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FINAL REPORT

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ALTERNATIVE SUPPORT SYSTEMS FOR CANTILEVER SIGNAL/SIGN STRUCTURES

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DISCLAIMER

Metric Conversion Table

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2

Technical Report Documentation Page

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16. Abstract

During the 2004 hurricane season, several anchor embedment failures of the support structures of cantilever signal/sign structures occurred. A previous research program determined the cause of these failures was by concrete breakout due to shear on the anchors directed parallel to the edge of the foundation. The purpose of the current research program was to take the knowledge obtained on the previous research program and identify a suitable alternative support structure without the use of anchor bolts. After a literature review and experimental testing, it was determined that an embedded pipe with welded plates was a suitable alternative support structure. The torsion could be adequately transferred to the support structure concrete through the vertical torsional plates and the flexure could be adequately transferred to the concrete through the welded annular plate on the bottom of the pipe. Furthermore, it was determined that the alternative selected was not only a viable alternative to the anchor bolt system, but it had greater strength for a given foundation size than the anchor bolt system.

The test specimens were designed to fail by concrete breakout originating from the torsional and flexural plates and to preclude other failure modes. The results of the testing indicated that the concrete breakout was the failure mode for the embedded pipe and plate configuration and that the concrete breakout strength could be accurately predicted using modified equations for concrete breakout from American Concrete Institute (ACI) 318-08 Appendix D. The results of these tests led to the development of guidelines for the design of the embedded pipe and plate configuration. Recommendations for future testing include an alternative base connection that precludes the use of annular plates.

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EXECUTIVE SUMMARY

During the 2004 hurricane season, the failure of several foundations of cantilever sign structures occurred along Florida highways. Those failures necessitated a review of the design and construction procedures for the foundations of cantilever sign structures. The failures were determined to be caused by concrete breakout of the anchors subjected to shear parallel to the edge caused by torsional loading. The research team tested a retrofit option using carbon fiber-reinforced polymer (CFRP) wrap and design guidelines for determining the susceptibility of failure for current systems and design of the CFRP wrap retrofit design were created. Having found the failure mechanism, alternative support structures were recommended for future research, which became the basis for the current project.

The primary objectives of this research program were as follows:

- Identify a viable alternative to transfer load from the superstructure to the foundation other than through anchor bolts.
- Provide design guidelines for the alternative selected.

In order to complete these objectives, a literature review and experimental program were conducted. The findings of the literature review were used to develop the experimental program. The literature review and the results of the experimental program were used to develop the design guidelines for the alternative selected. In addition to the primary objectives, alternative connections were also identified for consideration for future testing.

After a literature review and exploration of other industries' options, an embedded pipe and plate section was selected as a viable alternative. The clear load path and ability to handle both torsional and flexural load made the embedded pipe and plate section the most ideal alternative. Testing proved that the embedded pipe and plate section was able to transfer the torsional and flexural load to the concrete satisfactorily. Testing also proved that American Concrete Institute (ACI) 318 code equations for concrete breakout from applied shear could be modified to accurately predict the concrete breakout strength of the embedded pipe and plate section.

The accurate testing predictions using the modified code equations were the basis for the development of the design guidelines. The design guidelines account for the design of the base connection as well as the foundation, including the pipe and plates section and concrete pedestal and reinforcement.

Implementation of the recommended alternative and design guidelines for foundations of cantilever signal/sign structures should eliminate any concrete breakout problems associated with the anchor bolts. The recommended alternative connections are highly recommended for further investigation. The combination of the embedded pipe and plate section and a selected alternative connection would significantly reduce the number of failures of cantilever signal/sign structures.

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CHAPTER 1
INTRODUCTION

This project is in response to the failures of several cantilever sign structure foundations in Florida during the 2004 hurricane season (See Figure 1-1 and Figure 1-2). The initial research program resulting from these failures was completed in August 2007 and is Florida Department of Transportation (FDOT) Report No. BD545 RPWO #54, *Anchor Embedment Requirements for Signal/Sign Structures* (1). The objective of the initial project was to determine the cause of failure of the foundations and to recommend both design procedures and retrofit options. It was determined that torsional loading on the anchor bolt group in the foundation was the most likely cause of the failures. Design recommendations for torsional loading on the anchor group and recommendations for a retrofit are included in the project report (1). The initial project also provided recommendations for potential alternative foundation systems.

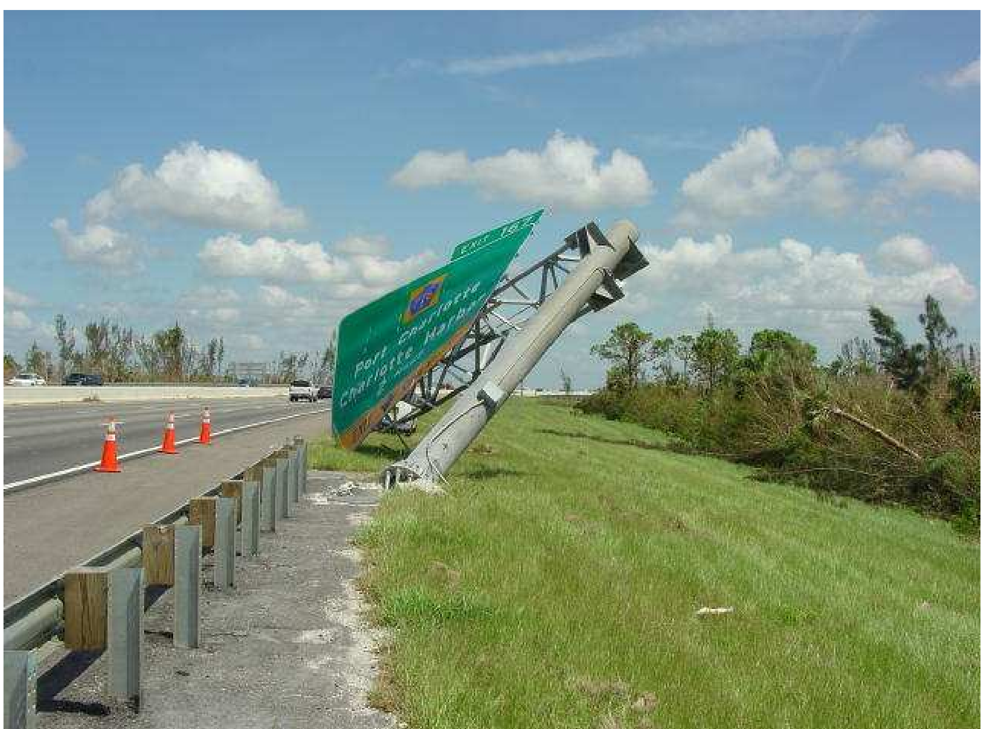


Figure 1-1. Failed cantilever sign structure[1]

Figure 1-2. Failed foundation during post-failure excavation[(1)]

The primary objective of this research project was to identify alternative support structure designs without anchor bolts that will be better equipped to handle transfer of the torsional load to the concrete than the current anchor bolt design and then to conduct an experimental investigation and develop design guidelines for the identified alternative support structure.

In order to complete the objective of this research program, a thorough investigation of alternative support structures used in other structural applications was completed. The findings of this investigation as well as the recommendations of FDOT Report BD545 RPWO #54 were used as the groundwork for the experimental investigation and design guidelines for the identified alternative support structure.

CHAPTER 2
BACKGROUND

The following sections cover the history of signal/sign anchor bolt foundations and present the various foundation systems recommended by FDOT Report BD545 RPWO #54 and alternatives used in other industries. The current anchor bolt foundation system is revisited so that its particular structural concerns can be identified and explored in alternative foundations. The recommended foundations are analyzed for potential problems and benefits, particularly on how they transfer load from the cantilever's monopole to the substructure. Based on the information gathered, a recommended alternative is identified.

2.1 Current Anchor Bolt Foundation System

During a recent survey (2) of state DOTs, an assessment of typical signal/sign foundations was conducted, particularly on the structural application of each foundation type and frequency of use (See Table 2-1). The information obtained from this survey shows that at present, reinforced cast-in-place foundations are the most common foundation types for overhead cantilever signs, with spread footings the next most common foundation.

Table 2-1. Support structure foundation frequency of use[(7)]

Structure type	Reinforced Cast-In-Place Drilled Shafts	Unreinforced Cast-In-Place Drilled Shafts	Steel Screw-In Foundation	Spread Footings	Directly Embedded
Overhead Cantilever	Common	None	Rare	Intermediate	None
Over Head Bridge	Intermediate	None	Rare	Intermediate	None
Road Side Sign	Intermediate	Rare	Rare	Rare	Rare
Street Light Poles	Intermediate	Rare	Rare	Rare	Rare
High-Level Lighting Poles	Common	None	None	Rare	None
Traffic Signal Supports	Common	None	None	Rare	Rare
Span Wire Supports	Intermediate	None	None	Rare	Rare

Notation
Common = 67-100% of the states reporting use
Intermediate = 34-66% of the states reporting use
Rare = 1-33% of the states reporting use
None = 0% of the states reporting use

These most common foundation systems utilize anchor bolts to transfer torsional and flexural moments from the monopole to the support structure. Figure 2-1 depicts how the torsional and flexural moments are transferred in the current anchor bolt design. American Association of State and Highway Transportation Officials(AASHTO) provides guidance in their *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (*Supports Specifications*) for the design of signal/sign supports (3). Many problems have been detected with the signal/sign support structures and the following will cover the history and problems associated with cantilever signal/signs and their support structures.

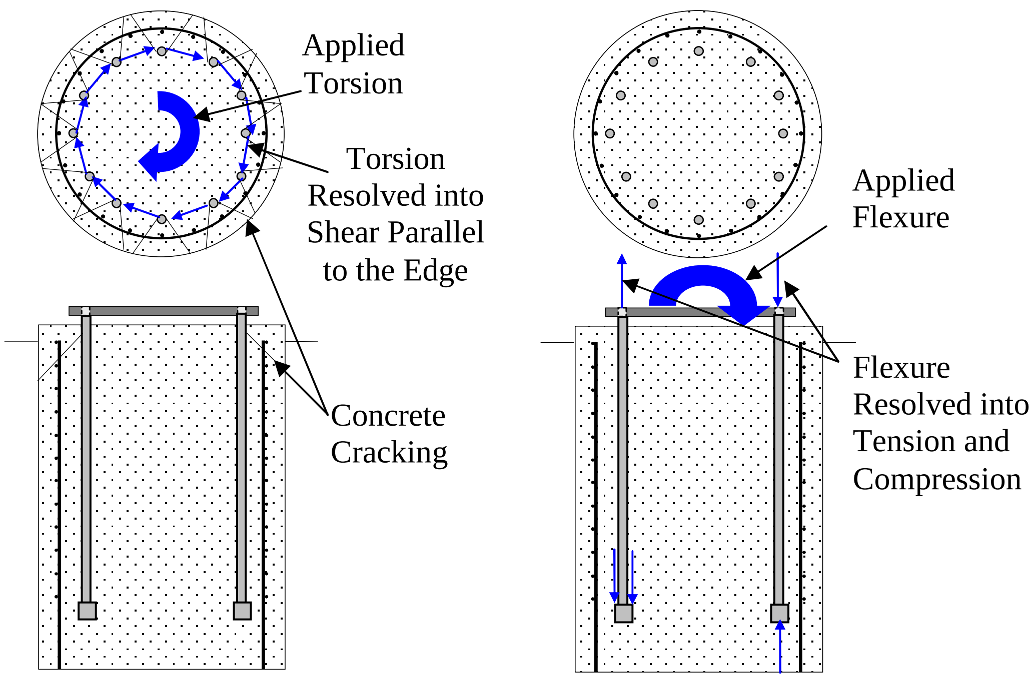


Figure 2-1. How torsional and flexural moments are transferred using anchor bolts(1)

In 1994, the National Cooperative Highway Research Program (NCHRP) initiated Project 17-10 at the University of Alabama at Birmingham (4). The scope of Project 17-10 was to update all aspects, excluding vibration and fatigue, of the 1994 *Supports Specifications* (4). One element of the Supports Specifications that required immediate updating was the information on anchorage systems. The 1994 Supports Specifications' information on anchor bolts was based on information obtained in the late 1960s and late 1970s (4). The updated anchor bolt information contained in Report 411 included an Appendix C which addressed minimum embedment length of headed cast-in-place anchor bolts, effect of edge distance, and the effect of spacing between anchor bolts (4). However, Appendix C of NCHRP Report 411 was not included in the 2001 Supports Specifications (2).

A second phase of Project 17-10 was initiated and published as NCHRP Report 494 in 2003. NCHRP Report 494 addressed additional updates to the *Supports Specifications*. In NCHRP Report 494, further information is provided regarding anchorage to concrete. In addition to restating the information in Appendix C of NCHRP Report 411, NCHRP Report 494 provided a simplified design method for design of anchorage to concrete based on the then recently added Appendix D to American Concrete Institute (ACI) 318-02 (2). The simplified design method for anchorage required the following conditions be met (2):

- Anchor bolts be hooked or headed
- Foundations have vertical reinforcing steel and vertical confinement, with anchor bolts placed inside of the reinforcement
- Foundation reinforcing steel is uncoated
- If hooked anchor bolts are used, the length of the hook is at least 4.5 times the anchor bolt diameter

The simplified design method would design the diameter and bearing area of a headed anchor or the required anchor bolt diameter of a hooked anchor as well as the bolt length so that the failure plane would intersect the foundation's reinforcing steel below the point at which the reinforcing steel is fully developed (2). The transfer of flexural moment is thoroughly addressed in the simplified design method through its treatment of tension. While the simplified method does well to address anchor bearing on concrete, it makes the assumption that if confining reinforcement is provided, failure by concrete breakout and concrete side-face blowout can be prevented (2). It also assumes that the shear force will not control because of the greater flexural moment. These simplified design guidelines have not been included in the *Support Structures*.

However, the information obtained on anchor bolts by the FDOT under contract number BD545 RPWO #54 entitled *Anchor Embedment Requirements for Signal/Sign Structures* indicates that concrete breakout is a problem even if confining reinforcement is provided. The reason for the report was several cantilever support structure failures in Florida during the 2004 hurricane season (See Figure 1-1 and Figure 1-2). The project predicted that the reason for the failure of the cantilever signal/sign foundations was the hurricane wind loads applied excessive torsional force on the foundation. The torsional force could be resolved into shear force acting on the anchors parallel to the edge of the foundation (See Figure 2-2). The shear force acting parallel to the edge was causing an anchor break-out phenomena that is described in Section D.5.2 of ACI 318-08 (5). Testing confirmed the prediction and an evaluation guideline as well as a CFRP wrap retrofit design guideline were detailed in the report.

Clearly, the information gathered on the present system shows a need to rethink the design where anchors are concerned. While the NCHRP Reports are designed to modify the Supports Specifications for the current anchor bolt design, the purpose for this research is to identify an alternative method of transferring torsional and flexural moments from the monopole to the concrete shaft other than through an anchor bolt connection.

The main concern addressed in this research project is the failure of concrete due to shear load on the anchor bolts parallel to the edge resulting from torsion on the anchor group. Therefore, a viable alternative will be one that avoids transferring shear through anchor bolts. Other concerns that have been identified are design practice and construction related. While these concerns are not the main objective of this research project, a new design may address these problems. The concern with fatigue has also been identified and is addressed in other research projects and is not in the scope of this project (6; 7). Recommendations for future testing regarding fatigue concerns will be addressed in Chapter 6.

2.2 Alternative Foundation Systems

The following alternatives are based upon the recommendations of the FDOT Report BD545 RPWO #54 (1). There are three cast-in-place concrete foundation alternatives and a drilled helical pipe alternative recommended from FDOT Report BD545 RPWO #54. Also included in this section is an embedded tapered section that was not included in the previous report but has been used in other DOT applications.

2.2.1 Steel Pipes with Plates Welded at Four Locations

This foundation system would use an embedded pipe with stiffener plates. Figure 2-2 shows the configuration of this system (1). The stiffener plates will be attached symmetrically around the shaft of the steel pipe. The purpose of the stiffener plates would be to provide for the transfer of torsional loading between the steel pipe and the concrete by bearing on the concrete during twisting.

The installation of this foundation would be relatively simple. After excavation for the concrete foundation, a reinforcement cage would be lowered into the excavation and aligned properly. The steel pipe and plate assembly would be lowered into the excavation and aligned. The concrete would then be poured into the excavation. Then the superstructure would be erected on top of the foundation (8). The superstructure could be aligned and leveled using a leveling nut detail shown in Figure 2-3. This connection would also eliminate problems with grout installation because none would be required.

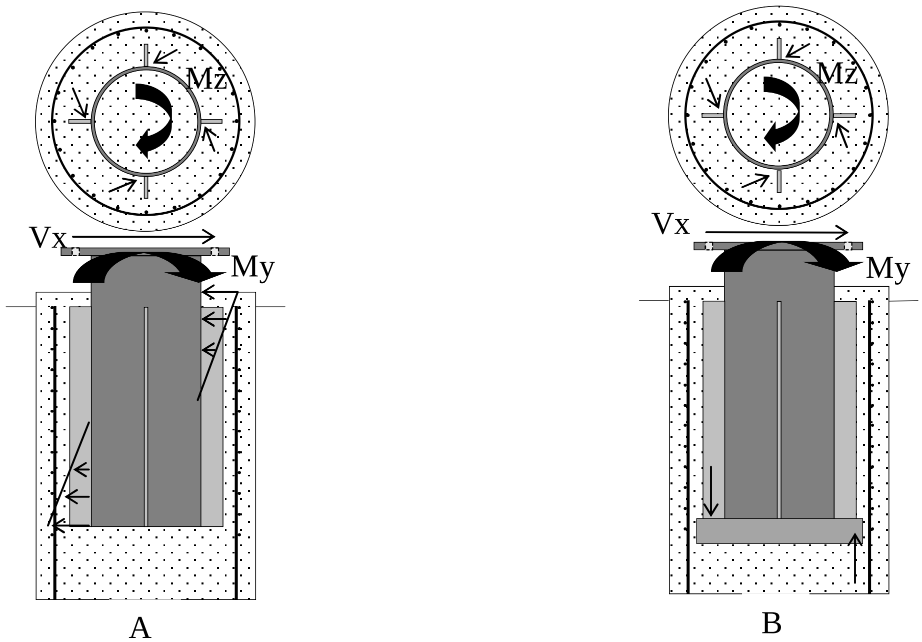


Figure 2-2. Alternative foundation: steel pipe with four welded plates

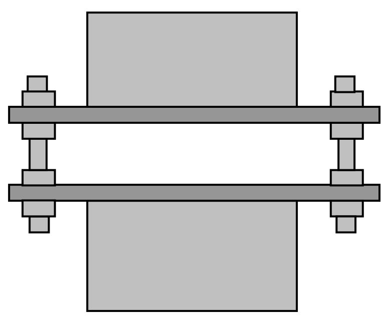


Figure 2-3. Leveling nut detail

As mentioned earlier, the vertical torsional plates would act similar to an anchor group for transferring load to the foundation. Figure 2-2 shows the possible force configuration that would be acting on the foundation and how the foundation would resist the forces. Option B has an annular plate welded to the bottom of the embedded pipe and plate section while option A does not. The purpose of the annular plate is to provide a stiff member to resist the bending moment induced on the foundation. If the plate were not a part of the configuration, then the pipe would likely resist the bending by bearing on the concrete, creating a potential problem with buckling of the pipe. As the biaxial moment acts on the foundation with the annular plate, it will induce a tensile reaction on one part of the concrete foundation and a compressive reaction on the opposite side, see Figure 2-2 The shear load will induce a distributed load on the sides of the foundation. The axial load will be distributed throughout the foundation by the annular plate. The torsional load will cause the stiffener plates to transfer the load as a shear force directed parallel to the edge of the concrete similar to an anchor loaded in shear parallel to the edge and bear on the concrete.

2.2.2 Geometric Hollow Section

This foundation would use an embedded geometric hollow section rather than a steel pipe. Figure 2-4 shows the configuration of this system (1). The purpose of the geometric shape would be to create additional torsional resistance through the geometry of the shape. The installation of this foundation would be very similar to the method mentioned for the embedded pipe and plate section.

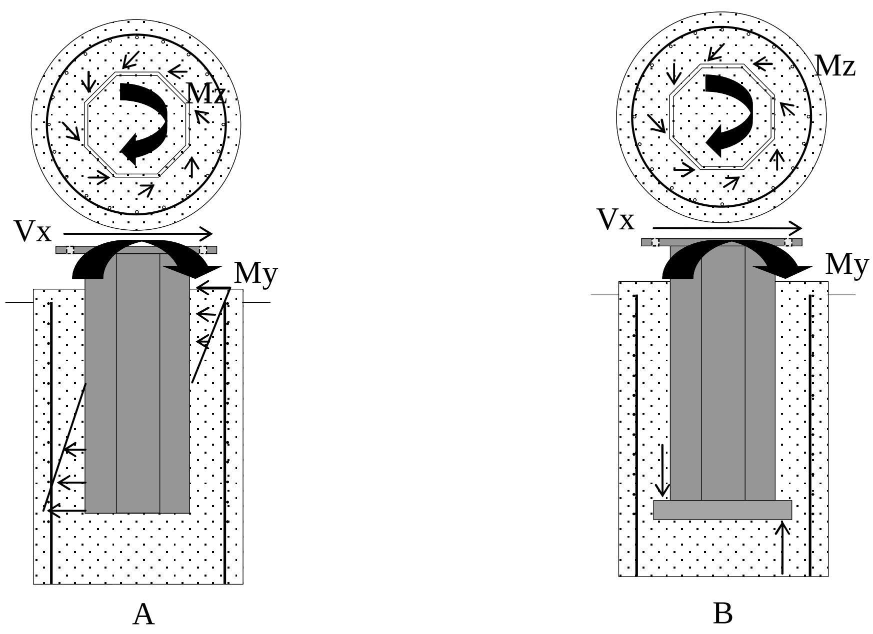


Figure 2-4. Alternate foundation: geometric hollow section

The geometric shape of the pipe would act as the way to transfer the load from the steel monopole to the concrete. The concrete would be able to resist the torsional rotation of the pipe embedded in the foundation through the geometric advantages of the section. The shear force would cause the concrete to resist as a distributed load. The moment would induce axial resistance. Figure 2-4(b) shows the force configuration acting on the foundation and how it would resist the force by bearing on the concrete.

2.2.3 Pipe with Welded Studs

In this option, the steel pipe would be welded with symmetrically oriented rows of steel studs through the depth of the foundation. The purpose of the studs would be to provide resistance to both flexural and torsional loading. The installation of this foundation would be the same as both the embedded pipe and plates and the embedded geometric hollow section foundations.

The welded studs would transfer the shear, flexure, and torsion from the steel superstructure to the concrete. All of the torsional and bending forces can be resolved into shears on the studs at their various angles of loading. The studs would resist the shear by bearing on the concrete. Figure 2-5 shows the force configuration acting on the foundation as well as the resistive bearing forces from the concrete.

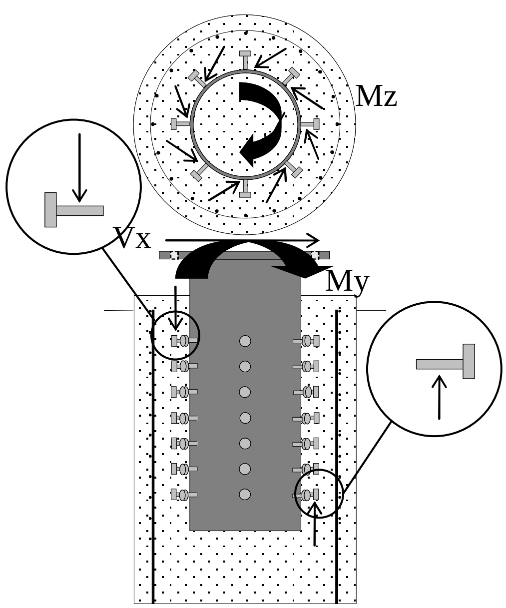


Figure 2-5. Alternate foundation: pipe with welded studs

2.2.4 Helical Pipes

This option would call for the helical pipes to be screwed directly into the soil. This alternative provides the benefit of removing concrete as a consideration in the design. See Figure 2-6 for the configuration of this foundation. The geometry of the pipe and the strength of the soil itself would provide the torsional resistance required in the design. The pipes would need to be first protected against corrosion and then screwed into the soil.

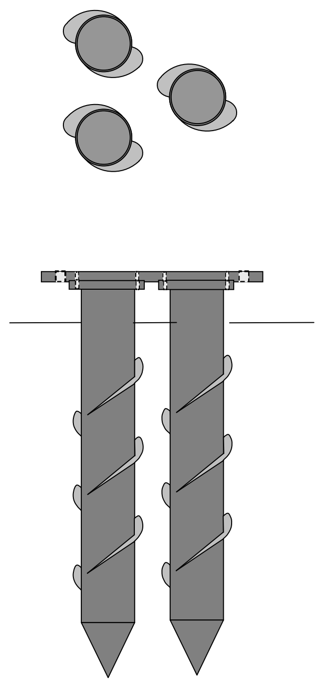


Figure 2-6. Alternate foundation: helical pipes

One possible drawback to this alternative would be that the helical piles would require frequent field inspections to ensure that the soil is not failing. The helical piles would not be an ideal option for Florida because of the prevalent poor soil conditions. Also, the helical piles would be highly susceptible to corrosion because of the direct contact with the soil and possible direct contact with the water table. In this foundation system, the load would not be transferred from the steel to the concrete, but rather directly from the steel to the soil. Therefore a thorough geotechnical assessment would be required before design could begin. Because of this, it would be very difficult to present standard design guidelines for this option.

2.2.5 Embedded Geometric Tapered Section

In this option, a geometric tapered section would be embedded into the drilled shaft (See Figure 2-7). The purpose of the geometric shape would be to create additional torsional resistance through the geometric qualities of the shape. This foundation would require similar construction methods as the other cast-in-place options.

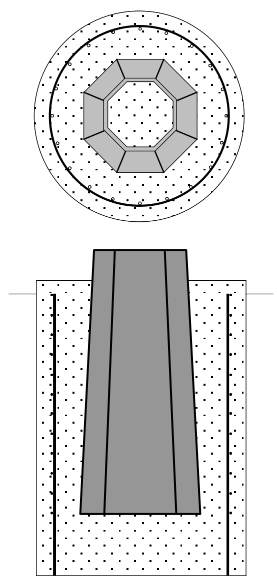


Figure 2-7. Alternate foundation: geometric tapered section

The geometrically varied shape of the tapered section would act as the way to transfer the load from the steel superstructure to the concrete. The concrete would be able to resist the twisting motion of the pipe embedded in the foundation through the geometry of the section. The shear force would cause the concrete to resist by bearing on the pipe in a distributed load. The moment would induce axial resistance. One problem associated with this configuration is the availability of large tapered sections to be embedded in the foundation. The large tapered sections can be costly and difficult to find, limiting the practicality of this option.

2.3 Alternative Foundations from Other Industries

An investigation into transmission line foundations, cellular tower foundations, wind turbine foundations, and large advertising sign foundations was completed. While investigating these fields it became apparent that despite the similarities in foundation requirements, the large torsion experienced by cantilever sign/signal foundations is not typically present in other industries and is not designed for. Because of this, the other industries' alternatives would most likely not be viable for the cantilever sign and signal applications. The following section will describe what was found in these other industries.

2.3.1 Transmission Line Foundations

An investigation into transmission line foundations showed that they often use cast-in-place concrete designs that are similar to the current anchor bolt design, using anchor bolts to connect the superstructure to the foundation; see Figure 2-8c (9). The other cast-in-place designs, Figure 2-8a, Figure 2-8b, are disparate from the current anchor bolt design. However, these are not viable alternative options because they are typically exposed to primarily axial and shear loads. The sizes of the members make direct embedment a more suitable option for their foundations than a cantilever sign/signal foundation. See Figure 2-9 for the loading that transmission line foundations are subject to (9). This loading pattern is similar, but not the same as the loading that cantilever sign/signal foundations are subject to. The torsional load that a cantilever superstructure induces on a foundation creates additional concerns for transferring load to the foundation that these foundations cannot address.

Other alternatives investigated in the transmission line industry seem unsuitable for sign/signal foundations because of construction sequencing, cost, and most importantly because they are unlikely to successfully transfer the torsional loading a sign/signal superstructure is likely to induce. The following are examples of unsuitable alternatives found in the transmission line industry:

- Drilled concrete piles, see Figure 2-10 (9)
- Prestressed anchors
- Grouted soil anchors

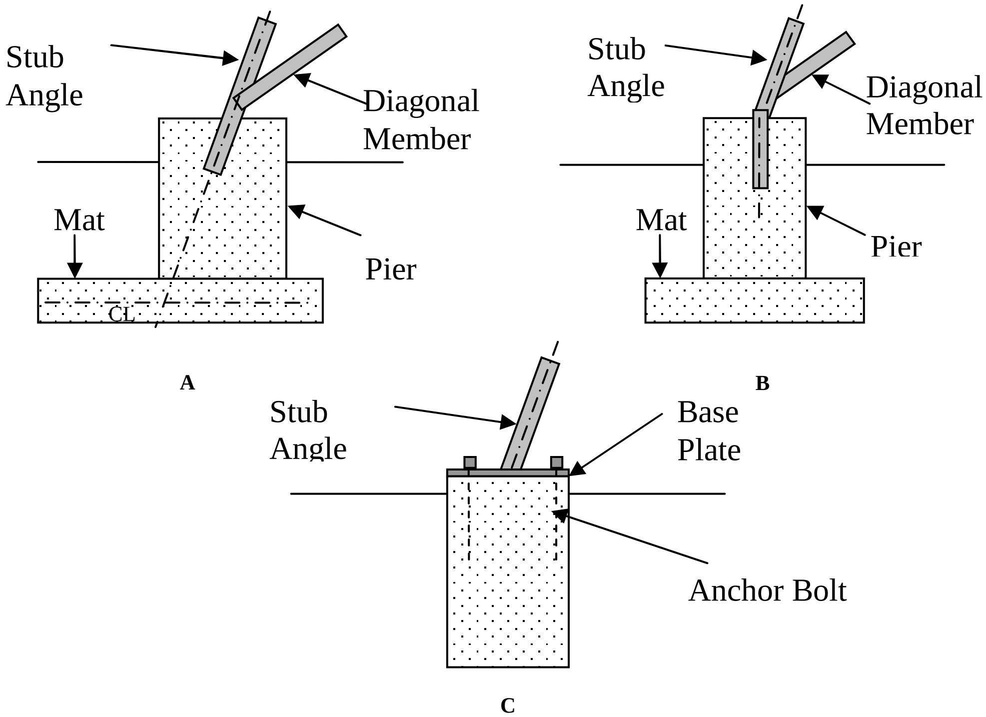


Figure 2-8. Cast-in-place foundation for transmission lines

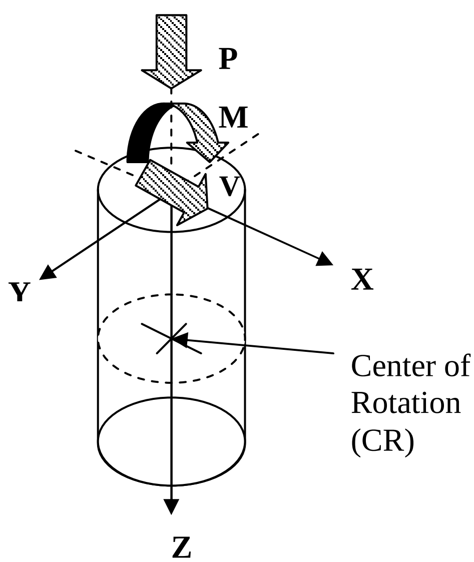


Figure 2-9. Potential forces acting on a transmission line foundation

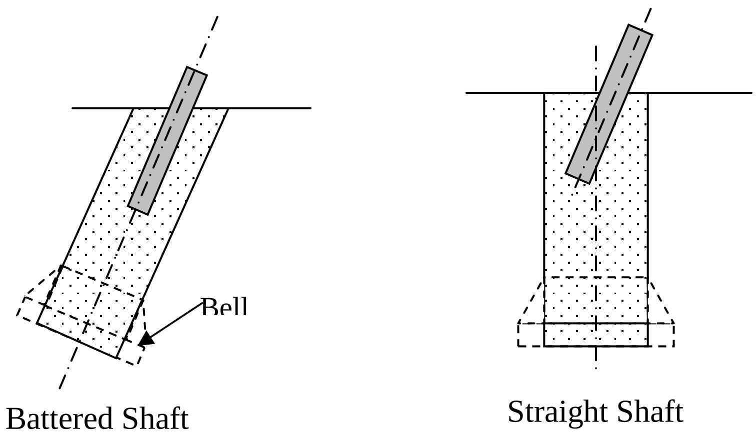


Figure 2-10. Drilled concrete piles for transmission lines

Drilled concrete piles are similar to the current anchor bolt design with the difference being that the guys are embedded in the cast-in-place foundation instead of anchor bolts (See Figure 2-10). These foundations handle axial, shear, and biaxial moments by transferring the loading from the embedded guys to the concrete (9). However, because a transmission line tower is supported by multiple legs, minimal torsional forces are present in each drilled concrete pile. Even the H-structures and single pole structures do not introduce much torsional force into the foundation because there is not a sufficient moment arm to produce significant torsional force. Figure 2-11 demonstrates the typical structural configurations of a lattice tower, H-structure, and single pole structure as well as a cantilever sign/signal structure (9).

Prestressed and grouted soil anchors are typically not suitable to handle torsional load. As described in the Institute for Electrical and Electronics Engineers (IEEE) Guide for Transmission Structure Foundation and Testing, anchors are primarily used to provide resistance to tensile forces (9). Prestressed anchors are typically expensive and should not be used in soils with time dependent compressibility (9). These factors make them typically unsuitable to use for cantilever sign/signal structures. See Figure 2-12 and Figure 2-13 for prestressed and grouted soil anchor configurations, respectively.

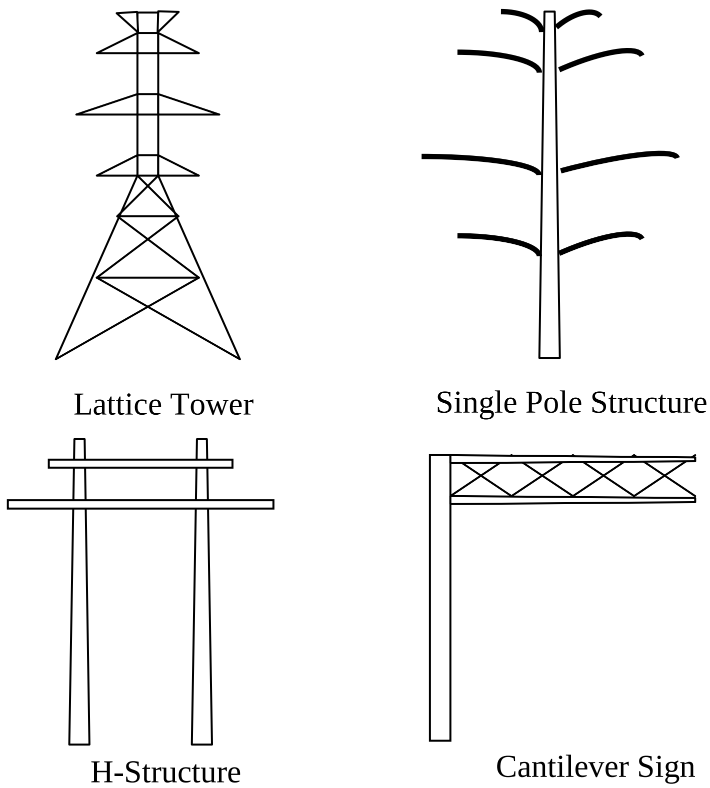


Figure 2-11. Typical transmission line structures compared to a cantilever sign structure

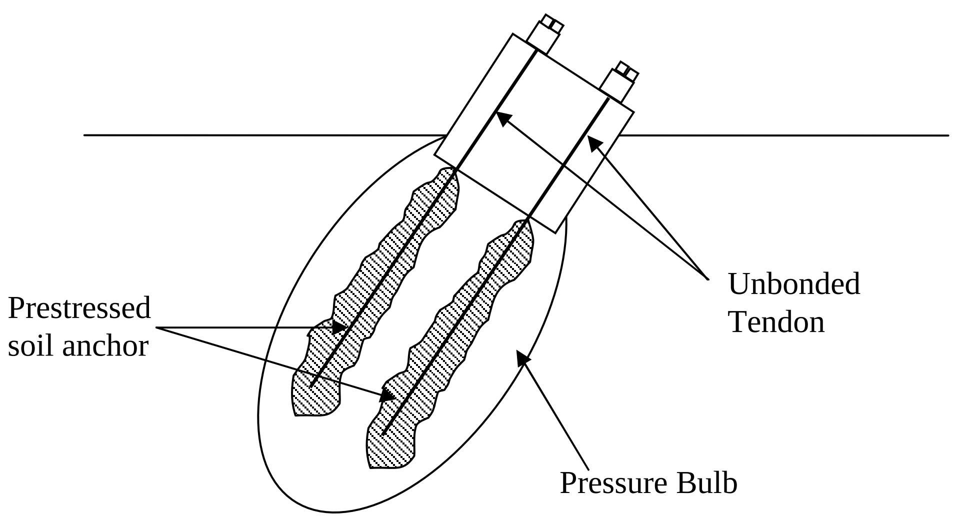


Figure 2-12. Prestressed soil anchor

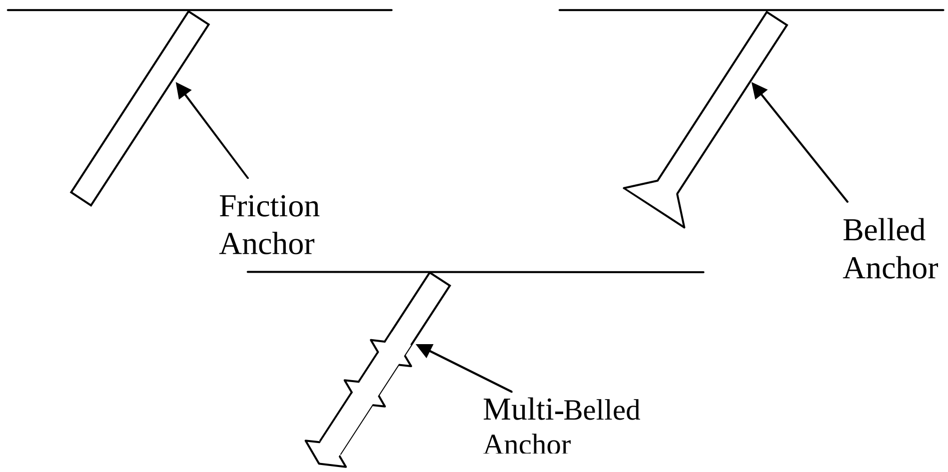


Figure 2-13. Grouted soil anchors

Grouted soil anchors are designed to transfer uplift or tensile loads from the superstructure directly to the soil (9). They do this through frictional resistance between the grout and soil, as well as through the end bearing strength from the increased diameter at the end of the anchor (9). However, the anchors do not provide much torsional resistance because of their smooth geometry.

Despite the fact that these are viable alternatives in the field of transmission line foundations, these are generally not preferable options for sign and signal foundation systems. The fact that sign and signal installations are sequenced at the end of highway construction make piles and anchors undesirable options. By the time the contractor is installing signs and signals, most of the large pile-driving equipment has been moved off the construction site and would create additional expense for the contractor. Time and expense are also reasons why these options are not preferred. Prestressed anchors and grouted soil anchors require geotechnical expertise as well as significant geotechnical analysis of the area and would need to be designed for individual projects which can be more costly. It would be difficult to produce a standard for these options.

2.3.2 Wind Turbine Foundations

The search into wind turbine foundations was initially promising, being that they are required to handle significant amounts of lateral force from the wind (10). However, the torsion experienced by a wind turbine is not significant because there is a limited moment arm. Of greater concern for a wind turbine is biaxial moments. Thus, the three primary designs for a monopole wind turbine that were specified included a mat foundation, a pad and pier foundation, and a pier foundation, all of which utilize anchor bolts to connect the superstructure to the foundation (11). There were guyed tower options as well, but these were not explored thoroughly because of their irrelevance to this project’s application and their similarity to the transmission line industry’s guyed tower foundations.

The mat foundation, found in Figure 2-14, has several elements that make it unsuitable. The primary fault with this option is that it uses anchor bolts, which is the purpose of this research project to eliminate. A mat foundation is also not ideal for the significant loads that a cantilever sign/signal structure will induce on a foundation. The uplift that is created by the cantilever structure will necessitate a deeper foundation.

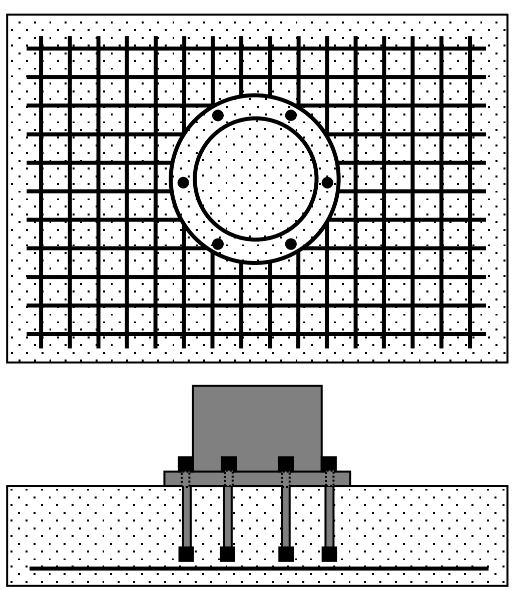


Figure 2-14. Mat foundation for wind turbines

The pad and pier foundation, found in Figure 2-15, and the pier alone foundations are similar to the current anchor bolt design. They are cast-in-place concrete foundations with a monopole attached to the foundation by anchor bolts. The pad and pier foundation is the same as the current anchor bolt design. These options do not hold any potential for a new design because they are the same as the current anchor bolt design. The loading configuration on a wind turbine is similar to that of the transmission line structures. While the wind turbine and transmission line structures will exceed the height of the cantilever sign/signal structure, they do not have sufficient moment arms to create a torsion that is equivalent to the torsion experienced in a cantilever sign/signal structure.

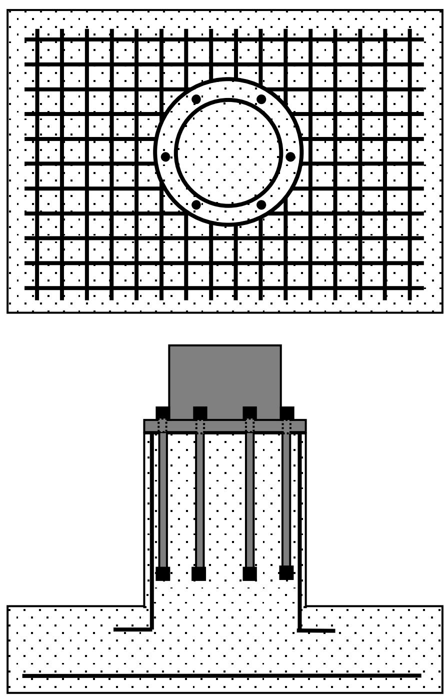


Figure 2-15. Pad and pier foundations for wind turbines

2.3.3 Cellular Tower Foundations

The cellular tower industry was consulted regarding alternative foundations, particularly on which of the recommended designs from FDOT Report BD545 RPWO #54 seemed the most promising. Contact was made with Dave Hawkins, P.E. of Paul J. Ford & Co. from the

Columbus, OH office. Hawkins is a member of the TIA TR14.7 committee which produces the TIA-222 Standard. The TIA-222 Standard governs the design criteria for telecommunications tower structures. Paul J. Ford & Co. is a structural consulting firm that works in the design of communications towers and monopoles as well as transmission towers. Their specialization in this field made them an appropriate choice with which to discuss relevant alternatives.

In a discussion with Hawkins, he stated that from his perspective, the steel pipe with welded plates or the geometric hollow section would be most preferred in his industry. The advantages he pointed out for the steel pipe with welded plates are as follows:

- The stiffeners would act similarly to an anchor group
- Relatively easily cast-in-place
- No direct contact between the steel and soil, reducing corrosion issues

Some possible problems with this configuration are mostly construction related. If the substructure is not placed properly, then the superstructure would not align levelly. This is a concern with the current anchor bolt design, and will be a concern in most cast-in-place designs. The current anchor bolt method uses leveling nuts, as seen in Figure 2-3, to properly align the monopole with the foundation.

The geometric hollow section is also a preferred option for the cellular tower monopole industry because they currently use 12-sided, 16-sided, and 18-sided poles. Hawkins explained that any relevant research pertaining to these designs has not been conducted yet and would be very useful to the telecommunications industry.

2.3.4 Advertising Monopole Foundations

For standards pertaining to monopole foundations in the advertising industry, the International Sign Association (ISA) was contacted. Contact was made with Bill Dundas, who is the ISA’s Director of Technical Affairs. Given FDOT Report BD545 RPWO #54, Dundas forwarded this information to the ISA’s Mechanical and Structural Subcommittee to make comments and recommendations on preferences from the options selected in FDOT Report BD545 RPWO #54 as well as suggest any additional designs. Based on the information gathered from ISA's Mechanical and Structural Subcommittee, the pipe with welded studs seemed to be a preferred option. The subcommittee commented that this detail had been used in larger pipes from 48 inches to 96 inches in diameter.

2.4 Selection

The purpose of the literature review and investigation into alternative support structures was to identify viable foundation alternatives on which to conduct an experimental program. The primary consideration taken into account for the selection of the alternative support system was its ability to properly handle the loading configuration present in a cantilever sign/signal support system. Constructability, time, and expense were taken into consideration, though not fully explored. The foundation systems of other industries were investigated and considered.

Some of the least viable options were the drilled helical pipes, the soil anchors, and the piles. These options would not only be expensive, they would likely not sufficiently handle the loading conditions encountered by the foundation of a cantilever sign/signal configuration. The cast-in-place options seemed most viable as they are the currently used design and seem to be preferred by industry professionals. They can sufficiently handle the loading conditions, are less expensive than other options, and can be easily constructed.

While the investigation into other industries provided insight into how different industries are addressing issues with shear, biaxial bending, and axial load, they do not necessarily provide solutions to implementing a design to transfer torsional load from the steel to the concrete. The recommended cast-in-place designs from FDOT Report BD545 RPWO #54 are the designs with the most potential for applications of sign/signal foundations (1). Therefore, the recommendation for potential design was the pipe with welded plates. The clear load path associated with this option makes it ideal to design for. Industry professionals found this option to be effective at transferring load and easy to design. This design holds potential for a wider range of connections from the foundation to the monopole superstructure. Also, this option seemed to be potentially cost-efficient and effective at transferring the load appropriately to the foundation.

CHAPTER 3
DESIGN IMPLICATIONS

Based on the literature review and investigation into other industries, the embedded pipe with welded plates (See Figure 2-2) was chosen as a suitable alternative to the anchor bolt design (See Figure 2-1). Design provisions for determining the strength of this option and how the forces are transferred from the steel to the concrete are not available. Therefore, some approximations must be made on how this new configuration will transfer the load. The forces that were primarily transferred through the anchor bolts were the torsional moment and flexural moment. Each of these forces will need to be designed for and a failure mode predicted in order for the design to be feasible.

3.1 Design for Torsion

The first parameter to consider is the torsional moment. One estimate is that the welded plates will act similarly to an anchor group when transferring force to the concrete. Assuming this is a valid hypothesis, it would be equally valid to assume that the failure of this foundation would be similar to that of an anchor group failure. Therefore, the concepts that will be explored in this section include viewing the foundation failure as a concrete breakout or concrete side-face blowout (See Figure 3-2).

3.1.1 Equivalent Concrete Breakout Strength in Shear

One method used to estimate the torsional strength of this section was to assume the failure would be similar to a modified concrete breakout failure from shear applied parallel to the edge. In FDOT Report BD545 RPWO #54, it was determined that the previous failures experienced by the foundations were concrete breakout failures from torsional loads applying shear parallel to the edge on the anchor bolt group (See Figure 3-1) (1). It was because of this failure that the alternative support structures research project was initiated. Therefore, during the experiment it would be useful to determine the equivalent torsional strength from concrete breakout and design the rest of the test to preclude other failure modes. In order to calculate an estimated strength of the concrete breakout, the anchor breakout equations need to be modified to account for the differences between an anchor breakout and the pipe and plate breakout.

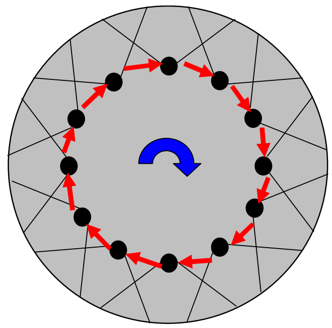


Figure 3-1. Concrete breakout of an anchor caused by shear directed parallel to the edge for a cylindrical foundation

An anchor breakout failure occurs at the surface of the concrete in which it is installed, typically with a ≈35° breakout failure cone. The embedded pipe and stiffener configuration would cause the stiffeners to cause a similar ≈35° breakout failure cone, though not at the top of the shaft. The breakout would occur where the plates are embedded in the concrete. As a result of this expected concrete breakout, the breakout surface would be considerably larger than that of a typical concrete breakout for an anchor loaded in shear because it will create a breakout cone in both the top and bottom of the welded plate. Figure 3-2 depicts the differences between the typical anchor concrete breakout and the expected breakout caused by the welded plates.

In order to quantify the difference in these breakout configurations, some manipulation of the governing equations for concrete breakout of an anchor loaded in shear from ACI 318-08 Appendix D (5) will be required. First, the breakout strength of an anchor loaded in shear needs to be described. The basic breakout strength of a single anchor in cracked concrete loaded in shear perpendicular to an edge (See Figure 3-3) is described in ACI 318-08 Equation D-24 and is shown below as Equation 3-1 (5).

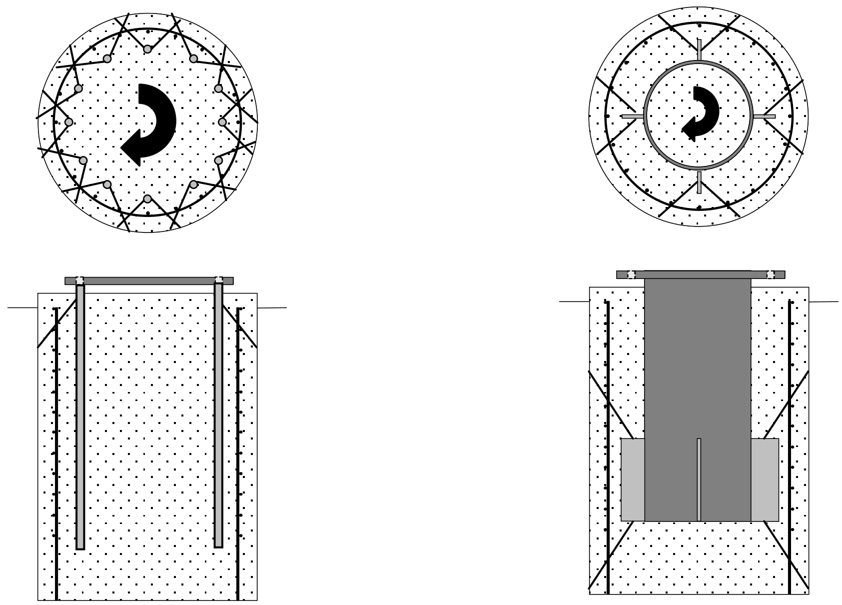


Figure 3-2. Differences between concrete breakout failures for anchor bolts in shear and embedded pipe and plate section in torsion

$$V_b = \left(7\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda\sqrt{f'_c}\,(c_{a1})^{1.5} \qquad \text{(3-1)}$$

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.)
$= h_{ef} < 8\, d_a$

d_a = outside diameter of anchor (in.)

λ = 1.0 for normal weight concrete

f'_c = specified compressive strength of concrete (psi)

c_{a1} = distance from the center of an anchor shaft to the edge of concrete taken in the direction of the applied shear (in.)

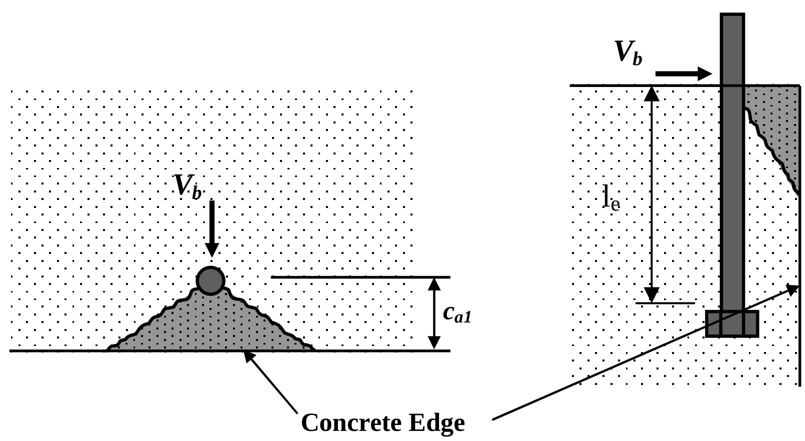


Figure 3-3. Concrete breakout formula for an anchor loaded in shear[1]

The maximum length for ℓ_e is limited to $8d_a$ as delineated in ACI 318-08 D6.2.2. The constant 7 from Equation 3-1 was determined from a 5% fractile with cracked concrete. The constant 7 becomes a constant 13 for the mean breakout strength of a single anchor in uncracked concrete loaded in shear perpendicular to the edge. The mean breakout strength is described in Equation 3-2, as shown below (12).

$$V_b = \left(13 \left(\frac{\ell_e}{d_a} \right)^{0.2} \sqrt{d_a} \right) \lambda \sqrt{f_c'} (c_{a1})^{1.5} \qquad (3\text{-}2)$$

ACI 318-08 (5) describes the nominal breakout strength of an anchor loaded in shear perpendicular to the edge in Equation D-21 and is described below as Equation 3-3. Figure 3-4 depicts the projected concrete failure area of a single anchor in rectangular concrete. Figure 3-5 depicts the projected concrete failure area of a single anchor in cylindrical concrete. An important distinction to note between the failure area of a single anchor in rectangular concrete and cylindrical concrete is the edge distance c_{a1}. Equation 3-4 details how to calculate the value of c_{a1} for an anchor adjacent to a circular edge.

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad (3\text{-}3)$$

Where

V_{cb} = The nominal concrete breakout strength in shear of a single anchor (lb.)

A_{Vc} = The projected area of the failure surface for a single or group of anchors, used to determine the shear strength (in^2)

A_{Vco} = The 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 2-16)

$\psi_{ed,V}$ = The factor used to modify shear strength of anchors for edge effects, ACI 318-08 Section D.6.2.6

$\psi_{c,V}$ = The 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 318-08 Section D.6.2.7, accounted for in Equation 2-2

$\psi_{h,V}$ = The factor used to modify shear strength of anchors based on anchor location and effective length of anchor, ACI 318-08 Section D.6.2.8

$$c_{a1} = \frac{\sqrt{(r_b)^2 + 3.25\left[(r_s)^2 - (r_b)^2\right]} - r_b}{3.25} \tag{3-4}$$

Where

c_{a1} = distance from the center of an anchor shaft to the edge of concrete taken in the direction of the applied shear (in.)

r_b = The distance from the center of the cylindrical shaft to the center of the anchor bolt (in.)

r_s = The radius of the cylindrical shaft (in.)

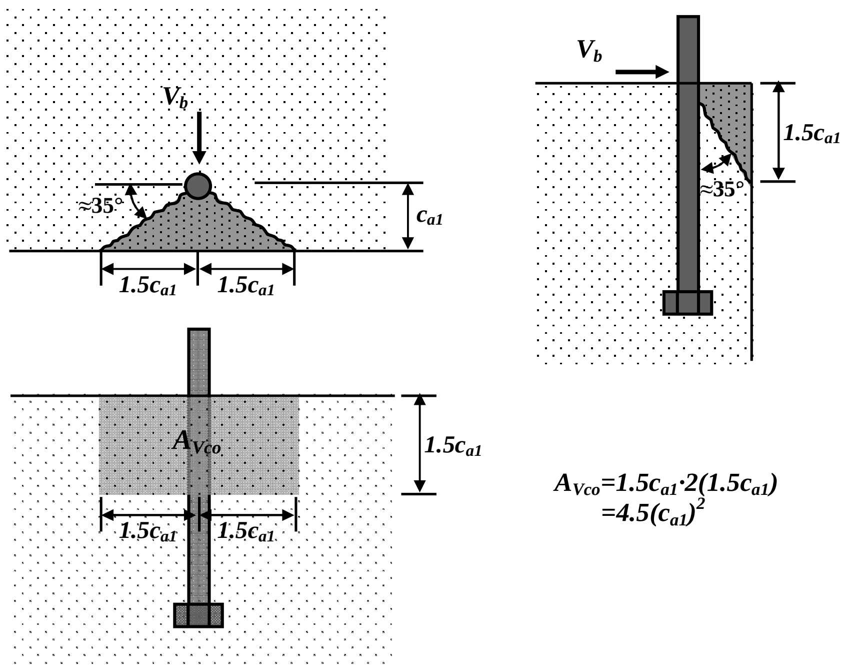


Figure 3-4. Shear breakout of a single anchor in rectangular concrete

As FDOT Report BD545 RPWO #54 determined, the failure loading on the foundation's anchor group was torsion (1). This torsion can be resolved into shear forces acting parallel to an edge. ACI 318-08 prescribes in section D6.2.1 that the nominal concrete breakout strength of a single anchor loaded in shear parallel to an edge shall be permitted to be twice the value of the shear force determined as V_{cb}, which assumes shear loading perpendicular to an edge.

Now that the basic equations for concrete breakout due to shear on anchor bolts have been established, it is appropriate to address the changes in these equations to satisfy the differences between the anchor breakout and the expected experimental breakout. The mean breakout

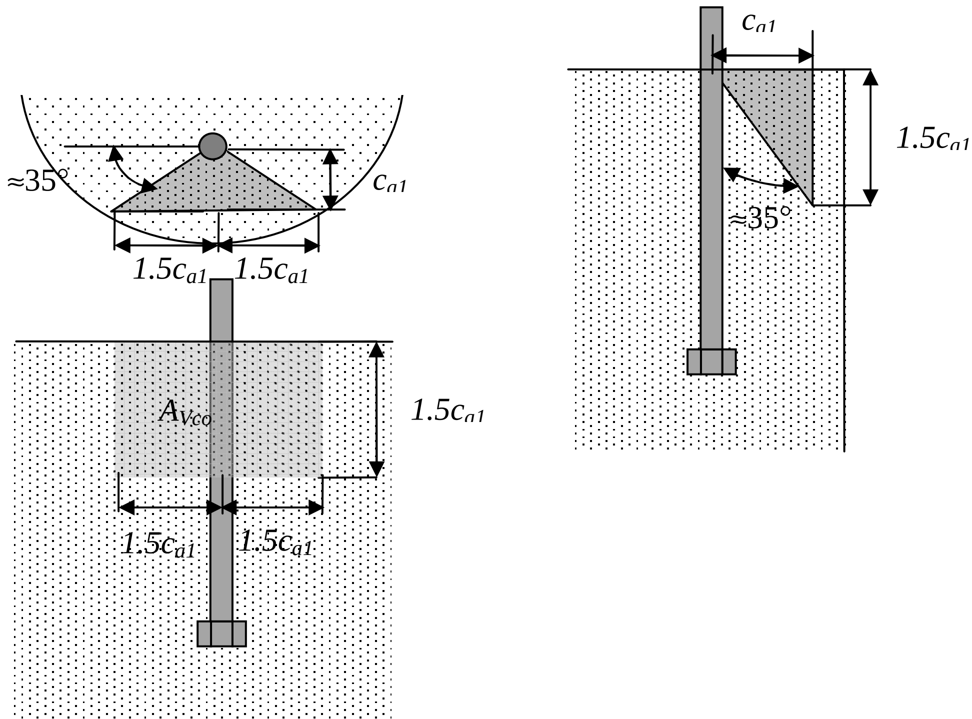


Figure 3-5. Shear breakout for a single anchor in cylindrical concrete

strength of a single plate in shear acting perpendicular to the edge has been modified from Equation 3-2 to Equation 3-5 listed below by substituting the geometric qualities from the anchor bolt system to the appropriate geometric qualities of the embedded pipe and plate system.

$$V_b = \left(13 \left(\frac{\ell_e}{t_p} \right)^{0.2} \sqrt{t_p} \right) \lambda \sqrt{f_c'} \left(c_{a1} \right)^{1.5} \qquad (3\text{-}5)$$

Where

V_b = basic concrete breakout strength in shear of a single plate in uncracked concrete (lb.)

ℓ_e = load bearing length of plate for shear (in.)

t_p = thickness of plate (in.)

c_{a1} = distance from the center of the plate to the edge of concrete taken in the direction of the applied shear (in.)

The arrangement of the plates in this specific design does not allow them to be analyzed as a group because their ≈35° breakout failure cones do not overlap. Therefore, Equation 3-3 was utilized to determine the strength of a single plate. However, the A_{vcp}, or the projected area of the breakout surface for a single plate, was modified from A_{vc} to account for the differences in the breakout surface. Figure 3-6 depicts the area A_{Vcp}.

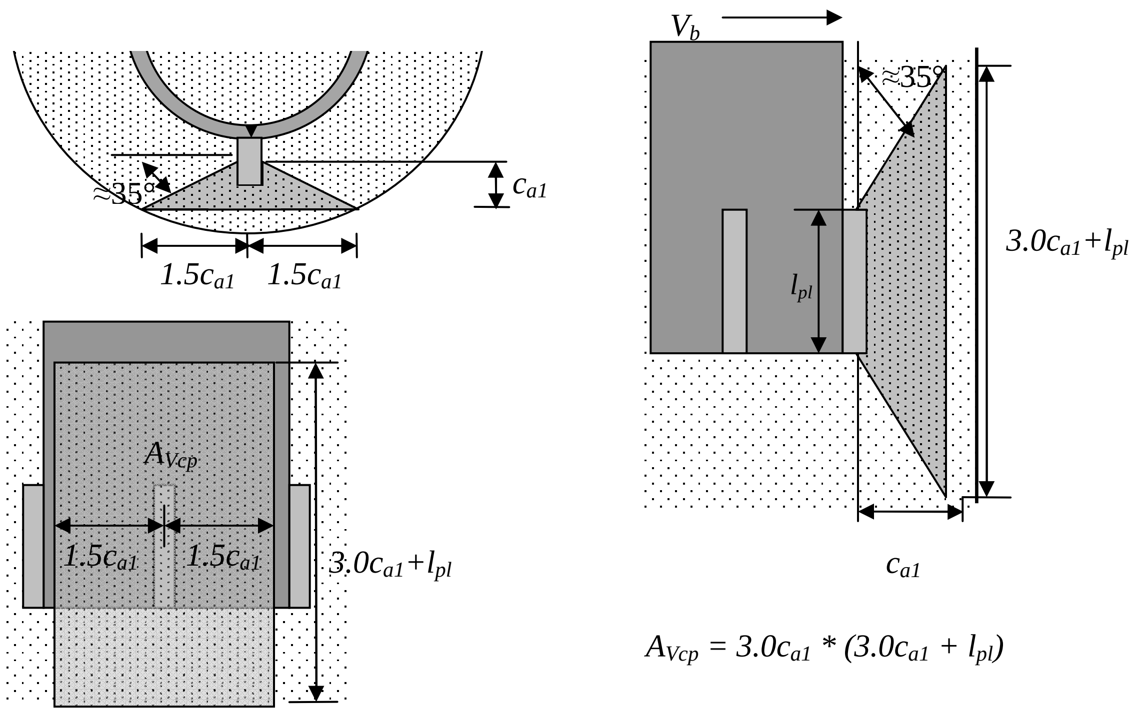


Figure 3-6. Determination of A_{Vcp} based on ≈35° failure cone for embedded pipe and plate section

Because the concrete breakout area for the plate is much larger than that of the anchor bolt, the ratio of the plate breakout area to the anchor bolt breakout area will include the increase in breakout strength for the plate due to the larger breakout area. There will be an increase in strength because it will take more force to cause a breakout on a larger volume of concrete. Equation 3-6 accounts for the additional strength of a concrete breakout for the embedded plate because the ratio of A_{Vcp} to A_{Vco} will be greater than one as can be seen by comparing Figure 3-5 and Figure 3-6. Equation 3-6 displays the equation utilized to determine the concrete breakout strength of a single plate.

$$V_{cbp} = \frac{A_{Vcp}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad (3\text{-}6)$$

Where

V_{cbp} = The nominal concrete breakout strength in shear of a single plate (lb.)

A_{Vcp} = The projected area of the failure surface for a single plate, used to determine the shear strength (in^2)
=$3.0c_{a1}$*($3.0c_{a1} + l_{pl}$)

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 3-4)

The contribution of each plate to the overall torsional strength of the embedded pipe (T_{cbp}) is twice the expected breakout strength (V_{cbp}) multiplied by the moment arm. It is twice the expected breakout strength because as mentioned earlier, the shear strength when loaded parallel to the edge of concrete is permitted to be twice that of the shear strength when loaded perpendicular to the edge of concrete and Equations 3-5 and 3-6 are for loading perpendicular to the edge of concrete.

$$T_{cbp} = 2V_{cbp} n r_p \qquad (3\text{-}7)$$

Where

T_{cbp} = The nominal torsional strength of the pedestal from concrete breakout (kip-ft)

V_{cbp} = The nominal concrete breakout strength in shear of plate configuration where the plates are not acting as a group (lb.)

n = The number of torsional plates in the configuration; the plates are not acting in a group

r_p = The radius of the pipe (in.)

3.1.2 Equivalent Side-Face Blowout Strength

Another method to determine the torsional strength of the embedded pipe and plate section is to determine the available bearing strength of concrete for the embedded pipe and plate section. The bearing strength was expected to be calculated similarly to the side-face blowout strength of a headed anchor in tension. The side-face blowout strength of a headed anchor in tension represents the bearing strength of the concrete at the head of the anchor. Figure 3-7 depicts the similarities in anticipated failure cones for the embedded pipe and plate section and the headed anchor configuration.

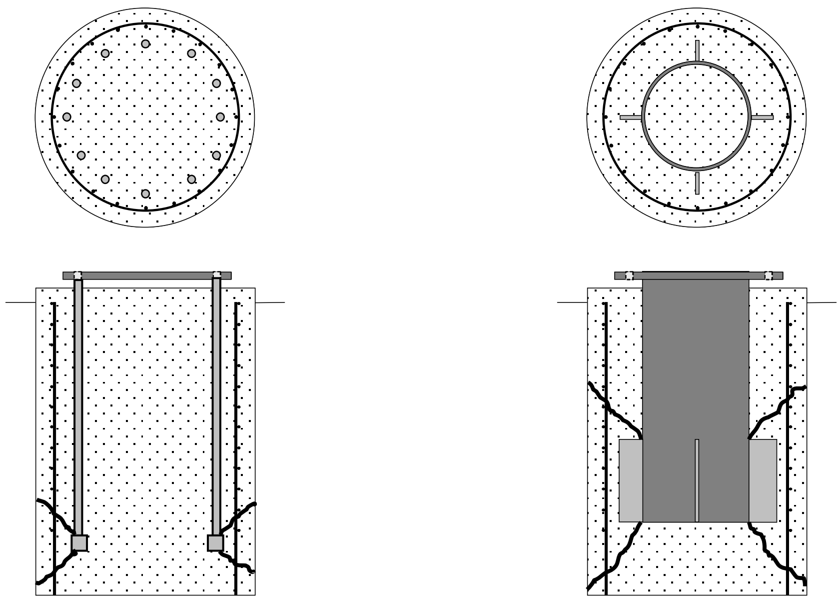


Figure 3-7. Similarities of failure cones in side-face blowout of a headed anchor in tension and the embedded pipe and plate section in torsion

The similarity in these failures shows that there requires little manipulation of the equation to determine the bearing strength for the embedded pipe and plate section. ACI 318-08 Appendix D determines the nominal side-face blowout strength of a headed anchor in tension (See Figure 3-8) in Equation D-17 and is shown below as Equation 3-8.

$$N_{sb} = 160 c_{a1} \sqrt{A_{brg}} \sqrt{f'_c} \qquad (3\text{-}8)$$

Where

N_{sb} = the nominal concrete side-face blowout strength of a single headed anchor in tension (lb.)

c_{a1} = distance from the center of an anchor shaft to the edge of concrete taken in the direction of the closest edge (in.)

A_{brg} = bearing area of the head of anchor bolt (in.^{2})

f'_c = specified compressive strength of concrete (psi)

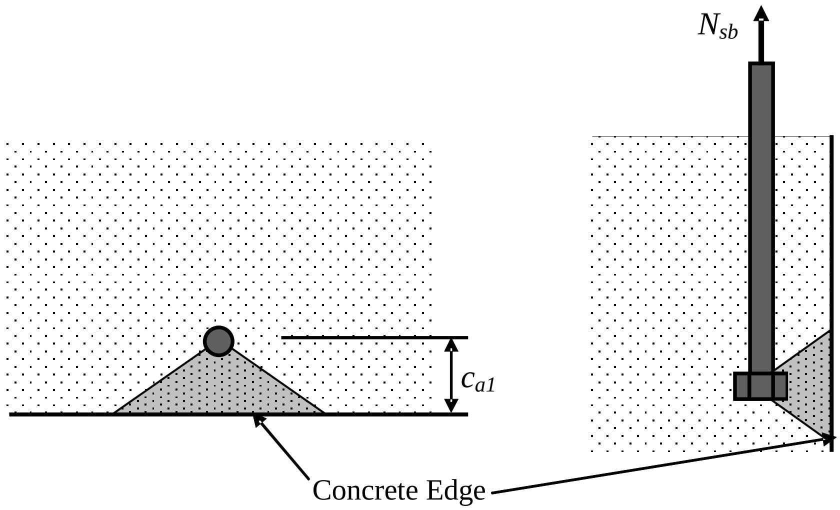


Figure 3-8. Concrete side-face blowout equation for a headed anchor in tension

The constant 160 from Equation 3-8 was determined from a 5% fractile in cracked concrete and is used to determine the nominal strength. By removing the safety factor attached to the 5% fractile and the cracked concrete, the constant for the mean side-face blowout strength of a single headed anchor in uncracked concrete loaded in tension is 200 (13). The mean side-face blowout strength of a single headed anchor in uncracked concrete is described in Equation 3-9, as shown below.

$$N_{sb} = 200 c_{a1} \sqrt{A_{brg}} \sqrt{f'_c} \tag{3-9}$$

The modifications necessary to Equation 3-9 to account for the embedded pipe and plate section was to substitute A_{brg} from the bearing area of the head of the anchor bolt to the bearing area of the plate and substitute the rectangular concrete's edge distance c_{a1} to the cylindrical concrete's edge distance c_{a1} (See Equation 3-4). The equivalent torsional strength was derived using N_{sb} and multiplying it by the number of plates and moment arm, which is equivalent to the radius of the pipe. See Equation 3-10 for how to calculate the torsional strength using N_{sb}.

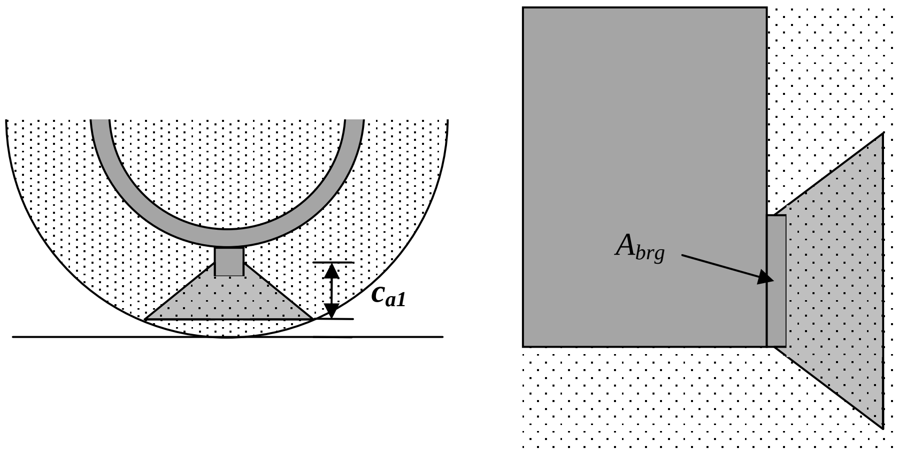


Figure 3-9. Schematic of anticipated failure and bearing area of torsion plate

$$T_{sb} = N_{sb} n r_p \tag{3-10}$$

Where

T_{sb} = The nominal torsional strength of the concrete pedestal from side-face blowout (kip-ft)

N_{sb} = The nominal concrete side-face blowout strength of a single plate in tension (lb.)

n = The number of torsional plates in the configuration

r_p = The radius of the embedded pipe (in.)

3.2 Design for Flexure

The next parameter to be designed for is flexure. One method of handling flexure would be to weld an annular plate to the bottom of the pipe. The plate would be able to resist the tensile and compressive forces induced by the flexure by bearing on the concrete. This failure would also produce a concrete breakout or side-face blowout that can also be compared to an anchor bolt failure.

3.2.1 Equivalent Concrete Breakout Strength in Shear

One method of hypothesizing the predicted behavior of the embedded section would be to treat it as a typical annular base plate with anchor bolts. When analyzing flexure on this setup, the flexure can be resolved into a compressive force on one side of the plate and a tensile force on the other side of the flexural plate (See Figure 3-10). The resolved forces can be viewed to act in one of two ways: shear parallel to an edge and an equivalent bearing pressure causing side-face blowout. In this section the hypothetical failure mode associated with shear parallel to the edge will be discussed.

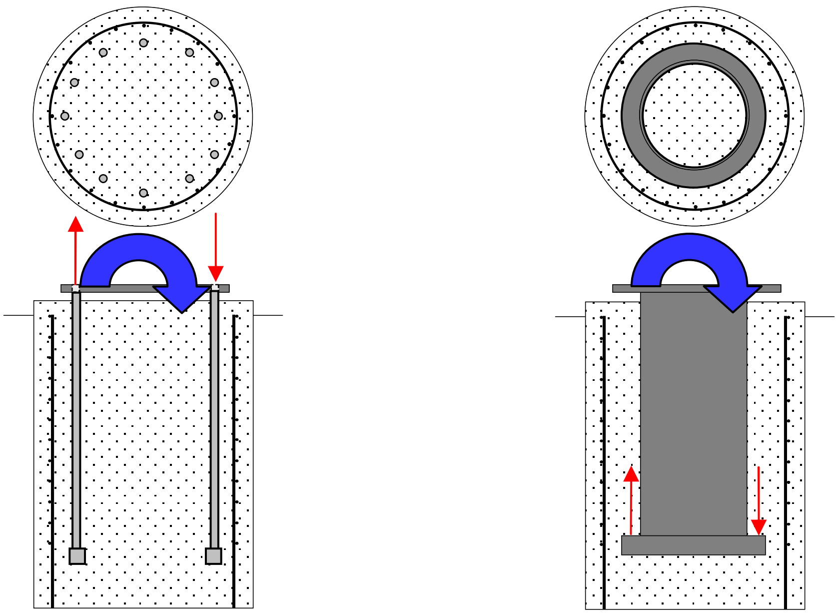


Figure 3-10. Flexure resolved into a tension and compression on an anchor bolt system and the proposed system

As shown in Figure 3-10, the flexural moment can be resolved into a tension and compression acting on opposite sides of the plate. Another way of looking at the tension and compression forces would be to rotate the foundation 90 degrees to more clearly see it as shear acting parallel to an edge (See Figure 3-11). These shears will create a breakout failure similar to that experienced during torsional loading on the welded plates. Modifying Equation 3-2 to account for the differences in the anchor bolt configuration and the embedded pipe and plate configuration yields Equation 3-11, shown below.

$$V_{bfp} = \left(13 \left(\frac{\ell_e}{t_{fp}} \right)^{0.2} \sqrt{b_{fp}} \right) \lambda \sqrt{f'_c} (c_{a1})^{1.5} \qquad (3\text{-}11)$$

Where

V_{bfp} = the basic concrete breakout strength in shear of one side of a flexural plate in cracked concrete (lb.)

ℓ_e = the equivalent bearing length of the annular plate, taken conservatively as 1/8 of the circumference of the centerline of the plate (in.)

t_{fp} = the thickness of the annular plate (in.)

b_{fp} = the bearing width of the annular plate (in.)

f'_c = specified compressive strength of concrete (psi)

c_{a1} = the edge distance, taken from the center of the width of the plate to the nearest concrete edge (in.)

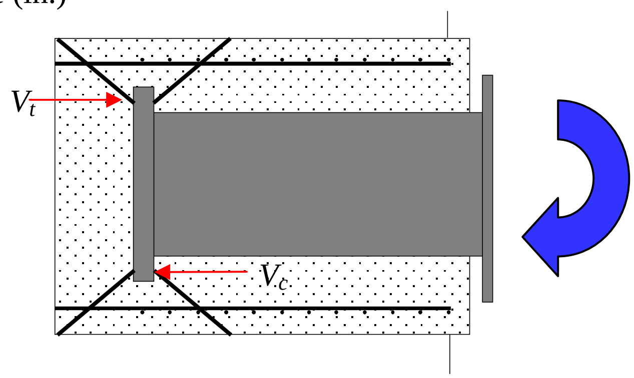


Figure 3-11. The tensile and compressive forces seen as shears acting parallel to an edge

Once the basic concrete breakout strength of one plate bearing area has been determined, then the total shear breakout capacity can be determined using Equation 3-12. Equations 3-11 and 3-12 are used to determine the shear strength perpendicular to an edge. To determine the shear strength parallel to an edge, the perpendicular shear strengths obtained need to be doubled. See Figure 3-12 for a visual representation of the values in Equations 3-11 and 3-12.

$$V_{cbfp} = \frac{A_{Vcp}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_{bfp} \qquad (3\text{-}12)$$

Where

V_{cbfp} = The nominal concrete breakout strength in shear of plate configuration where the plate bearing areas are not acting as a group (lb.)

A_{Vcfp} = The projected area of the failure surface for a single bearing location on the plate, used to determine the shear strength (in^2)
=(3.0c_{a1}+ l_e)*(3.0c_{a1} + t_{fp})

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 3-4)

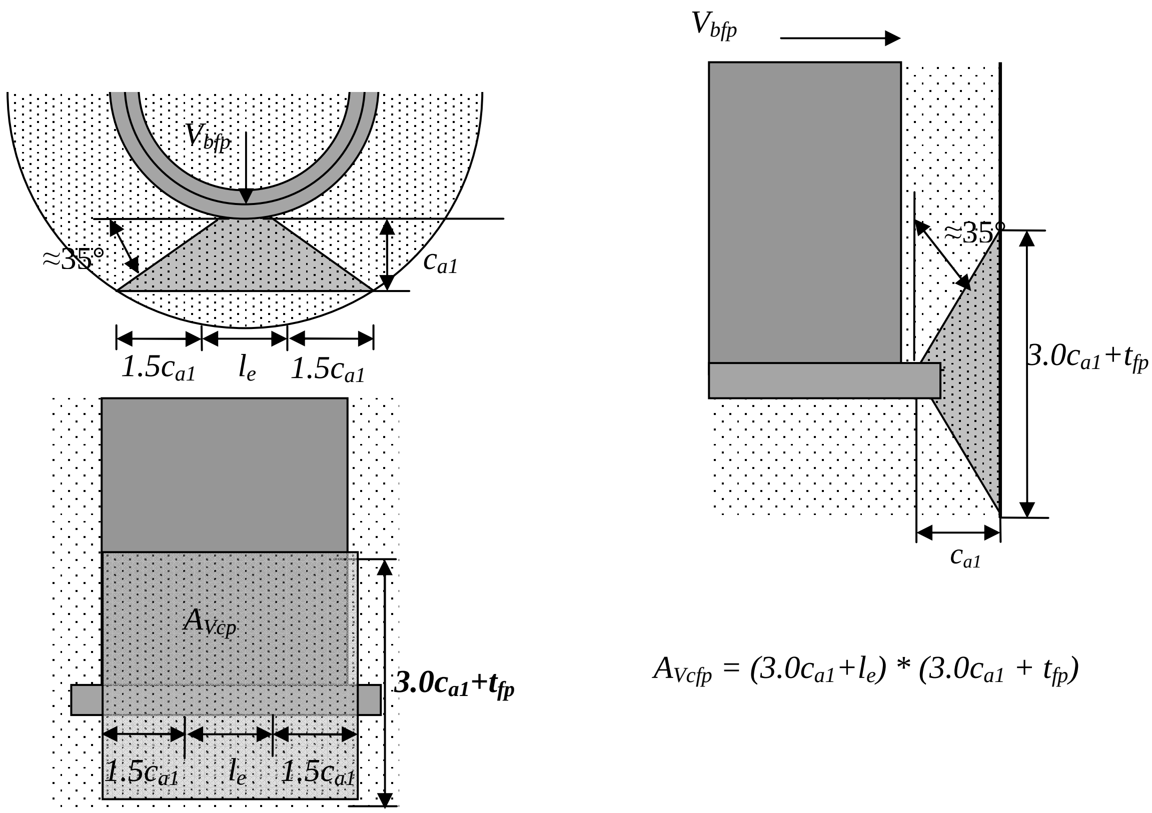


Figure 3-12. Determination of A_{Vcfp} based on ≈35° failure cone for embedded pipe and plate section

Using the value obtained from Equation 3-12, an equivalent flexural strength can be calculated using Equation 3-13.

$$M_{cbfp} = 2V_{cbfp} d_{fp} \tag{3-13}$$

Where

M_{cbfp} = The nominal flexural concrete breakout strength in shear of plate configuration where the plate bearing areas are not acting as a group (lb.)

V_{cbfp} = The nominal concrete breakout strength in shear of plate configuration where the plate bearing areas are not acting as a group (lb.)

d_{fp} = The diameter of the centerline of the flexural plate (in.)

3.2.2 Equivalent Side-Face Blowout Strength

The other method to determine the flexural strength of the embedded pipe and plate section is to determine the available side-face blowout strength of concrete for the embedded pipe and plate section. The side-face blowout strength was expected to be calculated similarly to the side-face blowout strength of a headed anchor in tension, with the bearing area modified from the head of the anchor to the bearing area of the flexural plate.

The similarity in these failures shows that there requires little manipulation of the equation to determine the side-face blowout strength for the embedded pipe and plate section. Equation 3-8 seen earlier in the chapter describes the nominal side-face blowout strength of a headed anchor in tension while Equation 3-9 describes the mean side-face blowout strength of a headed anchor in tension. Equation 3-9 would be used to determine the strength for each bearing area on the flexural plate. Figure 3-13 illustrates the bearing area for one location on the flexural plate.

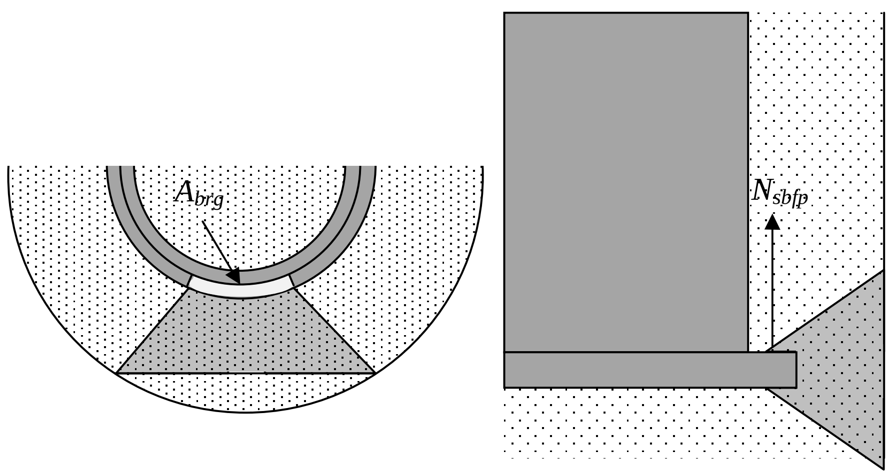


Figure 3-13. Illustration of bearing area on flexural plate for side-face blowout calculations

The difference in Equation 3-9 for a headed anchor bolt and the flexural plate system would be that the A_{brg} would be the bearing area of the flexural plate rather than the headed anchor. In order to quantify this, a recent study on tension and compression testing of signal/sign base plates utilizing anchor bolts compared bearing areas for calculating the bearing strength of headed anchor bolts was looked into (14). The current method utilizes a bearing area equivalent to the head area. This was found to be a very conservative approach, with the field tests yielding more than double the strength predicted using the equivalent bearing area equivalent to the head area. The recommendation of the paper was to utilize the spacing between bolts and the entire width of the embedded template as the bearing area (14). Based on this information, it would seem reasonable to utilize the same principles to estimate the bearing area of the plate. However, since there would be 4 bearing areas on the plate, it seems unreasonable to assume that the bearing area would be one quarter of the plate area. In order to be conservative it was assumed that the bearing area would be one eighth of the plate area. See Figure 3-14 for an illustration of the bearing area comparison.

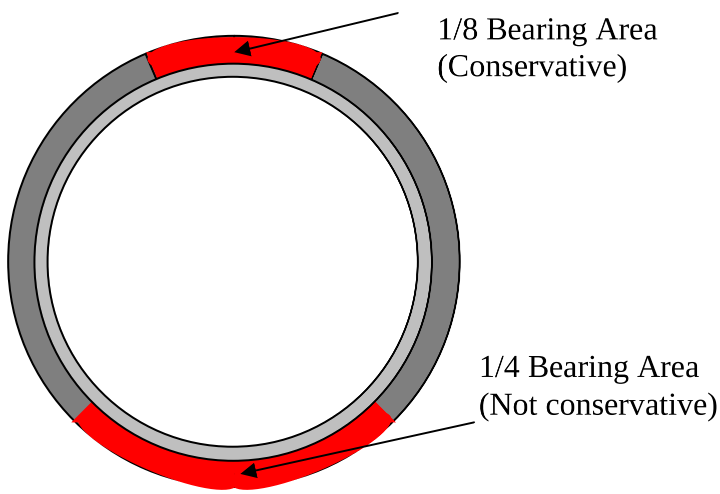


Figure 3-14. Flexural plate bearing area for side-face blowout calculations

By using this technique to calculate the bearing strength of the plate and using a moment arm of the diameter of the centerline of the plate, an equivalent flexural strength can be computed. See Equation 3-14 below to determine the equivalent flexural strength from side-face blowout.

$$M_{sb} = N_{sb} d_{fp} \qquad (3\text{-}14)$$

Where

M_{sb} = The nominal flexural strength of the concrete pedestal from side-face blowout (kip-ft)

N_{sb} = The nominal concrete side-face blowout strength of a single bearing area on the flexural plate in tension (lb.)

r_{fp} = The radius of the centerline diameter of the flexural plate (in.)

3.3 Design Implications Summary

By modifying the concrete breakout and bearing strength equations from ACI 318-08, a reasonable estimate of the torsional and flexural strength of the embedded pipe and plate section could be calculated. The estimated torsional and flexural strengths of the embedded pipe and plate section were calculated as approximately twice that of the traditional anchor bolt setup.

CHAPTER 4
DEVELOPMENT OF EXPERIMENTAL PROGRAM

After the background investigation, it was determined that the embedded steel pipe with welded plates would be the alternative used to develop the experimental program. The experimental program for the initial testing would be similar to that conducted on FDOT Project BD545 RPWO #54, using a lever arm to create primarily torsional loading on the foundation. The second test would induce both torsional and flexural loading on the alternative design. Based on the alternative identified from the background investigation, torsion from the attached member is transferred by bearing on the embedded plates. The flexure from the attached member is transferred by creating a tension and a compression on the embedded welded annular plate. A potential failure mode needed to be identified and a strength for this predicted failure mode quantified. The predicted torsional failure mode was a concrete breakout failure caused by bearing on the welded plates would occur as shown in Figure 4-1. Two possible methods of quantifying this were identified and are described in the previous chapter.

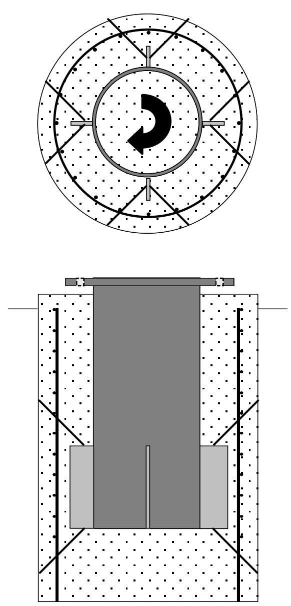


Figure 4-1. Predicted concrete breakout failure

One method to quantify the failure strength was to reference the equations from Appendix D of ACI 318-08 regarding anchors loaded in shear parallel to an edge and modify them to account for the additional concrete breakout area encountered by the plate configuration (5). Another potential way to determine the failure capacity of the embedded pipe and plate section was to consider the side-face blowout strength of the concrete caused by the welded plates similar to that of a headed anchor loaded in tension. In order to quantify this failure, the equations from Appendix D of ACI 318-08 were modified to account for the differences between the pipe and plate assembly and a headed anchor (5).

Based on the quantified values from these potential failure modes, the rest of the test apparatus was designed to preclude other failure modes and determine the tested strength of the pipe and plate assembly in order to develop design guidelines. This chapter elaborates on the development of the experimental test program.

As a side note, in both torsion and flexure, the predicted concrete breakout strength was less than the predicted side-face blowout strength. These strengths were utilized to determine the required strengths of the remainder of the test apparatus. Therefore, if the nominal strength of a portion of the test design did not exceed the predicted side-face blowout strength, yet exceeded the predicted concrete breakout strength, it was deemed sufficient.

4.1 Description of Test Apparatus

The test from FDOT Report BD545 RPWO #54 was designed to be a half-size model of field conditions for testing at the Florida Department of Transportation (FDOT) Structures Research Center. Therefore, the starting point for this test was to design the concrete shaft the same size as the half-size model from the previous report. During design of the first test, the concrete shaft was modified from the original half-size design of a 30” diameter to a 26” diameter to reduce the capacity of the concrete shaft so that the previously fabricated lever arm would be sufficient for the test. This process will be described in detail in the subsequent sections. The second test that was conducted for flexure and torsion was designed using the original half-size design of a 30” diameter. A schematic of the torsion test apparatus is shown in Figure 4-2. A schematic of the flexure and torsion test apparatus is shown in Figure 4-3. The final design for the torsion test apparatus consisted of the following:

- A 26” diameter concrete shaft that extended 3’-0” outward from the concrete block
- A 16” diameter steel pipe assembly with 4 welded 1” x 1” x 7” steel plates
- The 16” diameter embedded pipe assembly welded to a 24” diameter, 1” thick steel base plate with 12-1.75” diameter holes drilled for the anchor bolts to provide the connection this lever arm assembly and the embedded pipe assembly
- A 16” diameter, 10’-0” long steel pipe lever arm assembly
- Twelve 4.5” long, 1.5” diameter A490 bolts and associated nuts and washers to connect the lever arm assembly and the embedded pipe assembly
- A 6’-0” x 10’-0’ x 2’-6” reinforced concrete block to provide a fixed support at the base of the concrete shaft
- Two assemblies of C12x30 steel channels and plates to attach the block to the floor

The final design for the torsion and flexure test apparatus consisted of the following:

- A 30” diameter concrete shaft that extended 3’-0” outward from the concrete block
- A 16” diameter steel pipe assembly with 4 welded 1” x 1” x 7” steel plates and a welded 20” outside diameter annular plate
- A 16” diameter, 10’-0” long steel pipe lever arm assembly
- A 16” diameter, 7’-0” long steel extension pipe assembly
- The 16” diameter embedded pipe assembly was also welded to a 24” diameter, 1” thick steel base plate with 12-1.75” diameter holes drilled for bolts to provide the connection between this embedded pipe assembly and the lever arm assembly
- 12- 4.5” long, 1.5” diameter A490 bolts and associated nuts and washers to connect the extension pipe assembly and the embedded pipe assembly

- An additional 12-4.5” long, 1.5” diameter A490 bolts and associated nuts and washers to connect the extension pipe assembly and the lever arm assembly

- A 6’-0” x 10’-0” x 2’-6” reinforced concrete block to provide a fixed support at the base of the concrete shaft

- Two assemblies of C12x30 steel channels and plates to attach the block to the floor

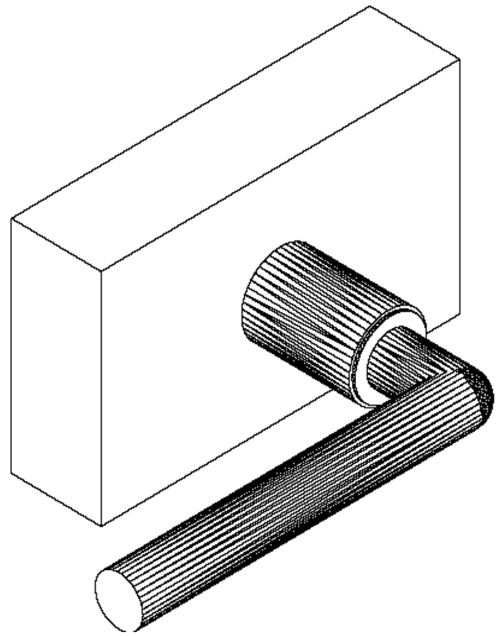


Figure 4-2. Schematic of torsion test specimen

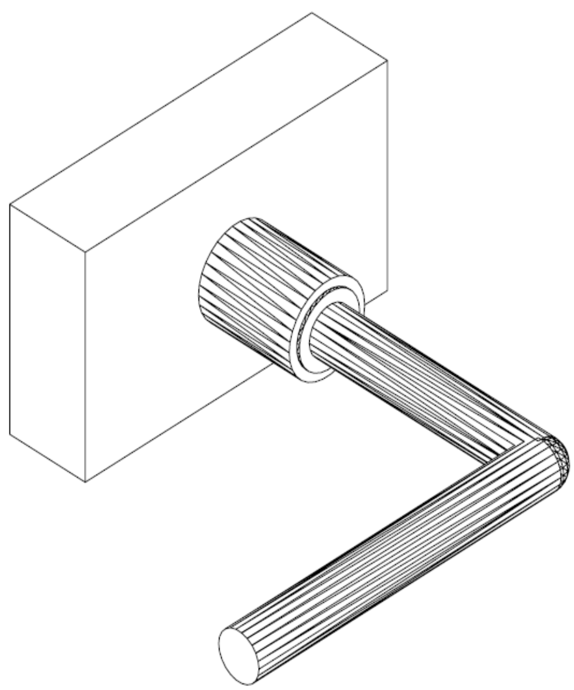


Figure 4-3. Schematic of torsion and flexure test specimen

The basis for the selection of the concrete shaft's diameter was one half of the diameter of a typical field design. One problem that also needed to be addressed was to maintain the torsional strength of the concrete shaft below that of the previously fabricated lever arm assembly. Based on the quantified strength of the embedded pipe and plate assembly, the remaining components of the test apparatus were designed to preclude all failure modes other than the concrete breakout or side-face blowout of the welded torsional plates and/or flexural plate.

More detailed information regarding the design of the components of the test apparatus is provided in the subsequent sections. Much of the design of the embedded pipe and plate apparatus and reinforced concrete shaft was performed using an iterative process. Therefore, the following sections will be organized as chronologically as possible, though some information in later sections was necessary to design components in earlier sections. Figure 4-4, Figure 4-5, Figure 4-6, and Figure 4-7 provide more detailed drawings of the torsion test apparatus. The flexural test apparatus was very similar with the main differences being an inclusion of a flexural plate on the embedded section and a flexural extension pipe on the testing assembly. Figure 4-8 shows a 3-D isometric view of the embedded section for the second test. For larger scale, dimensioned drawings for both tests, refer to Appendix A. Complete design calculations are located in Appendix B.

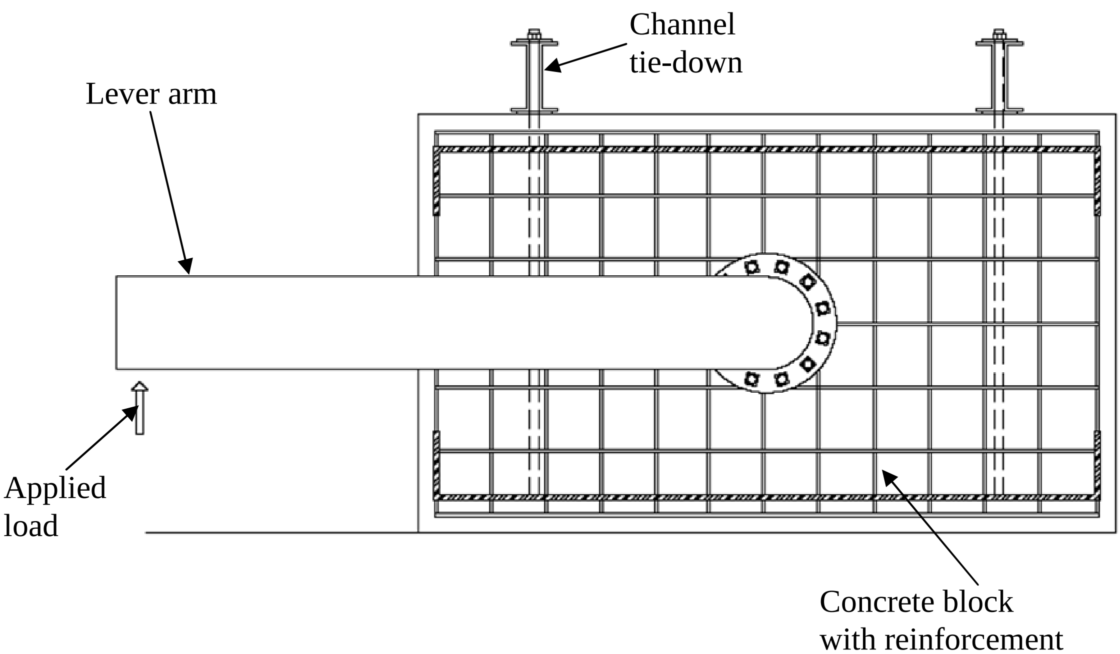


Figure 4-4. Front view of torsion test setup

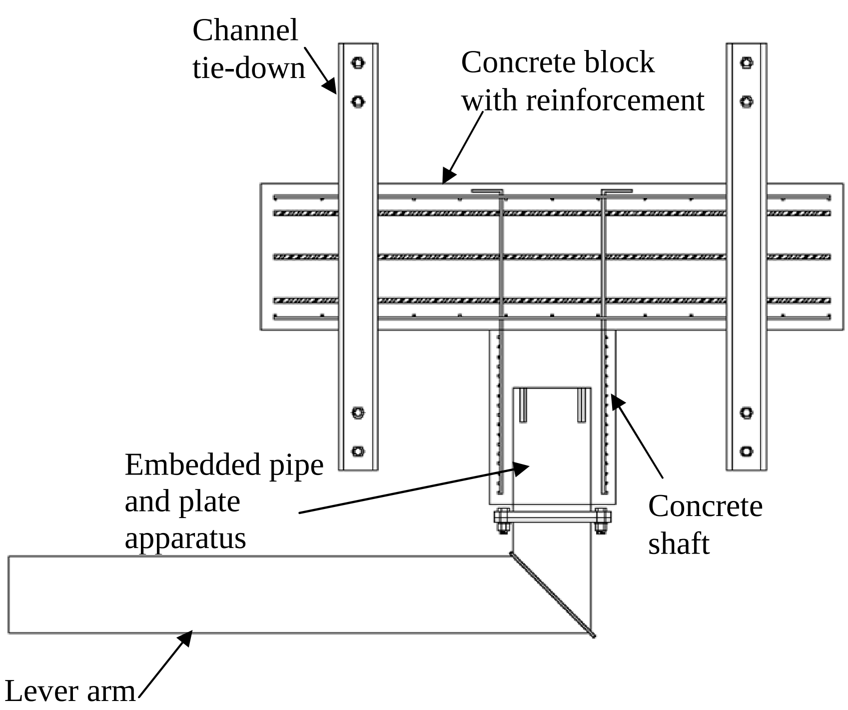


Figure 4-5. Top view of torsion test setup

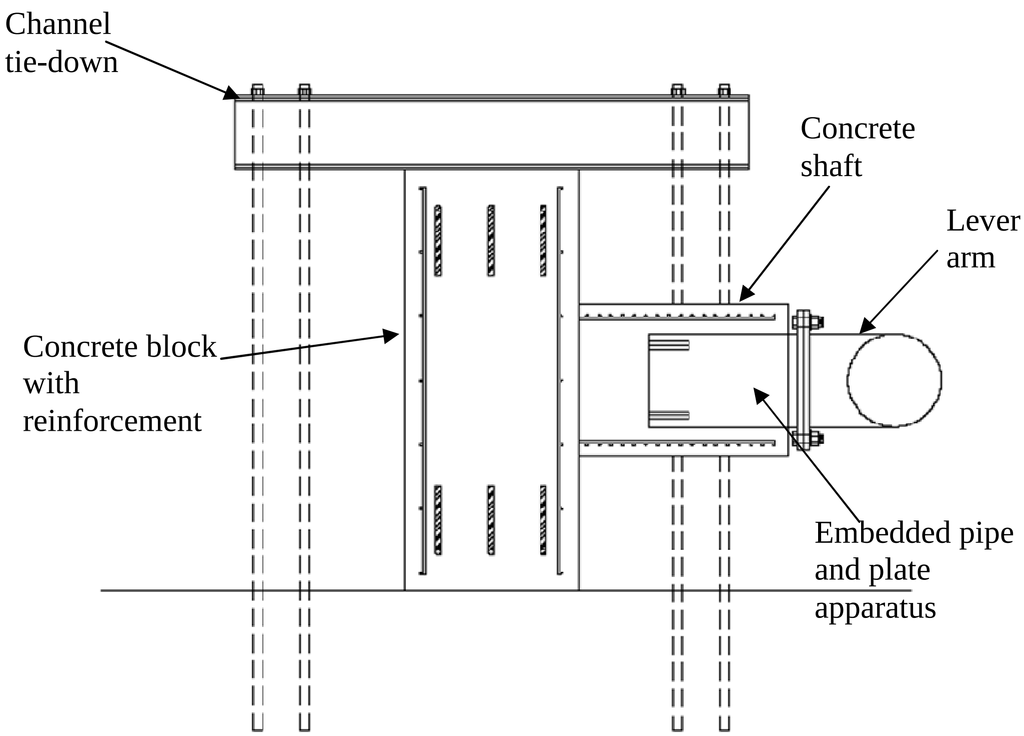


Figure 4-6. Side view of torsion test setup

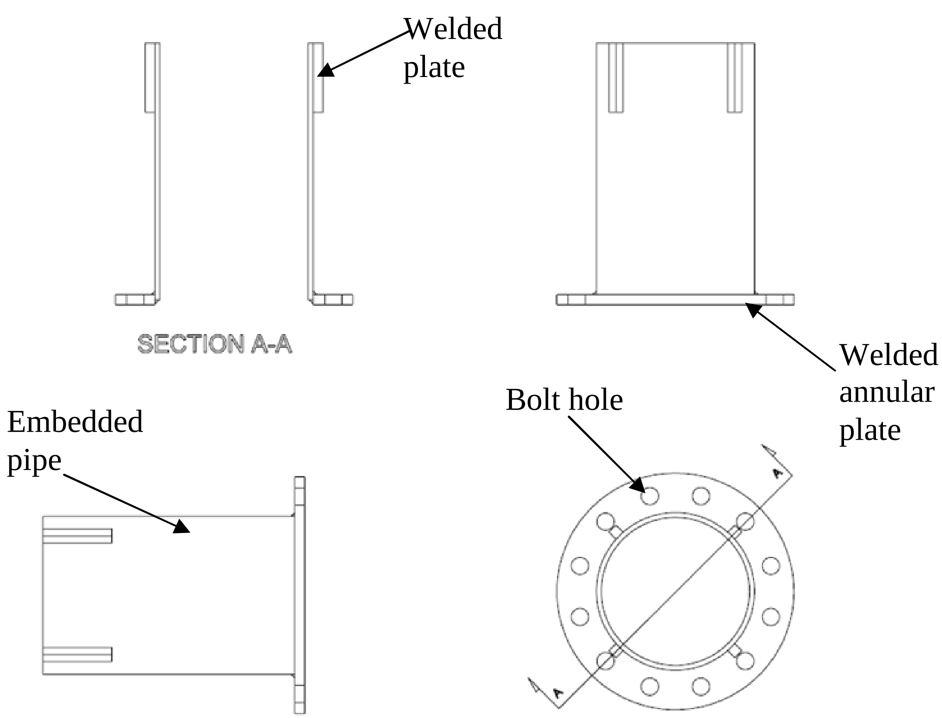


Figure 4-7. Views of the embedded torsion pipe section

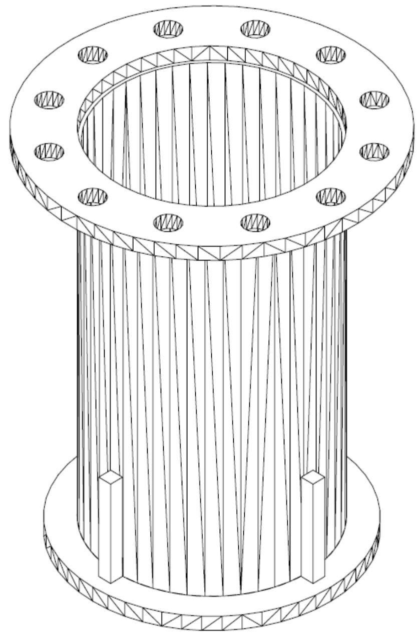


Figure 4-8. Isometric view of embedded torsion and flexural pipe section for the second test

4.2 Embedded Pipe and Plate Design

The embedded pipe and plate sections' design was based upon the strength of the lever arm(s) and on the flexural and torsional strength requirements of the test procedure. The embedded pipe and plate section must be at least as strong as or stronger than the traditional anchor bolt design in order to be a viable alternative. The embedded pipe and plate section would be bolted to the lever arm assembly, which was designed in the previous experiment as an HSS 16"x.500" with a 24" diameter annular plate. It seemed beneficial to size the pipe and base plate the same as the lever arm assembly. Based on this configuration, the welded stiffener plates, welds, concrete breakout and side-face blowout strengths were determined.

4.2.1 Concrete Breakout and Bearing Strength

The facet of the design that dictated the rest of the design was the predicted concrete breakout strength and side-face blowout strength of the embedded pipe and plate apparatus. For design purposes, a concrete strength of 5500 psi was assumed. This value was adjusted for more accurate strength prediction when the average 28-day compressive strength of the concrete cylinders was obtained.

By using Equation 3-7, the torsional breakout strength for the assembly was determined to be 249 kip-ft for the torsion test apparatus. Similarly, by using Equation 3-10, the torsional side-face blowout strength for the assembly was determined to be 390 kip-ft. See Figure 4-1 for the expected breakout configuration of the torsional test assembly. The expected torsional breakout and side-face blowout strengths of the torsional and flexural test assembly were calculated similarly using Equations 3-7 and 3-10. The expected torsional breakout strength of the torsion and flexure test assembly was 348 kip-ft while the expected torsional side-face blowout strength of the second test assembly was 523 kip-ft.

By using Equations 3-13 and 3-14, the flexural breakout and side-face blowout strengths could be determined. Equation 3-13 determined a flexural breakout strength of 218 kip-ft. Equation 3-14 determined a flexural side-face blowout strength of 337 kip-ft. Of concern in the combined torsion and flexure test was the potential interaction between torsional and flexural breakout due to overlap in the breakout surfaces (Figure 4-9). A linear interaction diagram between torsion and flexural strengths was produced to predict a testing failure load (See Figure 4-10). Because of the test arrangement, a 1 kip applied load would produce 9 kip-ft of torsional moment and 8 kip-ft of flexural moment. Therefore if a completely linear interaction occurred then the maximum flexural moment would be 128 kip-ft and the maximum torsional moment would be 144 kip-ft.

4.2.2 Welded Stiffener Plates Design

The starting point for the design of the welded stiffener plates was to determine their width and thickness. It was determined that a 1”x1” plate would be approximately equivalent to an

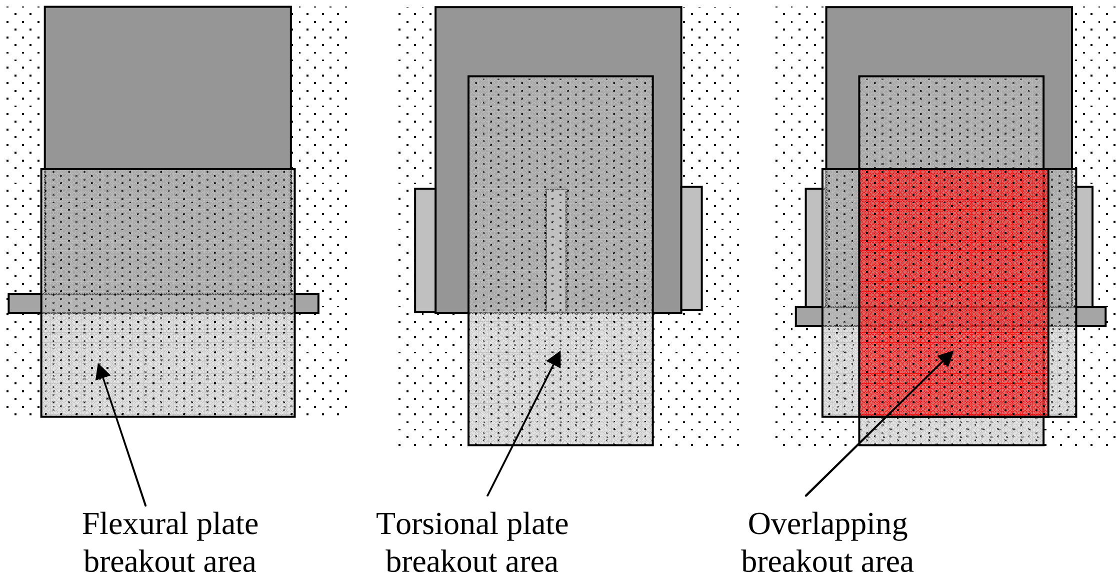


Figure 4-9. Breakout overlap of the torsional and flexural breakouts

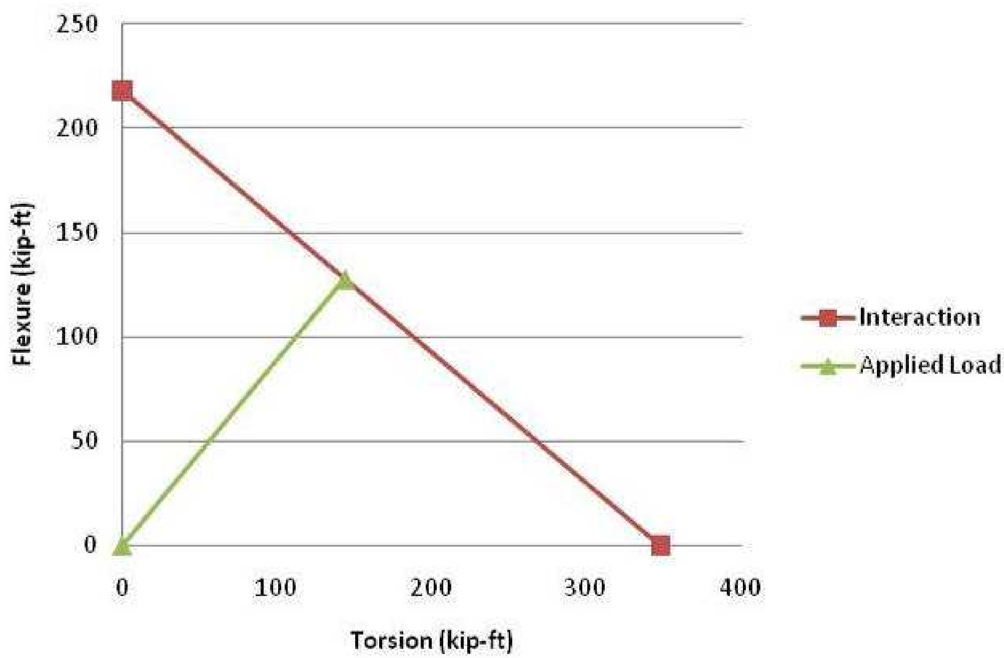


Figure 4-10. Interaction between torsion and flexure for concrete breakout

anchor bolt. The length of the plate was determined by the required 3/8" fillet weld length that corresponded to the resolved shear force acting on the plates, 93 kips, which was determined from the equivalent torsional concrete breakout strength of 249 kip-ft. The required weld length was determined as 6". To be conservative, the plates were designed to be 1"x1"x7".

In order to be sure that the force would be transferred to the plates as predicted, it was necessary to ensure that the longitudinal reinforcement had enough length to be fully developed before the cone of the concrete breakout reached the longitudinal reinforcement. The longitudinal reinforcement was based upon that determined in FDOT Report BD545 RPWO #54, 24 #4 bars evenly spaced. The development length was calculated using ACI 318-08 12.2.3 and was determined to be approximately 8” (5). The breakout length above the 7” stiffener plate was determined to be approximately 5.6”. Therefore, when the embedded pipe was placed at a depth of 24” in the concrete shaft and the welded plates were placed at the bottom of the pipe, enough concrete shaft length would be available for full development of the longitudinal reinforcement.

4.2.3 Annular Flexural Plate Design

The annular flexural plate needed to be designed to have an adequate bearing area for the load to be transferred to the concrete. The welds needed to be designed to preclude failure from the applied flexure. Therefore, the starting point of the design of the annular plate was to use the same thickness as that used in the base plate, which was 1”, to preclude yielding. The plate was designed to have a 20” outside diameter and a 16” inside diameter. The outside diameter was designed as 20” in order to allow for the concrete’s aggregate to be able to pass between the flexural plate and the reinforcement cage of the concrete shaft. The assumed bearing area, as described in the previous chapter, was 1/8 of the circumference of the centerline of the plate, or in this design 7”, by the half the width of the plate, which was 2”. This bearing area is considered conservative due to recent findings (14). The welding for the plate was determined to be the same as the previous design’s base plate welds, or 3/8” fillet welds on the exterior and interior of the annular plate and pipe connection.

4.2.4 Annular Base Plate Design

The annular base plate for the embedded pipe and plate was designed to align with the annular base plate of the lever arm apparatus. It was designed to have a 24” diameter, 1” thickness, with 12-1.75” diameter holes centered on the plate. Standard A490 1.5” diameter bolts were designed to replace the 1.5” diameter anchor bolts utilized in the previous design. The equivalent torsional bolt bearing strength and bolt shear were calculated as 2418 kip-ft and 1272 kip-ft respectively, which greatly exceeds the concrete breakout and side-face blowout strengths calculated earlier. The welding for the plate was determined to be the same as the previous design, or 3/8” fillet welds on the exterior and interior of the annular plate and pipe connection.

4.2.5 Pipe Design

The pipe was determined to be embedded in the foundation 24”, which is approximately equivalent to the 26” embedment length of the anchor bolts in the previous design. In order to allow for the bolts to be fastened at the base plate, an additional 2.5” was included in the length. As stated earlier, the pipe was designed as an HSS 16”x.500” with a yield strength of 42 kips/in^2 and an ultimate strength of 58 kips/in^2.

The torsional strength of an HSS 16”x.500” pipe was determined using AISC 2005 Specification H3.1 as 359 kip-ft (15). This was a limiting factor on the size of the concrete shaft, as will be explained in the subsequent section. Figure 4-11 shows the fabricated pipe and plate section.

4.3 Concrete Shaft Design

The design of the concrete shaft was initially based on the same dimensions as the concrete shaft used in Project BD545 RPWO #54. The reasoning behind this was to obtain comparable results to determine the benefits and drawbacks to the new design as compared to the anchor bolt design. The previous concrete shaft was based upon developing a test specimen approximately

Figure 4-11. Fabricated pipe and plate apparatus

one half of the size of the foundation that was investigated in a site visit for that project (1). However, based on the 30” diameter of the concrete shaft used in the previous design, it became apparent that the calculated torsional strength of the embedded pipe and plate apparatus would exceed the torsional strength of the lever arm utilized in the previous test. Therefore, the concrete shaft diameter was reduced to 26”. From there, the torsional and flexural capacity was determined using ACI 318-08 requirements, taking care to prevent failure before the concrete breakout or bearing strength was encountered and exceeded. A concrete strength of 5500 psi was utilized in the calculations, which is the strength indicated on FDOT standard drawings.

4.3.1 Concrete Shaft Diameter Design

The starting point for the concrete shaft diameter was 30”, the same as that of the previous project. Using this concrete shaft diameter, the value of c_{a1} was determined to be approximately 5”. The calculated equivalent torsional concrete breakout strength was determined to be 296 kip-ft. The calculated equivalent torsional bearing strength was determined to be 446 kip-ft. The torsional strength of the lever arm pipe was calculated to be only 359 kip-ft, which exceeds the concrete breakout strength and does not exceed the bearing strength. Since the estimated strength will likely lie between those values, the lever arm pipe does not provide enough strength. For the first test the concrete shaft diameter was reduced to 26" to decrease the concrete breakout and bearing strength to 212 kip-ft and 333 kip-ft, respectively. However, for the second test a 30" concrete shaft diameter was chosen because it was thought that the interaction of the flexural and torsional failure modes would reduce the overall strength of each failure mode and the increased value of c_{a1} would be compensated for.

4.3.2 Torsion Design

The basic threshold torsional strength of the concrete shaft was calculated using ACI 318-08 11.6.1(a) to be 18 kip-ft (5). The threshold torsional strength does not take into account the reinforcement present in the concrete shaft and therefore will likely be exceeded. Therefore, the nominal torsional strength, which does take into account reinforcement, was used as the design torsional strength. The cracking torsional strength was determined from ACI 318-08 R11.6.1 as 73 kip-ft (5). Since the concrete breakout and side-face blowout torsional strengths exceeded this value, it indicated that there would be torsional cracks in the concrete shaft before it fails.

In order to calculate the nominal torsional strength of the concrete shaft, the reinforcement needed to be specified. The starting point was derived from the previous design, with the transverse hoop steel being comprised of #3 bars spaced at 2.5". However, it became clear that the torsional strength with this reinforcement scheme, 191 kip-ft, was insufficient to exceed the concrete breakout or bearing strength of the section, 212 kip-ft or 333 kip-ft, respectively. Therefore, the hoop steel size was increased to #4 bars and the spacing decreased to 2" to yield a nominal torsional strength of 426 kip-ft, which exceeded the concrete breakout strength of the section and almost attained the bearing strength of the section. This was sufficient because it was estimated that the experimental strength would lie somewhere between these values.

4.3.3 Longitudinal and Transverse Reinforcement

As was previously stated, the hoop steel for the torsion test was comprised of #4 bars spaced at 2”. The hoop steel for the torsion and flexure test was comprised of #3 bars spaced at 2.5”. The hoop steel’s center-to-center diameter was determined to be 22” for the torsion test and 27” for the torsion and flexure test. The splice length of the hoop steel was determined using ACI 318-08 12.2.3 to be approximately 16” (5).

The longitudinal steel layout for the torsion test comprised of 24 #4 bars evenly spaced around a 21” center-to-center diameter. The longitudinal steel layout for the torsion and flexure test comprised of 24 #4 bars evenly spaced around a 26” center-to-center diameter. The longitudinal steel required a 6” hook and a development length of 8” into the concrete block. The longitudinal bars extended 27” into the concrete block for ease of construction, which exceeded the development length.

4.3.4 Flexure Design

The flexural capacity of the concrete shaft was also deemed necessary because the setup of the test imposed both torsion and flexure on the concrete shaft. The longitudinal bars detailed in the previous section would provide the flexural reinforcement for the concrete shaft. The ACI stress block method detailed in ACI 318-08 Chapter 10 (5) was utilized to determine the flexural strength. It was determined that the flexural strength of the torsion test’s section was 245 kip-ft and the torsion and flexure test’s section was 296 kip-ft. The anticipated maximum applied flexure for the first test was 125 kip-ft. The anticipated maximum applied flexure for the second test would be transferred to the concrete by the flexural plate on the bottom of the pipe.

4.4 Concrete Block and Tie-Down Design

For both tests, the concrete block was designed to provide a fixed base for the concrete shaft. The design of the reinforcement was based upon a strut-and-tie model design outlined in ACI 318-08 Appendix A (5). The reinforcement was also analyzed using the beam theory to be sure that the reinforcement was adequate in shear and flexure. The information obtained from these approaches determined that 6 #8 bars, each with a 12 in. hook on each end, would be sufficient. 3 of the #8 bars would be placed on the top of the block and the remaining 3 #8 bars would be placed on the bottom of the block. Additional reinforcement included two cages of #4 bars placed in the block's front and back faces. These additional reinforcement cages would meet the supplementary reinforcement requirements. Using this reinforcement arrangement, the concrete block was determined to be a fixed base for the concrete shaft.

The tie-down was designed to be comprised of two channels connected by welded plates. The channels individually and as a channel assembly were designed for flexure and local buckling as specified in AISC 2005 (15). Each channel assembly's resistance was required to not exceed the floor capacity of 100 kips on either end, or 200 kips total. The bearing capacity of the concrete at the point of contact between the channel assembly and the concrete block was also checked to ensure that the loading from the channel would not cause the concrete to fail in that region.

4.5 Instrumentation

To successfully obtain data from the experimental program, a plan for instrumentation needed to be designed. The rotational stiffness of the concrete shaft was necessary to understand the behavior of the newly designed concrete shaft. To obtain this information, a system of linear variable displacement transducers (LVDTs) would need to be arranged.

To accurately determine the rotational stiffness of the concrete shaft, a system with 11 LVDTs was arranged. The arrangement of the LVDTs is detailed in Figure 4-12 through Figure 4-15. There will be one LVDT 6" from the point of applied force. There will be 4 LVDTs on the base plate, 3 measuring vertical displacement, 1 measuring horizontal displacement (See Figure 4-12). The measurement from D4 (as seen in Figure 4-12) will measure the horizontal displacement of the base plate. The rotation of the base plate was calculated using Equation 4-1.

$$R = \tan^{-1}\left(\frac{D_1 + D_3}{D_{gage}}\right) \tag{4-1}$$

Where

R = base plate rotation (rad)

D_1 = displacement of LVDT D1 (in.)

D_3 = displacement of LVDT D3 (in.)

D_{gage} = distance between LVDTs D1 and D3 (in.)

Figure 4-13 shows the arrangement of the LVDTs on the top of the concrete shaft. Figure 4-14 shows the arrangement of the LVDTs on the bottom of the concrete shaft, where the concrete shaft meets the block. The purpose of these LVDTs was to measure the rotation of the concrete shaft relative to the base plate. Figure 4-15 shows the LVDT 6" from the load location.

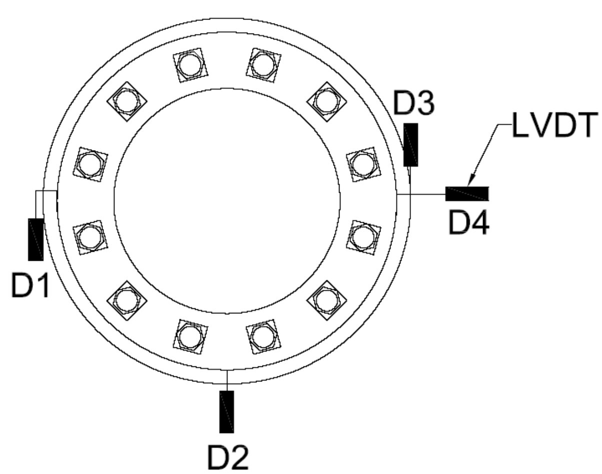


Figure 4-12. Arrangement of the LVDTs on base plate

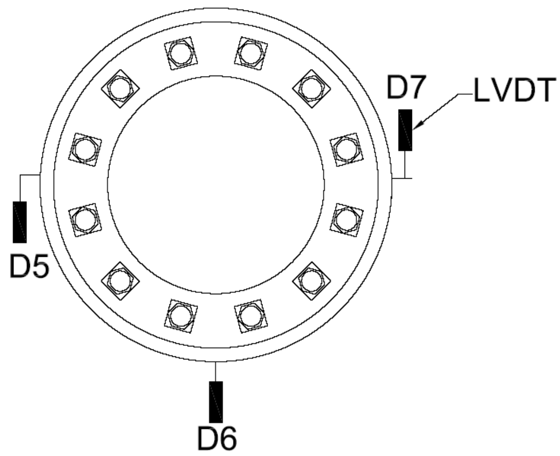


Figure 4-13. Arrangement of the LVDTs on the top of the concrete shaft

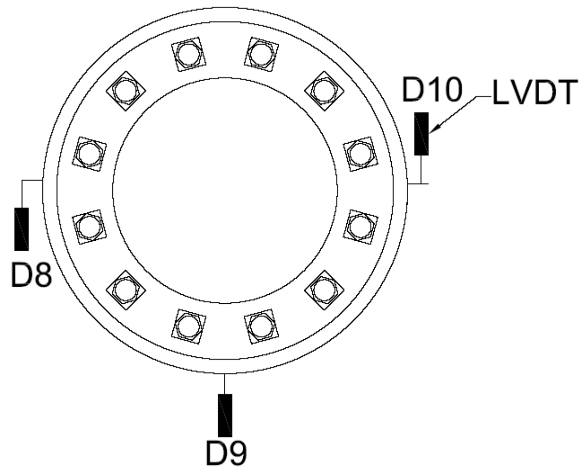


Figure 4-14. Arrangement of the LVDTs on the bottom of the concrete shaft

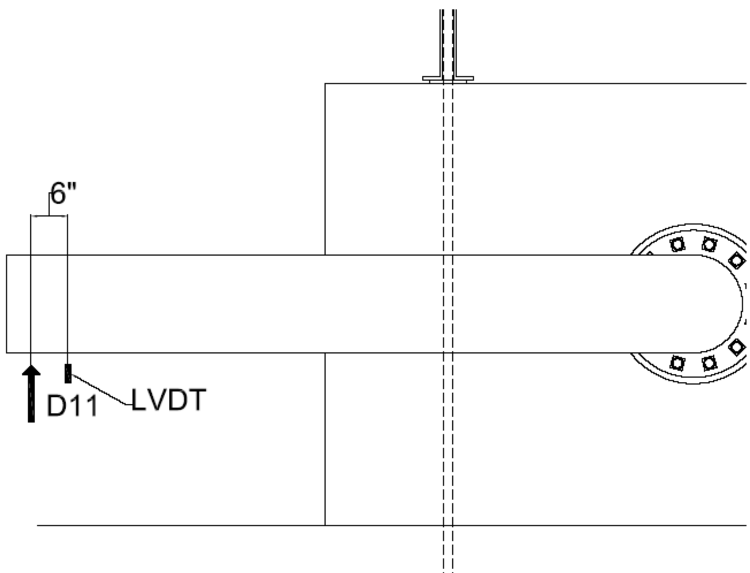


Figure 4-15. Arrangement of the LVDTs at the load location

4.6 Summary of Torsion Design

To summarize, the previous sections describe the design of the various components of both experimental programs. For the torsion test, the key element of the design that dictated the rest of the design was the concrete shaft. The concrete breakout or bearing strength of the shaft with the embedded pipe and plate apparatus was the ultimate strength of the entire system. All other components of the system were designed to preclude failure from these elements. This way the experimental strength of the embedded pipe and plate system could be observed and appropriate design guidelines could be written to detail the strength of the new system. Appendix A shows detailed and dimensioned drawings of the testing apparatus. Appendix B shows detailed calculations for the test apparatus.

The most critical components of the design were the embedded pipe and plate apparatus and the reinforced concrete shaft. As long as the components of the concrete shaft and embedded pipe and plate section exceeded that of the equivalent torsional concrete breakout strength then the design was sufficient.Table 4-1, shown below, summarizes the essential design components, their equivalent torsional strengths, whether the strengths are mean or nominal, and their ratio compared to the concrete breakout strength.

Table 4-1. Summary of pertinent design strengths for torsion test with 5500 psi concrete

Failure Mode	Capacity	Mean or Nominal?	Predicted Load	Ratio of Failure Capacities
Embedded Pipe and Stiffeners				
Equivalent Torsion from Shear Parallel to an Edge	249 kip-ft	Mean	27.67	1.00
Equivalent Torsion from Side Face Blowout	391 kip-ft	Mean	43.44	1.57
Circular Shaft - 26"				
Torsion	373 kip-ft	Nominal	41.44	1.50
Flexure	252 kip-ft	Nominal	126.00	2.02
"Superstructure" Pipes - 16" x .5"				
Torsion	359 kip-ft	Nominal	39.89	1.44
Flexure	392 kip-ft	Nominal	196.00	3.14

4.7 Summary of Torsion and Flexure Design

For the torsion and flexure test, once again the key element of the design that dictated the rest of the design was the concrete shaft. The concrete breakout or bearing strength of the shaft with the embedded pipe and plate apparatus in torsion and flexure was the ultimate strength of the entire system. All other components of the system were designed to preclude failure from these elements. Appendix A shows detailed and dimensioned drawings of the testing apparatus. Appendix B shows detailed calculations for the test apparatus.

Table 4-2, shown below, summarizes the essential design components, their equivalent torsional and flexural strengths, whether the strengths are mean or nominal, and their ratio compared to the interaction torsional and flexural strength values.

Table 4-2. Summary of pertinent design strengths for torsion and flexure test with 5500 psi concrete

Failure Mode	Capacity	Mean or Nominal?	Predicted Load	Ratio of Failure Capacities
Embedded Pipe and Stiffeners				
Equivalent Torsion from Shear Parallel to an Edge	348 kip-ft	Mean	38.67	2.42
Equivalent Torsion from Side Face Blowout	523 kip-ft	Mean	58.11	3.63
Equivalent Flexure from Shear Parallel to an Edge	218 kip-ft	Mean	27.25	1.70
Equivalent Flexure from Side Face Blowout	337 kip-ft	Mean	42.13	2.63
Anticipated Interaction				
Torsional Strength	144 kip-ft		16.00	1.00
Flexural Strength	128 kip-ft		16.00	1.00
Circular Shaft - 30"				
Torsion	253 kip-ft	Nominal	28.11	1.76
Flexure	296 kip-ft	Nominal	37.00	2.31
"Superstructure" Pipes - 16" x .5"				
Torsion	359 kip-ft	Nominal	39.89	2.49
Flexure	392 kip-ft	Nominal	49.00	3.06

CHAPTER 5
EXPERIMENTAL TEST RESULTS

Two separate tests were conducted on different specimens. The first test was conducted to determine the viability of the alternative chosen in torsion only. This was determined by the comparison of the experimental strength to an equivalent anchor bolt assembly's calculated strength (See Appendix A). The second test was conducted to determine the viability of the alternative chosen in torsion and flexure and to determine the interaction of the torsion and flexure failure modes. This also was determined by comparing the experimental strength of the system to an equivalent anchor bolt assembly's calculated strength (See Appendix A).

5.1 Torsion Test

5.1.1 Behavior of Specimen During Testing

The first test comprising of primarily torsional loading was conducted on September 23, 2009 at the Florida Department of Transportation Structures Research Center. The test specimen was gradually loaded and the formation of cracks on the surface of the concrete was monitored. At approximately 76.5 kip-ft, the bolts in the base connection slipped. This was because 1.5" diameter bolts were used in 1.75" diameter bolt holes. Approximately 1/4" slip occurred. This can be seen in Figure 5-1. At approximately 85.5 kip-ft, torsional cracks began to form on the concrete shaft (See Figure 5-2). At approximately 153 kip-ft, concrete breakout failure cracks began to form on the concrete shaft while the torsional cracks continued to widen (See Figure 5-3). At approximately 191 kip-ft, the concrete breakout failure cracks began to widen noticeably (See Figure 5-4). The foundation continued to be loaded until the specimen stopped taking on more load. The torsion load peaked at approximately 250 kip-ft (See Figure 5-5). At failure the foundation displayed the predicted breakout cone extending into the foundation. As intended, the rest of the test specimen did not fail before the predicted breakout failure occurred.

Figure 5-1. Lines drawn on base plate to show bolt slippage

Figure 5-2. Formation of torsional cracks

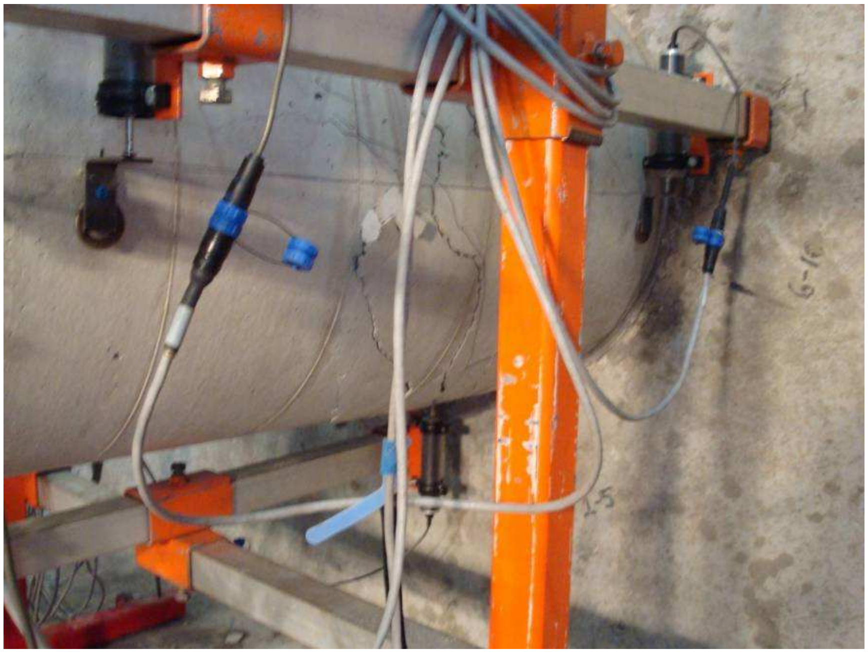


Figure 5-3. Formation of concrete breakout failure cracks

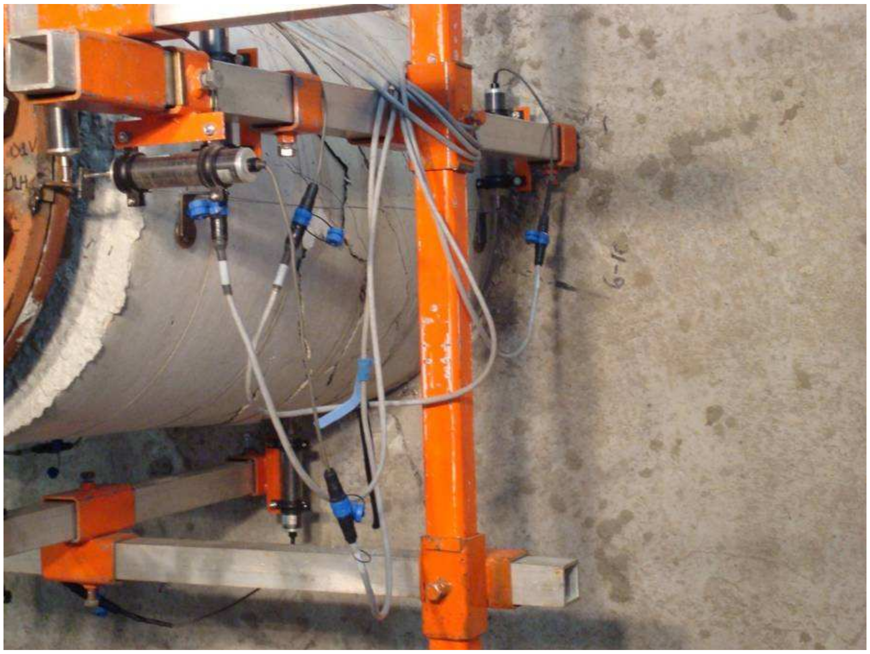


Figure 5-4. Concrete breakout failure cracks widen

Figure 5-5. Specimen at failure

5.1.2 Summary of LVDT Test Results

Data was reduced to formulate an applied torsion versus plate rotation plot. The plot shows that the embedded pipe and plate configuration ceased taking on additional load at 250 kip-ft after the concrete breakout failure due to shear applied parallel to the edge resulting from the applied torsion. The cylinder tests indicated that the compressive strength of concrete on the day of testing was 5550 psi. When the predictions with the 28-day concrete strength were made, the concrete breakout predicted 250 kip-ft and the side-face blowout method predicted 392 kip-ft. The applied torsion versus plate rotation plot also shows a change in slope when the specimen experienced a redistribution of load due to bolt slippage, formation of various cracks, and widening of cracks. See Figure 5-6 for the graph of applied torsion versus plate rotation. A comparison between the experimental loading and the predicted strength can be accomplished by comparing Table 4-1and Figure 5-6.

LVDT information was gathered at the front base plate, the face of the shaft, and the rear of the shaft. As shown in Figure 5-7, the base plate rotated significantly more than the face of the shaft. This can be attributed to the fact that bolt slippage occurred, resulting in approximately 1/4” additional rotation, which can contribute approximately 1.25° of additional rotation for the base plate at failure. The rear of the shaft was designed to be a fixed support and proved to be so until failure occurred and the entire shaft rotated.

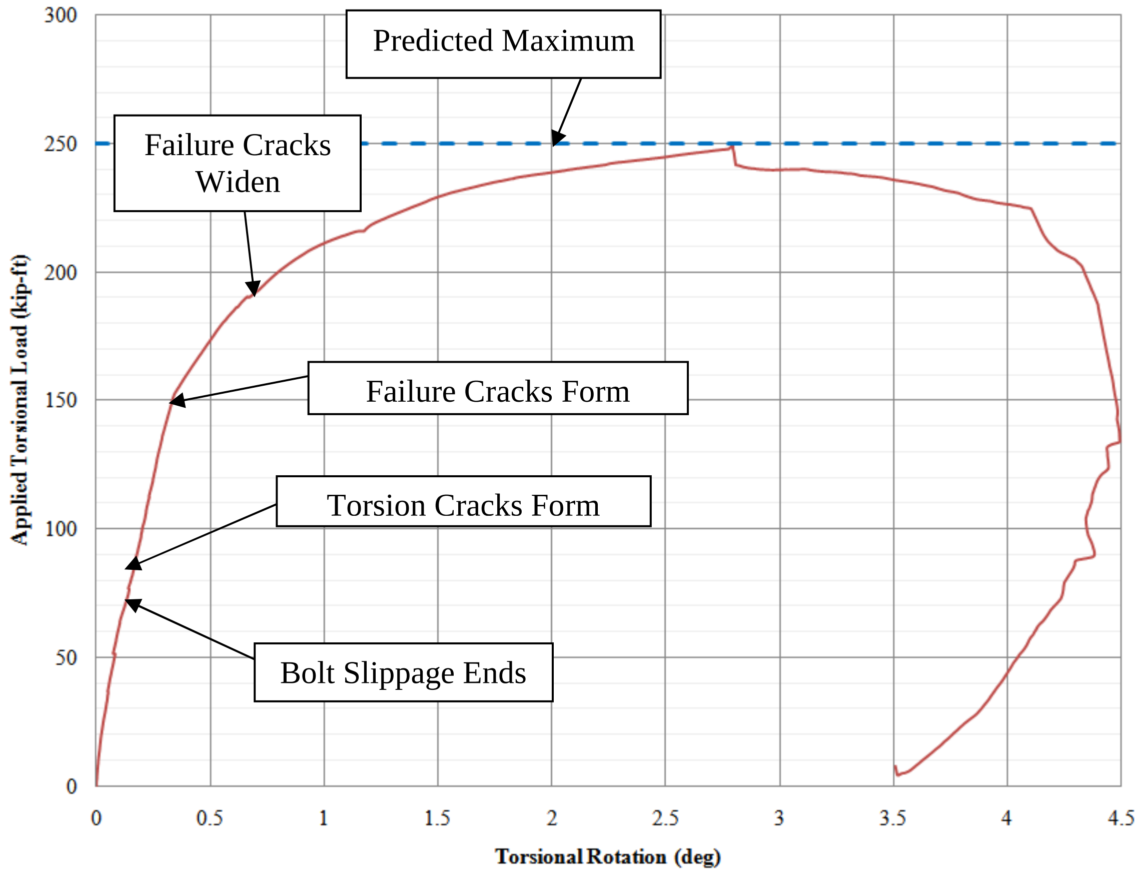


Figure 5-6. Torsional moment and rotation plot for base plate of torsion test

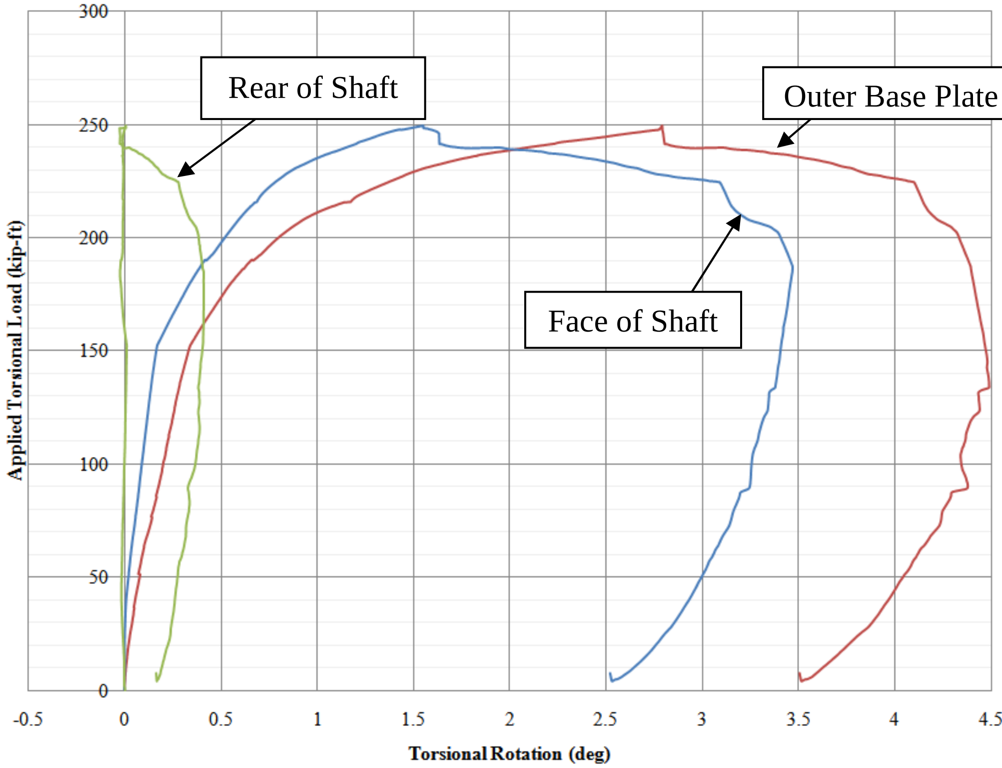


Figure 5-7. Torsional moment and rotation plot for torsion test

5.1.3 Summary of Torsion Test

The alternative support structure proved effective at transferring torsional load during the initial testing. It was determined that the modified anchor breakout equations accurately predicted the behavior and strength of the failure within 0.16% error. See Table 4-1 and Equations 3-5, 3-6, and 3-7 for the predicted strength that the experimental results verified and the equations that derived the predicted strength. The test specimen had a cone shaped blowout failure within the foundation at the approximate location of the torsional plates. It was also determined that the alternative tested had approximately twice the strength of the calculated strength of an equivalent anchor bolt system (See Appendix B). For more details on the calculated strength of an equivalent anchor bolt system compared to the test apparatus' strength, see the test apparatus calculations in Appendix B.

5.2 Torsion and Flexure Test

5.2.1 Behavior of Specimen During Testing

The second test comprising of both flexural and torsional loading was conducted on January 6, 2010 at the Florida Department of Transportation Structures Research Center. There were concerns with bolt slippage due to both the flexural and torsional moment arm connections. Prior to testing, the system was loaded with the crane only to remove some of the initial rotation due to bolt slippage (See Figure 5-8). During testing, the test specimen was loaded at approximately 100 pounds force per second and the formation of cracks on the surface of the concrete was monitored. At approximately 10.8 kips, bond between the concrete and the embedded pipe loosened, causing a change in stiffness. At approximately 14.3 kips, flexural and torsional cracks began to form on the concrete shaft (See Figure 5-9). At approximately 20.2 kips, concrete breakout failure cracks began to form on the concrete shaft while the torsional cracks continued to widen (See Figure 5-10). At approximately 24.5 kips, the concrete breakout failure cracks began to widen noticeably (See Figure 5-11). The foundation continued to be loaded until the specimen stopped taking on additional load. The applied load peaked at approximately 26.3 kips. At failure the foundation displayed the predicted breakout cone indicated by bulging concrete deep within the foundation. As intended, the rest of the test specimen did not fail before the predicted breakout failure occurred. Note that an applied load of 1 kip produces a flexural moment of 8 kip-ft and a torsional moment of 9 kip-ft.

The formation of the initial cracks was noteworthy because it indicated a change in the concrete behavior from a concrete pedestal with anchor bolts and confining reinforcement. rather than the 45 degree torsional cracks forming at the surface of the concrete closest to the base plate, cracks parallel to the embedded pipe formed at the surface closest to the base plate. These parallel cracks extended several inches down the foundation and then began to exhibit typical torsional 45 degree crack formation. This cracking behavior shows that the torsional load is being transferred from the steel to the concrete deeper in the foundation. This will be beneficial because the frequent construction mistake of placing the rebar cage too deep in the foundation often leaves the surface of the concrete under reinforced. If the load will be transferred into the concrete deeper in the foundation, the problem of the under reinforced surface concrete will be partially negated.

Figure 5-8. Test specimen prior to testing

Figure 5-9. Torsional and flexural cracks forming

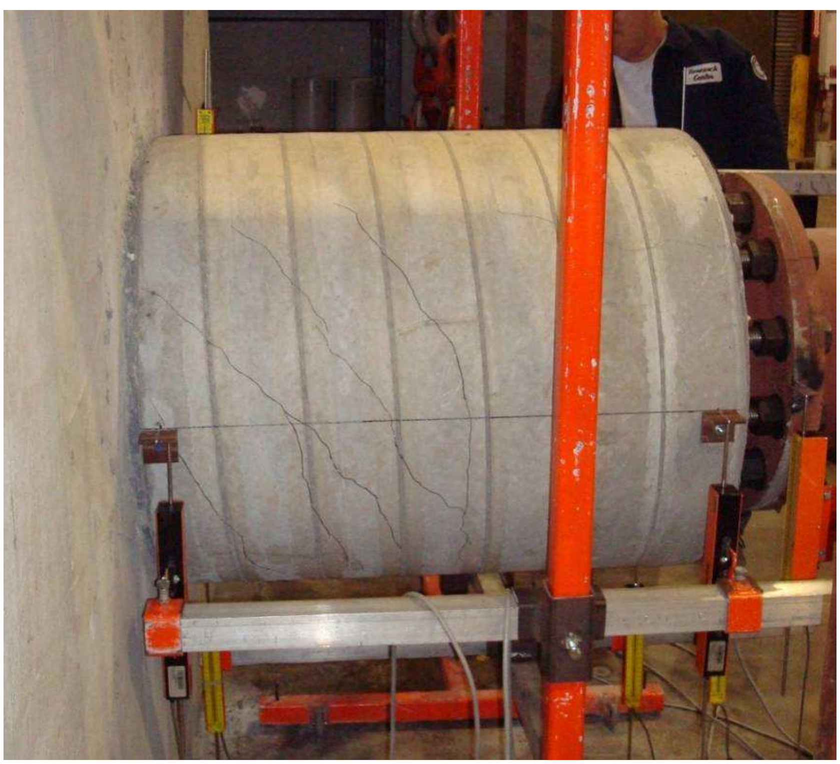


Figure 5-10. Formation of concrete breakout failure cracks in second test

Figure 5-11. Widening of concrete breakout failure cracks in second test

5.2.2 Summary of LVDT Test Results

Data was reduced to formulate an applied load versus rotation plot for both flexure and torsion. The plots show that the embedded pipe and plate configuration ceased taking on additional load after 26.3 kips after the concrete breakout failure from flexure resulting from the applied bending moment. The cylinder tests indicated that the compressive strength of concrete on the day of testing was 5180 psi. When the predictions with the 28 day concrete strength were made, the flexural concrete breakout was predicted to be 26.4 kips and the torsional concrete breakout was predicted to be 37.5 kips. When the predictions with the 28 day concrete strength were made, the flexural side-face blowout strength was predicted to be 40.9 kips and the torsional side-face blowout strength was predicted to be 56.4 kips. The applied load versus torsional plate rotation plot also shows a change in slope when the specimen experienced a redistribution of load due to bolt slippage, bond changes, formation of various cracks, and widening of cracks. See Figure 5-12for the graph of applied load versus torsional plate rotation.

See Figure 5-12 for the graph of the applied load versus flexural rotation. gathered from LVDT's place on the base plate of the embedded pipe. The graph showing the torsional rotation of the base plate for this test (Figure 5-12) shows significantly less rotation than the plate of the previous test (Figure 5-6). This can be attributed to the fact that the LVDT was placed on the base plate attached to the moment arm on the previous test and the LVDT was placed on the base plate attached to the embedded pipe on this test. The moment arm base plate would feel more rotation because of the bolt slippage occurring at the connection.

The graph of load versus flexural rotation was gathered from the LVDT's placed on the bottom of the base plate, front of shaft, and rear of shaft. The graph shows that the rotation between the face of the shaft and the base plate was significantly greater than the rotation between the face of the shaft and the rear of the shaft (See Figure 5-13). This can be attributed to several things, including the steel pipe and base connection was less stiff than the concrete pedestal as well as the concrete block was adequately designed as a fixed support, which would have restrained the rotation at the base and created a deflection that could be adequately described by an applied moment on a fixed cantilever.

As stated earlier, the LVDT's gathered information from the base plate, the front of the concrete pedestal and the rear of the concrete pedestal. The base plate's torsional rotation exceeded the rotations from the front of the concrete pedestal and the back of the concrete pedestal (See Figure 5-14). This shows that the steel pipe and base plate was less stiff than the concrete pedestal. The lack of considerable rotation in the rear of the concrete pedestal once again shows that the concrete block connected to the concrete pedestal was adequately designed as a fixed support.

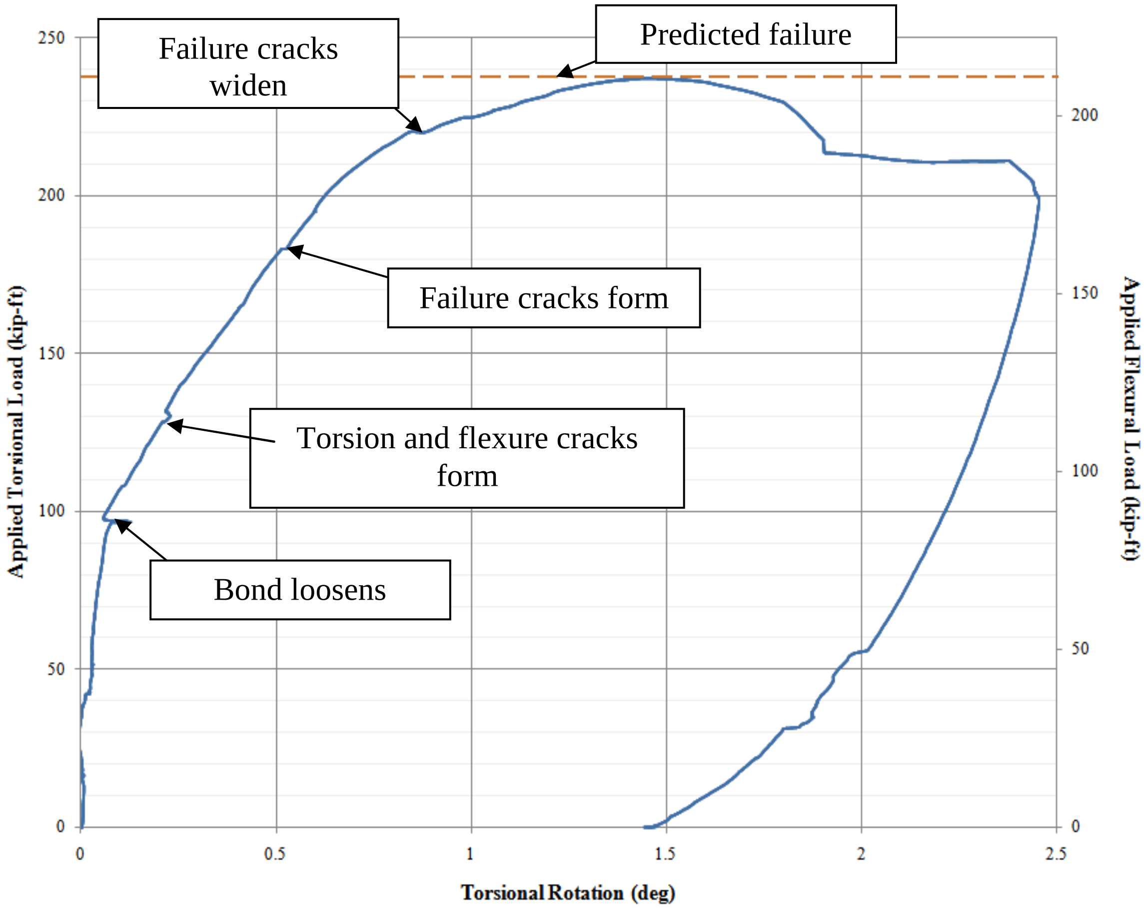


Figure 5-12. Load and torsional rotation of base plate for torsion and flexure test

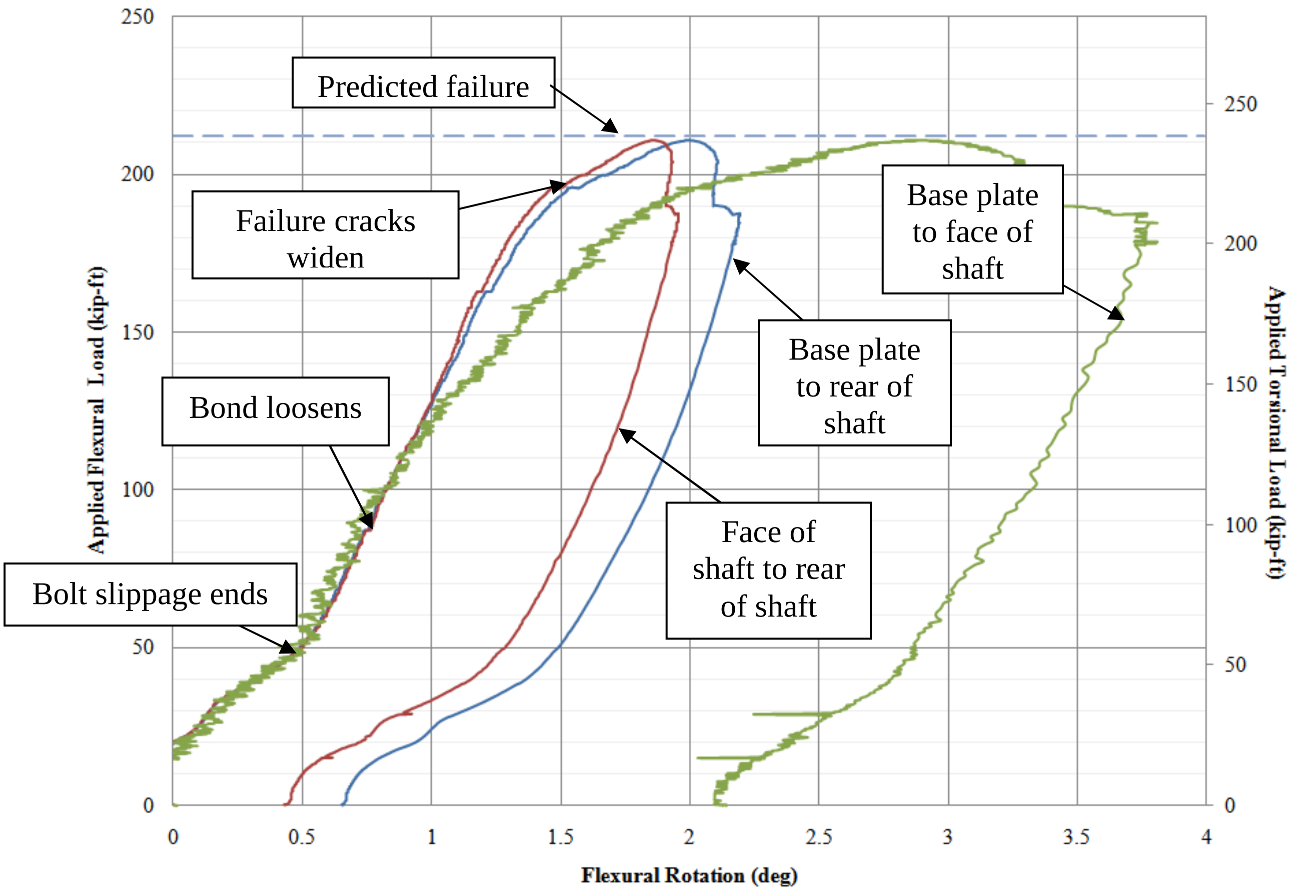


Figure 5-13. Load and flexural rotation for the second test

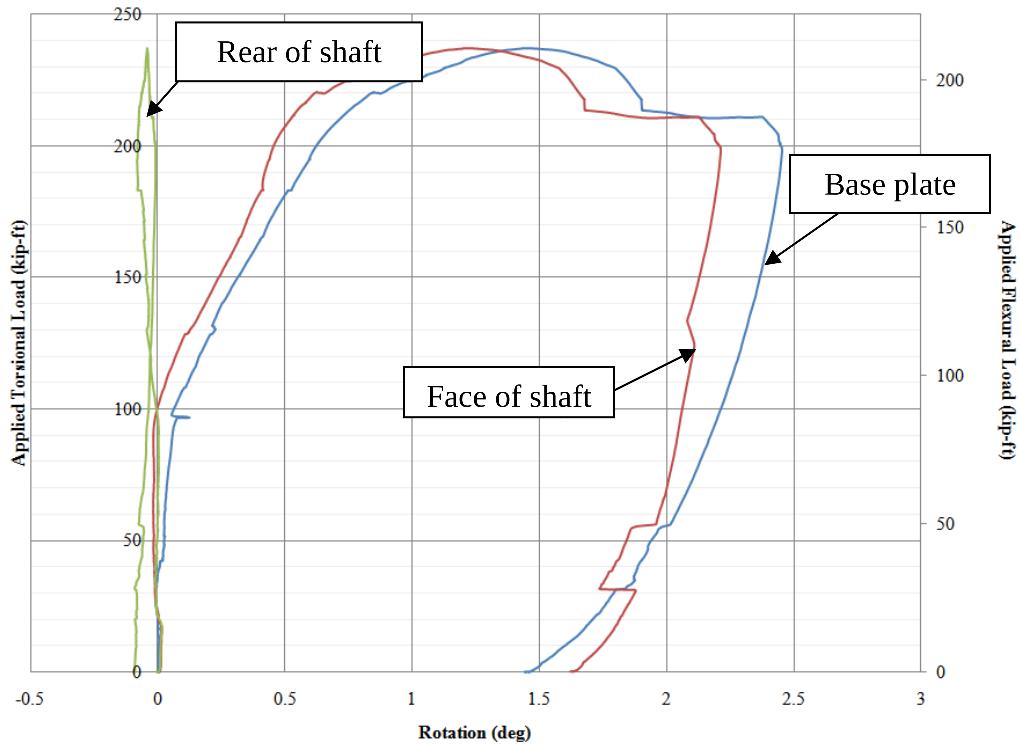


Figure 5-14. Load and torsional rotation for test specimen for the second test

5.2.3 Summary of Torsion and Flexure Test

Overall, this test proved the embedded pipe and plates section was successful at transferring load from the superstructure to the substructure. It was determined that the modified anchor breakout equations for flexure also accurately predicted the behavior and strength of the failure (See Equations 3-11, 3-12, and 3-13). The predicted failure load for the concrete breakout in flexure was 26.4 kips (See Table 4-2) and the applied failure load was 26.3 kips, with the largest breakout occurring on the bottom of the test specimen, indicating a flexure failure. The test specimen had a breakout failure deep within the foundation at the approximate location of the flexural plate. It was also determined that the alternative tested had approximately twice the strength of the calculated strength of an equivalent 12 anchor bolt system. For more details on the calculated strength of an equivalent anchor bolt system compared to the test apparatus' strength, see the test apparatus calculations in Appendix B.

CHAPTER 6
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The purpose of this research program was to determine a suitable alternative support structure for cantilever sign/signal structures and test the selected alternative to verify its viability. After a review of the problems with the current anchor bolt design and research into alternatives found in other fields, an embedded pipe and plate configuration was selected for testing. In order to quantify the strength of the embedded pipe and plate configuration, a review of current ACI 318 formulas relating to anchorage to concrete was conducted. The applicable equations regarding anchor breakout due to shear applied parallel to an edge as well as side-face blowout due to an anchor in tension were modified to accommodate the differences in geometry and behavior of an anchor and the embedded pipe and plate system. Once the predicted strength in torsion and flexure was quantified, testing was conducted on two different specimens. The purpose of the first experiment was to test primarily torsion, and the second experiment tested both torsion and flexure. The first test proved that the alternative selected was a viable alternative to transfer torsional load from the monopole to the foundation.

6.1 Implications of Test Results

6.1.1 Torsion Test

The implication of the torsion test is that the alternative selected is a viable alternative for transferring torsion from the monopole to the foundation. A comparison of the torsion test results and the calculated strength of an equivalent anchor bolt system in torsion show that the embedded pipe and plate configuration has double the strength of the equivalent anchor bolt system (See design calculations in Appendix B). The predicted breakout pattern of a failure cone within the foundation at the approximate location of the torsion plates was exhibited during testing, signifying that the predicted behavior was likely correct. The modified concrete breakout equations for torsion (See Equations 3-5 and 3-6) were proven accurate as the predicted failure load with these equations was less than 1% disparate from the tested failure load.

The results imply that the embedded pipe and plate configuration in torsion alone would be an adequate alternative to the current anchor bolt system. The torsional strength of the alternative is greater than the anchor bolt system and can be accurately predicted using the modified concrete breakout equations for torsion.

6.1.2 Torsion and Flexure Test

The implication of the torsion and flexure test is that the alternative selected, the embedded pipe with torsion and flexure plates, is a suitable alternative to the current design using anchor bolts. A comparison of the experimental test values and the calculated equivalent strength of an anchor bolt setup show that the experimental test strength in flexure is approximately twice that of the equivalent anchor bolt system (See design calculations in Appendix B). A large bulge of concrete on the bottom of the shaft signifies a concrete breakout of the embedded flexure plate, verifying the breakout was the failure mode. The modified concrete breakout equations for flexure (See Equations 3-8 and 3-9) were proven accurate as the predicted failure load with these equations was less than 1% off from the tested failure load.

These results imply that the tested system with the embedded pipe and torsion and flexure plates is a viable alternative to the current anchor bolt system. The failure can be predicted accurately using both the torsion and flexure plates and can easily be quantified using the modified concrete breakout equations.

6.2 Recommendations for Future Testing

6.2.1 Introduction and Background

Cantilever sign/signal structures typically have a single monopole supported by a cast-in-place foundation. As was mentioned in Chapter 2, the most common method of connecting the monopole to the foundation is through the use of anchor bolts attached to an annular plate welded to the monopole (See Figure 6-1). Although this connection is the most widely used, many studies in the past few years have reported that fatigue of the annular plate and anchor bolt configuration is a significant concern.

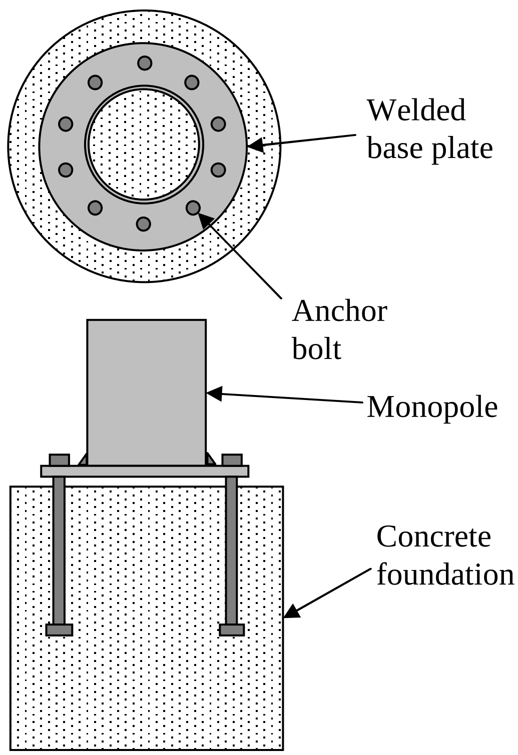


Figure 6-1. Typical sign/signal base connection

In the early 1990s it became evident that the *Supports Specifications* were not providing enough guidance on designing for vibration and fatigue. In response to the large problems with vibration and fatigue in cantilever signal/sign support structures, the National Cooperative Highway Research Program (NCHRP) initiated project 10-38 in 1993 (6). The information obtained from project 10-38 was published as NCHRP Report 412. The recommendations provided in NCHRP Report 412 were incorporated into the design provisions in the 2001 *Supports Specifications*.

NCHRP Report 412 found that galloping, vortex shedding, natural wind gusts, and truck-induced wind gusts were the primary wind-loading mechanisms that were responsible for most vibration and fatigue-related stresses on cantilever structures (7). Based on this information, importance factors were assigned for each of the four wind-loading mechanisms on three fatigue categories. Report 412 describes, “Structures classified as Category I would present a high hazard in the event of failure and should be designed to resist rarely occurring wind loading and vibration phenomena” (7).

The fatigue design approach recommended by NCHRP Report 412, and adopted by the 2001 *Supports Specifications*, was to design cantilever support structures to resist specified static wind loads, modified by the importance factors (3). The stresses obtained from the modified static wind loads would be designed to satisfy the requirements of their recommended detail categories for an infinite life fatigue design (3).

Due to the lack of proper guidance on vibration and fatigue design in the *Supports Specifications* until the 2001 edition, many of the supports structures designed prior to the 2001 edition are now experiencing fatigue problems, particularly on the welded annular base plate and anchor bolt connection (3).

Despite the fact that NCHRP Report 412 finally gave guidance to designers on fatigue design for cantilever signal/sign support structures, the rate of fatigue cracking and failure has continued and may have even increased (6). Because of this, NCHRP Project 10-38(2) was initiated to further address fatigue-resistant design of the cantilever support structures. The information obtained from Project 10-38(2) was published as NCHRP Report 469. NCHRP

Report 469 partially attributes the continued fatigue problems with the increasing use of longer horizontal spans of the cantilever sign/signal structures (6). Past inspections have shown that the following typical and special problems on cantilever signal/sign structures are prevalent (16):

- Cracked anchor bolts both above and within the concrete
- Loose nuts and missing connectors, both on anchor bolts and structural bolts
- Cracked and broken welds
- Split tubes
- Plugged drain holes, debris accumulation and corrosion
- Internal corrosion of tubular members
- Poor fit-up of flanged connections with cracking and missing bolts
- Structure overload due to installation of signs exceeding design square footage

Some of the recommended revisions proposed in NCHRP Report 469 to the 2001 *Supports Specifications* fatigue design and partially incorporated into the *2006 Interim to Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* include the following (6; 17):

- Clearly define criteria for categorizing the structure fatigue categories

- Galloping mitigation devices (sign blanks or other proved mitigation devices) not be used to remove the galloping design load entirely, but would instead alter the fatigue category from Category I to Category II

- The equivalent static pressure range be changed from $1760C_D$ to $900C_D$ for truck-induced wind gusts

- A statement be included in the vortex-shedding section, similar to that in the galloping section of the 2001 Supports Specifications, allowing for mitigation of vibration due to vortex shedding after a problem with vibration in double-curvature has been observed

- Minor changes to the design some of the fatigue-resistant details, with the inclusion of an additional fatigue-resistant detail to be considered

The problems identified with the fatigue of the steel annular base plate and the concrete breakout from the anchor bolts necessitates looking at alternatives to the current anchor bolt and base plate connection. The following are some options to explore regarding alternative connections that do not use the same anchor bolt and annular base plate connection.

6.2.2 Tapered Embedded Steel Pipe and Plate Option with Bolted Slip Base Connection

In this option, a tapered welded pipe and plate configuration will be embedded into the foundation with a portion of the pipe projecting from the foundation. The monopole will be placed over the projecting pipe, acting as a sleeve, and secured into place by several bolts that will extend through the diameter of the pole. See Figure 6-2 for a sketch of this connection.

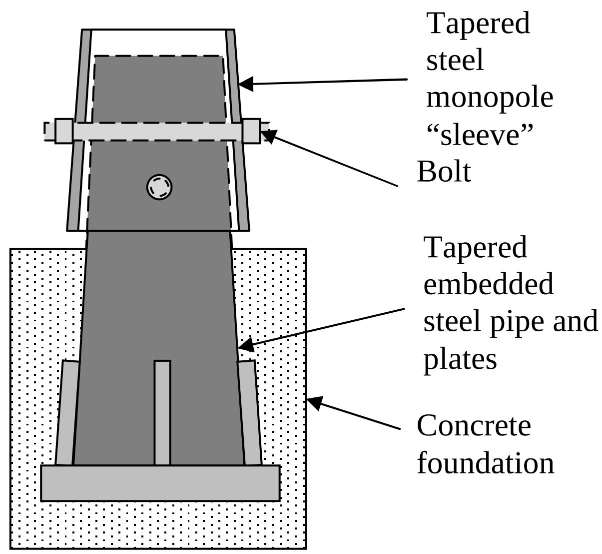


Figure 6-2. Embedded steel pipe and plate option with slip base connection

The primary benefit associated with this connection is that the annular plate and anchor bolts have been removed, thus eliminating the questionable connecting elements of the design. The design calculations for the bolted connection would be relatively easy. The bolts would need to be designed for shear strength and the bearing strength of the bolt holes would also be a primary consideration. The embedded pipe and plate section has been tested to determine its torsional and flexural viability. Since the embedded pipe and plate alternative has been proven effective at transferring load, this connection would seem a likely candidate for consideration.

However, one of the drawbacks to this design is the construction feasibility. A typical monopole's taper is 0.14 in/ft. In order to provide the shorter embedded tapered section, an additional pole would need to be ordered and cut to the appropriate length at the appropriate point on the pole. This process may prove tedious and time consuming. The connecting bolts bearing on the monopole may require an increase in pipe thickness for the monopole which could lead to additional expense. Additionally, this option would include corrosion as a potential problem since the entire connection is steel.

Alignment of this connection may be difficult to accomplish during construction. One method possible to control the alignment would be to place the sleeve flush with the top of the concrete foundation. However, if a standoff was required, there might be difficulty leveling the monopole for placement. The bolt holes will ensure the final product will be level because they need to be aligned properly to ensure the bolts will fit through the holes. If a bolt is forced into place because of improper alignment it may incur additional stress.

Design strength considerations for this connection include, but are not limited to, the following:

- Bolt shear strength
- Bolt bearing strength (on steel pipes)
- Fatigue (of bolts)
- Breakout strength (of embedded section on concrete foundation)
- Torsional strength
- Flexural strength

This option provides a suitable alternative to the current annular plate and anchor bolt connection. The FDOT currently uses a detail similar to this in Index No. 11860, Single Column Ground Signs, in their Design Standards (18). See Figure 6-3 for a sketch of the FDOT detail. However, this detail has been specified for use with aluminum single column posts for ground signs and not for steel monopoles.

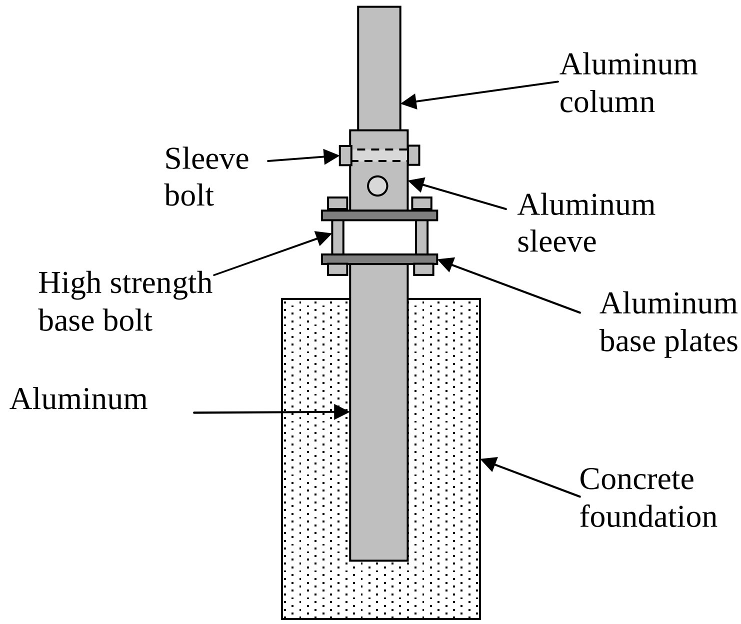


Figure 6-3. FDOT *Design Standards* Index No. 11860[18]

6.2.3 Embedded Steel Pipe and Plate Option with Grouted Slip Base Connection

In this option, a standard welded pipe and plate configuration will be embedded into the foundation with a portion of the pipe projecting from the foundation. The monopole will be placed over the projecting pipe, acting as a sleeve, and secured into place by several bolts that will extend through the diameter of the pole. The gap between the tapered monopole and the embedded pipe's projection will be filled with high-strength grout. See Figure 6-4 for a sketch of this connection.

As with the tapered embedded steel pipe and plate option, the primary benefit associated with this connection is that the annular plate and anchor bolts have been removed. The design calculations for the bolted connection would be relatively easy. The bolts would need to be designed for shear strength and the bearing strength of the bolt holes would also be a primary consideration. A benefit of this design over the tapered steel pipe design would be that the embedded steel pipe would be more easily obtained.

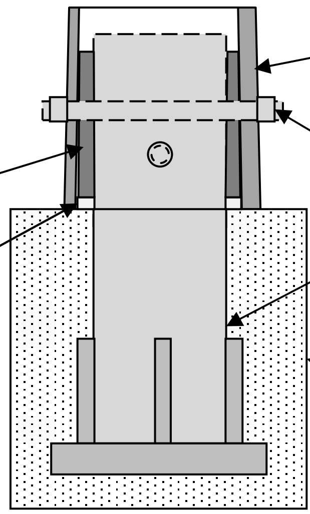


Figure 6-4. Embedded steel pipe and plate option with grouted slip base connection

One of the drawbacks to this design is the added complication of high-strength grout. Grout was found to be improperly placed in the current anchor bolt and base plate connection and has the potential to be improperly placed in this connection. Another potential drawback is that the connecting bolts bearing on the monopole may require an increase in pipe thickness for the monopole which could lead to additional expense. Additionally, this option would include corrosion as a potential problem since the entire connection is steel.

Alignment of this option during construction may prove difficult because of the small tolerance for error on aligning the bolt holes. Allowing the monopole to be placed directly on the concrete foundation will reduce some error.

Design strength considerations for this connection include, but are not limited to, the following:

- Bolt shear strength
- Bolt bearing strength (on steel pipes)
- Fatigue (of bolts)
- Breakout strength (of embedded section on concrete foundation)
- Torsional strength
- Flexural strength

This option provides a possible alternative to the current annular plate and anchor bolt connection. The FDOT currently uses a detail similar to this in Index No. 11860 in their Design Standards (See Figure 6-3). However, this detail has only been used with aluminum single column posts for ground signs.

6.2.4 Embedded Concrete Pipe with Bolts Option with Bolted Slip Base Connection

In this option, a prestressed concrete pipe with bolts option, either tapered or not tapered, will be embedded into the foundation with a portion of the pipe extending beyond the foundation. This option is very similar to the embedded steel pipe and plate option with slip base connection. One obvious difference would be that the embedded pipe would be concrete rather than steel. Another difference is that the embedded portion would have bolts acting in a manner similar to the plates. The bolts would connect plates to the concrete section. As explained later in this section, the embedded concrete pipe with bolts may be replaced with a geometric section without bolts if necessary. See Figure 6-5 for the overall setup of this connection as a concrete pipe with bolts.

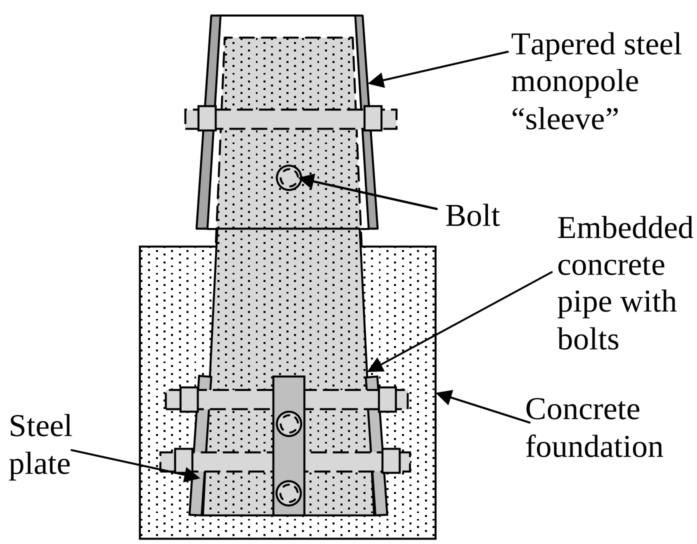


Figure 6-5. Embedded concrete pipe and plate option with slip base connection

One immediate benefit associated with this configuration is that the annular plate and anchor bolt connection has been removed. Another benefit over the embedded steel pipe and plate option is that this embedded concrete option removes corrosion of the embedded pipe as a potential problem. As with the previous option, the bolted connection bearing on the monopole may require an increase in thickness for the monopole, leading to additional expense.

A potentially difficult piece to construct would be the embedded concrete pipe with bolts. One option would be to order spun concrete poles from a manufacturer. The poles would include prestressed strands as well as spiral reinforcement and would be light and durable. The through bolt holes would be included by using a cast-in-place PVC pipe during fabrication. Another option would be to use a geometric section without bolts instead of the round section with bolts. The geometric section would provide the required torsional resistance once embedded in the foundation that the bolts are providing in the round section.

Design strength considerations for this connection include, but are not limited to, the following:

- Bolt shear strength
- Bolt bearing strength (on steel monopole)
- Bolt bearing strength (on embedded concrete section)
- Fatigue (of bolts)
- Breakout strength (of embedded section on concrete foundation)
- Torsional strength
- Flexural strength

As with the previous option, the embedded steel pipe and plate option, this configuration may provide a suitable alternative to the current annular base plate and anchor bolt connection. As mentioned before, the FDOT currently uses a detail similar to this in Index No. 11860 in their Design Standards. Given that, the detail in the Design Standards has only been specified for use with aluminum single column posts for ground signs.

6.2.5 Cast-in-Place Solid Concrete Pedestal with Bolted Slip Base Connection

In this option, a cast-in-place solid concrete pedestal would be poured projecting from the foundation with the tapered steel monopole placed over the pedestal projection and the two connected with bolts. Some longitudinal rebar would connect the solid concrete pedestal projection to the foundation. See Figure 6-6 for the setup of this connection.

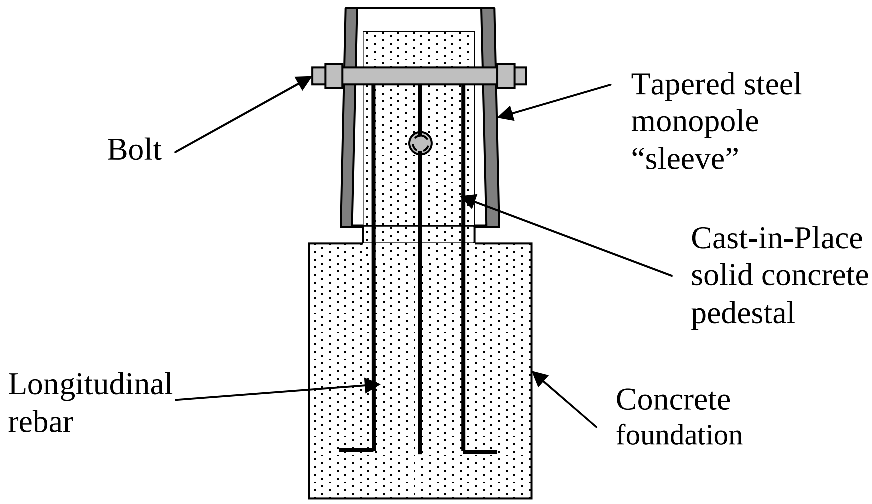


Figure 6-6. Cast-in-Place solid concrete pedestal with slip base connection

As with the previous option, one benefit to this connection would be that the annular base plate and anchor bolt connection would be eliminated. Another benefit to this connection is that the construction would be relatively easy because it’s all cast-in-place. One problem with this connection is that the connection may have less flexural strength because the rebar would be the only flexural reinforcement. And as with the other bolted slip connections, the bolt bearing may require an increase in monopole member thickness.

Design strength considerations for this connection include, but are not limited to, the following:

- Bolt shear strength
- Bolt bearing strength (on steel monopole)
- Bolt bearing strength (on cast-in-place solid concrete pedestal)
- Fatigue (of bolts)

- Torsional strength
- Flexural strength

6.2.6 Embedded Concrete Pipe with Bolts Option with Grouted Splice to Concrete Monopole

In this option, a prestressed concrete pipe with bolted plates would be embedded into a concrete foundation. The bolts and plates would resist torsion by bearing on the surrounding concrete foundation. The splice would be similar to that presented in FDOT Project BC354-80 Final Report, Volume 2 (19). See Figure 6-7 for the setup of this connection. The splice connection would be a steel HSS pipe with welded rebar hoops placed in the hollow core of the prestressed spun concrete pipe and then pressure grouted into place.

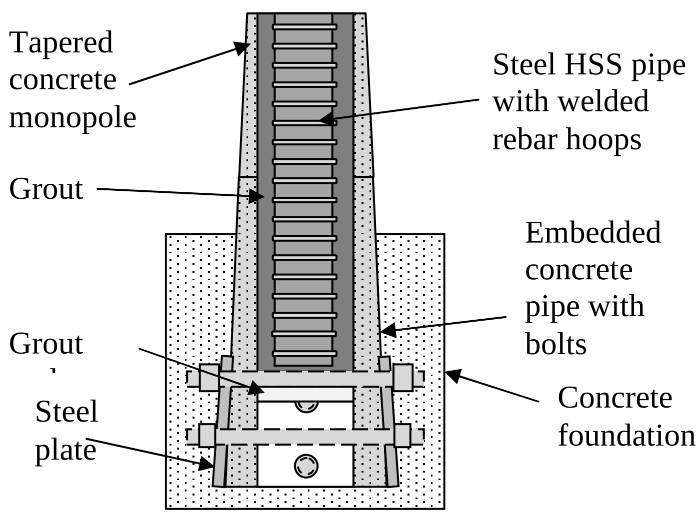


Figure 6-7. Embedded concrete pipe with bolts option with grouted splice to concrete monopole

This option has several advantages over the current annular plate and anchor bolt connection. The primary advantage is that the connection does not use annular plates or anchor bolts and will eliminate the fatigue problems associated with the current connection option. Another advantage is that since the steel portion of the connection is grouted in the core of the spun concrete poles, the steel will not suffer as much corrosion unless one of the grout inlet holes is compromised.

This option does have some disadvantages as well. The construction will be more tedious and time consuming than the current connection option. This connection can be more costly than the current connection option because of the increase in number of elements as well as the cost of each element. The inclusion of grout adds an additional complication for design and construction error. The viability of a concrete monopole for cantilever use is also questionable. The horizontal member that needs to be attached for sign/signal purposes may be too large to attach to the concrete monopole. This connection may be more difficult to monitor and repair than a visible connection.

Design strength considerations for this connection include, but are not limited to, the following:

- Breakout strength (of embedded section on concrete foundation)
- Grout strength
- Torsional strength
- Flexural strength
- Monopole to horizontal member connection

6.2.7 Embedded Steel Pipe and Hoops with Grouted Slip Base Connection

This option entails using a steel pipe and hoops embedded into the foundation and stubbing out from the foundation. The steel pipe and hoops would then be covered by a concrete monopole and pressure grouted into place. See Figure 6-8 for this connection configuration.

The primary benefit of this connection is that it removes the fatigue-prone elements of the current anchor bolt and annular base plate connection. It offers good torsional and flexural resistance with the embedded pipe and hoop section. The pipe and hoop section may be expensive to fabricate. The pressure grouting has the potential to be a problem during construction as it has been a problem in the past. As mentioned earlier, the concrete monopole

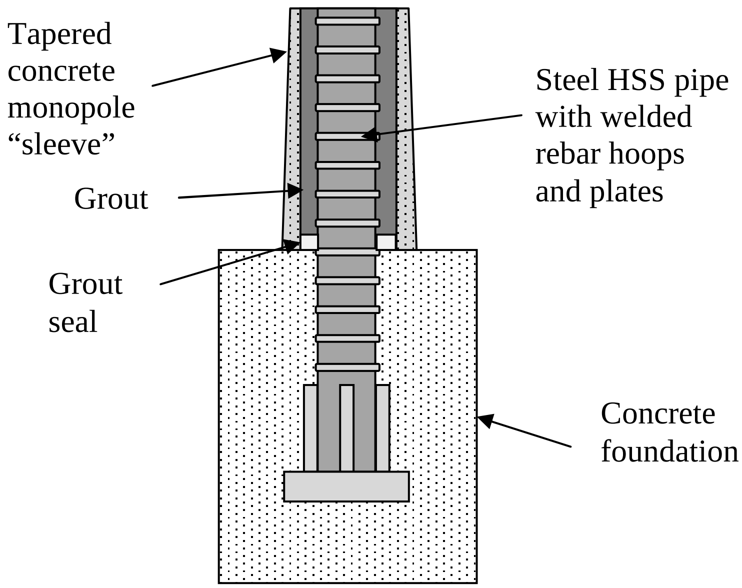


Figure 6-8. Embedded steel pipe and hoops with grouted slip base connection

may not be viable to connect to the steel horizontal member. This connection may also be more difficult to monitor or repair than a visible connection.

Design strength considerations for this connection include, but are not limited to, the following:

- Breakout strength (of embedded section on concrete foundation)
- Grout strength
- Torsional strength
- Flexural strength
- Monopole to horizontal member connection

6.2.8 Embedded Steel Pipe and Plates with Bolted Plate Connection

In this option, an annular plate would be welded to both the monopole and the stub of the embedded steel pipe protruding from the foundation. The two annular plates would be bolted together, allowing for space between for leveling nuts to be used. The leveling nuts would make it easier to ensure the monopole was erected properly. See Figure 6-9 for the connection setup.

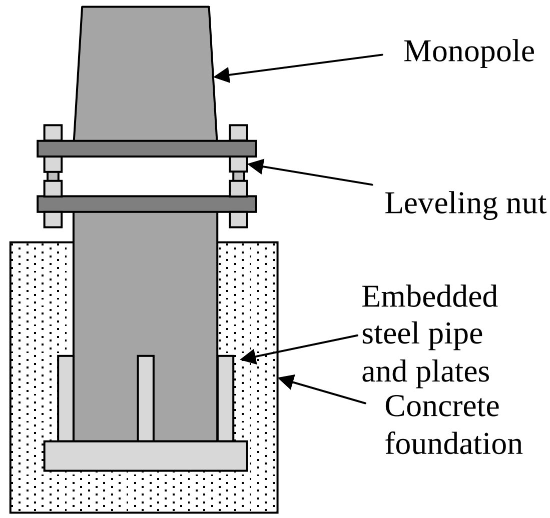


Figure 6-9. Embedded steel pipe and plates with bolted plate connection

One benefit with this connection is that it would be easy to construct and the materials would be easy to obtain. Since this option is very similar to the current base plate and anchor bolt option, it would not be difficult for designers to transition to this design. This option also provides the benefit that the embedded pipe would not need to be tapered and therefore could be more easily constructed by using a standard circular HSS section.

However, one major drawback to this design is that it has the potential to have fatigue problems similar to the current base plate and anchor bolt design. The welds, bolts, and plates could experience fatigue cracking after the cyclical wind stresses are imposed on the connection. Corrosion would also remain an issue with this connection. This connection does not necessarily fix the problem with fatigue associated with the current annular plate and anchor bolt design, but it does offer another alternative.

Design considerations for this connection include, but are not limited to the following:

- Bolt shear strength
- Bolt bearing strength (on annular plates)
- Flexural strength (of annular plates and bolts)
- Weld strength
- Axial strength (of annular plates and bolts)

- Fatigue (of bolts, welds, and annular plates)

As demonstrated by the increase in the number of design considerations, this option has more possibilities for failure. It does include welds, annular base plates, and bolts as the current base plate and anchor bolt option does. Therefore, this option does not eliminate the problems associated with the current base plate and anchor bolt design, other than removing anchor bolts as a potential failure and replacing it with a standard bolted connection.

6.2.9 Embedded Steel Pipe and Plates with Welded Sleeve Connection

In this connection, a steel pipe and plate configuration would be embedded in the concrete foundation and connected to the steel monopole by a welded sleeve. The sleeve would consist of a high strength steel pipe section fillet welded to the monopole and embedded steel pipe and plates around the perimeter of the pipes. See Figure 6-10 for the connection detail.

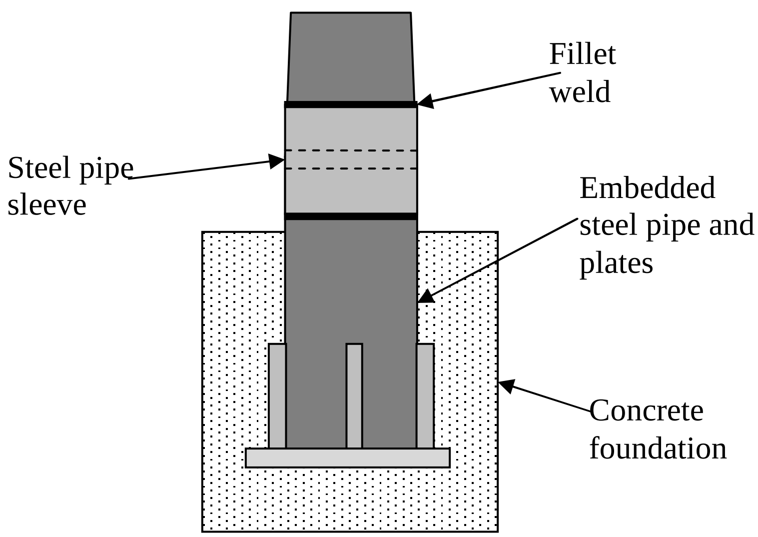


Figure 6-10. Embedded steel pipe and plate with welded sleeve connection

One benefit of this connection is that it would be relatively easy to construct and the materials would be easy to obtain. This connection does not use bolts and thus removes bolt fatigue as a problem. The annular plate is also removed, also eliminating fatigue problems with this component of a connection.

However, the fillet welds are susceptible to fatigue cracking similar to the current base plate and anchor bolt section. Corrosion would also remain an issue because the connection is comprised totally of steel. The welded sleeve's pipe thickness would need to be large to handle the large flexural and torsional moments presented at the connection. This connection may make it difficult to align the monopole correctly during construction.

Design considerations for this connection include, but are not limited to the following:

- Weld strength
- Fatigue (of welds)
- Flexural strength
- Torsional strength
- Breakout strength (of embedded section on concrete foundation)

This option provides a solution to part of the fatigue problems associated with the annular plates, bolts, and welds. The welds will remain a fatigue problem. While this option may not solve all of the problems, it does provide a solution that may be relatively easy to construct.

6.2.10 Summary of Recommendations for Future Testing

Future testing of alternative connections to resolve the fatigue and vibration problems exhibited in the current base plate and anchor bolt connection is highly recommended. The embedded pipe and plates configuration has been proven to be effective at transferring torsion and flexure and therefore a connection incorporating the embedded pipe and plates would be ideal. The option that may have the greatest potential that incorporates the embedded pipe and plates option is the grouted slip base connection. The benefits of this connection are that it includes the embedded pipe and plates, the design would be relatively simple, the anchor bolts are removed, and the fatigue prone welds that present a problem are eliminated.

6.3 Summary

The alternative selected, the embedded pipe and plate configuration, has worked in transferring torsional and flexural load from the monopole to the foundation during experimental testing. The controlling failure behavior of the system is characterized by a concrete breakout in the shape of a cone in the vicinity of the embedded plates. The strength of the failure modes can be quantified by using modified ACI 318 equations. The embedded pipe and plate configuration also has potential to work in an alternative base connection that is recommended for future testing. The proper use of the findings in this testing program will allow for future prevention of the types of failures exhibited in the 2004 hurricane season.

APPENDIX A
TEST APPARATUS DRAWINGS

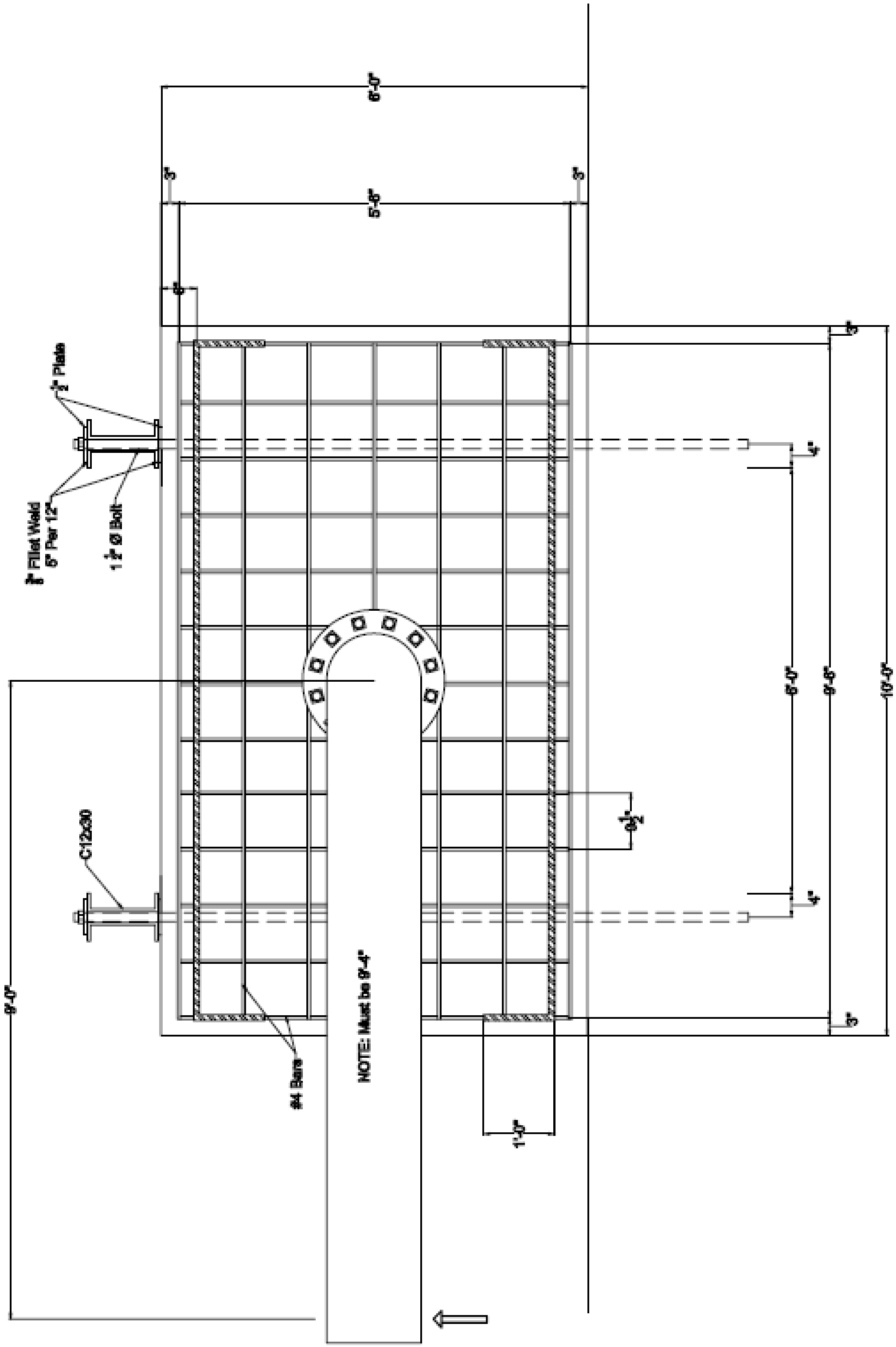


Figure A-1. Dimensioned front elevation drawing of torsion test apparatus

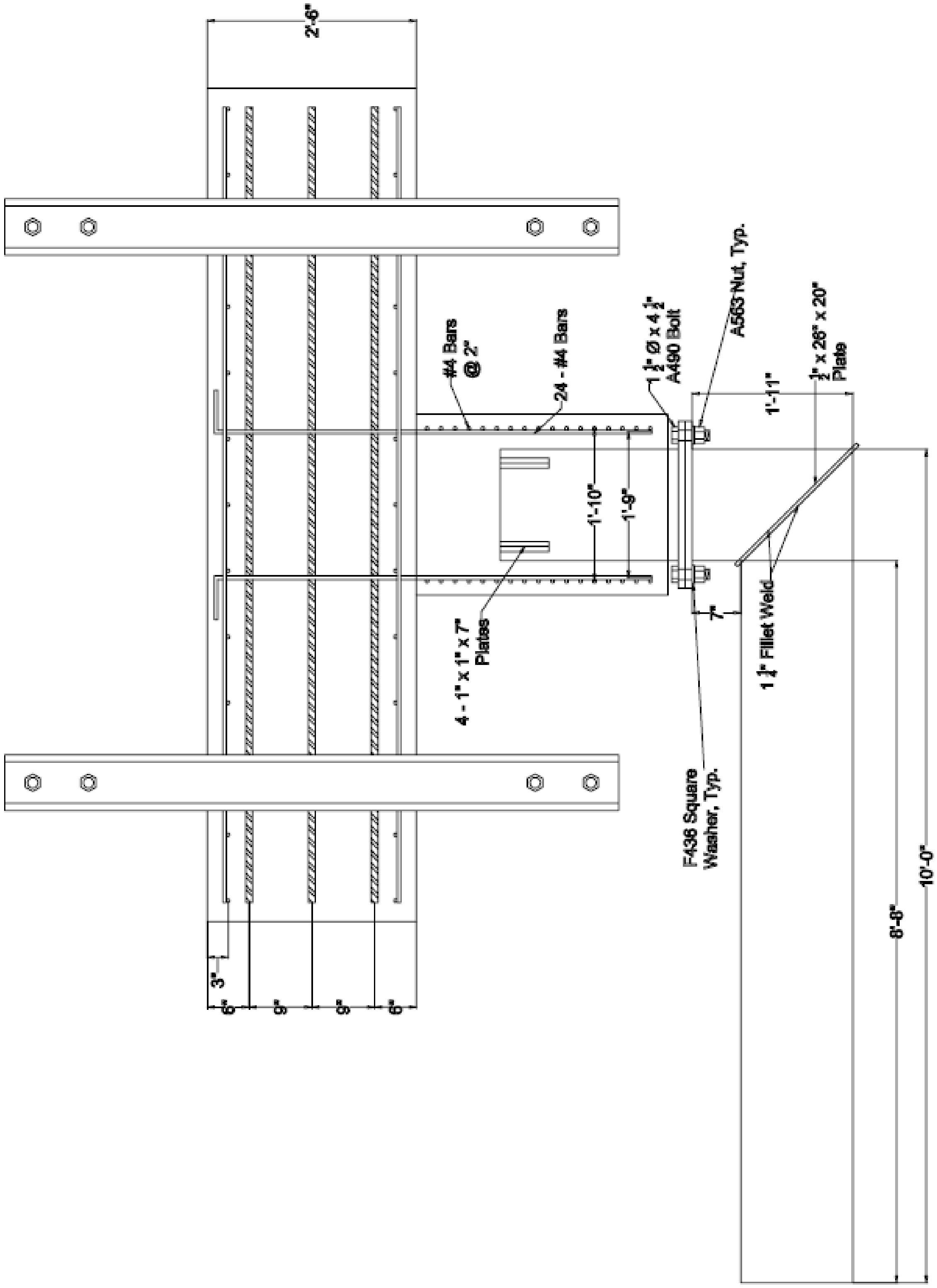


Figure A-2. Dimensioned plan view drawing of torsion test apparatus

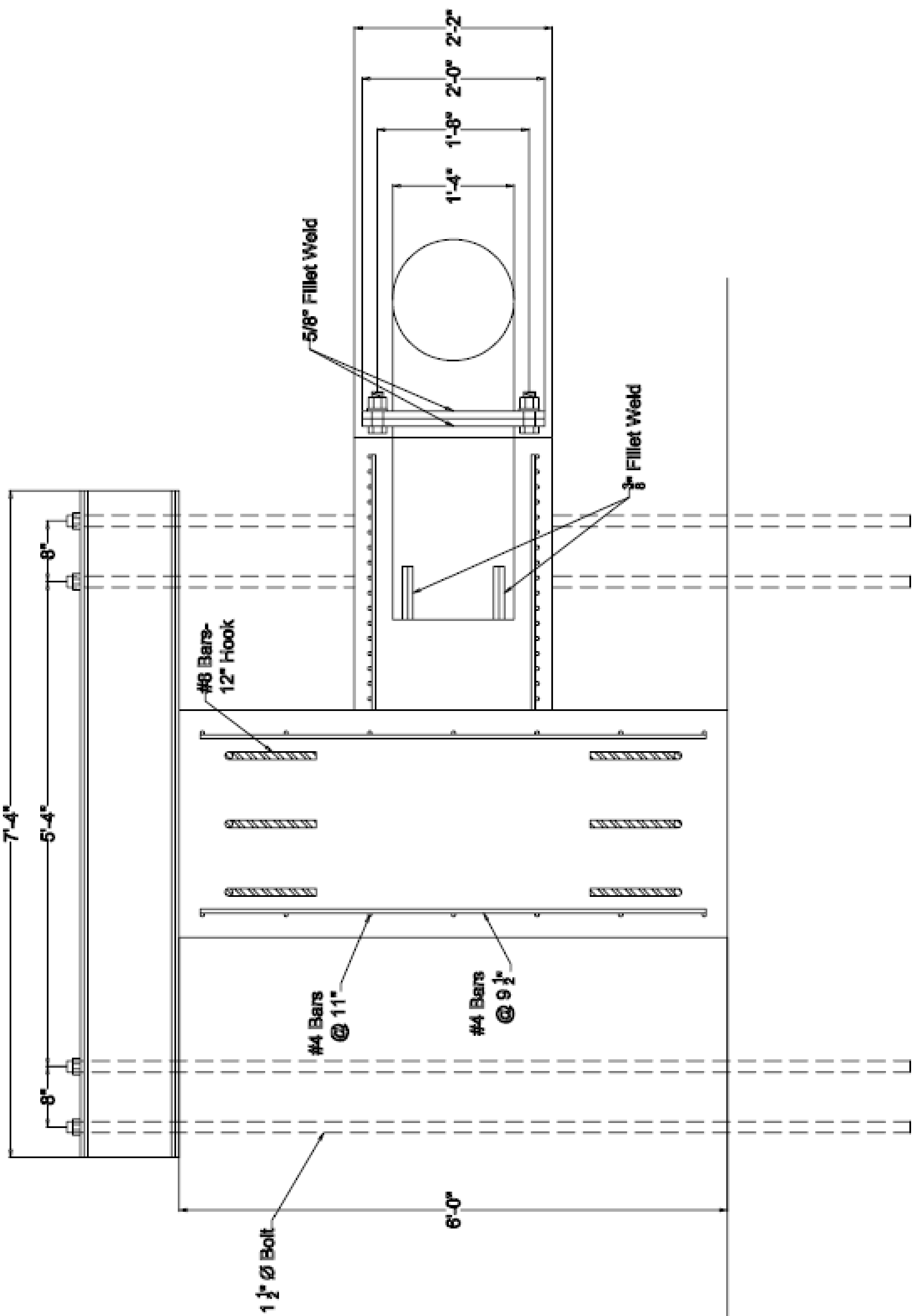


Figure A-3. Dimensioned side elevation drawing of torsion test apparatus

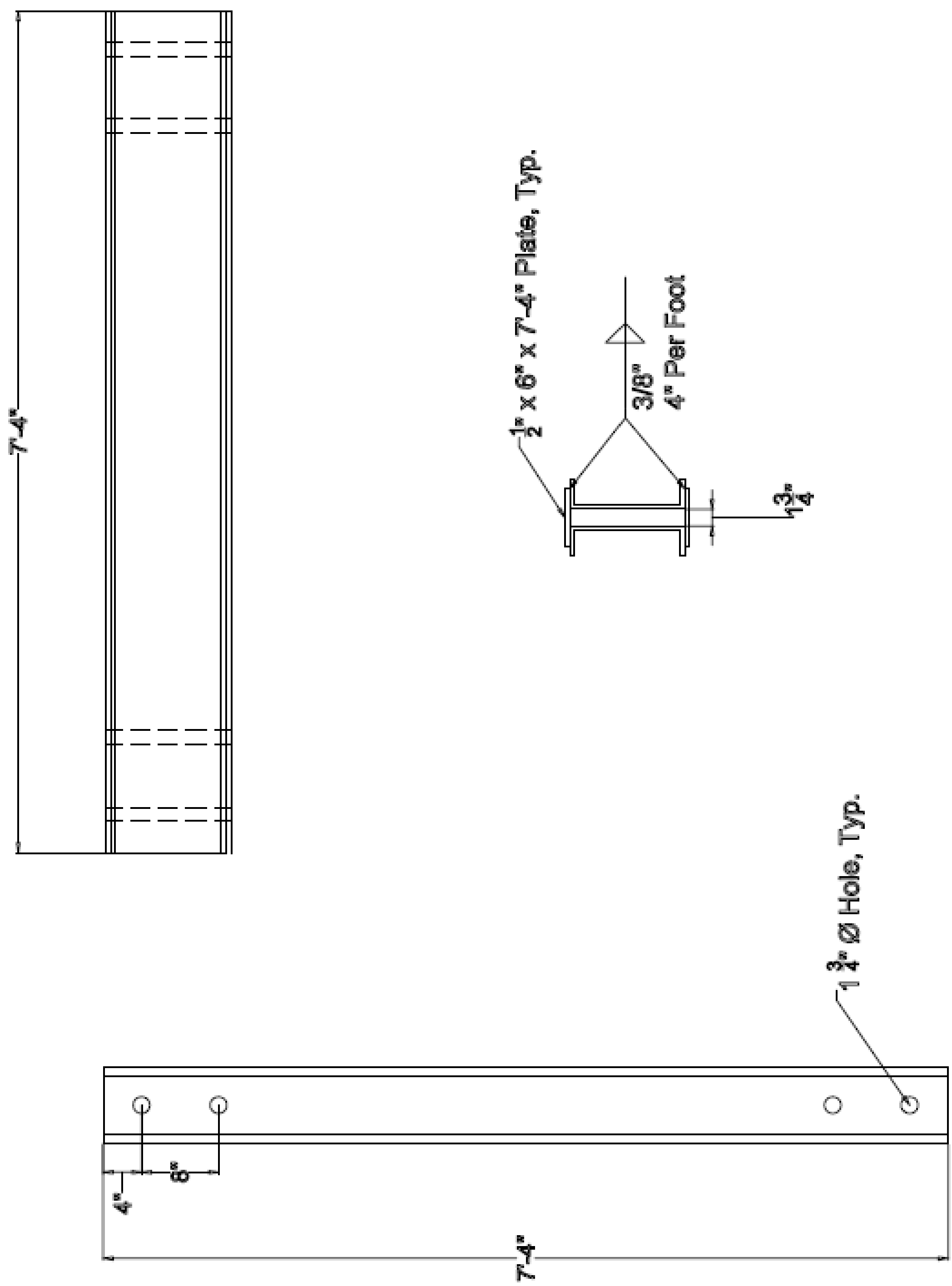


Figure A-4. Dimensioned view of channel tie-down for torsion test apparatus

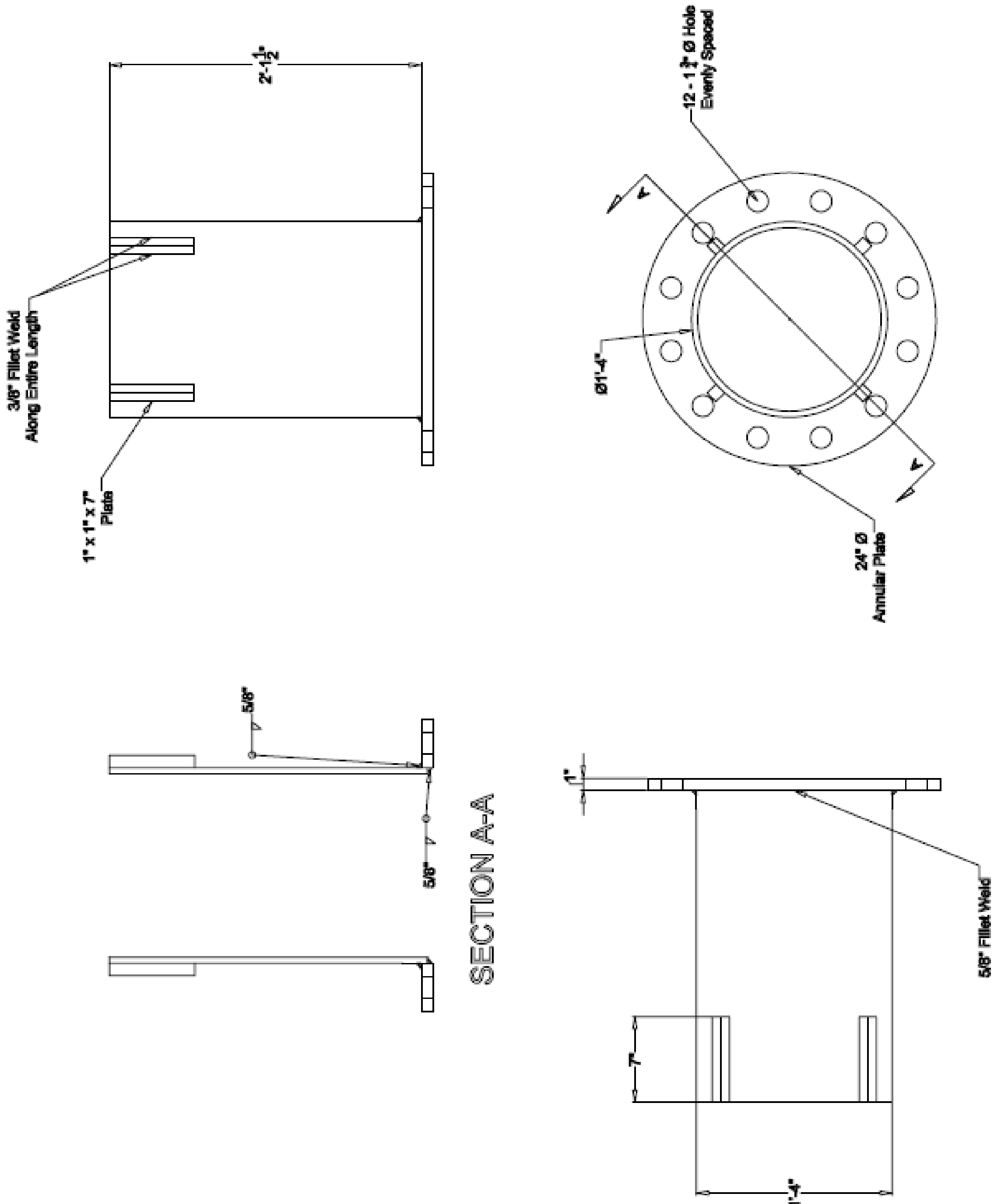


Figure A-5. Dimensioned drawings of embedded pipe and plate for torsion test

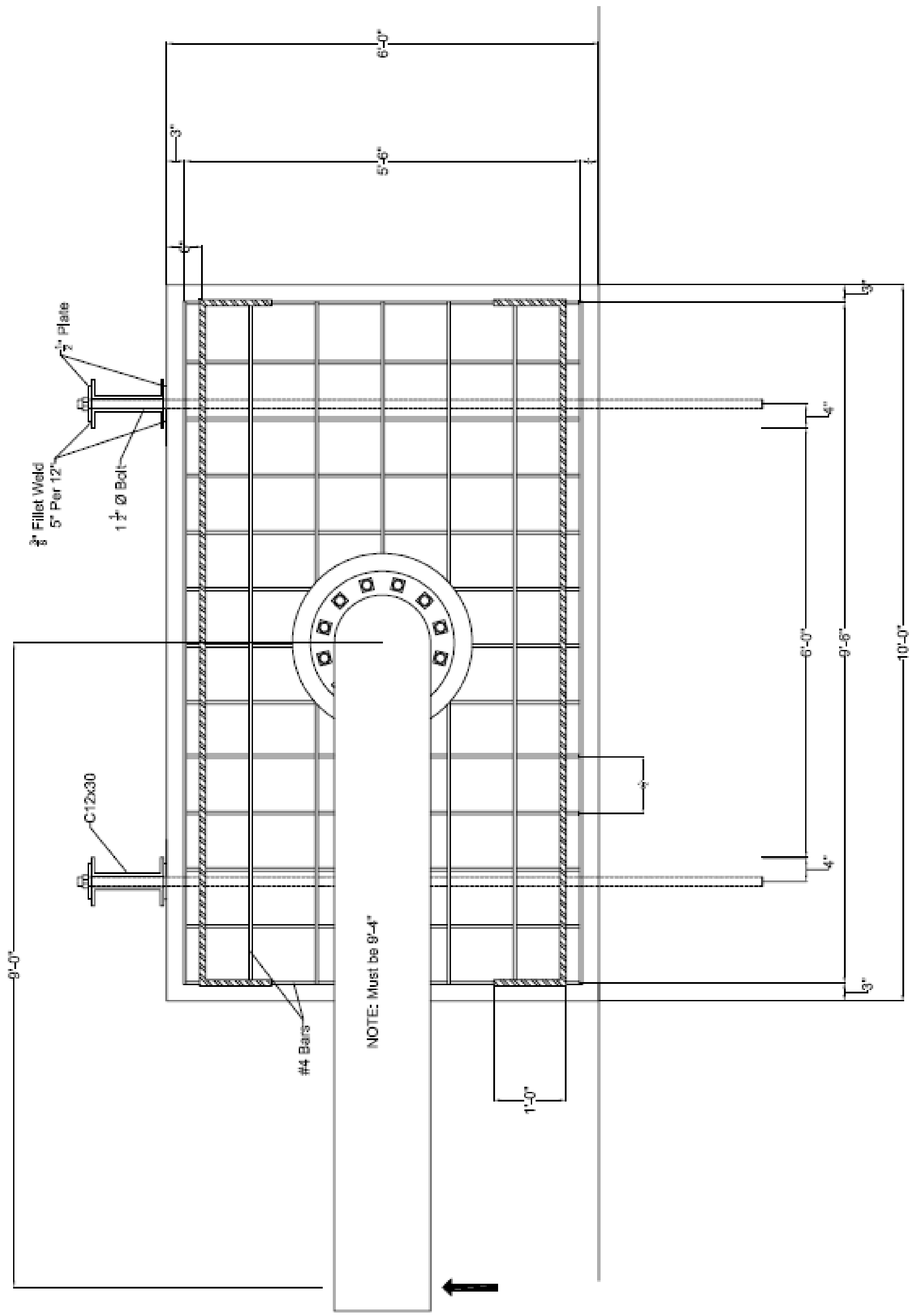


Figure A-6. Dimensioned front elevation drawing of torsion and flexure test apparatus

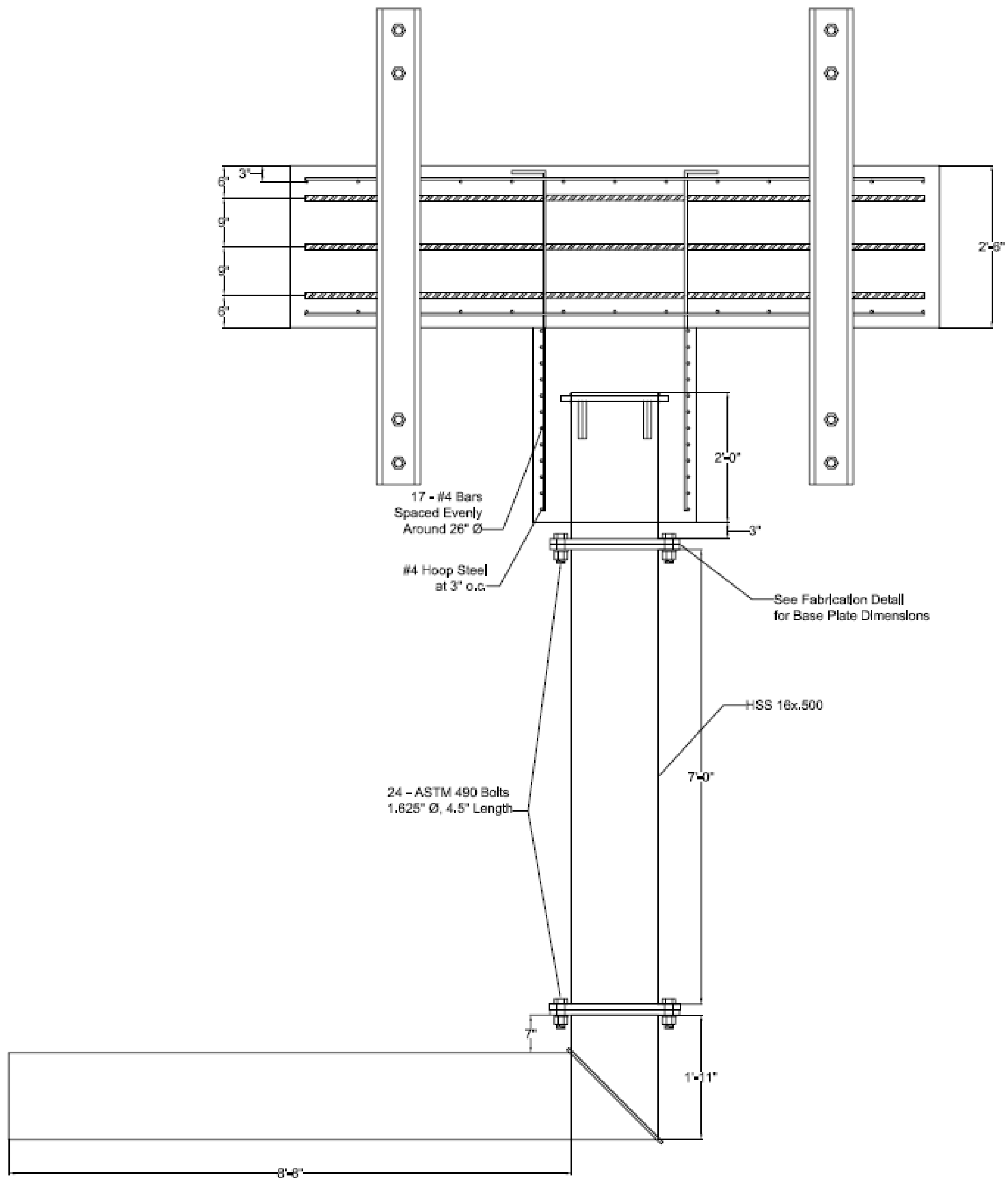


Figure A-7. Dimensioned plan drawing of torsion and flexure test apparatus

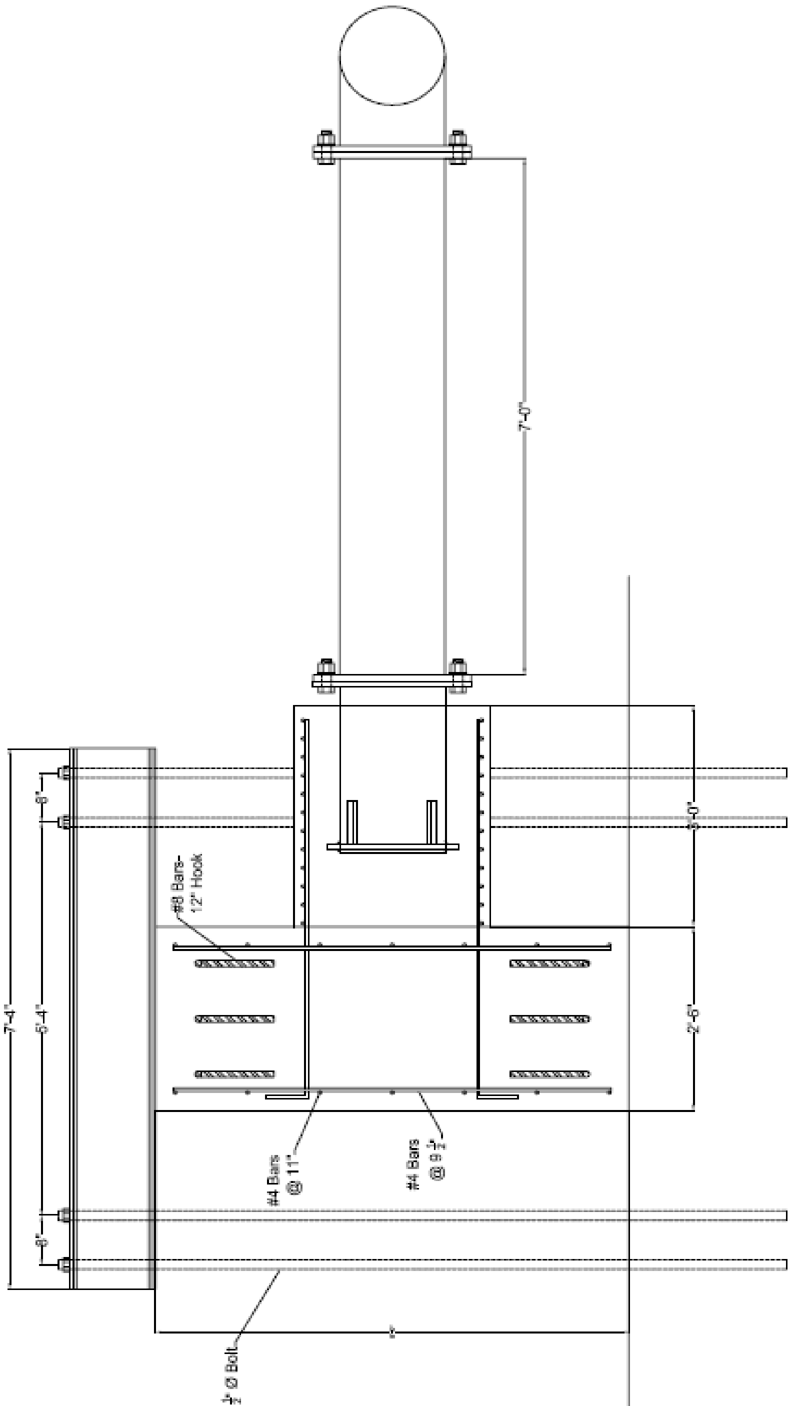


Figure A-8. Dimensioned side view drawing of torsion and flexure test apparatus

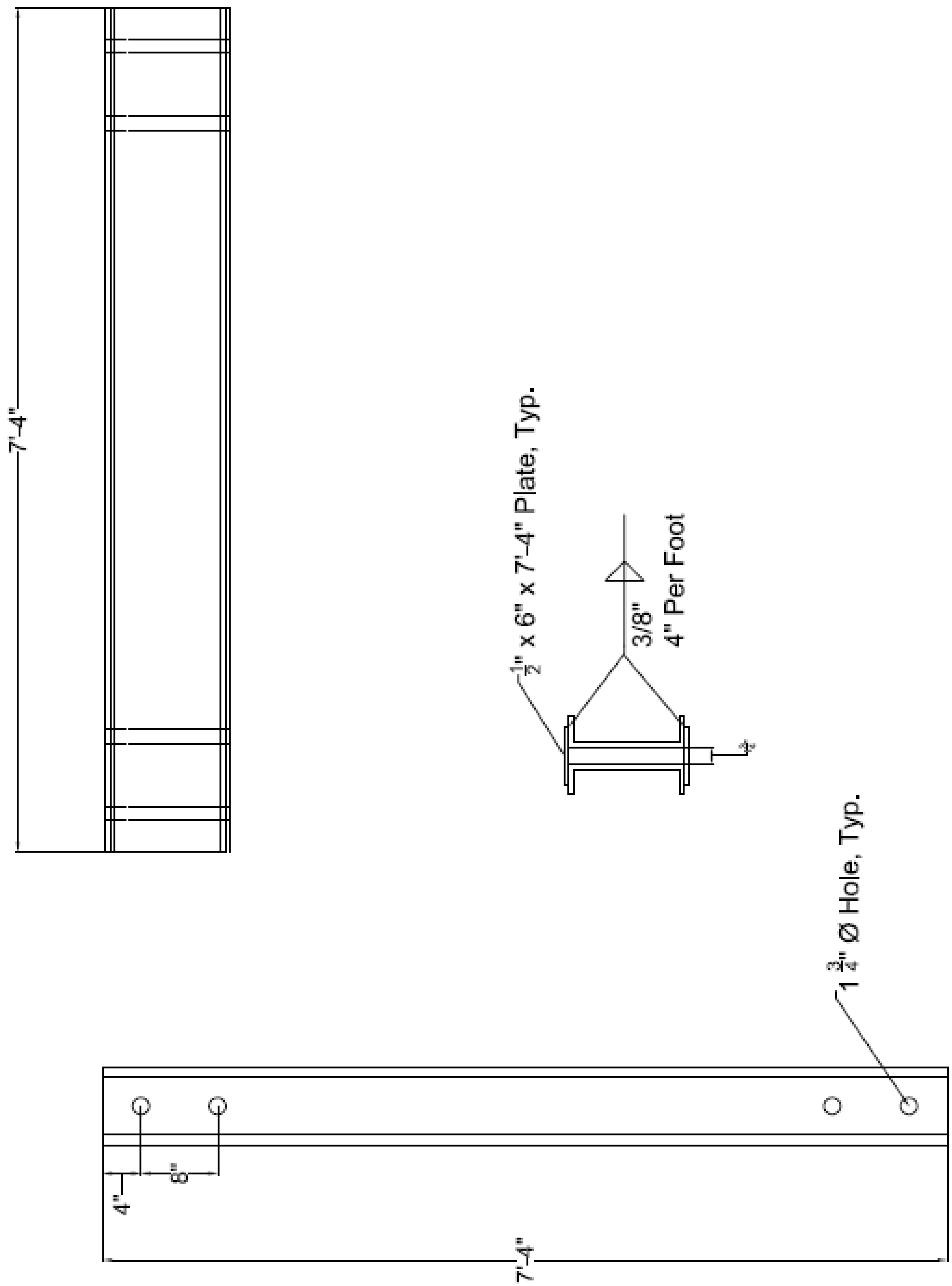


Figure A-9. Dimensioned drawing of channel tie-down for torsion and flexure test

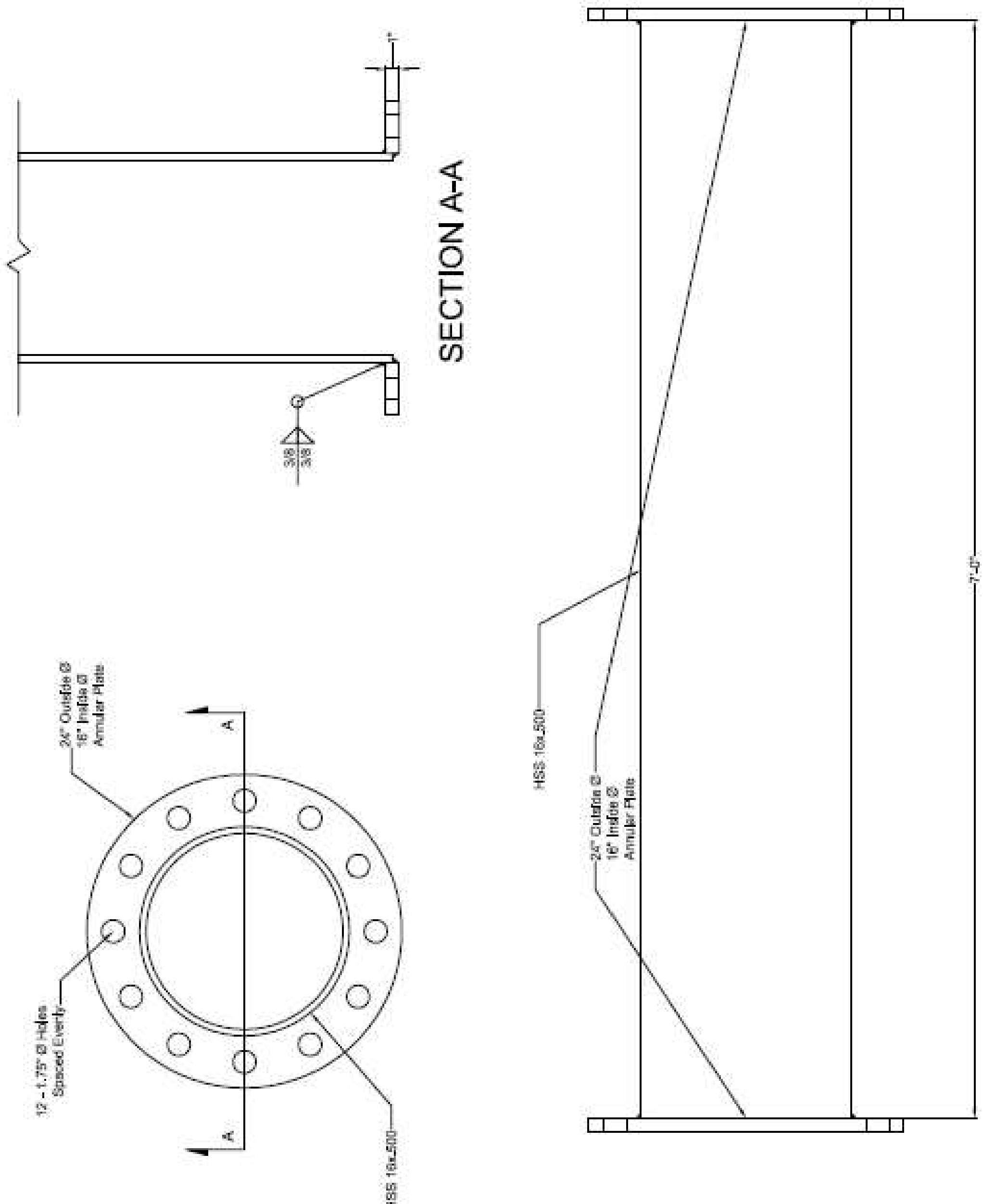


Figure A-10. Dimensioned drawing of flexure extension pipe for torsion and flexure test

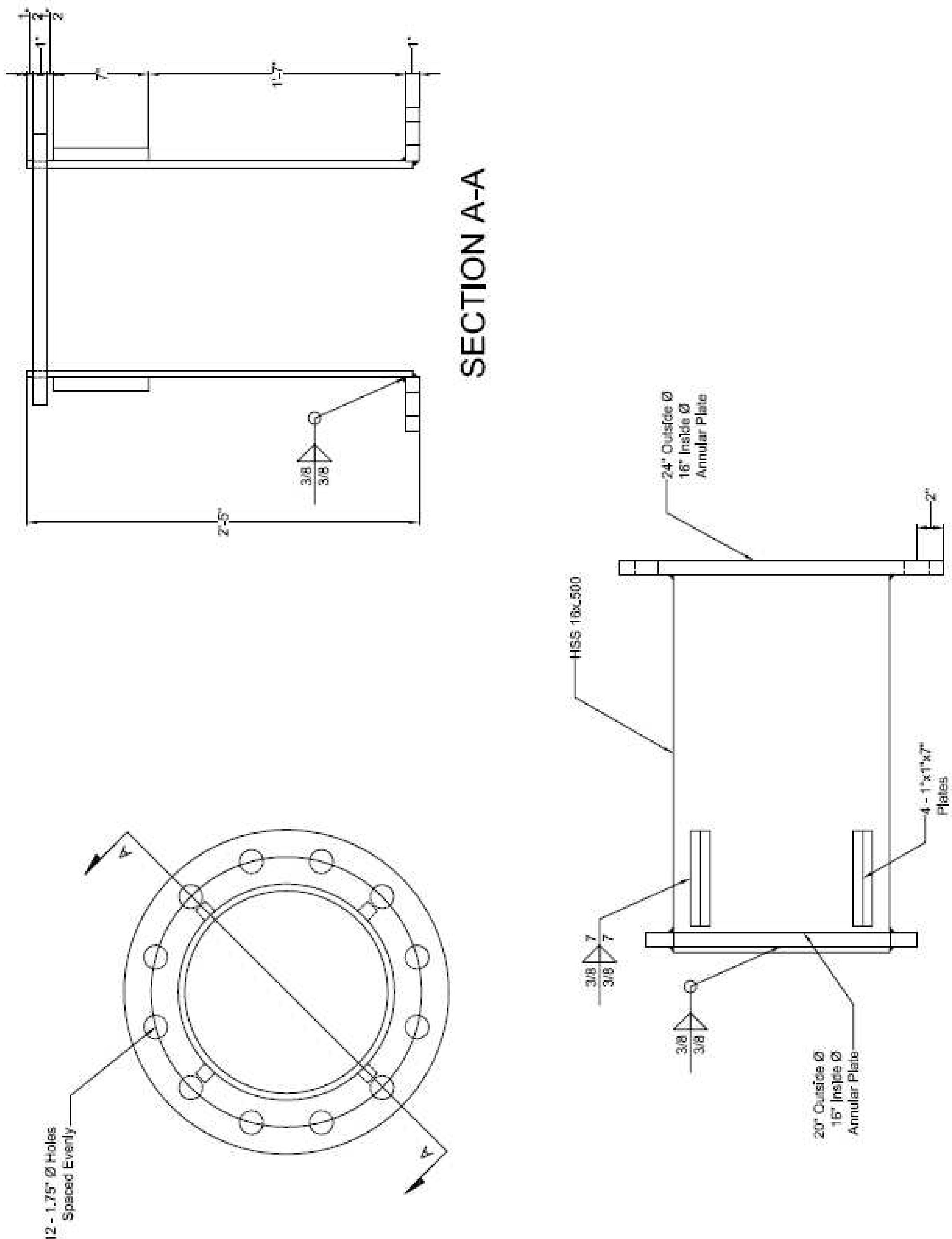


Figure A-11. Dimensioned view of embedded pipe and plates for torsion and flexure test

APPENDIX B
DESIGN CALCULATIONS

Torsion Design Calculations

Input and Properties

Shaft

Diameter of the Shaft	$d_s := 26in$
Concrete Strength	$f_c := 5500psi$
Lenth of Shaft	$L_s := 36in$

Hoop Steel

Hoop Steel Area	$A_{hoop} := .20in^2$
Hoop Steel Diameter	$d_{hoop} := .50in$
Spacing of Hoop Steel	$s_{hoop} := 2in$
Yield Strength of Hoop Steel	$f_{y\_hoop} := 60ksi$
Centerline of Hoop Steel Diamter	$d_h := 23.5in$

Longitudinal Steel

Longitudinal Steel Area	$A_{long} := .2in^2$
Longitudinal Steel Diameter	$d_{long} := .5in$
Yield Strength of Longitudinal Steel	$f_{y\_long} := 60ksi$
Number of Long Steel Bars	$n_{long} := 24$

Torsional Stiffener Plates

Thickness of the plate	$t := 1in$
Width of the plate	$b := 1in$
Length of plate	$L := 7in$
Yield strength of the plate	$f_{y\_plate} := 50ksi$

Embedded Pipe

Thickness of the pipe	$t_{pipe} := .465in$
Diameter of the pipe	$d_{pipe} := 16in$
	$F_{y\_pipe} := 42ksi$
	$F_{u\_pipe} := 58ksi$
Moment Arm	$Tors\_Moment\_Arm := 9ft$

Input and Properties

STIFFENER DESIGN

Calculation of Capacity with Anchor Bolts

Input

Shaft

Diameter of the Shaft $d_s = 26\,in$

Concrete Strength $f_c = 5.5\,ksi$

Equivalent Anchor Bolt

Diameter of the bolt $d_o := 1.5in$

Center-to-center diameter of bolts $d_b := 20in$

Number of bolts $No\_Bolts\_equiv := 12$

Yield strength of bolts $f_{y\_bolt\_equiv} := 105ksi$

Concrete Breakout Equivalent Torsional Strength

Based on ACI 318 Appendix D - Design requirements for shear loading

$$cover := \frac{(d_s - d_b)}{2} \qquad cover = 3\,in$$

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{d_b}{2}\right)^2 + 3.25\cdot\left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_b}{2}\right)^2\right]} - \left(\frac{d_b}{2}\right)\right]}{3.25} \qquad c_{a1} = 2.46\,in$$

$$A := \frac{360deg}{No\_Bolts\_equiv} \qquad A = 30\cdot deg$$

$$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right) \qquad chord\_group = 6.73\,in$$

$$A_{min\_group} := 2\cdot asin\left(\frac{3.0\cdot c_{a1}}{d_s}\right) \qquad A_{min\_group} = 33.03\cdot deg$$

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

Check_Group_Effect = "Group Effect"

$$A_{Vc} := No\_Bolts\_equiv\cdot chord\_group\cdot 1.5\cdot c_{a1} \qquad A_{Vc} = 298.42\,in^2$$

$$A_{Vco} := 4.5\cdot c_{a1}^{\,2} \qquad A_{Vco} = 27.31\,in^2$$

$$l_e := 8\cdot d_o \qquad l_e = 12\,in$$

$$V_b := 13\cdot\left(\frac{l_e}{d_o}\right)^{.2}\cdot\sqrt{\frac{d_o}{in}}\cdot\sqrt{\frac{f_c}{psi}}\cdot\left(\frac{c_{a1}}{in}\right)^{1.5}\cdot lbf \qquad V_b = 6.92\,kip$$

$\psi_{cV} := 1.4$

$\psi_{ecV} := 1.0$

$\psi_{edV} := 1.0$

$$V_{cbg} := \begin{cases} \left(No\_Bolts\_equiv \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b\right) & \text{if } Check\_Group\_Effect = \text{"No Group Effect"} \\ \left[\left(\dfrac{A_{Vc}}{A_{Vco}}\right) \cdot \psi_{ecV} \cdot \psi_{edV} \cdot V_b\right] & \text{if } Check\_Group\_Effect = \text{"Group Effect"} \end{cases}$$

$V_{cbg} = 75.62\,kip$

$V_{cbg\_parallel} := 2 \cdot V_{cbg}$

$V_{cbg\_parallel} = 151.23\,kip$

$$T_{n\_breakout\_ACI} := V_{cbg\_parallel} \cdot \left(\frac{d_b}{2}\right)$$

$T_{n\_breakout\_ACI} = 126.03\,ft \cdot kip$

Calculation of Capacity with Anchor Bolts

Torsional Capacity Using Breakout Capacity

Input

Width of the stiffener plates $b = 1\,in$

Thickness of the stiffener plates $t = 1\,in$

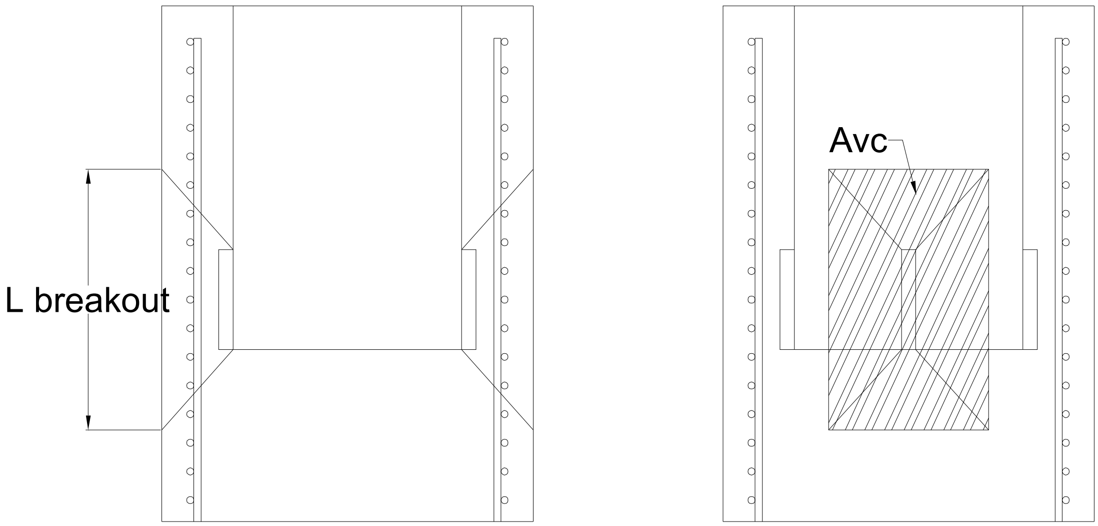
Length of the stiffener plates $L = 7\,in$

Length of the shaft $L_s = 36\,in$

Diameter of upright/embedded pipe $d_{pipe} = 16\,in$

Diameter of stiffeners $d_{st} := d_{pipe}$

Number of stiffeners $No\_Stiff := 4$



Concrete Breakout Equivalent Torsional Strength

Based on ACI 318 Appendix D - Design requirements for shear loading

$$cover := \frac{(d_s - d_{st})}{2} \qquad cover = 5\ in$$

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{d_{st}}{2}\right)^2 + 3.25\cdot\left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_{st}}{2}\right)^2\right]} - \left(\frac{d_{st}}{2}\right)\right]}{3.25} \qquad c_{a1} = 3.73\ in$$

$$A := \frac{360deg}{No\_Stiff} \qquad A = 90\cdot deg$$

$$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right) \qquad chord\_group = 18.38\ in$$

$$A_{min\_group} := 2\cdot asin\left(\frac{3.0\cdot c_{a1}}{d_s}\right) \qquad A_{min\_group} = 51.02\cdot deg$$

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

$l_{breakout} := L + 2 \cdot 1.5 c_{a1}$ $\quad$ $l_{breakout} = 18.2\,in$

$A_{Vc} := \min(l_{breakout}, L_s) \cdot 3 \cdot c_{a1}$ $\quad$ $A_{Vc} = 203.77\,in^2$

$A_{Vco} := 4.5 \cdot c_{a1}^{\ 2}$ $\quad$ $A_{Vco} = 62.69\,in^2$

$l_e := L$ $\quad$ $l_e = 7\,in$

$$V_b := 13 \cdot \left(\frac{l_e}{b}\right)^{.2} \cdot \sqrt{\frac{b}{in}} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf \qquad V_b = 10.26\,kip$$

$$V_{cbg} := \left(\frac{A_{Vc}}{A_{Vco}}\right) \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b \qquad V_{cbg} = 46.69\,kip$$

$V_{cbg\_parallel} := 2 \cdot V_{cbg}$ $\quad$ $V_{cbg\_parallel} = 93.37\,kip$

$V_c := V_{cbg\_parallel} \cdot No\_Stiff$ $\quad$ $V_c = 373.5\,kip$

$$T_{n\_breakout\_plate} := V_c \cdot \left(\frac{d_{st}}{2}\right) \qquad T_{n\_breakout\_plate} = 249\,ft \cdot kip$$

Torsional Capacity Using Breakout Capacity

Torsional Capacity Using Side-Face Blowout Capacity

Input

Width of the stiffener plates $\quad$ $b = 1\,in$

Thickness of the stiffener plates $\quad$ $t = 1\,in$

Length of the stiffener plates $\quad$ $L = 7\,in$

Length of the shaft $\quad$ $L_s = 36\,in$

Diameter of upright/embedded pipe $\quad$ $d_{pipe} = 16\,in$

Diameter of stiffeners $\quad$ $d_{st} := d_{pipe}$

Number of stiffeners $\quad$ $No\_Stiff := 4$

Concrete Breakout Equivalent Torsional Strength

Based on ACI 318 Appendix D

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{d_{st}}{2}\right)^2 + 3.25 \cdot \left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_{st}}{2}\right)^2\right]} - \left(\frac{d_{st}}{2}\right)\right]}{3.25} \qquad c_{a1} = 3.73\,in$$

$A_{brg} := L \cdot b = 7\,in^2$

$N_{sb} := 200 \cdot c_{a1} \cdot \sqrt{A_{brg}} \cdot f_c^{\,.5} \cdot psi^{.5}$ $N_{sb} = 146.48\,kip$

$T_{n\_blowout} := No\_Stiff \cdot N_{sb} \cdot \frac{d_{st}}{2}$ $T_{n\_blowout} = 390.6\,ft \cdot kip$

Torsional Capacity Using Side-Face Blowout Capacity

Capacity Check

$$Check\_Capacity := \begin{cases} \text{"Sufficient Strength"} & \text{if } T_{n\_breakout\_plate} \geq T_{n\_breakout\_ACI} \\ \text{"Insufficient Strength"} & \text{otherwise} \end{cases}$$

Check_Capacity = "Sufficient Strength"

$$\frac{T_{n\_breakout\_plate}}{T_{n\_breakout\_ACI}} = 1.98$$

Capacity Check

Welding for Stiffener Plates

Weld Design

$V_{weld} := \frac{T_{n\_blowout}}{4(.5d_s)}$ $V_{weld} = 90.14\,kip$

$t = 1\,in$

$t_{pipe} = 0.47\,in$

$Weld\_Size := \frac{3}{8}in$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$ AISC Spec. J2 Table J2.5

$Throat := .707 \cdot Weld\_Size$

$R_{n\_weld} := Throat \cdot F_W$ $R_{n\_weld} = 11.14 \cdot \frac{kip}{in}$

$R_{n\_yield} := .6 \cdot F_{y\_pipe} \cdot \frac{t}{2}$ $R_{n\_yield} = 12.6 \cdot \frac{kip}{in}$

$R_{n\_rupture} := .45 \cdot F_{u\_pipe} \cdot \frac{t}{2}$ $R_{n\_rupture} = 13.05 \cdot \frac{kip}{in}$

$R_n := \min\left(R_{n\_weld}, R_{n\_yield}, R_{n\_rupture}\right)$ $R_n = 11.14 \cdot \frac{kip}{in}$

$Required\_Length\_Each\_Side := \frac{V_{weld}}{2 \cdot R_n}$ $Required\_Length\_Each\_Side = 4.05\ in$

$ceil\left(\frac{Required\_Length\_Each\_Side}{in}\right) \cdot in = 5\ in$

Welding for Stiffener Plates

$T_{n\_breakout\_ACI} = 126.03\ ft \cdot kip$

$T_{n\_breakout\_plate} = 249\ ft \cdot kip$

$T_{n\_blowout} = 390.6\ ft \cdot kip$

FLEXURAL CAPACITY

Flexural Capacity of Shaft

Check Flexural Capacity of Shaft

Input

Radius of Shaft $R := \frac{d_s}{2} = 13\ in$

Area of shaft $A_s := \pi \cdot \left(\frac{d_s}{2}\right)^2$

Longitudinal Reinforcement

Number of Longitudinal Bars $n_{long} = 24$

Yield Strength of Longitudinal Reinforcement $f_{y\_long} = 60 \cdot ksi$

Longitudinal Steel Area $A_{long} = 0.2\ in^2$

Number of Bars Yielded (Assumption) $n_{long\_yield} := 17$

Embedded Pipe

Cross sectional area of pipe $A_{pipe} := 24in \cdot .688in$

Inside diameter of pipe $d_{pipe} = 16\ in$

Yield Strength of Pipe $f_{y\_pipe} := 50ksi$

Calculations Using ACI Stress Block at the Point Below the Embedded Pipe

$$\beta_1(f_c) := \begin{cases} .85 & \text{if } f_c < 4000\text{psi} \\ .65 & \text{if } f_c > 8000\text{psi} \\ \left[.85 - .05\cdot\left[\dfrac{(f_c - 4000\text{psi})}{1000\text{psi}}\right]\right] & \text{if } 4000\text{psi} \le f_c \le 8000\text{psi} \end{cases}$$

$\beta_1(f_c) = 0.78$

ACI 10.2.7.3

$$A_{comp} := \frac{(n_{long\_yield}\cdot A_{long}\cdot f_{y\_long})}{.85\cdot f_c}$$

$A_{comp} = 43.64\,\text{in}^2$

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

$$a := \text{root}(A_{compcircle}(h), h, 0\text{in}, R)$$

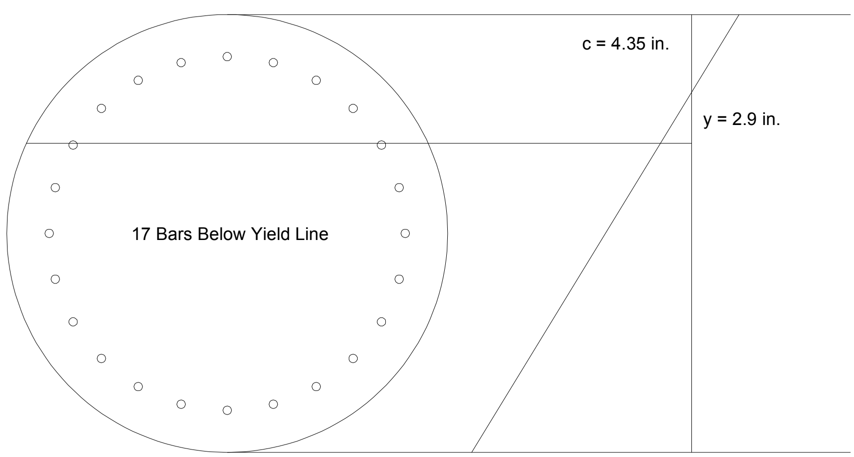
$a = 3.55\,\text{in}$

$$c := \frac{a}{\beta_1(f_c)}$$

$c = 4.59\,\text{in}$

$$y := .002\cdot\frac{c}{.003}$$

$y = 3.06\,\text{in}$



$$d_{bar} := \begin{pmatrix} 9.2502 \\ 12.0237 \\ 15 \\ 17.9763 \\ 20.7498 \\ 23.1314 \\ 25.0189 \\ 26.1677 \end{pmatrix}\text{in}$$

$$d_{bars} := \frac{\left[\left[\sum_{i=0}^{7} \left(d_{bar_i} \cdot A_{long} \cdot 2\right)\right] + 26.5in \cdot A_{long}\right]}{n_{long\_yield} \cdot A_{long}}$$

$d_{bars} = 19.13\,in$

$$M_{n\_shaft} := n_{long\_yield} \cdot A_{long} \cdot f_{y\_long} \cdot \left(d_{bars} - \frac{a}{2}\right)$$

$M_{n\_shaft} = 294.93\,ft \cdot kip$

Flexural Capacity of Shaft

Flexural Capacity of Pipe

$Z := 112in^3$

Embedded Pipe

Cross sectional area of pipe $A_{pipe} := 28.5in^2$

Inside diameter of pipe $d_{pipe} = 16\,in$

Pipe wall thickness $t_{pipe} = 0.47\,in$

Yield Strength of Pipe $F_{y\_pipe} = 42\,ksi$

Diameter to thickness ratio $D\_t := 43.0$

Length of the pipe $L_{pipe} := 3ft$

$E := 29000ksi$

Determine Shear Strength of Round HSS

$$L_v := \frac{L_{pipe}}{2}$$

$$F_{cr\_1} := \max\left[\left[\frac{(1.6 \cdot E)}{\sqrt{\frac{L_v}{d_{pipe}}} \cdot (D\_t)^{\left(\frac{5}{4}\right)}}\right], \left[\frac{(.78 \cdot E)}{(D\_t)^{\left(\frac{3}{2}\right)}}\right]\right]$$

$F_{cr\_1} = 397.29\,ksi$

$$F_{cr} := \min\left(F_{cr\_1}, .6 \cdot F_{y\_pipe}\right)$$

$F_{cr} = 25.2\,ksi$

$$V_{n\_pipe} := \frac{F_{cr} \cdot A_{pipe}}{2}$$

$V_{n\_pipe} = 359.1\,kip$

Determine Flexural Capacity of Round HSS

$$Check\_Applicable := if\left[D\_t < \left(\frac{.45 \cdot E}{F_{y\_pipe}}\right), "Applicable", "N/A"\right]$$

$Check\_Applicable = "Applicable"$

$$\lambda_p := .07 \cdot \frac{E}{F_{y\_pipe}}$$

$$\lambda_r := .31 \cdot \frac{E}{F_{y\_pipe}}$$

$$Check\_Compact := \left| \begin{array}{l} "Compact" \quad if \ D\_t \le \lambda_p \\ "Noncompact" \quad if \ \lambda_p < D\_t \le \lambda_r \\ "Slender" \quad if \ D\_t > \lambda_r \end{array} \right.$$

$Check\_Compact = "Compact"$

$$M_p := F_{y\_pipe} \cdot Z$$

$$M_{n\_pipe} := M_p$$

$M_{n\_pipe} = 392 \ ft \cdot kip$

Flexural Capacity of Pipe

$M_{n\_pipe} = 392 \ ft \cdot kip$

$M_{n\_shaft} = 294.93 \ ft \cdot kip$

FAILURE EQUATIONS

Torsion

Threshold Torsion

$$A_{cp} := \pi \cdot \left(\frac{d_s}{2}\right)^2 = 530.93\ in^2$$

$$p_{cp} := \pi \cdot d_s = 81.68\ in$$

$$T_{threshold} := \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \frac{\left(A_{cp}{}^2\right)}{p_{cp}}$$

$T_{threshold} = 21.33\ ft \cdot kip$

ACI 11.6.1a

Cracking Torsion

$$T_{cr} := 4 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}{}^2}{p_{cp}}\right)$$

$T_{cr} = 85.31\ ft \cdot kip$

ACI R11.6.1

Nominal Torsional Strength

$$A_o := \pi \cdot \left(\frac{d_h}{2}\right)^2 = 433.74\ in^2$$

$$A_t := \pi \cdot \left(\frac{d_{hoop}}{2}\right)^2 = 0.2\ in^2$$

$$\theta := 45 deg$$

$\theta = 0.79 \cdot rad$

$$T_{torsion} := \frac{2 \cdot A_o \cdot A_t \cdot f_{y\_hoop}}{s_{hoop}} \cdot \cot(\theta)$$

$T_{torsion} = 425.82\ ft \cdot kip$

ACI 318-05 11.6.3.6 (11-21)

$$T_{n\_shaft} := T_{torsion}$$

$T_{n\_shaft} = 425.82\ ft \cdot kip$

ACI 318-05 11.6.3.5 (11-20)

Torsion

$T_{n\_breakout\_plate} = 249\ ft \cdot kip$

$T_{n\_blowout} = 390.6\ ft \cdot kip$

$T_{n\_shaft} = 425.82\ ft \cdot kip$

DEVELOPMENT LENGTHS OF FLEXURAL REINF.

Development Length of Longitudinal Bars

Input

Longitudinal Steel

Longitudinal Steel Area $A_{long} = 0.2\,in^2$

Longitudinal Steel Diameter $d_{long} = 0.5\,in$

Yield Strength of Longitudinal Steel $f_{y\_long} = 60\,ksi$

Development Length of Longitudinal Reinforcement

$\Psi_t := 1.3$ ACI 318-05 12.5.2

$\Psi_e := 1.0$

$\Psi_s := 1.0$

$\lambda := 1.0$

$Cb\_Ktr := 2.5in$ ACI 318-05 12.2.3

$$l_{dh\_long} := \left[\left(\frac{3}{40}\right)\cdot\left(\frac{f_{y\_long}}{\sqrt{\frac{f_c}{psi}}\cdot psi}\right)\cdot\frac{(\Psi_t\cdot\Psi_e\cdot\Psi_s\cdot\lambda)}{\left(\frac{Cb\_Ktr}{d_{long}}\right)}\right]\cdot d_{long}$$

$l_{dh\_long} = 7.89\,in$

ACI 318-05 12.2.3

$$l_{d\_long} := l_{dh\_long}$$

$l_{d\_long} = 7.89\,in$

ACI 318-05 12.2.5

$$l_{d\_1} := ceil\left(\frac{l_{d\_long}}{in}\right)\cdot in$$

$l_{d\_1} = 8\,in$

Development Length of Longitudinal Bars

Length of Shaft Required

Length of Stiffeners $L = 7\,in$

Length of Breakout $l_{breakout} = 18.2\,in$

Length of Shaft $L_s = 36\,in$

Development Length of Longitudinal Reinforcement $l_{d\_1} = 8\,in$

Required Cover $c\_cover := 2.5in$

Required Length of Shaft Based on Breakout and Development Length

$l_{shaft} := l_{breakout} + c\_cover + l_{d\_1}$ $\qquad$ $l_{shaft} = 28.7\,in$

$Check\_Shaft\_Length := if(L_s \geq l_{shaft}, \text{"Sufficient"}, \text{"Not Sufficient"})$

$Check\_Shaft\_Length = \text{"Sufficient"}$

Length of Shaft Required

SUPERSTRUCTURE

Superstructure Assembly Strength - Pipes

Superstructure Test Assembly Pipe

Pipe Properties - HSS 16x.500

Design Wall Thickness	$t_{pipe} := .465in$
Cross Sectional Area of Pipe	$A_{pipe} := 22.7in^2$
Diameter to Wall Thickness Ratio	$D\_t := 34.4$
Nominal Weight	$W_{pipe} := 82.85\frac{lbf}{ft}$
Moment of Inertia	$I_{pipe} := 685in^4$
Elastic Section Modulus	$S_{pipe} := 85.7in^3$
Radius of Gyration	$r_{pipe} := 5.49in$
Plastic Section Modulus	$Z_{pipe} := 112in^3$
Diameter of the Pipe	$D_{pipe} := 20in$
Torsional Constant	$J_{pipe} := 1370in^4$
HSS Torsional Constant	$C_{pipe} := 171in^3$
Yield Strength	$F_{y\_pipe} = 42\,ksi$
Ultimate Strength	$F_{u\_pipe} = 58\,ksi$
Modulus of Elasticity	$E := 29000ksi$
Length of Short Superstructure Pipe	$L_{s\_pipe} := 17in$
Length of Long Superstructure Pipe	$L_{l\_pipe} := 9ft$

Short Pipe

Design Flexural Strength

$\phi_{flexure} := .9$

AISC Spec. F1

$$M_{n\_s\_pipe} := \begin{cases} \left(F_{y\_pipe} \cdot Z_{pipe}\right) & \text{if } D\_t \le \left(.45 \cdot \dfrac{E}{F_{y\_pipe}}\right) \\ \text{"Equation Invalid"} & \text{otherwise} \end{cases}$$

$M_{n\_s\_pipe} = 392 \text{ ft} \cdot \text{kip}$

AISC Spec. F2.1

Design Shear Strength

$\phi_{shear} := .9$

AISC Spec. G1

$$F_{cr} := \begin{cases} \left[\left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right]\right] & \text{if } \left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] \ge \left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right] \\ \left[\left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right]\right] & \text{otherwise} \end{cases}$$

$F_{cr\_shear} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_shear} = 25.2 \text{ ksi}$

$$V_{n\_s\_pipe} := \frac{\phi_{shear} \cdot \left(F_{cr\_shear} \cdot A_{pipe}\right)}{2}$$

$V_{n\_s\_pipe} = 257.42 \text{ kip}$

AISC Spec. G6

Design Torsional Strength

$\phi_{torsion} := .75$

AISC Spec. H3.1

$$F_{cr} := \begin{cases} \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] & \text{if } \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] \ge \dfrac{(.60 \cdot E)}{(D\_t)^{1.5}} \\ \left[\dfrac{(.60 \cdot E)}{(D\_t)^{1.5}}\right] & \text{otherwise} \end{cases}$$

$F_{cr\_torsion} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_torsion} = 25.2 \text{ ksi}$

$T_{n\_s\_pipe} := F_{cr\_torsion} \cdot C_{pipe}$

$T_{n\_s\_pipe} = 359.1 \text{ ft} \cdot \text{kip}$

AISC Spec. H3.1

Design Axial Strength

$\phi_{comp} := .90$

$\lambda_r := .31 \cdot \frac{E}{F_{y\_pipe}}$ $\lambda_r = 214.05$

$\lambda_p := .07 \cdot \frac{E}{F_{y\_pipe}}$

$\lambda := \begin{cases} \text{"Compact"} & \text{if } D\_t \le \lambda_p \\ \text{"Noncompact"} & \text{if } \lambda_p < D\_t \le \lambda_r \\ \text{"Slender"} & \text{if } D\_t > \lambda_r \end{cases}$ $\lambda = \text{"Compact"}$

AISC Spec. B4

$k_{s\_pipe} := .5$

$F_{e\_short} := \frac{(\pi^2 \cdot E)}{\left(k_{s\_pipe} \cdot \frac{L_{s\_pipe}}{r_{pipe}}\right)^2}$ $F_{e\_short} = 1.19 \times 10^5\ \text{ksi}$

AISC Equation E3-4

$F_{cr\_short} := \begin{cases} \left[\left[.658^{\left(\frac{F_{y\_pipe}}{F_{e\_short}}\right)} \cdot F_{y\_pipe}\right]\right] & \text{if } F_{e\_short} \ge .44 \cdot F_{y\_pipe} \\ (.877 \cdot F_{e\_short}) & \text{if } F_{e\_short} < .44 \cdot F_{y\_pipe} \end{cases}$

$P_{n\_s\_pipe} := \phi_{comp} \cdot F_{cr\_short} \cdot A_{pipe}$ $P_{n\_s\_pipe} = 857.93\ \text{kip}$

AISC Equation E3-1

Summary for Short Pipe

Flexural Strength $M_{n\_s\_pipe} = 392\ \text{ft} \cdot \text{kip}$

Shear Strength $V_{n\_s\_pipe} = 257.42\ \text{kip}$

Torsional Strength $T_{n\_s\_pipe} = 359.1\ \text{ft} \cdot \text{kip}$

Axial Strength $P_{n\_s\_pipe} = 857.93\ \text{kip}$

Long Pipe

Design Flexural Strength

$\phi_{flexure} = 0.9$

AISC Spec. F1

$$M_{n\_1\_pipe} := \begin{cases} \left(\phi_{flexure} \cdot F_{y\_pipe} \cdot Z_{pipe}\right) & \text{if } D\_t \le \left(.45 \cdot \dfrac{E}{F_{y\_pipe}}\right) \\ \text{"Equation Invalid"} & \text{otherwise} \end{cases}$$

$M_{n\_1\_pipe} = 352.8\ \text{ft}\cdot\text{kip}$

AISC Spec. F2.1

Design Shear Strength

$\phi_{shear} = 0.9$

AISC Spec. G1

$$F_{cr} := \begin{cases} \left[\left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right]\right] & \text{if } \left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] \ge \left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right] \\ \left[\left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right]\right] & \text{otherwise} \end{cases}$$

$F_{cr\_shear} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_shear} = 25.2\ \text{ksi}$

$$V_{n\_1\_pipe} := \frac{\phi_{shear} \cdot \left(F_{cr\_shear} \cdot A_{pipe}\right)}{2}$$

$V_{n\_1\_pipe} = 257.42\ \text{kip}$

AISC Spec. G6

Design Torsional Strength

$\phi_{torsion} = 0.75$

AISC Spec. H3.1

$$F_{cr} := \begin{cases} \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] & \text{if } \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] \ge \dfrac{(.60 \cdot E)}{(D\_t)^{1.5}} \\ \left[\dfrac{(.60 \cdot E)}{(D\_t)^{1.5}}\right] & \text{otherwise} \end{cases}$$

$F_{cr\_torsion} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_torsion} = 25.2\ \text{ksi}$

$T_{n\_1\_pipe} := F_{cr\_torsion} \cdot C_{pipe}$

$T_{n\_1\_pipe} = 359.1\ \text{ft}\cdot\text{kip}$

AISC Spec. H3.1

Design Axial Strength

$\phi_{comp} := .90$

$$\lambda_r := .31 \cdot \frac{E}{F_{y\_pipe}}$$

$\lambda_r = 214.05$

$$\lambda_p := .07 \cdot \frac{E}{F_{y\_pipe}}$$

$$\lambda := \begin{cases} \text{"Compact"} & \text{if } D\_t \le \lambda_p \\ \text{"Noncompact"} & \text{if } \lambda_p < D\_t \le \lambda_r \\ \text{"Slender"} & \text{if } D\_t > \lambda_r \end{cases}$$

$\lambda = \text{"Compact"}$

AISC Spec. B4

$k_{long\_pipe} := 2.0$

$$F_{e\_long} := \frac{\left(\pi^2 \cdot E\right)}{\left(k_{long\_pipe} \cdot \frac{L_{1\_pipe}}{r_{pipe}}\right)^2}$$

$F_{e\_long} = 184.9\,\text{ksi}$

AISC Equation E3-4

$$F_{cr\_long} := \begin{cases} \left[\left[.658^{\left(\frac{F_{y\_pipe}}{F_{e\_long}}\right)} \cdot F_{y\_pipe}\right]\right] & \text{if } F_{e\_long} \ge .44 \cdot F_{y\_pipe} \\ \left(.877 \cdot F_{e\_long}\right) & \text{if } F_{e\_long} < .44 \cdot F_{y\_pipe} \end{cases}$$

$$P_{n\_1\_pipe} := \phi_{comp} \cdot F_{cr\_long} \cdot A_{pipe}$$

$P_{n\_1\_pipe} = 780.24\,\text{kip}$

AISC Equation E3-1

Summary for Long Pipe

Flexural Strength $M_{n\_1\_pipe} = 352.8\,\text{ft} \cdot \text{kip}$

Shear Strength $V_{n\_1\_pipe} = 257.42\,\text{kip}$

Torsional Strength $T_{n\_1\_pipe} = 359.1\,\text{ft} \cdot \text{kip}$

Axial Strength $P_{n\_1\_pipe} = 780.24\,\text{kip}$

Superstructure Assembly Strength - Pipes

Superstructure Assembly Strength - Connecting Plates

Superstructure Test HSS Connection Plate

Plate Properties - PL1/2" x 32" x 24"

Plate thickness $t_p := .5in$

Plate length $h_p := 32in$

Plate width $b_p := 24in$

Yield strength $F_{y\_plate} := 50ksi$

Ultimate strength $F_{u\_plate} := 62ksi$

Design Tensile Strength

$\phi_{t\_yield} := .9$

$P_{n\_yield} := \phi_{t\_yield} \cdot F_{y\_plate} \cdot t_p \cdot b_p$ $\qquad P_{n\_yield} = 540\,kip$

AISC Spec. D2a

$U := 1.0$ $\qquad$ AISC Table D3.1

$A_n := t_p \cdot b_p$ $\qquad$ AISC D3.2

$A_e := U \cdot A_n$ $\qquad A_e = 12\,in^2$

AISC D3.3

$\phi_{t\_rupt} := .75$

$P_{n\_rupture} := \phi_{t\_rupt} \cdot A_e \cdot F_{u\_plate}$ $\qquad P_{n\_rupture} = 558\,kip$

$$P_{n\_plate} := \begin{cases} P_{n\_yield} & \text{if } P_{n\_yield} \le P_{n\_rupture} \\ P_{n\_rupture} & \text{otherwise} \end{cases}$$

$P_{n\_plate} = 540\,kip$

AISC D2b

Design Flexural Strength

$\phi_{flexure} = 0.9$

$A_g := t_p \cdot b_p$ $\qquad A_g = 12\,in^2$

$L_b := 16in$

$$I_p := \frac{b_p \cdot t_p^{\,3}}{3}$$

$$c := \frac{t_p}{2}$$

$$S_p := \frac{I_p}{c}$$ $\qquad S_p = 4\,in^3$

$M_y := S_p \cdot F_{y\_plate}$ $\qquad M_y = 16.67\,ft \cdot kip$

$$Z_p := \left(t_p \cdot \frac{b_p}{2}\right) \cdot \frac{t_p}{2}$$ $Z_p = 1.5\,in^3$

$$M_p := F_{y\_plate} \cdot Z_p$$ $M_p = 6.25\,ft \cdot kip$

$$M_{p\_yield} := \begin{cases} (1.6 \cdot M_y) & \text{if } 1.6 \cdot M_y \le M_p \\ M_p & \text{otherwise} \end{cases}$$ $M_{p\_yield} = 6.25\,ft \cdot kip$

$$LTB\_Equation\_Check := \begin{cases} \text{"Equation F11-2"} & \text{if } \frac{(.08 \cdot E)}{F_{y\_plate}} < \left[\frac{(L_b \cdot b_p)}{t_p^{\,2}}\right] \le \frac{(1.9 \cdot E)}{F_{y\_plate}} \\ \text{"Equation F11-3"} & \text{if } \left[\frac{(L_b \cdot b_p)}{t_p^{\,2}}\right] > \frac{(1.9 \cdot E)}{F_{y\_plate}} \end{cases}$$

$LTB\_Equation\_Check =$ "Equation F11-3"

$C_b := 1.0$ AISC F1

$$F_{cr} := \frac{(1.9 \cdot E \cdot C_b)}{\frac{(L_b \cdot b_p)}{t_p^{\,2}}}$$ $F_{cr} = 35.87\,ksi$

$$M_{n\_ltb} := \begin{cases} (F_{cr} \cdot S_p) & \text{if } LTB\_Equation\_Check = \text{"Equation F11-3"} \\ \left[C_b \cdot \left[1.52 - .274 \cdot \left(L_b \cdot \frac{\frac{b_p}{2}}{t_p}\right) \cdot \frac{F_{y\_plate}}{E}\right]\right] \cdot kip \cdot in & \text{otherwise} \end{cases}$$ $M_{n\_ltb} = 11.96\,ft \cdot kip$

$$M_{n\_plate} := \phi_{flexure} \cdot \begin{cases} M_{n\_ltb} & \text{if } M_{n\_ltb} \le M_{p\_yield} \\ M_{p\_yield} & \text{otherwise} \end{cases}$$ $M_{n\_plate} = 5.62\,ft \cdot kip$

Design Torsional Strength

$\phi_{torsion} = 0.75$

$$M_{t\_plate} := \phi_{torsion} \cdot F_{y\_plate} \cdot .6$$ $M_{t\_plate} = 22.5\,ksi$

Summary for Plate Connector

Tensile Strength $P_{n\_plate} = 540\,kip$

Flexural Strength $M_{n\_plate} = 5.62\,ft \cdot kip$

Torsional Strength $M_{t\_plate} = 22.5\,ksi$

Weld Design

$$V_{weld} := \frac{T_{n\_blowout}}{Tors\_Moment\_Arm}$$ $V_{weld} = 43.4\,kip$

$t_p = 0.5\,in$

$t_{pipe} = 0.47\,in$

$Weld\_Size := \frac{3}{16}in$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$ AISC Spec. J2 Table J2.5

$\phi_{weld} := .75$

$Throat := .707 \cdot Weld\_Size$

$\phi R_{n\_weld} := \phi_{weld} \cdot Throat \cdot F_W$ $\phi R_{n\_weld} = 4.18 \cdot \frac{kip}{in}$

$\phi R_{n\_yield} := .6 \cdot F_{y\_pipe} \cdot t_p$ $\phi R_{n\_yield} = 12.6 \cdot \frac{kip}{in}$

$\phi R_{n\_rupture} := .45 \cdot F_{u\_pipe} \cdot t_p$ $\phi R_{n\_rupture} = 13.05 \cdot \frac{kip}{in}$

$\phi R_n := \min(\phi R_{n\_weld}, \phi R_{n\_yield}, \phi R_{n\_rupture})$ $\phi R_n = 4.18 \cdot \frac{kip}{in}$

$$Required\_Length\_Each\_Side := \frac{V_{weld}}{\phi R_n}$$ $Required\_Length\_Each\_Side = 10.39\,in$

$$ceil\left(\frac{Required\_Length\_Each\_Side}{in}\right) \cdot in = 11\,in$$

Superstructure Assembly Strength - Connecting Plates

Base Connection

Superstructure Test Base Connection Plate

Plate Properties - Annular Plate

Plate diameter $B_p := 24in$

Yield strength $f_{y\_ann} := 50ksi$

Ultimate strength $f_{u\_ann} := 75ksi$

Thickness of plate $t_{plate} := 1.00in$

Bolt Properties - D1" ASTM A325

Center to Center Radius of Bolts $r_b := 12in$

Number of Bolts $No\_Bolts := 12$

Field Strength of Bolts $f_{y\_bolt\_field} := 55ksi$

Ultimate Strength of Bolts $f_{u\_bolt} := 105ksi$

Radius of the pipe $r_p := \frac{D_{pipe}}{2} = 10\,in$

Diameter of the bolt $d_{bolt} := 1.50in$

Bolt Bearing Strength

$$L_c := \frac{\left(B_p - d_b - .5 \cdot d_{bolt}\right)}{2} = 1.62\,in$$

$$\phi_{shear} := .75$$

$$\phi R_n := 1.2 \cdot F_{u\_plate} \cdot L_c \cdot t_{plate} \qquad \phi R_n = 120.9\,kip$$

$$Rn\_parallel := 2 \cdot \phi R_n \qquad Rn\_parallel = 241.8\,kip$$

$$T_{n\_bolt\_bearing} := No\_Bolts \cdot Rn\_parallel \cdot \left(\frac{d_b}{2}\right) \qquad T_{n\_bolt\_bearing} = 2.42 \times 10^3\,ft \cdot kip$$

$$Check\_Bolt\_Bearing := \begin{array}{|l} \text{"Sufficient Strength"} \quad \text{if } T_{n\_bolt\_bearing} \geq T_{n\_blowout} \\ \text{"Insufficient Strength"} \quad \text{otherwise} \end{array}$$

$$Check\_Bolt\_Bearing = \text{"Sufficient Strength"}$$

Check Bolt Spacing

$$s_{req} := 2.67 \cdot d_{bolt} = 4\,in$$

$$s_{actual} := \frac{\pi \cdot d_b}{12} = 5.24\,in$$

$$Check\_Bolt\_Spacing := \begin{array}{|l} \text{"Sufficient"} \quad \text{if } s_{actual} \geq s_{req} \\ \text{"Insufficient"} \quad \text{otherwise} \end{array} \qquad Check\_Bolt\_Spacing = \text{"Sufficient"}$$

Check Bolt Shear

$$A_b := \pi \cdot \left(.5 \cdot d_{bolt}\right)^2 = 1.77\,in^2$$

$$F_{nv} := .4 \cdot 120ksi = 48\,ksi$$

$$\phi V_n := \phi_{shear} \cdot A_b \cdot F_{nv} \qquad \phi V_n = 63.62\,kip$$

$$V_{n\_parallel} := \phi V_n \cdot 2 \qquad V_{n\_parallel} = 127.23\,kip$$

$$T_{bolt\_shear} := No\_Bolts \cdot V_{n\_parallel} \cdot \left(\frac{d_b}{2}\right) \qquad T_{bolt\_shear} = 1.27 \times 10^3\,ft \cdot kip$$

$$Check\_Bolt\_Shear := \begin{cases} \text{"Sufficient Strength"} & \text{if } T_{bolt\_shear} \geq T_{n\_blowout} \\ \text{"Insufficient Strength"} & \text{otherwise} \end{cases}$$

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

Weld Design

Weld Connecting Annular Plate to Pipe

$t_{pipe} = 0.47\,in$

$Weld\_Size := \frac{3}{8}in$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$ AISC Spec. J2 Table J2.5

$Throat := .707 \cdot Weld\_Size$

$R_{n\_weld} := Throat \cdot F_W$ $R_{n\_weld} = 11.14 \cdot \frac{kip}{in}$

$R_{n\_yield} := .6 \cdot F_{y\_pipe} \cdot t_{pipe}$ $R_{n\_yield} = 11.72 \cdot \frac{kip}{in}$

$R_{n\_rupture} := .45 \cdot F_{u\_pipe} \cdot t_{pipe}$ $R_{n\_rupture} = 12.14 \cdot \frac{kip}{in}$

$R_n := \min(R_{n\_weld}, R_{n\_yield}, R_{n\_rupture})$ $R_n = 11.14 \cdot \frac{kip}{in}$

$R_{weld} := R_n \cdot \pi \cdot d_{pipe}$ $R_{weld} = 559.72\,kip$

$T_{weld} := R_{weld} \cdot \frac{d_{pipe}}{2}$ $T_{weld} = 373.15\,ft \cdot kip$

$M_{weld} := R_{weld} \cdot \frac{d_{pipe}}{2}$ $M_{weld} = 373.15\,ft \cdot kip$

Base Connection

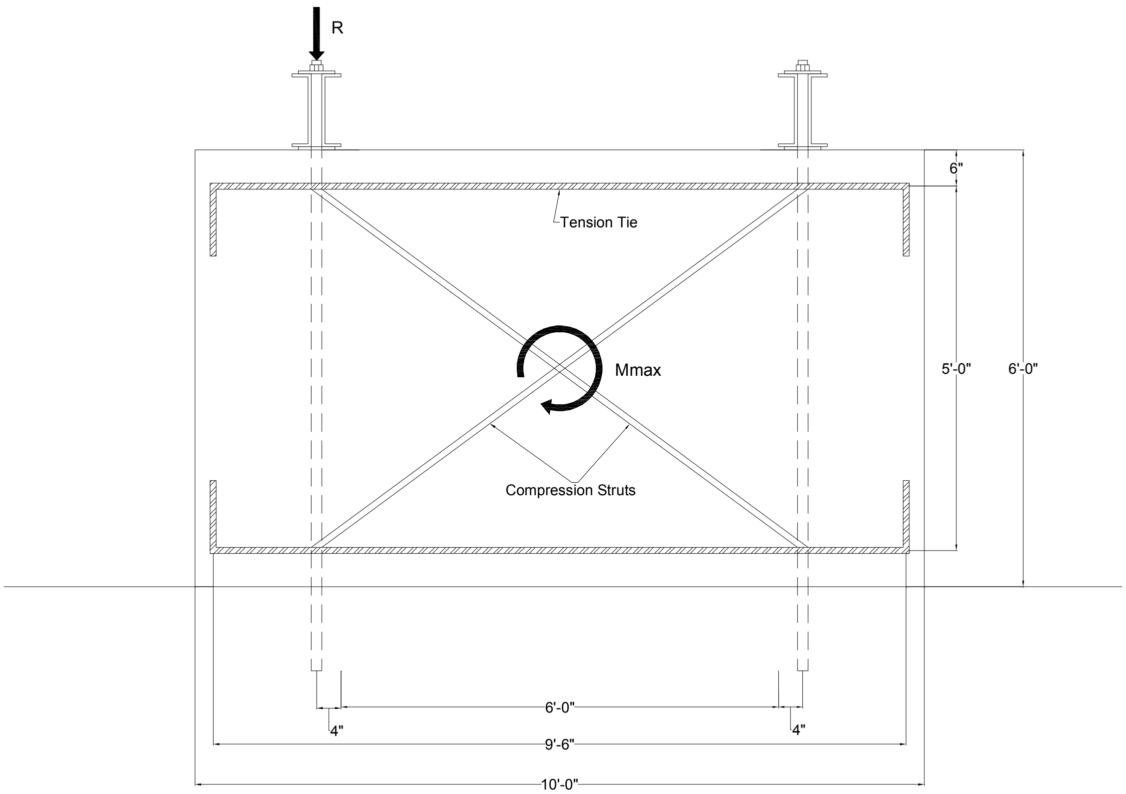
$T_{n\_s\_pipe} = 359.1\,ft \cdot kip$

$T_{weld} = 373.15\,ft \cdot kip$

$M_{weld} = 373.15\,ft \cdot kip$

CONCRETE BLOCK

Concrete Block Design - Strut-and-Tie Model



Based on ACI 318 Appendix A

$M_{max} := T_{n\_breakout\_plate} \cdot \frac{4.5ft}{9ft}$ $\qquad$ $M_{max} = 124.5\,ft \cdot kip$

$d := 6ft + 8in$ $\qquad$ $d = 80\,in$

$R := \frac{M_{max}}{d}$ $\qquad$ $R = 18.67\,kip$

Node A

$\theta := atan\left(5\frac{ft}{d}\right)$ $\qquad$ $\theta = 36.87 \cdot deg$

$C := \frac{R}{\sin(\theta)}$ $\qquad$ $C = 31.12\,kip$

$T := C \cdot \cos(\theta)$ $\qquad$ $T = 24.9\,kip$

Check Reinforcement

$No\_Bars\_Block\_Reinf := 3$

$Block\_Reinf\_Bar\_No := 8$

$f_{y\_block\_reinf} := 60ksi$

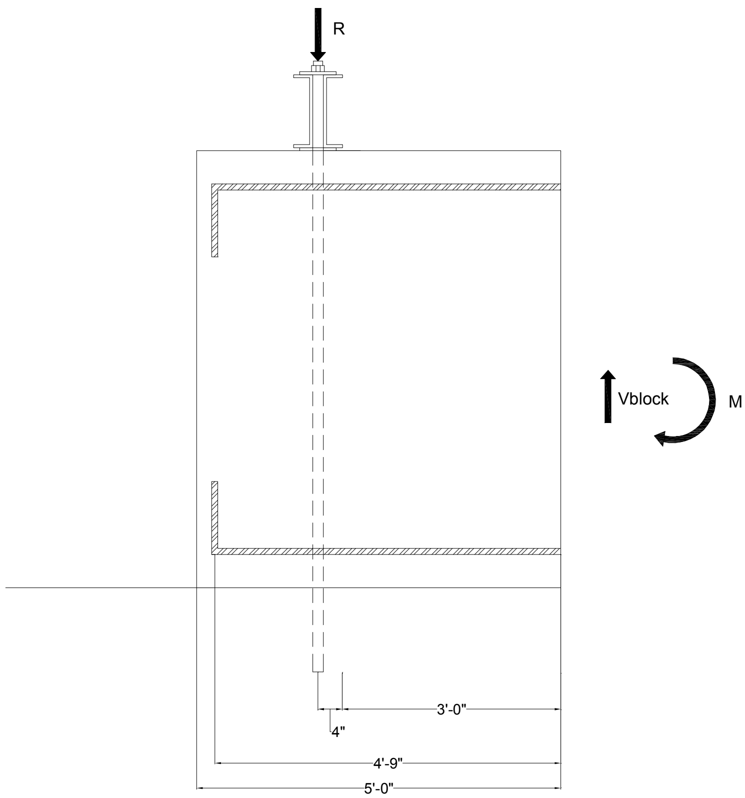
$A_{block\_reinf} := No\_Bars\_Block\_Reinf \cdot \pi \cdot \left(\frac{Block\_Reinf\_Bar\_No \div 8}{2}\right)^2 in^2$ $A_{block\_reinf} = 2.36\,in^2$

$$Check\_Block\_Reinf\_A := \begin{cases} \text{"Sufficient"} & \text{if } A_{block\_reinf} \cdot f_{y\_block\_reinf} \geq T \\ \text{"Not Sufficient"} & \text{otherwise} \end{cases}$$

$Check\_Block\_Reinf\_A = \text{"Sufficient"}$

Concrete Block Design - Strut-and-Tie Model

Concrete Block Design - Beam Theory



$V_{block} := R$ $V_{block} = 18.67\,kip$

$M_{block} := R \cdot (3ft + 4in)$ $M_{block} = 62.25\,ft \cdot kip$

From strut-and-tie model...

$A_{block\_reinf} = 2.36\,in^2$

$f_{y\_block\_reinf} = 60\,ksi$

Check Shear

$$Check\_Shear\_B := \begin{cases} \text{"Sufficient"} & \text{if } A_{block\_reinf} \cdot f_{y\_block\_reinf} \geq V_{block} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

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

Check Flexure

$b_{block} := 30in$

$h_{block} := 6ft$

$d_{block} := 5.5ft$

$T_{given} := A_{block\_reinf} \cdot f_{y\_block\_reinf}$ $\qquad T_{given} = 141.37\,kip$

$C(a) := .85 \cdot f_c \cdot b_{block} \cdot a$

$P(a) := C(a) - T_{given}$

$a := root(P(a), a, 0in, h_{block})$ $\qquad a = 1.01\,in$

$\beta_1(f_c) = 0.78$

$$c := \frac{a}{\beta_1(f_c)}$$ $\qquad c = 1.3\,in$

$$M_{n\_block} := T_{given} \cdot \left(d_{block} - \frac{a}{2}\right)$$ $\qquad M_{n\_block} = 771.61\,ft \cdot kip$

$$Check\_Flexure\_B := \begin{cases} \text{"Sufficient"} & \text{if } M_{n\_block} \geq M_{block} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

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

Required Hook Length for a #8 bar

$$Hook\_No\_8 := 12 \cdot \left(\frac{Block\_Reinf\_Bar\_No}{8} \cdot in\right)$$

$Hook\_No\_8 = 12\,in$

ACI 318-05 Fig. 12.5

Concrete Block Design - Beam Theory

Summary of Concrete Block Reinforcement

Block_Reinf_Bar_No = 8

No_Bars_Block_Reinf = 3

Check_Block_Reinf_A = "Sufficient"

Check_Shear_B = "Sufficient"

Check_Flexure_B = "Sufficient"

Summary of Concrete Block Reinforcement

Tie-Down Design

Block Properties

Width of the block $b_{block} = 30\,in$

Height of the block $h_{block} = 6 \cdot ft$

Length of the block $l_{block} := 10ft$

Diameter of the shaft $d_s = 26\,in$

Length of the shaft $l_{shaft} := 36in$

Weight of concrete $w_c := 150pcf$

Maximum shear applied $V_{max} := \dfrac{T_{n\_breakout\_plate}}{Tors\_Moment\_Arm} = 27.67\,kip$

Channel Assembly - 2 C12x30 Channels with 1.75" between

Moment of inertia about strong axis $I_x := 162in^4$

$S_x := 27.0in^3$

Radius of gyration about strong axis $r_x := 4.29in$

$Z_x := 33.8in^3$

Cross sectional area $A_{channel} := 8.81in^2$

Moment of inertia about weak axis $I_y := 5.12in^4$

Radius of gyration about weak axis $r_y := .762in$

$x\_bar := .674in$

Yield strength $F_{y\_channel} := 50ksi$

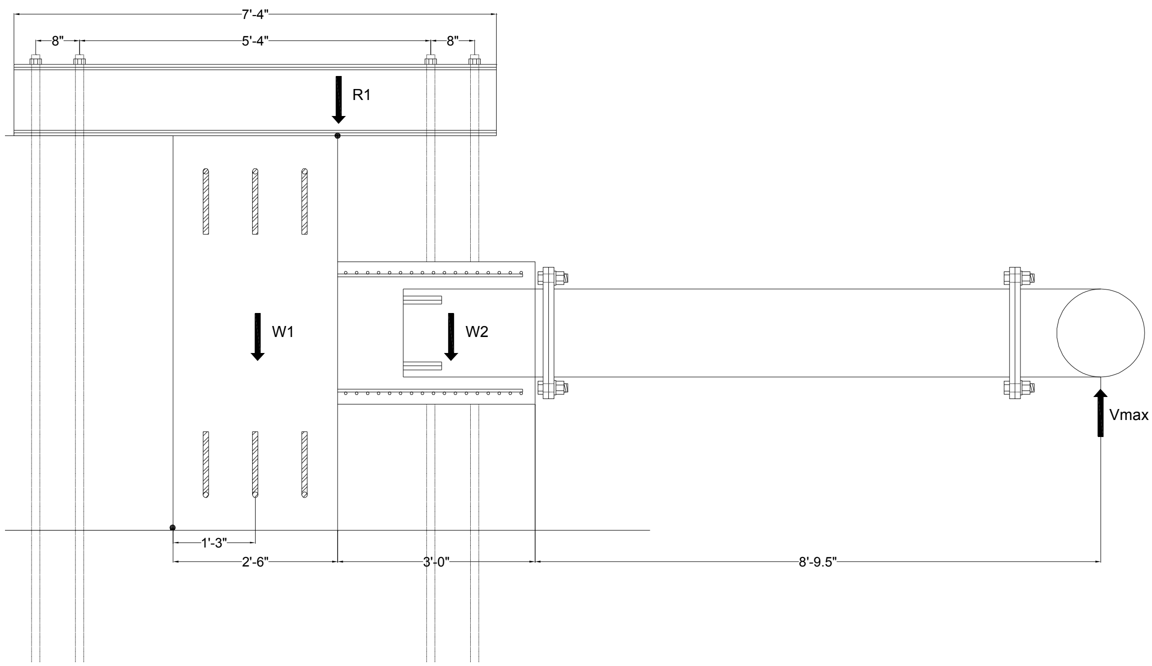
Modulus of elasticity $E = 2.9 \times 10^4\,ksi$

Web thickness $t_w := .510in$

Flange width $b_f := 3.17in$

Flange thickness $t_f := .501in$

Depth $h := 12in$



Calculate self-weight of block

$$W_1 := h_{block} \cdot b_{block} \cdot l_{block} \cdot w_c$$ $W_1 = 22.5\,\text{kip}$

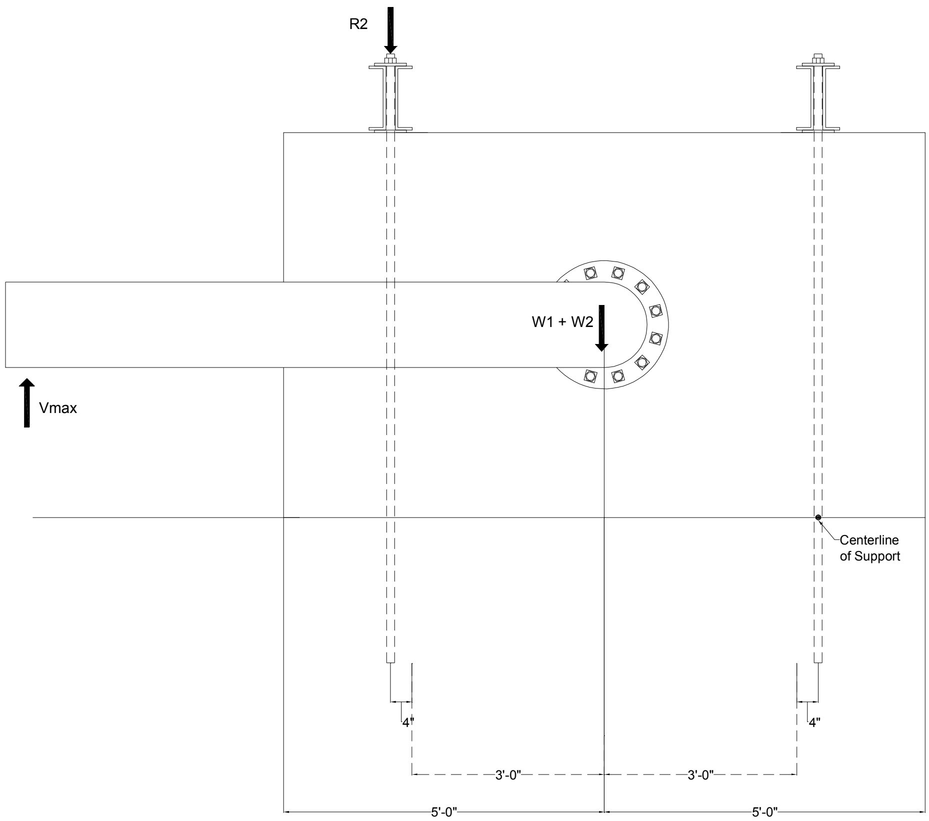
$$W_2 := l_{shaft} \cdot \left(\frac{\pi}{4} \cdot d_s^{\,2}\right) \cdot w_c$$ $W_2 = 1.66\,\text{kip}$

Calculate the Load that the Tie-down must resist in each direction

$$R_1 := \frac{-\left[W_2 \cdot \left(\frac{l_{shaft}}{2} + b_{block}\right) + W_1 \cdot \left(\frac{b_{block}}{2}\right) - V_{max} \cdot \left(l_{shaft} + 17.5\text{in} + b_{block}\right)\right]}{b_{block}}$$

$R_1 = 63.1\,\text{kip}$

$$R_2 := \frac{\left[V_{max} \cdot \left(L_{1\_pipe} + 3\text{ft} + 4\text{in}\right) - \left(W_1 + W_2\right) \cdot (3\text{ft} + 4\text{in})\right]}{6.67\text{ft}}$$ $R_2 = 39.08\,\text{kip}$



Total Load that the Tie-down must support

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

$R = 70.63\,\text{kip}$

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

$Floor\_Capacity := 200kip$

$$Check\_Floor\_Capacity := \begin{cases} \text{"Sufficient"} & \text{if } Floor\_Capacity \geq R \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

$Check\_Floor\_Capacity = \text{"Sufficient"}$

Check Bearing Strength of Concrete

Conservatively assume that the load bears on 1 in. of concrete across the length of the block...

$A_{bearing} := l_{block} \cdot 1in$

$A_{bearing} = 120\,in^2$

$\phi_{bearing} := .65$

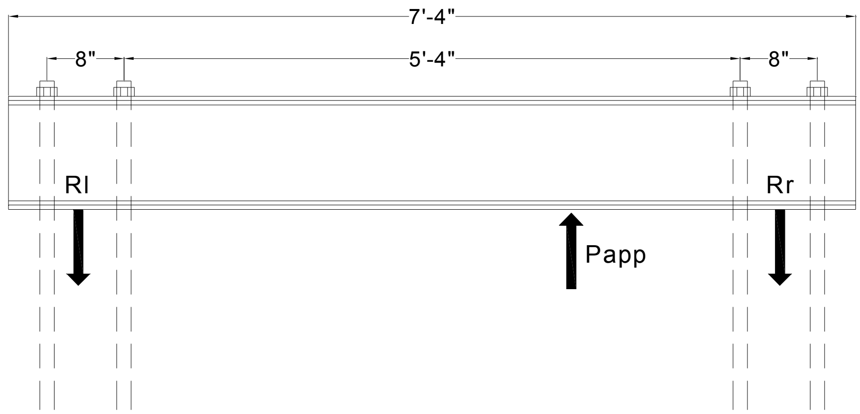
$Bearing\_Strength := \phi_{bearing} \cdot .85 \cdot f_c \cdot A_{bearing}$

$Bearing\_Strength = 364.65\,\text{kip}$

$$Check\_Bearing\_Capacity := \begin{cases} \text{"Sufficient"} & \text{if } Bearing\_Strength \geq 2 \cdot R \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

$Check\_Bearing\_Capacity = \text{"Sufficient"}$

Required Capacity of the Channel Assembly



$P_{app} := R$ $\qquad$ $P_{app} = 70.63\,\text{kip}$

$R_L := \dfrac{P_{app} \cdot 2ft}{6ft}$ $\qquad$ $R_L = 23.54\,\text{kip}$

$R_R := P_{app} - R_L$ $\qquad$ $R_R = 47.09\,\text{kip}$

$a := 4ft$

$b := 2ft$

$M_{max\_tiedown} := \dfrac{P_{app} \cdot a \cdot b}{a + b}$ $\qquad$ $M_{max\_tiedown} = 94.18\,\text{ft} \cdot \text{kip}$

Flexural Capacity of Each Channel

$M_{n\_tiedown} := F_{y\_channel} \cdot (2 \cdot Z_x)$ $\qquad$ $M_{n\_tiedown} = 281.67\,\text{ft} \cdot \text{kip}$

AISC Spec. F2.1
(F2-1)

$$Check\_Flexure\_Channels := \begin{cases} \text{"Sufficient"} & \text{if } M_{n\_tiedown} \geq M_{max\_tiedown} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

Check_Flexure_Channels = "Sufficient"

Buckling Check of Each Channel

Treated as 2 separate channels

$\lambda_f := \dfrac{b_f}{2 \cdot t_f}$ $\qquad$ $\lambda_f = 3.16$

$\lambda_w := \dfrac{h}{t_w}$ $\qquad$ $\lambda_w = 23.53$

$$\lambda_{pf} := .38\cdot\sqrt{\frac{E}{F_{y\_channel}}}$$

$\lambda_{pf} = 9.15$

AISC Spec. B4

$$\lambda_{pw} := 3.76\cdot\sqrt{\frac{E}{F_{y\_channel}}}$$

$\lambda_{pw} = 90.55$

AISC Spec. B4

$$Check\_Flange\_Compact := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pf} > \lambda_f \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

Check_Flange_Compact = "Compact"

AISC Spec. B4

$$Check\_Web\_Compact := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pw} > \lambda_w \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

Check_Web_Compact = "Compact"

AISC Spec. B4

Bracing Check of Each Channel

$L_b := b$

$L_b = 2\cdot ft$

$$L_p := 1.76\cdot r_y\cdot\sqrt{\frac{E}{F_{y\_channel}}}$$

$L_p = 2.69\cdot ft$

AISC Spec. F2.2 (F2-5)

$$Bracing\_Check := \begin{cases} \text{"Braced"} & \text{if } L_p > L_b \\ \text{"Unbraced"} & \text{otherwise} \end{cases}$$

Bracing_Check = "Braced"

Checking Channel Assembly

$$I_{y\_unit} := 2\cdot\left[I_y + \left[A_{channel}\cdot\left[x\_bar + \left(\frac{1.75in}{2}\right)\right]^2\right]\right]$$

$I_{y\_unit} = 52.52\,in^4$

$$r_{y\_unit} := \sqrt{\frac{I_{y\_unit}}{A_{channel}}}$$

$r_{y\_unit} = 2.44\,in$

$$L_{p\_unit} := 1.76\cdot r_{y\_unit}\cdot\sqrt{\frac{E}{F_{y\_channel}}}$$

$L_{p\_unit} = 103.49\,in$

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

$b_{f\_unit} = 8.09\,in$

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

$t_{w\_unit} = 2.77\,in$

$$\lambda_{f\_unit} := \frac{b_{f\_unit}}{2\cdot t_f}$$

$\lambda_{f\_unit} = 8.07$

$$\lambda_{w\_unit} := \frac{h}{t_{w\_unit}}$$

$\lambda_{w\_unit} = 4.33$

$$Check\_Flange\_Compact\_Unit := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pf} > \lambda_{f\_unit} \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

$Check\_Flange\_Compact\_Unit = \text{"Compact"}$

$$Check\_Web\_Compact\_Unit := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pw} > \lambda_{w\_unit} \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

$Check\_Web\_Compact\_Unit = \text{"Compact"}$

$$Bracing\_Check\_Unit := \begin{cases} \text{"Braced"} & \text{if } L_{p\_unit} > L_b \\ \text{"Unbraced"} & \text{otherwise} \end{cases}$$

$Bracing\_Check\_Unit = \text{"Braced"}$

Weld Design

$V_{weld} := \max(R_L, R_R)$

$V_{weld} = 47.09\,kip$

$t_{pl} := .5in$

$b_{pl} := 5in$

$$Q := (t_{pl} \cdot b_{pl}) \cdot \left[\left(\frac{t_{pl}}{2} \right) + \left(\frac{h}{2} \right) \right]$$

$Q = 15.62\,in^3$

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

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

$$Required\_Load\_per\_Foot := \frac{V_{weld} \cdot Q}{I_{weld}}$$

$Required\_Load\_per\_Foot = 17 \cdot \frac{kip}{ft}$

$Weld\_Size := \frac{3}{8}in$

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode} = 42\,ksi$

AISC Spec. J2
Table J2.5

$\phi_{weld} := .75$

$Throat := .707 \cdot Weld\_Size$

$$Required\_Length\_Per\_Foot := \frac{Required\_Load\_per\_Foot}{\phi_{weld} \cdot F_W \cdot Throat}$$

$Required\_Length\_Per\_Foot = 2.04 \cdot \frac{in}{ft}$

Specify 4" per foot of Weld

AISC Spec. J2.4
(J2-3)

Tie-Down Design

$T_{max} := T_{n\_breakout\_plate} = 249\,ft \cdot kip$

$V_{max} = 27.67\,kip$

$M_{max} = 124.5\,ft \cdot kip$

Torsion and Flexure Design Calculations

Input and Properties

Shaft

Diameter of the Shaft	$d_s := 30in$
Concrete Strength	$f_c := 5500psi$
Lenth of Shaft	$L_s := 36in$

Hoop Steel

Hoop Steel Area	$A_{hoop} := .11in^2$
Hoop Steel Diameter	$d_{hoop} := .375in$
Spacing of Hoop Steel	$s_{hoop} := 2.5in$
Yield Strength of Hoop Steel	$f_{y\_hoop} := 60ksi$
Centerline of Hoop Steel Diamter	$d_h := 27in$

Longitudinal Steel

Longitudinal Steel Area	$A_{long} := .2in^2$
Longitudinal Steel Diameter	$d_{long} := .5in$
Yield Strength of Longitudinal Steel	$f_{y\_long} := 60ksi$
Number of Long Steel Bars	$n_{long} := 24$

Torsional Stiffener Plates

Thickness of the plate	$t := 1in$
Width of the plate	$b := 1in$
Length of plate	$L := 7in$
Yield strength of the plate	$f_{y\_plate} := 50ksi$

Flexural Stiffener Plates

Width of the stiffener plates	$b_{flex\_plate} := 1in$
Thickness of the stiffener plates	$t_{flex\_plate} := 1in$
Length of the stiffener plates	$L_{flex\_plate} := 3in$

Embedded Pipe

Thickness of the pipe	$t_{pipe} := .465in$
Diameter of the pipe	$d_{pipe} := 16in$
	$F_{y\_pipe} := 42ksi$
	$F_{u\_pipe} := 58ksi$

Moment Arm	$Tors\_Moment\_Arm := 9ft$ $Flex\_Moment\_Arm := 8ft$

Input and Properties

STIFFENER DESIGN

Calculation of Capacity with Anchor Bolts

Input

Shaft

Diameter of the Shaft $d_s = 30\,in$

Concrete Strength $f_c = 5.5\,ksi$

Equivalent Anchor Bolt

Diameter of the bolt $d_o := 1.5in$

Center-to-center diameter of bolts $d_b := 20in$

Number of bolts $No\_Bolts\_equiv := 12$

Yield strength of bolts $f_{y\_bolt\_equiv} := 105ksi$

Concrete Breakout Equivalent Torsional Strength

Based on ACI 318 Appendix D - Design requirements for shear loading

$$cover := \frac{(d_s - d_b)}{2} \qquad cover = 5\,in$$

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

$$A := \frac{360deg}{No\_Bolts\_equiv} \qquad A = 30\cdot deg$$

$$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right) \qquad chord\_group = 7.76\,in$$

$$A_{min\_group} := 2\cdot asin\left(\frac{3.0\cdot c_{a1}}{d_s}\right) \qquad A_{min\_group} = 45.24\cdot deg$$

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

Check_Group_Effect = "Group Effect"

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

$$A_{Vco} := 4.5\cdot c_{a1}^{\;2} \qquad A_{Vco} = 66.57\,in^2$$

$$l_e := 8\cdot d_o \qquad l_e = 12\,in$$

$$V_b := 13\cdot\left(\frac{l_e}{d_o}\right)^{.2}\cdot\sqrt{\frac{d_o}{in}}\cdot\sqrt{\frac{f_c}{psi}}\cdot\left(\frac{c_{a1}}{in}\right)^{1.5}\cdot lbf \qquad V_b = 13.5\,kip$$

$\psi_{cV} := 1.4$

$\psi_{ecV} := 1.0$

$\psi_{edV} := 1.0$

$$V_{cbg} := \begin{cases} \left(No\_Bolts\_equiv \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b\right) & \text{if } Check\_Group\_Effect = \text{"No Group Effect"} \\ \left[\left(\frac{A_{Vc}}{A_{Vco}}\right) \cdot \psi_{ecV} \cdot \psi_{edV} \cdot V_b\right] & \text{if } Check\_Group\_Effect = \text{"Group Effect"} \end{cases}$$

$V_{cbg} = 109.01\ kip$

$V_{cbg\_parallel} := 2 \cdot V_{cbg}$

$V_{cbg\_parallel} = 218.03\ kip$

$T_{n\_breakout\_ACI} := V_{cbg\_parallel} \cdot \left(\frac{d_b}{2}\right)$

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

Calculation of Capacity with Anchor Bolts

Torsional Capacity Using Breakout Capacity

Input

Width of the stiffener plates $b = 1\ in$

Thickness of the stiffener plates $t = 1\ in$

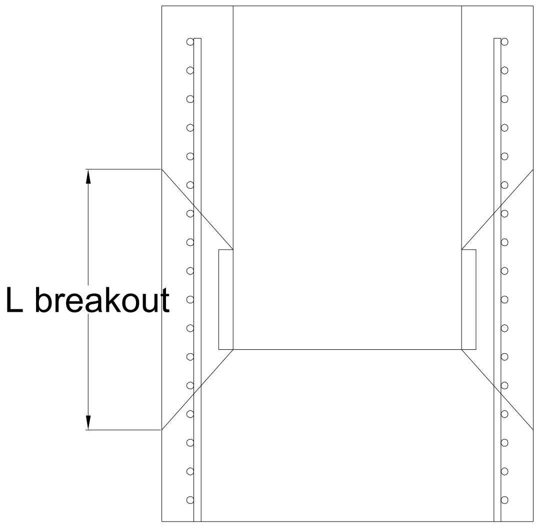
Length of the stiffener plates $L = 7\ in$

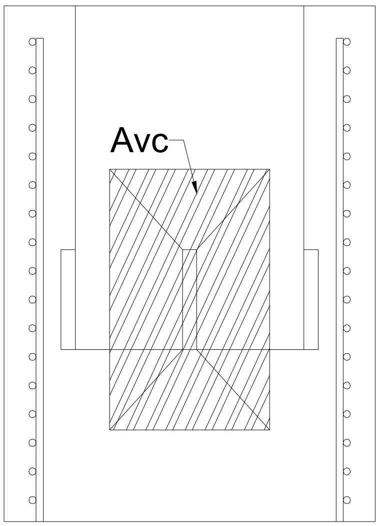
Length of the shaft $L_s = 36\ in$

Diameter of upright/embedded pipe $d_{pipe} = 16\ in$

Diameter of stiffeners $d_{st} := d_{pipe}$

Number of stiffeners $No\_Stiff := 4$





Concrete Breakout Equivalent Torsional Strength

Based on ACI 318 Appendix D - Design requirements for shear loading

$$cover := \frac{(d_s - d_{st})}{2} \qquad cover = 7\,in$$

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{d_{st}}{2}\right)^2 + 3.25\cdot\left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_{st}}{2}\right)^2\right]} - \left(\frac{d_{st}}{2}\right)\right]}{3.25} \qquad c_{a1} = 4.99\,in$$

$$A := \frac{360deg}{No\_Stiff} \qquad A = 90\cdot deg$$

$$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right) \qquad chord\_group = 21.21\,in$$

$$A_{min\_group} := 2\cdot\operatorname{asin}\left(\frac{3.0\cdot c_{a1}}{d_s}\right) \qquad A_{min\_group} = 59.93\cdot deg$$

$$Check\_Group\_Effect := \begin{array}{|l} \text{"Group Effect"} \quad \text{if } A \le A_{min\_group} \\ \text{"No Group Effect"} \quad \text{otherwise} \end{array} \qquad Check\_Group\_Effect = \text{"No Group Effect"}$$

$l_{breakout} := L + 2 \cdot 1.5 c_{a1}$ $\qquad$ $l_{breakout} = 21.98\,in$

$A_{Vc} := \min(l_{breakout}, L_s) \cdot 3 \cdot c_{a1}$ $\qquad$ $A_{Vc} = 329.43\,in^2$

$A_{Vco} := 4.5 \cdot c_{a1}^{\;2}$ $\qquad$ $A_{Vco} = 112.27\,in^2$

$l_e := L$ $\qquad$ $l_e = 7\,in$

$$V_b := 13 \cdot \left(\frac{l_e}{b}\right)^{.2} \cdot \sqrt{\frac{b}{in}} \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf$$ $\qquad$ $V_b = 15.88\,kip$

$$V_{cbg} := \left(\frac{A_{Vc}}{A_{Vco}}\right) \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b$$ $\qquad$ $V_{cbg} = 65.25\,kip$

$V_{cbg\_parallel} := 2 \cdot V_{cbg}$ $\qquad$ $V_{cbg\_parallel} = 130.49\,kip$

$V_c := V_{cbg\_parallel} \cdot No\_Stiff$ $\qquad$ $V_c = 521.97\,kip$

$$T_{n\_breakout\_plate} := V_c \cdot \left(\frac{d_{st}}{2}\right)$$ $\qquad$ $T_{n\_breakout\_plate} = 347.98\,ft \cdot kip$

Torsional Capacity Using Breakout Capacity

Torsional Capacity Using Side-Face Blowout Capacity

Input

Width of the stiffener plates	$b = 1\,in$
Thickness of the stiffener plates	$t = 1\,in$
Length of the stiffener plates	$L = 7\,in$
Length of the shaft	$L_s = 36\,in$
Diameter of upright/embedded pipe	$d_{pipe} = 16\,in$
Diameter of stiffeners	$d_{st} := d_{pipe}$
Number of stiffeners	$No\_Stiff := 4$

Concrete Breakout Equivalent Torsional Strength

Based on ACI 318 Appendix D

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{d_{st}}{2}\right)^2 + 3.25 \cdot \left[\left(\frac{d_s}{2}\right)^2 - \left(\frac{d_{st}}{2}\right)^2\right]} - \left(\frac{d_{st}}{2}\right)\right]}{3.25}$$ $\qquad$ $c_{a1} = 4.99\,in$

$A_{brg} := L \cdot b = 7\ in^2$

$N_{sb} := 200 \cdot c_{a1} \cdot \sqrt{A_{brg}} \cdot f_c^{\ .5} \cdot psi^{.5}$ $\qquad N_{sb} = 196.01\ kip$

$T_{n\_blowout} := No\_Stiff \cdot N_{sb} \cdot \frac{d_{st}}{2}$ $\qquad T_{n\_blowout} = 522.7\ ft \cdot kip$

Torsional Capacity Using Side-Face Blowout Capacity

Capacity Check

$$Check\_Capacity := \begin{array}{|ll} \text{"Sufficient Strength"} & \text{if } T_{n\_breakout\_plate} \geq T_{n\_breakout\_ACI} \\ \text{"Insufficient Strength"} & \text{otherwise} \end{array}$$

Check_Capacity = "Sufficient Strength"

$\frac{T_{n\_breakout\_plate}}{T_{n\_breakout\_ACI}} = 1.92$

Capacity Check

Welding for Stiffener Plates

Weld Design

$V_{weld} := \frac{T_{n\_blowout}}{4(.5d_s)}$ $\qquad V_{weld} = 104.54\ kip$

$t = 1\ in$

$t_{pipe} = 0.47\ in$

$Weld\_Size := \frac{3}{8} in$ $\qquad$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$ $\qquad$ AISC Spec. J2 Table J2.5

$Throat := .707 \cdot Weld\_Size$

$R_{n\_weld} := Throat \cdot F_W$ $R_{n\_weld} = 11.14 \cdot \frac{kip}{in}$

$R_{n\_yield} := .6 \cdot F_{y\_pipe} \cdot \frac{t}{2}$ $R_{n\_yield} = 12.6 \cdot \frac{kip}{in}$

$R_{n\_rupture} := .45 \cdot F_{u\_pipe} \cdot \frac{t}{2}$ $R_{n\_rupture} = 13.05 \cdot \frac{kip}{in}$

$R_n := \min(R_{n\_weld}, R_{n\_yield}, R_{n\_rupture})$ $R_n = 11.14 \cdot \frac{kip}{in}$

$Required\_Length\_Each\_Side := \frac{V_{weld}}{2 \cdot R_n}$ $Required\_Length\_Each\_Side = 4.69\ in$

$ceil\left(\frac{Required\_Length\_Each\_Side}{in}\right) \cdot in = 5\ in$

Welding for Stiffener Plates

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

$T_{n\_breakout\_plate} = 347.98\ ft \cdot kip$

$T_{n\_blowout} = 522.7\ ft \cdot kip$

FLEXURAL CAPACITY

Equivalent Bolt Flexural Capacity

Input

Shaft

Diameter of the Shaft $d_s = 30\ in$

Concrete Strength $f_c = 5.5\ ksi$

Equivalent Anchor Bolt

Diameter of the bolt $d_o := 1.5in$

Center-to-center diameter of bolts $d_b := 20in$

Number of bolts $No\_Bolts\_equiv := 12$

Yield strength of bolts $f_{u\_bolt\_equiv} := 125ksi$

Calculate flexural capacity...

$A_b := \left[\pi \cdot \left(\frac{d_o}{2}\right)^2\right]$ $A_b = 1.77\ in^2$

$M_{n\_bolt} := A_b \cdot f_{u\_bolt\_equiv} \cdot No\_Bolts\_equiv \cdot \frac{d_o}{4}$ $M_{n\_bolt} = 82.83\ ft \cdot kip$

Equivalent Bolt Flexural Capacity

Flexural Capacity of Shaft

Check Flexural Capacity of Shaft

Input

Radius of Shaft $R := \frac{d_s}{2} = 15\,\text{in}$

Area of shaft $A_s := \pi\cdot\left(\frac{d_s}{2}\right)^2$

Longitudinal Reinforcement

Number of Longitudinal Bars $n_{long} = 24$

Yield Strength of Longitudinal Reinforcement $f_{y\_long} = 60\cdot\text{ksi}$

Longitudinal Steel Area $A_{long} = 0.2\,\text{in}^2$

Number of Bars Yielded (Assumption) $n_{long\_yield} := 17$

Embedded Pipe

Cross sectional area of pipe $A_{pipe} := 24\text{in}\cdot .688\text{in}$

Inside diameter of pipe $d_{pipe} = 16\,\text{in}$

Yield Strength of Pipe $f_{y\_pipe} := 50\text{ksi}$

Calculations Using ACI Stress Block at the Point Below the Embedded Pipe

$$\beta_1(f_c) := \begin{cases} .85 & \text{if } f_c < 4000\text{psi} \\ .65 & \text{if } f_c > 8000\text{psi} \\ \left[.85 - .05\cdot\left[\frac{(f_c - 4000\text{psi})}{1000\text{psi}}\right]\right] & \text{if } 4000\text{psi} \le f_c \le 8000\text{psi} \end{cases}$$

$\beta_1(f_c) = 0.78$

ACI 10.2.7.3

$$A_{comp} := \frac{(n_{long\_yield}\cdot A_{long}\cdot f_{y\_long})}{.85\cdot f_c}$$

$A_{comp} = 43.64\,\text{in}^2$

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

$$a := \text{root}(A_{compcircle}(h), h, 0\text{in}, R)$$

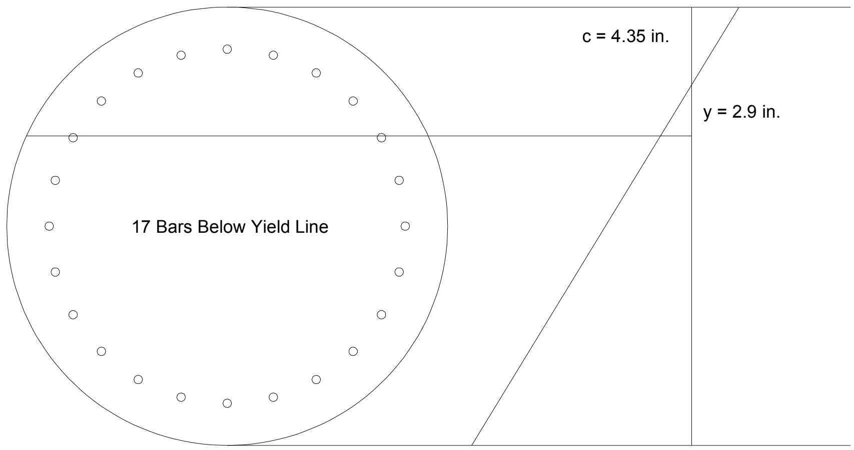
$a = 3.37\,\text{in}$

$$c := \frac{a}{\beta_1(f_c)}$$

$c = 4.35\,\text{in}$

$$y := .002\cdot\frac{c}{.003}$$

$y = 2.9\,\text{in}$



$$d_{bar} := \begin{pmatrix} 9.2502 \\ 12.0237 \\ 15 \\ 17.9763 \\ 20.7498 \\ 23.1314 \\ 25.0189 \\ 26.1677 \end{pmatrix} in \qquad d_{bars} := \frac{\left[\left[\sum_{i=0}^{7} \left(d_{bar_i} \cdot A_{long} \cdot 2\right)\right] + 26.5in \cdot A_{long}\right]}{n_{long\_yield} \cdot A_{long}} \qquad d_{bars} = 19.13\,in$$

$$M_{n\_shaft} := n_{long\_yield} \cdot A_{long} \cdot f_{y\_long} \cdot \left(d_{bars} - \frac{a}{2}\right) \qquad M_{n\_shaft} = 296.49\,ft \cdot kip$$

Flexural Capacity of Shaft

Flexural Capacity of Pipe

$Z := 112in^3$

Embedded Pipe

Cross sectional area of pipe $A_{pipe} := 28.5in^2$

Inside diameter of pipe $d_{pipe} = 16\,in$

Pipe wall thickness $t_{pipe} = 0.47\,in$

Yield Strength of Pipe $F_{y\_pipe} = 42\,ksi$

Diameter to thickness ratio $D\_t := 43.0$

Length of the pipe $L_{pipe} := 3ft$

$E := 29000ksi$

Determine Shear Strength of Round HSS

$$L_v := \frac{L_{pipe}}{2}$$

$$F_{cr\_1} := \max\left[\left[\frac{(1.6 \cdot E)}{\sqrt{\frac{L_v}{d_{pipe}}} \cdot (D\_t)^{\left(\frac{5}{4}\right)}}\right], \left[\frac{(.78 \cdot E)}{(D\_t)^{\left(\frac{3}{2}\right)}}\right]\right]$$

$F_{cr\_1} = 397.29\ ksi$

$$F_{cr} := \min\left(F_{cr\_1}, .6 \cdot F_{y\_pipe}\right)$$

$F_{cr} = 25.2\ ksi$

$$V_{n\_pipe} := \frac{F_{cr} \cdot A_{pipe}}{2}$$

$V_{n\_pipe} = 359.1\ kip$

Determine Flexural Capacity of Round HSS

$$Check\_Applicable := if\left[D\_t < \left(\frac{.45 \cdot E}{F_{y\_pipe}}\right), \text{"Applicable"}, \text{"N/A"}\right]$$

$Check\_Applicable = \text{"Applicable"}$

$$\lambda_p := .07 \cdot \frac{E}{F_{y\_pipe}}$$

$$\lambda_r := .31 \cdot \frac{E}{F_{y\_pipe}}$$

$$Check\_Compact := \left| \begin{array}{l} \text{"Compact"} \quad \text{if } D\_t \le \lambda_p \\ \text{"Noncompact"} \quad \text{if } \lambda_p < D\_t \le \lambda_r \\ \text{"Slender"} \quad \text{if } D\_t > \lambda_r \end{array} \right.$$

$Check\_Compact = \text{"Compact"}$

$$M_p := F_{y\_pipe} \cdot Z$$

$$M_{n\_pipe} := M_p$$

$M_{n\_pipe} = 392\ ft \cdot kip$

Flexural Capacity of Pipe

Flexural Capacity of T-Plates Using Side-Face Blowout Capacity

Flexural Stiffener Plates

Width of the stiffener plates $b_{flex\_plate} := 2in$

Thickness of the stiffener plates $t_{flex\_plate} := 1in$

Length of the stiffener plates $L_{flex\_plate} := .125\pi \cdot d_{pipe}$

Length of the shaft $L_s = 36\,in$

Diameter of upright/embedded pipe $d_{pipe} = 16\,in$

Diameter of stiffeners $d_{st} := d_{pipe} + 2 \cdot b_{flex\_plate}$

Concrete Breakout Equivalent Flexural Strength

Based on ACI 318 Appendix D

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

$$A_{brg} := L_{flex\_plate} \cdot b_{flex\_plate} \qquad A_{brg} = 12.57\,in^2$$

$$N_{sb} := 200 \cdot c_{a1} \cdot \sqrt{A_{brg}} \cdot f_c^{.5} \cdot psi^{.5} \qquad N_{sb} = 202.23\,kip$$

$$M_{n\_blowout} := N_{sb} \cdot d_{st} \qquad M_{n\_blowout} = 337.05\,ft \cdot kip$$

Flexural Capacity of T-Plates Using Side-Face Blowout Capacity

Flexural Capacity Using Breakout Capacity

Input

Width of the stiffener plates $b_{flex\_plate} := 1in$

Thickness of the stiffener plates $t_{flex\_plate} = 1\,in$

Length of the stiffener plates $L_{flex\_plate} := 7in$

Length of the shaft $L_s = 36\,in$

Diameter of upright/embedded pipe $d_{pipe} = 16\,in$

Diameter of stiffeners $d_{st} := d_{pipe} + 4in$

Number of stiffeners $No\_Stiff := 4$

Concrete Breakout Equivalent Flexural Strength

Based on ACI 318 Appendix D - Design requirements for shear loading

$$cover := \frac{(d_s - d_{st})}{2} \qquad cover = 5\,in$$

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

$$A := \frac{360deg}{No\_Stiff} \qquad A = 90\cdot deg$$

$$chord\_group := 2\cdot\frac{d_s}{2}\cdot\sin\left(\frac{A}{2}\right) \qquad chord\_group = 21.21\,in$$

$$A_{min\_group} := 2\cdot asin\left(\frac{3.0\cdot c_{a1}}{d_s}\right) \qquad A_{min\_group} = 45.24\cdot deg$$

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

$$l_{breakout} := t_{flex\_plate} + 2\cdot 1.5c_{a1} \qquad l_{breakout} = 12.54\,in$$

$$A_{Vc} := \min\left(l_{breakout}, L_s\right)\cdot 3\cdot c_{a1} \qquad A_{Vc} = 144.67\,in^2$$

$$A_{Vco} := 4.5\cdot c_{a1}{}^2 \qquad A_{Vco} = 66.57\,in^2$$

$$l_e := L_{flex\_plate} \qquad l_e = 7\,in$$

$$V_b := 13\cdot\left(\frac{l_e}{t_{flex\_plate}}\right)^{.2}\cdot\sqrt{\frac{b_{flex\_plate}}{in}}\cdot\sqrt{\frac{f_c}{psi}}\cdot\left(\frac{c_{a1}}{in}\right)^{1.5}\cdot lbf \qquad V_b = 10.73\,kip$$

$$V_{cbg} := \left(\frac{A_{Vc}}{A_{Vco}}\right)\cdot\psi_{ecV}\cdot\psi_{edV}\cdot\psi_{cV}\cdot V_b \qquad V_{cbg} = 32.65\,kip$$

$$V_{cbg\_parallel} := 2\cdot V_{cbg} \qquad V_{cbg\_parallel} = 65.31\,kip$$

$$V_c := V_{cbg\_parallel}\cdot No\_Stiff \qquad V_c = 261.23\,kip$$

$$M_{n\_breakout} := V_c\cdot\left(\frac{d_{st}}{2}\right) \qquad M_{n\_breakout} = 217.69\,ft\cdot kip$$

Flexural Capacity Using Breakout Capacity

$M_{n\_bolt} = 82.83\,ft\cdot kip$

$M_{n\_pipe} = 392\,ft\cdot kip$

$M_{n\_shaft} = 296.49\,ft\cdot kip$

$M_{n\_blowout} = 337.05\,ft\cdot kip$

$M_{n\_breakout} = 217.69\,ft\cdot kip$

FAILURE EQUATIONS

Torsion

Threshold Torsion

$$A_{cp} := \pi \cdot \left(\frac{d_s}{2}\right)^2 = 706.86\,in^2$$

$$p_{cp} := \pi \cdot d_s = 94.25\,in$$

$$T_{threshold} := \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \frac{\left(A_{cp}{}^2\right)}{p_{cp}}$$

$T_{threshold} = 32.76\,ft \cdot kip$

ACI 11.6.1a

Cracking Torsion

$$T_{cr} := 4 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \left(\frac{A_{cp}{}^2}{p_{cp}}\right)$$

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

ACI R11.6.1

$$A_o := \pi \cdot \left(\frac{d_h}{2}\right)^2 = 572.56\,in^2$$

$$A_t := \pi \cdot \left(\frac{d_{hoop}}{2}\right)^2 = 0.11\,in^2$$

$\theta := 45deg$

$\theta = 0.79 \cdot rad$

$$T_{torsion} := \frac{2 \cdot A_o \cdot A_t \cdot f_{y\_hoop}}{s_{hoop}} \cdot \cot(\theta)$$

$T_{torsion} = 252.95\,ft \cdot kip$

ACI 318-05 11.6.3.6
(11-21)

$T_{n\_shaft} := T_{torsion}$

$T_{n\_shaft} = 252.95\,ft \cdot kip$

ACI 318-05 11.6.3.5
(11-20)

Torsion

$T_{n\_breakout\_plate} = 347.98\,ft \cdot kip$

$T_{n\_blowout} = 522.7\,ft \cdot kip$

$T_{n\_shaft} = 252.95\,ft \cdot kip$

DEVELOPMENT LENGTHS OF FLEXURAL REINF.

Development Length of Longitudinal Bars

Input

Longitudinal Steel

Longitudinal Steel Area $A_{long} = 0.2\,in^2$

Longitudinal Steel Diameter $d_{long} = 0.5\,in$

Yield Strength of Longitudinal Steel $f_{y\_long} = 60\,ksi$

Development Length of Longitudinal Reinforcement

$\Psi_t := 1.3$ ACI 318-05 12.5.2

$\Psi_e := 1.0$

$\Psi_s := 1.0$

$\lambda := 1.0$

$Cb\_Ktr := 2.5in$ ACI 318-05 12.2.3

$$l_{dh\_long} := \left[\left(\frac{3}{40}\right)\cdot\left(\frac{f_{y\_long}}{\sqrt{\frac{f_c}{psi}}\cdot psi}\right)\cdot\frac{(\Psi_t\cdot\Psi_e\cdot\Psi_s\cdot\lambda)}{\left(\frac{Cb\_Ktr}{d_{long}}\right)}\right]\cdot d_{long}$$

$l_{dh\_long} = 7.89\,in$

ACI 318-05 12.2.3

$l_{d\_long} := l_{dh\_long}$ $l_{d\_long} = 7.89\,in$

ACI 318-05 12.2.5

$l_{d\_1} := ceil\left(\frac{l_{d\_long}}{in}\right)\cdot in$ $l_{d\_1} = 8\,in$

Development Length of Longitudinal Bars

Length of Shaft Required

Length of Stiffeners $L = 7\,in$

Length of Breakout $l_{breakout} = 12.54\,in$

Length of Shaft $L_s = 36\,in$

Development Length of Longitudinal Reinforcement $l_{d\_1} = 8\,in$

Required Cover $c\_cover := 2.5in$

Required Length of Shaft Based on Breakout and Development Length

$l_{shaft} := l_{breakout} + c\_cover + l_{d\_1}$ $\qquad l_{shaft} = 23.04\,in$

$Check\_Shaft\_Length := if\left(L_s \ge l_{shaft}, "Sufficient", "Not\ Sufficient"\right)$

$Check\_Shaft\_Length = "Sufficient"$

Length of Shaft Required

SUPERSTRUCTURE

Superstructure Assembly Strength - Pipes

Superstructure Test Assembly Pipe

Pipe Properties - HSS 16x.500

Design Wall Thickness $\qquad t_{pipe} := .465in$

Cross Sectional Area of Pipe $\qquad A_{pipe} := 22.7in^2$

Diameter to Wall Thickness Ratio $\qquad D\_t := 34.4$

Nominal Weight $\qquad W_{pipe} := 82.85\frac{lbf}{ft}$

Moment of Inertia $\qquad I_{pipe} := 685in^4$

Elastic Section Modulus $\qquad S_{pipe} := 85.7in^3$

Radius of Gyration $\qquad r_{pipe} := 5.49in$

Plastic Section Modulus $\qquad Z_{pipe} := 112in^3$

Diameter of the Pipe $\qquad D_{pipe} := 20in$

Torsional Constant $\qquad J_{pipe} := 1370in^4$

HSS Torsional Constant $\qquad C_{pipe} := 171in^3$

Yield Strength $\qquad F_{y\_pipe} = 42\,ksi$

Ultimate Strength $\qquad F_{u\_pipe} = 58\,ksi$

Modulus of Elasticity $\qquad E := 29000ksi$

Length of Short Superstructure Pipe $\qquad L_{s\_pipe} := 17in$

Length of Long Superstructure Pipe $\qquad L_{l\_pipe} := 9ft$

Short Pipe

Design Flexural Strength

$\phi_{flexure} := .9$

AISC Spec. F1

$$M_{n\_s\_pipe} := \begin{cases} \left(F_{y\_pipe} \cdot Z_{pipe}\right) & \text{if } D\_t \le \left(.45 \cdot \dfrac{E}{F_{y\_pipe}}\right) \\ \text{"Equation Invalid"} & \text{otherwise} \end{cases}$$

$M_{n\_s\_pipe} = 392 \text{ ft} \cdot \text{kip}$

AISC Spec. F2.1

Design Shear Strength

$\phi_{shear} := .9$

AISC Spec. G1

$$F_{cr} := \begin{cases} \left[\left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}} \cdot (D\_t)^{1.25}}}\right]\right] & \text{if } \left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}} \cdot (D\_t)^{1.25}}}\right] \ge \left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right] \\ \left[\left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right]\right] & \text{otherwise} \end{cases}$$

$F_{cr\_shear} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_shear} = 25.2 \text{ ksi}$

$$V_{n\_s\_pipe} := \frac{\phi_{shear} \cdot \left(F_{cr\_shear} \cdot A_{pipe}\right)}{2}$$

$V_{n\_s\_pipe} = 257.42 \text{ kip}$

AISC Spec. G6

Design Torsional Strength

$\phi_{torsion} := .75$

AISC Spec. H3.1

$$F_{cr} := \begin{cases} \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}} \cdot (D\_t)^{1.25}}}\right] & \text{if } \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{s\_pipe}}{D_{pipe}} \cdot (D\_t)^{1.25}}}\right] \ge \dfrac{(.60 \cdot E)}{(D\_t)^{1.5}} \\ \left[\dfrac{(.60 \cdot E)}{(D\_t)^{1.5}}\right] & \text{otherwise} \end{cases}$$

$F_{cr\_torsion} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_torsion} = 25.2 \text{ ksi}$

$T_{n\_s\_pipe} := F_{cr\_torsion} \cdot C_{pipe}$

$T_{n\_s\_pipe} = 359.1 \text{ ft} \cdot \text{kip}$

AISC Spec. H3.1

Design Axial Strength

$\phi_{comp} := .90$

$$\lambda_r := .31\cdot\frac{E}{F_{y\_pipe}}$$

$\lambda_r = 214.05$

$$\lambda_p := .07\cdot\frac{E}{F_{y\_pipe}}$$

$$\lambda := \begin{cases} \text{"Compact"} & \text{if } D\_t \le \lambda_p \\ \text{"Noncompact"} & \text{if } \lambda_p < D\_t \le \lambda_r \\ \text{"Slender"} & \text{if } D\_t > \lambda_r \end{cases}$$

$\lambda =$ "Compact"

AISC Spec. B4

$k_{s\_pipe} := .5$

$$F_{e\_short} := \frac{\left(\pi^2\cdot E\right)}{\left(k_{s\_pipe}\cdot\frac{L_{s\_pipe}}{r_{pipe}}\right)^2}$$

$F_{e\_short} = 1.19\times10^5\ \text{ksi}$

AISC Equation E3-4

$$F_{cr\_short} := \begin{cases} \left[\left[.658^{\left(\frac{F_{y\_pipe}}{F_{e\_short}}\right)}\cdot F_{y\_pipe}\right]\right] & \text{if } F_{e\_short} \ge .44\cdot F_{y\_pipe} \\ \left(.877\cdot F_{e\_short}\right) & \text{if } F_{e\_short} < .44\cdot F_{y\_pipe} \end{cases}$$

$P_{n\_s\_pipe} := \phi_{comp}\cdot F_{cr\_short}\cdot A_{pipe}$

$P_{n\_s\_pipe} = 857.93\ \text{kip}$

AISC Equation E3-1

Summary for Short Pipe

Flexural Strength $M_{n\_s\_pipe} = 392\ \text{ft}\cdot\text{kip}$

Shear Strength $V_{n\_s\_pipe} = 257.42\ \text{kip}$

Torsional Strength $T_{n\_s\_pipe} = 359.1\ \text{ft}\cdot\text{kip}$

Axial Strength $P_{n\_s\_pipe} = 857.93\ \text{kip}$

Long Pipe

Design Flexural Strength

$\phi_{flexure} = 0.9$

AISC Spec. F1

$$M_{n\_1\_pipe} := \begin{cases} \left(\phi_{flexure} \cdot F_{y\_pipe} \cdot Z_{pipe}\right) & \text{if } D\_t \le \left(.45 \cdot \dfrac{E}{F_{y\_pipe}}\right) \\ \text{"Equation Invalid"} & \text{otherwise} \end{cases}$$

$M_{n\_1\_pipe} = 352.8\,\text{ft}\cdot\text{kip}$

AISC Spec. F2.1

Design Shear Strength

$\phi_{shear} = 0.9$

AISC Spec. G1

$$F_{cr} := \begin{cases} \left[\left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right]\right] & \text{if } \left[\dfrac{(1.60 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] \ge \left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right] \\ \left[\left[\dfrac{(.78 \cdot E)}{(D\_t)^{1.5}}\right]\right] & \text{otherwise} \end{cases}$$

$F_{cr\_shear} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_shear} = 25.2\,\text{ksi}$

$$V_{n\_1\_pipe} := \frac{\phi_{shear} \cdot \left(F_{cr\_shear} \cdot A_{pipe}\right)}{2}$$

$V_{n\_1\_pipe} = 257.42\,\text{kip}$

AISC Spec. G6

Design Torsional Strength

$\phi_{torsion} = 0.75$

AISC Spec. H3.1

$$F_{cr} := \begin{cases} \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] & \text{if } \left[\dfrac{(1.23 \cdot E)}{\sqrt{\dfrac{L_{1\_pipe}}{D_{pipe}}} \cdot (D\_t)^{1.25}}\right] \ge \dfrac{(.60 \cdot E)}{(D\_t)^{1.5}} \\ \left[\dfrac{(.60 \cdot E)}{(D\_t)^{1.5}}\right] & \text{otherwise} \end{cases}$$

$F_{cr\_torsion} := \min\left(F_{cr}, .6 \cdot F_{y\_pipe}\right)$

$F_{cr\_torsion} = 25.2\,\text{ksi}$

$T_{n\_1\_pipe} := F_{cr\_torsion} \cdot C_{pipe}$

$T_{n\_1\_pipe} = 359.1\,\text{ft}\cdot\text{kip}$

AISC Spec. H3.1

Design Axial Strength

$\phi_{comp} := .90$

$\lambda_r := .31 \cdot \frac{E}{F_{y\_pipe}}$ $\lambda_r = 214.05$

$\lambda_p := .07 \cdot \frac{E}{F_{y\_pipe}}$

$\lambda := \begin{vmatrix} \text{"Compact"} \quad \text{if } D\_t \le \lambda_p \\ \text{"Noncompact"} \quad \text{if } \lambda_p < D\_t \le \lambda_r \\ \text{"Slender"} \quad \text{if } D\_t > \lambda_r \end{vmatrix}$ $\lambda = \text{"Compact"}$

AISC Spec. B4

$k_{long\_pipe} := 2.0$

$F_{e\_long} := \frac{\left(\pi^2 \cdot E\right)}{\left(k_{long\_pipe} \cdot \frac{L_{1\_pipe}}{r_{pipe}}\right)^2}$ $F_{e\_long} = 184.9\,\text{ksi}$

AISC Equation E3-4

$F_{cr\_long} := \begin{vmatrix} \left[\left[.658^{\left(\frac{F_{y\_pipe}}{F_{e\_long}}\right)} \cdot F_{y\_pipe}\right]\right] \quad \text{if } F_{e\_long} \ge .44 \cdot F_{y\_pipe} \\ \left(.877 \cdot F_{e\_long}\right) \quad \text{if } F_{e\_long} < .44 \cdot F_{y\_pipe} \end{vmatrix}$

$P_{n\_1\_pipe} := \phi_{comp} \cdot F_{cr\_long} \cdot A_{pipe}$ $P_{n\_1\_pipe} = 780.24\,\text{kip}$

AISC Equation E3-1

Summary for Long Pipe

Flexural Strength $M_{n\_1\_pipe} = 352.8\,\text{ft} \cdot \text{kip}$

Shear Strength $V_{n\_1\_pipe} = 257.42\,\text{kip}$

Torsional Strength $T_{n\_1\_pipe} = 359.1\,\text{ft} \cdot \text{kip}$

Axial Strength $P_{n\_1\_pipe} = 780.24\,\text{kip}$

Superstructure Assembly Strength - Pipes

Superstructure Assembly Strength - Connecting Plates

Superstructure Test HSS Connection Plate

Plate Properties - PL1/2" x 32" x 24"

Plate thickness $t_p := .5in$

Plate length $h_p := 32in$

Plate width $b_p := 24in$

Yield strength $F_{y\_plate} := 50ksi$

Ultimate strength $F_{u\_plate} := 62ksi$

Design Tensile Strength

$\phi_{t\_yield} := .9$

$P_{n\_yield} := \phi_{t\_yield} \cdot F_{y\_plate} \cdot t_p \cdot b_p$ $\qquad P_{n\_yield} = 540\,kip$

AISC Spec. D2a

AISC Table D3.1

$U := 1.0$

AISC D3.2

$A_n := t_p \cdot b_p$

$A_e := U \cdot A_n$ $\qquad A_e = 12\,in^2$

AISC D3.3

$\phi_{t\_rupt} := .75$

$P_{n\_rupture} := \phi_{t\_rupt} \cdot A_e \cdot F_{u\_plate}$ $\qquad P_{n\_rupture} = 558\,kip$

$$P_{n\_plate} := \begin{cases} P_{n\_yield} & \text{if } P_{n\_yield} \le P_{n\_rupture} \\ P_{n\_rupture} & \text{otherwise} \end{cases}$$

$P_{n\_plate} = 540\,kip$

AISC D2b

Design Flexural Strength

$\phi_{flexure} = 0.9$

$A_g := t_p \cdot b_p$ $\qquad A_g = 12\,in^2$

$L_b := 16in$

$$I_p := \frac{b_p \cdot t_p^{\,3}}{3}$$

$$c := \frac{t_p}{2}$$

$$S_p := \frac{I_p}{c}$$ $\qquad S_p = 4\,in^3$

$M_y := S_p \cdot F_{y\_plate}$ $\qquad M_y = 16.67\,ft \cdot kip$

$$Z_p := \left(t_p \cdot \frac{b_p}{2}\right) \cdot \frac{t_p}{2}$$ $Z_p = 1.5\,in^3$

$$M_p := F_{y\_plate} \cdot Z_p$$ $M_p = 6.25\,ft \cdot kip$

$$M_{p\_yield} := \begin{cases} (1.6 \cdot M_y) & \text{if } 1.6 \cdot M_y \le M_p \\ M_p & \text{otherwise} \end{cases}$$ $M_{p\_yield} = 6.25\,ft \cdot kip$

$$LTB\_Equation\_Check := \begin{cases} \text{"Equation F11-2"} & \text{if } \frac{(.08 \cdot E)}{F_{y\_plate}} < \left[\frac{(L_b \cdot b_p)}{t_p^{\,2}}\right] \le \frac{(1.9 \cdot E)}{F_{y\_plate}} \\ \text{"Equation F11-3"} & \text{if } \left[\frac{(L_b \cdot b_p)}{t_p^{\,2}}\right] > \frac{(1.9 \cdot E)}{F_{y\_plate}} \end{cases}$$

LTB_Equation_Check = "Equation F11-3"

$C_b := 1.0$ AISC F1

$$F_{cr} := \frac{(1.9 \cdot E \cdot C_b)}{\frac{(L_b \cdot b_p)}{t_p^{\,2}}}$$ $F_{cr} = 35.87\,ksi$

$$M_{n\_ltb} := \begin{cases} (F_{cr} \cdot S_p) & \text{if } LTB\_Equation\_Check = \text{"Equation F11-3"} \\ \left[C_b \cdot \left[1.52 - .274 \cdot \left(L_b \cdot \frac{b_p}{t_p^{\,2}}\right) \cdot \frac{F_{y\_plate}}{E}\right]\right] \cdot kip \cdot in & \text{otherwise} \end{cases}$$ $M_{n\_ltb} = 11.96\,ft \cdot kip$

$$M_{n\_plate} := \phi_{flexure} \cdot \begin{cases} M_{n\_ltb} & \text{if } M_{n\_ltb} \le M_{p\_yield} \\ M_{p\_yield} & \text{otherwise} \end{cases}$$ $M_{n\_plate} = 5.62\,ft \cdot kip$

Design Torsional Strength

$\phi_{torsion} = 0.75$

$$M_{t\_plate} := \phi_{torsion} \cdot F_{y\_plate} \cdot .6$$ $M_{t\_plate} = 22.5\,ksi$

Summary for Plate Connector

Tensile Strength $P_{n\_plate} = 540\,kip$

Flexural Strength $M_{n\_plate} = 5.62\,ft \cdot kip$

Torsional Strength $M_{t\_plate} = 22.5\,ksi$

Weld Design

$V_{weld} := \dfrac{T_{n\_blowout}}{Tors\_Moment\_Arm}$ $V_{weld} = 58.08\,kip$

$t_p = 0.5\,in$

$t_{pipe} = 0.47\,in$

$Weld\_Size := \dfrac{3}{16}in$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$ AISC Spec. J2 Table J2.5

$\phi_{weld} := .75$

$Throat := .707 \cdot Weld\_Size$

$\phi R_{n\_weld} := \phi_{weld} \cdot Throat \cdot F_W$ $\phi R_{n\_weld} = 4.18 \cdot \dfrac{kip}{in}$

$\phi R_{n\_yield} := .6 \cdot F_{y\_pipe} \cdot t_p$ $\phi R_{n\_yield} = 12.6 \cdot \dfrac{kip}{in}$

$\phi R_{n\_rupture} := .45 \cdot F_{u\_pipe} \cdot t_p$ $\phi R_{n\_rupture} = 13.05 \cdot \dfrac{kip}{in}$

$\phi R_n := \min\left(\phi R_{n\_weld}, \phi R_{n\_yield}, \phi R_{n\_rupture}\right)$ $\phi R_n = 4.18 \cdot \dfrac{kip}{in}$

$Required\_Length\_Each\_Side := \dfrac{V_{weld}}{\phi R_n}$ $Required\_Length\_Each\_Side = 13.91\,in$

$ceil\left(\dfrac{Required\_Length\_Each\_Side}{in}\right) \cdot in = 14\,in$

Superstructure Assembly Strength - Connecting Plates

Base Connection

Superstructure Test Base Connection Plate

Plate Properties - Annular Plate

Plate diameter $B_p := 24in$

Yield strength $f_{y\_ann} := 50ksi$

Ultimate strength $f_{u\_ann} := 75ksi$

Thickness of plate $t_{plate} := 1.00in$

Bolt Properties - D1" ASTM A325

Center to Center Radius of Bolts $r_b := 12in$

Number of Bolts $No\_Bolts := 12$

Field Strength of Bolts $f_{y\_bolt\_field} := 55ksi$

Ultimate Strength of Bolts $f_{u\_bolt} := 105ksi$

Radius of the pipe $r_p := \frac{D_{pipe}}{2} = 10\text{ in}$

Diameter of the bolt $d_{bolt} := 1.50in$

Bolt Bearing Strength

$$L_c := \frac{\left(B_p - d_b - .5 \cdot d_{bolt}\right)}{2} = 1.62\text{ in}$$

$$\phi_{shear} := .75$$

$$\phi R_n := 1.2 \cdot F_{u\_plate} \cdot L_c \cdot t_{plate} \qquad \phi R_n = 120.9\,\text{kip}$$

$$Rn\_parallel := 2 \cdot \phi R_n \qquad Rn\_parallel = 241.8\,\text{kip}$$

$$T_{n\_bolt\_bearing} := No\_Bolts \cdot Rn\_parallel \cdot \left(\frac{d_b}{2}\right) \qquad T_{n\_bolt\_bearing} = 2.42 \times 10^3\,\text{ft} \cdot \text{kip}$$

$$Check\_Bolt\_Bearing := \begin{cases} \text{"Sufficient Strength"} & \text{if } T_{n\_bolt\_bearing} \geq T_{n\_blowout} \\ \text{"Insufficient Strength"} & \text{otherwise} \end{cases}$$

$Check\_Bolt\_Bearing =$ "Sufficient Strength"

Check Bolt Spacing

$$s_{req} := 2.67 \cdot d_{bolt} = 4\text{ in}$$

$$s_{actual} := \frac{\pi \cdot d_b}{12} = 5.24\text{ in}$$

$$Check\_Bolt\_Spacing := \begin{cases} \text{"Sufficient"} & \text{if } s_{actual} \geq s_{req} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

$Check\_Bolt\_Spacing =$ "Sufficient"

Check Bolt Shear

$$A_b := \pi \cdot \left(.5 \cdot d_{bolt}\right)^2 = 1.77\text{ in}^2$$

$$F_{nv} := .4 \cdot 120ksi = 48\text{ ksi}$$

$$\phi V_n := \phi_{shear} \cdot A_b \cdot F_{nv} \qquad \phi V_n = 63.62\,\text{kip}$$

$$V_{n\_parallel} := \phi V_n \cdot 2 \qquad V_{n\_parallel} = 127.23\,\text{kip}$$

$T_{bolt\_shear} := No\_Bolts \cdot V_{n\_parallel} \cdot \left(\frac{d_b}{2}\right)$ $\quad$ $T_{bolt\_shear} = 1.27 \times 10^3\ ft \cdot kip$

$Check\_Bolt\_Shear := \begin{cases} \text{"Sufficient Strength"} & \text{if } T_{bolt\_shear} \geq T_{n\_blowout} \\ \text{"Insufficient Strength"} & \text{otherwise} \end{cases}$

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

Weld Design

Weld Connecting Annular Plate to Pipe

$t_{pipe} = 0.47\ in$

$Weld\_Size := \frac{3}{8} in$ $\quad$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$ $\quad$ AISC Spec. J2 Table J2.5

$Throat := .707 \cdot Weld\_Size$

$R_{n\_weld} := Throat \cdot F_W$ $\quad$ $R_{n\_weld} = 11.14 \cdot \frac{kip}{in}$

$R_{n\_yield} := .6 \cdot F_{y\_pipe} \cdot t_{pipe}$ $\quad$ $R_{n\_yield} = 11.72 \cdot \frac{kip}{in}$

$R_{n\_rupture} := .45 \cdot F_{u\_pipe} \cdot t_{pipe}$ $\quad$ $R_{n\_rupture} = 12.14 \cdot \frac{kip}{in}$

$R_n := \min(R_{n\_weld}, R_{n\_yield}, R_{n\_rupture})$ $\quad$ $R_n = 11.14 \cdot \frac{kip}{in}$

$R_{weld} := R_n \cdot \pi \cdot d_{pipe}$ $\quad$ $R_{weld} = 559.72\ kip$

$T_{weld} := R_{weld} \cdot \frac{d_{pipe}}{2}$ $\quad$ $T_{weld} = 373.15\ ft \cdot kip$

$M_{weld} := R_{weld} \cdot \frac{d_{pipe}}{2}$ $\quad$ $M_{weld} = 373.15\ ft \cdot kip$

Base Connection

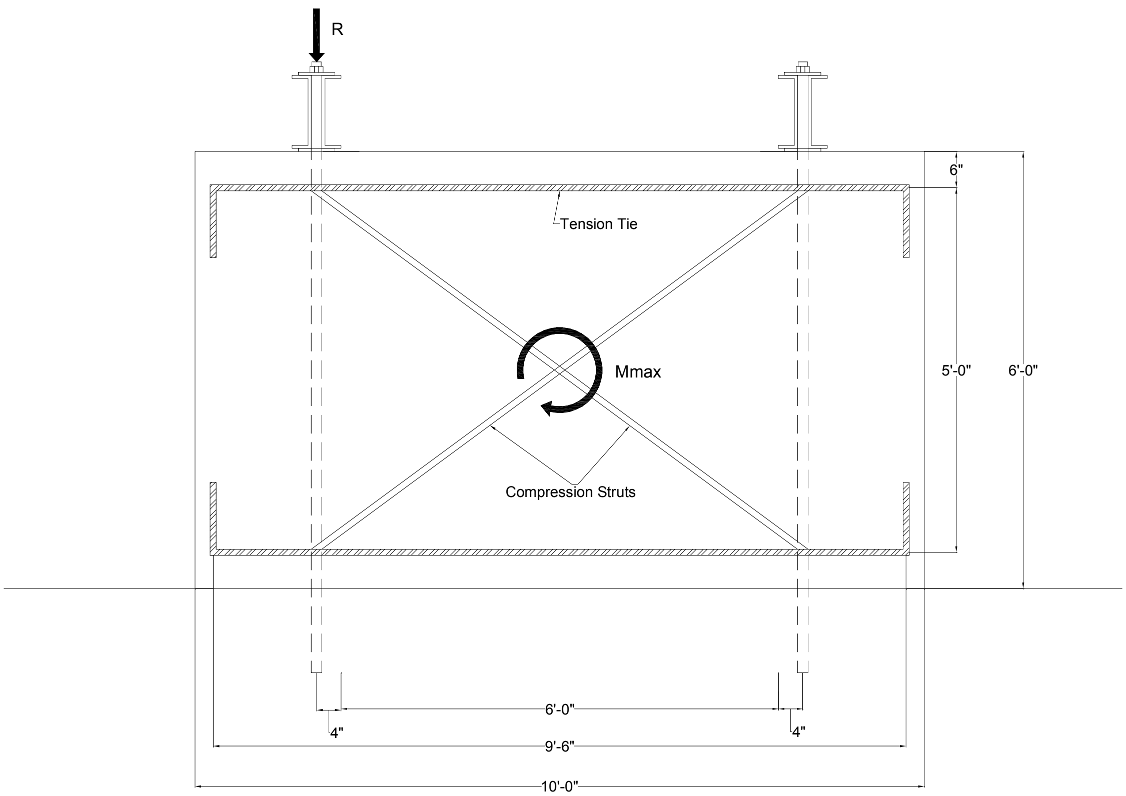
$T_{n\_s\_pipe} = 359.1\ ft \cdot kip$

$T_{weld} = 373.15\ ft \cdot kip$

$M_{weld} = 373.15\ ft \cdot kip$

CONCRETE BLOCK

Concrete Block Design - Strut-and-Tie Model



Based on ACI 318 Appendix A

$V_{max} := \dfrac{T_{n\_breakout\_plate}}{Tors\_Moment\_Arm}$ $\qquad V_{max} = 38.66\,kip$

$M_{max} := V_{max} \cdot Flex\_Moment\_Arm$ $\qquad M_{max} = 309.32\,ft \cdot kip$

$d := 6ft + 8in$ $\qquad d = 80\,in$

$R := \dfrac{M_{max}}{d}$ $\qquad R = 46.4\,kip$

Node A

$\theta := \operatorname{atan}\left(5\dfrac{ft}{d}\right)$ $\qquad \theta = 36.87 \cdot deg$

$C := \dfrac{R}{\sin(\theta)}$ $\qquad C = 77.33\,kip$

$T := C \cdot \cos(\theta)$ $\qquad T = 61.86\,kip$

Check Reinforcement

$No\_Bars\_Block\_Reinf := 3$

$Block\_Reinf\_Bar\_No := 8$

$f_{y\_block\_reinf} := 60ksi$

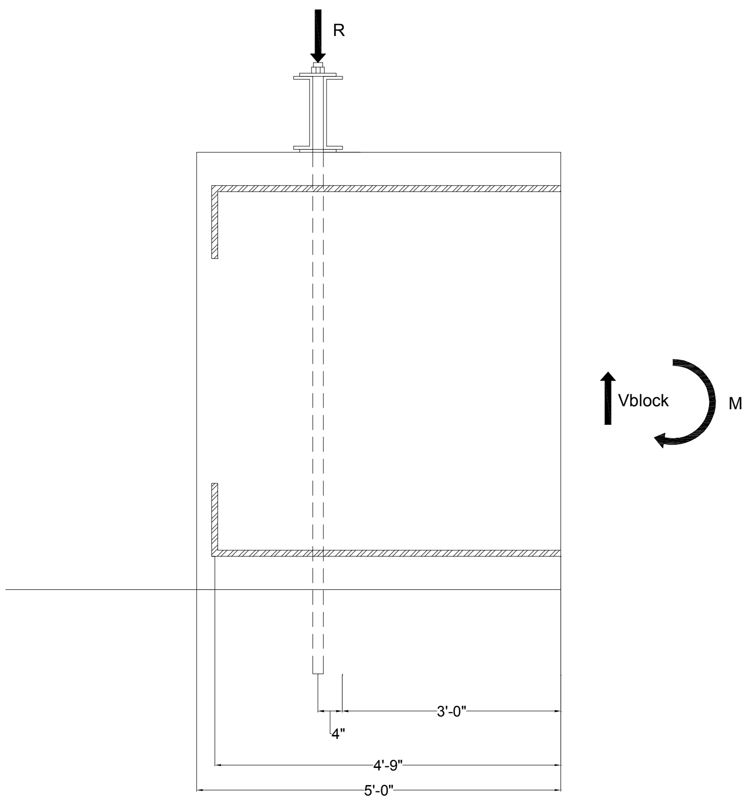
$A_{block\_reinf} := No\_Bars\_Block\_Reinf \cdot \pi \cdot \left(\frac{Block\_Reinf\_Bar\_No \div 8}{2}\right)^2 in^2$ $\quad A_{block\_reinf} = 2.36\, in^2$

$$Check\_Block\_Reinf\_A := \begin{array}{|l} \text{"Sufficient"} \quad \text{if } A_{block\_reinf} \cdot f_{y\_block\_reinf} \geq T \\ \text{"Not Sufficient"} \quad \text{otherwise} \end{array}$$

$Check\_Block\_Reinf\_A = \text{"Sufficient"}$

Concrete Block Design - Strut-and-Tie Model

Concrete Block Design - Beam Theory



$V_{block} := R$ $\quad V_{block} = 46.4\, kip$

$M_{block} := R \cdot (3ft + 4in)$ $\quad M_{block} = 154.66\, ft \cdot kip$

From strut-and-tie model...

$A_{block\_reinf} = 2.36\, in^2$

$f_{y\_block\_reinf} = 60\, ksi$

Check Shear

$$Check\_Shear\_B := \begin{cases} \text{"Sufficient"} & \text{if } A_{block\_reinf} \cdot f_{y\_block\_reinf} \geq V_{block} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

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

Check Flexure

$b_{block} := 30in$

$h_{block} := 6ft$

$d_{block} := 5.5ft$

$T_{given} := A_{block\_reinf} \cdot f_{y\_block\_reinf}$ $\qquad T_{given} = 141.37\,kip$

$C(a) := .85 \cdot f_c \cdot b_{block} \cdot a$

$P(a) := C(a) - T_{given}$

$a := root(P(a), a, 0in, h_{block})$ $\qquad a = 1.01\,in$

$\beta_1(f_c) = 0.78$

$$c := \frac{a}{\beta_1(f_c)}$$ $\qquad c = 1.3\,in$

$$M_{n\_block} := T_{given} \cdot \left(d_{block} - \frac{a}{2}\right)$$

$M_{n\_block} = 771.61\,ft \cdot kip$

$$Check\_Flexure\_B := \begin{cases} \text{"Sufficient"} & \text{if } M_{n\_block} \geq M_{block} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

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

Required Hook Length for a #8 bar

$$Hook\_No\_8 := 12 \cdot \left(\frac{Block\_Reinf\_Bar\_No}{8} \cdot in\right)$$

$Hook\_No\_8 = 12\,in$

ACI 318-05 Fig. 12.5

Concrete Block Design - Beam Theory

Summary of Concrete Block Reinforcement

$Block\_Reinf\_Bar\_No = 8$

$No\_Bars\_Block\_Reinf = 3$

$Check\_Block\_Reinf\_A = \text{"Sufficient"}$

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

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

Summary of Concrete Block Reinforcement

Tie-Down Design

Block Properties

Width of the block $b_{block} = 30\,in$

Height of the block $h_{block} = 6 \cdot ft$

Length of the block $l_{block} := 10ft$

Diameter of the shaft $d_s = 30\,in$

Length of the shaft $l_{shaft} := 36in$

Weight of concrete $w_c := 150pcf$

Maximum shear applied $V_{max} = 38.66\,kip$

Channel Assembly - 2 C12x30 Channels with 1.75" between

Moment of inertia about strong axis $I_x := 162in^4$

$S_x := 27.0in^3$

Radius of gyration about strong axis $r_x := 4.29in$

$Z_x := 33.8in^3$

Cross sectional area $A_{channel} := 8.81in^2$

Moment of inertia about weak axis $I_y := 5.12in^4$

Radius of gyration about weak axis $r_y := .762in$

$x\_bar := .674in$

Yield strength $F_{y\_channel} := 50ksi$

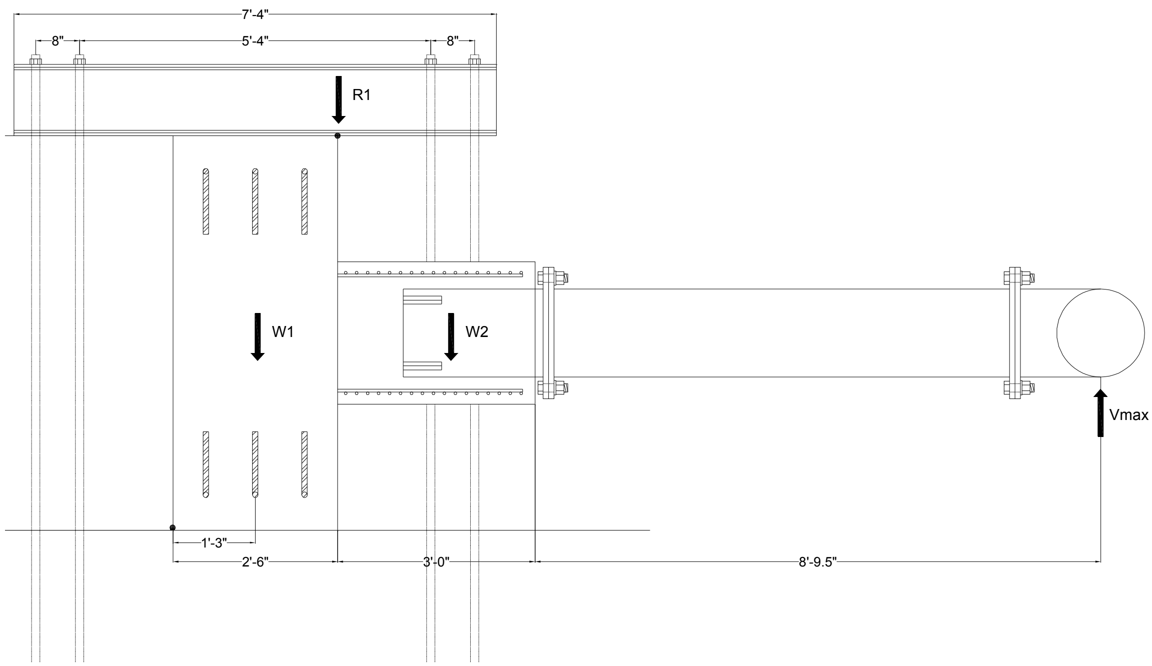
Modulus of elasticity $E = 2.9 \times 10^4\,ksi$

Web thickness $t_w := .510in$

Flange width $b_f := 3.17in$

Flange thickness $t_f := .501in$

Depth $h := 12in$



Calculate self-weight of block

$$W_1 := h_{block} \cdot b_{block} \cdot l_{block} \cdot w_c$$ $W_1 = 22.5\,kip$

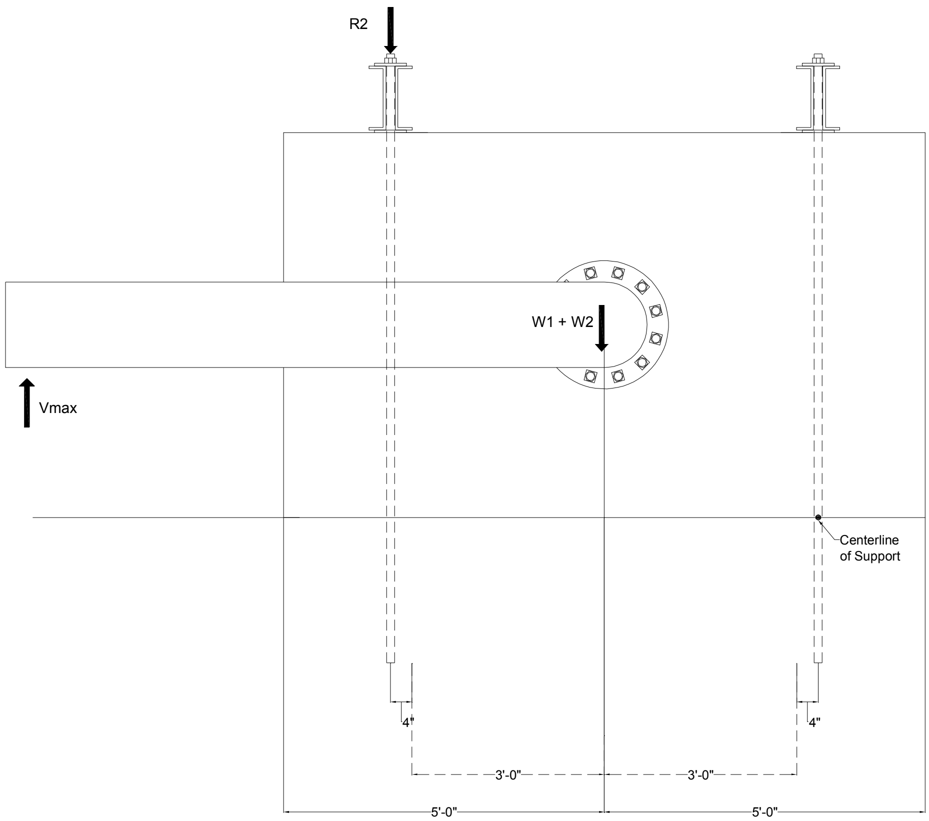
$$W_2 := l_{shaft} \cdot \left(\frac{\pi}{4} \cdot d_s^{\,2}\right) \cdot w_c$$ $W_2 = 2.21\,kip$

Calculate the Load that the Tie-down must resist in each direction

$$R_1 := \frac{-\left[W_2 \cdot \left(\frac{l_{shaft}}{2} + b_{block}\right) + W_1 \cdot \left(\frac{b_{block}}{2}\right) - V_{max} \cdot \left(l_{shaft} + 17.5in + b_{block}\right)\right]}{b_{block}}$$

$R_1 = 92.83\,kip$

$$R_2 := \frac{\left[V_{max} \cdot \left(L_{1\_pipe} + 3ft + 4in\right) - \left(W_1 + W_2\right) \cdot (3ft + 4in)\right]}{6.67ft}$$ $R_2 = 59.15\,kip$



Total Load that the Tie-down must support

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

$R = 105.56\,\text{kip}$

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

$\text{Floor\_Capacity} := 200\text{kip}$

$$\text{Check\_Floor\_Capacity} := \begin{vmatrix} \text{"Sufficient"} & \text{if Floor\_Capacity} \geq R \\ \text{"Insufficient"} & \text{otherwise} \end{vmatrix}$$

$\text{Check\_Floor\_Capacity} = \text{"Sufficient"}$

Check Bearing Strength of Concrete

Conservatively assume that the load bears on 1 in. of concrete across the length of the block...

$A_{bearing} := l_{block} \cdot 1\text{in}$

$A_{bearing} = 120\,\text{in}^2$

$\phi_{bearing} := .65$

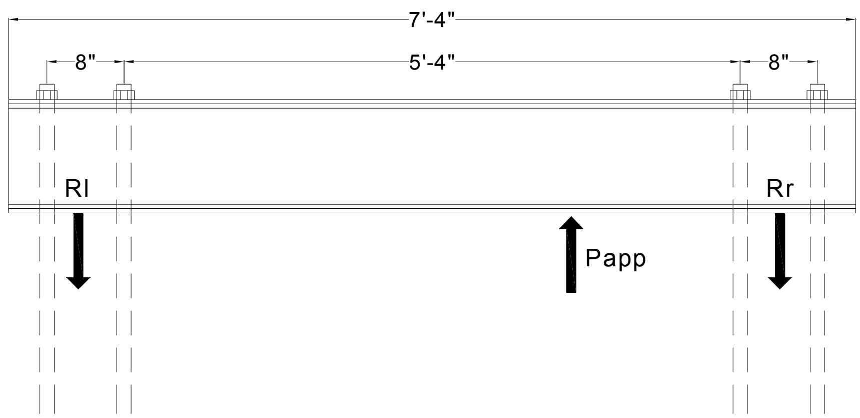
$\text{Bearing\_Strength} := \phi_{bearing} \cdot .85 \cdot f_c \cdot A_{bearing}$

$\text{Bearing\_Strength} = 364.65\,\text{kip}$

$$\text{Check\_Bearing\_Capacity} := \begin{vmatrix} \text{"Sufficient"} & \text{if Bearing\_Strength} \geq 2 \cdot R \\ \text{"Insufficient"} & \text{otherwise} \end{vmatrix}$$

$\text{Check\_Bearing\_Capacity} = \text{"Sufficient"}$

Required Capacity of the Channel Assembly



$P_{app} := R$

$P_{app} = 105.56\,kip$

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

$R_L = 35.19\,kip$

$R_R := P_{app} - R_L$

$R_R = 70.37\,kip$

$a := 4ft$

$b := 2ft$

$M_{max\_tiedown} := \dfrac{P_{app} \cdot a \cdot b}{a + b}$

$M_{max\_tiedown} = 140.75\,ft \cdot kip$

Flexural Capacity of Each Channel

$M_{n\_tiedown} := F_{y\_channel} \cdot (2 \cdot Z_x)$

$M_{n\_tiedown} = 281.67\,ft \cdot kip$

AISC Spec. F2.1 (F2-1)

$$Check\_Flexure\_Channels := \begin{cases} \text{"Sufficient"} & \text{if } M_{n\_tiedown} \geq M_{max\_tiedown} \\ \text{"Insufficient"} & \text{otherwise} \end{cases}$$

Check_Flexure_Channels = "Sufficient"

Buckling Check of Each Channel

Treated as 2 separate channels

$\lambda_f := \dfrac{b_f}{2 \cdot t_f}$

$\lambda_f = 3.16$

$\lambda_w := \dfrac{h}{t_w}$

$\lambda_w = 23.53$

$\lambda_{pf} := .38 \cdot \sqrt{\frac{E}{F_{y\_channel}}}$

$\lambda_{pf} = 9.15$

AISC Spec. B4

$\lambda_{pw} := 3.76 \cdot \sqrt{\frac{E}{F_{y\_channel}}}$

$\lambda_{pw} = 90.55$

AISC Spec. B4

$$\text{Check\_Flange\_Compact} := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pf} > \lambda_f \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

Check_Flange_Compact = "Compact"

AISC Spec. B4

$$\text{Check\_Web\_Compact} := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pw} > \lambda_w \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

Check_Web_Compact = "Compact"

AISC Spec. B4

Bracing Check of Each Channel

$L_b := b$

$L_b = 2 \cdot ft$

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_{y\_channel}}}$

$L_p = 2.69 \cdot ft$

AISC Spec. F2.2 (F2-5)

$$\text{Bracing\_Check} := \begin{cases} \text{"Braced"} & \text{if } L_p > L_b \\ \text{"Unbraced"} & \text{otherwise} \end{cases}$$

Bracing_Check = "Braced"

Checking Channel Assembly

$I_{y\_unit} := 2 \cdot \left[I_y + \left[A_{channel} \cdot \left[x\_bar + \left(\frac{1.75in}{2} \right) \right]^2 \right] \right]$

$I_{y\_unit} = 52.52\, in^4$

$r_{y\_unit} := \sqrt{\frac{I_{y\_unit}}{A_{channel}}}$

$r_{y\_unit} = 2.44\, in$

$L_{p\_unit} := 1.76 \cdot r_{y\_unit} \cdot \sqrt{\frac{E}{F_{y\_channel}}}$

$L_{p\_unit} = 103.49\, in$

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

$b_{f\_unit} = 8.09\, in$

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

$t_{w\_unit} = 2.77\, in$

$\lambda_{f\_unit} := \frac{b_{f\_unit}}{2 \cdot t_f}$

$\lambda_{f\_unit} = 8.07$

$\lambda_{w\_unit} := \frac{h}{t_{w\_unit}}$

$\lambda_{w\_unit} = 4.33$

$$Check\_Flange\_Compact\_Unit := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pf} > \lambda_{f\_unit} \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

$Check\_Flange\_Compact\_Unit = \text{"Compact"}$

$$Check\_Web\_Compact\_Unit := \begin{cases} \text{"Compact"} & \text{if } \lambda_{pw} > \lambda_{w\_unit} \\ \text{"Not Compact"} & \text{otherwise} \end{cases}$$

$Check\_Web\_Compact\_Unit = \text{"Compact"}$

$$Bracing\_Check\_Unit := \begin{cases} \text{"Braced"} & \text{if } L_{p\_unit} > L_b \\ \text{"Unbraced"} & \text{otherwise} \end{cases}$$

$Bracing\_Check\_Unit = \text{"Braced"}$

Weld Design

$V_{weld} := \max(R_L, R_R)$ $\qquad V_{weld} = 70.37\,kip$

$t_{pl} := .5in$

$b_{pl} := 5in$

$$Q := (t_{pl} \cdot b_{pl}) \cdot \left[\left(\frac{t_{pl}}{2} \right) + \left(\frac{h}{2} \right) \right]$$

$Q = 15.62\,in^3$

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

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

$$Required\_Load\_per\_Foot := \frac{V_{weld} \cdot Q}{I_{weld}}$$

$Required\_Load\_per\_Foot = 25.4 \cdot \frac{kip}{ft}$

$Weld\_Size := \frac{3}{8} in$

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode} = 42\,ksi$

AISC Spec. J2
Table J2.5

$\phi_{weld} := .75$

$Throat := .707 \cdot Weld\_Size$

$$Required\_Length\_Per\_Foot := \frac{Required\_Load\_per\_Foot}{\phi_{weld} \cdot F_W \cdot Throat}$$

$Required\_Length\_Per\_Foot = 3.04 \cdot \frac{in}{ft}$

Specify 4" per foot of Weld

AISC Spec. J2.4
(J2-3)

Tie-Down Design

$T_{max} := T_{n\_breakout\_plate} = 347.98\,ft \cdot kip$

$V_{max} = 38.66\,kip$

$M_{max} = 309.32\,ft \cdot kip$

APPENDIX C
TEST DATA

Torsion Test Data

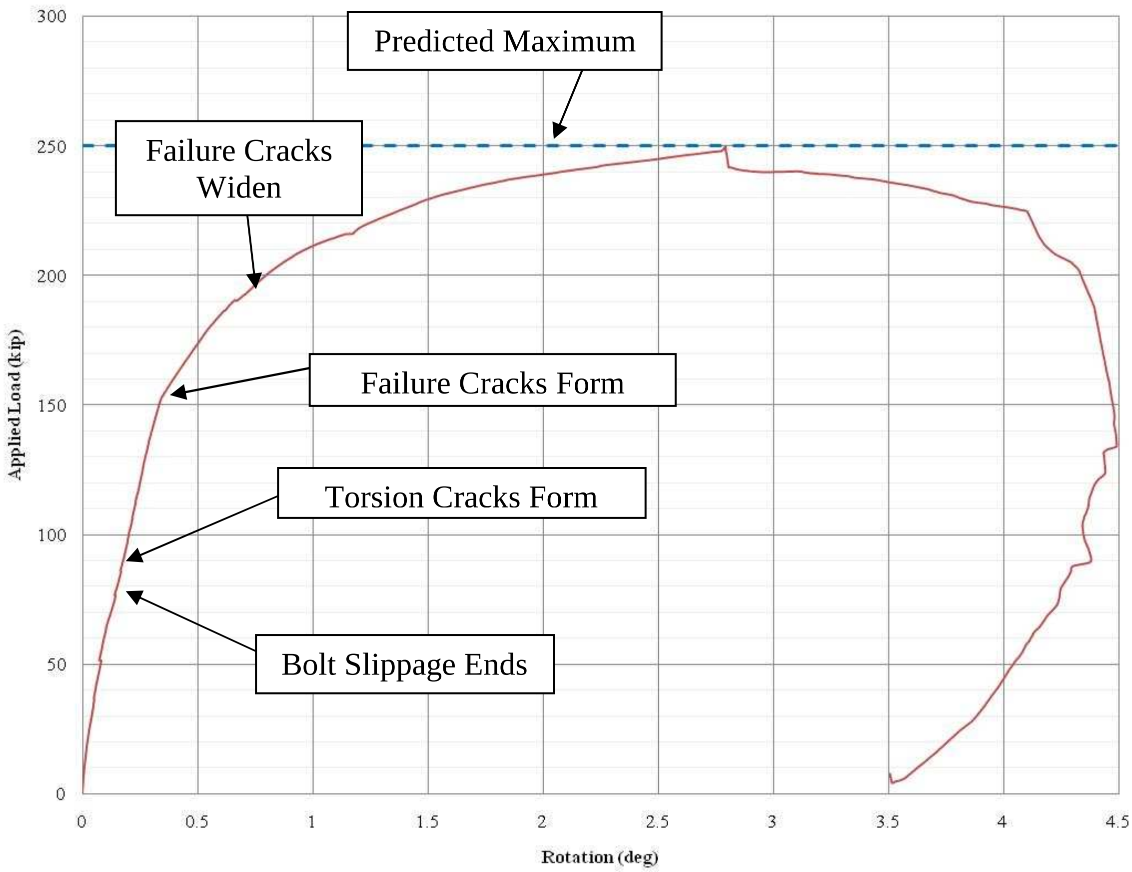


Figure C-1. Moment and rotation plot for base plate of torsion test

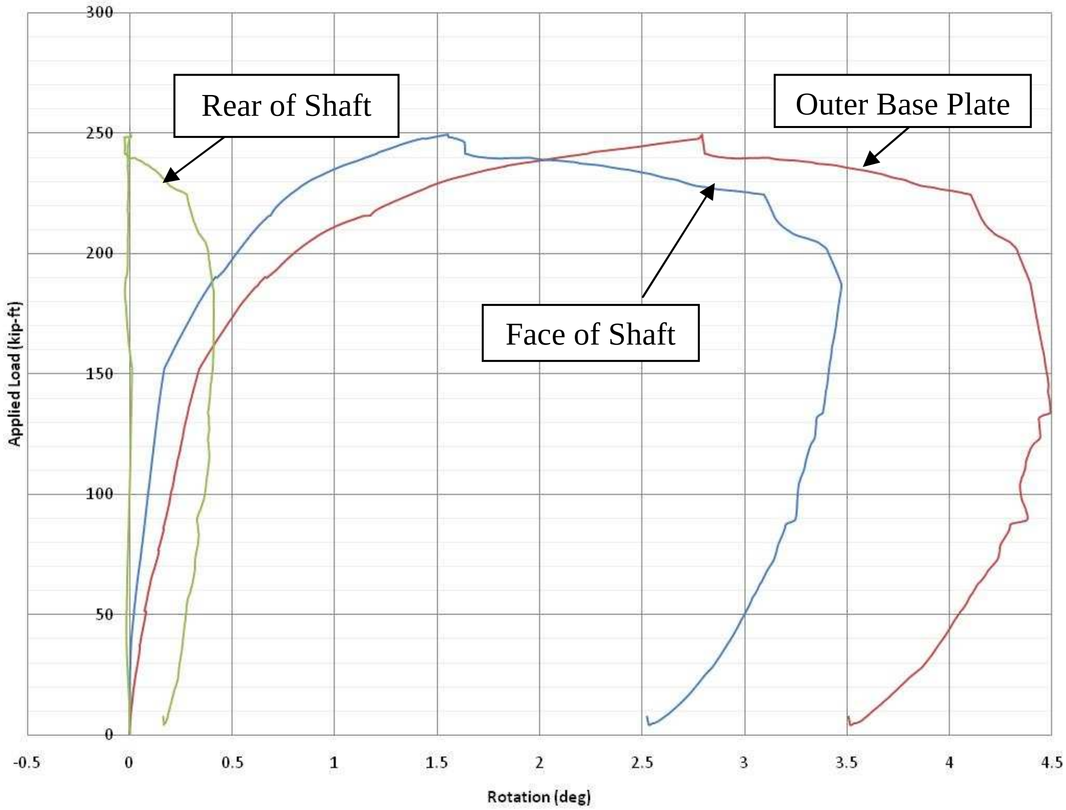


Figure C-2. Moment and torsional rotation plot for torsion test

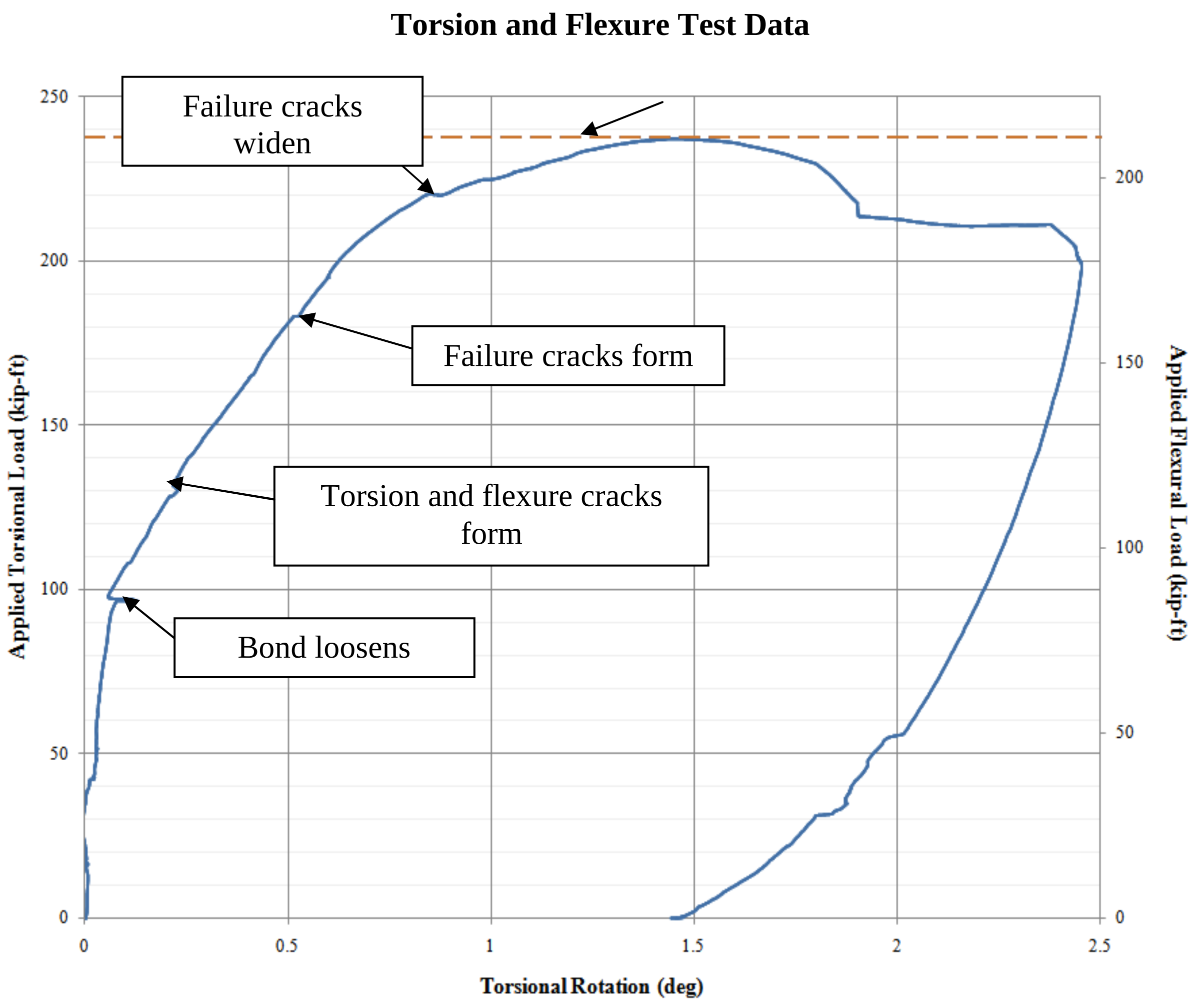


Figure C-3. Load and torsional rotation of base plate for torsion and flexure test

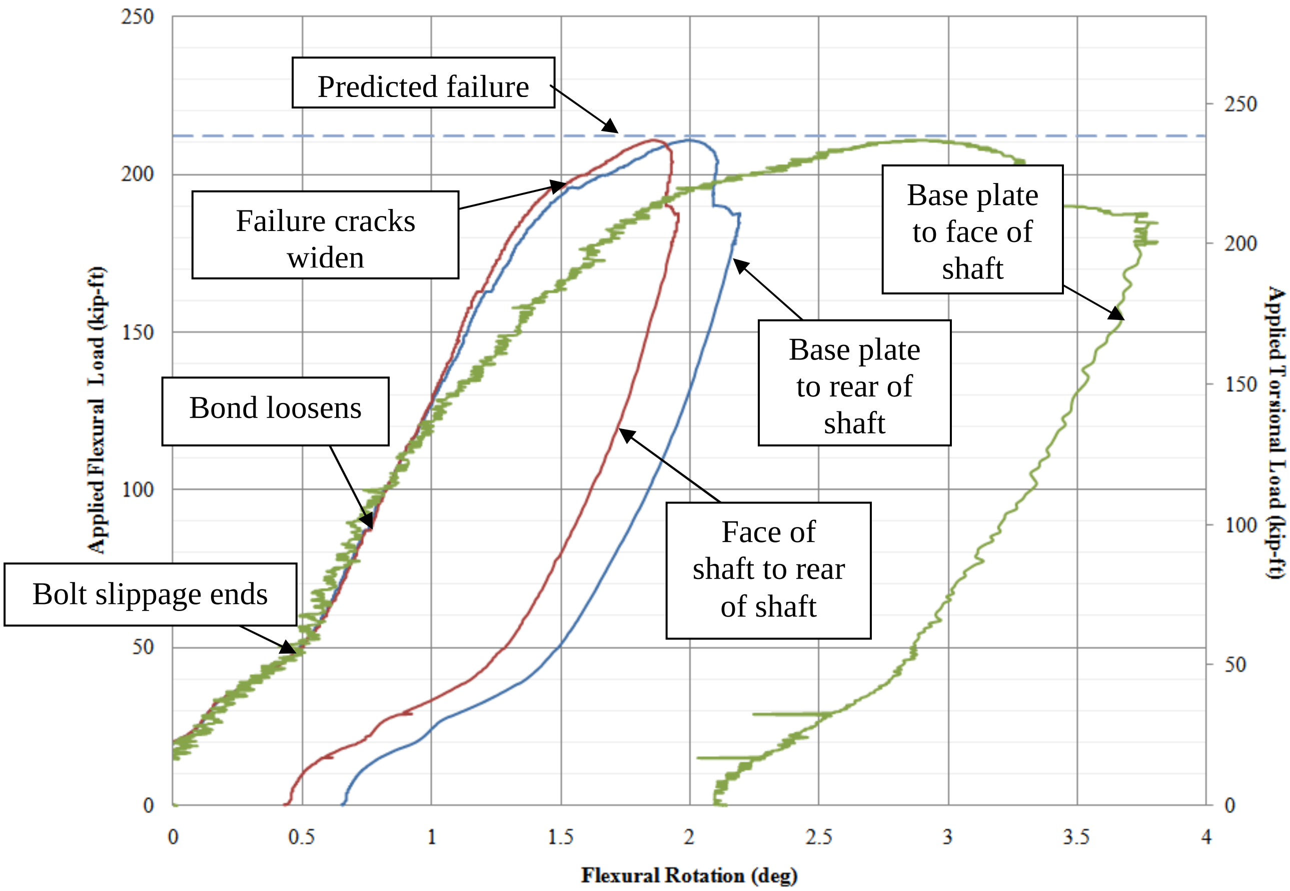


Figure C-4. Load and flexural rotation for torsion and flexure test

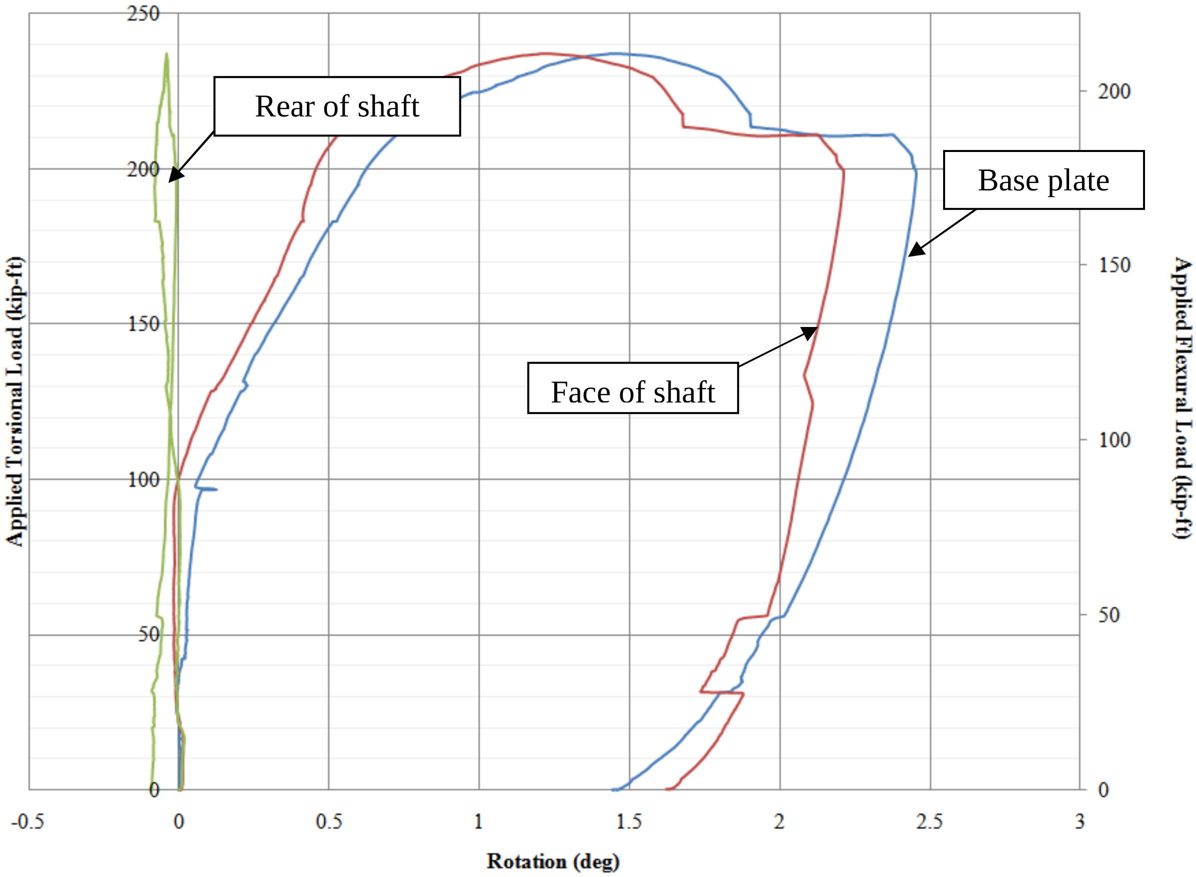


Figure C-5. Load and torsional rotation for torsion and flexure test

APPENDIX D
DESIGN GUIDELINES

For the purposes of these design guidelines, the design of a typical sign/signal can be divided into three areas. The first of these would be the superstructure, which would include the vertical column, horizontal member, connection between the horizontal and vertical members, and any other design components above the base connection. The second design area would be the interface with the foundation or the base connection. This second design area can be subdivided into the superstructure interface and the foundation interface. The last of the design areas would be the foundation. These design guidelines will only cover the base connection and the foundation. It is assumed that the superstructure will be designed appropriately using other FDOT design guidelines. The FDOT offers a MathCAD worksheet program called MastArm v4.3 on their website that includes the design of the superstructure including the horizontal arm, connection to the vertical column, the vertical column, and the annular base plate.

For the concerns of this design guideline the interface with the foundation and the foundation will need to be designed for shear, torsion, and flexure. The foundation interface will need to be designed to match the annular base plate from the MastArm v4.3 output and the connecting bolts and welds will need to be designed. The foundation will need to have the embedded steel pipe and plates, their welded connections, the concrete, and the concrete reinforcement designed. A design example will be displayed on the following pages.

Base Connection Design

For the design recommended in these design guidelines the information obtained from the MastArm v4.3 program will be the basis for the design. See Figure D-1 for a clarification of terminology. For the design of the base plate, the designer will need the following information from the FDOT program or their own design:

- Design loads for shear, flexure, and torsion (V_u, T_u, and M_u)
- Superstructure interface base plate sized

The designer should then use this information and their own design knowledge to design the following for the base connection:

- Size the foundation interface base plate to match the superstructure interface base plate
- Size the leveling bolts for design shear, flexure, and torsion
- Ensure shear capacity of bolt holes exceeds design shear
- Size the leveling nuts

Embedded Pipe Design

For the design of the embedded pipe, the designer will need the following information from the FDOT program or their own design:

- Design loads for shear, flexure, and torsion (V_u, T_u, and M_u)

- Superstructure monopole sized

- Welded connection from superstructure monopole to superstructure interface base plate sized

The designer should then use this information and their own design knowledge to design the following for the embedded pipe:

- Size the cross section of the embedded pipe to have the same diameter and wall thickness as the superstructure monopole. The embedded pipe can be either a tapered section or an HSS pipe.

- Size the welded connection from the embedded pipe to the foundation interface base plate to be the same as the welded connection from the superstructure monopole to the superstructure interface base plate.

Embedded Pipe and Torsion Plates Design

For the design of the torsion plates, the designer will need the following information from the FDOT program or their own design:

- Design loads for shear, flexure, and torsion (V_u, T_u, and M_u)
- Diameter and length of the circular pedestal portion of the concrete foundation
- Specified concrete strength of the circular pedestal portion of the concrete foundation
- Cross section geometry of the embedded pipe

The designer should then use this information and their own design knowledge to design the following for the embedded pipe and plate section (See Figure D-2):

- Determine the number of torsion plates by engineering judgment (minimum of 4, $N_{torsion\ plates}$)
- Determine the length of torsion plates by engineering judgment (minimum of 6 inches)
- Determine width and thickness of torsion plates by engineering judgment (minimum of 1 inch for each)
- Determine the breakout edge distance, c_{a1}

$$c_{a1} = \frac{\left[\sqrt{\left(\frac{Diameter\ of\ pipe}{2}\right)^2 + 3.25\left[\left(\frac{Diameter\ of\ shaft}{2}\right)^2 - \left(\frac{Diameter\ of\ pipe}{2}\right)^2\right]}\right] - \left(\frac{Diameter\ of\ pipe}{2}\right)}{3.25}$$

- Determine the angle available for each plate to breakout

$$A = \frac{360\ degrees}{N_{torsion\ plates}}$$

- Determine the angle required for group effect

$$A_{min\ \ group} = 2\operatorname{asin}\left(\frac{3.0c_{a1}}{Diameter\ of\ shaft}\right)$$

- Check group effect. If group effect present, reduce the number of plates or diameter of shaft until no group effect occurs.

$$if\ A \leq A_{min\ group}\ then\ group\ effect\ exists$$

- Determine length of breakout

$$L_{breakout} = L_{torsion\ plate} + 3.0c_{a1}$$

- Determine breakout area of one plate

$$A_{Vc} = L_{breakout} * 3.0c_{a1}$$

- Determine breakout area of equivalent anchor bolt

$$A_{Vco} = 4.5(c_{a1})^2$$

- Determine basic shear strength of one torsional plate

$$V_b = 13\left(\frac{Length\ of\ torsion\ plate}{Width\ of\ torsion\ plate}\right)^{0.2} * \sqrt{Width\ of\ torsion\ plate} * \sqrt{f'_c} * (c_{a1})^{1.5}$$

- Determine the breakout strength of one torsional plate

$$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}}\right) * V_b$$

- Determine the breakout strength of the system of torsional plates

$$V_c = 2V_{cbg} * N_{torsion\ plates}$$

- Determine the torsional strength of the system of plates

$$T_{n\ breakout} = \varphi_{torsion} * V_c * \left(\frac{Diameter\ \ of\ pipe}{2}\right)$$

- Check to ensure that the $T_{n\ breakout}$ is greater than T_u

- Based on the breakout length above, choose depth of embedment for pipe - the breakout should not reach the surface of the concrete

- Size welds to handle design loads (AASHTO LRFD Bridge Design Specifications Section 6.13.3.2.4)

Embedded Pipe and Flexure Plate Design

For the design of the flexural plate, the designer will need the following information from the FDOT program or their own design:

- Design loads for shear, flexure, and torsion (V_u, T_u, and M_u)
- Diameter and length of the circular pedestal portion of the foundation
- Specified compressive strength of concrete
- Embedded pipe dimensions

The designer should then use this information and their own design knowledge to design the following for the embedded pipe and plate section (See Figure D-3):

- Assume the number of idealized flexural bearing positions on flexure plate is 2

- Determine thickness of flexure plate, minimum of 1 inch

- Determine diameter of flexure plate, minimum of 2 inches greater than embedded pipe outside diameter (maintain aspect ratio of width to thickness less than 2:1 to avoid prying action)

- Determine the breakout edge distance, c_{a1}

$$c_{a1} = \frac{\left[\sqrt{\left(\frac{Diam.of\ \ flexure\ plate}{2}\right)^2 + 3.25\left[\left(\frac{Diam.\ of\ shaft}{2}\right)^2 - \left(\frac{Diam.of\ \ flexure\ plate}{2}\right)^2\right]}\right] - \left(\frac{Diam.of\ \ pipe}{2}\right)}{3.25}$$

- Determine the angle available for each plate to breakout

$$A = \frac{360\ degrees}{Number\ of\ flexural\ bearing\ positions}$$

- Determine the angle required for group effect

$$A_{\min\ \ group} = 2\text{asin}\left(\frac{3.0c_{a1}}{Diameter\ of\ shaft}\right)$$

- Check group effect. If group effect present, reduce the number of plates or diameter of shaft until no group effect occurs.

$$if\ A \leq A_{\min group}\ then\ group\ effect\ exists$$

- Determine length of breakout

$$L_{breakout} = t_{flexure\ plate} + 3.0c_{a1}$$

- Determine breakout area of one plate

$$A_{Vc} = L_{breakout} * 3.0c_{a1}$$

- Determine breakout area of equivalent anchor bolt

$$A_{Vco} = 4.5(c_{a1})^2$$

- Determine basic shear strength of one torsional plate

$$V_b = 13\left(\frac{Length\ of\ flexure\ plate}{Thickness\ of\ torsion\ plate}\right)^{0.2} * \sqrt{Width\ of\ torsion\ plate} * \sqrt{f'_c} * (c_{a1})^{1.5}$$

- Determine the breakout strength of one torsional plate

$$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}}\right) * V_b$$

- Determine the breakout strength of the system of torsional plates

$$V_c = 2V_{cbg} * .5 * Number\ of\ flexural\ bearing\ positions$$

- Determine the torsional strength of the system of plates

$$M_{n\ breakout} = \varphi_{flexure} * V_c * \left(\frac{Diameter\ of\ pipe}{2}\right)$$

- Check to ensure that the $M_{n\ breakout}$ is greater than M_u

- Based on the breakout length above, verify depth of embedment for pipe - the breakout should not reach the surface of the concrete

- Size welds to handle design loads (AASHTO LRFD Bridge Design Specifications Section 6.13.3.2.4)

Concrete Pedestal Reinforcement

For the design of the concrete pedestal reinforcement, the designer will need the following information from the FDOT program or their own design:

- Design loads for shear, flexure, and torsion (V_u, T_u, and M_u)
- Diameter and length of the circular pedestal portion of the foundation
- Specified compressive strength of concrete

The designer should then use this information and their own design knowledge to design the following for the embedded pipe and plate section:

- Reinforcement for flexure (AASHTO LRFD Bridge Design Specifications Section 5.7.3.2.4)

- Reinforcement for torsion (AASHTO LRFD Bridge Design Specifications Equation 5.8.3.6.2-1)

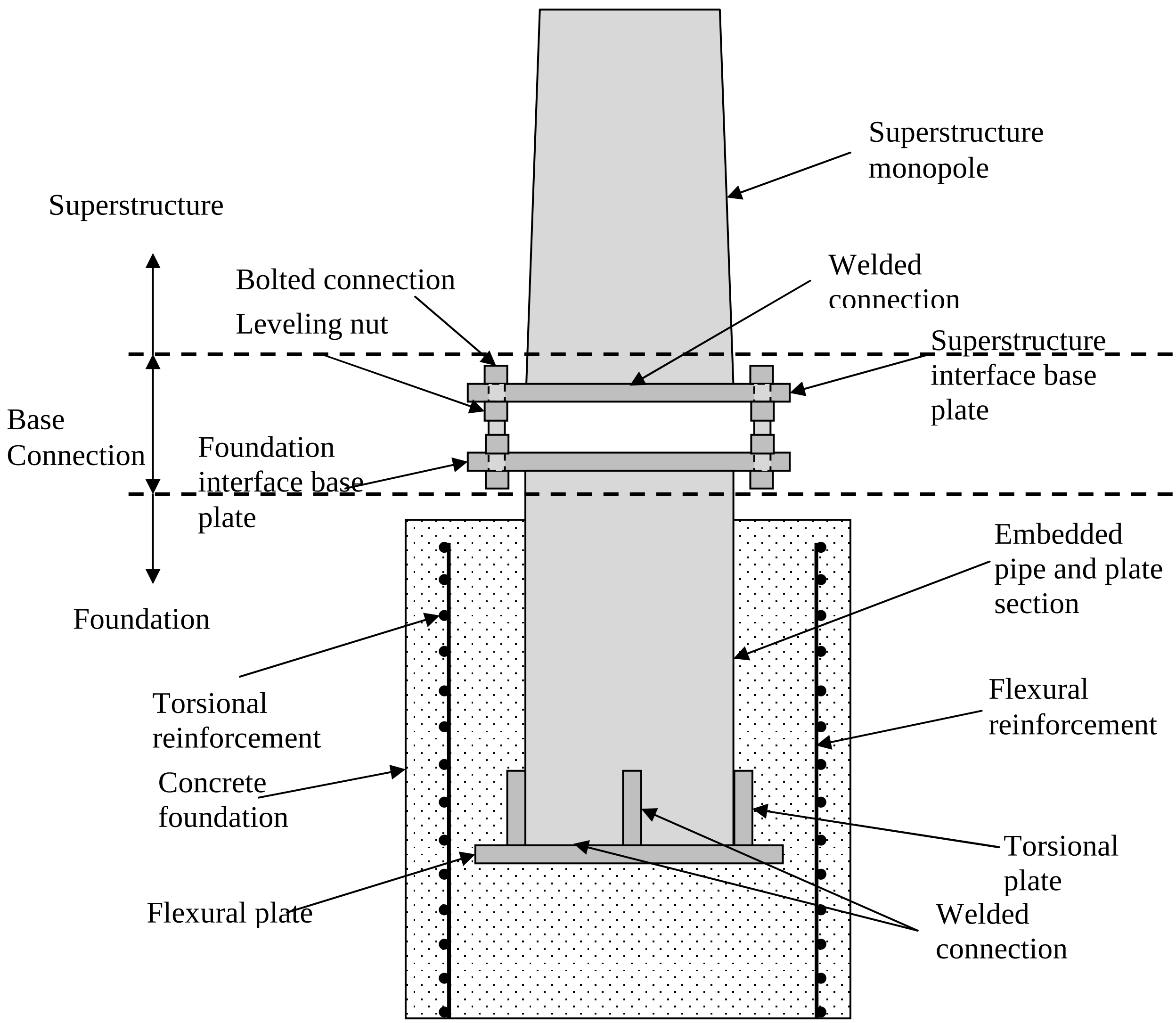


Figure D-1. Depiction of the elements described in the design guidelines

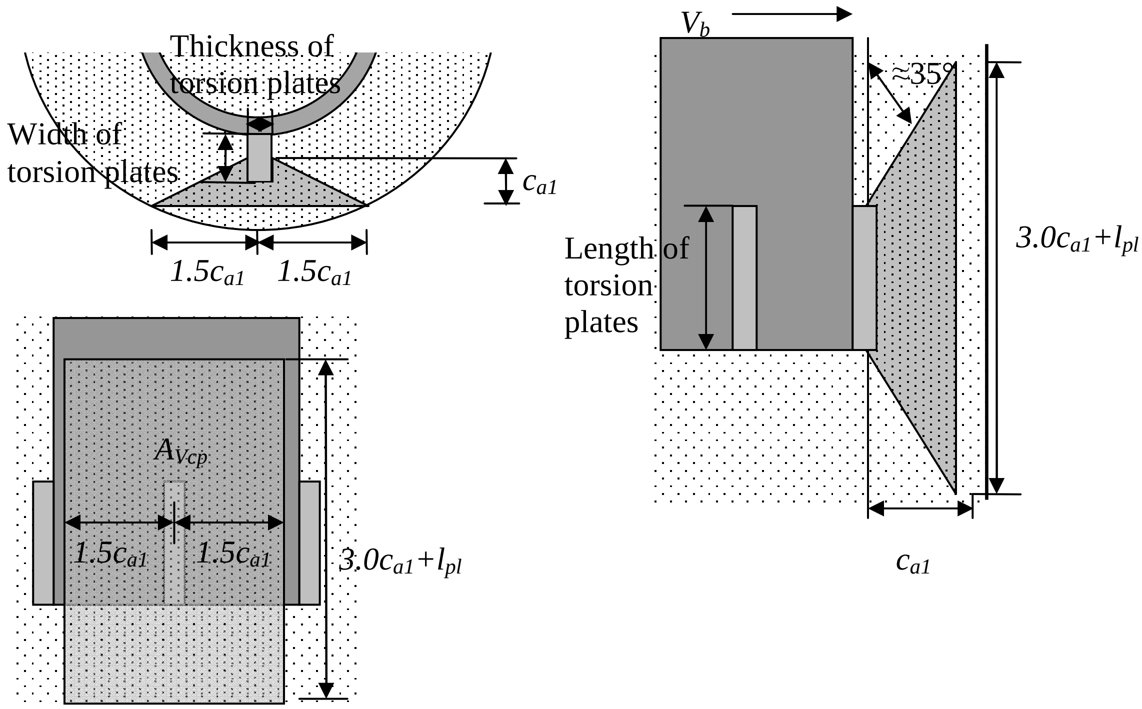


Figure D-2. Depiction of dimensions required for torsion plate design

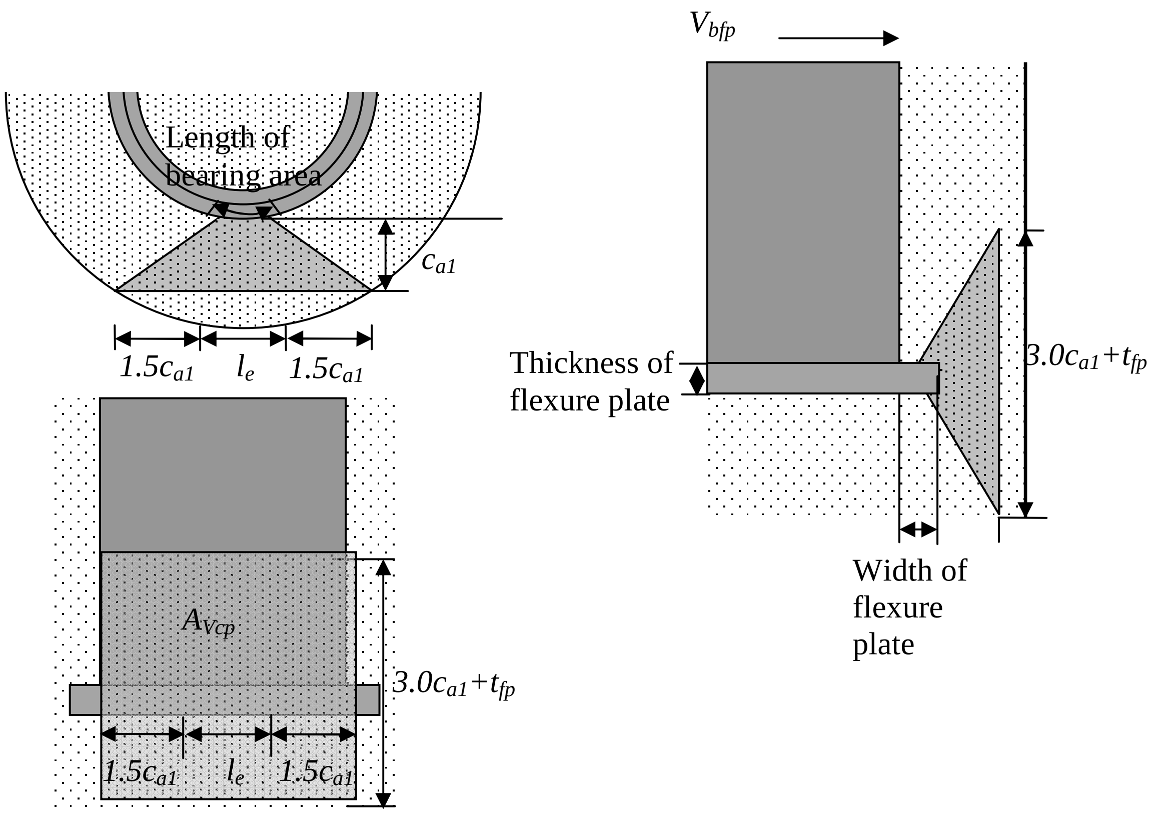


Figure D-3. Depiction of dimensions required for flexure plate design

Guidelines for the Design of the Embedded Pipe and Plate Section

The guidelines presented below are intended **ONLY** to design the embedded pipe and plate section. The remainder of the design should be designed according to applicable FDOT and AASHTO design guidelines.

Note: Yellow highlighting requires user INPUT and green highlighting denotes OUTPUT

Input from MastArm Program v4.3

Design Loads

Derived using an example from FDOT Program MastArm Program v4.3

$V_u := 1.2 \cdot 1.04\text{kip} + 1.6 \cdot 4.87\text{kip} = 9.04 \cdot \text{kip}$

$T_u := V_u \cdot 22\text{ft} = 198.88 \cdot \text{kip} \cdot \text{ft}$

$M_u := 1.2 \cdot 18.33\text{kip} \cdot \text{ft} + 1.6 \cdot 89.59\text{kip} \cdot \text{ft} = 165.34 \cdot \text{kip} \cdot \text{ft}$

Foundation Geometry

Derived using an example from FDOT Program MastArm Program v4.3

$L_{shaft} := 12\text{ft}$

$Diameter_{base.pole} := 16\text{in}$

$t_{wall.pole} := .375\text{in}$

$Diameter_{baseplate.pole} := 30\text{in}$

$t_{baseplate.pole} := 1.63\text{in}$

$Diameter_{shaft} := 3.5\text{ft}$

$Diameter_{boltcircle.pole} := 23\text{in}$

$Diameter_{rebar.circle} := 27.7\text{in}$

$No_{long.rebar} := 11$

$Diameter_{long.rebar} := 1.27\text{in}$

Input from MastArm Program v4.3

Base Connection Design

Step 1) Base Connection Design

Given

$\phi_{flex} := .9$

$f_{y.pipe} := 42\text{ksi}$

$f_{u.pipe} := 58\text{ksi}$

$f_{y.baseplate} := 36\text{ksi}$

$f_{u.baseplate} := 58\text{ksi}$

$E_s := 29000\text{ksi}$

$Diameter_{base.pole} = 16 \cdot \text{in}$

$t_{wall.pole} = 0.375 \cdot \text{in}$

$Diameter_{baseplate.pole} = 30 \cdot \text{in}$

$t_{baseplate.pole} = 1.63 \cdot \text{in}$

$Diameter_{boltcircle.pole} = 23 \cdot \text{in}$

$V_u = 9.04\,kip$

$T_{.u} = 198.88\,ft \cdot kip$

$M_u = 165.34\,ft \cdot kip$

Size the foundation interface base plate

$Diameter_{baseplate.found} := Diameter_{baseplate.pole} = 30 \cdot in$

$t_{baseplate.found} := t_{baseplate.pole} = 1.63 \cdot in$

Size the bolts for design shear, flexure, and torsion

Using threaded rods with A36 steel

$F_{y.threadedrod} := 36ksi$

$F_{u.threadedrod} := 58ksi$

$F_{nt.threadedrod} := .75 \cdot F_{u.threadedrod} = 43.5 \cdot ksi$ AISC Table J3.2

$F_{nv.threadedrod} := .4 \cdot F_{u.threadedrod} = 23.2 \cdot ksi$ AISC Table J3.2

Select the number of bolts to use

$Number_{threadedrod} := 12$

Determine the required diameter of bolts for torsion resolved into shear

$$V_{required} := \frac{T_u}{Number_{threadedrod} \cdot Diameter_{boltcircle.pole}} = 8.647\,kip$$

$$A_{threadedrod.tors} := \frac{V_{required}}{F_{nv.threadedrod}} = 0.373 \cdot in^2$$

$$Diameter_{threadedrod.tors} := \sqrt{\frac{(4 \cdot A_{threadedrod.tors})}{\pi}} = 0.689 \cdot in$$

Round up to the nearest 1/8"

$$Diameter_{threadedrod.tors} := Ceil\left(Diameter_{threadedrod.tors}, \frac{1}{8}in\right) \qquad Diameter_{threadedrod.tors} = 0.75 \cdot in$$

Determine the required diameter of bolts for flexure resolved into tension

$$P_{required} := \frac{M_u}{Diameter_{boltcircle.pole}} = 86.264\,kip$$

$$A_{threadedrod.flex} := \frac{P_{required}}{F_{nt.threadedrod}} = 1.983 \cdot in^2$$

$$Diameter_{threadedrod.flex} := \sqrt{\frac{(4 \cdot A_{threadedrod.flex})}{\pi}} = 1.589 \cdot in$$

Round up to the nearest 1/8"

$$Diameter_{threadedrod.flex} := Ceil\left(Diameter_{threadedrod.flex}, \frac{1}{8}in\right) \qquad Diameter_{threadedrod.flex} = 1.625 \cdot in$$

Determine the required diameter of bolts for shear

$$A_{threadedrod.shear} := \frac{V_u}{F_{nv.threadedrod}} = 0.39 \cdot in^2$$

$$Diameter_{threadedrod.shear} := \sqrt{\frac{(4 \cdot A_{threadedrod.shear})}{\pi}} = 0.704 \cdot in$$

Round up to the nearest 1/8"

$$Diameter_{threadedrod.shear} := Ceil\left(Diameter_{threadedrod.shear}, \frac{1}{8} in\right) \quad Diameter_{threadedrod.shear} = 0.75 \cdot in$$

Use the controlling diameter

$$Diameter_{threadedrod} := max\left(Diameter_{threadedrod.tors}, Diameter_{threadedrod.flex}, Diameter_{threadedrod.shear}\right)$$

$$Diameter_{threadedrod} = 1.625 \cdot in$$

Determine the length of the threaded rod after determining the size of the nuts and washers

Check minimum spacing and edge distance

$$s_{threadedrod} := \frac{(\pi \cdot Diameter_{boltcircle.pole})}{Number_{threadedrod}} = 6.021 \cdot in$$

$$s_{min} := \frac{8}{3} \cdot Diameter_{threadedrod} = 4.333 \cdot in$$ AISC J3.3

$$Check\_spacing := if\left(s_{threadedrod} \ge s_{min}, \text{"Sufficient"}, \text{"Not Sufficient"}\right)$$

$$Check\_spacing = \text{"Sufficient"}$$

$$L_{c.threadedrod} := \frac{(Diameter_{baseplate.pole} - Diameter_{boltcircle.pole})}{2} = 3.5 \cdot in$$

$$L_{min} := 1.75 \cdot Diameter_{threadedrod} = 2.844 \cdot in$$ AISC Table J3.4

$$Check\_edgedist := if\left(L_{c.threadedrod} \ge L_{min}, \text{"Sufficient"}, \text{"Not Sufficient"}\right)$$

$$Check\_edgedist = \text{"Sufficient"}$$

Check bearing strength of bolt holes

$$P_{n.bolthole} := 1.2 \cdot L_{c.threadedrod} \cdot t_{baseplate.pole} \cdot f_{u.baseplate} = 397.068\ kip$$

$$P_{max.bolthole} := 2.4 \cdot Diameter_{threadedrod} \cdot t_{baseplate.pole} \cdot f_{u.baseplate} = 368.706\ kip$$

$$Check\_bearing := if\left(P_{n.bolthole} \le P_{max.bolthole}, \text{"Sufficient"}, \text{"Not Sufficient"}\right)$$

$$Check\_bearing = \text{"Not Sufficient"}$$

Increase threaded rod diameter to 2 in

$Diameter_{threadedrod} := 2in$

$P_{max.bolthole} := 2.4 \cdot Diameter_{threadedrod} \cdot t_{baseplate.pole} \cdot f_{u.baseplate} = 453.792\ kip$

$Check\_bearing := if\left(P_{n.bolthole} \le P_{max.bolthole}, \text{"Sufficient"}, \text{"Not Sufficient"}\right)$

$Check\_bearing = \text{"Sufficient"}$

Size the leveling nuts

Use Heavy Hex nuts

$W_{level.nut} := 3.125in$ AISC Table 7-20

$C_{level.nut} := 3.625in$ AISC Table 7-20

$N_{level.nut} := 2in$ AISC Table 7-20

Base Connection Design

Embedded Pipe Design

Step 2) Embedded Pipe Design

Given

$\phi_{flex} = 0.9$

$\phi_{weld} := .75$

$f_{y.pipe} := 42ksi$

$f_{u.pipe} := 58ksi$

$f_{y.baseplate} := 36ksi$

$f_{u.baseplate} := 58ksi$

$E_s = 2.9 \times 10^4 \cdot ksi$

$V_u = 9.04\ kip$

$T_u = 198.88\ ft \cdot kip$

$M_u = 165.34\ ft \cdot kip$

$Diameter_{base.pole} = 16 \cdot in$

$t_{wall.pole} = 0.375 \cdot in$

$Diameter_{baseplate.pole} = 30 \cdot in$

$t_{baseplate.pole} = 1.63 \cdot in$

$Diameter_{boltcircle.pole} = 23 \cdot in$

Size the cross section of the embedded pipe

$Diameter_{embed.pipe} := Diameter_{base.pole} = 16 \cdot in$

$t_{wall.embed.pipe} := t_{wall.pole} = 0.375 \cdot in$

Size the welded connection from the embedded pipe to the foundation base plate

$$V_{weld.reqd} := \max\left(V_u, \frac{T_u}{Diameter_{embed.pipe}}\right) = 149.16\,kip$$

Select weld properties and revise if necessary

$$Weld\_Size := \frac{1}{4}in$$

$$F_{electrode} := 70ksi$$

AISC Spec. J2
Table J2.4

$$F_W := .6 \cdot F_{electrode}$$

AISC Spec. J2
Table J2.5

$$Throat := .707 \cdot Weld\_Size$$

$$\phi R_{n\_weld} := \phi_{weld} \cdot Throat \cdot F_W \qquad \phi R_{n\_weld} = 5.568 \cdot \frac{kip}{in}$$

$$\phi R_{n\_yield} := .6 \cdot f_{y.pipe} \cdot t_{wall.embed.pipe} \qquad \phi R_{n\_yield} = 9.45 \cdot \frac{kip}{in}$$

$$\phi R_{n\_rupture} := .45 \cdot f_{u.pipe} \cdot t_{wall.embed.pipe} \qquad \phi R_{n\_rupture} = 9.787 \cdot \frac{kip}{in}$$

$$\phi R_n := \min\left(\phi R_{n\_weld}, \phi R_{n\_yield}, \phi R_{n\_rupture}\right) \qquad \phi R_n = 5.568 \cdot \frac{kip}{in}$$

$$Required\_Length\_Plate := \frac{V_{weld.reqd}}{\phi R_n} \qquad Required\_Length\_Plate = 26.791 \cdot in$$

$$Ceil(Required\_Length\_Plate, 1in) = 27 \cdot in$$

$$Check\_Length := \begin{cases} \text{"Sufficient"} & \text{if } \left(\pi \cdot Diameter_{embed.pipe}\right) \geq Required\_Length\_Plate \\ \text{"Not Sufficient"} & \text{otherwise} \end{cases}$$

$$Check\_Length = \text{"Sufficient"}$$

Embedded Pipe Design

Step 3) Embedded Pipe and Torsion Plates Design

Given

$\phi_{flex} = 0.9$

$\phi_{weld} = 0.75$

$\phi_{tor} := .9$

$f_{y.pipe} = 42 \cdot ksi$

$f_{u.pipe} = 58 \cdot ksi$

$E_s = 2.9 \times 10^4 \cdot ksi$

$V_u = 9.04\,kip$

$T_u = 198.88\,ft \cdot kip$

$M_u = 165.34\,ft \cdot kip$

$Diameter_{embed.pipe} = 16 \cdot in$

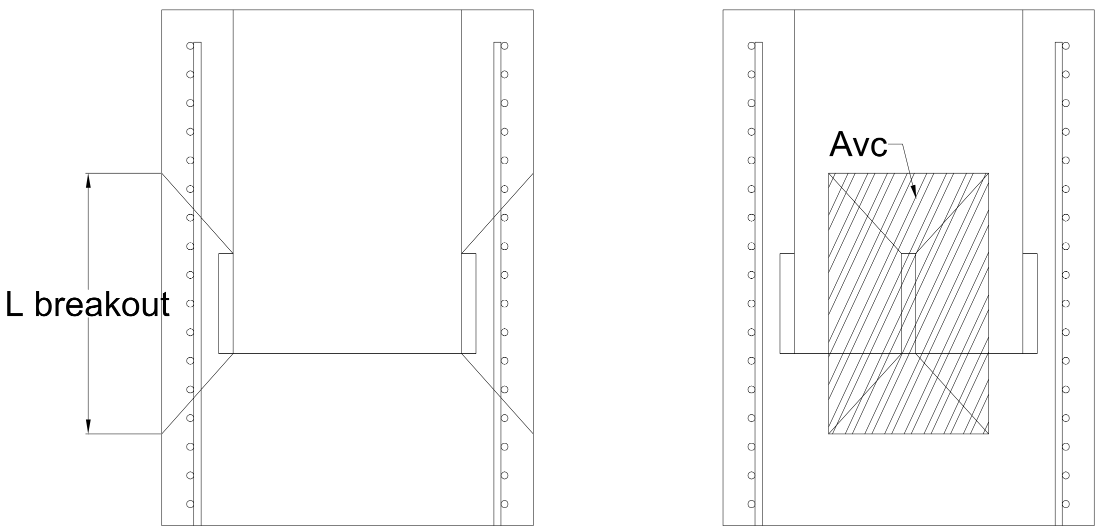
$t_{wall.embed.pipe} = 0.375 \cdot in$

$Diameter_{shaft} = 42 \cdot in$

$L_{shaft} = 12\,ft$

$f'_c := 5500psi$

Based on ACI 318-08 Appendix D - Anchorage to Concrete



Estimate torsion plate section properties and refine if necessary

$N_{tor.plate} := 4$ *Number of torsion plates, minimum of 4*

$L_{tor.plate} := 6in$ *Length of torsion plate, minimum of 6 in.*

$b_{tor.plate} := 1in$ *Width of torsion plate, minimum of 1 in.*

$t_{tor.plate} := 1in$ *Thickness of torsion plate, minimum of 1 in.*

$$cover := \frac{Diameter_{shaft} - Diameter_{embed.pipe}}{2} \qquad cover = 13 \cdot in$$

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{Diameter_{embed.pipe}}{2}\right)^2 + 3.25 \cdot \left[\left(\frac{Diameter_{shaft}}{2}\right)^2 - \left(\frac{Diameter_{embed.pipe}}{2}\right)^2\right]} - \left(\frac{Diameter_{embed.pipe}}{2}\right.\right.}{3.25}$$

$$c_{a1} = 8.587 \cdot in$$

$$A := \frac{360deg}{N_{tor.plate}} = 1.571 \qquad A = 90 \cdot deg$$

$$chord\_group := 2 \cdot \frac{Diameter_{shaft}}{2} \cdot \sin\left(\frac{A}{2}\right) \qquad chord\_group = 2.475\,ft$$

$$A_{min\_group} := 2 \cdot asin\left(\frac{3.0 \cdot c_{a1}}{Diameter_{shaft}}\right) \qquad A_{min\_group} = 75.66 \cdot deg$$

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

$$L_{breakout} := L_{tor.plate} + 2 \cdot 1.5 c_{a1} \qquad L_{breakout} = 31.76 \cdot in$$

$$A_{Vc} := \min\left(L_{breakout}, L_{shaft}\right) \cdot 3 \cdot c_{a1} \qquad A_{Vc} = 818.109 \cdot in^2$$

$$A_{Vco} := 4.5 \cdot c_{a1}^{\,2} \qquad A_{Vco} = 331.776 \cdot in^2$$

$$l_e := L_{tor.plate} \qquad l_e = 0.5\,ft$$

$$V_b := 13 \cdot \left(\frac{l_e}{b_{tor.plate}}\right)^{.2} \cdot \sqrt{\frac{b_{tor.plate}}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf \qquad V_b = 34.712\,kip$$

$\psi_{cV} := 1.4$ *Modification factor for cracking in concrete*

$\psi_{ecV} := 1.0$ *Modification factor for anchor groups*

$\psi_{edV} := 1.0$ *Modification factor for edge effects*

$$V_{cbg} := \left(\frac{A_{Vc}}{A_{Vco}}\right) \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b \qquad V_{cbg} = 119.832\,kip$$

$$V_{cbg\_parallel} := 2 \cdot V_{cbg} \qquad V_{cbg\_parallel} = 239.664\,kip$$

$$V_c := V_{cbg\_parallel} \cdot N_{tor.plate} \qquad V_c = 958.658\,kip$$

$$T_{n.breakout.plate} := \phi_{tor} \cdot V_c \cdot \left(\frac{Diameter_{embed.pipe}}{2}\right) \qquad T_{n.breakout.plate} = 575.195\,ft \cdot kip$$

$$Check\_Breakout\_Torsion := \begin{cases} \text{"Sufficient"} & \text{if } T_{n.breakout.plate} \geq T_u \\ \text{"Not Sufficient"} & \text{if } T_{n.breakout.plate} < T_u \end{cases}$$

$Check\_Breakout\_Torsion = \text{"Sufficient"}$

Based on breakout length above, choose depth of embedment for pipe

$L_{clearance} := 3in$ *Length of clearance between breakout and top of shaft*

$L_{embedment} := L_{clearance} + L_{breakout} - 1.5 \cdot c_{a1}$ $L_{embedment} = 21.88 \cdot in$

$L_{embedment} := ceil\left(\frac{L_{embedment}}{in}\right) \cdot in$ $L_{embedment} = 22 \cdot in$

Weld Design for Torsion Plates

$V_{weld.tor} := \frac{T_{n.breakout.plate}}{Diameter_{shaft}}$ $V_{weld.tor} = 164.341\ kip$

Select weld properties and revise if necessary

$Weld\_Size := \frac{3}{8} in$

AISC Spec. J2 Table J2.4

$F_{electrode} := 70ksi$

$F_W := .6 \cdot F_{electrode}$

AISC Spec. J2 Table J2.5

$Throat := .707 \cdot Weld\_Size$

$\phi R_{n\_weld} := \phi_{weld} \cdot Throat \cdot F_W$ $\phi R_{n\_weld} = 8.351 \cdot \frac{kip}{in}$

$\phi R_{n\_yield} := .6 \cdot f_{y.pipe} \cdot t_{wall.embed.pipe}$ $\phi R_{n\_yield} = 9.45 \cdot \frac{kip}{in}$

$\phi R_{n\_rupture} := .45 \cdot f_{u.pipe} \cdot t_{wall.embed.pipe}$ $\phi R_{n\_rupture} = 9.787 \cdot \frac{kip}{in}$

$\phi R_n := \min(\phi R_{n\_weld}, \phi R_{n\_yield}, \phi R_{n\_rupture})$ $\phi R_n = 8.351 \cdot \frac{kip}{in}$

$Required\_Length\_Each\_Plate := \frac{V_{weld.tor}}{N_{tor.plate} \phi R_n}$ $Required\_Length\_Each\_Plate = 4.92 \cdot in$

$ceil\left(\frac{Required\_Length\_Each\_Plate}{in}\right) \cdot in = 5 \cdot in$

$$Check\_Length := \begin{cases} \text{"Sufficient"} & \text{if } L_{tor.plate} \geq Required\_Length\_Each\_Plate \\ \text{"Not Sufficient"} & \text{otherwise} \end{cases}$$

$Check\_Length = \text{"Sufficient"}$

Embedded Pipe and Torsion Plates Design

Step 4) Embedded Pipe and Flexure Plate Design

Given

Given

$\phi_{flex} = 0.9$

$\phi_{weld} = 0.75$

$\phi_{tor} = 0.9$

$f_{y.pipe} = 42 \cdot ksi$

$f_{u.pipe} = 58 \cdot ksi$

$E_s = 2.9 \times 10^4 \cdot ksi$

$V_u = 9.04\,kip$

$T_u = 198.88\,ft \cdot kip$

$M_u = 165.34\,ft \cdot kip$

$Diameter_{embed.pipe} = 16 \cdot in$

$t_{wall.embed.pipe} = 0.375 \cdot in$

$Diameter_{shaft} = 42 \cdot in$

$L_{shaft} = 12\,ft$

$f'_c = 5.5 \cdot ksi$

Based on ACI 318-08 Appendix D - Anchorage to Concrete

Estimate flexure plate section properties and refine if necessary

$N_{flex.plate.bear} := 4$ *Number of idealized flexural bearing positions on flexure plate*

$t_{flex.plate} := 1in$ *Thickness of flexure plate, minimum of 1 in.*

$Diameter_{flex.plate} := 20in$ *Diameter of flexure plate, minimum of 2 in. greater than embedded plate*

$$cover := \frac{Diameter_{shaft} - Diameter_{flex.plate}}{2} \qquad cover = 11 \cdot in$$

$$c_{a1} := \frac{\left[\sqrt{\left(\frac{Diameter_{flex.plate}}{2}\right)^2 + 3.25 \cdot \left[\left(\frac{Diameter_{shaft}}{2}\right)^2 - \left(\frac{Diameter_{flex.plate}}{2}\right)^2\right]} - \left(\frac{Diameter_{flex.plate}}{2}\right)\right]}{3.25}$$

$$c_{a1} = 7.618 \cdot in$$

$$A := \frac{360deg}{N_{flex.plate.bear}} = 1.571 \qquad A = 90 \cdot deg$$

$$chord\_group := 2 \cdot \frac{Diameter_{shaft}}{2} \cdot \sin\left(\frac{A}{2}\right) \qquad chord\_group = 2.475\,ft$$

$$A_{min\_group} := 2 \cdot asin\left(\frac{3.0 \cdot c_{a1}}{Diameter_{shaft}}\right) \qquad A_{min\_group} = 65.936 \cdot deg$$

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

$L_{breakout} := t_{flex.plate} + 2 \cdot 1.5 c_{a1}$ $\quad L_{breakout} = 23.855 \cdot in$

$A_{Vc} := \min(L_{breakout}, L_{shaft}) \cdot 3 \cdot c_{a1}$ $\quad A_{Vc} = 545.219 \cdot in^2$

$A_{Vco} := 4.5 \cdot c_{a1}{}^2$ $\quad A_{Vco} = 261.182 \cdot in^2$

$b_{flex.plate} := .5 \cdot (Diameter_{flex.plate} - Diameter_{embed.pipe})$ $\quad b_{flex.plate} = 2 \cdot in$

$L_{flex.plate} := \frac{\pi}{8} \cdot (Diameter_{embed.pipe} + 2 \cdot .5 \cdot b_{flex.plate})$ $\quad L_{flex.plate} = 7.069 \cdot in$

$l_e := L_{flex.plate}$ $\quad l_e = 7.069 \cdot in$

$$V_b := 13 \cdot \left(\frac{l_e}{t_{flex.plate}}\right)^{.2} \cdot \sqrt{\frac{b_{flex.plate}}{in}} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5} \cdot lbf \qquad V_b = 42.394\ kip$$

$\psi_{cV} := 1.4$ *Modification factor for cracking in concrete*

$\psi_{ecV} := 1.0$ *Modification factor for anchor groups*

$\psi_{edV} := 1.0$ *Modification factor for edge effects*

$$V_{cbg} := \left(\frac{A_{Vc}}{A_{Vco}}\right) \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot V_b \qquad V_{cbg} = 123.897\ kip$$

$V_{cbg\_parallel} := 2 \cdot V_{cbg}$ $\quad V_{cbg\_parallel} = 247.794\ kip$

$V_c := V_{cbg\_parallel} \cdot .5 N_{flex.plate.bear}$ $\quad V_c = 495.587\ kip$

$$M_{n.breakout.plate} := \phi_{tor} \cdot V_c \cdot \left(\frac{Diameter_{embed.pipe}}{2}\right) \qquad M_{n.breakout.plate} = 297.352\ ft \cdot kip$$

$$Check\_Breakout\_Flexure := \begin{cases} \text{"Sufficient"} & \text{if } M_{n.breakout.plate} \ge T_u \\ \text{"Not Sufficient"} & \text{if } M_{n.breakout.plate} < T_u \end{cases}$$

Check_Breakout_Flexure = "Sufficient"

Weld Design for Flexure Plate

$$V_{weld.flex} := \frac{\max(M_u, M_{n.breakout.plate})}{Diameter_{flex.plate}} \qquad V_{weld.flex} = 178.411\ kip$$

Select weld properties and revise if necessary

$Weld\_Size := \frac{1}{4} in$ $\quad$ AISC Spec. J2 Table J2.4

$F_{electrode} := 70 ksi$

$F_W := .6 \cdot F_{electrode}$ $\quad$ AISC Spec. J2 Table J2.5

$Throat := .707 \cdot Weld\_Size$

$\phi R_{n\_weld} := \phi_{weld} \cdot Throat \cdot F_W$ $\qquad \phi R_{n\_weld} = 5.568 \cdot \frac{kip}{in}$

$\phi R_{n\_yield} := .6 \cdot f_{y.pipe} \cdot t_{wall.embed.pipe}$ $\qquad \phi R_{n\_yield} = 9.45 \cdot \frac{kip}{in}$

$\phi R_{n\_rupture} := .45 \cdot f_{u.pipe} \cdot t_{wall.embed.pipe}$ $\qquad \phi R_{n\_rupture} = 9.787 \cdot \frac{kip}{in}$

$\phi R_n := \min(\phi R_{n\_weld}, \phi R_{n\_yield}, \phi R_{n\_rupture})$ $\qquad \phi R_n = 5.568 \cdot \frac{kip}{in}$

$Required\_Length\_Plate := \frac{V_{weld.flex}}{\phi R_n}$ $\qquad Required\_Length\_Plate = 32.044 \cdot in$

$ceil\left(\frac{Required\_Length\_Plate}{in}\right) \cdot in = 33 \cdot in$

$$Check\_Length := \begin{cases} \text{"Sufficient"} & \text{if } (\pi \cdot Diameter_{embed.pipe}) \ge Required\_Length\_Plate \\ \text{"Not Sufficient"} & \text{otherwise} \end{cases}$$

$Check\_Length = \text{"Sufficient"}$

Embedded Pipe and Flexure Plate Design

Concrete Pedestal Reinforcement

Step 5) Concrete Pedestal Reinforcement

Given

$Diameter_{shaft} = 42 \cdot in$ $\qquad f_{y.rebar} := 50ksi$

$L_{shaft} = 12\ ft$ $\qquad Diameter_{hoop.rebar} := .500in$

$f'_c = 5.5 \cdot ksi$ $\qquad R := .5Diameter_{shaft} = 21 \cdot in$

$V_u = 9.04\ kip$ $\qquad Diameter_{flex.rebar} := 1.375in$

$T_u = 198.88\ ft \cdot kip$ $\qquad Diameter_{rebar.circle} := 27.5in$

$M_u = 165.34\ ft \cdot kip$

Reinforcement for Flexure

Determine properties of flexural reinforcement

$Number_{flex.rebar} := No_{long.rebar} = 11$

$A_{flex.rebar} := .25 \cdot \pi \cdot Diameter_{flex.rebar}^{\ 2}$

Assume 8 of the 11 bars yield

$n_{flex.yield} := 8$

Calculations Using ACI Stress Block

$$\beta_1(f_c) := \begin{cases} .85 & \text{if } f_c < 4000psi \\ .65 & \text{if } f_c > 8000psi \\ \left[.85 - .05\cdot\left[\frac{(f_c - 4000psi)}{1000psi}\right]\right] & \text{if } 4000psi \le f_c \le 8000psi \end{cases}$$

$\beta_1(f'_c) = 0.775$

ACI 10.2.7.3

$$A_{comp} := \frac{(n_{flex.yield}\cdot A_{flex.rebar}\cdot f_{y.rebar})}{.85\cdot f'_c}$$

$A_{comp} = 0.882\,ft^2$

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

$$a := root(A_{compcircle}(h), h, 0in, R)$$

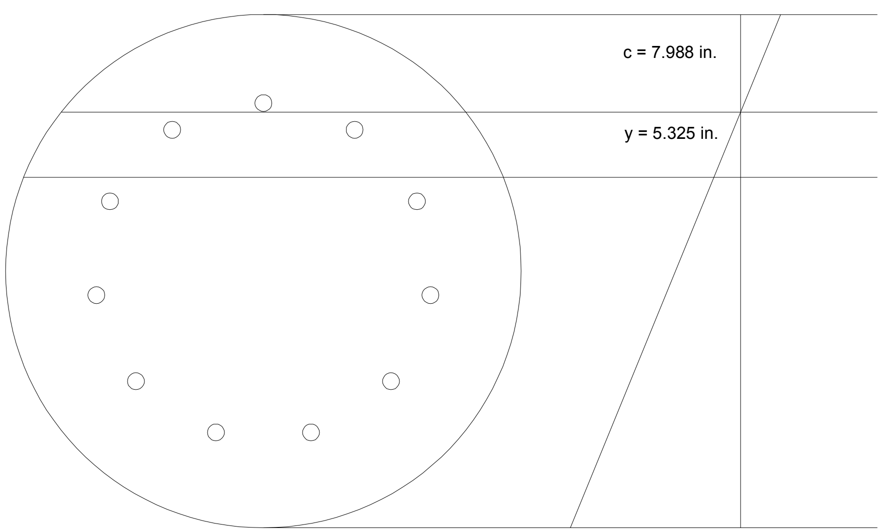
$a = 6.191\cdot in$

$$c := \frac{a}{\beta_1(f'_c)}$$

$c = 7.988\cdot in$

$$y := .002\cdot\frac{c}{.003}$$

$y = 5.325\cdot in$



$$d_{bars} := .5\cdot(Diameter_{shaft} - c - y) + c + y$$

$d_{bars} = 27.657\cdot in$

$$M_{n.shaft} := \phi_{flex}\cdot n_{flex.yield}\cdot A_{flex.rebar}\cdot f_{y.rebar}\cdot\left(d_{bars} - \frac{a}{2}\right)$$

$M_{n.shaft} = 1.094\times 10^3\ ft\cdot kip$

Check_Flexure := if$(M_{n.shaft} \ge M_u$, "Sufficient" , "Not Sufficient"$)$

Check_Flexure = "Sufficient"

Reinforcement for Torsion

$$A_{tors.rebar} := \pi \cdot \left(.5 \cdot Diameter_{hoop.rebar}\right)^2 = 0.196 \cdot in^2$$

$$A_o := \pi \cdot \left(.5Diameter_{rebar.circle} + .5 \cdot Diameter_{long.rebar} + .5 \cdot Diameter_{hoop.rebar}\right)^2 = 672.876 \cdot in^2$$

$$s_{tors.rebar} := 4in$$

$$T_{n.shaft} := \frac{\phi_{tor} \cdot \left(2 \cdot A_o \cdot A_{tors.rebar} \cdot f_{y.rebar}\right)}{s_{tors.rebar}} \cdot \cot(45deg)$$

$$T_{n.shaft} = 247.723 \, ft \cdot kip$$

$$Check\_Torsion := if\left(T_{n.shaft} \geq T_u, \text{"Sufficient"}, \text{"Not Sufficient"}\right)$$

Check_Torsion = "Sufficient"

Concrete Pedestal Reinforcement

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