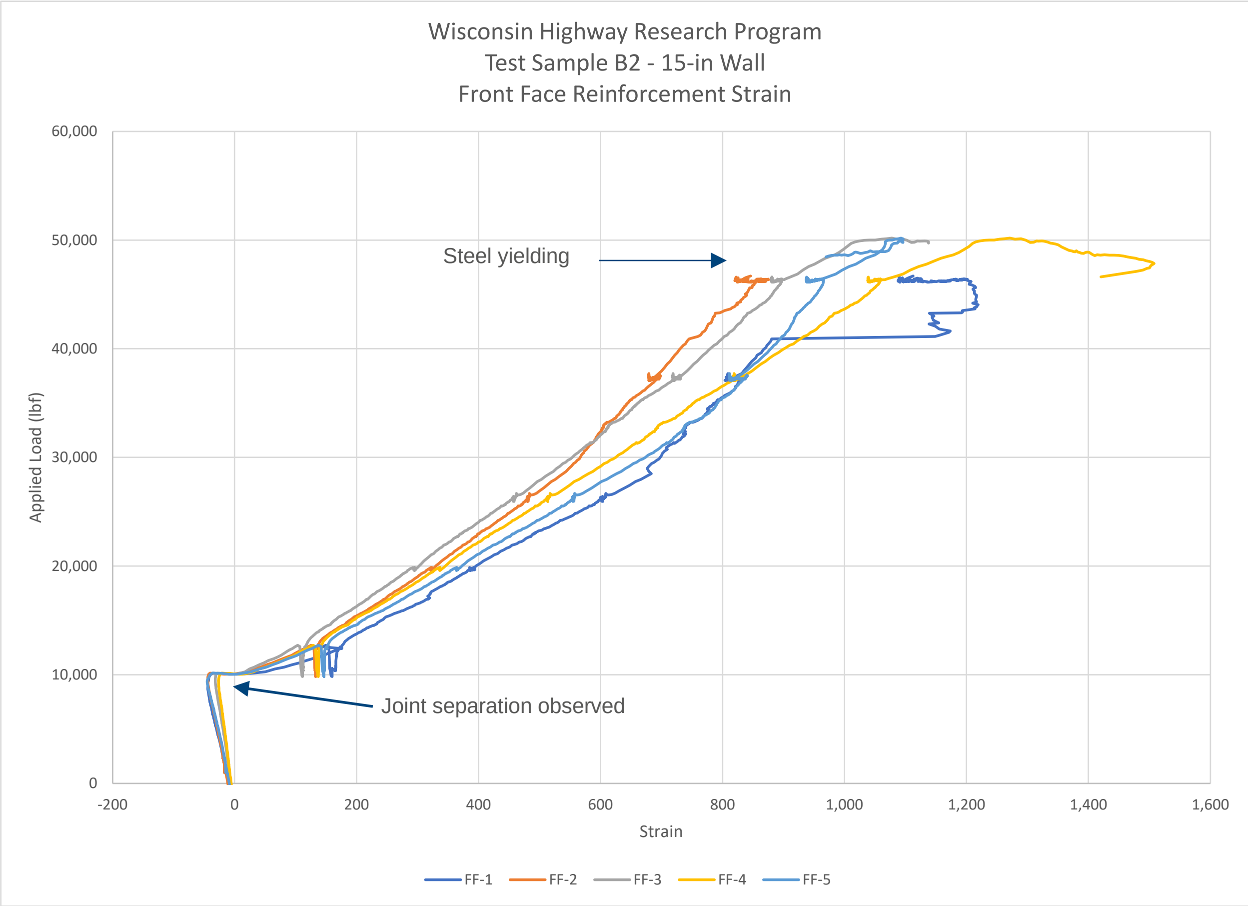


Figure D.15. Wall B2 load-strain plot for front face reinforcement

Wall Capacity Calculations

Calculations were made based on ACI design equations without using strength reduction factors, ϕ, to determine the anticipated failure mode of the adhesively anchored reinforcement. The failure modes considered were concrete breakout, reinforcing steel yield and fracture, and bond failure. The controlling calculated failure mode was concrete breakout. The calculations for these failure modes are included. Calculations were also made for a conventionally reinforced wall and are also included in this appendix. It is noted that ACI 318-19, Section R17.6.5.1 states the strength in tension of adhesive anchors is limited by concrete breakout strength as given by Equations 17.6.2.1a and 17.6.2.1b. ACI 318-19, Section R17.6.5.1 further states "The tensile strength of closely spaced adhesive anchors with low bond strength may significantly exceed the value given by Eq. (17.6.5.1b). A correction factor is given in the literature (Eligehausen et al. 2006a) to address this issue, but for simplicity, this factor is not included in the Code." This statement seems to hold true based on the test results and the calculated capacity for concrete breakout.

Based on a concrete breakout failure mode, the anticipated maximum test load was 23,700-lbf at a loading distance of 48-in from the wingwall joint. The anticipated maximum test moment was 94,800 lbf-ft.

The capacity of a conventionally reinforced upper wingwall cast against a lower wingwall using concrete strengths achieved for the test walls and the mill certificate reinforcing bar yield strength (f_y = 68.7 ksi) was also determined. The capacity for a conventionally reinforced wingwall based on reinforcing steel yielding is 33,600-lbf. The moment capacity is 134,400 lbf-ft.

The calculated load values for concrete breakout and conventionally reinforced wingwall based on reinforcing steel yield strength are shown on the load-displacement plots in Figure D.9 and Figure D.10.

The failure mode for the 15-in wingwalls was yielding of the tension reinforcing steel (back face reinforcing) and concrete cracking/crushing. Upper wingwall concrete was removed to examine the lower wingwall concrete at the adhesively anchored reinforcing steel areas for each test sample. Concrete cracks were observed that extended between anchors in the back face and front face rows (Figure D.15 and Figure D.16). Indications of concrete breakout or bond failure manifesting in anchor movement beyond the lower wingwall surface were not observed.

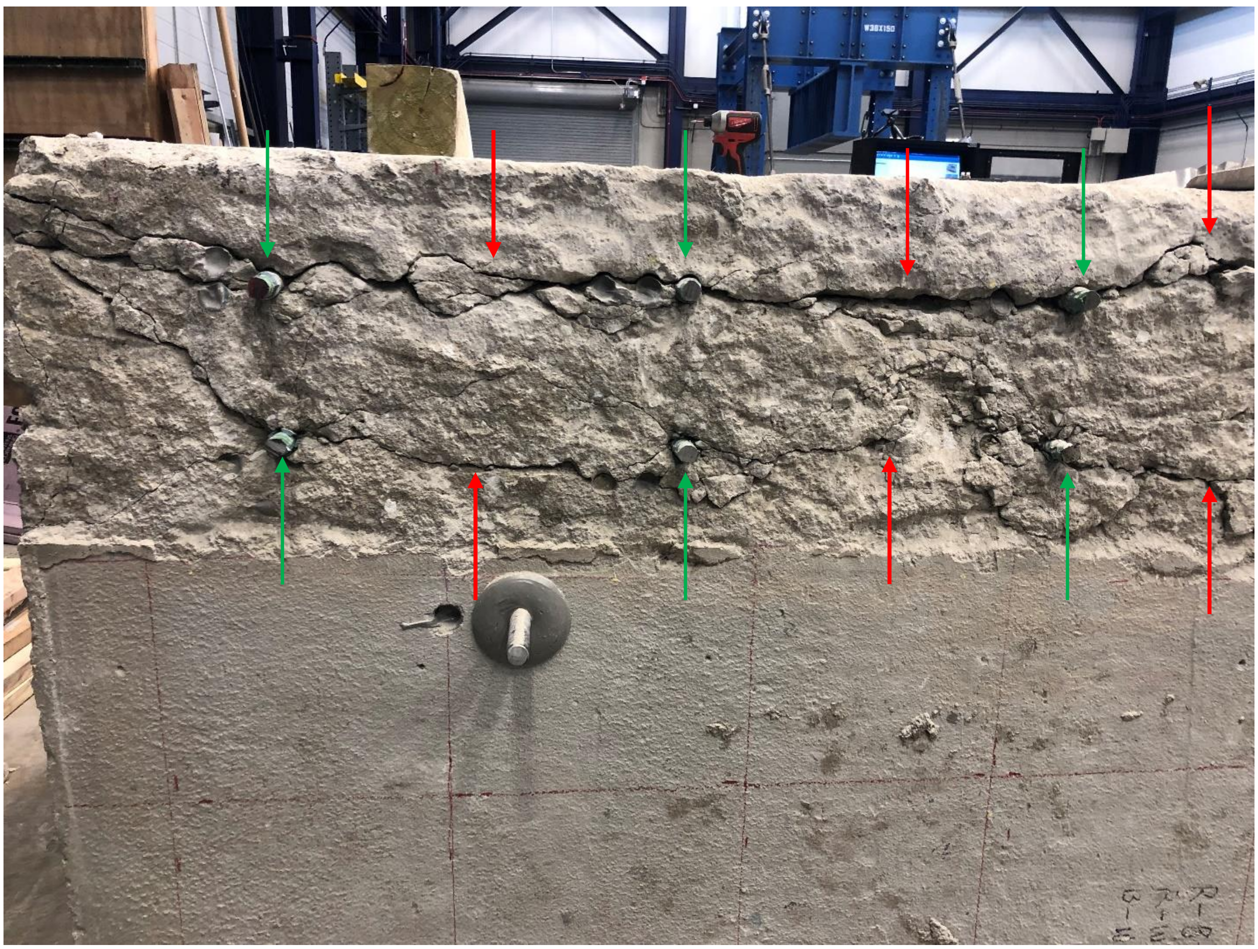


Figure D.16. Test Sample B1. Green arrows point to the reinforcement anchors. Red arrows point to cracks between a row of anchors.

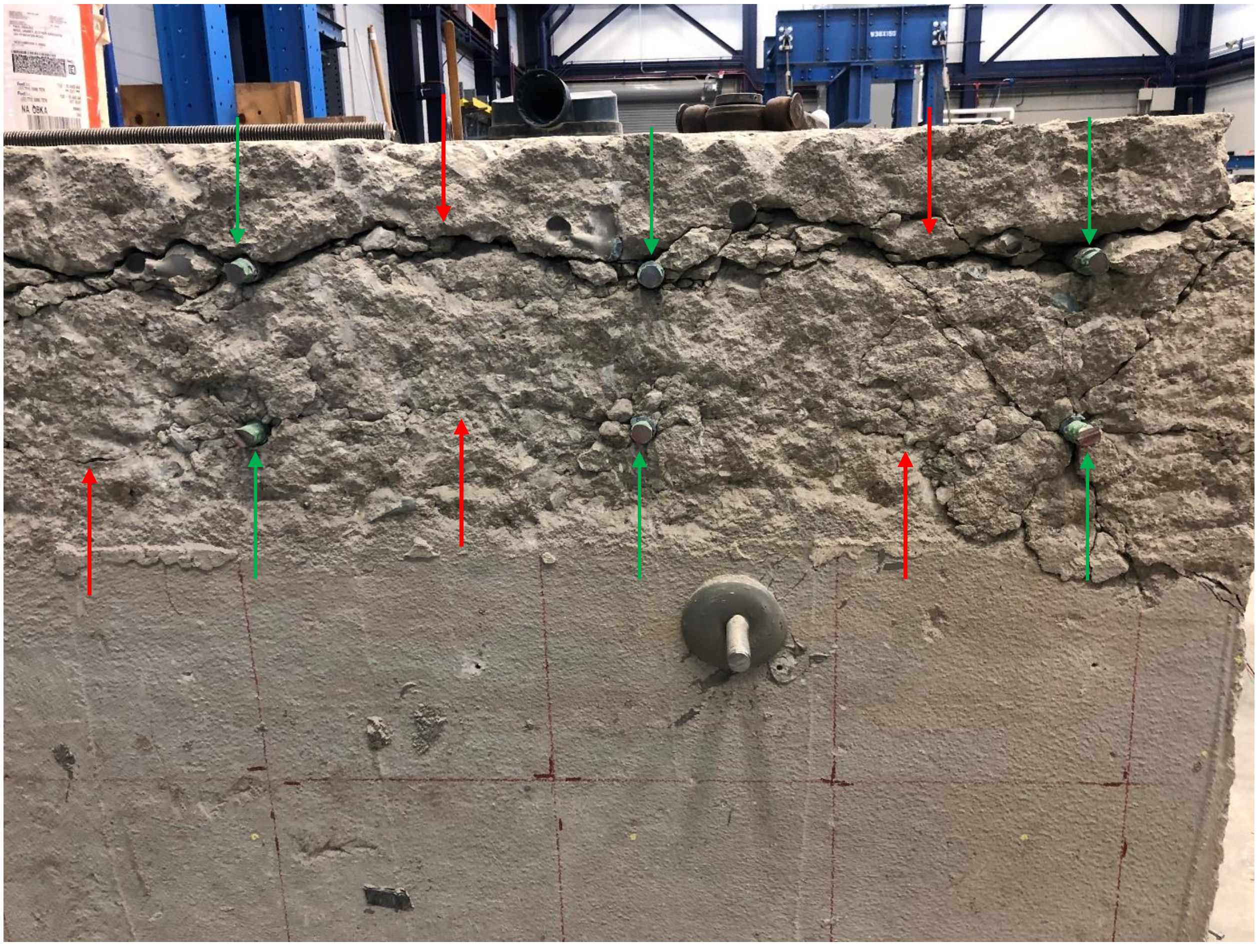


Figure D.17. Test Sample B2. Green arrows point to the reinforcement anchors. Red arrows point to cracks between a row of anchors.

Based on the visual examination of the lower wingwalls surface, it appears bending of the front face anchors contributed to concrete cracking. A similar occurrence with bending of the back face anchors but to a lesser degree because of a larger amount of concrete cover in the direction of bending.

Other observations made regarding the 15-in wingwall tests:

- The embedment depth for the adhesively anchored No. 6 reinforcing bar was 20 times the anchor diameter (15-in) per Chapter 40 of the Wisconsin Bridge Design Manual. This embedment depth is less than the calculated development length for a No. 6 epoxy coated reinforcing bar. This implies the bond strength of the adhesive used (Simpson AT-XP, which had the lowest listed bond strength) can develop the yield strength of a No. 6 epoxy coated reinforcing bar at 15-in embedment when installed in dry, uncracked concrete.
- The failure mode observed during the 15-in wingwall tests are the same as would be expected when designing cast-in-place reinforced concrete.
- Concrete breakout was the controlling failure mode based on ACI 318-19 design equations. The maximum load achieved was approximately 210 percent of the calculated code design concrete breakout strength.
- The maximum load achieved for each test sample was approximately 150 percent of the calculated flexural yield capcity of a similar conventionally reinforced wingwall configuration.

CALCULATIONS

CALCULATIONS OF WINGWALL TEST SPECIMEN

General Information

Calculate tensile and shear strengths of adhesive anchor for wingwall replacement.

References

- AASHTO LRFD-9th Edition
- ACI 318-19 Building Code Requirements for Structural Concrete and Commentary
- WisDOT Bridge Manual 2020

Note: all referenced sections in the calculations refer to WisDOT Bridge Manual 2020 unless otherwise noted.

Legend:

Input

Design Results/Check

Design Parameters

Concrete and Steel Anchor Properties

$w_c := 150pcf$	Unit weight of concrete
$f'_c := 5110psi$	Concrete compressive strength of lower wall
$f_{ua} := 106.6ksi$	Steel anchor tensile strength (from mill certs)
$f_{ya} := 68.7ksi$	Steel anchor yield strength (from mill certs)
$E_s := 29000ksi$	

Characteristic Bond Stress

$\tau_{uncr} := 955psi$	Simpson AT-XP for #6 anchors
$\tau_{cr} := 790psi$	

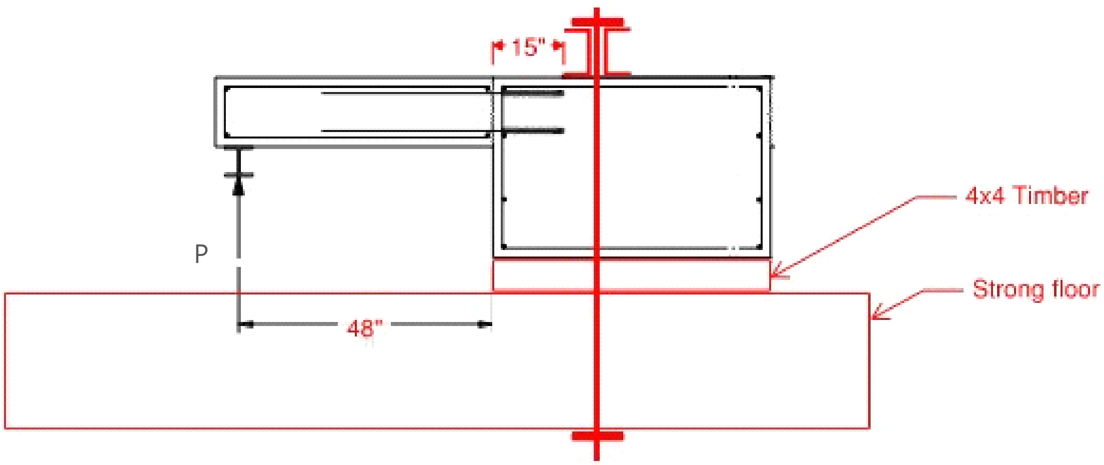
Resistance Factors, LRFD 5.5.4.2

$\phi_{ts} := 0.75$	Strength reduction factor for steel in tension for ductile steel, 40.16.2. Rebars are considered ductile.
$\phi_{tc} := 0.75$	Strength reduction factor for concrete breakout and bond in tension with supplementary reinforcement, 40.16.3 & ACI 318 17.5.3 Anchor Category. Assuming deck reinforcement functions as supplementary reinforcement for adhesive anchors.
$\phi_{vs} := 0.65$	Strength reduction factor for steel in tension for ductile steel, 40.16.2 & ACI 318 17.5.3. Rebars are considered ductile.
$\phi_{vc} := 0.75$	Strength reduction factor for concrete breakout in shear with supplementary reinforcement, 40.16.4 & ACI 318 17.5.3.

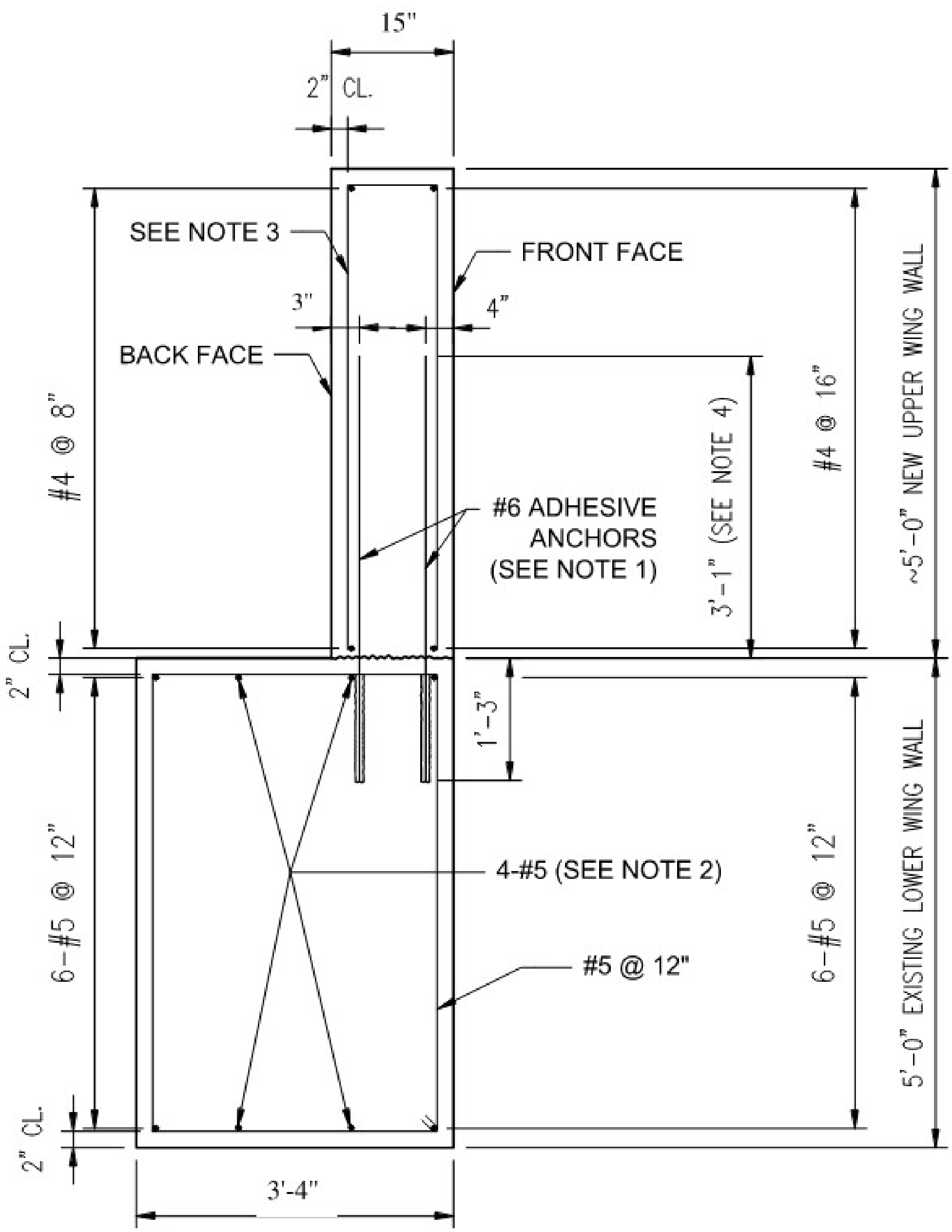
$\phi_{vp} := 0.70$

Strength reduction factor for concrete pryout in shear with supplementary reinforcement, 40.16.4 & ACI 318 17.5.3.

Geometry



Schematic of Test Setup



Elevation Cross-Section of Test Sample

$B_U := 15in$

$H_U := 5ft$

$H_L := 5ft$

$h_a := H_L = 60\ in$ Concrete thickness

$B_L := 40in$ Actual width

$L_L := 6ft + 8in = 80\ in$ Length of wall

$d_c := 3.5in$ Concrete cover to center of anchors

$n_a := 5$ Number of anchors

$d_a := 0.75in$ Diameter of anchor

$s_a := 16in$ Anchor spacing longitudinally

$l_{ebd} := 15in$ Actual embedment depth

$c_{a1} := B_U - d_c = 11.5\ in$ Edge distance parallel to shear force

$c_{a3} := B_L - c_{a1} = 28.5\ in$

$c_{a2} := \frac{s_a}{2} = 8\ in$ Edge distance perpendicular to shear force

$4 \cdot d_a \le l_{ebd} \le 20d_a = 0$ OK Embedment depth limit, ACI 318 17.3.3

$h_{ef} := \min(l_{ebd}, 20 \cdot d_a) = 15\ in$ Effective embedded length. ACI 318 17.3.3

$A_{se\_N} := \frac{\pi \cdot d_a^2}{4} = 0.44 \cdot in^2$ Anchor cross-sectional area

$A_{se\_V} := \frac{\pi \cdot d_a^2}{4} = 0.44 \cdot in^2$

$c_{a\_min} := \min(c_{a1}, c_{a2}) = 8\ in$ Min edge distance

Calculations of Anchor Strengths

Anchor tensile strength

Tensile strength - steel

$f_{uta} := \min\left(f_{ua}, 1.9 \cdot f_{ya}, 125ksi\right) = 106.6 \cdot ksi$

$n_a = 5$

$N_{sa} := n_a \cdot A_{se\_N} \cdot f_{uta} = 235.5 \cdot kip$

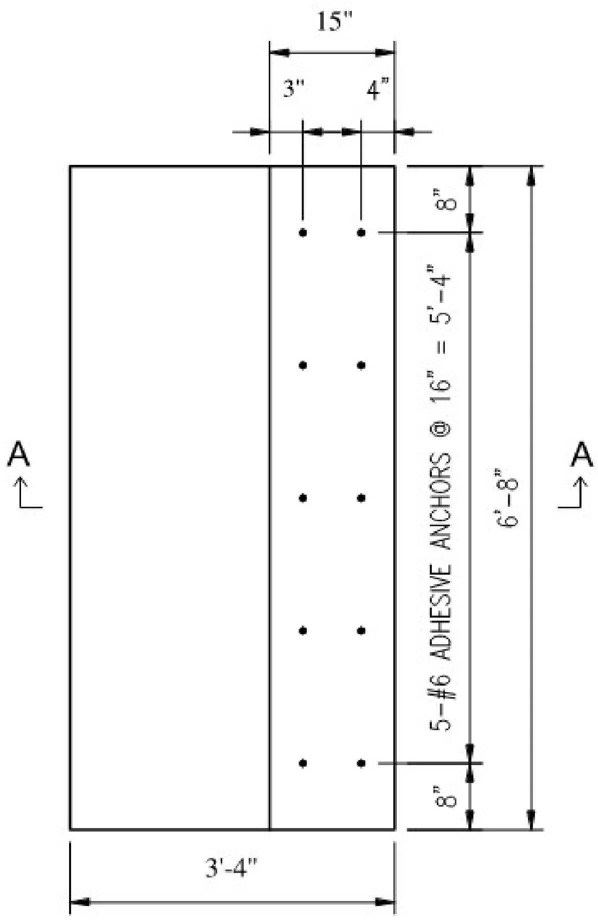
$\phi_{ts} \cdot N_{sa} = 176.6 \cdot kip$

Yield strength of steel anchor

$N_{ya} := n_a \cdot A_{se\_N} \cdot f_{ya} = 152 \cdot kip$

Tensile strength - Concrete breakout

Consider a group of five anchors



$n_a = 5$

$c_{a1} = 11.5\,in$ $\quad c_{a2} = 8\,in$ $\quad c_{a3} = 28.5\,in$

$A_{Nco} := 9 \cdot h_{ef}^2 = 2025\,in^2$ ACI 318 17.6.2.1.4

$n_a \cdot A_{Nco} = 10125\,in^2$

$A_{Nc1} := \left(\min\left(1.5 \cdot h_{ef}, c_{a1}\right) + \min\left(1.5h_{ef}, c_{a3}\right)\right) \cdot L_L = 2720\,in^2$

$A_{Nc} := \min\left(A_{Nc1}, n_a \cdot A_{Nco}\right) = 2720\,in^2$ ACI 17.6.2.1.1

$\psi_{ec\_N} := 1$ The eccentricity factor for a group of anchors does not apply beacause only the interior row of anchors is considered.

$$\psi_{ed\_N} := \begin{cases} 1 & \text{if } c_{a\_min} \ge 1.5 \cdot h_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{1.5 \cdot h_{ef}} & \text{otherwise} \end{cases}$$

The modifcation factor for edge effects

40.16.3 and ACI 318 17.6.2.4

$$\psi_{ed\_N} = 0.81$$

$$\psi_{c\_N} := 1.4$$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels; conservative

$$\psi_{cp\_N} := 1$$

Breakout splitting factor for concrete with supplementary reinforcement (ACI 318 17.6.2.6)

$$k_c := 17 \qquad f'_c = 5110\,psi$$

$k_c = 17$ for post-installed anchor, ACI 318 17.6.2.2.1

$$N_b := k_c \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} lbf = 70.6 \cdot kip$$

Basic concrete breakout strength of a single anchor in tension in cracked concrete (ACI 318-14 17.4.2.2a)

$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ec\_N} \cdot \psi_{ed\_N} \cdot \psi_{c\_N} \cdot \psi_{cp\_N} \cdot N_b = 107.09 \cdot kip$$

Concrete breakout strength in tension for anchor group (ACI 318-14 17.4.2.1b)

$$\phi_{tc} \cdot N_{cbg} = 80.32 \cdot kip$$

Anchor bond strength

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100psi}} = 6.99\,in$$

$$A_{Nao} := 4 \cdot c_{Na}^{\ 2} = 195.3\,in^2$$

ACI 318 17.6.5.1.2a

$$n_a \cdot A_{Nao} = 976.7\,in^2$$

$$A_{Na1} := \left(\min\left(c_{Na}, c_{a1}\right) + \min\left(c_{Na}, c_{a3}\right)\right) \cdot L_L = 1118.1\,in^2$$

$$A_{Na} := \min\left(A_{Na1}, n_a \cdot A_{Nao}\right) = 976.7\,in^2$$

$$\psi_{ed\_Na} := \begin{cases} 1 & \text{if } c_{a\_min} \ge c_{Na} \\ 0.7 + 0.3 \cdot \dfrac{c_{a\_min}}{c_{Na}} & \text{otherwise} \end{cases}$$

Breakout edge effect factor , 40.16.3 and ACI 318 17.6.5.4

$\psi_{ed\_Na} = 1$

$\psi_{cp\_Na} := 1$

Bond splitting factor assuming reinforcement in the lower wing wall functions as supplementary reinforcement, 40.16.3 and ACI 17.6.5.5.2.

$N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef} = 33.75\,\text{kip}$

Assuming cracked concrete given uncertain condition of the existing lower wing wall. 40.16.3 and ACI 318 17.6.5.2.1

$$N_{ag} := \frac{A_{Na}}{A_{Nao}} \cdot \psi_{ed\_Na} \cdot \psi_{cp\_Na} \cdot N_{ba} = 168.8\,\text{kip}$$

40.16.3 and ACI 318 17.6.5.1

$\phi_{tc} \cdot N_{ag} = 126.6\,\text{kip}$

Determine governing failure mode:

$N_{sa} = 235.5\,\text{kip}$

$N_{ya} = 151.8\,\text{kip}$

$N_{cbg} = 107.1\,\text{kip}$

$N_{ag} = 168.8\,\text{kip}$

$N_n := \min\left(N_{sa}, N_{cbg}, N_{ag}\right) = 107.09\,\text{kip}$

Adhesive anchor tensile resistance, concrete breakout controls

Shear

$$V_r = \phi_{vs} V_{sa} \le \phi_{vc} V_{cb} \le \phi_{vp} V_{cp}$$

Steel shear strength

$V_{sa} := 0.6 \cdot A_{se\_V} \cdot f_{uta} = 28.3\,\text{kip}$

$\phi_{vs} \cdot V_{sa} = 18.4\,\text{kip}$

Shear strength - Concrete breakout

$H := \min(1.5c_{a1}, h_a) = 17.3\text{ in}$

$S_1 := \min\left(1.5c_{a1}, \frac{s_a}{2}\right) = 8\text{ in}$

$S_2 := \min\left(1.5c_{a1}, \frac{s_a}{2}\right) = 8\text{ in}$

S1 and S2 are calculated for an anchor within a row (i.e. there are anchors on both sides of the calculated anchor).

$A_{Vco} := 4.5c_{a1}^{\ 2} = 595.1\text{ in}^2$

ACI 318 17.7.2.1.3

$A_{Vc} := (S_1 + S_2)\cdot H = 276\text{ in}^2$

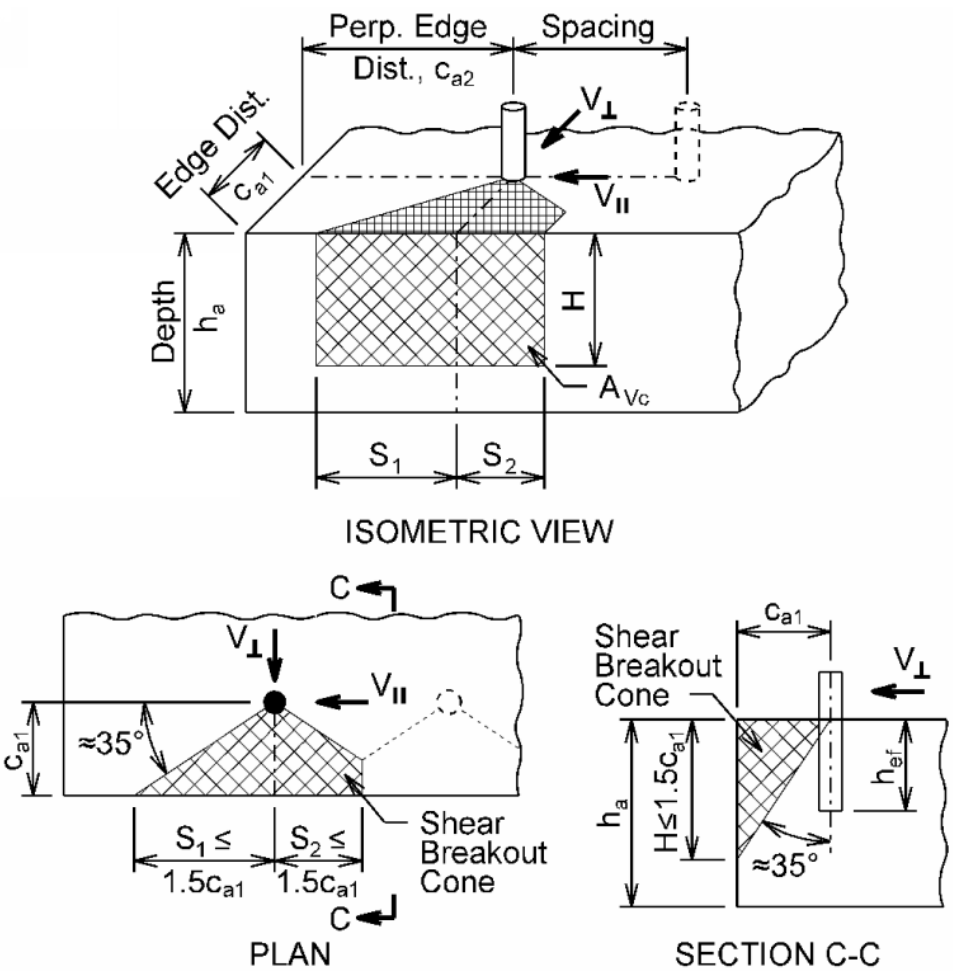


Figure 40.16-3
Concrete Breakout of Concrete Anchors in Shear

$\psi_{ed\_V} := 1$

The modifcation factor for edge effect for a single anchor or group of anchors loaded in shear (ACI 17.5.2.6). Perpendicular shear with $c_{a2} >= 1.5c_{a1}$

$\psi_{c\_V} := 1.2$

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of at least a No. 4 bar or greater between the anchor and the edge. Deck longitudinal reinforcment will likely meet this condition.

$$\psi_{h\_V} := \max\left(1, \sqrt{\frac{1.5 \cdot c_{a1}}{h_a}}\right) = 1$$

The modifcation factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$

$\psi_{p\_V} := 1$

Shear perpendicular to edge

$$l_e := \min\left(h_{ef}, 8 \cdot d_a\right) = 6\,\text{in}$$

$$V_{b1} := 7 \cdot \left[\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right] lbf = 25.6 \cdot kip$$

$$V_{b2} := 9 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left[\left(\frac{c_{a1}}{in}\right)^{1.5}\right] lbf = 25.1 \cdot kip$$

$$V_b := \min\left(V_{b1}, V_{b2}\right) = 25.1 \cdot kip$$

Basic breakout shear strength

$$V_{cb} := \frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ed\_V} \cdot \psi_{c\_V} \cdot \psi_{h\_V} \cdot V_b = 14 \cdot kip$$

Concrete breakout strength of anchor for shear force perpendicular to the edge on a single anchor (ACI 17.5.2.1)

$$\phi_{vc} \cdot V_{cb} = 10.5\,\text{kip}$$

Shear strength - Concrete pryout

WisDOT Manual 40.16.4

$N_{cp} := \min(N_{ag}, N_{cbg}) = 107.1 \cdot kip$

Anchor tensile strength

$h_{ef} \geq 2.5in$ OK

WisDOT Manual 40.16.4

$V_{cp} := 2 \cdot N_{cp}$

Anchor pryout strength

$\phi_{vp} \cdot V_{cp} = 149.9 \cdot kip$

$V_r := \min(\phi_{vs} \cdot V_{sa}, \phi_{vc} \cdot V_{cb}, \phi_{vp} \cdot V_{cp}) = 10.5 \cdot kip$

Concrete breakout controls shear strength.

$V_n := \min(V_{sa}, V_{cb}, V_{cp}) = 14 \cdot kip$

Check edge distances, spacings (40.16.1.1)

$s_{a\_min} := 6 \cdot d_a = 4.5 \cdot in$

Minimum spacing required, ACI 318 17.9.2

$c_{edge\_min} := 6 \cdot d_a = 4.5 \cdot in$

Minimum edge distance required, ACI 318 17.9.2

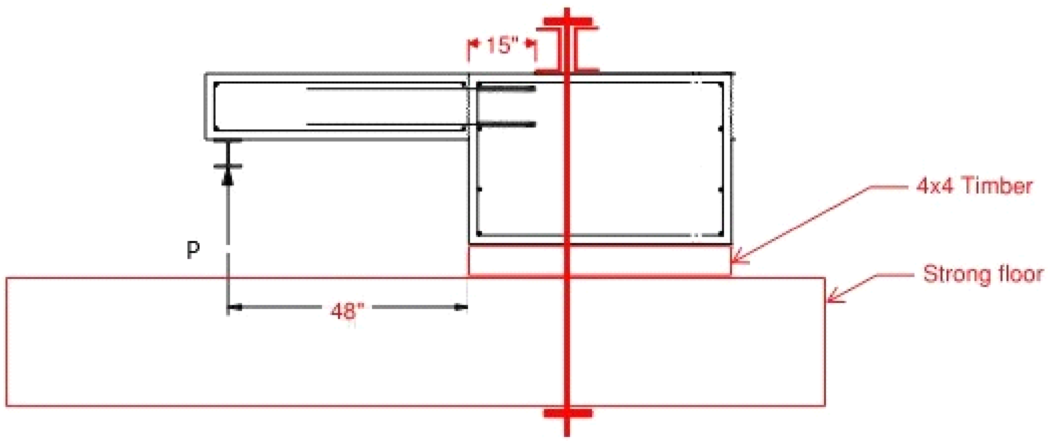
$s_a \geq s_{a\_min} = 1$

OK

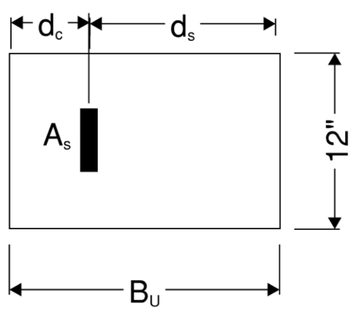
$c_{a1} \geq c_{edge\_min} = 1$

OK

Caculations of Wall Resistances in Flexure and Shear



Test Setup Schematic



Upper Wall in Flexure, LRFD 5.6.3

Flexural resistance of wall with adhesive anchors is determined based on LRFD 5.6.3 in which the anchor tensile resistance, T_n, is used instead of Asfs. The factored flexural strength of wall is determined using the anchor factored tensile resistance, where the resistance factors are in accordance with LRFD 5.13, which references ACI 318, Chapter 17.

$L_L = 80\,in$

$T_n := N_n = 107.1\cdot kip$ — Anchor nominal tensile resistance based on failure modes

$T_{n_1} := N_{cbg} = 107.1\cdot kip$ — concrete breakout

$T_{n_2} := N_{ag} = 169\cdot kip$ — bond strength

$T_{n_3} := N_{ya} = 151.8\cdot kip$ — steel yield

$T_{n_4} := N_{sa} = 235.5\cdot kip$ — steel strength

$$T_n = \begin{pmatrix} 107 \\ 169 \\ 152 \\ 235 \end{pmatrix} \cdot kip$$

$$d_s := B_U - d_c = 11.5\ in$$

$E_s := 29000ksi$ — Modulus of elasticity of steel reinforcment

$f'_{cu} := 1, 1.1 .. 4$ — Compressive strength of upper wall

$$E_{cu}(f'_{cu}) := 33000 \cdot \left(\frac{w_c}{1000pcf}\right)^{1.5} \cdot \sqrt{f'_{cu}} ksi$$

Modulus of elasticity of concrete, ACI 318 19.2.2.1a

Calculate flexural resistance of upper wall assuming linear compressive stress distribution in concrete and no tensile stress in concrete.

$$A_s := A_{se\_N} = 0.44 \cdot in^2$$

$$\rho := \frac{A_s}{d_s \cdot 12in} = 0.0032$$

$$n(f'_{cu}) := \frac{E_s}{E_{cu}(f'_{cu})}$$

$$k(f'_{cu}) := \sqrt{\left(\rho \cdot n(f'_{cu})\right)^2 + 2 \cdot \rho \cdot n(f'_{cu})} - \rho \cdot n(f'_{cu})$$

$$j(f'_{cu}) := 1 - \frac{k(f'_{cu})}{3}$$

$$M_n(f'_{cu}) := T_n \cdot j(f'_{cu}) \cdot d_s$$ Nominal flexural resistance of wall

$f'_{cu} := 2.02ksi$ Compressive strength of upper wall

$$E_{cu} := 33000 \cdot \left(\frac{w_c}{1000pcf}\right)^{1.5} \cdot \sqrt{\frac{f'_{cu}}{ksi}} ksi = 2725 \cdot ksi$$ Modulus of elasticity of concrete, ACI 318 19.2.2.1a

Calculate flexural resistance of upper wall assuming linear compressive stress distribution in concrete and no tensile stress in concrete.

$$A_s := A_{se\_N} = 0.44 \cdot in^2$$

$$\rho := \frac{A_s}{d_s \cdot 12in} = 0.0032$$

$$n := \frac{E_s}{E_{cu}} = 10.6$$

$$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n = 0.23$$

$$j := 1 - \frac{k}{3} = 0.92$$

$$d_s = 11.5\,in$$

$$M_n := T_n \cdot j \cdot d_s = \begin{pmatrix} 95 \\ 149 \\ 134 \\ 208 \end{pmatrix} \cdot kip \cdot ft$$

concrete breakout
bond strength
steel yield
steel strength

Nominal flexural resistance of wall

$H_P := 4.ft$

Max anticipated test load based on possible failure modes:

$$P_{max} := \frac{M_n}{H_P} = \begin{pmatrix} 23698 \\ 37344 \\ 33580 \\ 52105 \end{pmatrix} \cdot lbf$$

concrete breakout
bond strength
steel yield
steel strength

Upper Wall in Shear

Self-weight of upper wingwall:

$$P_{SW} := w_c \cdot L_L \cdot B_U \cdot H_U = 6\,kip$$

Shear at bottom of wall

$$V_{max} := P_{max} - P_{SW} = \begin{pmatrix} 17 \\ 31 \\ 27 \\ 46 \end{pmatrix} kip$$

$$\frac{M_n}{V_{max}} = \begin{pmatrix} 5.4 \\ 4.8 \\ 4.9 \\ 4.5 \end{pmatrix} \cdot ft$$

$$V_c := 2 \cdot \left[\sqrt{\frac{f'_{cu}}{psi}} \cdot \left(d_s \cdot L_L \right) psi \right] = 82.7\,kip$$

Interface shear resistance , LRFD 5.7.4.3

Horizontal shear at the bottom of the upper wing is resisted by the interface shear resistance between the upper and lower wing sections.

$b_{vi} := B_U = 15\,in$ — Concrete interface width considered to be engaged in shear transfer

$L_{vi} := L_L = 80\,in$ — Concrete interface length considered to be engaged in shear transfer

$A_{cv} := b_{vi} \cdot L_{vi} = 1200\,in^2$ — Concrete interface area considered to be engaged in shear transfer

$A_{vf} := n_a \cdot A_{se\_V} = 2.21\,in^2$ — Cross-sectional area of anchor group resisting shear

$\phi_v := 1$

Cohesion and friction factors, LRFD 5.7.4.4:

For calculations in this example, it was assumed concrete placed against a clean concrete surface, free of laitance, with surface not intentionally roughened. In practice, it is advisable that the concrete surface be intentionally roughened to an amplitude of 1/4 inch to improve the interface shear resistance.

$c := 0.075\text{ksi}$ Cohesion factor

$\mu := 0.6$ Friction factor

$K_1 := 0.2$ Fraction of concrete strength available to resist interface shear

$K_2 := 0.8\text{ksi}$ Limiting interface shear resistance (ksi)

Interface shear resistances

LRFD Equation 5.7.4.3-3 for shear resistance of the interface plane is based on the assumption that the interface reinforcement is stressed to its design yield stress, f_y. Adhesive anchors may not be stressed to its design yield stress at the nominal resistance; thus, the tensile force used to determine interface shear resistance is the lesser of:

- Yield strength of the anchor and
- Governing anchor tensile strength

$T_{ni} := \min\left(A_{vf} \cdot f_{ya}, N_{cbg}, N_{ag}\right) = 107.1\,\text{kip}$ Factored tensile force in adhesive anchor used to determine interface shear resistance

$P_c := 0\text{kip}$ Permanent net compressive force normal to the shear plane. Conservatively disregard compression force by self-weight of the upper wing at the interface.

$V_{ri1} := \phi_v \cdot c \cdot A_{cv} + \mu \cdot \left(T_{ni} + \phi_v \cdot P_c\right) = 154.3 \cdot \text{kip}$ LRFD Equation 5.7.4.3-3, modifed for adhesive anchors

$V_{ri2} := \phi_v \cdot K_1 \cdot f'_{cu} \cdot A_{cv} = 485 \cdot \text{kip}$ LRFD Equation 5.7.4.3-4

$V_{ri3} := \phi_v \cdot K_2 \cdot A_{cv} = 960 \cdot \text{kip}$ LRFD Equation 5.7.4.3-5

$V_{ri} := \min\left(V_{ri1}, V_{ri2}, V_{ri3}\right) = 154 \cdot \text{kip}$

Upper Wall in Flexure, conventionally reinforced

Flexural resistance of wall based on conventionally reinforced and cast using ACI 318 equations and tensile strength of reinforcing steel based on fy.

$$M_{n.Asfy} := n_a \cdot A_s \cdot f_{ya} \cdot j \cdot d_s = 134 \cdot kip \cdot ft$$

$$T_{n.Asfy} := \frac{M_{n.Asfy}}{H_P} = 33580 \cdot lbf$$

Maximum anticipated load applied to upper wingwall at 48-in distance from wingwall joint to cause yield of reinforcing steel.