Report No. BD545 RPWO #54 Date: July 2007
FINAL REPORT

Contract Title: Anchor Embedment Requirements for Signal/Sign Structures
UF Project No. 00054247
Contract No. BD545 RPWO #54

ANCHOR EMBEDMENT REQUIREMENTS FOR SIGNAL/SIGN STRUCTURES

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
BD545 RPWO #54		

4. Title and Subtitle	5. Report Date
Anchor Embedment Requirements for Signal/Sign Structures	July 2007
	6. Performing Organization Code
7. Author(s) R. A. Cook and K. M. Halcovage	8. Performing Organization Report No. 00054247
9. Performing Organization Name and Address University of Florida Department of Civil Engineering 345 Weil Hall / P.O. Box 116580 Gainesville, FL 32611-6580	10. Work Unit No. (TRAIS) 11. Contract or Grant No. BD545 RPWO #54
12. Sponsoring Agency Name and Address Florida Department of Transportation Research Management Center 605 Suwannee Street, MS 30 Tallahassee, FL 32301-8064	13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

During the 2004 hurricane season, several anchor embedment failures of the foundations of cantilever sign structures occurred. The purpose of this research program was to determine the cause of the failure of those foundations. After a literature review, in conjunction with site investigation, and testing, it was determined that the failure originated from the shear load on the anchors directed parallel to the edge of the foundation. The shear load resulted from the torsion loading on the anchor group that occurred during the hurricane. Investigation of this failure mode, based on the ACI 318-05 Appendix D provisions for concrete breakout of anchors, indicated that this is a failure mode not considered in the current design procedures for these types of foundations. Furthermore, it was determined that it very well describes the type of failure noted in the field investigation.

The test specimen was designed to preclude other possible failure modes not exhibited in the field (e.g., steel failure of the anchors, bending of the anchors, and torsion failure of the foundation). The results of the testing indicated the failure of the foundations was caused by concrete breakout due to shear on the anchors directed parallel to the free edge of the foundation. The test specimen failed at the torsion predicted by the ACI 318-05 Appendix D provisions based on the expected mean strength of the anchors for concrete breakout with shear directed parallel to the free edge. Additionally, the cracks that formed were the same type as those noted in the field investigation, and matched the expected pattern for concrete breakout failure.

Additional testing was performed to determine a viable repair/retrofit option. The repair/retrofit option used a carbon fiber reinforced polymer (CFRP) wrap around the top of the foundation. The results of this testing indicated that this repair/retrofit technique strengthens the foundation such that it not only meets its initial capacity for concrete breakout, but, also, can exceed this capacity. The results of this test led to the development of guidelines for the evaluation and repair/retrofit of existing foundations. Alternative foundations were also presented for future consideration.

17. Key Word	18. Distribution Statement
anchorage, cantilever sign structures, torsion, anchors, wind loads	No restrictions

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	128	

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ACKNOWLEDGMENTS

The authors acknowledge and thank the Florida Department of Transportation for providing the funding for this research project. This project was a collaborative effort between the University of Florida and the FDOT Structures Research Laboratory (Tallahassee). The authors wish to thank the FDOT Structures Research Laboratory personnel (David Allen, Frank Cobb, Steve Eudy, Chris Weigly, Paul Tighe) for constructing the specimen, installing instrumentation, data acquisition, and in general performing the tests required to make this project successful.

EXECUTIVE SUMMARY

During the 2004 hurricane season, the failure of several foundations of cantilever sign structures occurred along Florida highways. These failures necessitated a review of the current design and construction procedures for the foundations of cantilever sign structures.

The primary objectives of this research program were to:

- determine the cause of the failure of the cantilever sign structures and quantify what needs to be considered in design to preclude this type of failure
- develop a repair/retrofit option for the foundation

In order to fulfill these objectives, a literature review, site investigation, and experimental program were conducted. The findings of the literature review and site investigation were used to develop the experimental program. The findings of the experimental program were applied in the development of the repair/retrofit guidelines. In addition to the primary objectives, alternative support systems were also identified for future consideration.

After a literature review, in conjunction with site investigation, and testing, it was determined that the foundations failed as a result of an applied torsion from the wind loading which caused a concrete breakout failure due to shear on the anchors directed parallel to the edge of the foundation. This anchorage failure mode is detailed in ACI 318-05 Appendix D. This failure mode has not previously been incorporated in the design of the cantilever sign foundations. Cantilever sign foundations need to be designed for shear parallel to the edge on the anchor resulting from torsion.

Additional testing was performed to determine an acceptable repair/retrofit option. It was determined that applying a CFRP wrap to the foundation strengthens the foundation such that it not only meets its initial concrete breakout capacity, but, also, exceeds the capacity. The results of this test led to the development of guidelines for the evaluation and repair/retrofit of existing foundations. The guidelines were based on the following:

- Using either the torsional load from the design or, if not available, using the ACI nominal torsional strength (ACI 318-05 Section 11.6.3.6), determine the torsional capacity of the foundation.

- Calculate the concrete breakout strength in accordance with ACI Appendix D.

- If the concrete breakout strength is less than the maximum of the nominal torsional strength and design torsion, then the foundation is susceptible to failure.

- The amount of the carbon fabric required is calculated using the maximum of the nominal torsional strength and the design torsion. The amount required is given in layers of the CFRP wrap.

The guidelines may be used to evaluate and, if necessary, repair/retrofit existing foundations. It is critical that such foundations be evaluated in order to determine the susceptibility to this type of failure. Additionally, it is recommended that the alternative foundations identified be considered for further investigation.

Implementation of the recommended design and repair/retrofit methods for foundations of cantilever signal/sign structures should result in significantly less damage to these systems during hurricanes. This will result in a more reliable post-storm traffic system that should improve the time response for restoration of other critical services.

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CHAPTER 1
INTRODUCTION

During the 2004 hurricane season, the failure of several foundations of cantilever sign structures occurred along Florida highways (Figure 1-1). These failures necessitated a review of the current design and construction procedures for the foundations of cantilever sign structures.

The main objective of this research program was two-fold: to determine the cause of the failure of the cantilever sign structures; and, to propose a retrofit option for the foundation. In order to fulfill this objective, a thorough literature review, site investigation of a failed foundation, and experimental program were conducted. The findings of the literature review and site investigation were used to develop the experimental program. The findings of the experimental program were applied in the development of the retrofit guidelines.

Furthermore, this project tested whether or not the ACI 318-05, ACI (2005) Appendix D provisions for anchorage to concrete are applicable for circular foundations.

Figure 1-1. Failed cantilever sign structure

CHAPTER 2
BACKGROUND

While there have not been published reports detailing failures of sign structure foundations, such as those being investigated in this study, information on the behavior of anchor installations under various load conditions was found. The main subjects of much of the literature were the effects of fatigue and wind load on overhead sign structures. Additionally, there have been studies conducted on the failure modes of anchor installations, but these findings were not based on circular foundations. In later sections, one of these anchorage failure modes will be introduced for application in this research program.

This chapter presents the findings of the literature review, the conclusions drawn based on a site investigation of a failed foundation, and applicable design equations for the determination of the failure mode. The information presented in the chapter served as the base upon which the experimental program was developed.

2.1 Literature Review

Keshavarzian (2003) explores the wind design requirements and safety factors for utility poles and antenna monopoles from various specification manuals. It was found that the procedure outlined in ASCE (1991), ASCE 74- *Guidelines for Electrical Transmission Line Structural Loading*, resulted in the smallest factor of safety for the design. AASHTO (2001), *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*, was used as a part of the comparison for the design of the antenna monopole. The design from this specification was compared to that from ASCE (2000), ASCE 7-98-*Minimum Design Loads for Buildings and Other Structures*; TIA/EIA (1996), *Structural Standards for Steel Antenna Towers and Antenna Supporting Structures*; and, ASCE 74. The wind forces at the base were the same for ASCE 74, AASHTO, and ASCE 7-98. The forces using TIA/EIA were higher because it requires that a 1.69 gust response factor be applied to the design. Therefore, the pole designed using TIA/EIA would have between 30 and 40 percent extra capacity. ASCE 7-98 and AASHTO resulted in the same margin of safety. The paper did not include findings that were completely relative to this project, but it provided additional sources for design of structures for comparative purposes.

Keshavarzian and Priebe (2002) compares the design standards specified in ASCE (2000), ASCE 7-98, and IEEE (1997), NESC- *National Electrical Safety Code*. The NESC does not require that utility poles measuring less than 60 feet in height be designed for extreme wind conditions. Short utility poles were designed to satisfy NESC specifications (i.e. without extreme wind conditions). The poles were then evaluated according to the ASCE 7-98 wind load requirements. It was found that the poles did not meet the ASCE 7-98 requirements. Therefore, it was recommended that the exclusion for short utility poles in the NESC be reevaluated. The paper also mentioned AASHTO (1994), *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. It outlined that in the AASHTO specification, support structures exceeding 50 feet and overhead sign structures must be designed for a 50-year mean recurrence interval, or extreme wind loading condition.

MacGregor and Ghoneim (1995) presents the background information for the formulation of the thin-walled tube space truss analogy design method for torsion that was first adopted into ACI (1995), ACI 318-95. The design methodology was adopted because it was simpler to use than the previous method and was equally accurate. The basis for the derivation of the new method was based on tests that were conducted in Switzerland. Both solid and hollow beams were tested during that research. In comparing the data from both tests, it was discovered that after cracking the concrete in the center had little effect on the torsional strength of the beam.

Therefore, the center of the cross-section could be ignored, and the beam could be idealized as a hollow tube.

A space truss was formed by longitudinal bars in the corners, the vertical closed stirrups, and compression diagonals. The compression diagonals were spiraled around the member and extended between the torsion cracks. The paper also explained the shear stresses created by torsion on the member.

In addition to the derivation of the equations for torsion and shear, the authors discussed the limits for when torsion should be considered and the requirements for minimal torsional reinforcement. The tests, conducted on both reinforced and prestressed concrete beams, showed that there was acceptable agreement between the predicted strengths, as determined by the derived equations, and the test results. This agreement was comparable to the design equations from the ACI Code.

In addition to these papers, other reports reviewed include Lee and Breen (1966), Jirsa et al. (1984), Hasselwander et al. (1977), and Breen (1964). These four studies focused on important information regarding anchor bolt installations. Other reports that were examined for relevance were from the National Cooperative Highway Research Program (NCHRP). These are: Fouad et al. (1998), NCHRP Report 411; Kaczinski et al. (1998), NCHRP Report 412; and, Fouad et al. (2003), NCHRP Report 494.

Fouad et al. (2003) details the findings of NCHRP Project 17-10(2). The authors stated that AASHTO (2001) does not detail design requirements for anchorage to concrete. The ACI anchor bolt design procedure was also reviewed. Based on their findings, they developed a simplified design procedure. This procedure was based on the assumptions that the anchor bolts are hooked or headed, both longitudinal steel and hoop steel are present in the foundation, the anchor bolts are cast inside of the reinforcement, the reinforcement is uncoated, and, in the case of hooked bolts, the length of the hook is at least 4.5 times the anchor bolt diameter. If these assumptions did not apply, then the simplified procedure was invalid. The anchor bolt diameter was determined based on the tensile force on the bolt, and the required length was based on fully developing the longitudinal reinforcement between the embedded head of the anchor. The authors further stated that shear loads were assumed to be negligible, and concrete breakout and concrete side face blowout were controlled by adequate longitudinal and hoop steel. The design procedure was developed based on tensile loading, and did not address the shear load on the anchors directed parallel to the edge resulting from torsion.

Additionally, the authors presented the frequency of use of different foundation types by the state Departments of Transportation, expressed in percentages of states reporting use. According to the survey the most common foundation type used for overhead cantilever structures was reinforced cast-in-place drilled shafts (67-100%) followed by spread footings (34-66%) and steel screw-in foundations (1-33%). None of the states reported the use of directly embedded poles or unreinforced cast-in-place drilled shafts.

ASCE (2006), ASCE/SEI 48-05, entitled *Design of Steel Transmission Pole Structures* was obtained to gather information on the foundation design for transmission poles structures. The intent was to determine whether or not the design of such foundations was relevant to the evaluation of the foundations under examination in this research. In Section 9.0 of the standard, the provisions for the structural members and connections used in foundations was presented. Early in the section, the standard stated that the information in the section was not meant to be a foundation design guide. The proper design of the foundation must be ensured by the owner based on geotechnical principles. The section commented on the design of the anchor bolts. The standard focused on the structural stability of the bolts in the foundation; it looked at bolts in tension, bolts in shear, bolts in combined tension and shear, and the development length of such bolts. The standard did not present provisions for failure of the concrete.

2.2 Site Investigation

A site investigation was conducted at the site of one failed overhead cantilever signal/sign structure located at Exit 79 on Interstate 4 in Orlando (Figure 2-1). Figure 2-2 is the newly installed foundation at the site. The failed foundation had the same anchor and spacing specifications as the new foundation. This site visit coincided with the excavation of the failed anchor embedment. During the course of the excavation the following information was collected:

- The anchor bolts themselves did not fail. Rather, they were leaning in the foundation, which was indicative of a torsional load on the foundation. While the integrity of the anchor bolts held up during the wind loading, the concrete between the bolts and the surface of the foundation was cracked extensively (Figure 2-3). The concrete was gravelized between the anchors and the hoop steel. It should be noted that upon the removal and study of one anchor bolt, it was evident that there was no deformation of the bolt itself.
- The hoop steel did not start at the top of the foundation. It started approximately 15 in. (381 mm) into the foundation.

The concrete was not evenly dispersed around the foundation. The hoop steel was exposed at approximately three to four feet below grade. On the opposite side of the foundation there was excess concrete. It was assumed that during the construction of the foundation, there was soil failure allowing a portion of the side wall to displace the concrete.

Figure 2-1. Cantilever sign structure at Exit 79 on Interstate 4 in Orlando

Figure 2-2. New foundation installed at the site

Figure 2-3. Failed foundation during post-failure excavation

2.3 Applicable Code Provisions

The initial failure mode that was focused on in the background review was torsion. However, based on the results of the site investigation, it was determined that the most likely cause of failure was concrete breakout of an anchor (Figure 2-4). The equations for torsion are presented in this section as they were used during the design of the experimental program to prove that the concrete breakout failure will occur before the torsional failure. Torsion on the concrete cross-section is discussed in Sections 2.3.1 through 2.3.3. The concrete breakout failure of an anchor loaded in shear as displayed in Figure 2-4 is presented in Sections 2.3.4 through 2.3.5.

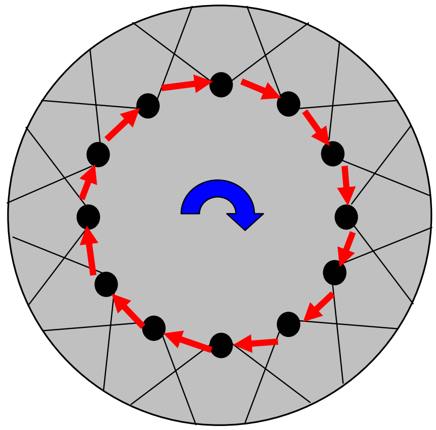


Figure 2-4. Concrete breakout of an anchor caused by shear directed parallel to the edge for a circular foundation

2.3.1 Cracking and Threshold Torsion

In a circular section, such as the foundation under review, the resulting torsion is oriented perpendicular to the radius or tangent to the edge. ACI 318-05 details the equation for the cracking torsion of a nonprestressed member. In Section R11.6.1, the equation for the cracking torsion, T_{cr}, is given (Equation 2-1). The equation was developed by assuming that the concrete will crack at a stress of $4\sqrt{f'_c}$.

$$T_{cr} = 4\sqrt{f'_c}\left(\frac{A_{cp}^{\;2}}{p_{cp}}\right) \tag{2-1}$$

Where

T_{cr} = cracking torsion (lb.-in.)
f'_c = specified compressive strength of the concrete (psi)
A_{cp} = area enclosed by the outside perimeter of the concrete cross-section (in.^{2})
= πr^2, for a circular section with radius r (in.)
p_{cp} = outside perimeter of the concrete cross-section (in.)
= $2\pi r$, for a circular section with radius r (in.)

This equation, when applied to a circular section, results in an equivalent value when compared to the basic equation (Equation 2-2) for torsion noted in Roark and Young (1975). The equality is a result of taking the shear stress as $4\sqrt{f'_c}$.

$$T = \frac{\tau \pi r^3}{2} \tag{2-2}$$

Where
T = torsional moment (lb.-in.)
τ = shear stress, $4\sqrt{f'_c}$, (psi)
r = radius of concrete cross-section (in.)

ACI 318-05 Section 11.6.1(a) provides the threshold torsion for a nonprestressed member (Equation 2-3). This is taken as one-quarter of the cracking torsion. If the factored ultimate torsional moment, T_u, exceeds this threshold torsion, then the effect of torsion on the member must be considered in the design.

$$T = \phi\sqrt{f'_c}\left(\frac{A_{cp}^{\ 2}}{p_{cp}}\right) \tag{2-3}$$

Where
Φ = strength reduction factor

AASHTO (2004), *AASHTO LRFD Bridge Design Specifications*, also presents equations for cracking torsion (Equation 2-4) and threshold torsion (Equation 2-5). Equation 2-4 corresponds with the AASHTO (2004) equation for cracking torsion with the exception of the components of the equation related to prestressing. That portion of the equation was omitted since the foundation was not prestressed. It must be noted that these equations are the same as the ACI 318-05 equations.

$$T_{cr} = 0.125\sqrt{f'_c}\,\frac{A_{cp}^{\ 2}}{p_c} \tag{2-4}$$

Where
T_{cr} = torsional cracking moment (kip-in.)
A_{cp} = total area enclosed by outside perimeter of the concrete cross-section (in.^{2})
p_c = the length of the outside perimeter of the concrete section (in.)

AASHTO (2004) also specifies the same provision as ACI 318-05 regarding the threshold torsion. In Section 5.8.2.1, it characterizes the threshold torsion as one-quarter of the cracking torsion multiplied by the reduction factor. Equation 2-5 corresponds with the threshold torsion portion of AASHTO (2004) equation.

$$T = 0.25\phi T_{cr} \qquad (2\text{-}5)$$

The above referenced equations considered the properties and dimensions of the concrete. They did not take into consideration the added strength provided by the presence of reinforcement in the member. For the purposes of this research, it was important to consider the impact of the reinforcement on the strength of the concrete shaft.

2.3.2 Nominal Torsional Strength

ACI 318-05 Section 11.6.3.5 states that if the ultimate factored design torsion exceeds the threshold torsion, then the design of the section must be based on the nominal torsional strength. The nominal torsional strength (Equation 2-6) takes into account the contribution of the reinforcement in the shaft.

$$T_n = \frac{2A_o A_t f_{yt}}{s}\cot\theta \qquad (2\text{-}6)$$

Where

T_n = nominal torsional moment strength (in.-lb.)
A_o = gross area enclosed by shear flow path (in.^{2})
A_t = area of one leg of a closed stirrup resisting torsion with spacing s (in.^{2})
f_{yt} = specified yield strength f_y of transverse reinforcement (psi)
s = center-to-center spacing of transverse reinforcement (in.)
θ = angle between axis of strut, compression diagonal, or compression field and the tension chord of the member

The angle, θ, is taken as 45°, if the member under consideration is nonprestressed. This equation, rather than taking into account the properties of the concrete, takes into account the properties of the reinforcement in the member. These inputs include the area enclosed by the reinforcement, the area of the reinforcement, the yield strength of the reinforcement, and the spacing of the reinforcement. For the purpose of this research, the reinforcement under consideration was the hoop steel.

AASHTO (2004) also outlines provisions for the nominal torsional resistance in Section 5.8.3.6.2. Equation 2-7 is the same equation that ACI 318-05 presents. The only difference is in the presentation of the equations. The variables are represented by different notation.

$$T_n = \frac{2A_o A_t f_y \cot\theta}{s} \tag{2-7}$$

Where

T_n = nominal torsional moment (kip-in.)

A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})

A_t = area of one leg of closed transverse torsion reinforcement (in.^{2})

θ = angle of crack

As the above referenced equation evidences, the ACI 318-05 and the AASHTO (2004) provisions for nominal torsional strength are the same. Based on the code provisions, the nominal torsional strength represents the torsional strength of the cross-section.

2.3.3 Combined Shear and Torsion

Another area that had to be considered in this research was the effect of combined shear and torsion. Both ACI 318-05 and AASHTO (2004) outline equations for the combined shear and torsion. Since the foundation had a shear load applied to it, it had to be determined whether the shear load was large enough to necessitate consideration. The ACI 318-05 equation (Equation 2-8) and the AASHTO (2004) equation (Equation 2-9) are presented hereafter. The ACI 318-05 equation is located in Section 11.6.3.1 of ACI 318-05, and the AASHTO (2004) equation is presented in Section 5.8.3.6.2 of that specification. The ACI 318-05 equation is presented with V_u substituted on the left-hand side.

$$V_u \leq \sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2} \tag{2-8}$$

Where

V_u = factored shear force at section (lb.)

$b_w d$ = area of section resisting shear, taken as A_{oh} (in.^{2})

T_u = factored torsional moment at section (in.-lb.)

p_h = perimeter of centerline of outermost closed transverse torsional reinforcement (in.)

A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement (in.^{2})

The AASHTO (2004) equation that is presented (Equation 2-9) is intended for the calculation of the factored shear force. For the purpose of this project, the right-hand side of the equation was considered.

$$V_u = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \qquad (2\text{-}9)$$

Where
V_u = factored shear force (kip)
p_h = perimeter of the centerline of the closed transverse reinforcement (in.)
T_u = factored torsional moment (kip-in.)

The determination of whether or not shear had to be considered was made based on a comparison of the magnitudes of the coefficients of these terms. This is investigated further in Chapter 3.

2.3.4 ACI Concrete Breakout Strength for Anchors

In ACI 318-05 Appendix D, the concrete breakout strength is defined as, "the strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member." A concrete breakout failure can result from either an applied tension or an applied shear. In this report, the concrete breakout strength of an anchor in shear, Section D.6.2, will be studied. The breakout strength for one anchor loaded by a shear force directed perpendicular to a free edge (Figure 2-5) is given in Equation 2-10.

$$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}\left(c_{a1}\right)^{1.5} \qquad (2\text{-}10)$$

Where
V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete (lb.)
ℓ_e = load bearing length of anchor for shear (in.)
d_o = outside diameter of anchor (in.)
c_{a1} = distance from the center of an anchor shaft to the edge of concrete in one

direction; taken in the direction of the applied shear (in.)

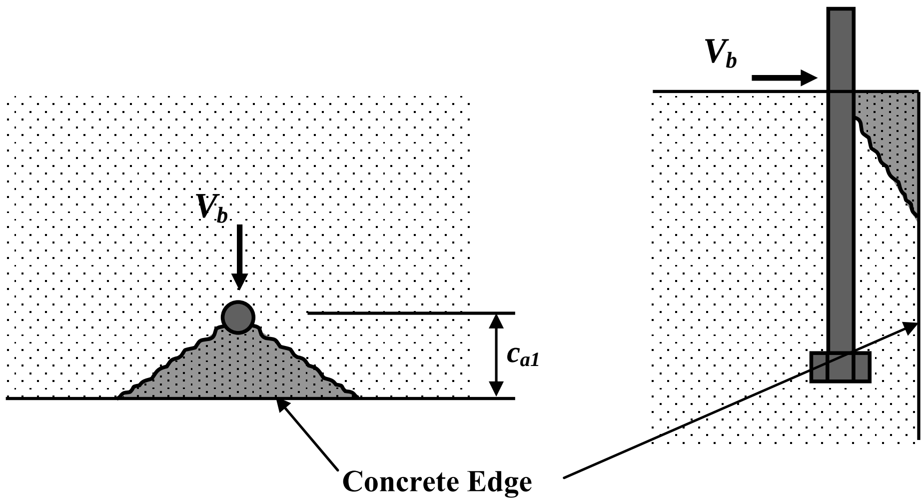


Figure 2-5. Concrete breakout failure for an anchor loaded in shear

The term ℓ_e is limited to $8d_o$ according to Section D.6.2.2. The equations in ACI 318-05 were developed based on a 5% fractile and with the strength in uncracked concrete equal to 1.4 times the strength in cracked concrete. The mean concrete breakout strength in uncracked concrete is provided in Fuchs et al. (1995) and given in Equation 2-11.

$$V_b = 13\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f'_c}\left(c_{a1}\right)^{1.5} \tag{2-11}$$

For a group of anchors, Equation 2-12 applies. This equation is the nominal concrete breakout strength for a group of anchors loaded perpendicular to the edge in shear.

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}V_b \tag{2-12}$$

Where

V_{cbg} = nominal concrete breakout strength in shear of a group of anchors (lb.)

A_{Vc} = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear (in.^{2})

A_{Vco} = projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness (in.^{2}) = $4.5(c_{a1})^2$, based on an ≈35° failure cone (Figure 2-6)

$\psi_{ec,V}$ = factor used to modify shear strength of anchors based on eccentricity of applied loads, ACI 318-05 Section D.6.2.5

$\psi_{ed,V}$ = factor used to modify shear strength of anchors based on proximity to edges of concrete member, ACI 318-05 Section D.6.2.6

$\psi_{c,V}$ = factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement, ACI 318-05 Section D.6.2.7, accounted for in Equation 2-11

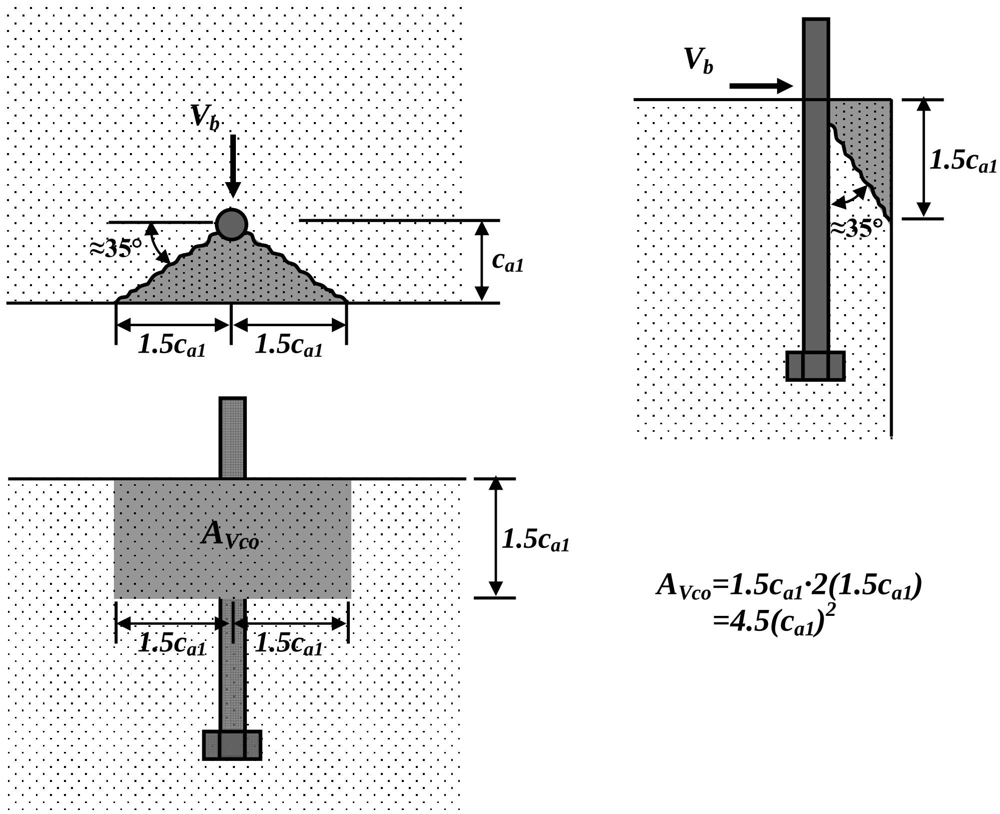


Figure 2-6. Determination of A_{Vco} based on the ≈35° failure cone

The resultant breakout strength is for a shear load directed perpendicular to the edge of the concrete. Therefore, an adjustment had to be made to account for the shear load acting parallel to the edge since this was the type of loading that resulted from torsion on the anchor group. In Section D.5.2.1(c) a multiplication factor of two is prescribed to convert the value to a shear directed parallel to the edge (Figure 2-7). Fuchs et al. (1995) notes that the multiplier is based on tests, which indicated that the shear load that can be resisted when applied parallel to the edge is approximately two times a shear load applied perpendicular to the edge.

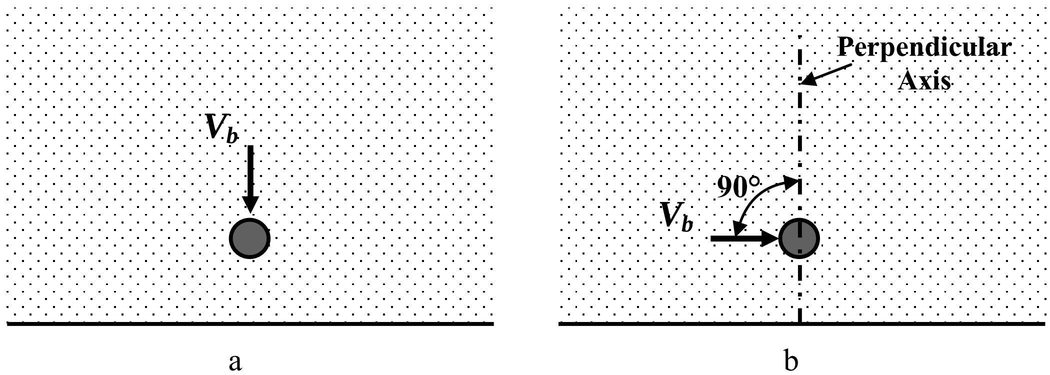


Figure 2-7. Shear load oriented (a) perpendicular to the edge and (b) parallel to the edge

In order to convert the breakout strength to a torsion, the dimensions of the test specimen were considered to calculate what was called the nominal torsional moment based on the concrete breakout strength, $T_{n,breakout}$.

2.3.5 Alternate Concrete Breakout Strength Provisions

In the book *Anchorage in Concrete Construction*, Eligehausen et al. (2006), the authors presented a series of equations for the determination of the concrete strength based on a concrete edge failure. These equations are presented in Chapter 4, Section 4.1.2.4 of the text. Equation 2-13 is the average concrete breakout strength of a single anchor loaded in shear. It must be noted that this equation is for uncracked concrete.

$$V_{u,c}^{0} = 3.0 \cdot d_o^{\alpha} \cdot \ell_e^{\beta} \cdot f_{cc200}^{0.5} \cdot c_{a1}^{1.5} \qquad (2\text{-}13)$$

Where

$V^0_{u,c}$ = concrete failure load of a near-edge shear loaded anchor (N)

d_o = outside diameter of anchor (mm)

ℓ_e = effective load transfer length (mm)

f_{cc200} = specified concrete compressive strength based on cube tests (N/mm^2) $\approx 1.18f'_c$

c_{a1} = edge distance, measured from the longitudinal axis of the anchor (mm)

$\alpha = 0.1 \cdot \left(\frac{\ell_e}{c_{a1}}\right)^{0.5}$

$\beta = 0.1 \cdot \left(\frac{d_o}{c_{a1}}\right)^{0.5}$

As was the case for the ACI 318-05 equations, the term ℓ_e is limited to $8d_o$. Equation 2-14 accounts for the group effect of the anchors loaded concentrically. The authors stated that cases where more than two anchors are present have not been extensively studied. They did, however, state that the equation should be applicable as long as there is no slip between the anchor and the base plate.

$$V_{u,c} = \frac{A_{Vc}}{A_{Vco}} \cdot V_{u,c}^{0} \tag{2-14}$$

Where

A_{Vc} = projected area of failure surface for the anchorage as defined by the overlap of individual idealized failure surfaces of adjacent anchors (mm^2)

A_{Vco} = projected area of the fully developed failure surface for a single anchor idealized as a half-pyramid with height c_{a1} and based lengths $1.5c_{a1}$ and $3c_{a1}$ (mm^2)

ACI 318-05 specifies that, in order to convert the failure shear directed perpendicular to the edge to the shear directed parallel to the edge, a multiplier of two be applied to the resultant load. The provisions outlined in this text take a more in-depth approach to determining this multiplier. The method for calculating this multiplier is detailed in Section 4.1.2.5 of Eligehausen et al. (2006). The authors stated that, based on previous research, the concrete edge breakout capacity for loading parallel to an edge is approximately two times the capacity for loading perpendicular to the edge if the edge distance is constant. The authors further moved to outline equations to calculate the multiplier based on the angle of loading. The first equation (Equation 2-15) that is presented in the text is a generalized approach for calculating the multiplier when the angle of loading is between 55° and 90° of the axis perpendicular to the edge. For loading parallel to the edge the angle is classified as 90° (Figure 2-7).

$$\psi_{\alpha,V} = \frac{1}{\cos\alpha + 0.5\sin\alpha} \tag{2-15}$$

Where

$\psi_{\alpha,V}$ = factor to account for the angle between the shear load applied and the direction perpendicular to the free edge of the concrete member

α = angle of the shear load with respect to the perpendicular load

This equation results in a factor of two for loading parallel to the edge. Equation 2-16 provides the concrete breakout strength for shear directed parallel to the edge using $\psi_{\alpha V}$.

$$V_{uc,\alpha V} = \psi_{\alpha V} \cdot V_{u,c} \qquad (2\text{-}16)$$

Where
$V_{uc,\alpha V}$= concrete failure load for shear directed parallel to an edge based on $\psi_{\alpha V}$ (N)

An alternate equation for calculating this factor is also presented in the Eligehausen et al. (2006) text. This equation is only valid for loading parallel to the edge. This equation is based on research proposing that the multiplier to calculate the concrete breakout capacity for loading parallel to the edge based on the capacity for loading perpendicular to the edge is not constant. Rather, it suggested that it is based on the concrete pressure generated by the anchor. The base equation for the application of this factor is Equation 2-17.

$$V_{u,c,parallel} = \psi_{parallel} \cdot V_{u,c} \qquad (2\text{-}17)$$

Where
$V_{u,c,parallel}$= concrete failure load in the case of shear parallel to the edge (N)
$\psi_{parallel}$ = factor to account for shear parallel to the edge
$V_{u,c}$ = concrete failure load in the case of shear perpendicular to the edge (N)

Equation 2-18 is used for the determination of the conversion factor $\psi_{parallel}$.

$$\psi_{parallel} = 4 \cdot k_4 \cdot \left[\frac{n \cdot d_o^{\,2} \cdot f_{cc}}{V_{u,c}} \right]^{0.5} \qquad (2\text{-}18)$$

Where
k_4 = 1.0 for fastenings without hole clearance
0.75 for fastenings with hole clearance
n = number of anchors loaded in shear
f_{cc} = specified compressive strength of the concrete (N/mm^2)
conversion to f'_c as specified for Equation 2-13

The results of Equation 2-13 through Equation 2-18 are presented alongside the ACI 318-05 equation results in Chapter 3. These are presented for comparative purposes only.

2.3.6 ACI 318-05 vs. AASHTO LRFD Bridge Design Specifications

In Sections 2.3.1 through 2.3.3, both the applicable design equations in ACI 318-05 and AASHTO (2004) were presented. As was shown, the ACI and AASHTO equations were the same. Additionally, the provisions for the concrete breakout failure capacity are only provided in ACI 318-05. AASHTO does not provide design guidelines for this failure. Therefore, the ACI 318-05 equations were used throughout the course of this research program.

2.4 Alternative Foundation Designs

Another focus of this research study was the identification of alternative foundation designs to be considered for future investigation. Four potential alternatives were identified. This section overviews the failure mode, ease of constructability, durability, and inspection considerations for each proposed foundation.

2.4.1 Reinforced Concrete Foundations

Three of the alternatives use reinforced concrete. The first alternative (Figure 2-8) is a steel pipe with plates welded at four points that is cast into a circular concrete foundation. Another alternative is shown in Figure 2-9 and incorporates a geometric hollow steel section, displayed as an octagon, cast into concrete. The third reinforced concrete foundation alternative (Figure 2-10) is a steel pipe with studs welded to the pipe. The foundation is shown with the steel pipe, but can also be integrated with the octagonal section.

These foundations must be designed to resist all applied loads (e.g., wind load, dead load). The failures modes that need to be considered for the reinforced concrete foundations are torsional failure of the steel pipe, torsional failure of the concrete, flexural failure of the concrete, and concrete breakout failure. For the foundation in Figure 2-8, the failure of the embedded plates and the failure of the weld between the plates and the pipe must be evaluated.

Additionally, for the foundation with the steel studs (Figure 2-10), the failure of the studs and the failure of the weld between the studs and the pipe must be considered.

These foundations will not require the installation of anchor bolts or a grout pad, which will simplify construction. However, the foundation will still require the use of reinforced concrete. The pipes will be cast into concrete protecting them against corrosion. The surface of the concrete should be routinely inspected for cracks indicative of a concrete breakout failure, and the steel pipe should be inspected for indications of failure and for corrosion of the steel where the pipe is exposed.

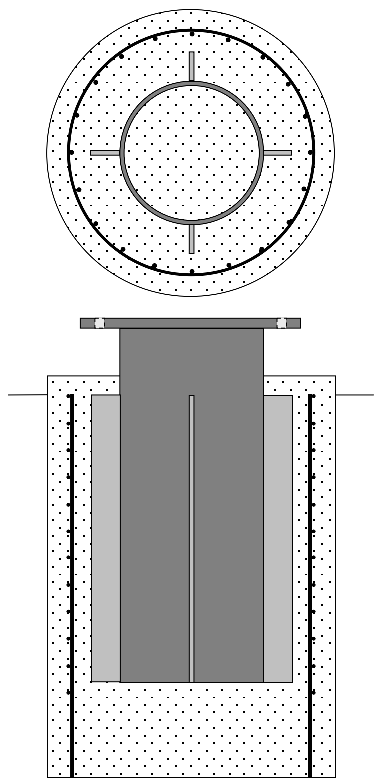


Figure 2-8. Alternate foundation: steel pipe with plates welded at four locations

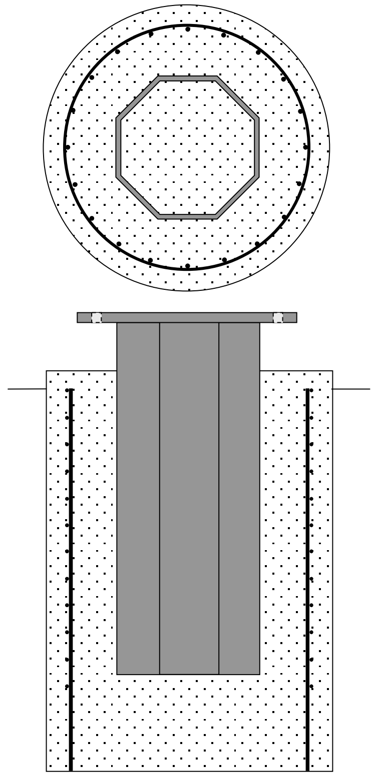


Figure 2-9. Alternate foundation: geometric hollow section

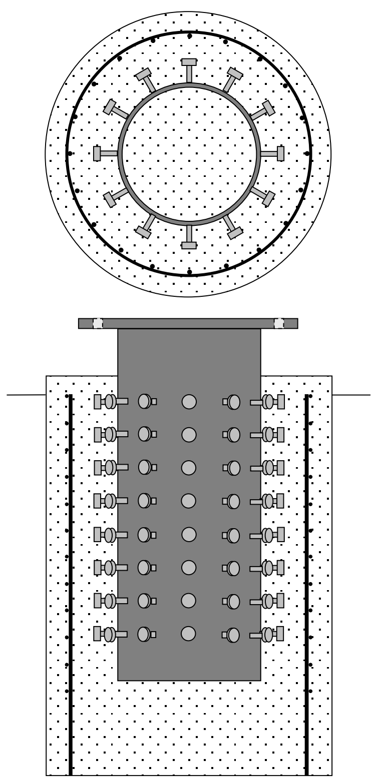


Figure 2-10. Alternate foundation: pipe with welded studs

2.4.2 Helical Pipe

The fourth alternative is involves use of a helical pipes (Figure 2-11), which would be screwed directly into the soil. For the helical pipes, the soil strength is of key importance along with the structural capacity of the steel. As with the previous three alternatives, this alternative will not require the installation of a grout pad or anchor bolts. In addition, the helical pipe alternative will not require the use a reinforced concrete foundation, which makes it the most construction-friendly of the three designs. However, care must be taken to ensure that the pipes are not damaged during the installation process. The helical pipes should also be galvanized or otherwise protected to guard against corrosion.

Inspection of the helical pipes below the surface of the soil will not be possible, which is why galvanized steel or other corrosion protection should be used. In addition to inspecting the steel above ground level, the deflection of structure should also need be routinely measured to ensure that it is securely anchored in the ground. One method of doing so would be to measure the rotation of the sign pole from the vertical plane. Geotechnical inspections in addition to structural inspections may need to be conducted for this foundation.

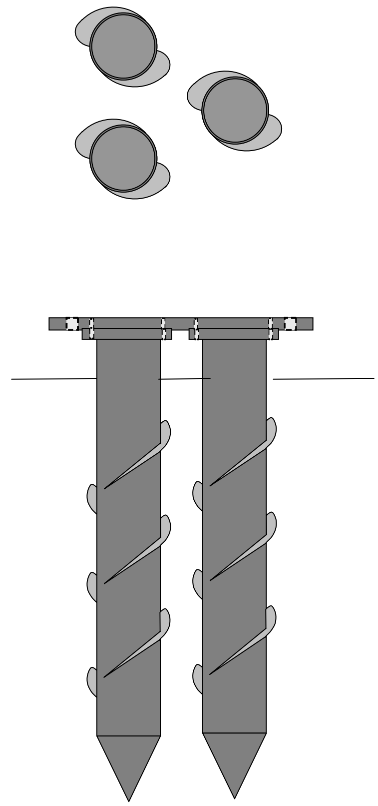


Figure 2-11. Alternate foundation: helical pipes

2.4.3 Summary

All of the proposed foundation designs are shown with annular base plates. An alternative method for attaching the sign pole for the first three options would be to telescope the sign pole over the foundation section. It is recommended that these alternative foundations be considered for further investigation to determine their viability for use in new foundations.

CHAPTER 3
DEVELOPMENT OF EXPERIMENTAL PROGRAM

After a thorough background investigation, it was determined that the most likely cause of the failure was the concrete breakout of an anchor loaded by a shear force directed parallel to a free edge. The shear force on the individual anchors was caused by torsion applied to the bolt group from the sign post. Based on this determination, an experimental program was formulated to determine if this was in fact the failure mode of the foundation. Therefore, it was of the utmost importance to design the test apparatus to preclude other failure modes. This chapter focuses on the development of the experimental program.

3.1 Description of Test Apparatus

The test apparatus was designed such that the field conditions could be closely modeled for testing at the Florida Department of Transportation (FDOT) Structures Research Center. A schematic of the test apparatus is shown in Figure 3-1. The test apparatus consisted of:

- A 30” (762 mm) diameter concrete shaft that extended 3’-0” (914 mm) outward from the concrete block
- Twelve 37” (940 mm), 1.5” (38.1 mm) diameter F1554 Grade 105 anchor bolts embedded into the concrete around a 20” (508 mm) diameter
- A 16” (406 mm) diameter steel pipe assembly welded to a 24” (610 mm) diameter, 1” (25.4 mm) thick steel base plate with holes drilled for the anchor bolts to provide the connection between the bolts and pipe assembly
- A 6’-0” x 10’-0” x 2’-6” (1830 mm x 3050 mm x 762 mm) reinforced concrete block to provide a fixed support at the base of the shaft
- Two assemblies of C12x30 steel channels and plates to attach the block to the floor

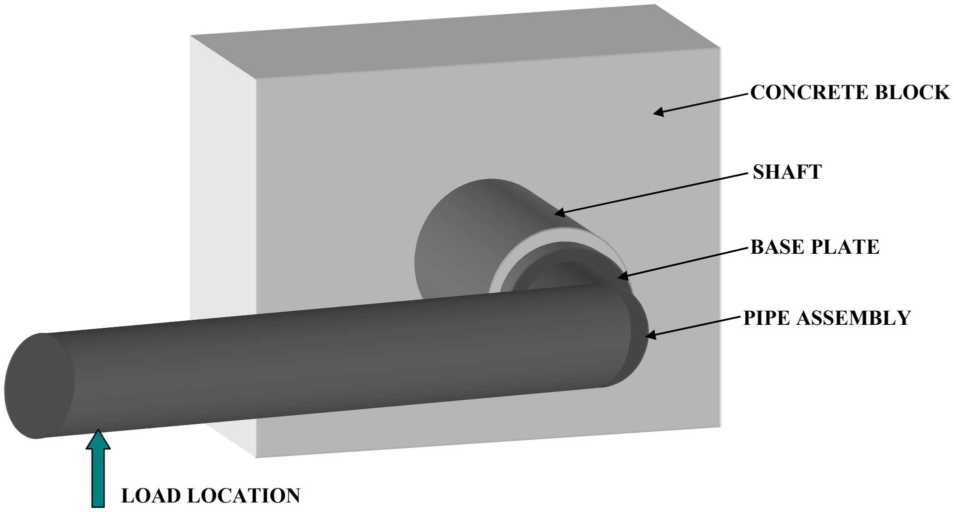


Figure 3-1. Schematic of test apparatus

The basis for the design of the various components of the test apparatus was one half of the size of the failed foundation investigated during the site visit. The dimensions of the field foundation are presented in Table 3-1. From that point, the elements of the test apparatus were designed to preclude all failure modes other than the concrete breakout failure of the anchors.

Information pertaining to the design of the components of the apparatus is presented in the following sections. Figures 3-2 through 3-4 are drawings of the test apparatus. For large scale dimensioned drawings, reference Appendix A. Complete design calculations are located in Appendix B.

Table 3-1. Field and test specimen dimensions

Component	Field Dimension	Test Specimen Dimension
Shaft Diameter	60 in.	30 in.
Hoop Steel Diameter	46 in	27 in.
Hoop Steel Size	#5	#3
Longitudinal Steel Size	#9	#4
Anchor Bolt Diameter	2 in.	1½ in.
Anchor Embedment	55 in.	26 in.
Bolt Spacing Diameter	36 in.	20 in.
Base Plate Diameter	42 in.	24 in.
Base Plate Thickness	1⅛ in.	1 in.

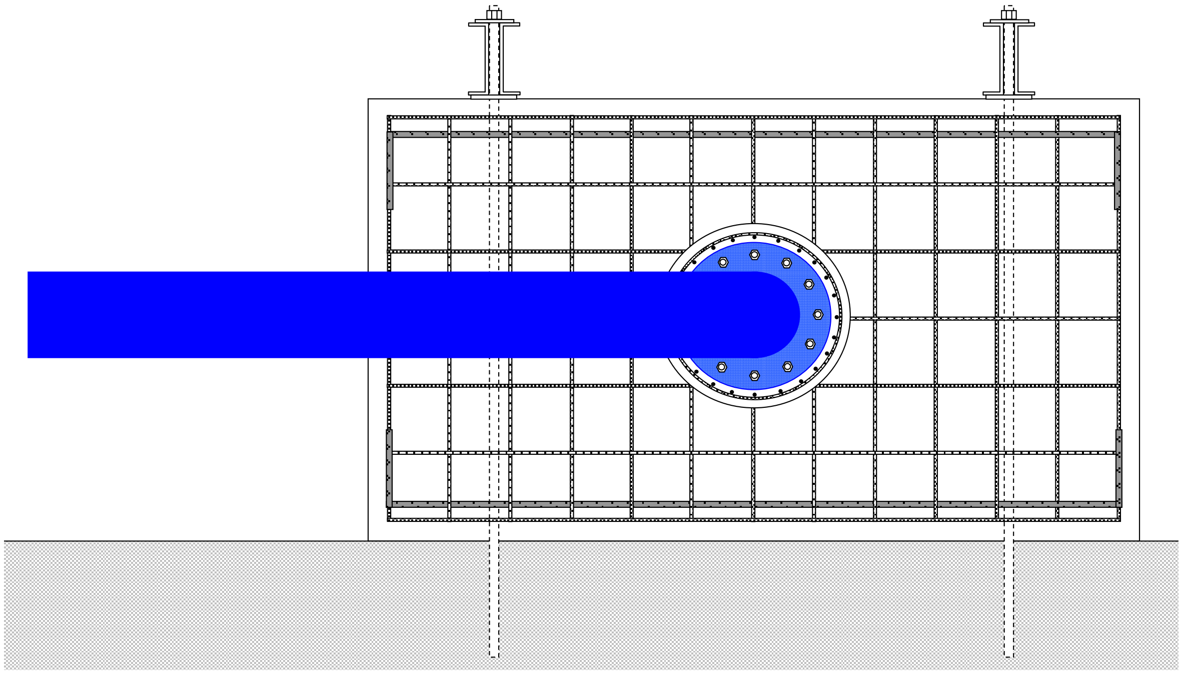


Figure 3-2. Front elevation of test apparatus

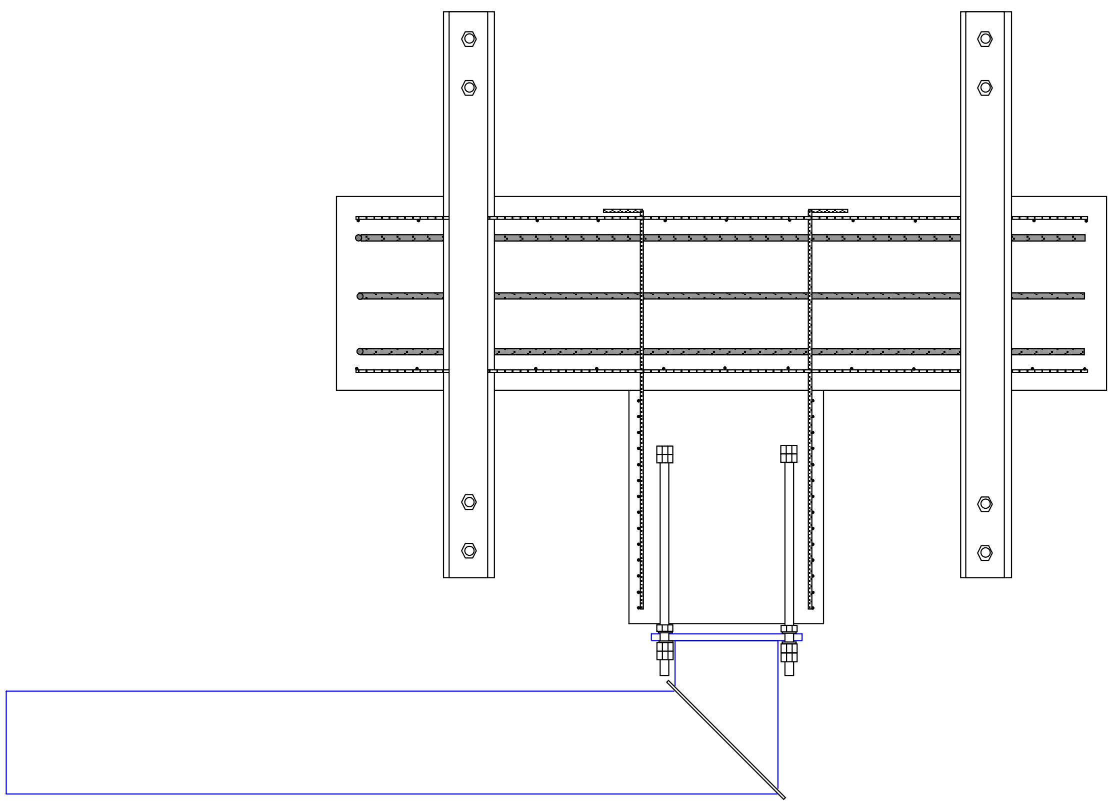


Figure 3-3. Plan view of test apparatus

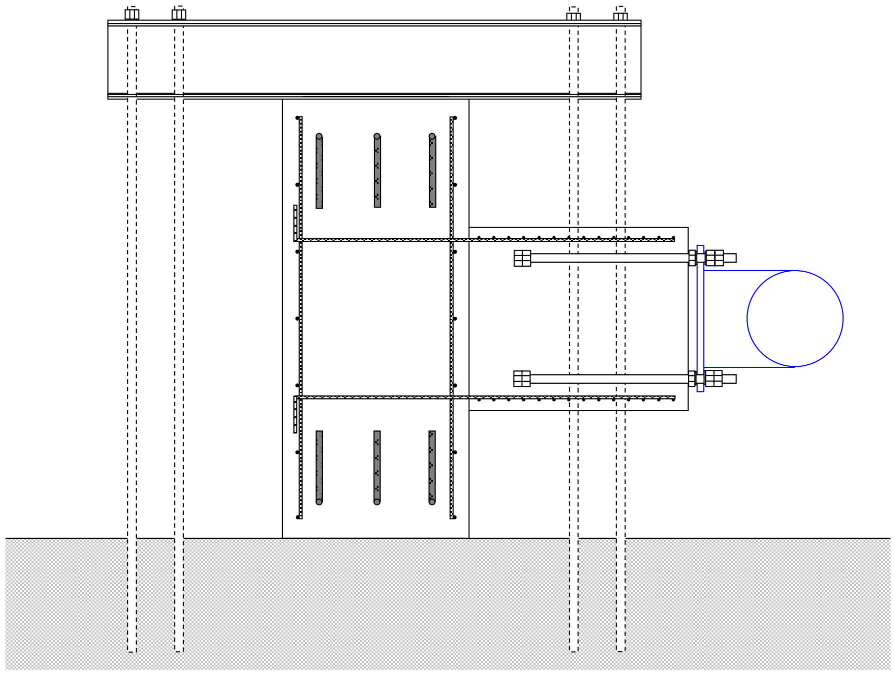


Figure 3-4. Side elevation of test apparatus

3.2 Shaft Design

The starting point for the design of the concrete shaft was based on developing a test specimen approximately one half of the size of the foundation that was investigated during the site visit. From there, the various components of the shaft were designed the meet the ACI 318-05 requirements, and to prevent failure before the concrete breakout strength was reached and exceeded. All of the strengths were calculated using a concrete strength of 5500 psi (37.9 MPa), which was the strength indicated on the FDOT standard drawings.

3.2.1 Torsion Design

The basic threshold torsional strength of the shaft, 24.6 kip-ft (33.4 kN-m) was calculated using the ACI 318-05 torsional strength equation (Equation 2-3). This strength, however, did not take into account the reinforcement in the shaft. Therefore, it was assumed that the threshold torsion would be exceeded. As a result, the torsional strength of the shaft was based on the nominal torsional strength.

In order to calculate the torsional strength that the shaft would exhibit during testing, the ACI nominal torsional strength equation was applied. Before the strength was calculated, the minimum requirements for the shaft reinforcement were followed as outlined in ACI 318-05 Section 7.10.5.6 and Section 11.6.5.1. The nominal torsional strength (Equation 2-6) was then calculated for the specimen. This value, 252 kip-ft (342 kN-m), was compared to the concrete breakout strength. The spacing of the hoop steel in the shaft was altered until the nominal torsional strength exceeded the concrete breakout strength. Hence, if the concrete breakout failure was the correct failure mode, it would occur before the torsional capacity of the shaft was exceeded during testing.

3.2.2 Longitudinal and Transverse Reinforcement

As was outlined in the previous section, the required amount of hoop steel to meet the ACI 318-05 specifications was determined using guidelines from Chapters 7 and 11 in the code. The resultant longitudinal steel layout was twenty-four #4 bars spaced evenly around a 27 in. (686 mm) diameter circle. The transverse hoops were comprised of #3 bars at 2.5 in. (635 mm) totaling fourteen #3 bar hoops. The required splice for the #3 bar was 12 in. (305 mm), and the length required to develop the #4 bar into the concrete block was 8 in. (203 mm) with a 6 in. (152 mm) hook. In the test setup, the #4 bars extended 27 in. (686 mm) into the block, which exceeded the required length. This length was used for simplicity in design and construction of the test setup. The #4 bars were tied into one of the cages of reinforcement in the concrete block.

3.2.3 Flexure

Due to the eccentric loading of the bolts, the flexural capacity of the shaft had to be calculated. It had to be determined that the shaft would not fail in flexure under the load applied during testing. The flexural reinforcement in the shaft was the longitudinal reinforcement, the #4 bars. The first step to determine the capacity was to assume the number of bars that would have yielded at the time of failure. From that point, the neutral axis of the shaft was located following the ACI 318-05 concrete stress block methodology presented in Chapter 10 of the code. It was then checked if the number of bars that had yielded was a good assumption. Once this was verified, the nominal moment capacity of the shaft was calculated, and, then, compared to the maximum flexural moment based on the concrete breakout capacity. The flexural capacity of the shaft, 267 kip-ft (355 kN-m), exceeded the maximum flexural moment on the shaft, 60.6 kip-ft (95.2 kN-m).

3.3 Anchor Design

3.3.1 Diameter of Anchor Bolts

The starting point for the diameter of the F1554 Grade 105 anchor bolts to be used in the test apparatus was based on half the diameter of those in the field specimen. The size determined using that methodology was 1 in. (25.4 mm). Once the concrete breakout strength capacity of the anchors was determined, the corresponding shear load on each of the bolts was calculated. The anchor bolt diameter was increased to 1.5 in. (38.1 mm) in order to ensure that the bolts would not experience steel failure in flexure or shear. This resulted in the area of the anchors used in the test being one half of those in the field specimen which is reasonable for a one-half scale model where the area of the anchor is a critical parameter. The maximum flexure on the bolts was calculated by taking the maximum shear applied to each bolt and calculating the corresponding maximum flexural moment (Figure 3-5). The lever arm (Equation 3-1) for the calculation of the capacity was defined in Eligehausen et al. (2006) Section 4.1.2.2 b.

$$l = e_1 + a_3 \tag{3-1}$$

Where:

l = lever arm for the shear load (in.)

e_1 = distance between the shear load and surface of concrete (in.)

a_3 = $0.5 \cdot d_o$, without presence of a nut on surface of concrete, Figure 3-5 (in.)
0, with a nut on surface of concrete

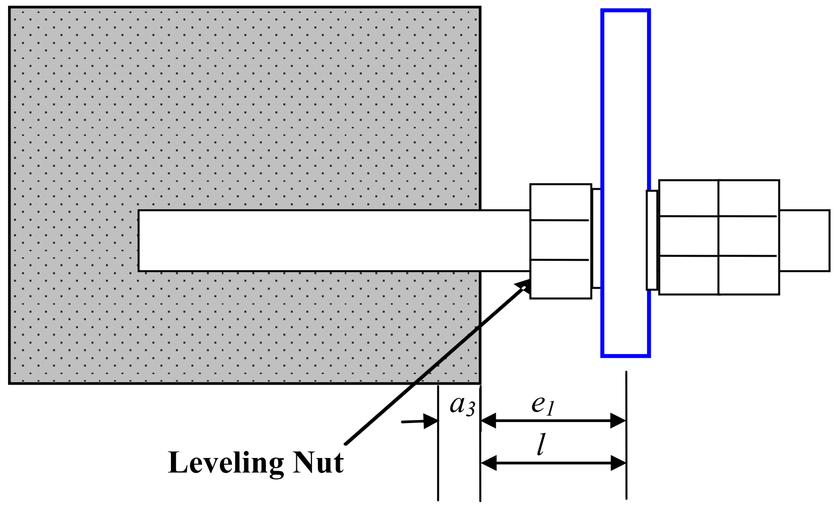


Figure 3-5. Lever arm for the calculation of bolt flexure

The base plate was restrained against rotation, and translation was only possible in the direction of the applied shear load. The maximum applied moment for each bolt was calculated based on these support conditions and the lever arm calculation. Full fixity occurred a distance a_3 into the shaft.

Using the section modulus of the bolts, the stress was then calculated and compared to the yield strength of the bolts, 105 ksi (724 MPa). The shear strength of the bolts was calculated using the provisions in Appendix D of ACI 318-05. In both cases it was determined that the bolts had sufficient strength.

3.3.2 Concrete Breakout Strength of Anchor in Shear Parallel to a Free Edge

The breakout strength provisions outlined in ACI 318-05 Appendix D and the breakout provisions introduced in Eligehausen et al. (2006) were applied to the design of the shaft. Equation 2-11, from ACI 318-05, was used as the primary equation for the calculation of the breakout strength. In order to apply the ACI provisions to the circular foundation a section of the concrete was ignored (Figure 3-6). If the full cover, *c*, was used in the calculation, the failure region would have included area outside of the circle. Rather than extending beyond the edge of the concrete, the ≈35 degree failure cone (Figure 2-6) was extended to the edge of the shaft as shown in Figure 3-6. Equation 3-2 was developed to determine the adjusted cover, c_{a1}.

$$c_{a1} = \frac{\sqrt{r_b^2 + 3.25\left(r^2 - r_b^2\right)} - r_b}{3.25} \qquad (3\text{-}2)$$

Where

r_b = radius measured from the centerline of the bolt to the center of the foundation (in.) (Figure 3-6)

r = radius of circular foundation (in.)

As presented in Section 2.3.4, the projected concrete failure area for a single anchor, A_{Vco}, is equivalent to $4.5(c_{a1})^2$. Figure 3-7 illustrates the development of the projected concrete failure area for a group of anchors, A_{Vc}, as a function of the number of bolts, n, the radius of the shaft, r, and the adjusted cover. The resultant concrete breakout strength using the adjusted cover approach was conservative relative to assuming the full cover.

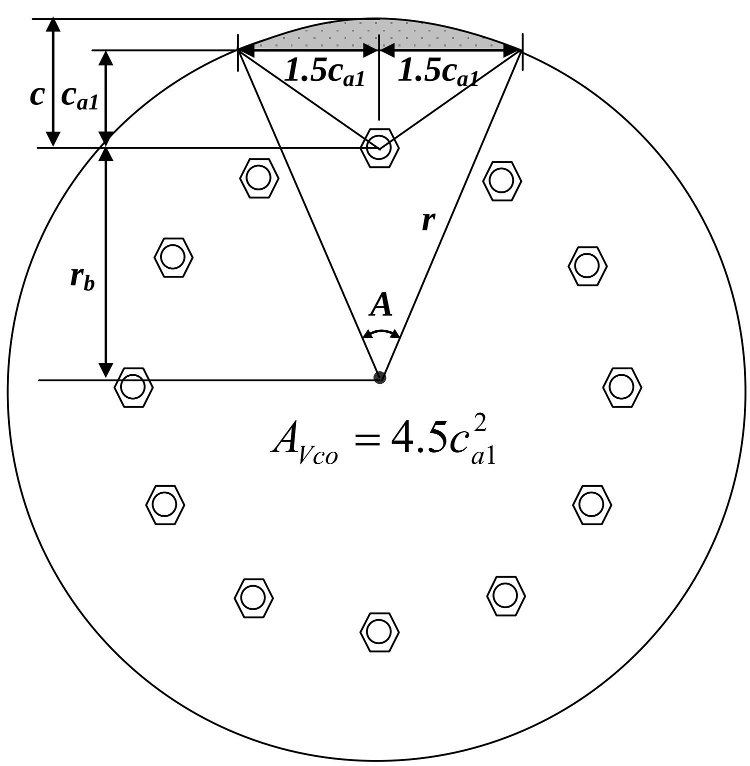


Figure 3-6. Adjusted cover based on a single anchor and ≈35° failure cone

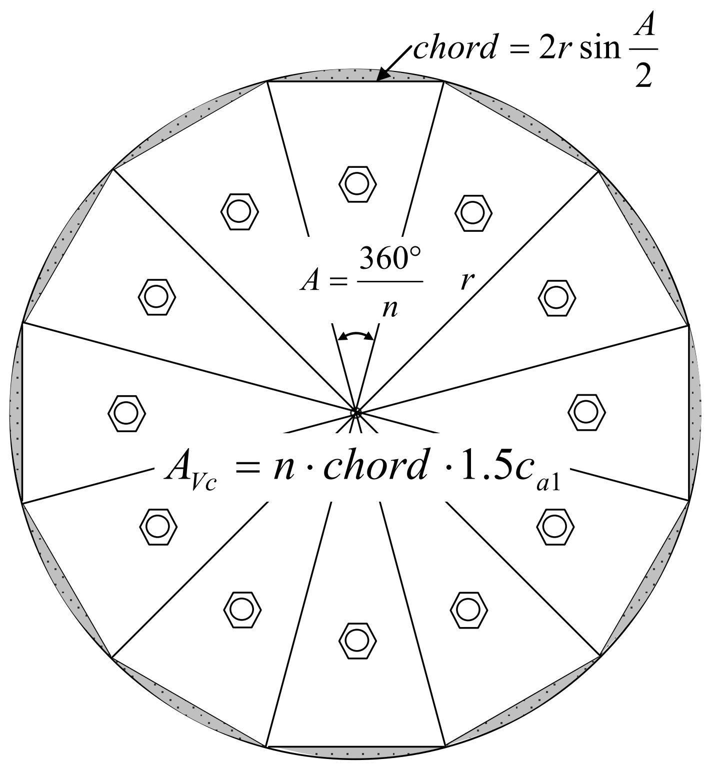


Figure 3-7. Development of the projected failure area for the group of anchors around a circular foundation

Equation 3-3 and Equation 3-4 are used to calculate the concrete breakout torsion, $T_{n,breakout}$, and are based on the ACI provisions for shear parallel to the free edge.

For $A \le \sin^{-1}\left(\frac{1.5c_{a1}}{r_b}\right)$

$$T_{n,breakout} = 2 \cdot \frac{A_{Vc}}{A_{Vco}} V_b \cdot r_b \tag{3-3}$$

For $A > \sin^{-1}\left(\frac{1.5c_{a1}}{r_b}\right)$ (i.e. no overlap of failure cones)

$$T_{n,breakout} = 2 \cdot n \cdot V_b \cdot r_b \tag{3-4}$$

Where

A = angle of circular sector for each bolt (deg) (Figure 3-7)
c_{a1} = adjusted cover (in.) (Equation 3-2)
r_b = radius measured from the centerline of the bolt to the center of the foundation (in.) (Figure 3-6)
A_{Vc} = projected concrete failure area of a group of anchors (in.^{2}) (Figure 3-7)
A_{Vco} = projected concrete failure area of a single anchor (in.^{2}) (Figure 3-6)

V_b = concrete breakout strength in shear for a single anchor calculated using Equation 2-11 with c_{a1} as calculated in Equation 3-2 (lb.)
n = number of bolts

During the analysis of the design equations, an issue arose regarding the calculation of the factor $\psi_{parallel}$. The result of Equation 2-18 was 4.06 compared to the ACI 318-05 factor and $\psi_{\alpha,V}$ of 2.0. This prompted an investigation of the application of the multiplier to the circular foundation in this research program.

The majority of the tests for the determination of $V_{u,c}$ (Equation 2-14) were for groups of two bolts. Therefore, it was investigated how the A_{Vc}/A_{Vco} term is affected by the spacing between the bolts and the number of bolts. Figure 3-8 shows that for spacing, s, of $3.0c_{a1}$ or greater there is no overlap of the breakout cones. In those cases the strength is the sum of the single anchor strengths. Figure 3-9 illustrates the overlap of the breakout cones. The A_{Vc}/A_{Vco} term is used to calculate the breakout strength for the case where the failure cones overlap.

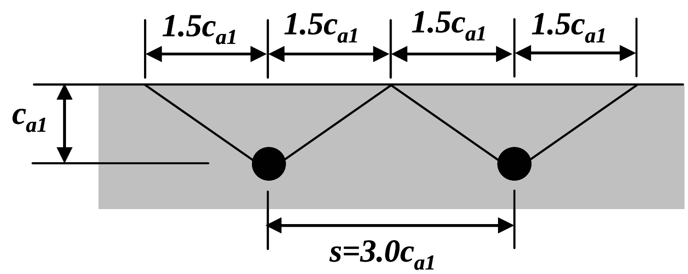


Figure 3-8. Two anchor arrangement displays the minimum spacing such that no overlap of the failure cones occurs

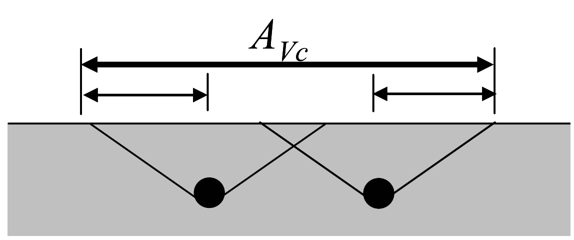


Figure 3-9. Overlap of failure cones

A_{Vc}/A_{Vco} can be normalized by dividing by the number of bolts. An increase in the number of bolts at the same spacing along a straight edge leads to a reduction in the normalized A_{Vc}/A_{Vco} term. This reduction is illustrated in Figure 3-10. The contribution of the failure cone outstanding "legs" at the ends of the group area, A_{Vc}, decreases as the number of bolts increases. For a circular foundation, with $s<3.0c_{a1}$, there is a constant overlap of the failure cones with no outstanding "legs" (Figure 3-11). The equivalent number of bolts along a straight edge is taken as infinity in order to represent a circular foundation (i.e. no outstanding "legs"). Therefore, the normalized A_{Vc}/A_{Vco} term for this case was calculated for an infinite number of bolts at the prescribed spacing for the foundation. To convert these ratios into a multiplier for $\psi_{parallel}$, the ratio of the normalized A_{Vc}/A_{Vco} for an infinite number of bolts to the normalized term for two bolts was calculated. That multiplier, 0.52, was applied to the $\psi_{parallel}$ term resulting in an adjusted $\psi_{parallel}$ of 2.1. This resulting value agreed with the ACI 318-05 factor and the Eligehausen et al. (2006) factor $\psi_{\alpha,V}$ of 2.0.

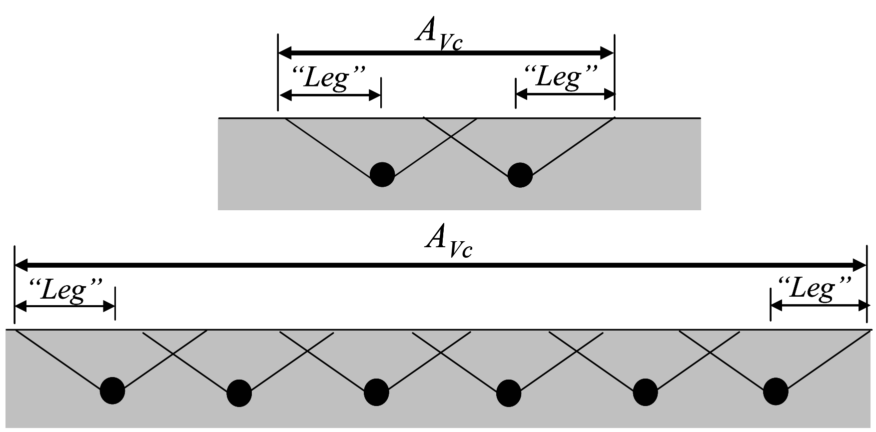


Figure 3-10. The contribution of the "legs" of the failure cone to A_{Vc} along a straight edge decreases as the number of bolts increases

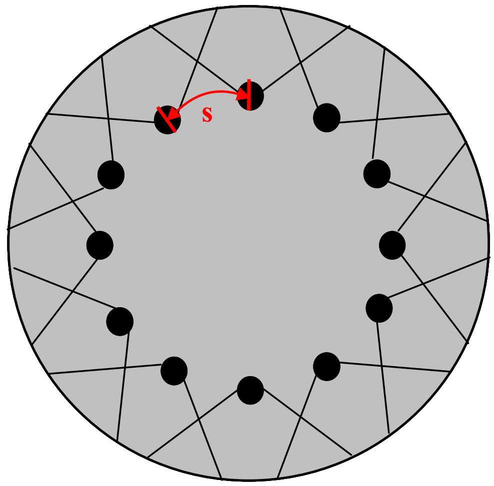


Figure 3-11. Overlap of failure cones for a circular foundation

The resultant concrete breakout torsions, based on the Eligehausen et al. (2006) concrete breakout strength (Equation 2-13), were 167 kip-ft (227 kN-m) using $\psi_{parallel}$ of 2.1 in Equation 2-17, and 159 kip-ft (216 kN-m) using $\psi_{\alpha,V}$ of 2.0 in Equation 2-16. These torsions were calculated using the same moment arm, r_b, used in Equation 3-3 and Equation 3-4. These results, in addition to the results of the other calculations, are summarized in Table 3-2.

Table 3-2. Summary of design calculations

Component	Design Type	Equation Reference	Result
Shaft			
	Cracking Torsion	(2-1)	131 kip-ft
	Basic Torsion	(2-2)	131 kip-ft
	Threshold Torsion	(2-3)	24.6 kip-ft
	Nominal Torsion	(2-4)	252 kip-ft
Anchor			
	ACI Concrete Breakout	(2-12)	182 kip-ft
	Eligehausen et. al. Concrete Breakout	(2-16)	159 kip-ft
	Eligehausen et. al. Concrete Breakout	(2-17)	167 kip-ft
	Bolt Flexure		253 kip-ft
	Bolt Shear		1756 kip-ft

3.3.3 Development Length of the Bolts

Another key aspect of the shaft design was to ensure that the anchor bolts were fully developed. As discussed in ACI 318-05 D.4.2.1 and RD.4.2.1, this can be accomplished by providing reinforcement fully anchored on each side of the breakout plane. In the case of the shaft, the breakout plane originates at the head of the anchors. In order to meet the code requirements, the splice length between the #4 bars and anchor bolts was calculated using the development length equations presented in ACI 318-05 Chapter 12. The bolts needed overlap the #4 bars across 26.7 in. (678 mm), and in the test setup the overlap was 29 in. (737 mm). Therefore, this requirement was met.

3.4 Steel Pipe Apparatus Design

The components of the steel pipe apparatus included the pipe, which was loaded during testing, and the base plate. The pipe design was based on the interaction between torsion, flexure, and shear as presented in AISC (2001), *LRFD Manual of Steel Construction-LRFD Specification for Steel Hollow Structural Sections*. Each of the individual capacities was calculated for various pipe diameters and thicknesses. The individual strengths were compared to the projected failure loads for testing, the concrete breakout failure loads. In addition to verifying that the capacity of the pipe exceeded those loads, the interaction of the three capacities was verified. The purpose was to check that the sum of the squares of the ultimate loads divided by the capacities was less than one. Based on this analysis, it was concluded that an HSS 16.000 x 0.500 pipe would provide sufficient strength.

In order to load the pipe, it needed to have a ninety degree bend in it. This was achieved by welding two portions of pipe cut on forty-five degree angles to a steel plate. The weld size for this connection was determined such that the effective throat thickness would equal the thickness of the pipe, which was 0.50 in. (12.7 mm).

The factors included in the design of the base plate were the diameter of the pipe, required weld size, bolt hole diameter, and the required distance between the edge of the bolt hole and the edge of the plate. The required width of the weld between the base plate and the pipe was calculated such that the applied torsion could be transferred to the plate without failing the weld. From that point, the bolt hole location diameter had to be checked to ensure that there was sufficient clearance between the weld and the nuts. It was important that the nuts could be fully tightened on the base plate. A 0.25 in. (6.35 mm) oversize was specified for the bolt hole diameter. The oversize for the bolt hole diameter was based on the FDOT Standard Drawing No. 11310 for cantilever sign structures. Note 12 on the drawing specifies that the maximum allowable oversize for anchor bolts is 0.50 in. (12.7 mm). The oversize dimension for the test specimen was taken as one half of the size of the maximum allowable, 0.25 in. (6.35 mm). Beyond that point, it was ensured that there would be sufficient cover distance between the bolt hole and the edge of the plate.

The design of the components of the steel pipe apparatus was crucial because these pieces had to operate efficiently in order to correctly apply load to the bolts. If the apparatus were to fail during testing, the objective of the research could not be achieved. The weight of the pipe apparatus was calculated in order to normalize the load during testing. The load applied to the anchorage would be the load cell reading less the weight of the pipe apparatus.

3.5 Concrete Block Design

The design of the concrete block was based on several key factors to ensure that it served its purpose as a fixed support at the base of the shaft. The amount of reinforcement required was based on a strut-and-tie model of the block as outlined in ACI 318-05 Appendix A and, as an alternate approach, beam theory to check the shear strength and flexural strength of the block. For the flexural capacity calculations, the ACI 318-05 concrete stress block provisions were utilized. Based on the results of both approaches, it was determined that 3 #8 bars, each with a 12 in. (305 mm) hook on both ends, spaced across the top and the bottom of the block were required. Additionally, two cages of #4 bars were placed in the block on the front and back faces meeting the appropriate cover requirements to serve as supplementary reinforcement. The purpose of reinforcing the block was to ensure structural stability of the block throughout the testing process.

Two channel apparatuses were provided to tie the block to the floor of the laboratory to resist overturning. The loads that had to be resisted by each tie-down were calculated such that the floor capacity of 100 kips (445 kN) per tie-down would not be exceeded. The channels were designed in accordance with the provisions set forth in the AISC (2001). In addition to assuring that the concrete block system had sufficient capacity to resist the applied load, the bearing strength of the concrete had to be calculated. This was done in order to verify that the concrete would not fail in the region that was in contact with the steel channels. The bearing strength was found to be sufficient. As a result, it was concluded that the concrete block system would efficiently serve as a fixed connection

3.6 Combined Shear and Torsion

As was presented in Chapter 2, a calculation was made to ensure that shear did not need to be considered in the design. Rather than inputting the values for the ultimate shear and ultimate torsion into Equation 2-8, the coefficients of these terms were calculated. The base for doing so was to input the torsion as a function of the shear. For the test specimen, the ultimate torsion, T_u, was taken as the moment arm multiplied by the ultimate shear, V_u. The moment arm for the load was 9 ft. (2740 mm). As an alternate approach, the actual concrete breakout strength and the corresponding shear could have been inputted into the equation rather than the generic variables. The result of the calculation to determine the coefficients was that the coefficient for the shear term was 1 compared to a coefficient of 88 for the torsion term. This calculation sufficiently verified that the shear contribution could be ignored in design.

3.7 Overview

The previous sections detailed the design of the various components of the experimental program. It was of the utmost importance that the concrete block system and pipe apparatus would not fail during testing. Furthermore, all other foundation failure modes had to be eliminated. This ensured that if shear breakout failure in shear was the failure mode, it would be observed during testing.

Figure 3-12 and Figure 3-13 show the fully assembled test specimen at the Florida Department of Transportation Structures Research Center.

Figure 3-12. Assembled test specimen

Figure 3-13. Shaft with pipe apparatus attached prior to instrumentation being attached

CHAPTER 4
IMPLEMENTATION OF TESTING PROGRAM

In order to proceed with the testing of the specimen presented in Chapter 3, important considerations had to be made. This chapter covers both materials and instrumentation. The material considerations were the concrete strength, bolt strength, and carbon fiber reinforced polymer (CFRP) wrap. The instrumentation used for testing were linear variable displacement transducers (LVDTs) and strain gages.

4.1 Materials

4.1.1 Concrete Strength

The initial calculations for the design of the test setup were based on an assumed concrete strength of 5500 psi (37.9 MPa). The concrete breakout strength was recalculated based on the concrete strength at the time of testing. On the date of the test, the concrete strength was 6230 psi (43 MPa). This strength was determined from the average of three 6 in. (152 mm) x 12 in. (305 mm) cylinder tests.

4.1.2 Bolt Strength

The yield strength of the F1554 Grade 105 anchor bolts was assumed to be 105 ksi (723.95 MPa). This was the strength used to calculate the flexural and shear strengths of the bolts.

4.1.3 Carbon Fiber Reinforced Polymer Wrap

The first test was stopped after significant cracking and the test specimen quit picking up additional load. The loading was ceased before the specimen completely collapsed. This allowed a second test to be performed on the specimen after it was retrofitted with a carbon fiber reinforced polymer (CFRP) wrap. The second test verified that a CFRP wrap is an acceptable means to retrofit the failed foundation.

The amount of CFRP that was applied to the shaft was determined by calculating the amount of CFRP required to bring the shaft back to its initial concrete breakout strength, $T_{n,breakout}$. The CFRP wrap that was used for the retrofit was SikaWrap Hex 230C. The property specifications for the SikaWrap were based on the mean strength minus 2 standard deviations. ACI 440.R-02 Section 3.3.1 specifies that the nominal strength to be used for design be based on the mean strength less 3 standard deviations. Therefore, the design strength provided by Sika was adjusted to ensure that the design met the ACI specifications.

Two methods were employed to determine the amount of the CFRP fabric that had to be applied around the foundation to return it to the initial concrete breakout strength. The equations were developed for a generic torsion, T. The first method, Figure 4-1, for calculating the amount of CFRP required was to convert the torsion to a shear load per bolt, $V_{parallel}$, Equation 4-1.

$$V_{parallel} = \frac{T}{r_b n} \tag{4-1}$$

Where

$V_{parallel}$ = shear load directed parallel to a free edge per bolt (kip)

T = torsion (kip-ft.)

r_b = radius measured from the centerline of the bolt to the center of the foundation (in.) (Figure 3-6)

n = number of bolts

The shear load, which was directed parallel to the edge, had to be converted to a load perpendicular to the edge. In order to do this, the load was divided by the ACI multiplier of 2 specified in ACI 318-05 Section D.5.2.1(c), Equation 4-2. As previously stated, the factor of 2 was based on tests that indicated that the concrete breakout strength for a shear load directed parallel to the edge is two times that of a shear load directed perpendicular to the edge.

$$V_{perpendicular} = \frac{V_{parallel}}{2} = \frac{T}{2 r_b n} \tag{4-2}$$

That load per bolt directed perpendicular to the edge, $V_{perpendicular}$, was converted to a pressure around the circumference of the shaft, p_r, Equation 4-3.

$$p_r = V_{perpendicular} \frac{n}{2\pi r} = \frac{T}{4\pi r_b r} \qquad (4\text{-}3)$$

The equivalent tension that had to be resisted by the CFRP wrap was then calculated. Equation 4-4 is the simplified equation for this tension.

$$F_{CFRP} = p_r\, r = \frac{T}{4\pi r_b} \qquad (4\text{-}4)$$

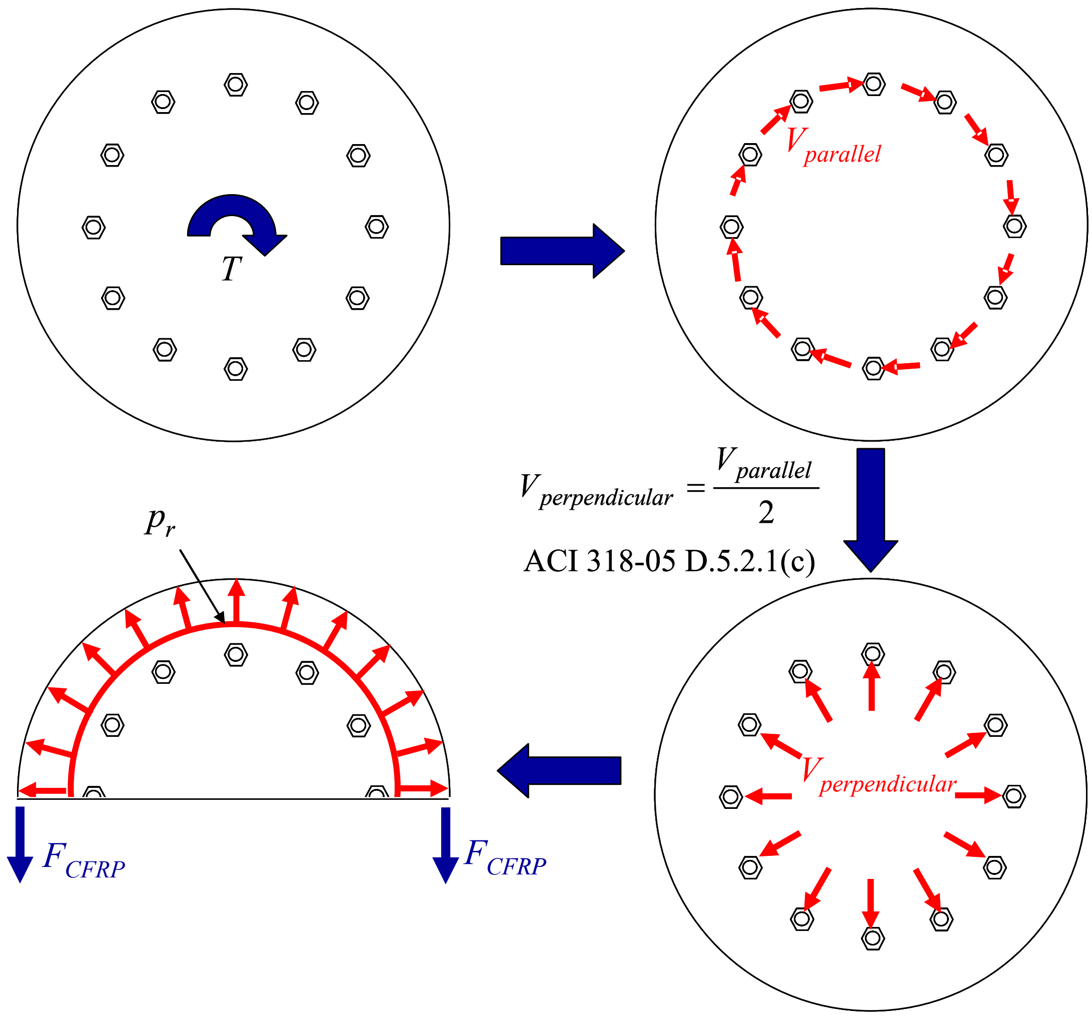


Figure 4-1. Method for the determination of the tension, F_{CFRP}, that must be resisted by the CFRP wrap using internal pressure

An alternate method to calculate the tension that must be resisted by the CFRP wrap was based on a strut-and-tie model, Figure 4-2.

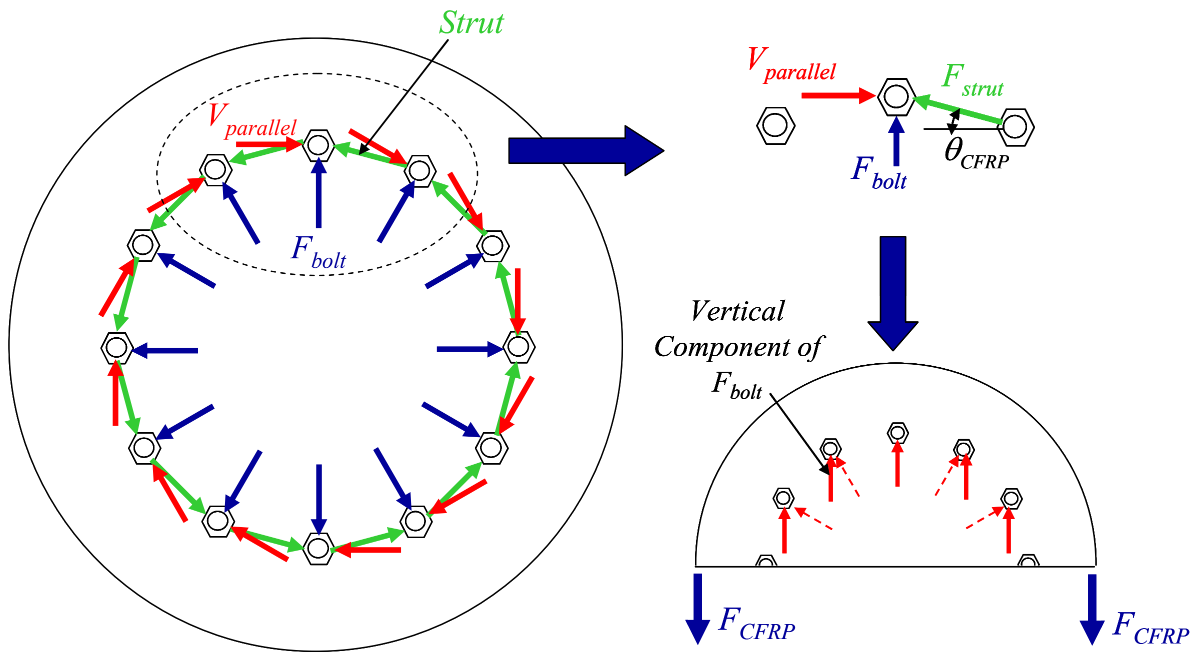


Figure 4-2. Alternate strut-and-tie method to calculate the tension, F_{CFRP}, that must be resisted by the CFRP wrap

Compression struts were directed between the bolts and ties were formed at the bolt location directed perpendicular to the edge. The forces were calculated using the angles formed by the circular sector for the individual bolts. The compression strut force, F_{strut}, was calculated using Equation 4-5 with the shear force directed parallel to the edge, $V_{parallel}$, as calculated using Equation 4-1.

$$F_{strut} = \frac{V_{parallel}}{\cos\theta_{CFRP}} = \frac{T}{r_b n \cos\theta_{CFRP}} \tag{4-5}$$

Where

$$\theta_{CFRP} = \frac{180^{\circ}}{n}$$

The force per bolt, F_{bolt}, directed normal to the edge was taken as a component of F_{strut}, Equation 4-6.

$$F_{bolt} = F_{strut} \sin \theta_{CFRP} = \frac{T}{r_b n} \tan \theta_{CFRP} \tag{4-6}$$

The value of T_{CFRP} for this method was determined by cutting the section in half and calculating the vertical components of F_{bolt} at each bolt location. Equation 4-7 is the simplified equation for F_{CFRP} after the substitutions were made for the calculation of the vertical components.

$$F_{CFRP} = \frac{T}{2 r_b n} \tag{4-7}$$

As previously mentioned, the concrete breakout strength, $T_{n,breakout}$, as calculated in Equations 3-3 and 3-4 was substituted for T in Equations 4-1 through 4-7 to determine the amount of CFRP that needed to be applied to the foundation. The tension calculated using the first method was 18.5 kip (82.1 kN) compared to a tension of 9.7 kip (43 kN) using the strut-and-tie approach. The prompted an investigation into the two approaches. First, the tension that must be resisted by the CFRP, F_{CFRP}, was plotted against the number of bolts, Figure 4-3.

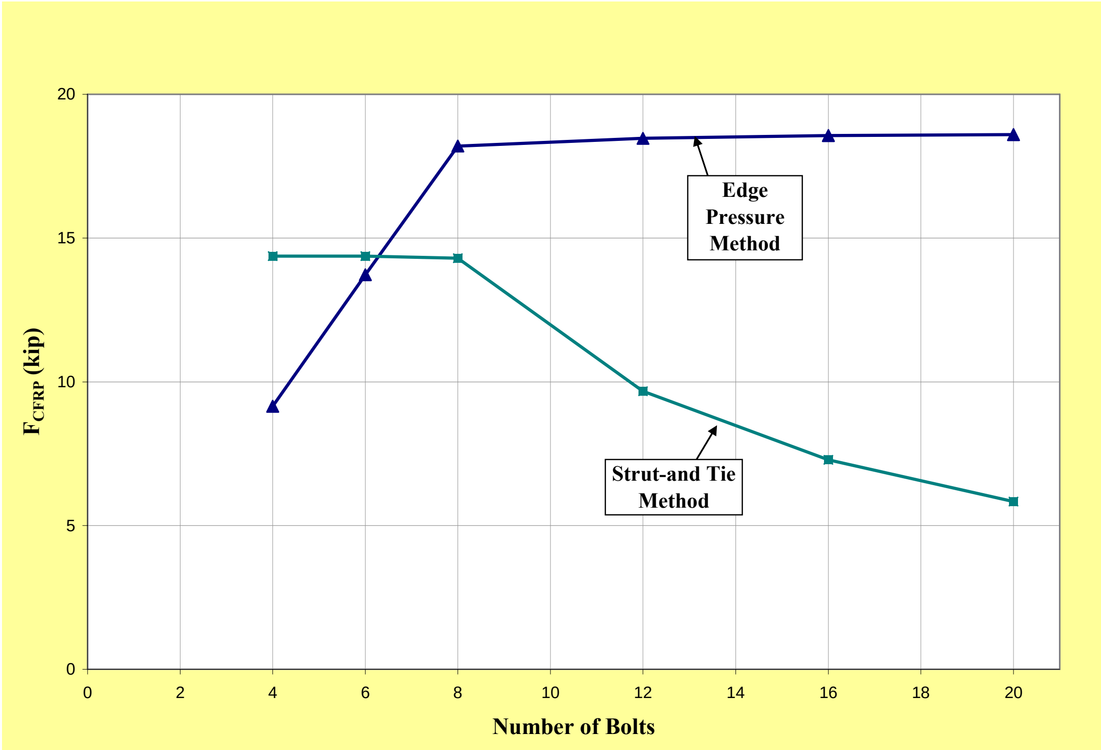


Figure 4-3. Comparison of methods for the calculation of F_{CFRP} for $T_{n,breakout}$

The plot illustrates that for up to six bolts the strut-and-tie method provides the more conservative result while beyond six bolts the edge pressure method provides the more conservative result.

Note that both equations, Equations 4-4 and 4-7, contain the terms T and r_b. A normalized plot of the two equations was developed by dividing out the term T/r_b, Figure 4-4. This plot illustrates that the edge pressure method is constant as compared to the strut-and-tie method which is inversely proportional to the number of bolts. Beyond six bolts (where overlap of the failure cones begins), it is recommended that until further testing is performed the tension in the CFRP be calculated using the edge pressure method. The strut-and-tie method is recommended for use when the failure cones do not overlap.

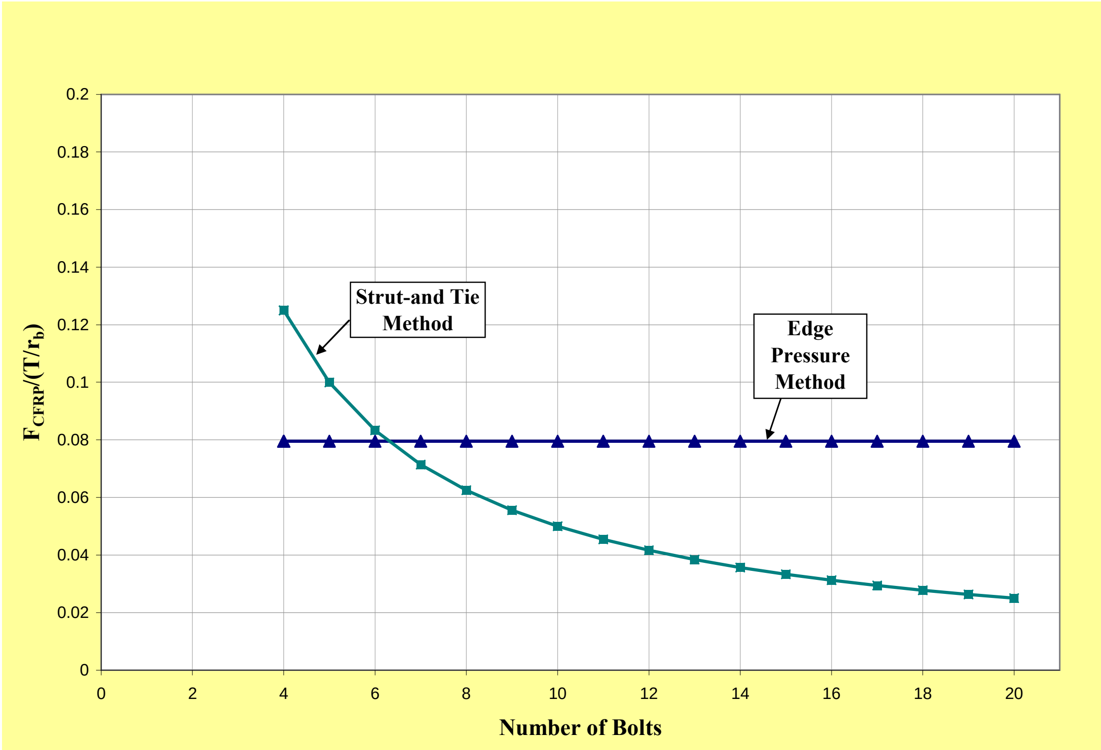


Figure 4-4. Normalized comparison of methods for the calculation of F_{CFRP}

Based on this analysis, the more conservative edge pressure method was used for the determination of F_{CFRP} for the test specimen since there were twelve bolts in the foundation. It should be noted that the strut-and-tie method is rationally based and in-fact may be the best method for determining the tension in the CFRP for all numbers of anchors. Until further testing is performed the edge pressure method was selected since it is more conservative when large numbers of anchors are used. The value of F_{CFRP} used in the analysis was 18.5 kip (82.1 kN).

It was assumed that the full 12 in. (305mm) width of the CFRP wrap would not be effective. Rather, the effective width was taken as the depth of the concrete breakout failure cone, *1.5·cover*. The resultant effective width was 7.5 in. (191mm). Using the effective width, two layers of the wrap were required to meet the ACI concrete breakout strength. Three layers of the CFRP wrap were applied to the specimen. The addition of the extra layer exceeded the required strength, so it was deemed acceptable. Once the wrap was set, the retrofit test was carried out. Calculations for the design of the CFRP wrap layout are located in Appendix B.

4.2 Instrumentation

4.2.1 Linear Variable Displacement Transducers

Linear Variable Displacement Transducers (LVDTs) were placed at the location of the load cell, and at various points along the shaft and base plate. A total of ten LVDTs were utilized in the project. Figure 4-5 is a schematic of the layout of the LVDTs on the base plate. Figure 4-6 and Figure 4-7 show the location of the LVDTs on the shaft, and Figure 4-8 shows the LVDT at the load location. The label for each of the LVDTs is also on the drawings. These identification codes were used to denote the LVDTs during testing. The purpose of the LVDTs along the shaft and base plate was to measure the rotation of the base plate during testing. The LVDTs at the front and back of the shaft were to allow for the rotation to be measured relative to the rotation of the shaft. The horizontal LVDT on the base plate was intended to indicate if there was any horizontal movement of the base plate. The rotation of the base plate was calculated using Equation 4-8.

$$R = \tan^{-1}\left(\frac{D_{1V} + D_3}{D_{gage}}\right) \tag{4-8}$$

Where

R = base plate rotation (rad)

D_{1V} = displacement of LVDT D1V (in.)

D_3 = displacement of LVDT D3 (in.)

D_{gage} = distance between LVDTs D1V and D3 (in.)

Once the test apparatus was assembled, the distance D_{gage} was measured. This distance was 26.31 in. (668 mm). Figure 4-9 shows LVDTs D1V and D4 on the test specimen.

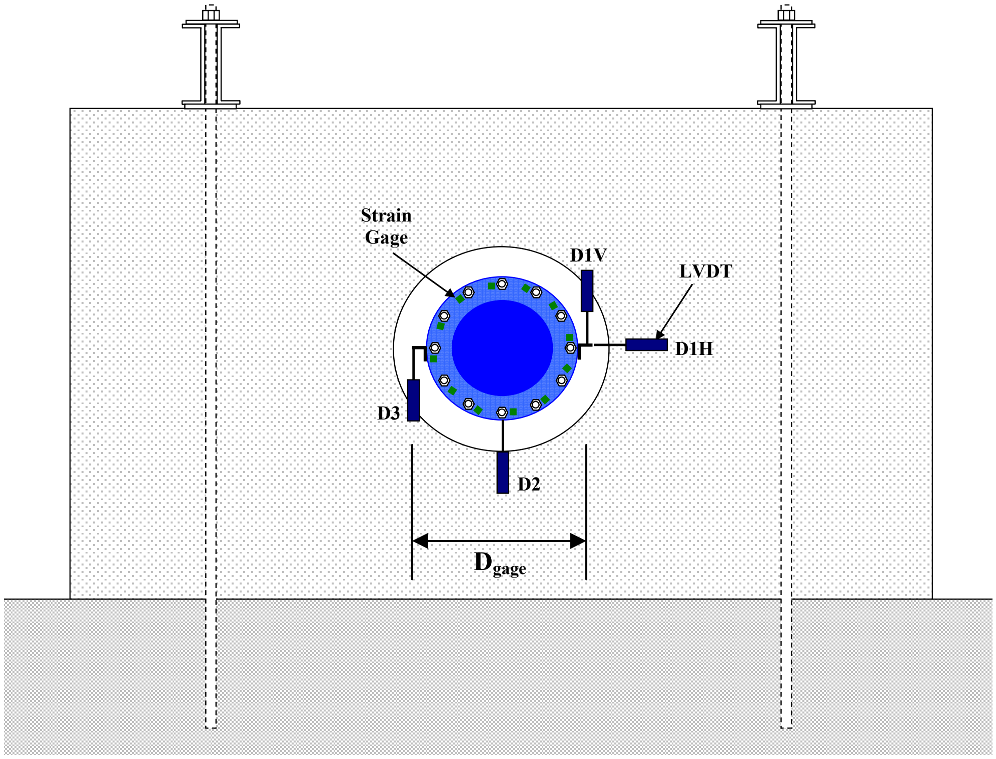


Figure 4-5. Instrumentation layout on the base plate

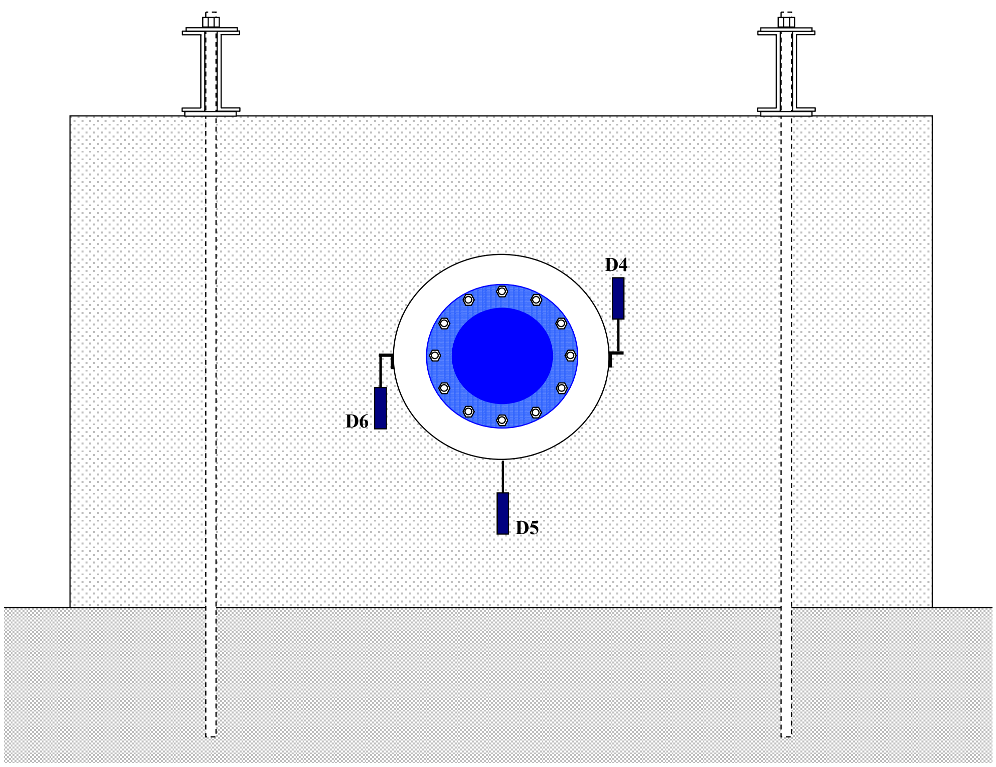


Figure 4-6. Instrumentation layout on face of shaft

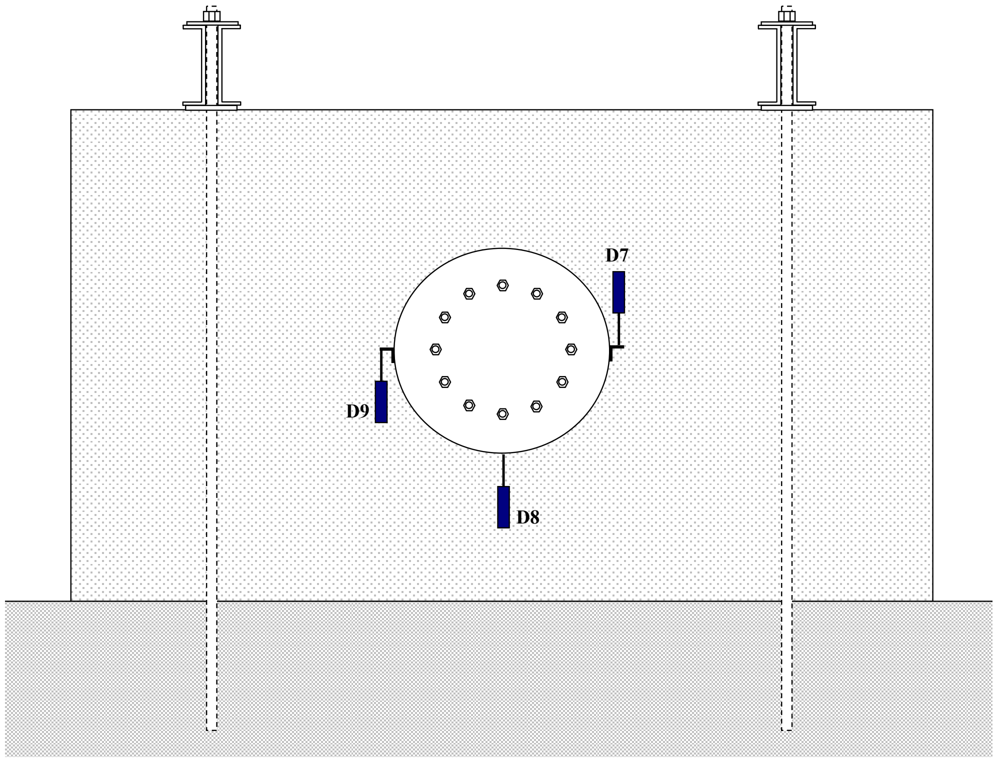


Figure 4-7. Instrumentation layout on rear of shaft/face of concrete block

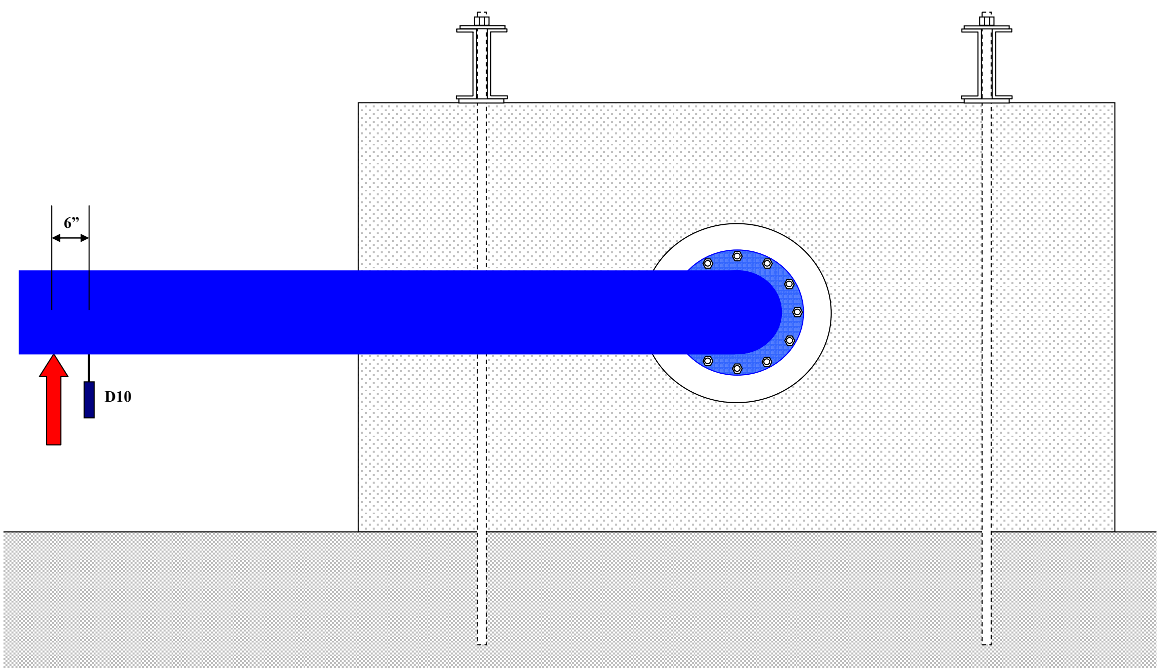


Figure 4-8. Instrumentation layout of pipe at load location

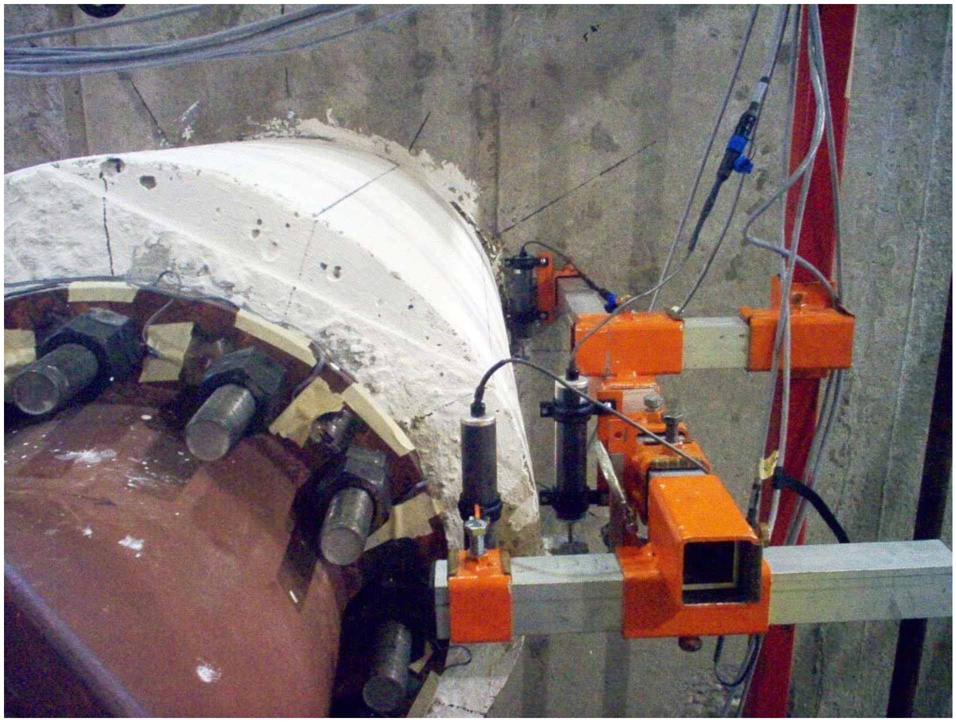


Figure 4-9. Location of LVDTs D1V, D4, and D7 on the test specimen.

4.2.2 Strain Gages

Strain gages were attached to the base plate on the outer surface adjacent to the bolt holes in order to determine how may bolts were actively transferring load given the 1.75 in. (44.5 mm) holes for the 1.5 in. (38.1 mm) anchors. In applying the ACI 318-05 equation for concrete breakout strength of an anchor in shear directed parallel to an edge (Equation 2-12) it was of key importance to know how many bolts were carrying the load. For instance, if two bolts were carrying the load, the concrete would fail at a lower load than if all twelve bolts were carrying the load. In addition to showing the placement of the LVDTs, Figure 4-5 also details the location of the strain gages on the base plate. Figure 4-10 shows the denotation of the strain gages relative to the bolt number, and Figure 4-11 shows a strain gage on the base plate of the test specimen. Note that the bolt numbering starts at one at the top of the plate and increases as you move clockwise around the base plate.

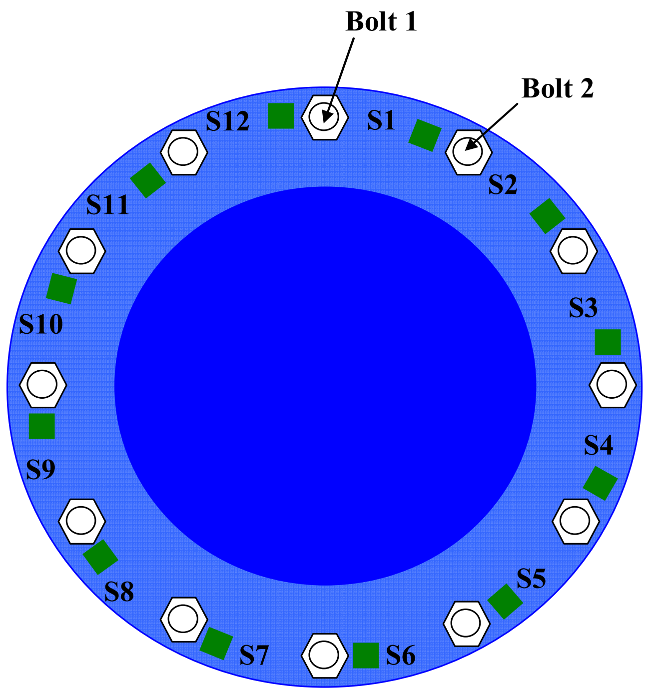


Figure 4-10. Strain gage layout on base plate

Figure 4-11. Strain gage on base plate of test specimen

CHAPTER 5
TEST RESULTS

Two tests were performed on the test specimen. The initial test was conducted to determine whether the concrete breakout failure was the failure mode demonstrated in the field. The verification of this was based on the crack pattern and the failure load recorded. If the failure torsion was the concrete breakout failure torsion, then the hypothesized failure mode would be verified. The retrofit test was performed on the same test specimen. This test was completed to establish whether a CFRP wrap was an acceptable retrofit for the foundation.

5.1 Initial Test

5.1.1 Behavior of Specimen During Testing

The initial test on the foundation was carried out on 31 August 2006 at the Florida Department of Transportation Structures Research Center. The test specimen was gradually loaded during the testing. Throughout the test, the formation of cracks on the surface of the concrete was monitored (Figures 5-1 and 5-2). At 90 kip-ft (122 kN-m), the first cracks began to form. When 108 kip-ft (146 kN-m) was reached, it was observed that the cracks were not extending further down the length of the shaft. Those cracks that had formed began to widen slightly. These cracks, Figure 5-1, were characteristic of those that formed during the concrete breakout failure. At 148 kip-ft (201 kN-m), cracks spanning between the bolts had formed (Figure 5-3). The foundation continued to be loaded until the specimen stopped taking on more load. The torsion load peaked at 200 kip-ft (271 kN-m). Loading ceased and was released when the applied torsion fell to 190 kip-ft (258 kN-m). The predicted concrete breakout capacity of the shaft at the time of testing was calculated as 193 kip-ft (262 kN-m) (Equation 3-3).

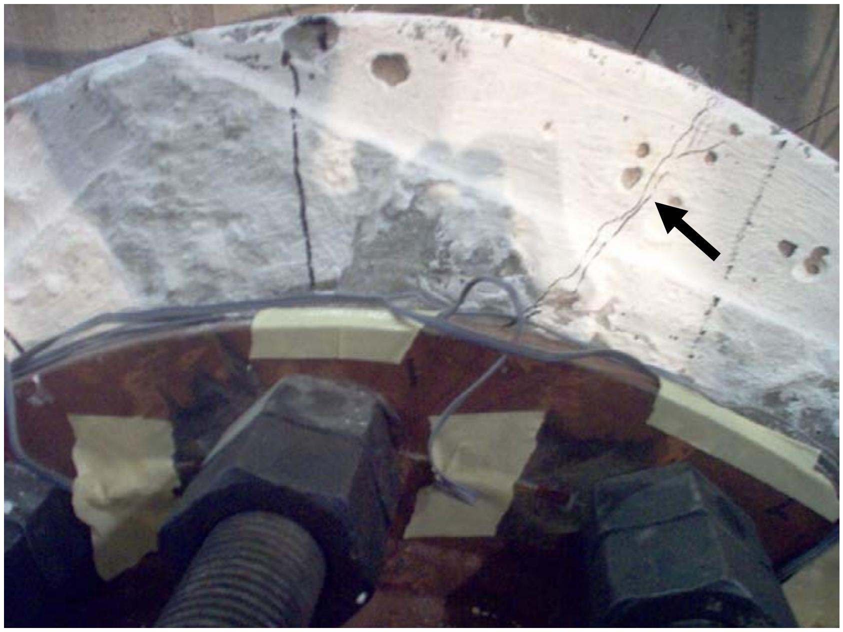


Figure 5-1. Initial cracks on face of shaft

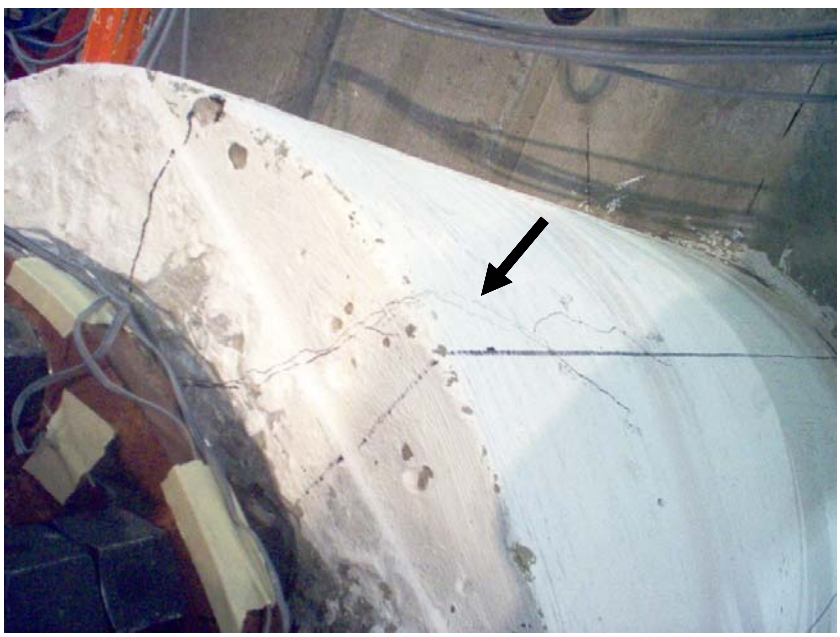


Figure 5-2. Initial cracks on face and side of shaft (alternate view of Figure 5-1)

Figure 5-3. Face of test specimen after testing exhibits cracks between the bolts along with the characteristic concrete breakout cracks

At failure, the foundation displayed the characteristic cracks that one would see in a concrete breakout failure (Figure 5-4). As intended, the bolts did not yield, and the shaft did not fail in torsion. Data was reduced to formulate applied torsion versus plate rotation and applied torsion versus plate strain plots. The Applied Torsion vs. Plate Rotation plot (Figure 5-5) shows that the bolts ceased taking on additional load after the noted concrete breakout failure due to the shear parallel to the edge resulting from the applied torsion. It also exhibits slope changes at the loads where crack development started or the existing cracks were altered. The first slope change at 108 kip-ft (146 kN-m) coincided with the widening of the characteristic diagonal cracks on the front face of the shaft. The second change occurred at 148 kip-ft (201 kN-m) corresponding with the formation of cracks between the bolts.

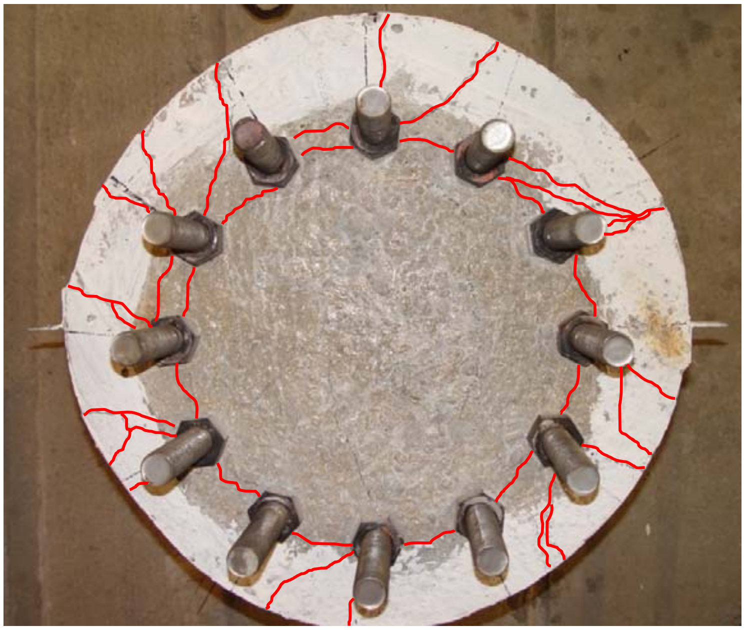


Figure 5-4. Crack pattern on face of shaft after testing depicts characteristic concrete breakout failure cracks

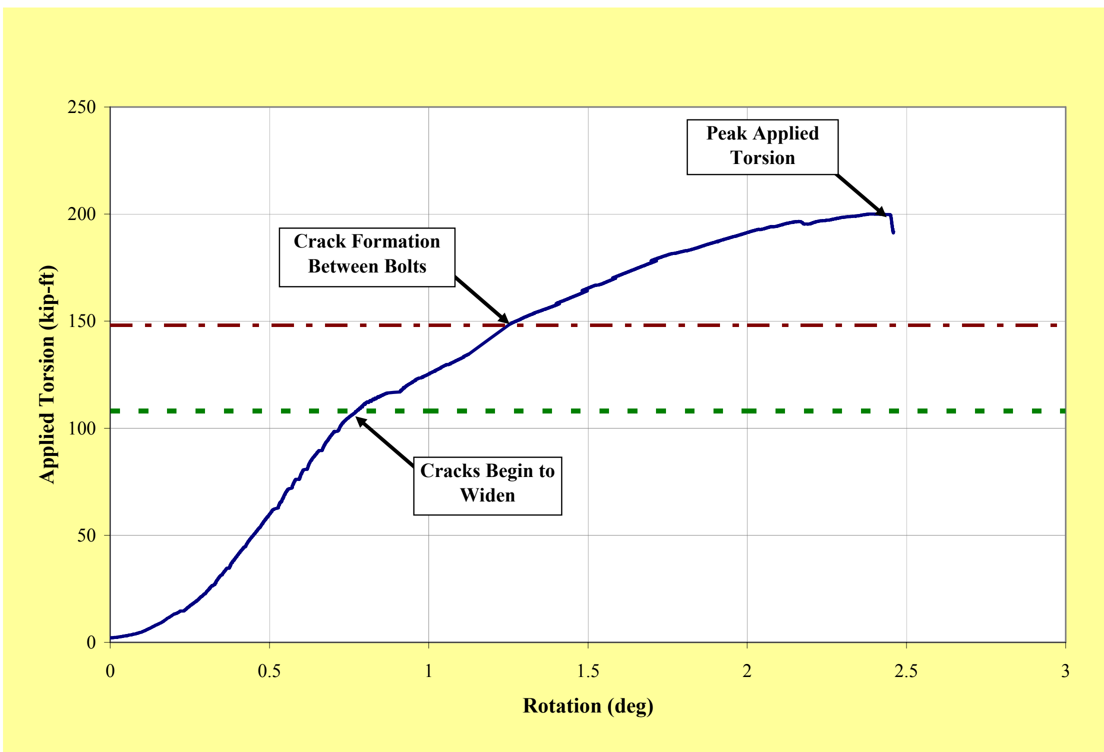


Figure 5-5. Applied Torsion vs. Plate Rotation Plot- Initial Test

5.1.2 Behavior of Strain Gages During Testing

Figure 5-6 displays the Applied Torsion vs. Plate Strain plots for strain in the top of the base plate for each bolt relative to its location on the foundation. The strain was a result of the bolt carrying load. The first line on the plots in Figure 5-6 is 50 kip-ft (67.8 kN-m). At this level, all of the bolts appear to be carrying load with the exception of bolts one, six, and eight. At the next level, 100 kip-ft (136 kN-m) bolt one picked up load, but bolts six and eight remained inactive.

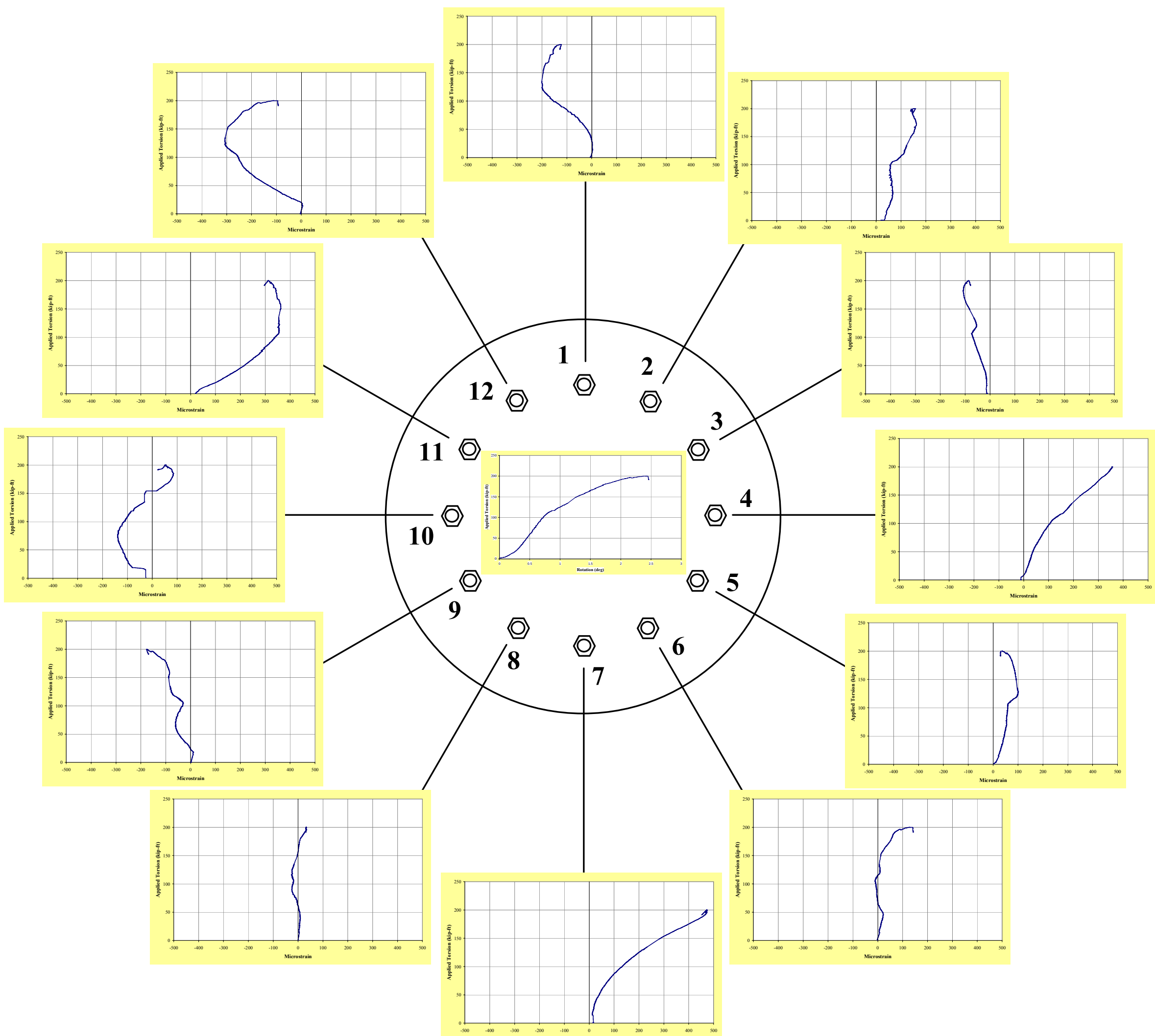


Figure 5-6. Applied Torsion vs. Plate Strain Plots for each bolt at the appropriate location on the base plate with Applied Torsion vs. Plate Rotation plot in center (full size plots in Appendix C)

It should be noted that, at 108 kip-ft (138 kN-m), which was the first slope change on the Applied Torsion vs. Plate Rotation Plot, a redistribution of the loading occurred. This redistribution is illustrated in Figure 5-7. As the cracks widened, those bolts that were transferring the majority of the load were able to move more freely, and, therefore, the other bolts became more active in transferring the load to the foundation. A similar redistribution to a lesser degree occurred at approximately 148 kip-ft (201 kN-m), which coincided with the first observation of cracks between the bolts.

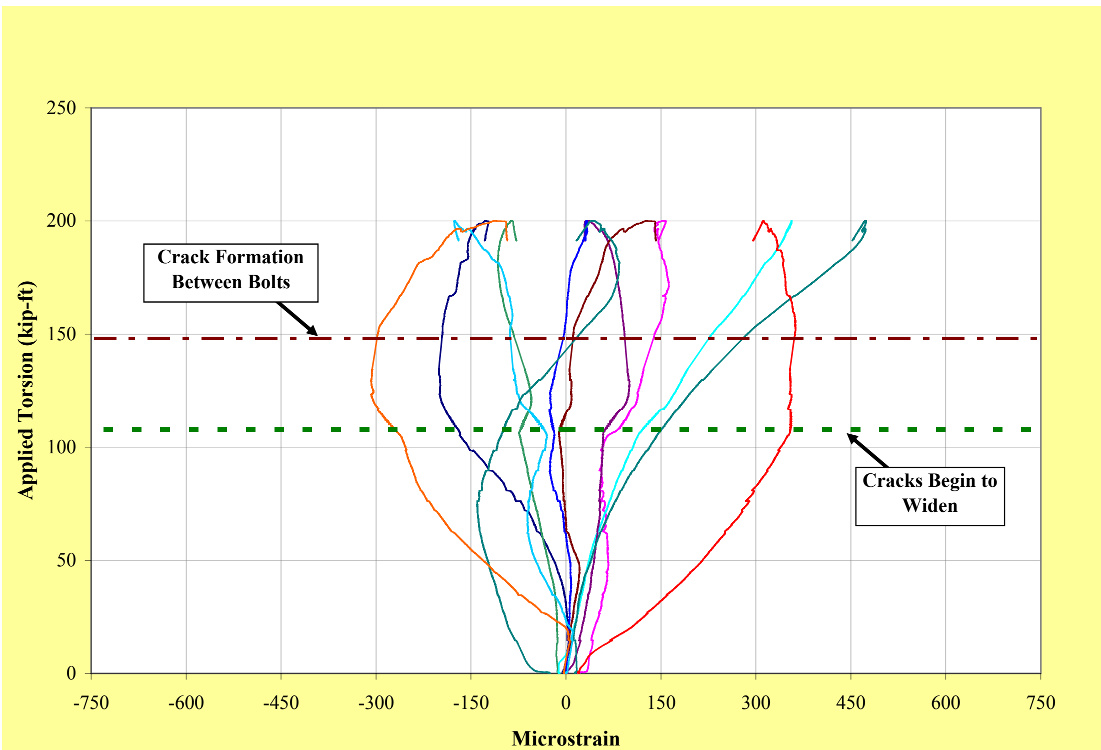


Figure 5-7. Plate Strain Comparison Plot for Initial Test exhibits the redistribution of the load coinciding with crack formations

As the various plots illustrate, some of the strain gages recorded negative strains, while others recorded positive strains. This was most likely due to the bearing location of the bolt on the base plate. To further explore this phenomenon, strain gages were placed on the bottom of the base plate in addition to those on the top for the second test.

5.1.3 Summary of Initial Test Results

The results of this test indicated that the concrete breakout failure was the failure mode observed in the site investigation. The characteristic cracks and the structural integrity of the bolts in the failed foundations, as observed during the site investigation, was the first step to arriving at this failure mode. The percent difference between the failure torsion and the predicted failure torsion (Equation 3-3) was 3.6%. Therefore, it was concluded that the foundation failed at the failure torsion for the predicted failure mode. These results indicated that the design methodology for cantilever sign foundations should include the concrete breakout failure due to shear directed parallel to an edge resulting from torsional loading. All plots for the first test are located in Appendix C.

5.2 CFRP Retrofit Test

After the results of the first test were reviewed, the need for a method to strengthen existing foundations became apparent. Since the concrete breakout failure had not been considered in the design of the cantilever sign structure foundations, a system had to be put in place to evaluate whether or not those existing foundations would be susceptible to failure. One economical method of retrofitting the existing foundations is the use of Carbon Fiber Reinforced Polymer (CFRP) wraps.

At the conclusion of the first test, the bolts had not yielded, and the concrete was still intact. This enabled a second test on the failed foundation to be carried out. The key focus of this second test was to determine if the foundation could reach its initial concrete breakout strength again. The foundation was retrofitted with three layers of 12 in. (305 mm) wide SikaWrap Hex 230C (Figure 5-8). This amount of CFRP exceeded the amount required to attain the concrete breakout strength, 193 kip-ft (262 kN-m). The torsional strength of the shaft with the retrofit was calculated. The resultant strength based on the effective width, Section 4.1.3, of

1.5·cover, or 7.5 in. (191 mm), was 229 kip-ft (310 kN-m). Since that effective depth was an assumption for design, the strength based on the full width, 12 in. (305 mm), of the wrap, 367 kip-ft (498 kN-m), was also calculated for reference.

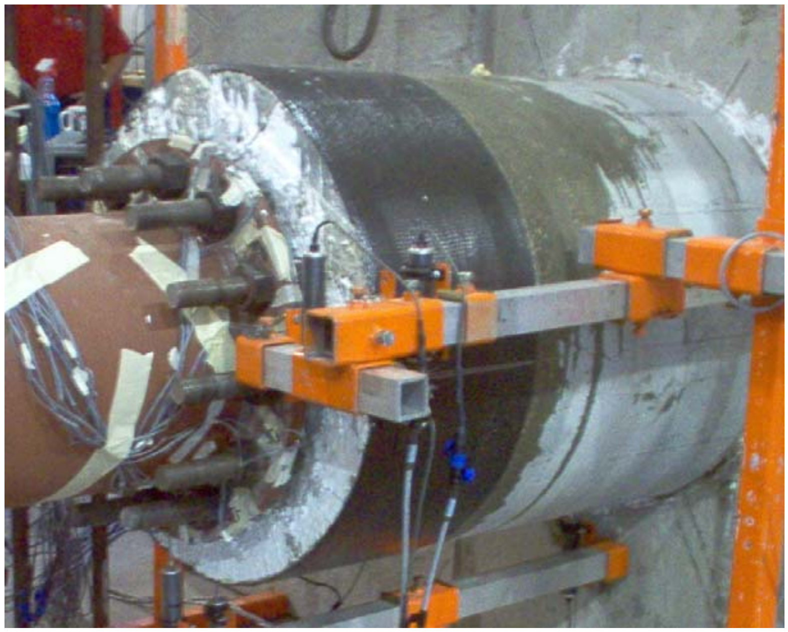


Figure 5-8. Shaft with the CFRP wrap applied prior to testing

5.2.1 Behavior of Specimen with CFRP Wrap During Testing

The second test was conducted on 13 September 2006. For this test, the concrete strength was not a critical parameter, since the concrete had already failed. The containment provided by the CFRP wrap, along with the anchor bolts, was the source of the strength of the foundation. As the purpose of the second test was to learn how much load the foundation could take, and if that load met or exceeded the concrete breakout strength, the load was not held for prolonged periods at regular intervals during the test. Figure 5-9 is the Applied Torsion vs. Plate Rotation plot for the second test. The foundation was closely monitored for crack formation along the shaft, propagation of existing cracks, and failure of the CFRP wrap.

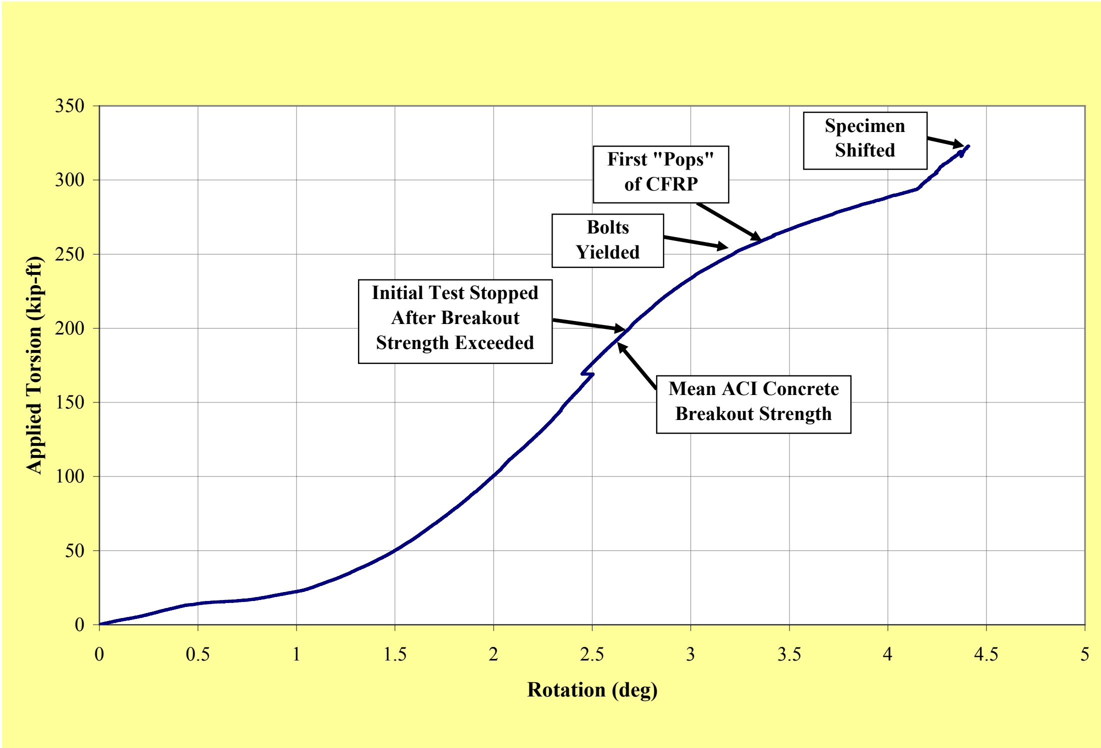


Figure 5-9. Applied Torsion vs. Plate Rotation Plot- Retrofit Test

The strength of the foundation exceeded the predicted concrete breakout strength of 193 kip-ft (262 kN-m). It was not until the loading reached 257 kip-ft (348 kN-m) that the first pops of the carbon fibers were heard. At that torsion load, the strength of the CFRP wrap based on the effective depth, 229 kip-ft (310 kN-m), was exceeded. Therefore, the effective depth of the wrap was a conservative assumption.

At approximately 288 kip-ft (390 kN-m) more pops within the CFRP wrap were heard. However, the entire carbon fiber wrap did not fail. During the course of the test, characteristic torsion cracks began to form along the shaft (Figure 5-10) and propagated to the base of the shaft. This occurred because the ACI 318-05 nominal torsional strength (Equation 2-6) of 252 kip-ft (342 kN-m) was exceeded. Although these cracks had formed, the foundation still had not failed. Another phenomenon that occurred was the yielding of the bolts. According to the calculations for the yield strength of the bolts, the bolts yielded at approximately 253 kip-ft (343 kN-m) of applied torsion. The strength was determined using the same methodology outlined in Section 3.3.1. This was the within the range in which the yielding was observed (Figure 5-9). The bolts were yielding, but they did not reach their ultimate strength. The test abruptly concluded when the concrete block shifted out of place, causing the load cell to be dislodged from its location on the pipe. This occurred at 323 kip-ft (438 kN-m).

Figure 5-10. Shaft exhibiting characteristic torsion cracks from face to base of shaft

5.2.3 Behavior of Strain Gages During Testing

For the retrofit test, strain gages were placed on the top and bottom of the base plate. Figure 5-11 shows each of the Applied Torsion vs. Plate Strain plots at the appropriate bolt locations. Note that as the loading increased, the bottom strain gages began to behave similarly for all of the bolts. The strain was increasing at a higher rate. This illustrated that as the bolts picked up load and began to bend, they were primarily in contact with the bottom of the base plate (Figure 5-12). The strains recorded by the bottom gages indicate that all of the bolts became active during the test.

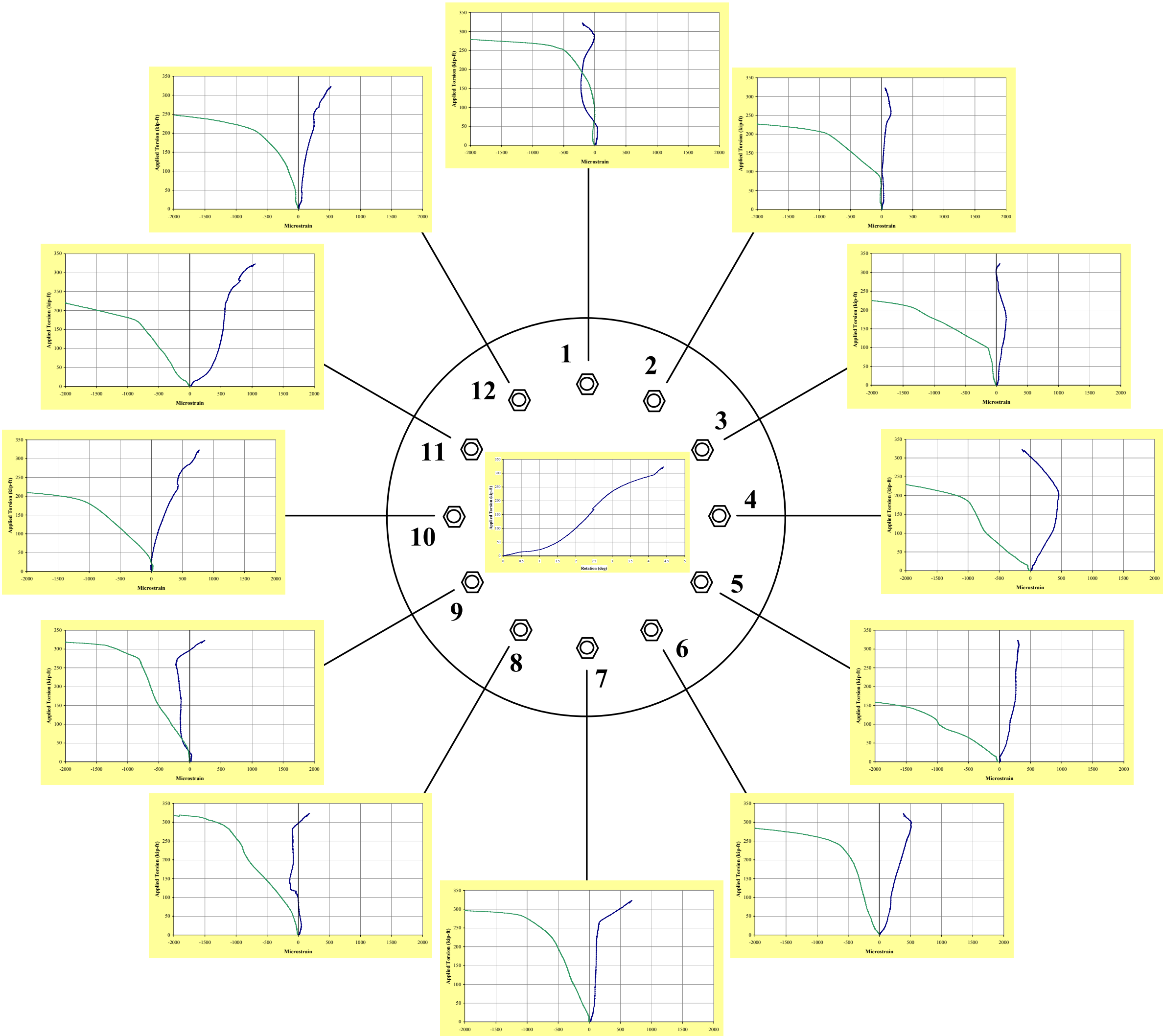


Figure 5-11. Applied Torsion vs. Plate Strain plots for the Retrofit Test at the appropriate bolt location around the base plate with Applied Torsion vs. Plate Rotation plot in center (full size plots in Appendix D)

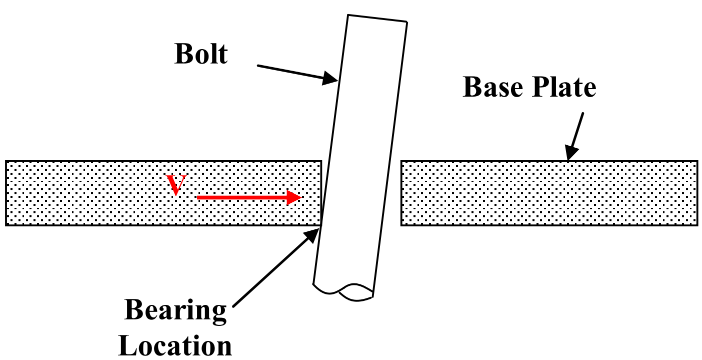


Figure 5-12. Bolt bearing on the bottom of the base plate during loading

Similar to the behavior of the bolts throughout the initial test, Figure 5-13 illustrates the changes in the plate strain data for the bottom gages corresponding with milestone loads during the test.

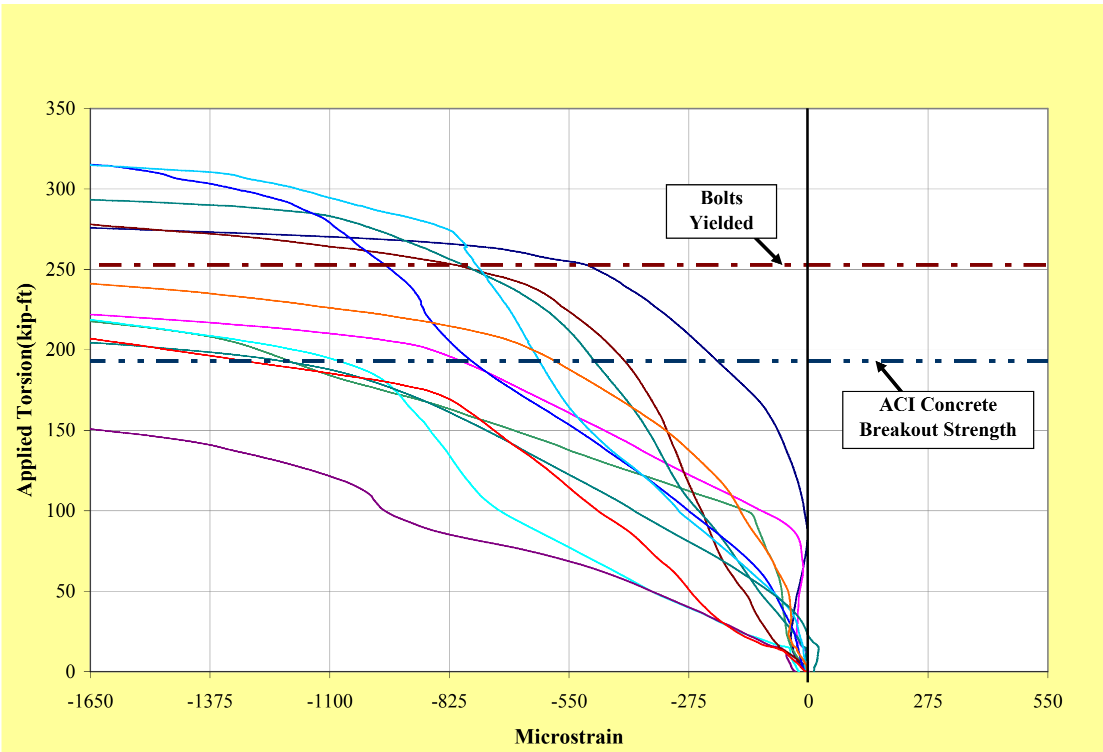


Figure 5-13. Plate Strain Comparison plot for the retrofit test exhibits slope changes at milestone loads

5.2.4 Summary of Test Results

Upon removal of the pipe apparatus, the crack pattern illustrated the concrete breakout failure, and torsional cracks in the center of the shaft verified that the concrete torsional capacity was exceeded during testing (Figure 5-14). Figure 5-15 details the characteristic torsion cracks on the side of the shaft after testing. The test proved that the CFRP wrap was an acceptable method for retrofitting the foundation. It exceeded the concrete breakout strength. The success of this retrofit test led to the development of guidelines for the evaluation of existing foundations and the guidelines for the retrofit of those foundations in need of repair. All plots for the retrofit test are located in Appendix D.

Figure 5-14. Face of shaft after test illustrates yielding of bolts, concrete breakout cracks around the perimeter, and torsion cracks in the center.

Figure 5-15. Torsion cracks along length of the shaft after the test

CHAPTER 6
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The purpose of this research program was to determine the cause of the failure of foundations of cantilever sign structures during the 2004 hurricane season. After a thorough literature review, in conjunction with the site investigation, and testing, it was determined that the foundations failed as a result of an applied torsion which caused a concrete breakout failure due to shear directed parallel to the edge on the anchors. This anchorage failure is detailed in ACI 318-05 Appendix D. Previous to this experimental research, this failure mode was not considered in the design of the cantilever sign foundations. Cantilever sign foundations need to be designed for shear parallel to the edge on the anchor resulting from torsion.

Test results indicate that the failure of the foundations was caused by concrete breakout due to shear directed parallel to the edge on the anchors. The test specimen failed at the torsion predicted by the ACI 318-05 Appendix D design equations. Additionally, the crack pattern matched the crack pattern exhibited in the field, and both foundations emulated the characteristic crack pattern of the shear directed parallel to an edge for concrete breakout failure. It is recommended that future tests be performed on circular foundations to further investigate the concrete breakout failure for a shear load directed both parallel and perpendicular to an edge.

Additional testing was performed to determine an acceptable retrofit option. It was determined that applying a CFRP wrap to the foundation strengthens the foundation such that it not only meets its initial concrete breakout capacity, but, also, exceeds the capacity. The results of this test led to the development of guidelines for the evaluation and repair of existing foundations. The guidelines were based on the following:

- Using either the torsional load from the design or, if not available, using the ACI nominal torsional strength (ACI 318-05 Section 11.6.3.6), determine the torsional capacity of the foundation.

- Calculate the concrete breakout strength in accordance with ACI Appendix D.
- If the concrete breakout strength is less than the maximum of the nominal torsional strength and design torsion, then the foundation is susceptible to failure.
- The amount of the carbon fabric required is calculated using the maximum of the nominal torsional strength and the design torsion. The amount required is given in layers of the CFRP wrap.

These guidelines (Appendix E) were submitted to the Florida Department of Transportation. The guidelines will be used to evaluate and, if necessary, repair the existing foundations. It is critical that such foundations be evaluated in order to determine the susceptibility to this type of failure. Additionally, it is recommended that the alternative foundations in Section 2-4 be considered for further investigation. The proper use of the findings of this research program will allow for future prevention of failures exhibited during the 2004 hurricane season.

APPENDIX A
TEST APPARATUS DRAWINGS

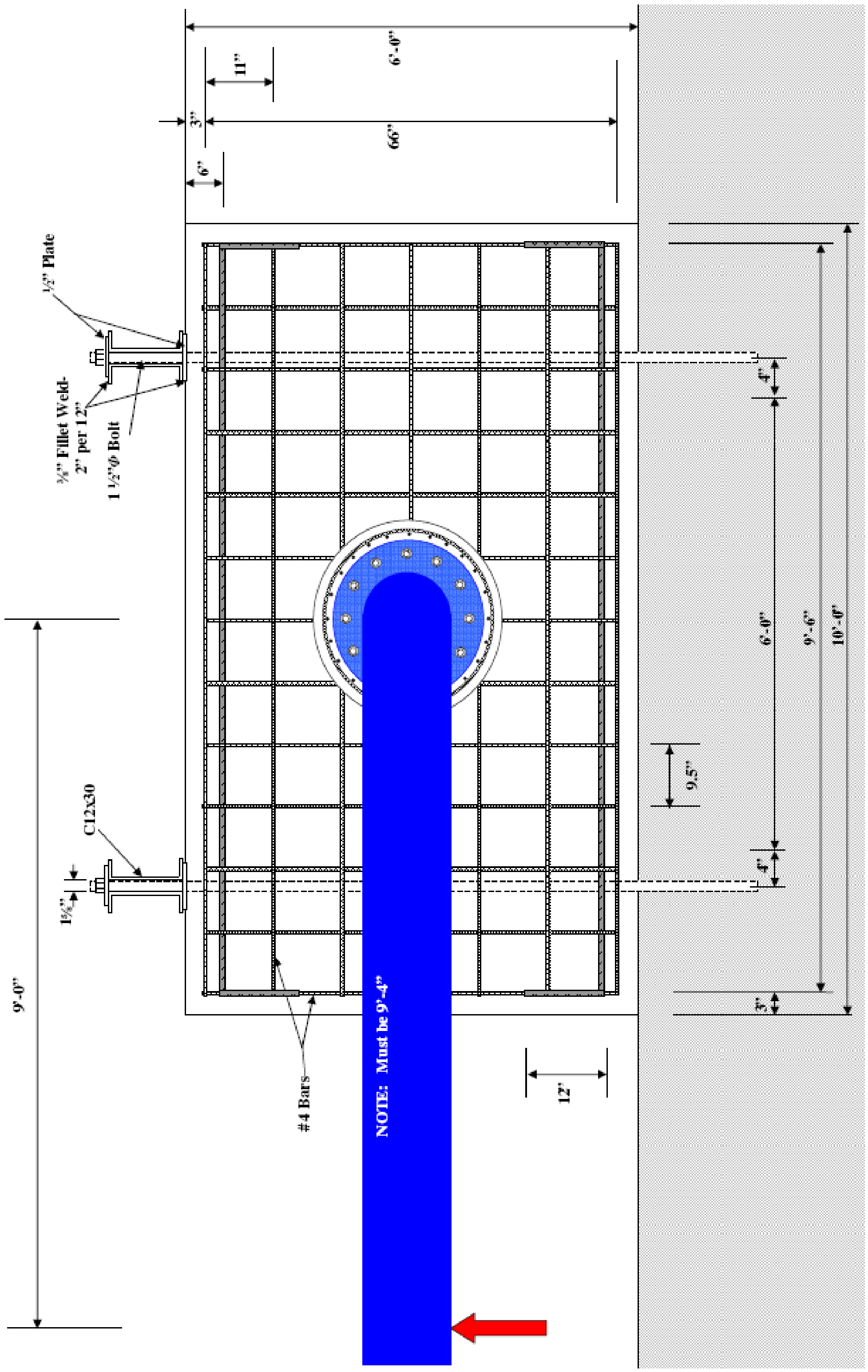


Figure A-1. Dimensioned front elevation drawing of test apparatus

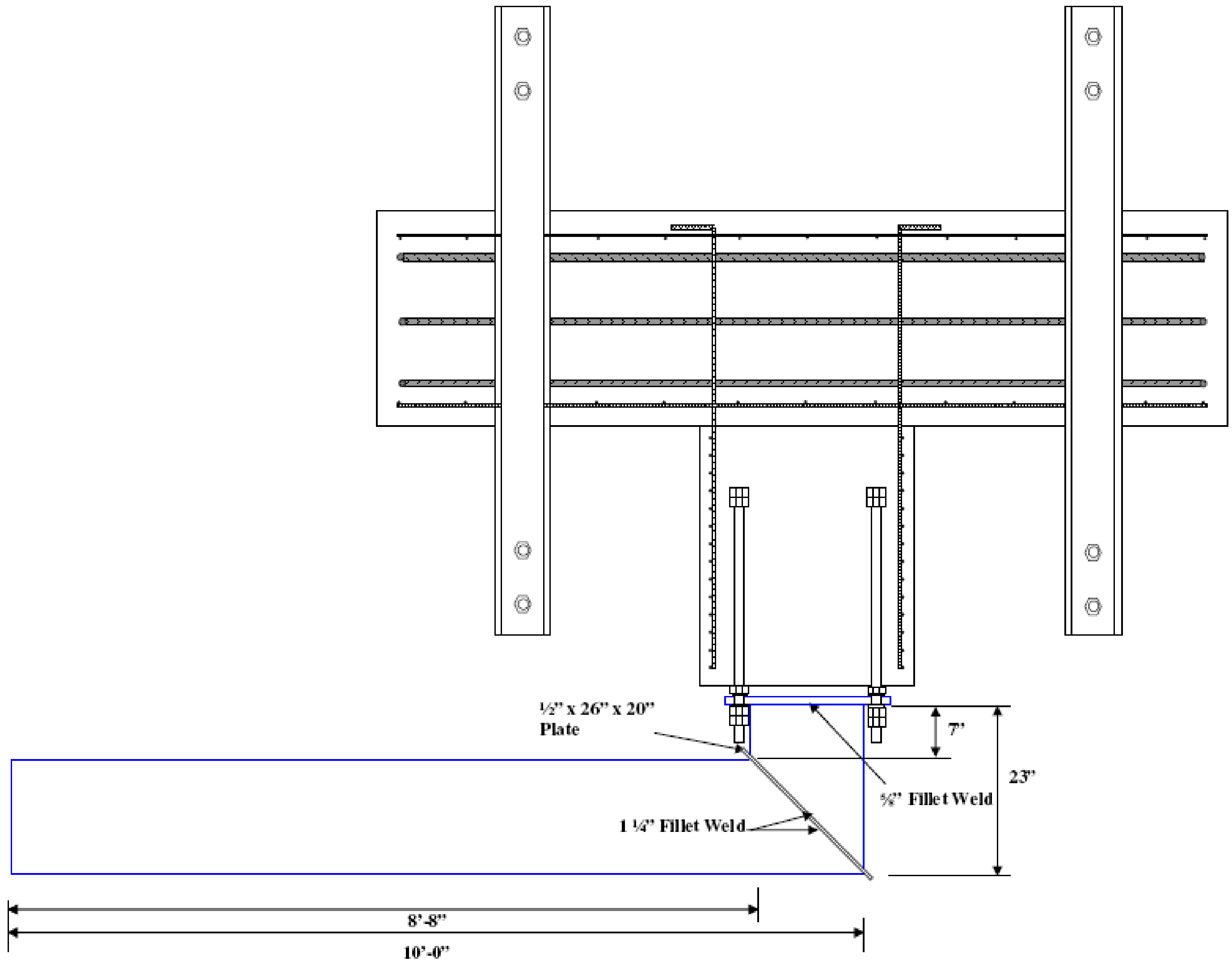


Figure A-2. Dimensioned plan drawing test apparatus

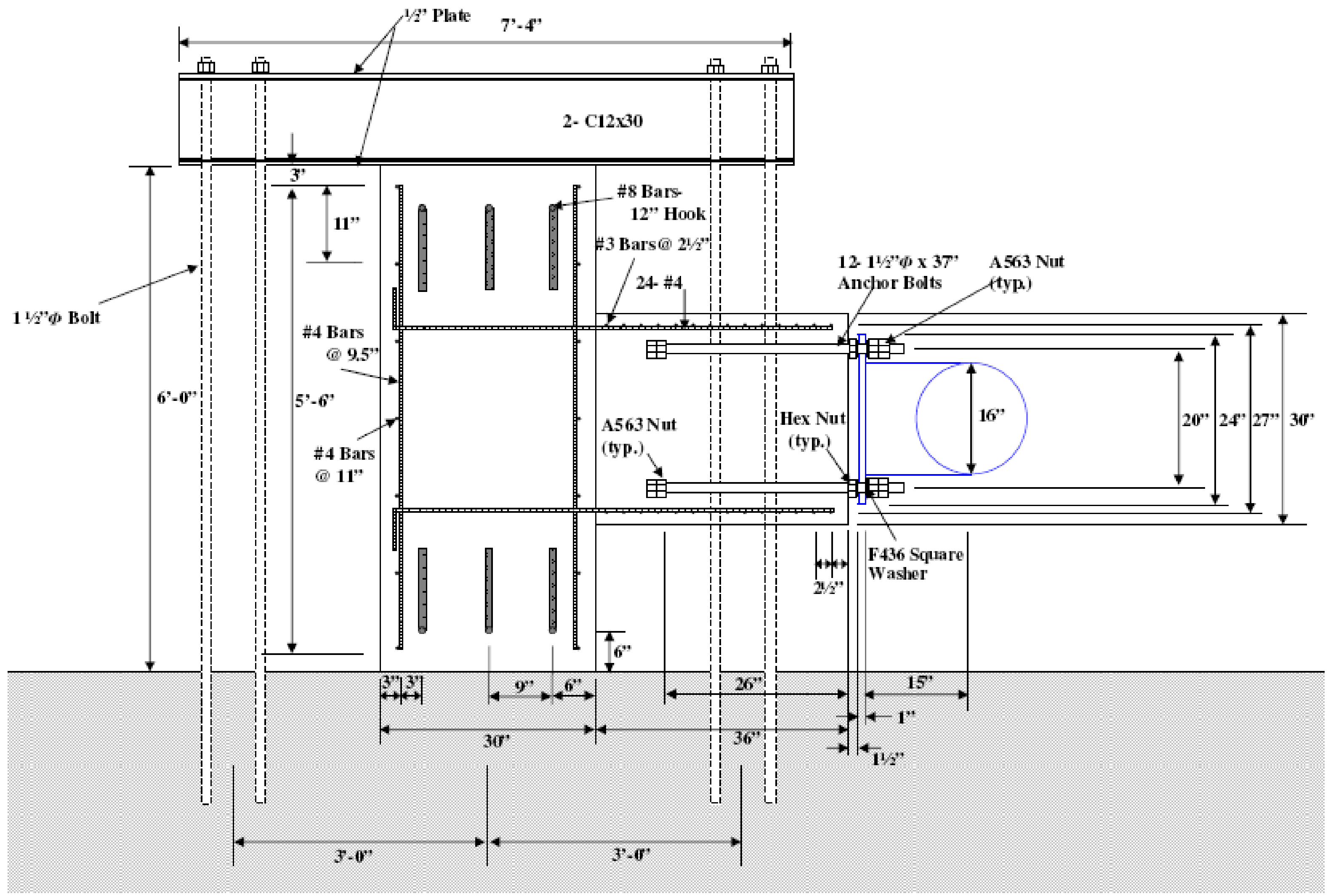
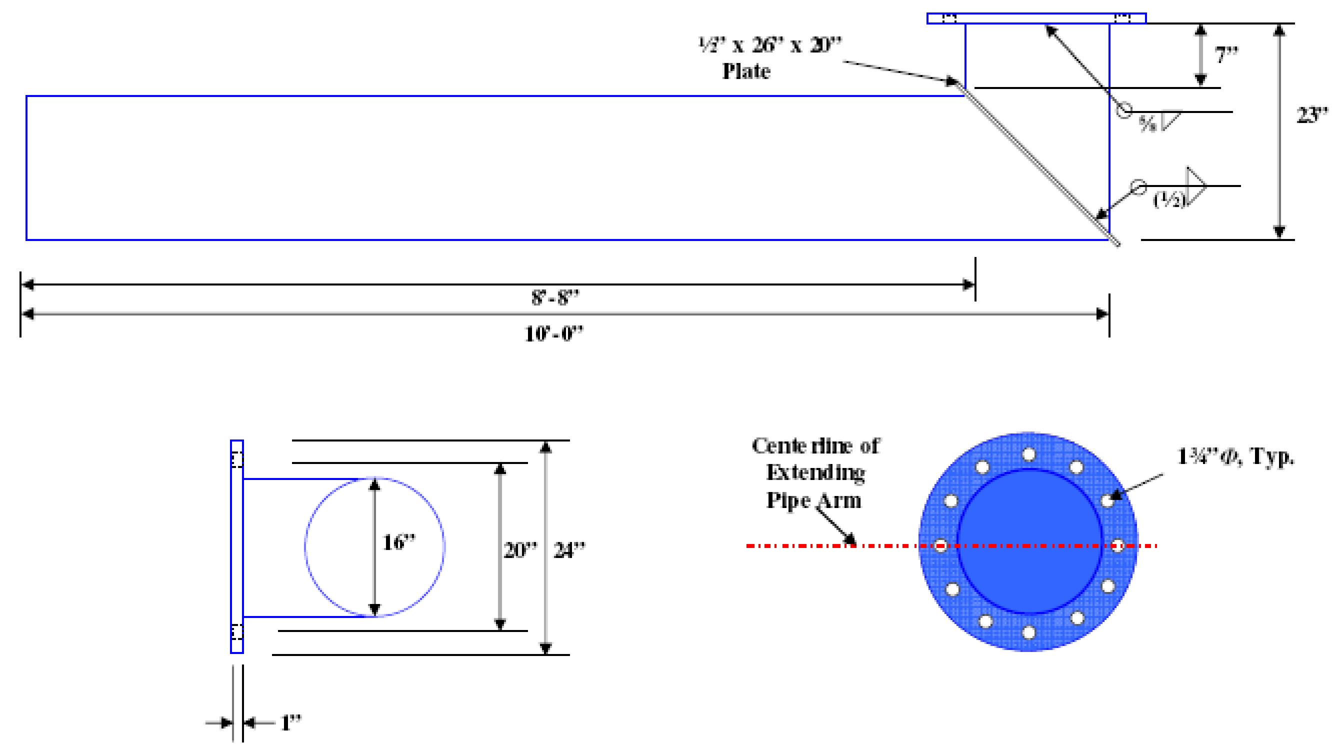


Figure A-3. Dimensioned side elevation drawing of test apparatus



Notes:
- 3- 24" Φ Base Plate (1" Thick)
- HSS 16.000 x 0.500 Pipe

Figure A-4. Dimensioned pipe apparatus drawing

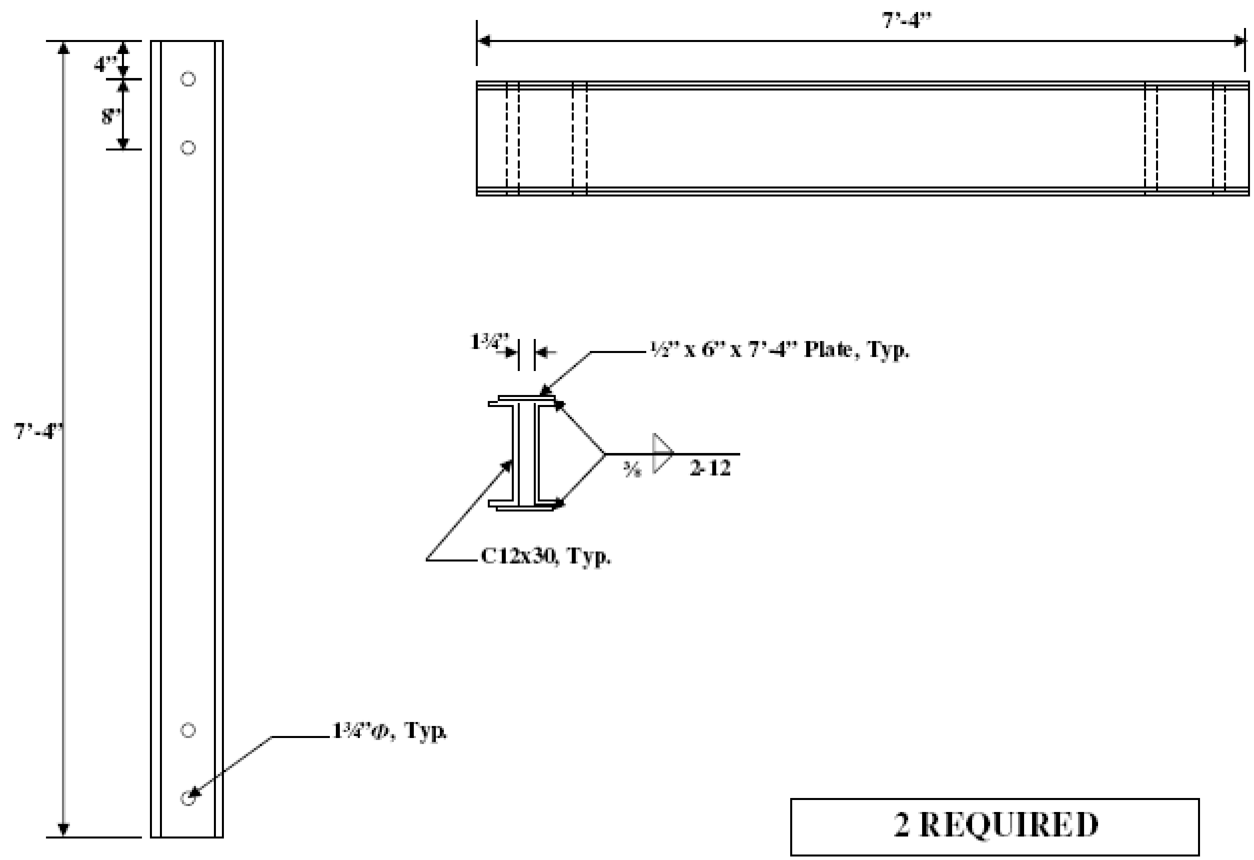


Figure A-5. Dimensioned channel tie-down drawing

APPENDIX B
DESIGN CALCULATIONS

Design of Test Program

ORIGIN ≡ 1

Input

Shaft

$d_s := 30in$ Diameter of the Shaft

$f'_c := 5500psi$ Concrete Strength

Hoop Steel

$Hoop\_Steel\_Area := 0.11in^2$ Hoop Steel Area

$Hoop\_Steel\_Diameter := \frac{3}{8}in$ Hoop Steel Diameter

$s_h := 2.5in$ Spacing of Hoop Steel

$f_{yt} := 60ksi$ Yield Strength of Hoop Steel

$d_h := 27in$ Centerline Hoop Steel Diameter

$Long\_Steel\_Area := 0.20in^2$ Longitudinal Steel Area

$Long\_Steel\_Diameter := 0.5in$ Longitudinal Steel Diameter

$\phi_{torsion} := 0.75$ ACI 318-05 9.2.3.2

$Moment\_Arm := 9ft$

Bolts

$d_o := 1.5in$ Diameter of the Bolt

$d_b := 20in$ Center-Center Diameter of Bolts

$f_{y\_bolt} := 105ksi$

$No\_Bolts := 12$ Number of Bolts

CFRP Properties

$t_{CFRP} := 0.015in$ Thickness of Wrap

$f_{n\_CFRP} := 91.1ksi$ Tensile Strength of CFRP

$w_{CFRP} := 12in$ Width of CFRP Sheet

For the field model, F1554 Grade 55 Anchor Bolts were used.

$f_{y\_bolt\_field} := 55ksi$

Failure Equations

Torsion

Cracking Torsion

$$A_{cp} := \pi \cdot \left(\frac{d_s}{2}\right)^2 \qquad p_{cp} := \pi \cdot d_s$$

$A_{cp} = 706.86\ in^2$

$p_{cp} = 94.25\ in$

$$T_{cr} := 4 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \left(\frac{A_{cp}^{\ 2}}{p_{cp}}\right) \qquad (2\text{-}1)$$

$T_{cr} = 131.06\ kip \cdot ft$

Basic Torsion

$$\tau := 4 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi$$

$$T_{basic} := \frac{\tau \cdot \pi \cdot \left(\frac{d_s}{2}\right)^3}{2} \qquad (2\text{-}2)$$

$T_{basic} = 131.06\ kip \cdot ft$

Threshold Torsion

$$T_{threshold} := \phi_{torsion} \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}^{\ 2}}{p_{cp}}\right) \qquad (2\text{-}3)$$

$T_{threshold} = 24.57\ kip \cdot ft$

Nominal Torsional Strength

$A_o := \pi \cdot \left(d_h \div 2\right)^2$ — Area enclosed by hoop steel — $A_o = 572.56\ in^2$

$A_t :=$ Hoop_Steel_Area — Area of hoop steel — $A_t = 0.11\ in^2$

$\theta := 45$ — ACI 318-05- 11.6.3.6 (a) — $\theta_{rad} := \theta \cdot \frac{\pi}{180}$ — $\theta_{rad} = 0.79$

$$T_{n\_torsion} := \frac{2 \cdot A_o \cdot A_t \cdot f_{yt}}{s_h} \cdot \cot\left(\theta_{rad}\right) \qquad (2\text{-}6)$$

$T_{n\_torsion} = 251.92\ kip \cdot ft$

Combined Shear and Torsion on Foundation

$$A_{oh} := A_o \qquad A_{oh} = 572.56\ \text{in}^2$$

$$T_u(V_u) := V_u \cdot \text{Moment\_Arm} \cdot \text{kip}$$

$$p_h := \pi \cdot d_h \qquad p_h = 84.82\ \text{in}$$

$$v(V_u) := \sqrt{\left(\frac{\frac{V_u}{1}}{\frac{A_{oh}}{\text{in}^2}}\right)^2 + \left[\frac{\frac{T_u(V_u)}{\text{kip}\cdot\text{in}} \cdot \frac{p_h}{\text{in}}}{1.7 \cdot \left(\frac{A_{oh}}{\text{in}^2}\right)^2}\right]^2} \qquad \text{(2-8)}$$

If $V_u := 1\text{kip}$ $\quad T_u(V_u) := V_u \cdot \text{Moment\_Arm}$

$$\text{Coefficient\_Shear} := \left(\frac{\frac{V_u}{\text{kip}}}{\frac{A_{oh}}{\text{in}^2}}\right)^2$$

$$\text{Coefficient\_Torsion} := \left[\frac{\frac{T_u(V_u)}{\text{kip}\cdot\text{in}} \cdot \frac{p_h}{\text{in}}}{1.7 \cdot \left(\frac{A_{oh}}{\text{in}^2}\right)^2}\right]^2$$

$$\frac{\text{Coefficient\_Torsion}}{\text{Coefficient\_Shear}} = 88.58$$

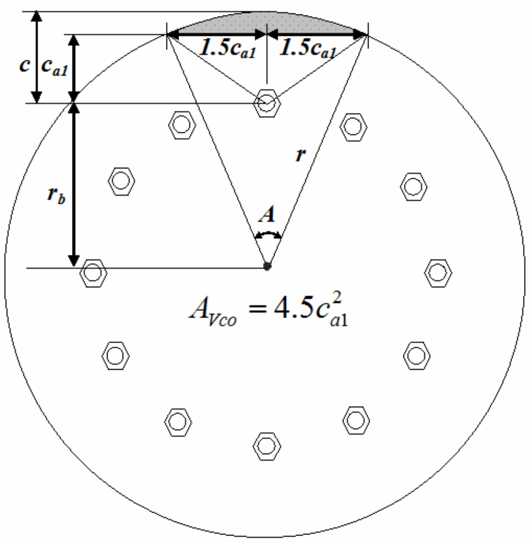
The coefficient for the torsion term is 88 times that of the shear term. Therefore, shear will not be considered.

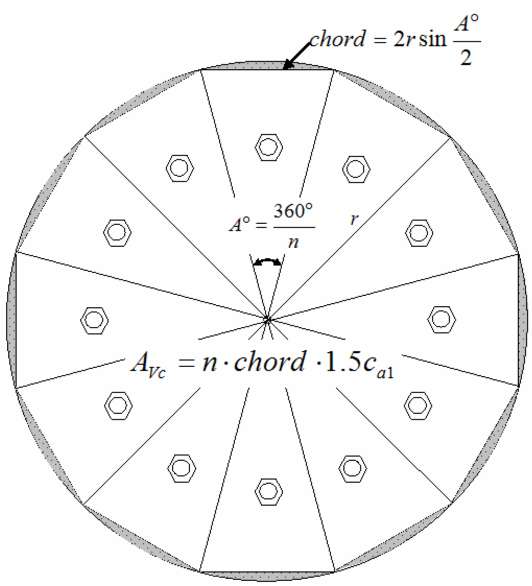
Concrete Breakout Strength

$cover := \frac{d_s - d_b}{2}$ Bolt Cover $cover = 5\ in$

$$c_{a1} := \frac{\sqrt{\left(\frac{d_b}{2}\right)^2 + 3.25\cdot\left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_b}{2}\right)^2\right]} - \left(\frac{d_b}{2}\right)}{3.25}$$ (3-2)

$c_{a1} = 3.85\ in$ Cover for the calculation of the anchor strength





$A := \frac{360}{No\_Bolts}deg$

$A = 30\ deg$

$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right)$

$chord\_group = 7.76\ in$

$A_{min\_group} := 2\cdot\operatorname{asin}\left(\frac{3.0\cdot c_{a1}}{d_s}\right)$

$A_{min\_group} = 45.24\ deg$

If A is greater than A_{min_group} then there is no group effect

$A_{Vc} := No\_Bolts\cdot chord\_group\cdot 1.5\cdot c_{a1}$ $A_{Vc} = 537.55\ in^2$ $A_{Vco} := 4.5\cdot c_{a1}^{\ 2}$ $A_{Vco} = 66.57\ in^2$

$$\text{Check\_Group\_Effect} := \begin{cases} \text{"Group Effect"} & \text{if } A \le A_{min\_group} \\ \text{"No Group Effect"} & \text{if } A > A_{min\_group} \end{cases}$$

$\text{Check\_Group\_Effect} = \text{"Group Effect"}$

ACI 318-05-D.6.2

$$l_e := 8 \cdot d_o$$

$$V_b := 13 \cdot \left(\frac{l_e}{d_o}\right)^{0.2} \cdot \sqrt{\frac{d_o}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf \qquad (2\text{-}11) \qquad V_b = 13.5\,kip$$

$\psi_{cV} := 1.4$ ACI 318-05- D.6.2.7

1.0 for cracked concrete with no supplementary reinforcement or reinforcement smaller than a No. 4 bar
1.2 for cracked concrete with edge reinforcement of a No.4 bar or greater
1.4 for uncracked concrete or with edge reinforcement of a No.4 bar or greater enclosed within stirrups spaced no more than 4 in. apart

$\psi_{ecV} := 1.0$ ACI 318-05- D.6.2.5 $\psi_{edV} := 1.0$ ACI 318-05- D.6.2.6

All anchors are loaded in shear in the same direction

$$V_{cbg} := \begin{cases} No\_Bolts \cdot \psi_{edV} \cdot V_b & \text{if } A > A_{min\_group} \\ \dfrac{A_{Vc}}{A_{Vco}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot V_b & \text{if } A \le A_{min\_group} \end{cases} \qquad (2\text{-}12) \qquad V_{cbg} = 109.01\,kip$$

$$V_{cbg\_parallel} := 2 \cdot V_{cbg} \qquad V_{cbg\_parallel} = 218.03\,kip$$

$$T_{n\_breakout\_ACI} := V_{cbg\_parallel} \cdot (d_b \div 2) \qquad T_{n\_breakout\_ACI} = 181.69\,kip \cdot ft$$

Calculation of V_{cbg} and $T_{n\_breakout\_ACI}$ are based on (3-3) and (3-4)

Eligehausen et al. (2006) Concrete Breakout Provisions

$f_{cc200} := 1.18f'_c$ 1.18 is the conversion factor between the cylinder test and cube test

$$\alpha := 0.1 \cdot \left(\frac{8 \cdot \frac{d_o}{mm}}{\frac{c_{a1}}{mm}} \right)^{0.5} \qquad \beta := 0.1 \cdot \left(\frac{\frac{d_o}{mm}}{\frac{c_{a1}}{mm}} \right)^{0.5}$$

$$V'_{uc} := 3.0 \cdot \left(\frac{d_o}{mm} \right)^{\alpha} \cdot \left(8 \cdot \frac{d_o}{mm} \right)^{\beta} \cdot \sqrt{\frac{f_{cc200}}{\frac{N}{mm^2}}} \cdot \left(\frac{c_{a1}}{mm} \right)^{1.5} \cdot N$$ (2-13)

$V'_{uc} = 52682.62\ N$

$V'_{uc} = 11.84\ kip$

$$V_{uc} := \frac{A_{Vc}}{A_{Vco}} \cdot V'_{uc}$$ (2-14)

$V_{uc} = 425420.27\ N$

$V_{uc} = 95.64\ kip$

$$\alpha_V := 90 \cdot \frac{\pi}{180}$$

$$\psi_{\alpha V} := \frac{1}{\cos(\alpha_V) + 0.5 \cdot \sin(\alpha_V)}$$ (2-15)

$\psi_{\alpha V} = 2$

$$V_{uc\_\alpha V} := V_{uc} \cdot \psi_{\alpha V}$$ (2-16)

$V_{uc\_\alpha V} = 191.28\ kip$

$$\psi_{parallel} := 4 \cdot k_4 \cdot \left[\frac{No\_Bolts \left(\frac{d_o}{mm} \right)^2 \cdot \frac{f_{cc200}}{\frac{N}{mm^2}}}{\frac{V_{uc}}{N}} \right]^{0.5}$$ $k_4 := 0.75$ (2-18)

$\psi_{parallel} = 4.06$

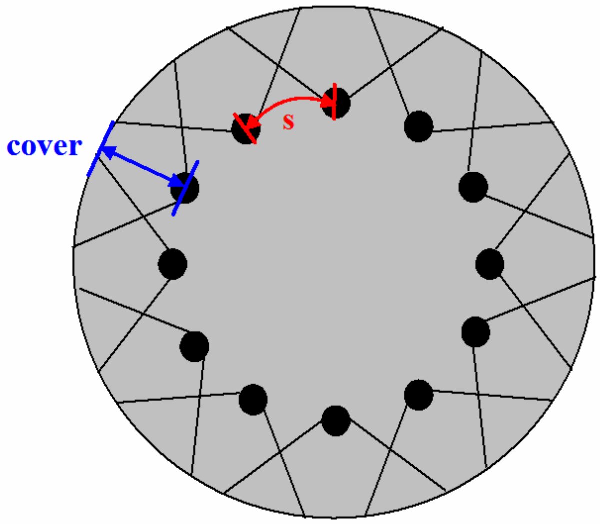
Equation 2-18 was based on tests with two bolts with a straight edge. Therefore, the applicability of the equation comes into question when there are more than two bolts being loaded. The following analysis will determine the ratio of V_{uc} for 2 bolts and V_{uc} for arrangements of 2 or more bolts.

Step 1: For 2 bolt arrangements, calculate V_{uc} as a function of spacing, s, and V'_{uc}.

The spacing will be taken as a function of the cover

A_v will be the area of the group of anchors for a unit depth

A_o will be the area of a single anchor for a unit depth

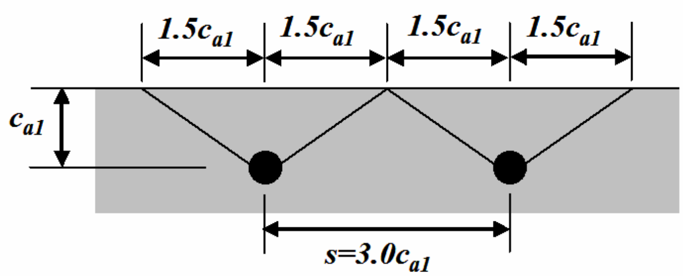


$\text{cover} = 5 \text{ in}$

$s_b := \pi \cdot d_b \div \text{No\_Bolts}$

$s_b = 5.24 \text{ in}$

$A_o(c) := 1.5 \cdot 2 \cdot c$ $\quad A_o(c) \rightarrow 3.0 \cdot c$ $\quad$ This value does not change as a result of changing the spacing. The generic variable for the cover is ***c***.



For the case where s>=3.0c_{a1}, there is no overlap.

$$s(c) := \begin{pmatrix} 3.0 \cdot c \\ 1.5 \cdot c \\ 1.0 \cdot c \\ 0.5 \cdot c \\ 0 \cdot c \end{pmatrix} \qquad A_v(c, s, n) := 2 \cdot (1.5 \cdot c) + s(c) \cdot (n - 1)$$

n is the number of bolts

For

$n := 2 \qquad A(c, s, n) \rightarrow 30 \cdot \text{deg}(c, \text{function}, 2)$

$$\frac{A_v(c, s, n)}{A_o(c) \cdot n} = \begin{pmatrix} 1 \\ 0.75 \\ 0.67 \\ 0.58 \\ 0.5 \end{pmatrix}$$

The ratios are normalized by factorin out the number of bolts under consideration such that the result may be compared to the ratio for 2 or more bolts.

For a circular foundation, ***n*** is considered equal to infinity because the bolts are continuous; there is no end as there is for a straight edge

$n := 1 \cdot 10^{20}$ For ***n***= infinity

$$\frac{A_V(c,s,n)}{A_o(c)\cdot n} = \begin{pmatrix} 1 \\ 0.5 \\ 0.33 \\ 0.17 \\ 0 \end{pmatrix}$$

Therefore, to calculate the reduction factor for the parallel conversion multiplier, the ratio of the A_{vc}/A_{vco} terms for 2 bolts and an infinite number of bolts will be calculated considering the proper ratio of s/c.

For the foundation under invesitgation,***n*** is equal to infinity

$$\frac{s_b}{cover} = 1.05 \qquad s\_ratio\_test(c) := \frac{s_b}{cover}\cdot c$$

$$Group\_Multiplier\_Infinity := \frac{A_V(c, s\_ratio\_test, n)}{A_o(c)\cdot n} \qquad Group\_Multiplier\_2\_Bolts := \frac{A_V(c, s\_ratio\_test, 2)}{A_o(c)\cdot 2}$$

$$Group\_Multiplier\_Infinity = 0.35 \qquad Group\_Multiplier\_2\_Bolts = 0.67$$

$$\frac{Group\_Multiplier\_Infinity}{Group\_Multiplier\_2\_Bolts} = 0.52$$

$$\Psi_{reduction\_s\_bolt} := \frac{Group\_Multiplier\_Infinity}{Group\_Multiplier\_2\_Bolts} \qquad \Psi_{reduction\_s\_bolt} = 0.52$$

$$\Psi_{parallel\_new} := \Psi_{parallel}\cdot\Psi_{reduction\_s\_bolt} \qquad \Psi_{parallel\_new} = 2.1$$

$$V_{ucparallel} := \Psi_{parallel\_new}\cdot V_{uc} \quad (2\text{-}17) \qquad V_{ucparallel} = 893988.63\ N$$

$$V_{ucparallel} = 200.98\ kip$$

$$T_{n\_breakout\_\psi\alpha V} := V_{uc\_\alpha V}\cdot\left(d_b \div 2\right) \qquad T_{n\_breakout\_\psi\alpha V} = 159.4\ kip\cdot ft$$

$$T_{n\_breakout\_\psi parallel} := V_{ucparallel}\cdot\left(d_b \div 2\right) \qquad T_{n\_breakout\_\psi parallel} = 167.5\ kip\cdot ft$$

Summary of Failure Equations

$T_{cr} = 131.06\ kip \cdot ft$

$T_{basic} = 131.06\ kip \cdot ft$

$T_{threshold} = 24.57\ kip \cdot ft$

$T_{n\_torsion} = 251.92\ kip \cdot ft$

$T_{n\_breakout\_ACI} = 181.69\ kip \cdot ft$

$T_{n\_breakout\_\psi\alpha V} = 159.4\ kip \cdot ft$

$T_{n\_breakout\_\psi parallel} = 167.48\ kip \cdot ft$

According to ACI 318-05 11.6.3.5, assuming the ultimate torsion exceeds the threshold torsion, the nominal torsional capacity is taken as the strength of the section.

For the design of the various components of the test specimen, the ACI Concrete Breakout Strength will be the maximum moment as it is the predicted failure mode. ($T_{n\_breakout\_ACI} < T_{n\_torsion}$)

$M_{max} := T_{n\_breakout\_ACI}$ $\quad M_{max} = 181.69\ kip \cdot ft$

$V_{max} := M_{max} \div Moment\_Arm$ $\quad V_{max} = 20.19\ kip$ Failure Load

Shaft Design

$T_{n\_torsion} = 251.92\ kip\cdot ft$ $M_{max\_shaft} := V_{max}\cdot 36in$ $M_{max\_shaft} = 60.56\ kip\cdot ft$

$Hoop\_Steel\_Area = 0.11\ in^2$ $Long\_Steel\_Area = 0.2\ in^2$ $Hoop\_Steel\_Diameter = 0.38\ in$

$s_h = 2.5\ in$ $f_y := 60ksi$ $Long\_Steel\_Diameter = 0.5\ in$

$f_{yt} = 60\ ksi$

$d_h = 27\ in$

Required splice for #3 bars

$\psi_t := 1.0$ $\psi_e := 1.0$ $\lambda := 1.0$ ACI 318-05 12.2.2

$$Required\_Splice\_Hoop := \frac{f_{yt}\cdot\psi_t\cdot\psi_e\cdot\lambda}{25\cdot\sqrt{\frac{f'_c}{psi}}\cdot psi}\cdot(Hoop\_Steel\_Diameter)$$

$Required\_Splice\_Hoop = 12.14\ in$

Required splice between #4 bars and anchor bolts

$\psi_s := 0.8$ Use the simplification for the $(c_b+K_{tr})/d_b$ term $cb\_Ktr\_term := 2.5$ ACI 318-05 12.2.3

$$l_d := \left(\frac{3}{40}\cdot\frac{f_{y\_bolt\_field}}{\sqrt{\frac{f'_c}{psi}}\cdot psi}\cdot\frac{\psi_t\cdot\psi_e\cdot\psi_s\cdot\lambda}{cb\_Ktr\_term}\right)\cdot(d_o)$$

$l_d = 26.7\ in$

Note: The yield strength in the field was used to determine the splice length to replicate the embedment in the field for the test setup.

Development length of #4 bars

ACI 318-05 12.5

$$l_{dh} := \frac{0.02\cdot\psi_e\cdot\lambda\cdot f_y}{\sqrt{\frac{f'_c}{psi}}\cdot psi}\cdot(Long\_Steel\_Diameter)$$

$l_{dh} = 8.09\ in$

$Hook\_Length := 12\cdot(Long\_Steel\_Diameter)$ $Hook\_Length = 6\ in$

Flexural Capacity of Shaft Calculated using to ACI Stress Block

$R := d_s \div 2$ $\quad R = 15\,in$ $\quad A_{long\_steel} := Long\_Steel\_Area$ $\quad A_{long\_steel} = 0.2\,in^2$

$f_y = 60\,ksi$ $\quad number\_of\_bars := 24$ $\quad number\_bars\_yielded := 17$

$$A_c := \frac{number\_bars\_yielded \cdot A_{long\_steel} \cdot f_y}{0.85 \cdot f'_c} \qquad A_c = 43.64\,in^2$$

$$A_{circseg}(h) := \left[R^2 \cdot acos\left(\frac{R-h}{R}\right) - (R-h)\cdot\sqrt{2\cdot R\cdot h - h^2}\right] - A_c$$

$h := root(A_{circseg}(h), h, 0in, 15in)$ $\quad h = 3.37\,in$

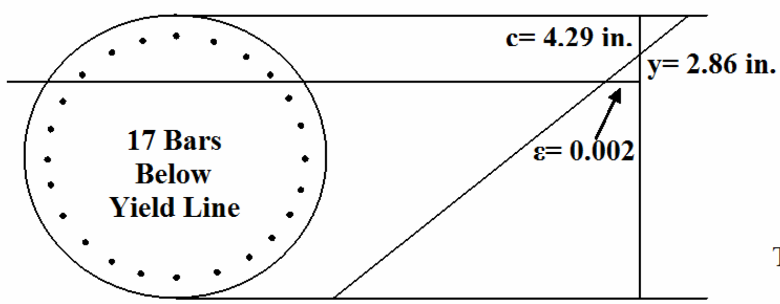
$$\beta_1(f'_c) := \begin{cases} 0.85 & \text{if } f'_c < 4000 \cdot psi \\ 0.65 & \text{if } f'_c > 8000 \cdot psi \\ 0.85 - 0.05\cdot\left(\frac{f'_c - 4000\cdot psi}{1000\cdot psi}\right) & \end{cases}$$

ACI 10.2.7.3

$\beta_1(f'_c) = 0.78$

$$c := \frac{h}{\beta_1(f'_c)} \qquad c = 4.35\,in$$

$$y := .002 \cdot \frac{c}{.003} \qquad y = 2.9\,in$$



The assumption was correct!

$$d_{bars} := \frac{9.2502in\cdot A_{long\_steel}\cdot 2 + 12.0237in\cdot A_{long\_steel}\cdot 2 + 15in\cdot A_{long\_steel}\cdot 2 \ldots + 17.9763in\cdot A_{long\_steel}\cdot 2 + 20.7498in\cdot A_{long\_steel}\cdot 2 + 23.1314in\cdot A_{long\_steel}\cdot 2 \ldots + 25.0189in\cdot A_{long\_steel}\cdot 2 + 26.1677in\cdot A_{long\_steel}\cdot 2 + 26.5in\cdot A_{long\_steel}}{17\cdot A_{long\_steel}}$$

$d_{bars} = 19.13\,in$ $\quad \phi_{flexure} := .90$

$$M_{n\_Shaft} := \phi_{flexure}\cdot number\_bars\_yielded \cdot A_{long\_steel}\cdot f_y \cdot \left(d_{bars} - \frac{h}{2}\right) \qquad M_{n\_Shaft} = 266.84\,kip\cdot ft$$

$$Check\_Flexure\_Shaft := \begin{cases} \text{"Sufficient Strength"} & \text{if } M_{n\_Shaft} \ge M_{max\_shaft} \\ \text{"Insufficient Strength"} & \text{if } M_{n\_Shaft} < M_{max\_shaft} \end{cases}$$

Check_Flexure_Shaft = "Sufficient Strength"

Anchor Design

Check Bolt Flexure and Shear

$$V_{max\_anchor} := \frac{M_{max}}{\frac{d_b}{2} \cdot No\_Bolts} \qquad M_{max\_anchor} := V_{max\_anchor} \cdot \frac{.5 \cdot d_o + 2in}{2} \qquad S_{bolt} := \frac{\pi \cdot \left(\frac{d_o}{2}\right)^3}{4}$$

$$V_{max\_anchor} = 18.17\,kip \qquad M_{max\_anchor} = 24.98\,kip \cdot in \qquad S_{bolt} = 0.33\,in^3$$

$$f_{bolt} := M_{max\_anchor} \div S_{bolt} \qquad f_{bolt} = 75.4\,ksi \qquad A_{se\_bolt} := 1.405\,in^2$$

$$f_{uta} := \min(1.9 \cdot f_{y\_bolt}, 125ksi) \qquad f_{uta} = 125\,ksi \qquad V_{bolt} := A_{se\_bolt} \cdot f_{uta}$$

$$Check\_Bolt\_Flexure := \begin{vmatrix} \text{"Sufficient Strength" if } f_{bolt} \le f_{y\_bolt} \\ \text{"Insufficient Strength" if } f_{bolt} > f_{y\_bolt} \end{vmatrix}$$

Check_Bolt_Flexure = "Sufficient Strength"

$$Check\_Bolt\_Shear := \begin{vmatrix} \text{"Sufficient Strength" if } V_{bolt} \ge V_{max\_anchor} \\ \text{"Insufficient Strength" if } V_{bolt} < V_{max\_anchor} \end{vmatrix}$$

Check_Bolt_Shear = "Sufficient Strength"

Calculate the load that will cause the bolts to yield

$$M_{bolt\_yield} := \frac{f_{y\_bolt} \cdot S_{bolt} \cdot d_b \cdot No\_Bolts}{0.5 \cdot d_o + 2in} \qquad M_{bolt\_yield} = 253.02\,kip \cdot ft$$

$$P_{bolt\_yield} := M_{bolt\_yield} \div Moment\_Arm \qquad P_{bolt\_yield} = 28.11\,kip$$

Pipe Apparatus Design

Based on AISC LRFD Manual of Steel Construction-

LRFD Specification for Steel Hollow Structural Sections

Pipe Properties-HSS 16.000 x 0.500

$t_{pipe} := 0.465$ $\quad A_{pipe} := 22.7in^2$ $\quad D\_t := 34.4$ $\quad W_{pipe} := 82.8\frac{lbf}{ft}$ $\quad F_{y\_pipe} := 42ksi$

$I_{pipe} := 685in^4$ $\quad S_{pipe} := 85.7in^3$ $\quad r_{pipe} := 5.49in$ $\quad Z_{pipe} := 112in^3$ $\quad F_{u\_pipe} := 58ksi$

$D_{pipe} := 16in$ $\quad J_{pipe} := 1370in^4$ $\quad C_{pipe} := 171in^3$ $\quad E := 29000ksi$

Design of Short Section- Applied Shear, Torsion, and Flexure

$L_{Short\_Pipe} := 15in$ $\quad V_{max} = 20.19\ kip$ $\quad M_{max} = 181.69\ kip\cdot ft$ Torsion

$M_{Flexure} := V_{max}\cdot L_{Short\_Pipe}$ $\qquad M_{Flexure} = 25.23\ kip\cdot ft$

Design Flexural Strength

$\phi_b := 0.90$ $\quad \lambda_{pipe} := D\_t$ $\quad \lambda_{p\_pipe} := 0.0714\cdot E \div F_{y\_pipe}$

$\lambda_{p\_pipe} = 49.3$ $\quad \lambda_{pipe} = 34.4$

$M_{d\_Short\_Pipe} := if(\lambda_{p\_pipe} \geq \lambda_{pipe}, \phi_b\cdot F_{y\_pipe}\cdot Z_{pipe}, \text{"Equation Invalid"})$

$M_{d\_Short\_Pipe} = 352.8\ kip\cdot ft$

$$Check\_Flexure\_Short\_Pipe := \begin{array}{|l} \text{"Sufficient Strength"} \ \text{ if } M_{d\_Short\_Pipe} \geq M_{Flexure} \\ \text{"Insufficient Strength"} \ \text{ if } M_{d\_Short\_Pipe} < M_{Flexure} \end{array}$$

Check_Flexure_Short_Pipe = "Sufficient Strength"

Design Shear Strength

$\phi_v := 0.9$

$$F_{cr} := max\left(\frac{1.60\cdot E}{\sqrt{\frac{L_{Short\_Pipe}}{D_{pipe}}}\cdot D\_t^{\frac{5}{4}}}, \frac{0.78E}{D\_t^{\frac{3}{2}}}\right) \qquad F_{cr} = 575.22\ ksi$$

$F_{cr} := min(0.6\cdot F_{y\_pipe}, F_{cr})$ $\qquad F_{cr} = 25.2\ ksi$

$V_{d\_Short\_Pipe} := \phi_v\cdot F_{cr}\cdot A_{pipe} \div 2$ $\qquad V_{d\_Short\_Pipe} = 257.42\ kip$

$$Check\_Shear\_Short\_Pipe := \begin{array}{|l} \text{"Sufficient Strength"} \ \text{ if } V_{d\_Short\_Pipe} \geq V_{max} \\ \text{"Insufficient Strength"} \ \text{ if } V_{d\_Short\_Pipe} < V_{max} \end{array}$$

Check_Shear_Short_Pipe = "Sufficient Strength"

Design Torsional Strength

$\phi_T := 0.90 \qquad T_{d\_pipe} := \phi_T \cdot F_{cr} \cdot C_{pipe}$ $\qquad T_{d\_pipe} = 323.19\ kip \cdot ft$

$$Check\_Torsion\_Short\_Pipe := \begin{array}{|l} \text{"Sufficient Strength" if } T_{d\_pipe} \geq M_{max} \\ \text{"Insufficient Strength" if } T_{d\_pipe} < M_{max} \end{array}$$

$Check\_Torsion\_Short\_Pipe = \text{"Sufficient Strength"}$

Check Interaction

$$Interaction\_Short\_Pipe := \left(\frac{M_{Flexure}}{M_{d\_Short\_Pipe}}\right) + \left(\frac{V_{max}}{V_{d\_Short\_Pipe}} + \frac{M_{max}}{T_{d\_pipe}}\right)^2$$

$Interaction\_Short\_Pipe = 0.48$

$$Check\_Interaction\_Short\_Pipe := \begin{array}{|l} \text{"Sufficient Strength" if } Interaction\_Short\_Pipe \leq 1 \\ \text{"Insufficient Strength" if } Interaction\_Short\_Pipe > 1 \end{array}$$

$Check\_Interaction\_Short\_Pipe = \text{"Sufficient Strength"}$

Design of Long Section- Applied Shear and Flexure

$L_{Long\_Pipe} := 9ft \qquad V_{max} = 20.19\ kip \qquad M_{max} = 181.69\ kip \cdot ft$

Design Flexural Strength

$M_{d\_Long\_Pipe} := if\left(\lambda_{p\_pipe} \geq \lambda_{pipe}, \phi_b \cdot F_{y\_pipe} \cdot Z_{pipe}, \text{"Equation Invalid"}\right)$

$M_{d\_Long\_Pipe} = 352.8\ kip \cdot ft$

$$Check\_Flexure\_Long\_Pipe := \begin{array}{|l} \text{"Sufficient Strength" if } M_{d\_Long\_Pipe} \geq M_{max} \\ \text{"Insufficient Strength" if } M_{d\_Long\_Pipe} < M_{max} \end{array}$$

$Check\_Flexure\_Long\_Pipe = \text{"Sufficient Strength"}$

Design Shear Strength

$V_{d\_Long\_Pipe} := \phi_v \cdot F_{cr} \cdot A_{pipe} \div 2$ $\qquad V_{d\_Long\_Pipe} = 257.42\ kip$

$$Check\_Shear\_Long\_Pipe := \begin{array}{|l} \text{"Sufficient Strength" if } V_{d\_Long\_Pipe} \geq V_{max} \\ \text{"Insufficient Strength" if } V_{d\_Long\_Pipe} < V_{max} \end{array}$$

$Check\_Shear\_Long\_Pipe = \text{"Sufficient Strength"}$

Check Interaction

$$\text{Interaction\_Long\_Pipe} := \left(\frac{M_{max}}{M_{d\_Long\_Pipe}}\right) + \left(\frac{V_{max}}{V_{d\_Long\_Pipe}}\right)^2$$

$$\text{Interaction\_Long\_Pipe} = 0.52$$

$$\text{Check\_Interaction\_Long\_Pipe} := \begin{cases} \text{"Sufficient Strength" if Interaction\_Long\_Pipe} \le 1 \\ \text{"Insufficient Strength" if Interaction\_Long\_Pipe} > 1 \end{cases}$$

$$\text{Check\_Interaction\_Long\_Pipe} = \text{"Sufficient Strength"}$$

Weight of Pipe Apparatus

$$W_{pipe\_app} := W_{pipe} \cdot L_{Short\_Pipe} + W_{pipe} \cdot L_{Long\_Pipe} + \underbrace{490 \frac{lbf}{ft^3} \cdot \pi \left(\frac{24in}{2}\right)^2 \cdot 1in}_{\text{Base Plate}} + \underbrace{26in \cdot 20in \cdot 0.5in \cdot 490 \cdot \frac{lbf}{ft^3}}_{\text{Weld Plate}}$$

$$W_{pipe\_app} = 1.05\,kip$$

This weight must be subtracted from the applied load to account for overcoming the weight of the pipe before the bolts were loaded. Will be used to normalize the test results

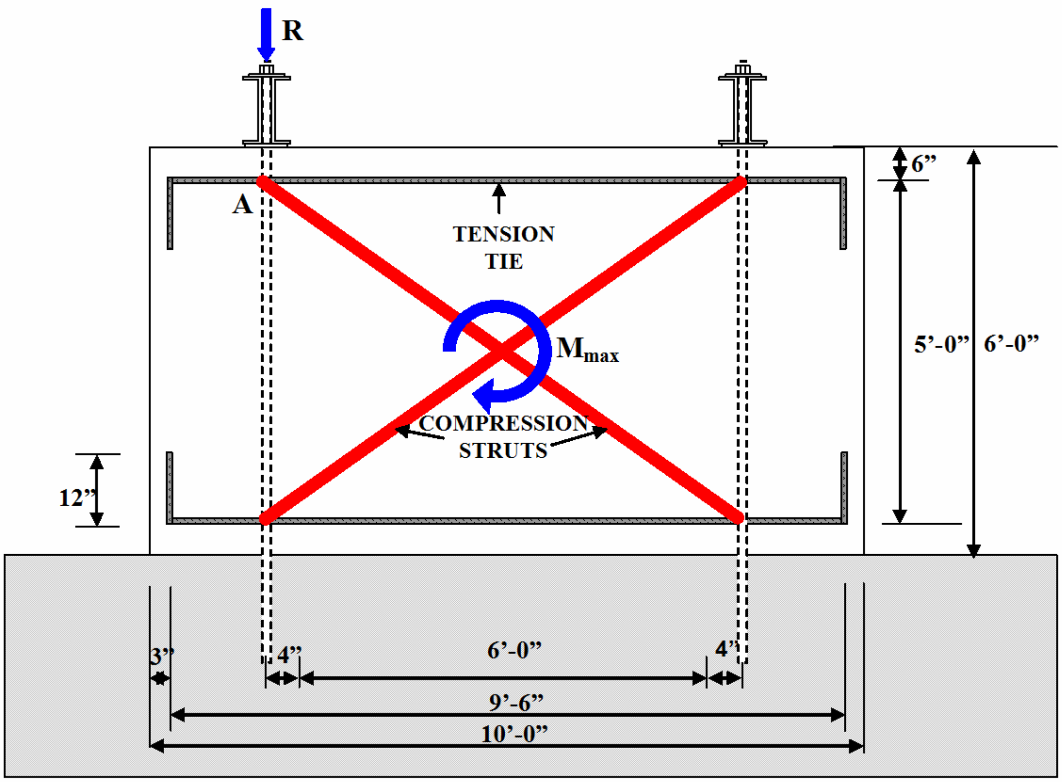
Concrete Block Design

Reinforcement

Two different methods were employed to check the reinforcement in the block:

A) Strut-and-Tie Model

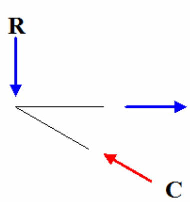
B) Beam Theory



A) Design by Strut-and-Tie Model ACI 318-05 Appendix A

$M_{max} = 181.69\ \text{kip}\cdot\text{ft}$ $\quad d := 6\text{ft} + 8\text{in}$ $\quad R := \dfrac{M_{max}}{d}$ $\quad R = 27.25\ \text{kip}$

NODE A



$\theta := \operatorname{atan}\left(\dfrac{5}{d \div \text{ft}}\right)$ $\quad \theta = 0.64$

$C := \dfrac{R}{\sin(\theta)}$ $\quad C = 45.42\ \text{kip}$

$T := C\cdot\cos(\theta)$ $\quad T = 36.34\ \text{kip}$

Check Reinforcement

$No\_Bars\_Block\_Reinforcement := 3$ $\quad Block\_Reinforcement\_Bar\_No := 8$ $\quad f_{y\_Block\_Reinforcement} := 60ksi$

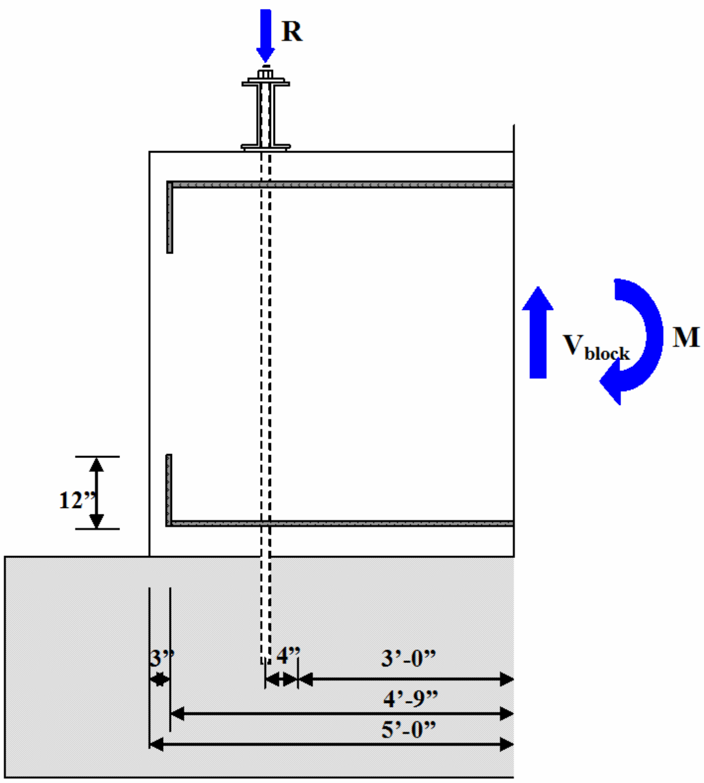
$$A_{Block\_Reinforcement} := No\_Bars\_Block\_Reinforcement \cdot \pi \cdot \left(\frac{Block\_Reinforcement\_Bar\_No \div 8}{2}\right)^2 \cdot in^2$$

$$A_{Block\_Reinforcement} = 2.36\ in^2$$

$$Check\_Reinforcement\_A := \left| \begin{array}{l} \text{"Sufficient Strength"} \text{ if } \left(A_{Block\_Reinforcement} \cdot f_{y\_Block\_Reinforcement}\right) \geq T \\ \text{"Insufficient Strength"} \text{ if } A_{Block\_Reinforcement} \cdot f_{y\_Block\_Reinforcement} < T \end{array} \right.$$

$$Check\_Reinforcement\_A = \text{"Sufficient Strength"}$$

B) Beam Theory



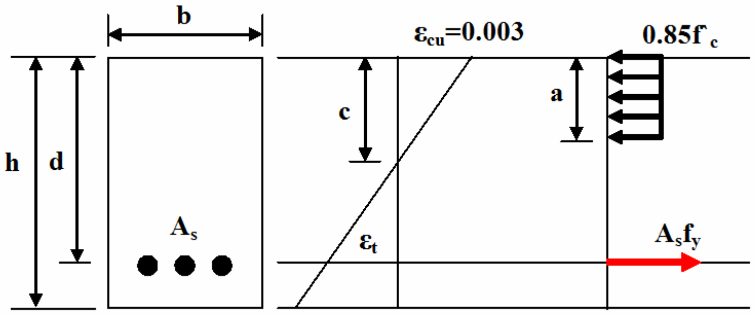
$V_{block} := R$ $\quad V_{block} = 27.25\ kip$ $\quad M := R \cdot (3ft + 4in)$ $\quad M = 90.84\ kip \cdot ft$

Check Shear

$$\text{Check\_Shear\_B} := \begin{cases} \text{"Sufficient Strength"} & \text{if } \left(A_{Block\_Reinforcement} \cdot f_{y\_Block\_Reinforcement}\right) \geq V_{block} \\ \text{"Insufficient Strength"} & \text{if } A_{Block\_Reinforcement} \cdot f_{y\_Block\_Reinforcement} < V_{block} \end{cases}$$

$\text{Check\_Shear\_B} = \text{"Sufficient Strength"}$

Check Flexure



$b := 30in$
$h := 6ft$
$d := 5.5ft$

Locate the neutral axis, c, such that (C=T)

$T := A_{Block\_Reinforcement} \cdot f_{y\_Block\_Reinforcement}$ $\quad T = 141.37\ kip$ $\quad C(a) := 0.85 \cdot f'_c \cdot b \cdot a$

$CT(a) := C(a) - T$ $\quad a := root(CT(a), a, 0, h)$ $\quad a = 1.01\ in$

$$\beta_1(f'_c) := \begin{cases} 0.85 & \text{if } f'_c < 4000 \cdot psi \\ 0.65 & \text{if } f'_c > 8000 \cdot psi \\ 0.85 - 0.05 \cdot \left(\dfrac{f'_c - 4000 \cdot psi}{1000 \cdot psi}\right) & \end{cases}$$

ACI 10.2.7.3

$\beta_1(f'_c) = 0.78$ $\quad \beta := \beta_1(f'_c)$ $\quad \beta = 0.78$

$c := \dfrac{a}{\beta}$ $\quad$ ACI 10.2.7.1 $\quad c = 16.14\ in$

Calculate the nominal moment capacity, M_n

Capacity Reduction Factor $\quad$ ACI 9.3.2, ACI 10.3.4, ACI 10.9.3

$M_{n\_Block} := T \cdot \left(d - \dfrac{a}{2}\right)$ $\quad M_{n\_Block} = 771.6\ kip \cdot ft$

$$\text{Check\_Flexure\_B} := \begin{cases} \text{"Sufficient Strength"} & \text{if } M_{n\_Block} \geq M \\ \text{"Insufficient Strength"} & \text{if } M_{n\_Block} < M \end{cases}$$

$\text{Check\_Flexure\_B} = \text{"Sufficient Strength"}$

Required Hook Length for a #8 Bar

$$\text{Hook\_No\_8} := 12 \cdot \left(\frac{\text{Block\_Reinforcement\_Bar\_No}}{8} \text{in} \right)$$ ACI R12.5

$\text{Hook\_No\_8} = 12\,\text{in}$

Summary of Concrete Block Design Reinforcement

Block_Reinforcement_Bar_No = 8

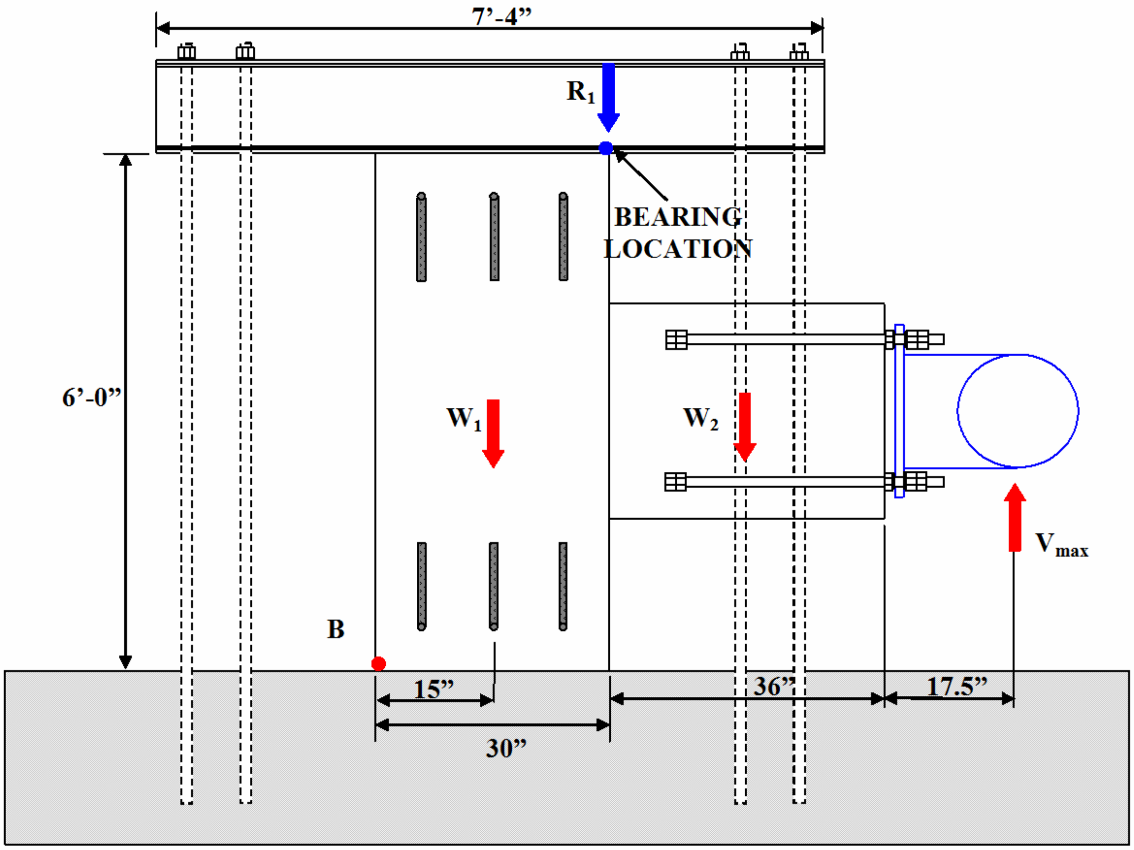
No_Bars_Block_Reinforcement = 3   3 bars on top and bottom

Check_Reinforcement_A = "Sufficient Strength"

Check_Shear_B = "Sufficient Strength"

Check_Flexure_B = "Sufficient Strength"

Tie-Down Design



Calculate the Load, $\mathbf{R_1}$, that the tie-down must resist in this direction

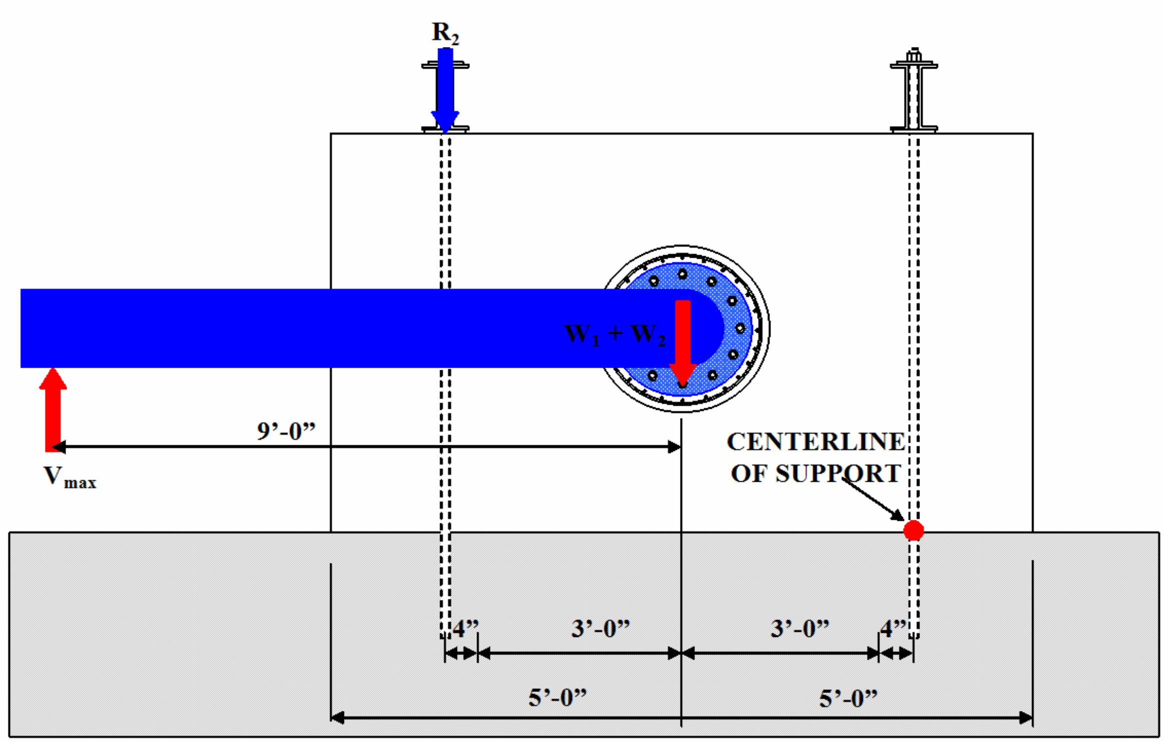
$$W_1 := (30in)\cdot(6ft)\cdot(10ft)\cdot 150\,\frac{lbf}{ft^3} \qquad W_1 = 22.5\,kip$$

$$W_2 := \pi\cdot\left(\frac{d_s}{2}\right)^2\cdot(36in)\cdot 150\,\frac{lbf}{ft^3} \qquad W_2 = 2.21\,kip$$

Note: The weight of the pipe apparatus was not accounted for because V_{max} cancels it out. The test results will be normalized to account for the weight of the pipe.

Sum of the Moments Around Point B

$$R_1 := -\frac{W_2\cdot\left(\frac{36in}{2} + 30in\right) + W_1\cdot(15in) - V_{max}\cdot(36in + 17.5in + 30in)}{30in} \qquad R_1 = 41.4\,kip$$



Calculate the Load, $\mathbf{R_2}$, that the tie-down must resist in this direction

$$R_2 := \frac{V_{max}\cdot(9ft + 3ft + 4in) - (W_1 + W_2)\cdot(3ft + 4in)}{6ft + 8in} \qquad R_2 = 24.99\ kip$$

Total Load that each tie-down must resist

$$R := \frac{R_1}{2} + R_2 \qquad R = 45.69\ kip$$

Check that the load is less than the capacity of the floor

Floor_Capacity := 100kip

Check_Floor_Capacity := if(Floor_Capacity ≥ R, "Capacity OK", "Capacity Exceeded")

Check_Floor_Capacity = "Capacity OK"

Check Bearing Strength of Concrete

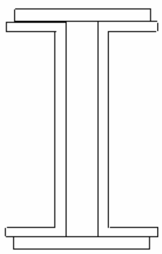
Assume the load bears on 1 in. of concrete across the length of the block

$A_{bearing} := 10ft\cdot 1in \qquad A_{bearing} = 120\ in^2 \qquad \phi_{bearing} := 0.65$

$\text{Bearing\_Strength} := \phi_{bearing}\cdot 0.85\cdot f_c\cdot A_{bearing} \qquad \text{Bearing\_Strength} = 364.65\ kip$

Check_Bearing_Capacity := if(Bearing_Strength ≥ 2·R, "Sufficient Strength", "Insufficient Strength")

Check_Bearing_Capacity = "Sufficient Strength"



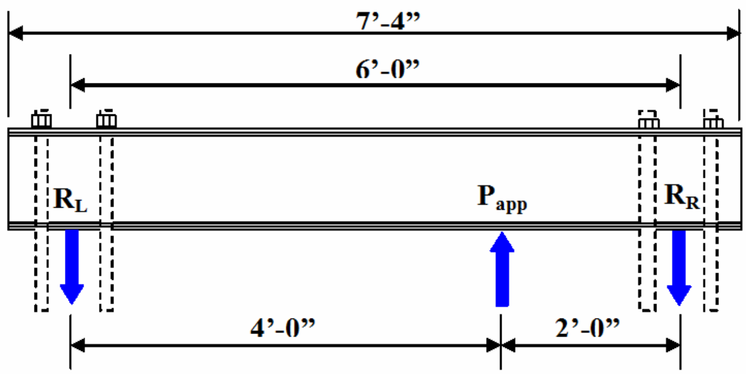
2 C12x30 Channels- 1.75" space between

$P_{app} := R$ $P_{app} = 45.69\ kip$

$I_x := 162in^4$ $S_x := 27.0in^3$ $r_x := 4.29in$ $Z_x := 33.8in^3$ $F_y := 50ksi$

$A := 8.81in^2$ $I_y := 5.12in^4$ $r_y := 0.762in$ $x\_bar := 0.674in$ $E := 29000ksi$

$t_w := 0.510in$ $b_f := 3.17in$ $t_f := 0.501in$ $h := 12in$



$$R_L := \frac{P_{app} \cdot 2ft}{6ft}$$

$R_L = 15.23\ kip$

$R_R := P_{app} - R_L$

$R_R = 30.46\ kip$

$a := 4ft$ $b := 2ft$ $M_{max\_tiedown} := \frac{P_{app} \cdot a \cdot b}{a + b}$ $M_{max\_tiedown} = 60.93\ kip \cdot ft$

Flexural Capacity of Channels

$M_{n\_tiedown} := F_y \cdot (2 \cdot Z_x)$ $M_{n\_tiedown} = 281.67\ kip \cdot ft$

Check_Flexure_Channels := if($M_{n\_tiedown} \geq M_{max\_tiedown}$, "Capacity OK", "Capacity Exceeded")

Check_Flexure_Channels = "Capacity OK"

Buckling Check

Treat as 2 Separate Channels

$\lambda_f := b_f \div (2 \cdot t_f)$ $\lambda_f = 3.16$ $\lambda_w := h \div t_w$ $\lambda_w = 23.53$

$\lambda_{pf} := 0.38 \cdot \sqrt{E \div F_y}$ $\lambda_{pf} = 9.15$ $\lambda_{pw} := 3.76 \cdot \sqrt{E \div F_y}$ $\lambda_{pw} = 90.55$

Check_Flange_Compact := if($\lambda_{pf} > \lambda_f$, "Flange is Compact", "Flange is Not Compact")

Check_Web_Compact := if($\lambda_{pw} > \lambda_w$, "Web is Compact", "Web is Not Compact")

Check_Flange_Compact = "Flange is Compact" Check_Web_Compact = "Web is Compact"

Bracing Check

$L_b := 2ft$ $L_p := 1.76 \cdot r_y \cdot \sqrt{E \div F_y}$ $L_p = 2.69\ ft$

Bracing_Check := if($L_p > L_b$, "Braced", "Unbraced") Bracing_Check = "Braced"

Treat as 1 unit

$I_{y\_unit} := 2\cdot\left[I_y + A\cdot[x\_bar + (1.75in \div 2)]^2\right]$ $\quad I_{y\_unit} = 52.52\ in^4$

$r_{y\_unit} := \sqrt{I_{y\_unit} \div A}$ $\quad r_{y\_unit} = 2.44\ in$

$L_{p\_unit} := 1.76\cdot r_{y\_unit}\cdot\sqrt{E \div F_y}$ $\quad L_{p\_unit} = 8.62\ ft$

$b_{f\_unit} := 2\cdot b_f + 1.75in$ $\quad b_{f\_unit} = 8.09\ in$

$t_{w\_unit} := 2\cdot t_w + 1.75in$ $\quad t_{w\_unit} = 2.77\ in$

$\lambda_{f\_unit} := b_{f\_unit} \div (2\cdot t_f)$ $\quad \lambda_{f\_unit} = 8.07$

$\lambda_{w\_unit} := h \div t_{w\_unit}$ $\quad \lambda_{w\_unit} = 4.33$

Check_Flange_Compact_Unit := if($\lambda_{pf} > \lambda_{f\_unit}$, "Flange is Compact", "Flange is Not Compact")

Check_Web_Compact_Unit := if($\lambda_{pw} > \lambda_{w\_unit}$, "Web is Compact", "Web is Not Compact")

Check_Flange_Compact_Unit = "Flange is Compact" Check_Web_Compact_Unit = "Web is Compact"

Bracing_Check_Unit := if($L_{p\_unit} > L_b$, "Braced", "Unbraced") Bracing_Check_Unit = "Braced"

Weld Design

$V_{weld} := \max(R_L, R_R)$ $V_{weld} = 30.46\ kip$ This shear must be transferred from the plate through the channels.

$t_{plate} := 0.5in$ $\quad b_{plate} := 5in$

$Q := (t_{plate}\cdot b_{plate})\cdot\left[(t_{plate} \div 2) + (h \div 2)\right]$ Plate area multiplied by distance to the neutral axis

$Q = 15.62\ in^3$

$I_{weld} := 2\cdot I_x + 2\cdot\left[(t_{plate}\cdot b_{plate})\cdot\left[(t_{plate} \div 2) + (h \div 2)\right]^2 + \frac{1}{12}\cdot b_{plate}\cdot t_{plate}^{\ 3}\right]$

$I_{weld} = 519.42\ in^4$

$\text{Required\_Load\_Per\_Foot} := \frac{V_{weld}\cdot Q}{I_{weld}}$ $\quad \text{Required\_Load\_Per\_Foot} = 11\ \frac{kip}{ft}$

Use a 3/8" weld

$\text{Weld\_Size} := \frac{3}{8}in$ $\quad F_{electrode} := 70ksi$ $\quad F_w := 0.6\cdot F_{electrode}$ $\quad F_w = 42\ ksi$

$\phi_{weld} := 0.75$ $\quad \text{Throat} := 0.707\cdot\text{Weld\_Size}$

$\text{Required\_Length\_Per\_Foot} := \frac{\text{Required\_Load\_Per\_Foot}}{\phi_{weld}\cdot F_w\cdot \text{Throat}}$ $\quad \text{Required\_Length\_Per\_Foot} = 1.32\ \frac{in}{ft}$

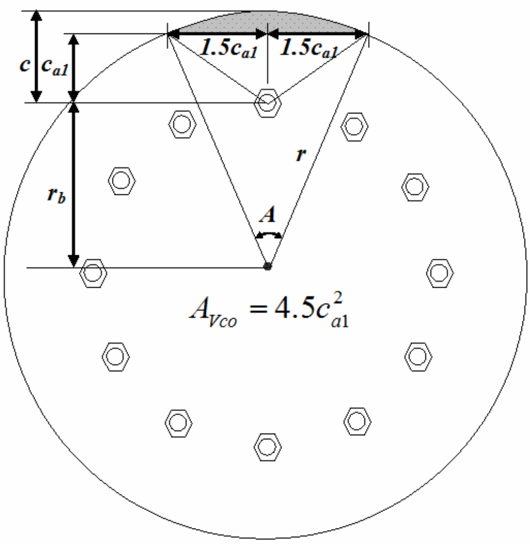
Specifiy 2" per foot of weld

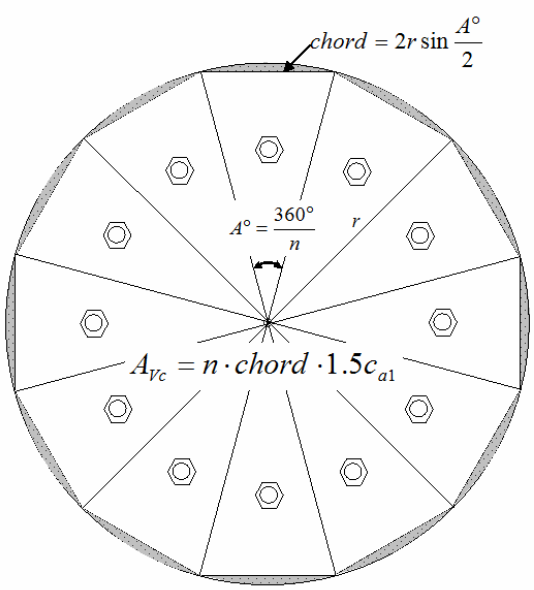
Failure Load Calculation

$f'_c := 6230\,\text{psi}$ $\quad \text{cover} := \dfrac{d_s - d_b}{2}$ Bolt Cover $\quad \text{cover} = 5\,\text{in}$

$$c_{a1} := \frac{\sqrt{\left(\frac{d_b}{2}\right)^2 + 3.25\cdot\left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_b}{2}\right)^2\right]} - \left(\frac{d_b}{2}\right)}{3.25} \quad \text{(3-2)} \qquad c_{a1} = 3.85\,\text{in}$$

Cover for the calculation of the anchor strength





$A := \dfrac{360}{\text{No\_Bolts}}\,\text{deg}$

$A = 30\,\text{deg}$

$\text{chord\_group} := 2\cdot\dfrac{d_s}{2}\cdot\sin\left(\dfrac{A}{2}\right)$

$\text{chord\_group} = 7.76\,\text{in}$

$A_{\text{min\_group}} := 2\cdot\text{asin}\left(\dfrac{3.0\cdot c_{a1}}{d_s}\right)$

$A_{\text{min\_group}} = 45.24\,\text{deg}$

If A is greater than $A_{\text{min\_group}}$ then there is no group effect

$A_{Vc} := \text{No\_Bolts}\cdot\text{chord\_group}\cdot 1.5\cdot c_{a1}$ $\quad A_{Vc} = 537.55\,\text{in}^2$ $\qquad A_{Vco} := 4.5\cdot c_{a1}^{\,2}$ $\quad A_{Vco} = 66.57\,\text{in}^2$

$$Check\_Group\_Effect := \begin{cases} \text{"Group Effect"} & \text{if } A \le A_{min\_group} \\ \text{"No Group Effect"} & \text{if } A > A_{min\_group} \end{cases}$$

$$Check\_Group\_Effect = \text{"Group Effect"}$$

ACI 318-05-D.6.2

$$l_e := 8 \cdot d_o$$

$$V_b := 13 \cdot \left(\frac{l_e}{d_o}\right)^{0.2} \cdot \sqrt{\frac{d_o}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf \qquad (2\text{-}11) \qquad V_b = 14.37\ kip$$

$\psi_{cV} := 1.4$ ACI 318-05- D.6.2.7

1.0 for cracked concrete with no supplementary reinforcement or reinforcement smaller than a No. 4 bar
1.2 for cracked concrete with edge reinforcement of a No.4 bar or greater
1.4 for uncracked concrete or with edge reinforcement of a No.4 bar or greater enclosed within stirrups spaced no more than 4 in. apart

$\psi_{ecV} := 1.0$ ACI 318-05- D.6.2.5 $\psi_{edV} := 1.0$ ACI 318-05- D.6.2.6

All anchors are loaded in shear in the same direction

$$V_{cbg} := \begin{cases} No\_Bolts \cdot \psi_{edV} \cdot V_b & \text{if } A > A_{min\_group} \\ \dfrac{A_{Vc}}{A_{Vco}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot V_b & \text{if } A \le A_{min\_group} \end{cases} \qquad (2\text{-}12) \qquad V_{cbg} = 116.02\ kip$$

$$V_{cbg\_parallel} := 2 \cdot V_{cbg} \qquad V_{cbg\_parallel} = 232.04\ kip$$

$$T_{n\_breakout\_ACI} := V_{cbg\_parallel} \cdot (d_b \div 2) \qquad T_{n\_breakout\_ACI} = 193.37\ kip \cdot ft$$

Calculation of V_{cbg} and $T_{n\_breakout\_ACI}$ are based on (3-3) and (3-4)

Retrofit Design

CFRP Properties

$t_{CFRP} = 0.02\ in$ Thickness of Wrap

$f_{n\_CFRP} = 91.1\ ksi$ Tensile Strength of CFRP

$w_{CFRP} = 12\ in$ Width of CFRP Sheet

Consider the twelve inch roll is only effective down to 1.5*cover

$w_{eff\_CFRP} := 1.5 \cdot cover \qquad w_{eff\_CFRP} = 7.5\ in$

Method 1: Conversion of $V_{perpendicular}$ to a Pressure, F_{edge}, on the Edge of the Concrete

$$V_{parallel}(T_i) := \frac{T_i}{\frac{d_b}{2} \cdot No\_Bolts} \qquad (4\text{-}1)$$

$$V_{perpendicular}(T_i) := \frac{V_{parallel}(T_i)}{2} \qquad (4\text{-}2)$$

$$p_r(T_i) := \frac{No\_Bolts}{2 \cdot \pi \cdot \frac{d_s}{2}} \cdot V_{perpendicular}(T_i) \qquad (4\text{-}3)$$

$$F_{CFRP\_Method\_1}(T_i) := p_r(T_i) \cdot \frac{d_s}{2} \qquad (4\text{-}4)$$

$F_{CFRP\_Method\_1}(T_{n\_breakout\_ACI}) = 18.47\ kip$

$F_{CFRP\_Method\_1} := F_{CFRP\_Method\_1}(T_{n\_breakout\_ACI})$

Method 2: Strut and Tie Approach

$$F_{CFRP\_Method\_2}(T_i) := \frac{T_i}{2 \cdot \left(\frac{d_b}{2}\right) \cdot No\_Bolts} \qquad (4\text{-}7)$$

$F_{CFRP\_Method\_2}(T_{n\_breakout\_ACI}) = 9.67\ kip$

$F_{CFRP\_Method\_2} := F_{CFRP\_Method\_2}(T_{n\_breakout\_ACI})$

Compare Results

$F_{CFRP\_Method\_1} = 18.47\ kip \qquad F_{CFRP\_Method\_2} = 9.67\ kip$

$F_{CFRP} := \text{if}(Check\_Group\_Effect = \text{"Group Effect"}, F_{CFRP\_Method\_1}, F_{CFRP\_Method\_2})$

$\psi_f := 0.95$ ACI 440.2R Table 10.1 $\phi := 0.75$ ACI 318-05 9.3.2.3

$$Layers\_Wrap\_Required := \frac{F_{CFRP}}{t_{CFRP} \cdot f_{n\_CFRP} \cdot w_{CFRP} \cdot \phi \cdot \psi_f}$$

The amount required will be calculated based on the concrete breakout strength as the intent is to return the speciment to that strength

$Amount\_of\_CFRP\_to\_Apply := Layers\_Wrap\_Required$

$Amount\_of\_CFRP\_to\_Apply = 1.58$

Round up to achieve a full layer

$Layers\_of\_CFRP\_to\_Apply := ceil(Amount\_of\_CFRP\_to\_Apply)$

$Layers\_of\_CFRP\_to\_Apply = 2$

Three layers of the Sika Wrap were applied to the specimen

Calculate the corresponding failure torsion

$Layers\_CFRP := 3$

$CFRP\_Eff\_Tensile\_Strength := Layers\_CFRP \cdot \left(t_{CFRP} \cdot f_{n\_CFRP} \cdot w_{eff\_CFRP} \cdot \phi \cdot \psi_f\right)$

$T_{Eff\_CFRP} := CFRP\_Eff\_Tensile\_Strength \cdot 2 \cdot \pi \cdot d_b$

$T_{Eff\_CFRP} = 229.41\ kip \cdot ft$

If the full 12 inches of the wrap were considered effective, the maximum load would be:

$Layers\_CFRP := 3$

$CFRP\_Max\_Tensile\_Strength := Layers\_CFRP \cdot \left(t_{CFRP} \cdot f_{n\_CFRP} \cdot w_{CFRP} \cdot \phi \cdot \psi_f\right)$

$T_{Max\_CFRP} := CFRP\_Max\_Tensile\_Strength \cdot 2 \cdot \pi \cdot d_b$

$T_{Max\_CFRP} = 367.05\ kip \cdot ft$

APPENDIX C
INITIAL TEST DATA

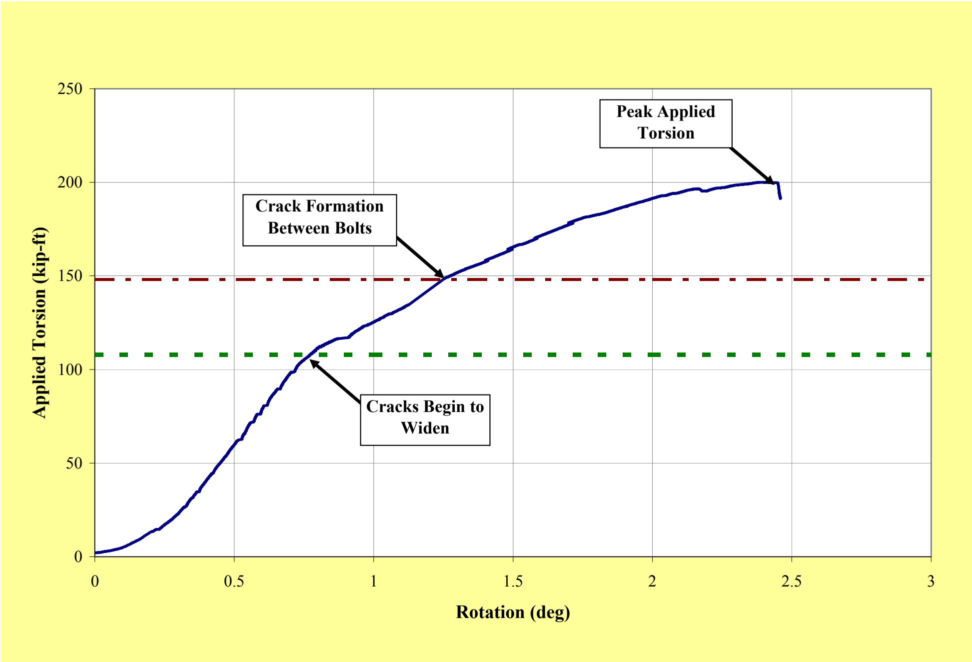


Figure C-1. Applied Torsion vs. Rotation Plot

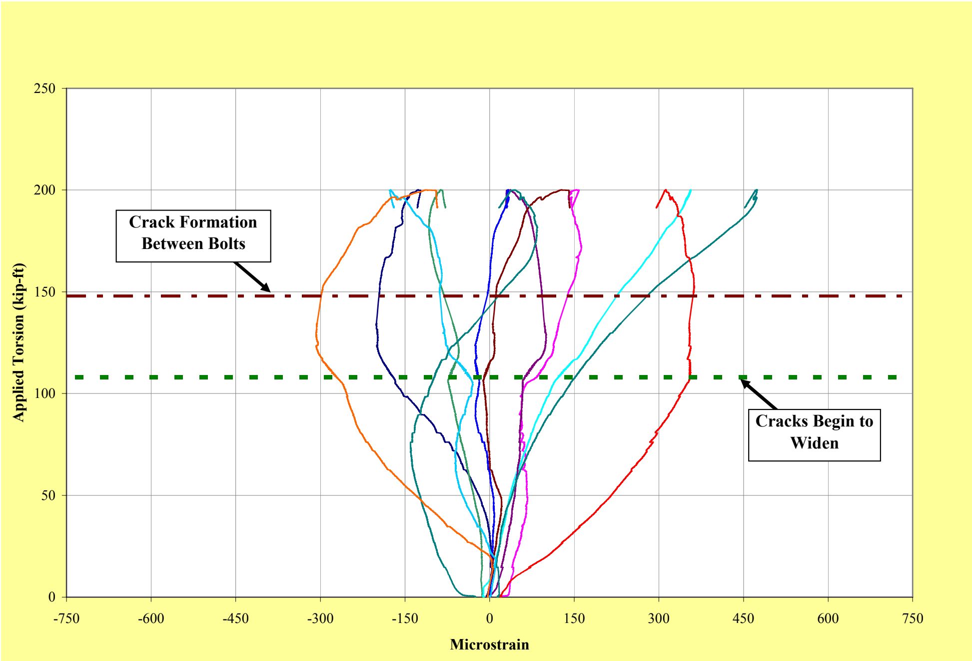
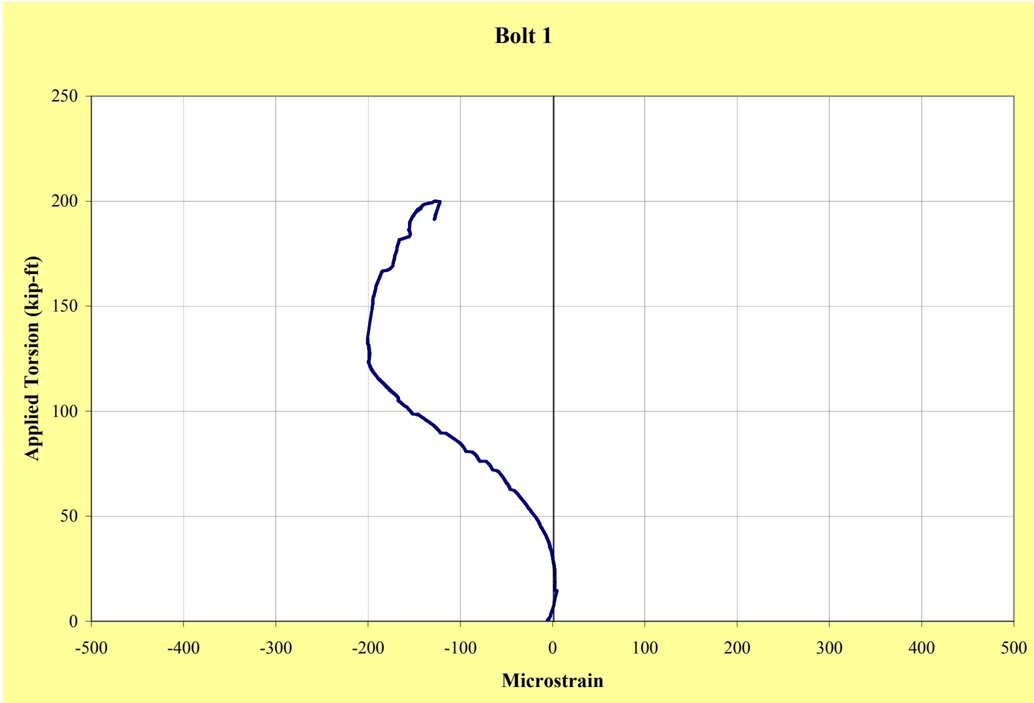
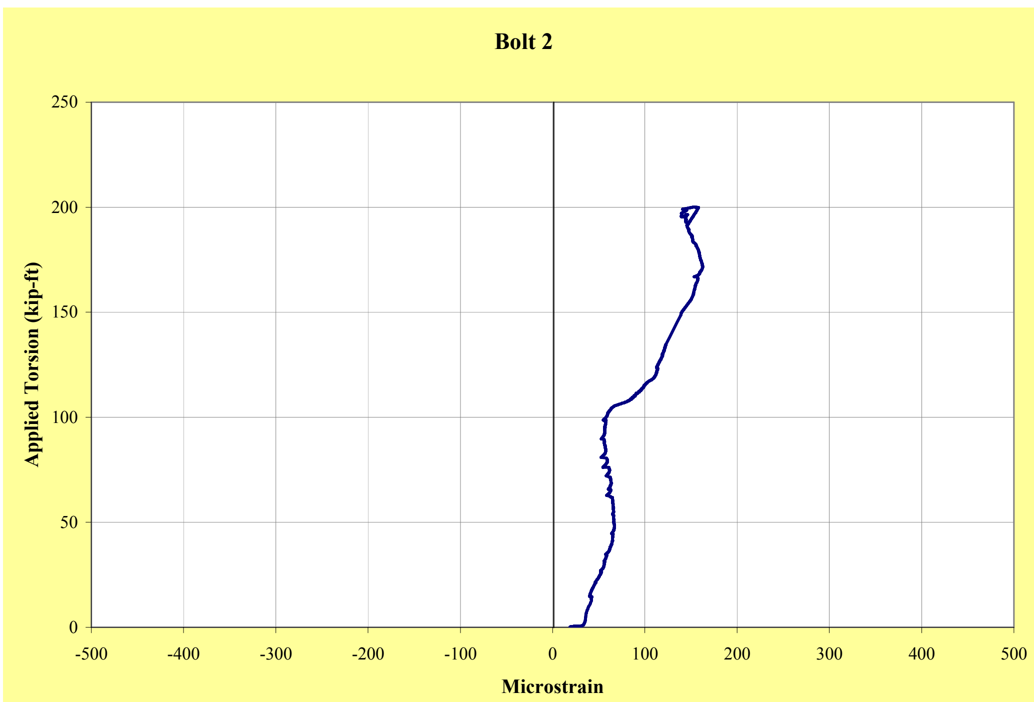


Figure C-2. Plate Strain Comparison Plot

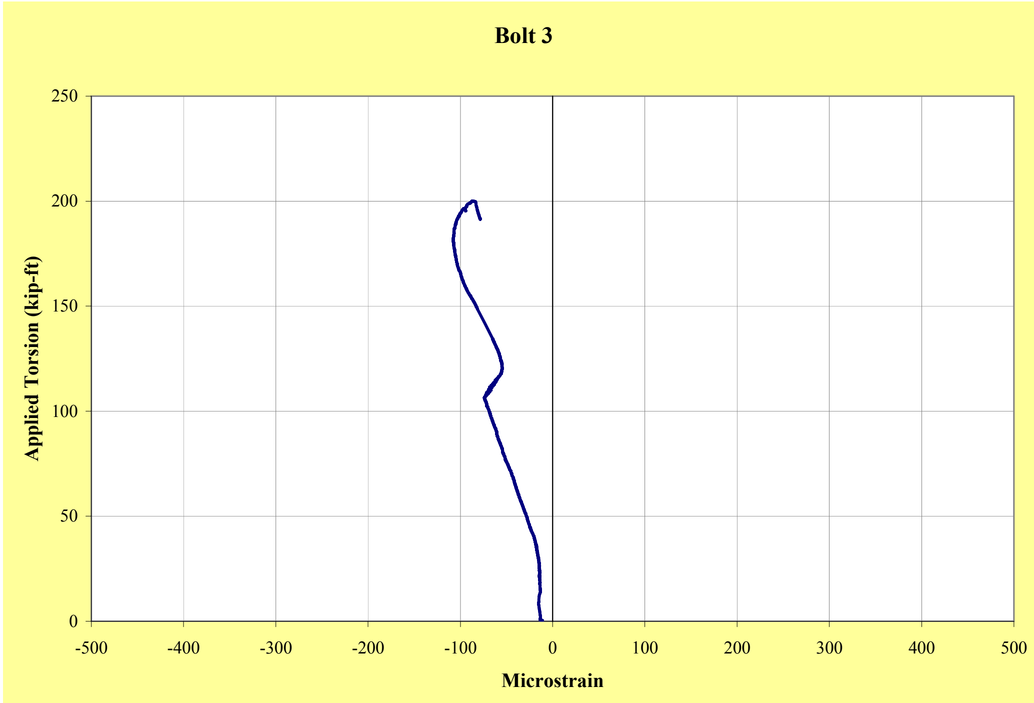


a

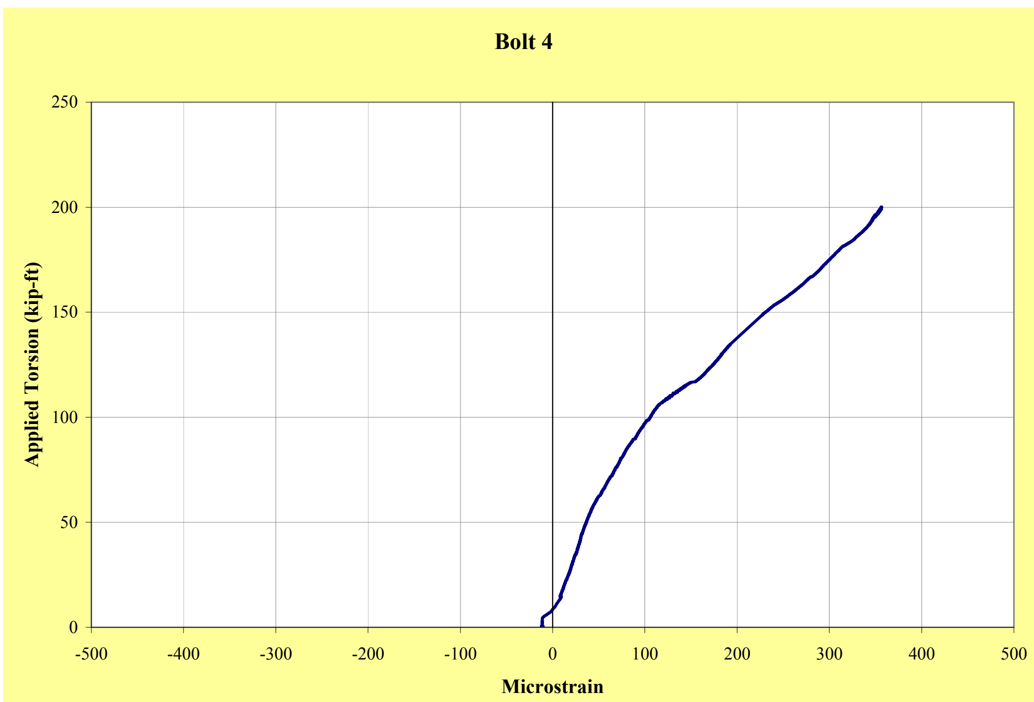


b

Figure C-3. Applied Torsion vs. Strain Plots for each bolt location

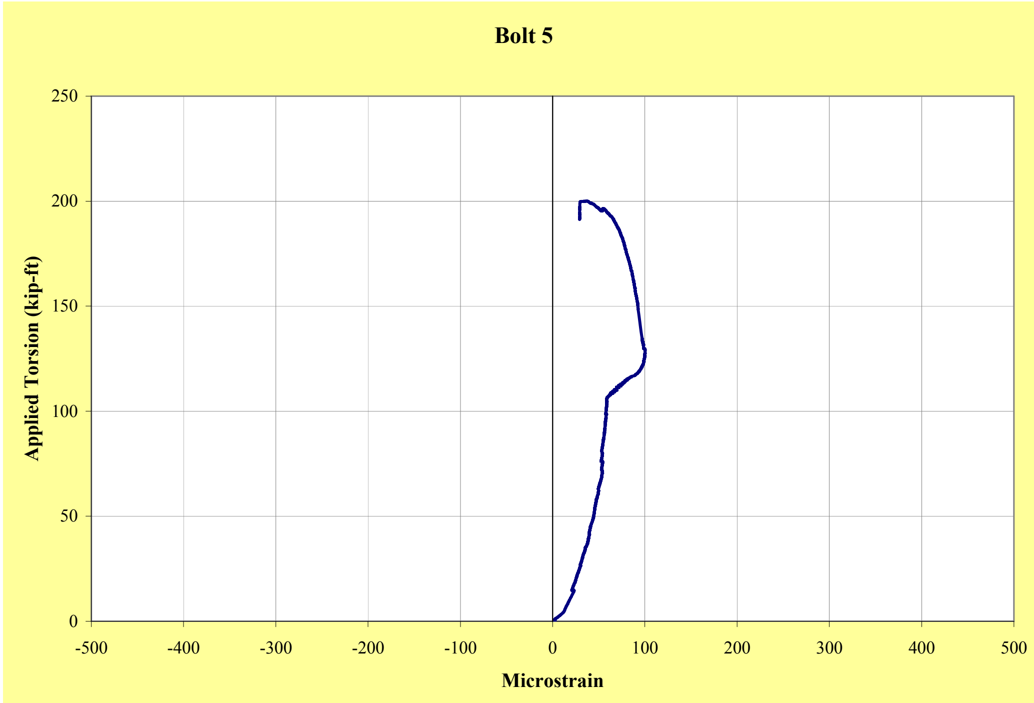


c



d

Figure C-3. Continued



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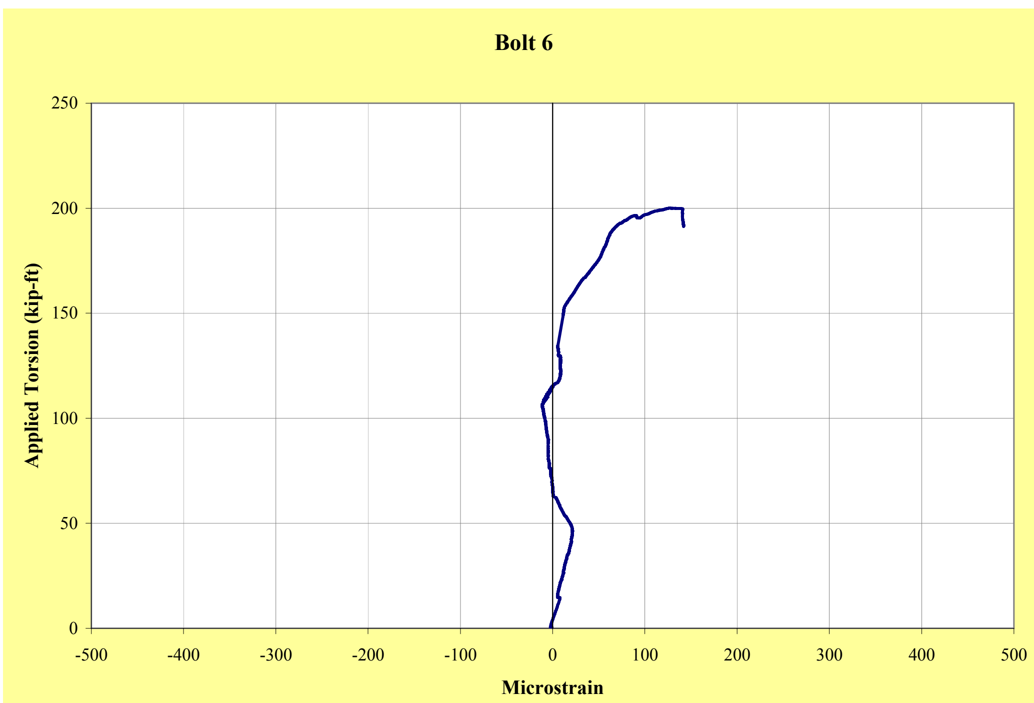
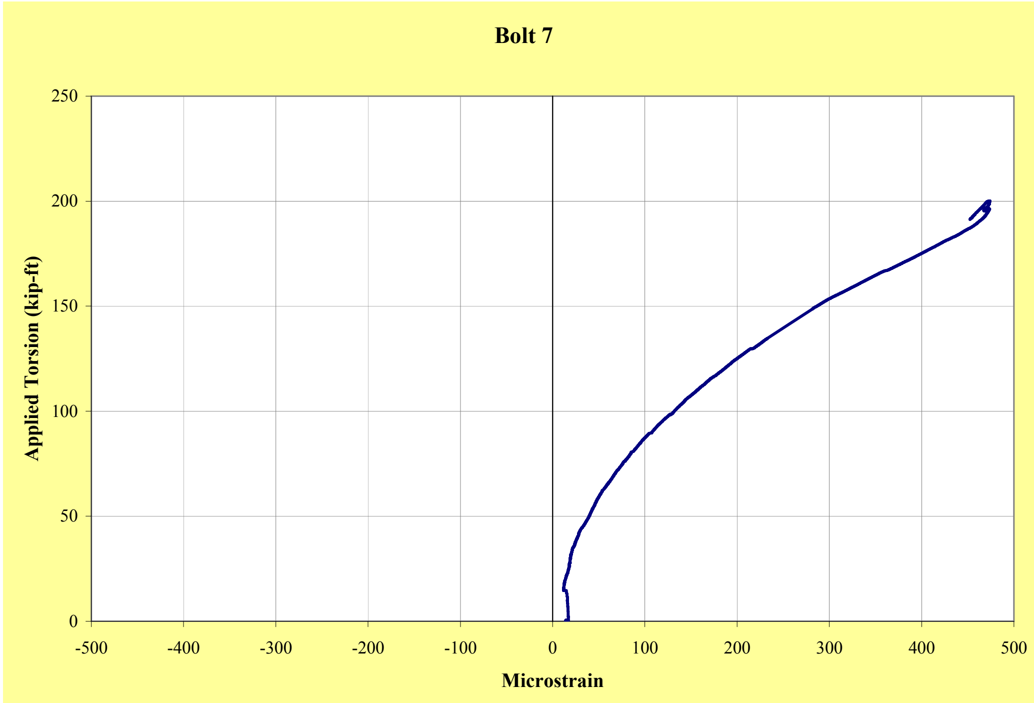


f

Figure C-3. Continued



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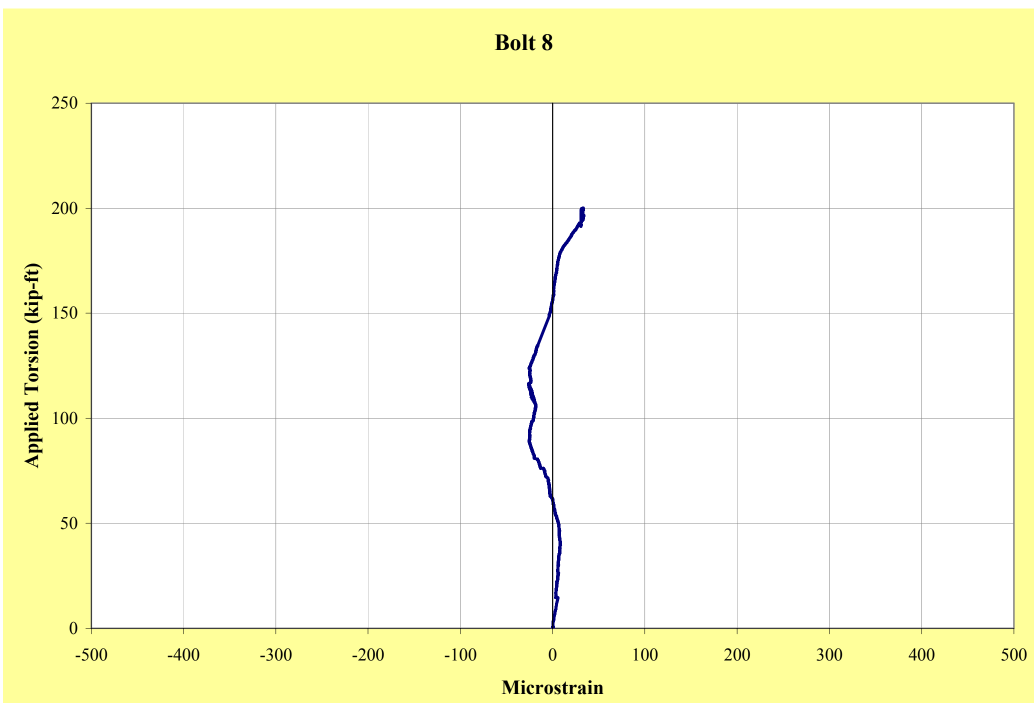
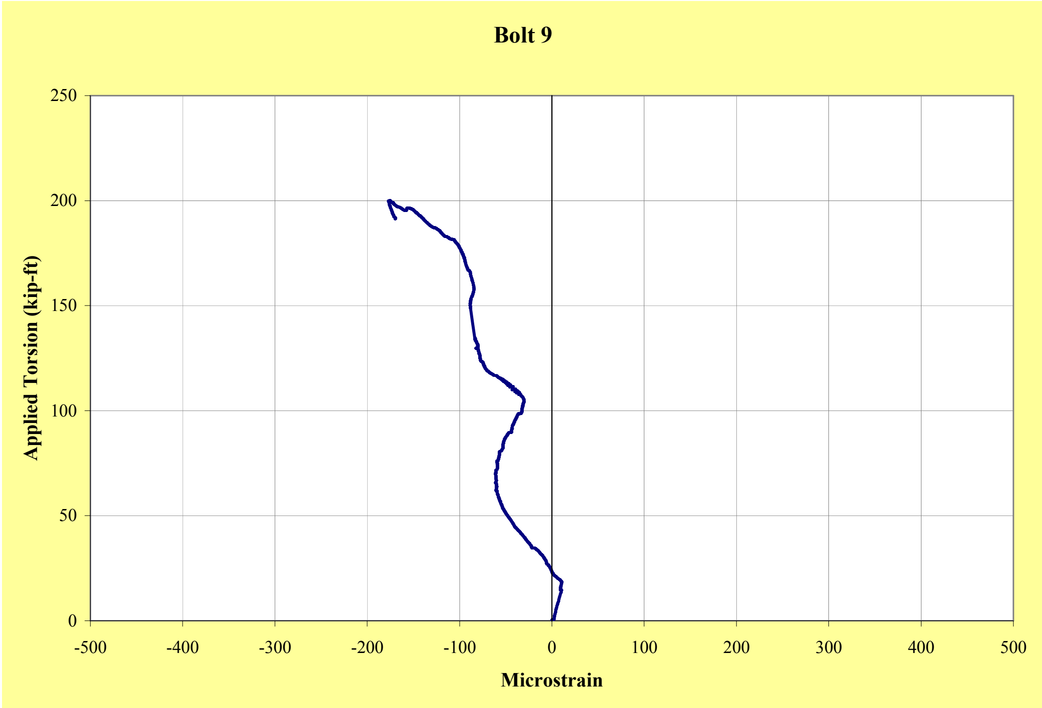
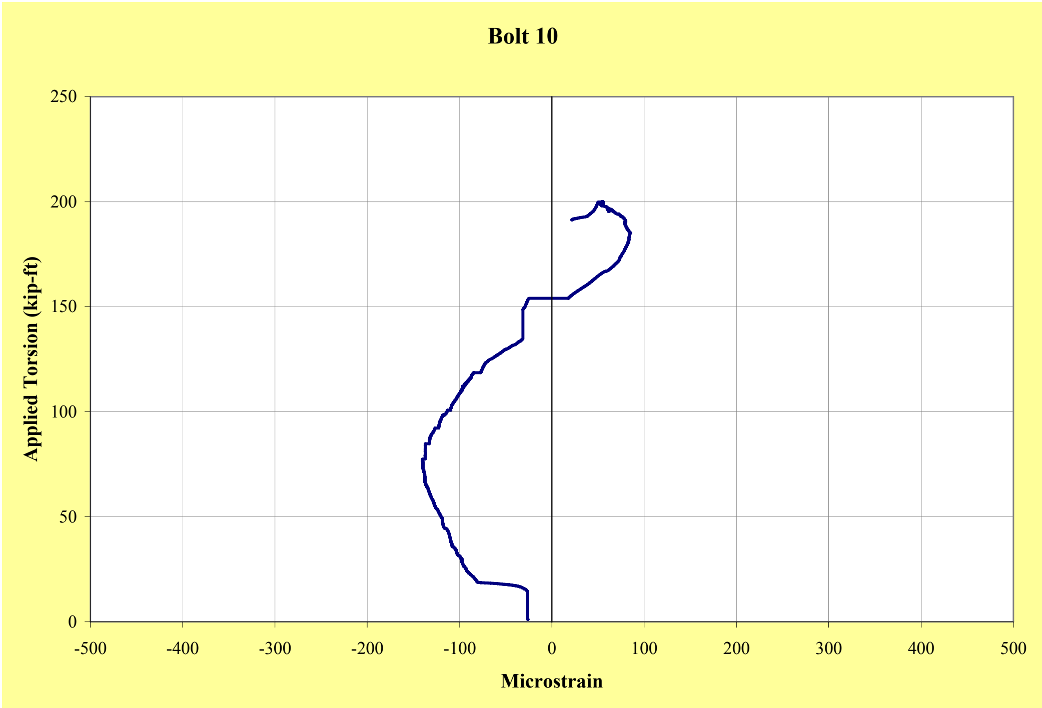


h

Figure C-3. Continued

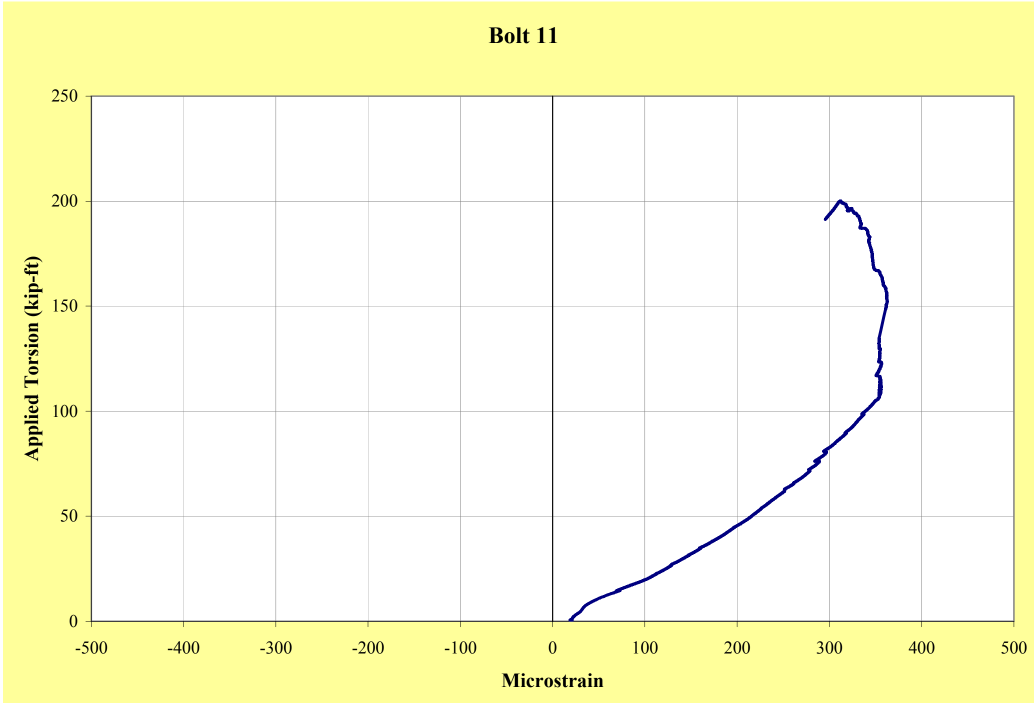


i



j

Figure C-3. Continued



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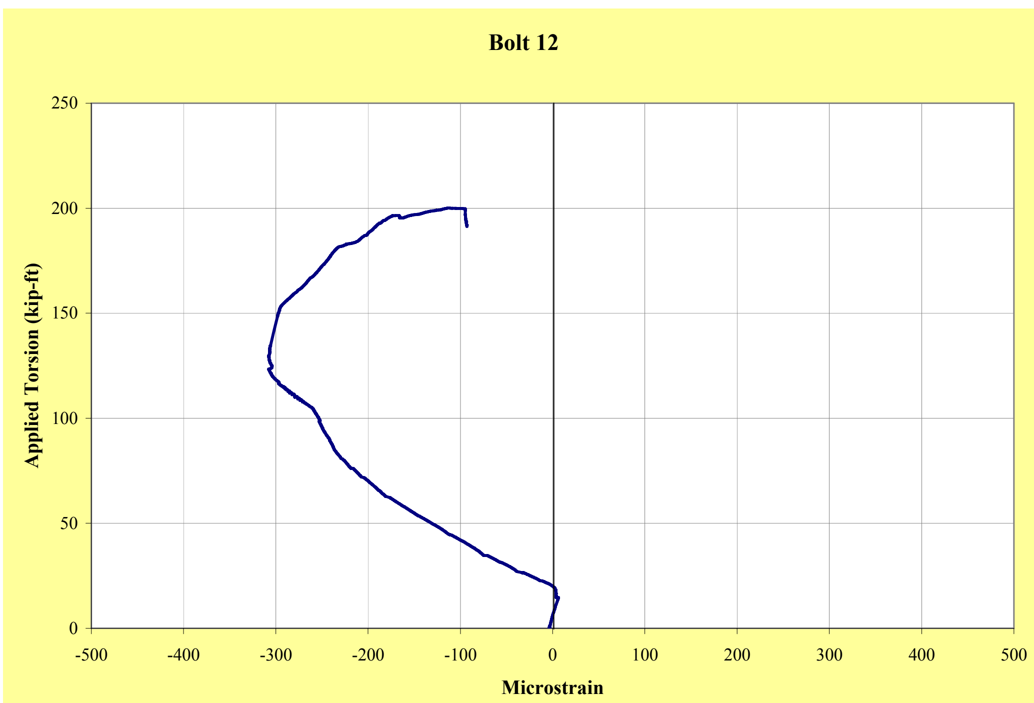


l

Figure C-3. Continued

APPENDIX D
RETROFIT TEST DATA

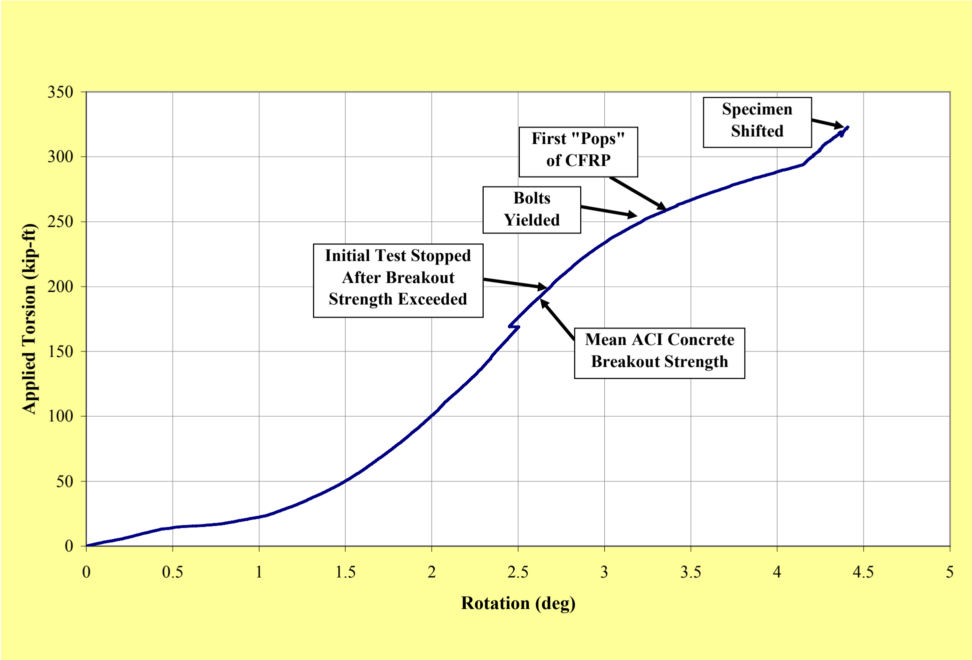


Figure D-1. Applied Torsion vs. Rotation Plot

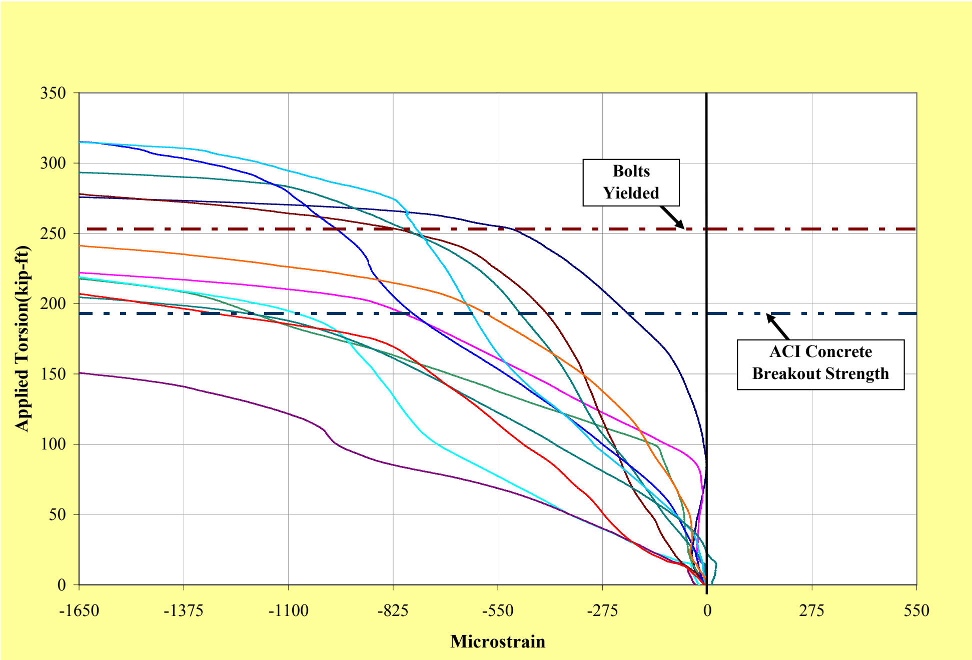
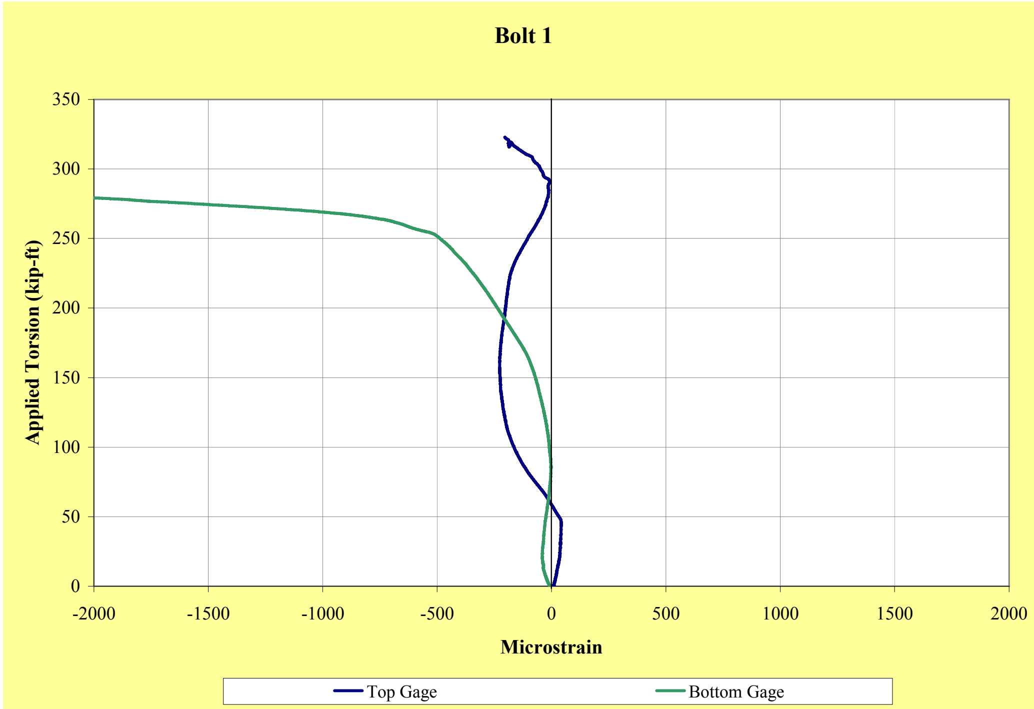
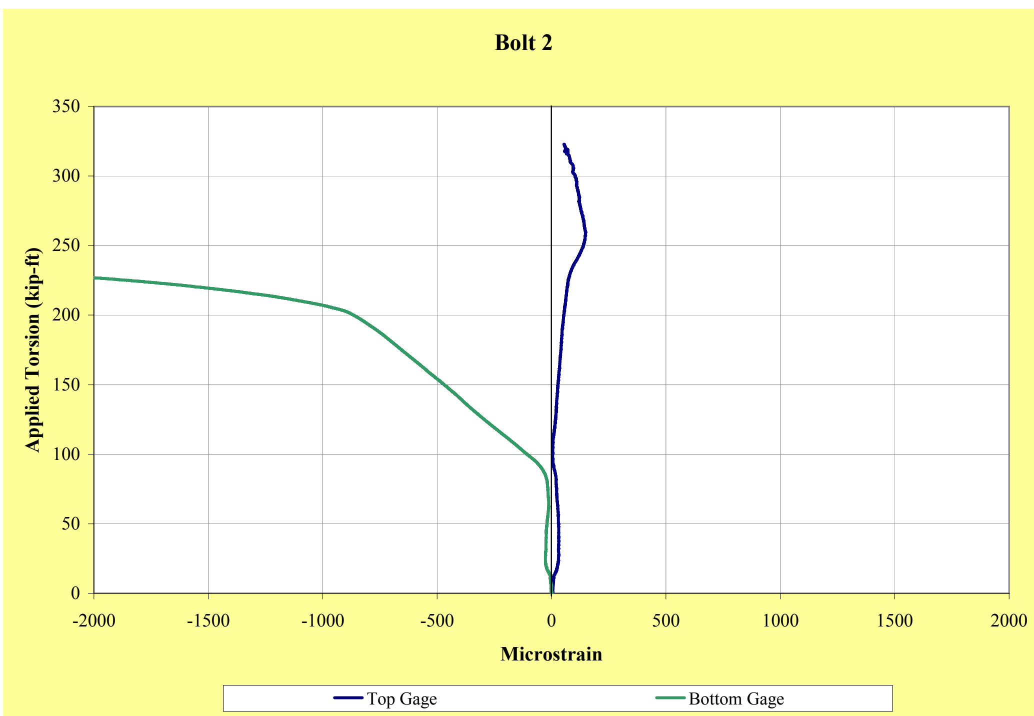


Figure D-2. Plate Strain Comparison Plot

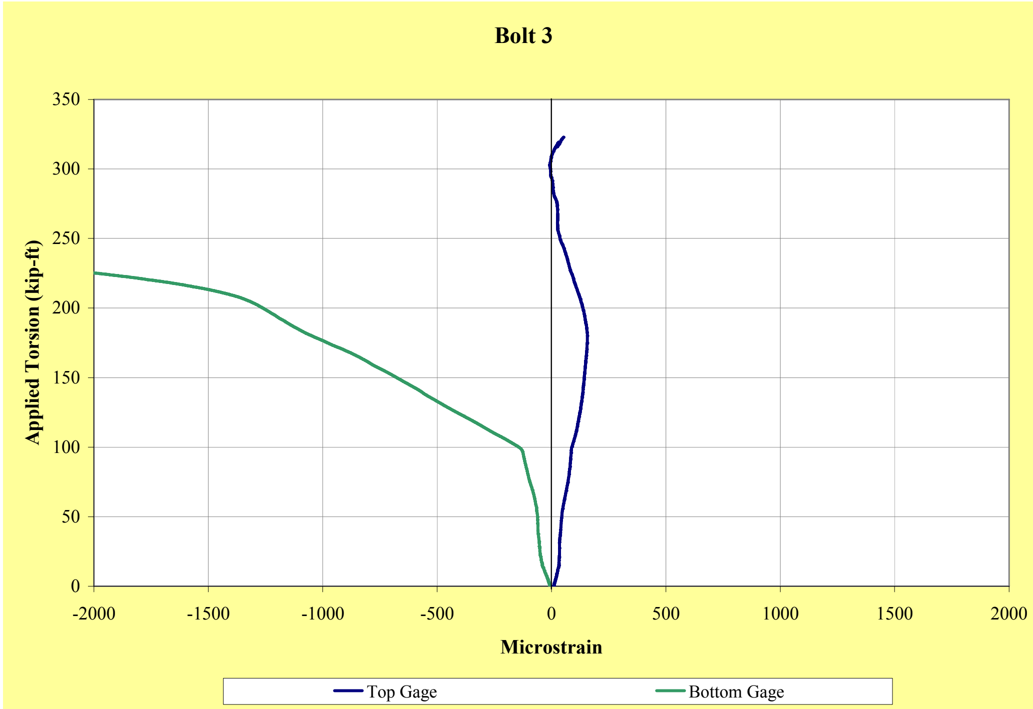


a

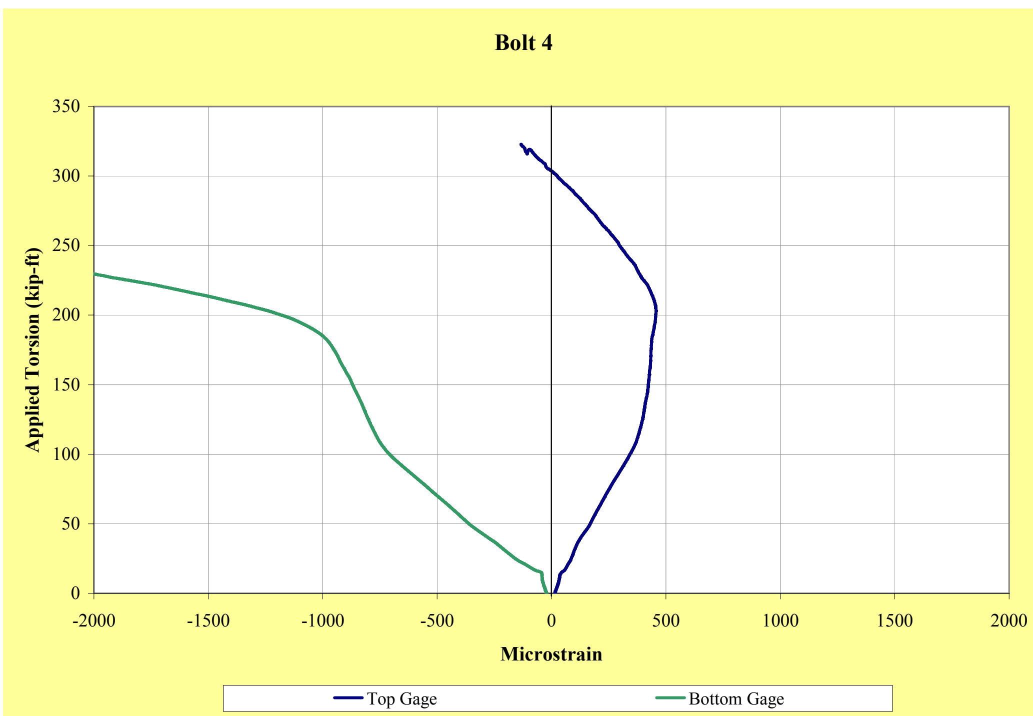


b

Figure D-3. Applied Torsion vs. Strain Plots for each bolt location

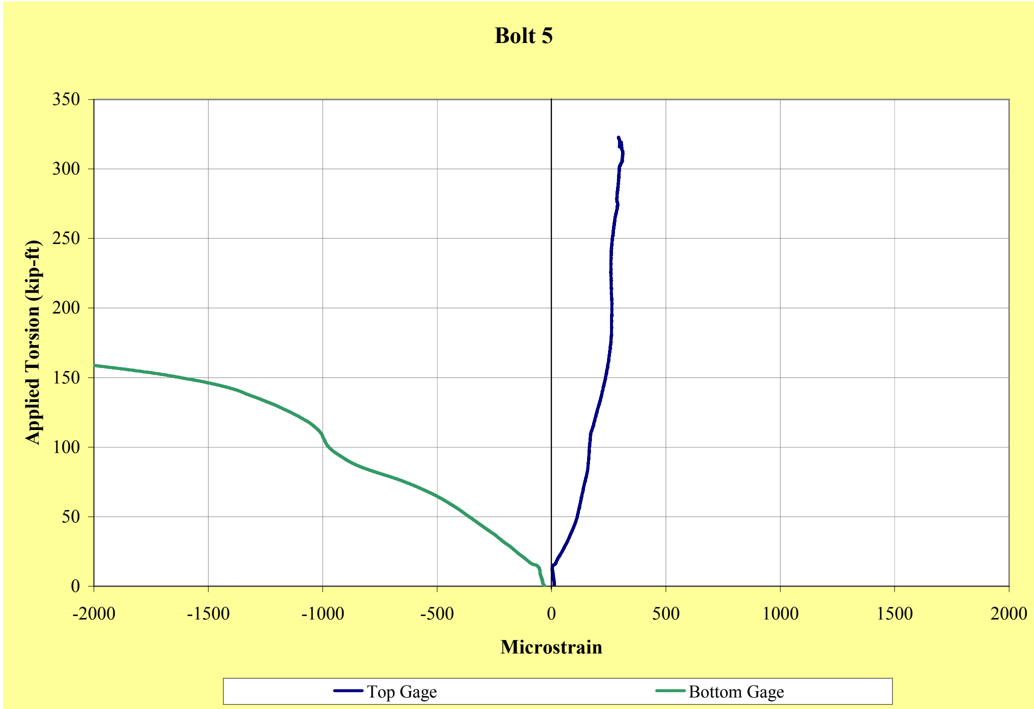


c

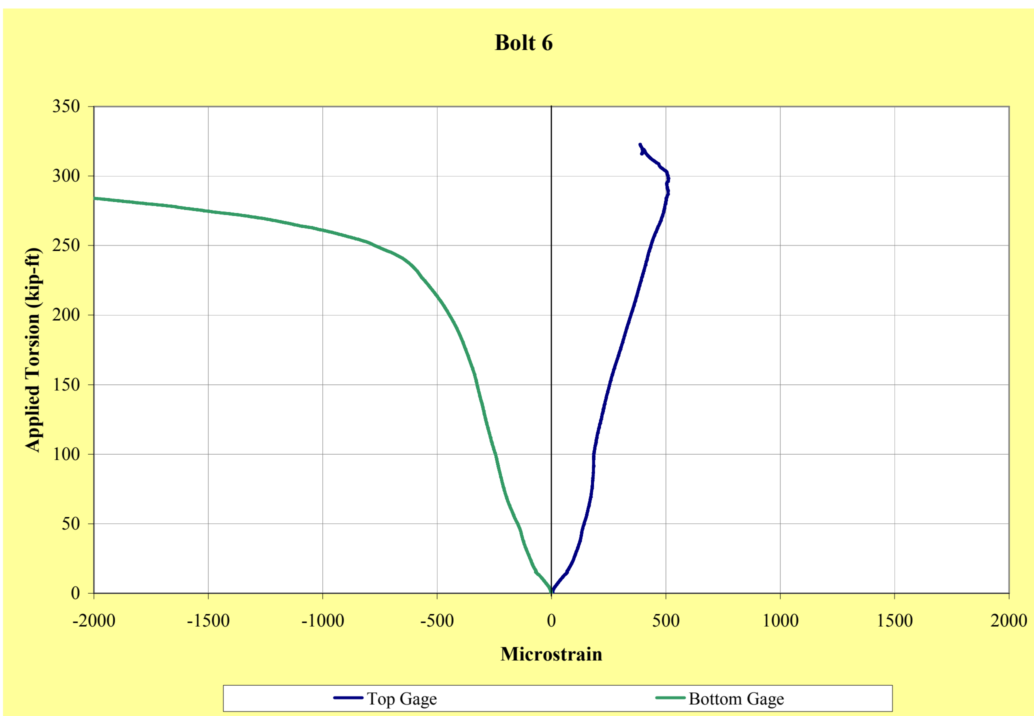


d

Figure D-3. Continued

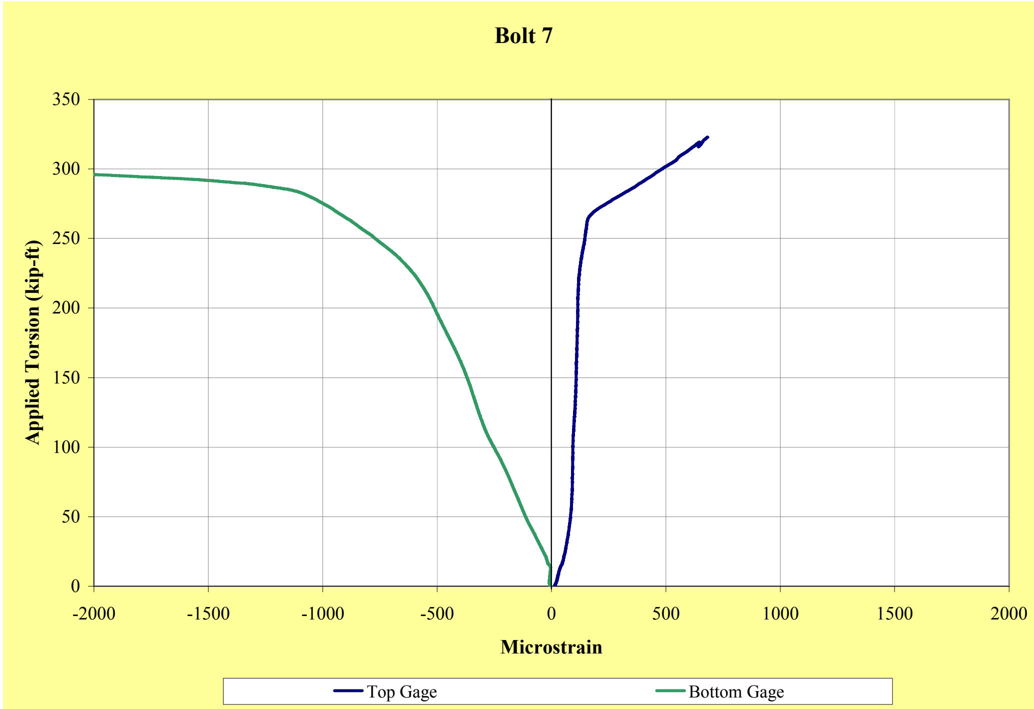


e

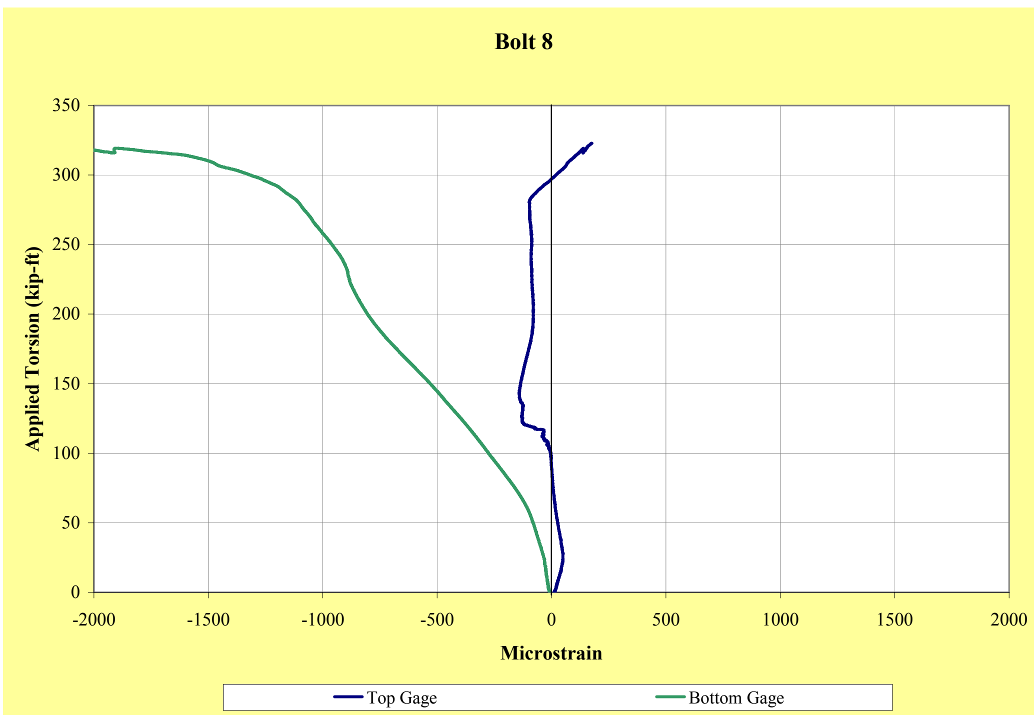


f

Figure D-3. Continued

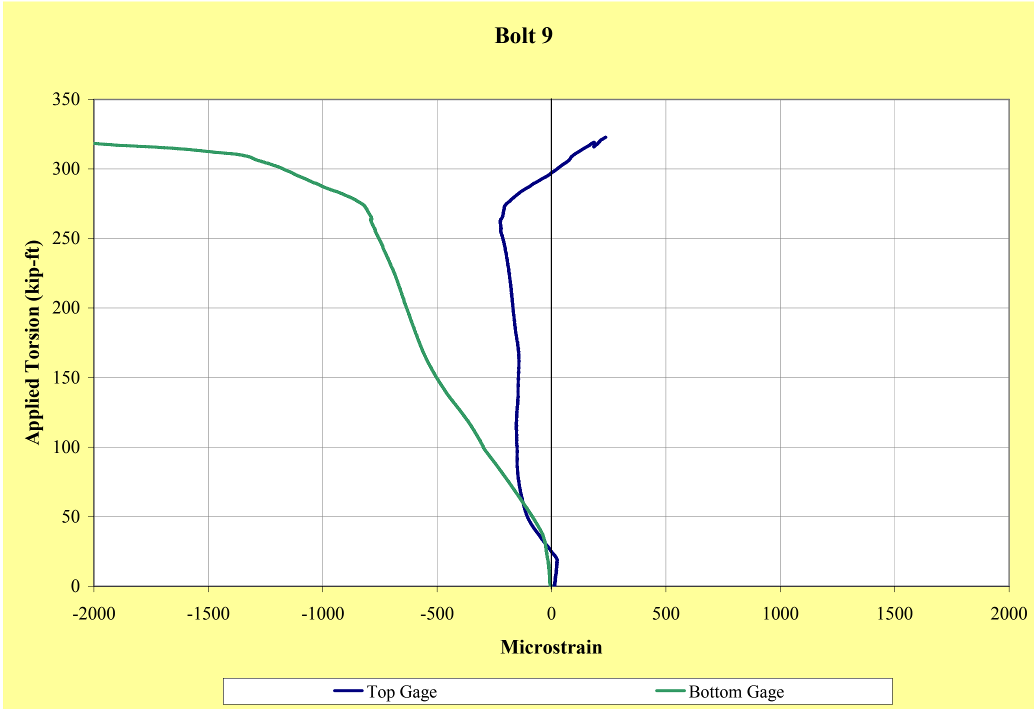


g



h

Figure D-3. Continued



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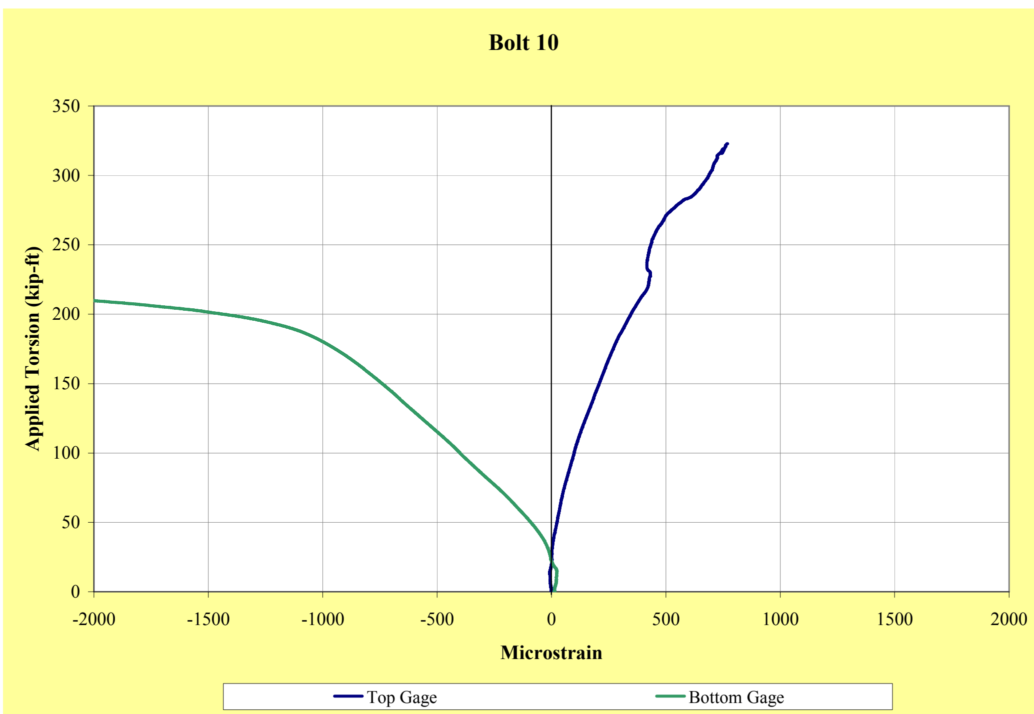
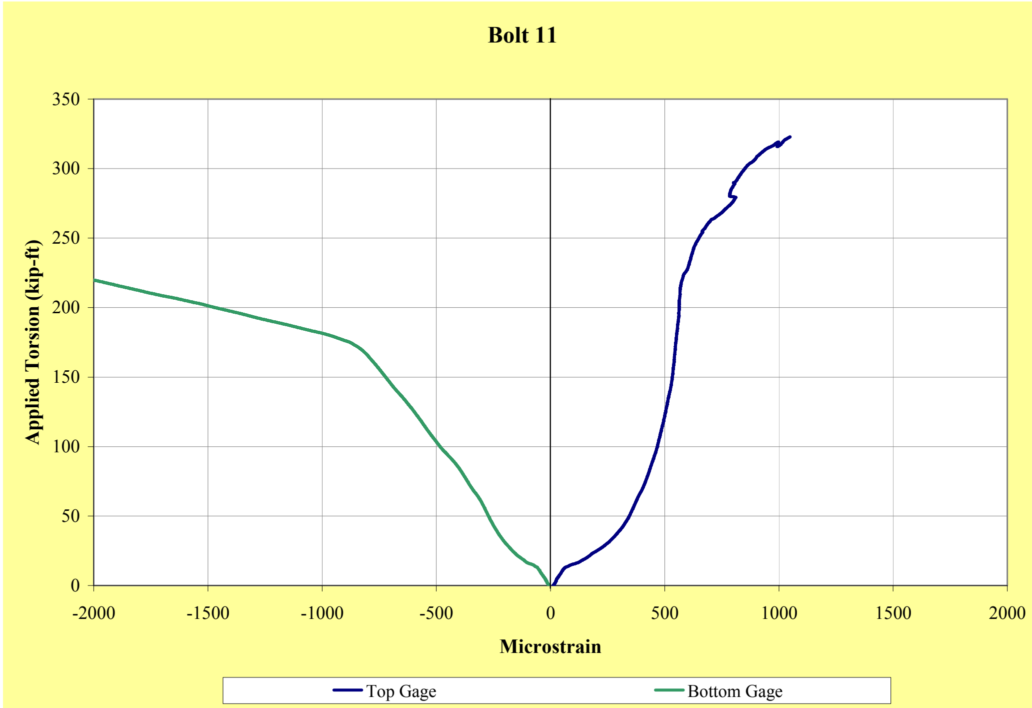


j

Figure D-3. Continued



k

l

Figure D-3. Continued

APPENDIX E
SAMPLE GUIDELINES FOR EVALUATION

Guidelines for the Evaluation of Existing Foundations

The guidelines presented below are intended **only** to check the torsional capacity of the foundation. It assumes that all other components of the sign structure have been designed to meet acceptable standards.

Input

Shaft

$d_s := 30in$ Diameter of the Shaft

$f'_c := 6230psi$ Concrete Strength

Hoop Steel

$Hoop\_Steel\_Area := 0.11in^2$ Hoop Steel Area

$Hoop\_Steel\_Diameter := \frac{3}{8}in$ Hoop Steel Diameter

$s_h := 2.5in$ Spacing of Hoop Steel

$f_{yt} := 60ksi$ Yield Strength of Hoop Steel

$d_h := 27in$ Centerline Hoop Steel Diameter

$Long\_Steel\_Area := 0.20in^2$ Longitudinal Steel Area

$Long\_Steel\_Diameter := 0.5in$ Longitudinal Steel Diameter

Bolts

$d_o := 1.5in$ Diameter of the Bolt

$d_b := 20in$ Center-Center Diameter of Bolts

$f_{y\_bolt} := 105ksi$

$No\_Bolts := 12$ Number of Bolts

CFRP Properties

$t_{CFRP} := 0.015in$ Thickness of Wrap

$f_{n\_CFRP} := 91.1ksi$ Tensile Strength of CFRP

$w_{CFRP} := 12in$ Width of CFRP Sheet

If you have the factored loading, input it below. If you do not have it leave it as 0

$T_u := 0kip \cdot ft$

Step 1: Calculate the design torsional strength

$A_o := \pi \cdot (d_h \div 2)^2$ Area enclosed by hoop steel $A_o = 572.56\ in^2$

$A_t := Hoop\_Steel\_Area$ Area of hoop steel $A_t = 0.11\ in^2$

$\theta := 45$ ACI 318-05- 11.6.3.6 (a) $\theta_{rad} := \theta \cdot \frac{\pi}{180}$ $\theta_{rad} = 0.79$

$$T_{n\_torsion} := \frac{2 \cdot A_o \cdot A_t \cdot f_{yt}}{s_h} \cdot \cot(\theta_{rad})$$ ACI 318-05 (11-21) $T_{n\_torsion} = 251.9\ kip \cdot ft$

$\phi_{torsion} := 0.75$ ACI 318-05 9.3.2.3

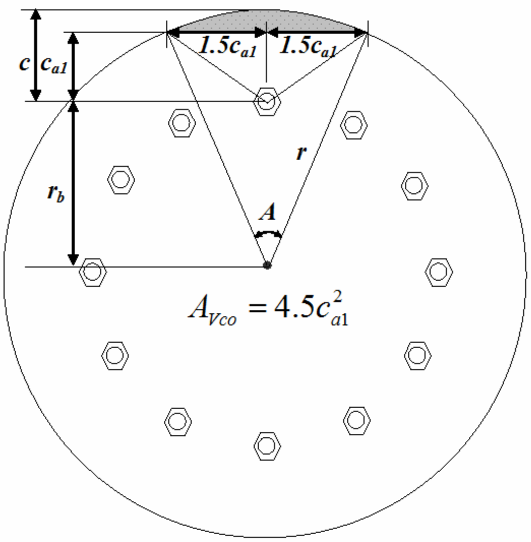
$T_{d\_torsion} := \phi_{torsion} \cdot T_{n\_torsion}$ $T_{d\_torsion} = 188.9\ kip \cdot ft$

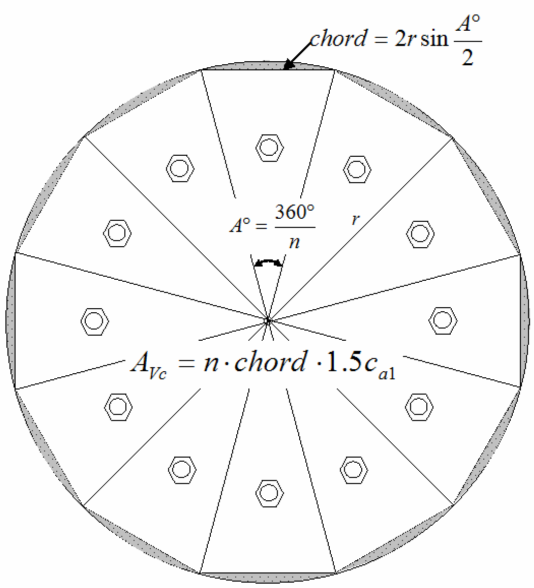
Step 2: Calculate the design concrete breakout strength

$$cover := \frac{d_s - d_b}{2}$$ Bolt Cover $cover = 5\,in$

$$c_{a1} := \frac{\sqrt{\left(\frac{d_b}{2}\right)^2 + 3.25\cdot\left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_b}{2}\right)^2\right]} - \left(\frac{d_b}{2}\right)}{3.25}$$

$c_{a1} = 3.85\,in$ Cover for the calculation o: the anchor strength





$$A := \frac{360}{No\_Bolts}deg$$

$A = 30\,deg$

$$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right)$$

$chord\_group = 7.76\,in$

$$A_{min\_group} := 2\cdot\operatorname{asin}\left(\frac{3.0\cdot c_{a1}}{d_s}\right)$$

$A_{min\_group} = 45.24\,deg$

If A is greater than A_{min_group} then there is no group effect

$A_{Vc} := No\_Bolts\cdot chord\_group\cdot 1.5\cdot c_{a1}$ $A_{Vc} = 537.55\,in^2$ $A_{Vco} := 4.5\cdot c_{a1}^2$ $A_{Vco} = 66.57\,in^2$

$$Check\_Group\_Effect := \begin{cases} \text{"Group Effect"} & \text{if } A \le A_{min\_group} \\ \text{"No Group Effect"} & \text{if } A > A_{min\_group} \end{cases}$$

$Check\_Group\_Effect = \text{"Group Effect"}$

$\psi_{cV} := 1.4$ ACI 318-05- D.6.2.7

1.0 for cracked concrete with no supplementary reinforcement or reinforcement smaller than a No. 4 bar
1.2 for cracked concrete with edge reinforcement of a No.4 bar or greater
1.4 for uncracked concrete or with edge reinforcement of a No.4 bar or greater enclosed within stirrups spaced no more than 4 in. apart

$\psi_{ecV} := 1.0$ ACI 318-05- D.6.2.5 $\psi_{edV} := 1.0$ ACI 318-05- D.6.2.6

All anchors are loaded in shear in the same direction

ACI 318-05-D.6.2

$l_e := 8 \cdot d_o$

$$V_b := 7 \cdot \left(\frac{l_e}{d_o}\right)^{0.2} \cdot \sqrt{\frac{d_o}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf \qquad \text{(D-24)}$$

$V_b = 7.74\,kip$

$$V_{cbg} := \begin{cases} No\_Bolts \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b & \text{if } A > A_{min\_group} \quad \text{(D-21)} \\ \dfrac{A_{Vc}}{A_{Vco}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b & \text{if } A \le A_{min\_group} \quad \text{(D-22)} \end{cases}$$

$V_{cbg} = 87.46\,kip$

$V_{cbg\_parallel} := 2 \cdot V_{cbg}$ ACI 318-05-D.6.2.1

$V_{cbg\_parallel} = 174.93\,kip$

$T_{n\_breakout} := V_{cbg\_parallel} \cdot (d_b \div 2)$

$T_{n\_breakout} = 145.77\,kip \cdot ft$

$\phi_{breakout} := 0.75$ ACI 318-05 9.3.2.3

$T_{d\_breakout} := \phi_{breakout} \cdot T_{n\_breakout}$

$T_{d\_breakout} = 109.33\,kip \cdot ft$

Step 3: Compare the concrete breakout strength to the torsional strength

$T_{d\_breakout} = 109.33\ kip \cdot ft$

$T := if(T_u = 0 kip \cdot ft, T_{d\_torsion}, max(T_u, T_{d\_torsion}))$ $T = 188.94\ kip \cdot ft$

$$Check\_Strength := \begin{array}{|l} \text{"Retrofit Required" if } T_{d\_breakout} < T \\ \text{"Sufficient Strength" if } T_{d\_breakout} \geq T \end{array}$$

Check_Strength = "Retrofit Required"

Step 4: Determine the amount of CFRP Wrap Required

Consider the twelve inch roll is only effective down to 1.5*cover

$w_{eff\_CFRP} := 1.5 \cdot cover$ $w_{eff\_CFRP} = 7.5\ in$

$$F_{CFRP} := if\left(Check\_Group\_Effect = \text{"Group Effect"}, \frac{T}{4 \cdot \pi \cdot \frac{d_b}{2}}, \frac{T}{2 \cdot \frac{d_b}{2} \cdot No\_Bolts}\right)$$

$F_{CFRP} = 18.04\ kip$ Tension calculated using a strut-and-tie model for cases where there is no group effect and edge pressure method when there is a group effect.

$\psi_f := 0.95$ ACI 440.2R Table 10.1

$\phi := 0.75$ ACI 318-05 9.3.2.3

$$Layers\_Wrap\_Required := \frac{F_{CFRP}}{t_{CFRP} \cdot f_{n\_CFRP} \cdot w_{eff\_CFRP} \cdot \phi \cdot \psi_f}$$

$$Amount\_of\_CFRP\_to\_Apply := \begin{array}{|l} Layers\_Wrap\_Required \text{ if } Check\_Strength = \text{"Retrofit Required"} \\ \text{"CFRP not required" if } Check\_Strength = \text{"Sufficient Strength"} \end{array}$$

Amount_of_CFRP_to_Apply = 2.47

Round up to achieve a full layer

$$Layers\_of\_CFRP\_to\_Apply := \begin{array}{|l} ceil(Amount\_of\_CFRP\_to\_Apply) \text{ if } Check\_Strength = \text{"Retrofit Required"} \\ \text{"CFRP not required" if } Check\_Strength = \text{"Sufficient Strength"} \end{array}$$

Layers_of_CFRP_to_Apply = 3

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