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ALTERNATIVES FOR PRECAST PILE SPLICES

Part 1

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16. Abstract

A pile splice was designed for a 30 inch square precast prestressed concrete pile using steel pipe and grout . The splice was designed for tension, compression, flexure, and shear. A testing apparatus for tension testing is included along with a flexure test apparatus. Field installation guidelines are given.

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1.0 Introduction

Prestressed concrete piles are commonly used on bridges and foundations. Problems often arise during the installation of prestressed concrete piles. These problems include cracking of long piles during handling, the extreme weight of piles, and the cost involved in handling long piles [3]. To alleviate these problems shorter lengths of pile are spliced together versus driving a single pile. Splicing is an attractive solution because shorter lengths of pile weigh less and are easier to handle and transport. Splicing also alleviates the need to calculate exact pile lengths prior to installation, which will reduce the waste of piles, which are too long. A splice must develop adequate strength in compression, tension, and moment during installation and in service [7]. Splices should be effective without significantly extending the duration of construction, be as durable as the pile, and be inexpensive.

Currently, the Florida Department of Transportation uses a dowel type splice for prestressed concrete piles [1]. The details consist of steel dowels and epoxy mortar. The size and number of dowels depend on the cross sectional area of the pile. There are no standard national guidelines on how to splice together piles; however guidelines suggest that a pile splice should be of equal strength and performance of the unspliced pile [8].

The splice considered in this report for the standard FDOT 30 inch pile will be designed to transfer tension loads, compression loads, flexural loads, and shear loads. The pile has an 18 inch circular void in the center. The splice consists of inserting a steel tube into the void and grouting between the tubing and the pile (Figure 1). The steel tubing has a diameter of 14 inches, a wall thickness of a half inch, and is made of Grade

42 steel. The steel tubing will be deformed in order to establish bond between the tubing and the grout.

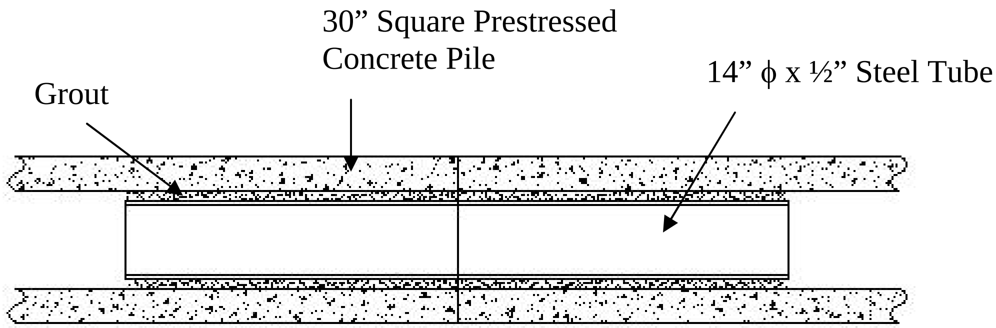


Fig 1. Basic Splice Detail

2.0 Background

Several types of splices are currently used throughout the world. There has also been research performed on various types of splices, including the splice that is designed for this report.

2.1 Splicing Methods

Bruce and Hebert [3] conducted research on sixteen pile splices which were presently in use in 1974. Below is a listing of the splices along with a brief description of how the splice works.

1. Marier Splice: A mechanical splice patented in Canada. Four flexible locking pins lock male and female steel caps cast into the piles together.
2. Herkules Splice: A mechanical splice patented in Sweden. A steel pin protruding from a male steel cap in the top pile aligns the piles while the top section is rotated to lock the piles together. Locking pins are inserted to hold the piles in the locked position.

3. ABB Splice: A mechanical splice patented in Sweden. Lock-blocks are welded to the reinforcing rods and cast with the pile. Once the piles are aligned, locking pins are driven into the joint block to draw the piles together.
4. NCS Splice: A welded splice used in the Pacific Northwest. Steel caps are cast into the ends of the pile using dowels bars to anchor the cap to the pile. When the piles are aligned, a fillet weld connects the two caps together.
5. Tokyu Splice: A welded splice patented in Japan. The prestressing bars are threaded through the male and female steel caps and bolted to fasten the caps to the piles. A butt weld is used to weld the caps together.
6. Raymond Cylinder Pile Splice: A welded spice for use on prestressed cylinder piles. Two steel ring caps with openings to allow post-tensioning are cast into the piles. After the post-tensioning is performed, the piles are aligned and butt welded together.
7. Bolognesi-Moretto Splice: A welded splice patented in Argentina. The piles are cast with steel base plates in the ends having the top base plate being smaller then the bottom. Once the piles are aligned, the base plates are connected by a fillet weld.
8. Japanese Bolted Splice: A bolted splice used in Japan. The bottom pile is cast with a steel piece in the end. The top pile is also cast with a steel piece and has eight bolts protruding from the section. The sections are locked together by fastening of the bolts.
9. Brunspile Connector Ring: A wedge splice patented and used in the United States. A connector ring is tapped onto the driven bottom pile. The top pile is

guided into the ring and driving completes the splice by wedging the piles together.

10. Anderson Splice: A sleeve splice patented in the United States. A steel sleeve is driven over the head of the bottom splice. The top pile is driven onto the sleeve to join the piles. Epoxy resin must be inserted between the sleeve and piles in order to develop tension.
11. Fuentes Splice: A sleeve splice patented in the United States and abroad. A steel band is welded to the reinforcement of both piles and is left exposed after casting. A steel sleeve is welded to the bands to connect the piles.
12. Hamilton Form Company Splice: A sleeve splice used in Texas. The splice is two semi-circular sleeves which are placed around the joint of the piles. Bolts are used to clamp the two sleeve sections together.
13. Cement-Dowel Splice or Epoxy-Dowel Splice: The splice is not patented and is currently used in Florida. The top pile is cast with dowel bars extending out of the section. The bottom has holes to receive the dowels. Holes are filled with cement or epoxy.
14. Macalloy Splice: A post-tensioned splice patented in Great Britain. Macalloy prestressing bars are threaded through both piles. The bars are screwed together by mechanical couplers and then bars are stressed which will post-tension the piles.
15. Mouton Splice: A combination splice patented in the United States. A connector ring and a connector pin lock the piles together.

16. Raymond Wedge Splice: A combination splice patented in the United States. The piles are connected so that a sleeve with square holes in it is around the joint. Steel wedges are placed in the holes of the sleeve and welded in place.
17. Sure-Lock Splice [14]: A mechanical splice patented in Canada. Steel bars driven into male and female plates form a shear key joint.

2.2 Previous Research

The FDOT has performed previous experimentation on the type of splice designed in this report. A report was written by Issa [8] on the results of the testing. Two test specimens were constructed and flexural tests were performed.

The first specimen was constructed with a 14 inch diameter steel tube with a length of five feet from each side of the splice for a total length of ten feet. The steel tubing was one half inch thick and made of Grade 42 steel. A No. 4 rebar was welded in a spiral configuration to the tube at a pitch of six inches. The void between the tubing and the pile was filled with non-shrink grout. The tubing was not filled with any grout or concrete material; it remained hollow. Hydraulic jacks were placed at a distance of two and half feet from either side of the splice (Figure 2). During the progression of the test, the pile began to crack horizontally near the joint at a moment of 255 kip-ft. Cracks that would commonly occur during the test are flexural cracks that travel vertically up from the bottom of the pile; therefore the horizontally cracking was unexpected. The cracking worsened and continued until the pile spiral reinforcement yielded. Failure occurred at a moment of 580.5 kip-ft even though the calculated nominal moment capacity of the unspliced pile section was 1,000 kip-ft.

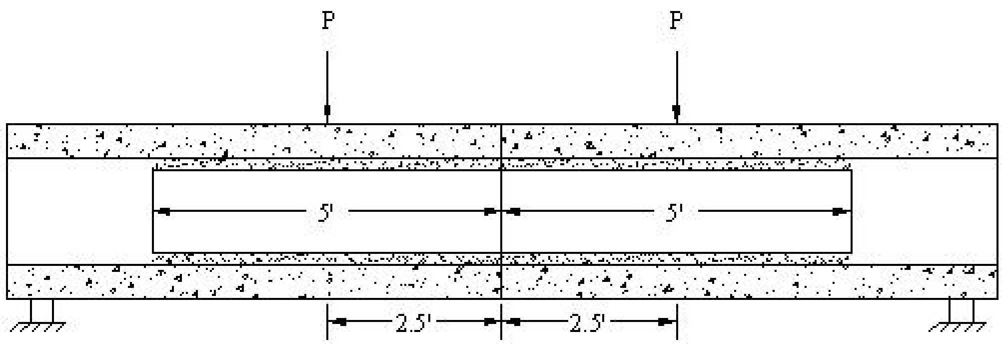


Fig 2. Test Specimen #1

The second specimen's steel tubing was a total of fifteen feet long having seven and a half feet on each side of splice. The tubing was the same size and grade steel as the first test. Rebar was welded to the outside of the tubing in the same manner and spacing as the first test splice. The tubing was filled with concrete. The void between the tubing and the pile was filled the same non-shrink grout as the first specimen. Hydraulic jacks were placed at a distance of five feet from either side of the splice (Figure 3). The ultimate test moment capacity was observed to be 840 kip-ft. Once again the unspliced pile had a calculated nominal moment capacity of 1000 kip-ft and the spliced pile section had a calculated nominal moment capacity of 878 kip-ft. Therefore the pile developed 84% of the calculated unspliced pile capacity and 96% of the calculated spliced pile capacity.

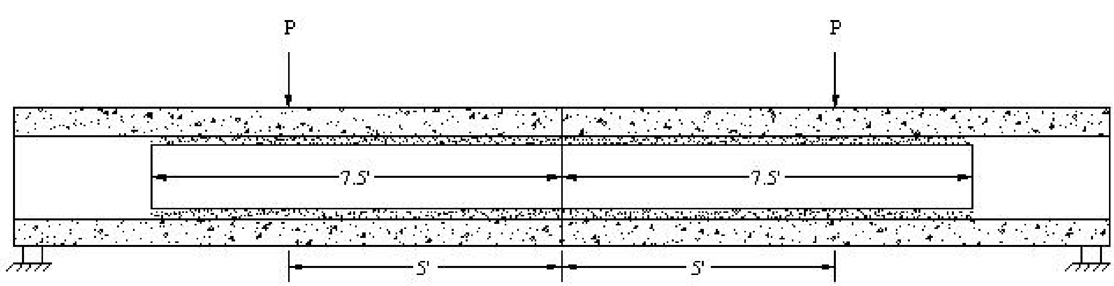


Fig 3. Test Specimen #2

After the laboratory testing of the splice, field testing was performed by driving spliced piles. The splice used in this test had a steel tubing of a total length of 20 feet [16]. A problem arose in that the spliced piles would not drive. Several issues may have contributed to the pile failure. First, the grout used was apparently not up to proper strength when driving took place. Second, the upper pile was not guided or supported while the grout was installed and allowed to cure. It was supported only by a vertical crane which may have allowed movement during curing. Last, the epoxy mortar bed between piles was created by placing steel shims at the joint. These steel shims were not removed prior to driving and therefore may have created four stiff points at the joint. The problems in the prior tests have been taken into consideration in the design of the splice.

3.0 Design of Splice

In the design of the splice several factors were considered. The splice's performance was designed for tension, flexure, shear and compression. The load path for each loading was considered and then designed in order to provide adequate capacity.

3.1 Tensile Design of Splice

Tensile loads in piles are caused by the impact wave introduced by a pile driving hammer. When a pile is impacted by a hammer, a wave travels down the pile causing compressive forces [15]. When the wave arrives at the bottom of the pile it is reflected back up the pile causing tensile forces. In order to design a splice, the load path of the tensile forces must be understood. Tensile forces during driving are transmitted from the lower pile to the steel tube. The steel tube carries the forces across the joint into the upper pile. The grout acts as a medium to transfer the forces between the steel tube and the upper and lower piles. At the joint between piles, the steel tube will be carrying the entire tensile force.

In order to design the splice for a tensile test, the magnitude of tensile load felt by a pile during driving must be determined. McVay [11] conducted a study on driven pile capacities which gathered data on the loads piles experience during driving. According to the report on driving data [11], the pile experienced a dynamic tensile load of 1124 kips during driving. The dynamic load can then be correlated to an equivalent static load for laboratory testing. The correlation is due to the strengthening of materials as strain rate increases [7]. Figure 19.11 of *Structural Dynamics* [17] depicts the different ratios for concrete between static and dynamic strength based on strain rate (Figure 4). As shown in Figure 2 the ratio of dynamic to static strength for concrete during pile driving is about 1.75. Therefore, the 1124 kip dynamic tensile load from pile driving has an equivalent static load of 650 kips. The tensile load test in the lab will need to test the pile to at least 650 kips in order to mimic what the pile feels during actual field driving. The steel tubing which is designated HSS 14.000 x 0.500 by AISC [10], has a cross sectional area of 19.8 in^2 and is Grade 42 steel, therefore the tubing can resist a tensile load of 832 kips before yielding.

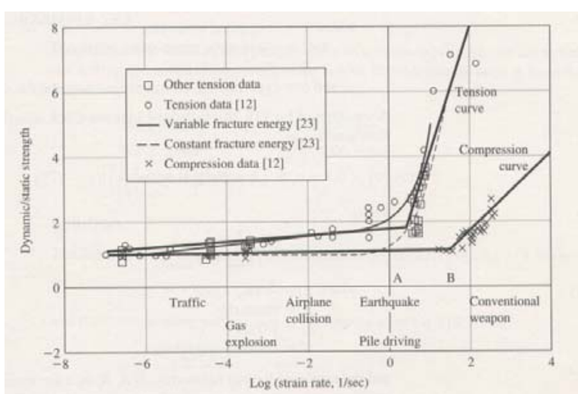


Fig 4. Structural Dynamics [17] Figure 19-11

The length of steel tubing required is controlled by several different factors.

- Strand development length
- Tube development length
- Bond stress between tube, grout, and pile

First, there needs to be enough length to give the prestressing strands and the tubing enough length to fully develop. The strands will be required to carry the full tension load at the end of the tubing, where the splice is terminated. The tubing will be carrying the full tensile load at the joint of the two piles.

ACI 318 [4] Equation 12-2 and AASHTO [9] Equation 5.11.4.1-1 compute the development length required for prestressing strands. The strands are one half inch in diameter and are Grade 270 low relaxation strand and require 57 inches to develop.

ACI 318 Equation 12-2:

$$l_d = \left(\frac{f_{se}}{3}\right)d_b + (f_{ps} - f_{se})d_b = 56.8\,\text{in}$$

where $d_b = 0.5$ in

$f_{se} = 194$ ksi

$f_{ps} = 243$ ksi

AASHTO Equation 5.11.4.1-1:

$$l_d = (f_{ps} - \frac{2}{3}f_{pe})d_b = 56.8\,\text{in}$$

where $d_b = 0.5$ in

$f_{pe} = 194$ ksi

$f_{ps} = 243$ ksi

In the above equations, the effective prestress after losses (f_{se} and f_{pe}) was taken as 80% of the yield strength of the prestressing steel based on AASHTO [9] Table 5.9.3.1-Stress Limits for Prestressing Strands. The yield strength of Grade 270 low relaxation steel is 243 ksi, which is 90% of f_{pu} based on AASHTO [9] Table 5.4.4.1.1-Properties of Prestressing Strand and Bar. The stress in prestressed reinforcement at nominal strength (f_{ps}) is taken as the yield strength which is the highest desirable stress during a tensile loading.

The steel tubing's development length is calculated by ACI 318 [4] Section 12.2.3, Equation 12-1. This equation determines the development length of deformed bars. The tubing will be acting similar to a deformed bar since deformations will be added to the tubing to establish bond. An equivalent diameter for the tubing was calculated from the area of the tubing. The length required for development is 82 inches.

ACI 318 Equation 12-1:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)}\right)d_b = 82\,\text{in}$$

For which the equivalent bar diameter is determined as: $d_b = \sqrt{\frac{4A_s}{\pi}} = 5.02\,\text{in}$

Where: $\left(\frac{c+K_{tr}}{d_b}\right) \leq 2.5$

A_s = 19.8 in^2

f_y = 42000 psi

f'_c = 6000 psi

c = 15 in

K_{tr} = 0

a = 1.0

b = 1.0

g = 1.0

l = 1.0

The splice length also needs to be long enough for the bond stress between the tubing and the grout to transfer the tension load between the tube and the pile. Finite element models of the splice were created to determine the bond stresses in the splice when a linear elastic stress distribution is followed. Axisymmetric models of various lengths depicting the steel tubing, grout, and prestressed concrete were created using ADINA. The models were assumed to remain in the elastic range of structural behavior while being loaded with 650 kips. This is the desired behavior of the pile so there is no cracking in the concrete. A minimum element length to width ratio of one to four was used in order to obtain accurate results from the model. Hand checks were performed to assess the accuracy of the model results. The concrete and grout were modeled with a modulus of elasticity of 4,000 ksi and a Poisson's ratio of 0.17. The steel was modeled with a modulus of elasticity of 29,000 ksi and a Poisson's ratio of 0.3. The bond stresses between the steel tubing and the grout were plotted for each trial length of the splice.

The resulting plots show that the peak bond stress is the same for all lengths. The bond stress follows a hyperbolic tangent shaped distribution due to the linear elastic analysis. The highest bond stress observed for all lengths was 2.5 ksi (Figure 5). Since the loading on the splice is high, the actual bond stress distribution will not follow the linear elastic model. Instead the bond stress will redistribute as the concrete and grout begin to yield [12]. The level of redistribution can be based on the ratio of length of embedment to diameter of embedded item. Research suggests that full redistribution occurs for anchors up to a depth to diameter ratio of 25 [6]. If an 84 inch splice length is used, the length to diameter ratio is 84 in/14 in = 6, therefore the bond stress should fully redistribute.

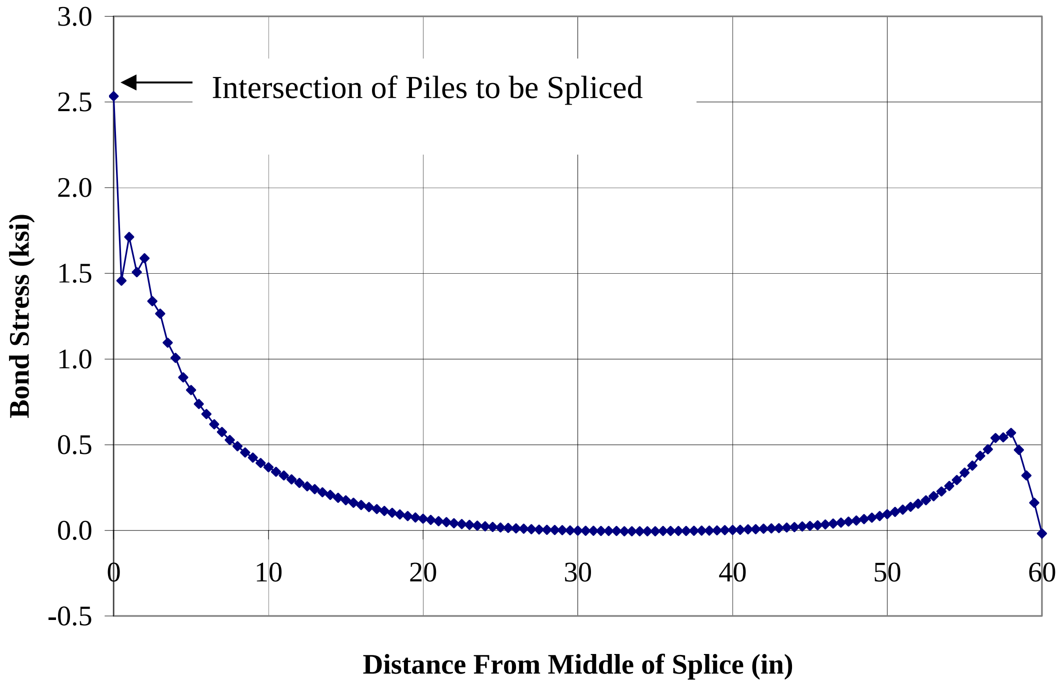


Fig 5a. Bond Stress Distribution at the Steel/Grout Interface for 60 inch Splice

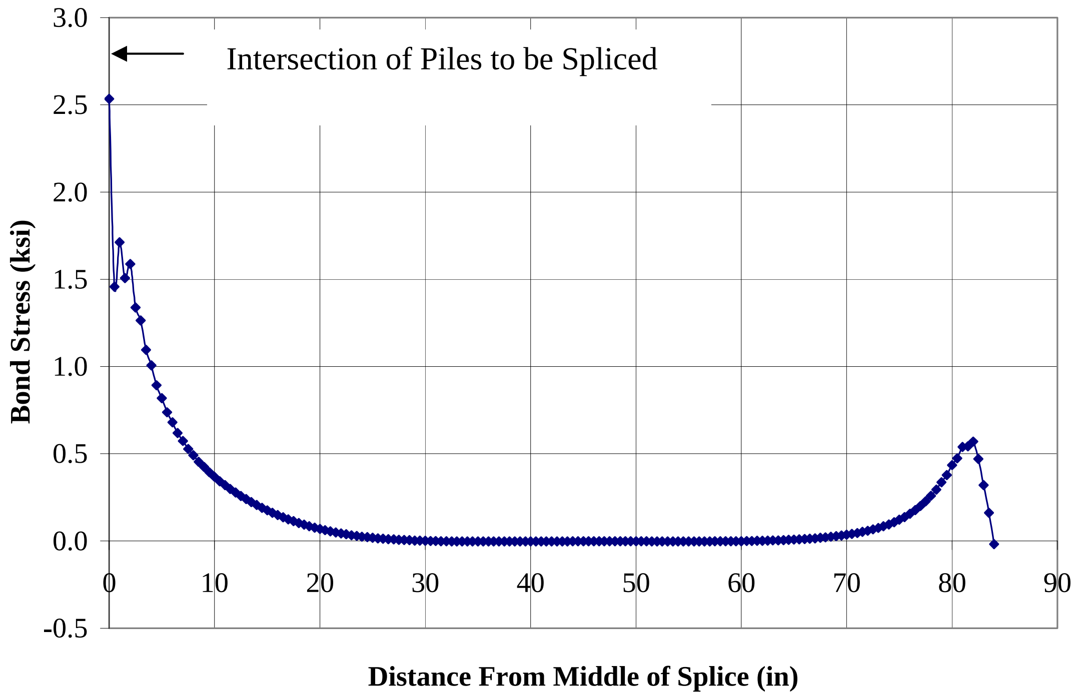


Fig 5b. Bond Stress Distribution at the Steel/Grout Interface for 84 inch Splice

Research by Cook [18] on bond stress for grouted threaded anchors that developed bond failure at the steel/grout interface found that the average uniform bond stress was 2,700 psi. Based on a 5% fractile, the design uniform bond stress along the steel/grout interface can be taken as 2,100 psi. Cook [18] found the average uniform bond stress for grouted anchors that developed bond failure at the grout/concrete interface to be 1,200 psi. Based on a 5% fractile, the grout/concrete interface design uniform bond stress can be taken as 900 psi. The design uniform bond stress values given above were used to determine the length required for the bond stress to transfer the tension load between the tube and the pile.

Steel/Grout Interface	Grout/Concrete Interface
$T = \pi \, d \, L \, \tau$	$T = \pi \, d \, L \, \tau$
$L = \dfrac{T}{\tau(\pi \, d)} = 7.0 \text{ in}$	$L = \dfrac{T}{\tau(\pi \, d)} = 16.4 \text{ in}$
T = 650 kips	T = 650 kips
$\tau = 2.1 \text{ ksi}$	$\tau = 0.9 \text{ ksi}$
d = 14 in	d = 14 in

The lengths determined from considering development of the strands, development of the tubing, and the bond stresses were compared to determine the minimum length of splice required. The development length of the tubing was the controlling length. Seven feet on each side of the splice for a total length of 14 feet is the minimum length of splice for the tensile design.

Other factors to consider when designing the splice are how to achieve an adequate bond between the tubing and the grout as well as between the grout and the concrete. The bond between the grout and the concrete will be achieved through the corrugated steel form liner, which is used during casting to form the void on the inside of the pile. The steel form liner provides adequate roughness for the grout and concrete to bond with each other. All piles which will be spliced must be cast with the correct form of corrugated steel.

The bond between the grout and the tubing will be achieved by welding of bars to the outside of the tubing. In determining what size of bar to weld and at what pitch, ASTM Standard A615 [2], Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement, was used. Using this standard, the tubing will have deformations welded onto it that satisfy the requirements for regular reinforcement. The requirements from ASTM for a No. 18 bar were used to determine the welds on the tubing. A ratio of the areas of the tubing and the No. 18 bar was used as a multiplier to determine the deformation requirements of the tubing. The ratio of the areas is $19.8in^2/4in^2 = 4.95$. Table 1 shows the requirements of the deformations of a No. 18 bar and the tubing.

	No. 18 Bar	HSS 14.000 x 0.500
Cross-Sectional Area, in^2	4	19.8
Maximum Average Spacing, in	1.58	$7.82 \approx 8$
Minimum Average Height, in	0.102	$0.51 \approx 0.5$

Table 1. Deformation Requirements

The maximum average spacing shall be measured by dividing a measured length of bar by the number of deformations and fractional parts of deformations on any one side of the bar. The minimum average height of deformation shall be determined by measuring at least two typical deformations. The weld shall be placed so that the included angle created between the line of deformations and the axis of the bar is not less than 45^{o}. The deformations shall be placed so that the deformations alternately reverse in direction on each side of the tubing. In accomplish this, two bars starting on opposite sides of the tube with be spirally wrapped around in the same direction. Plain steel bars with a diameter of a half an inch will be used for the weld since welding of rebar is discouraged. Figure 6 shows the details of deformations on the tubing.

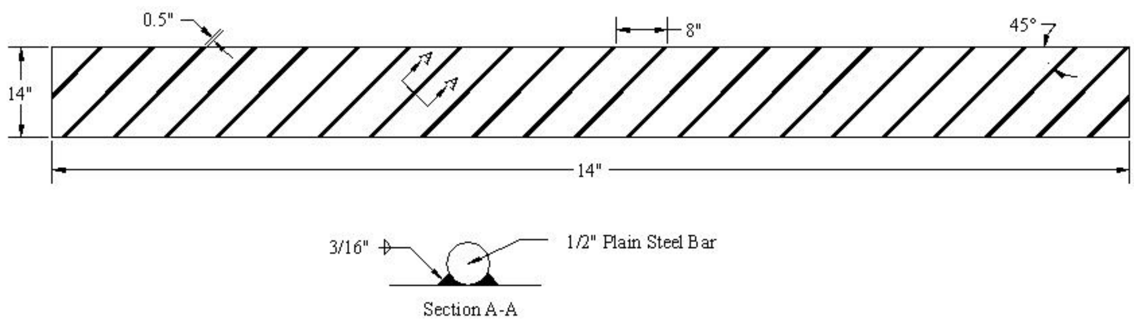


Fig 6. Deformation Detail

A second option for transferring load between the tubing and the grout which involves welding a round steel plate to the ends of the tubing was investigated. The steel plates would cause the splice to have a load path similar to that of an embedded bolt. A minimum length for the tubing in this scenario could be 60 inches on each side of the splice due to the development length requirements of the prestressing strands. The seven foot minimum would not be required because the steel tubing would not be acting as a piece of rebar. The steel plates would be 18 inches in diameter so that they could also act as plugs to stop the grout from extending down through the pile. However due to imperfections in the size and shape of the pile void created during precasting, this option was not considered viable. The imperfections in the pile void may prevent a steel plate from being able to be inserted into the void.

A long length of tubing may experience slenderness issues during compressive loading. In order to combat this problem, the steel tube is to be filled with grout or concrete. A concrete core inside the tube will brace the entire length of the tube and stop any buckling. This concrete will also help prevent corrosion of the steel tube. The concrete core creates a plug inside of the pile that will stop gases from escaping. To let gases escape, a 3 inch hollow pipe should be installed in the center of the plug. Figure 7 illustrates the details of the section tubing inside the pile.

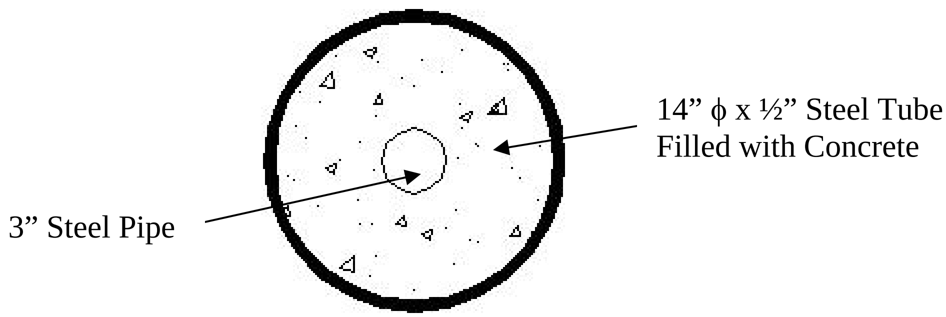


Fig 7. Interior Tube Detail

3.2 Flexural Design of Splice

When splicing a pile, the moment capacity of the spliced pile section should be close to the moment capacity of the unspliced pile section. This will ensure that the pile will perform as designed. The nominal moment capacity of an unspliced 30 inch pile was determined to be 966 kip-ft. Any strands that were in the compression zone of the section were neglected in the moment capacity calculation.   The calculation of f_{ps} is described in Section 3.1. A rectangular stress block was used to determine the force in the compression zone. The concrete strength was taken as 6,000 psi which meant that b_1 was 0.75 per ACI 318 [4] Section 10.2.7.3. Figure 8 illustrates the computation of the unspliced section nominal moment capacity.

$$a = \frac{A_p f_{ps}}{0.85 f'_c b} = 4.25 \text{ in}$$

$$M_n = A_p f_{ps}(d_p - \frac{a}{2}) = 966 \text{ kip - ft}$$

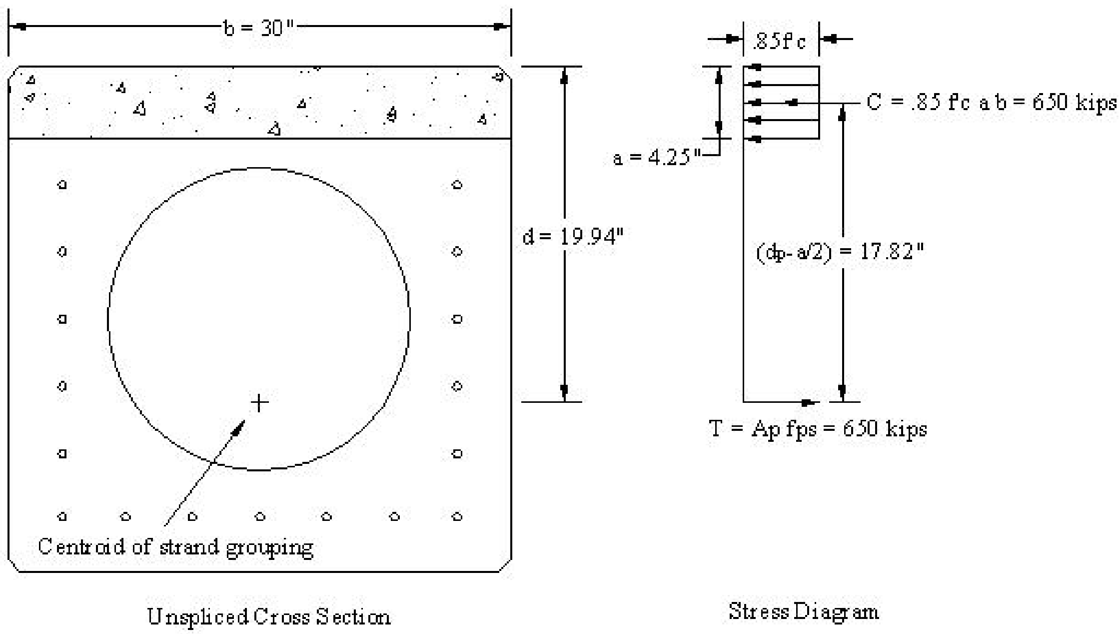


Fig 8. Unspliced Nominal Moment Capacity

The spliced section has a nominal moment capacity of 855 kip-ft. The moment capacity was computed using the composite section of the 14 inch pipe and concrete section at the joint without considering any prestressing across the joint. Any portion of the steel tubing that had not yielded was neglected for the moment calculation. The tubing is Grade 42 steel and the modulus of elasticity is 29,000 ksi, therefore the yield strain of the tubing was taken as .00145 in/in. A rectangular stress block was used to approximate the stress in the compression zone. The same concrete strength was used for the spliced section as was used for the unspliced section. The location of the neutral axis was determined by iterating until the compressive force and tensile force were equal.

The area of yielded steel was calculated to be 17.1 in^2. Figure 9 illustrates the computation of the spliced nominal moment capacity.

$$M_n = T(d - \frac{a}{2}) = 855 \text{ kip - ft}$$

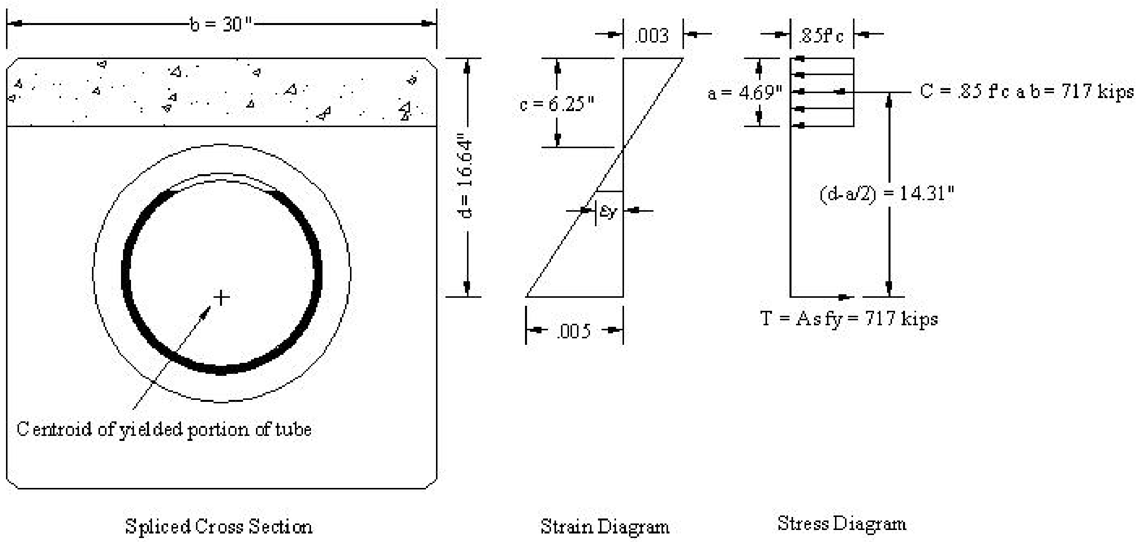


Fig 9. Spliced Nominal Moment Capacity

There is also a required length of splice based on the bond length needed to transfer flexural loadings through the splice. The design uniform bond stress of the grout is 900 psi as described in Section 3.1. Moment calculations for the spliced section show that a 717 kip load needs to be transferred by the grout. Assuming the interface between the portion of the steel tubing that was yielded and the grout is available to transfer the load; a length of 22.5 inches is required on each side of the splice.

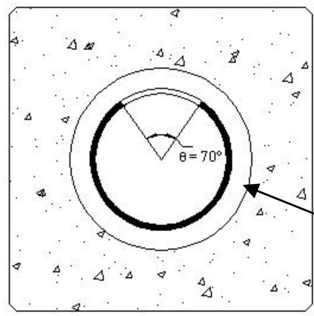
$$L = \frac{F}{\tau(2r\pi - \theta\frac{\pi}{180}r)} = 22.5\,\text{in}$$

Where: F = 717 kips

$\tau = 0.9$ ksi

r = 7”

$\theta = 70^{o}$



3.3 Shear Design of Splice

Earlier flexure testing done by the FDOT on this type of splice resulted in unexpected horizontal cracking and failure of the splice as described in Section 2.2. This failure can be explained by St. Venant’s Principle which states that stresses due to loading and geometric discontinuity approach a linear distribution at a distance approximately equal to the member height away from the discontinuity [4]. ACI 318 [4] Appendix A outlines a procedure on how to design members in the region within a distance equal to the member height from a loading or geometric discontinuity known as strut and tie modeling. A loading discontinuity can be envisioned in the splice by assuming no grout is present and a moment is applied as shown in Figure 10. The steel tube would be a separate unit inside of the pile and the point loads would develop.

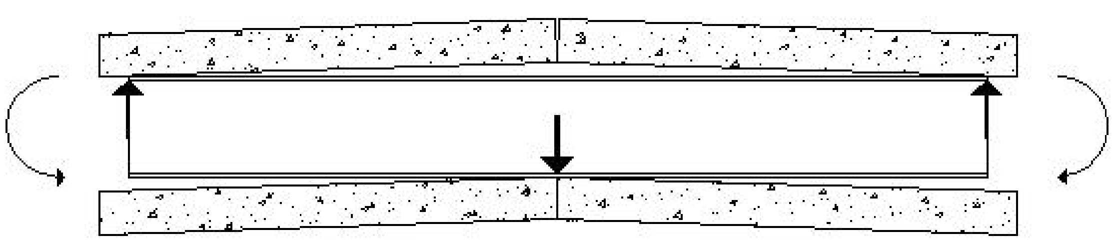


Fig 10. Concentrated Load Locations

Following St. Venant Principle, the splice will not behave as normally excepted within a distance of 30 inches from the concentrated point loads. A strut and tie model of this region has been designed as shown in Figure 11. Only one strut and tie model was designed and it will be used for both sides of the splice. The concentrated loads were calculated by assuming the applied moment was the spliced section's moment capacity of 855 kip-ft and dividing it by the tubing length of seven feet.

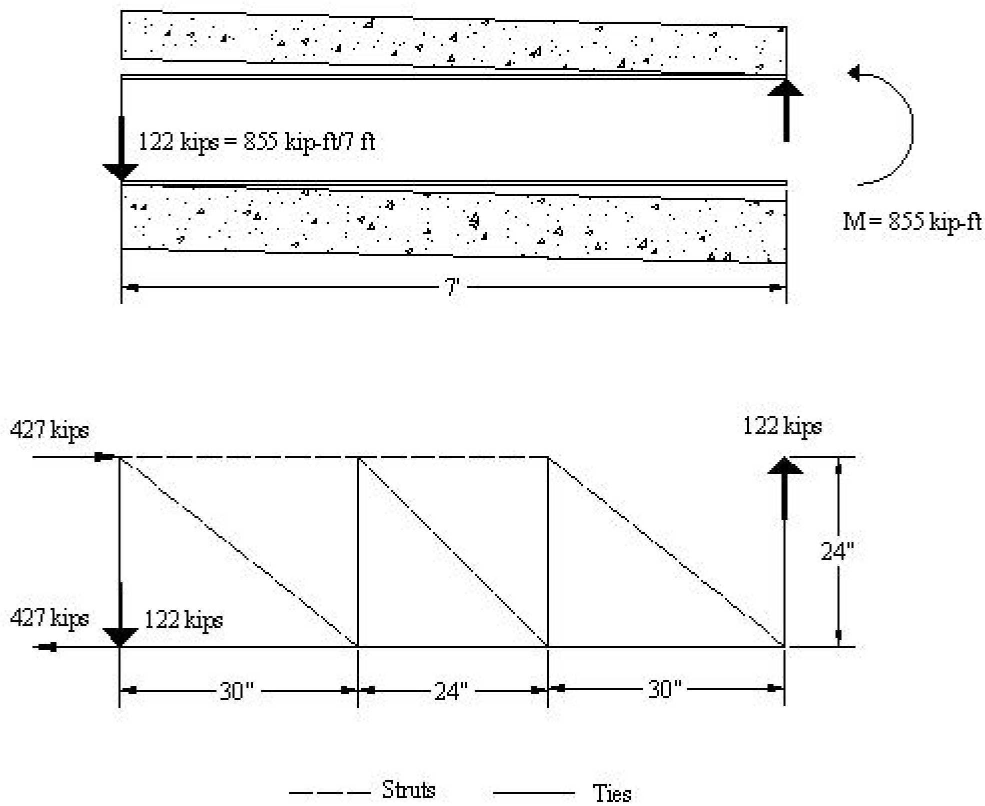


Fig 11. Strut and Tie Model

All forces were determined through truss analysis of the model. The vertical ties have a magnitude of 122 kips. The bottom chord tension ties and the top chord compression struts increase in magnitude from 153 kips to 427 kips. The prestressing strands will be acting as the bottom chord ties. Spiral ties will be acting as the vertical ties. The pile is currently constructed with W4 wire reinforcement [1] as shown in Figure 12 which has a nominal area of 0.04 in^2 [4]. The vertical ties require a steel area of 2.3 in^2 assuming the wire reinforcement has a yield strength of 70 ksi. With the assumption that the ties have a tributary width of 12 inches, W20 wire reinforcement would be required at a spacing of two inches (Figure 13) instead of the current spiral tie configuration. The spiral reinforcement would need to be placed for a distance of seven feet from the end of the pile.

$$A_{st} = \frac{F_u}{\phi f_y} = 2.3\,\text{in}^2$$

Where: F_u = 122 kips

$\phi = 0.75$

f_y = 70 ksi

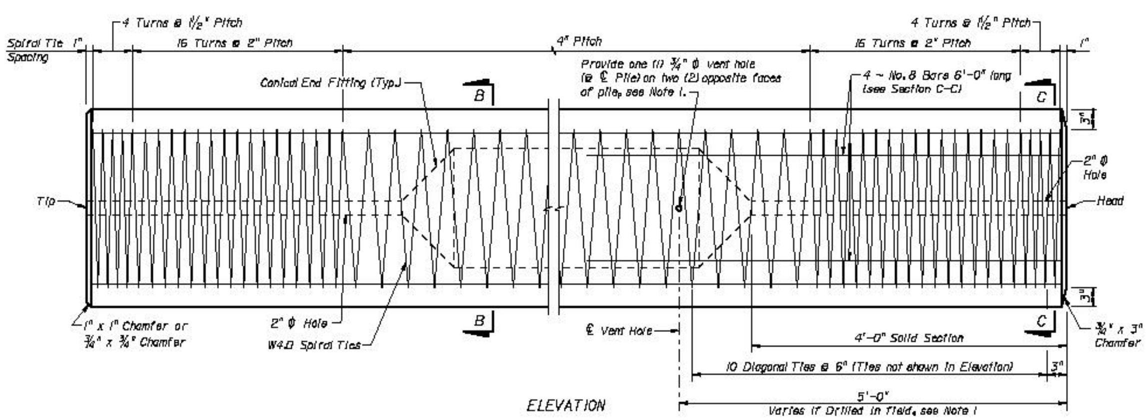


Fig 12. Current Spiral Tie Configuration on 30" Prestressed Pile

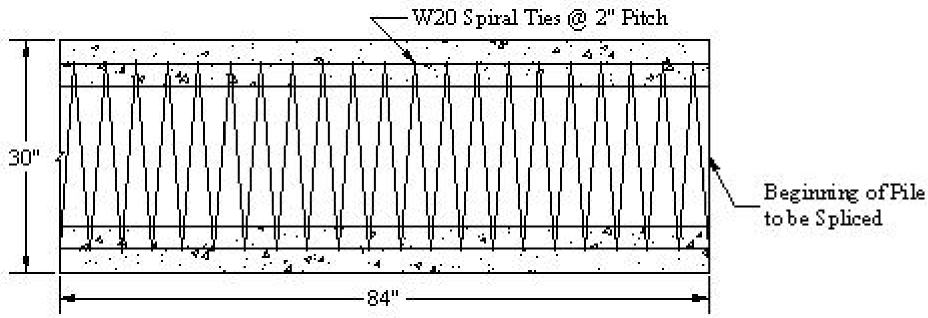


Fig 13. Spiral Tie Configuration for Region of Pile to be Spliced

3.4 Compressive Design of Splice

Compressive forces will be transferred between piles by mortar at the joint. The piles have beveled ends which will create a void at the mating surface (Figure 14). Mortar needs to be placed in this void in order to establish a bonded mating surface. Specifics on the mortar will be discussed in a later section of the report.

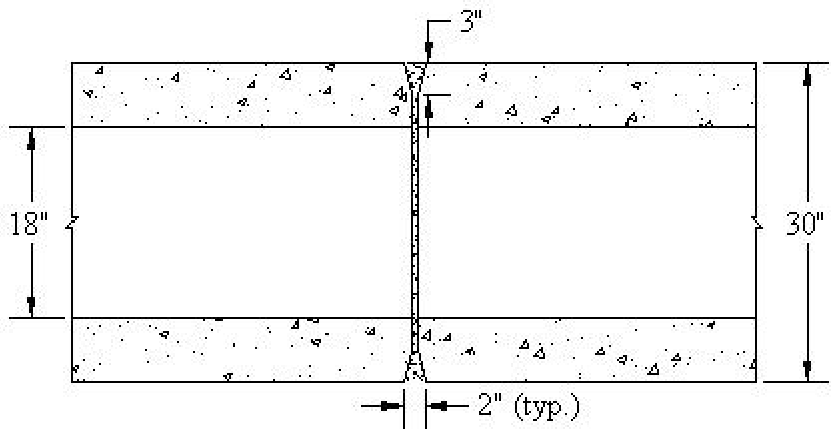


Fig 14. Beveled End Detail

In the event that the mating surface becomes unbonded, the compressive forces need to be transferred across the joint by the steel tube. The typical compressive stress on a pile during driving is 4,500 psi [11]. The grout must have enough length in order to transfer the compressive load between the concrete and the tubing. The grout has a design uniform bond stress of 900 ksi as described in Section 3.1. A length of 73 inches is required on each side of the splice to transfer the compressive forces from the concrete into the steel tubing.

$$L = \frac{f_c A_c}{\tau(\pi d)} = 73 \text{ in}$$

Where: f_c = 4500 psi

A_c = 646 in^2

$\tau = 900$ psi

d = 14 in

3.5 Grout

The grout used in the splice assembly has several requirements. First of all the grout must be fluid enough to move up through the splice and fill up the entire void between the tube and pile. It is very important that the grout makes a good bond for the entire length of the splice. The grout must also set up quickly so that the splicing operation does not slow down the driving process. Lastly, the grout must have the strength required to transfer the load from the tubing to the pile. One possible grout for this application is made by Sika and is called Sikadur 42, Grout-Pak [5]. Using this product will allow driving to continue after approximately ten hours.

Another material is needed to bond the mating surface between the two piles. A gap of a half an inch will be created and will be filled with the material. The material should be thick and come up to strength quickly so that construction may continue. One possible product is the Sikadur 31 [5].

4.0 Test Apparatus

The laboratory test phase of this project will involve performing tension and flexure tests on the splice.

4.1 Tension Test Apparatus

In order to simulate the stresses encountered by the splice during driving in the field, a testing apparatus for the tension test would normally load the pile a good distance away from the splice (Figure 15). This set up will not introduce forces into the system that would not be present in the field.

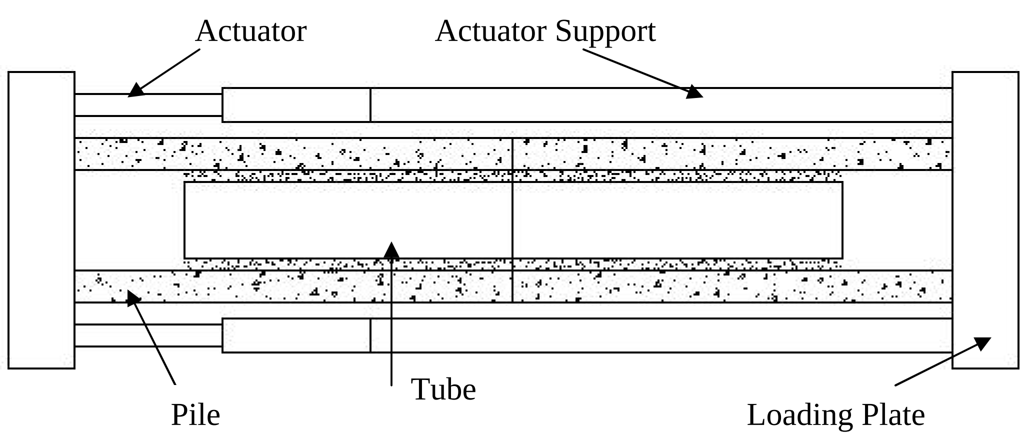


Fig 15. Test Apparatus Detail

Due to the long length of the splice, testing in this manner could be dangerous inside a laboratory since instability of a long actuator assembly could occur. A safer alternative to this test apparatus would be to load the pile over a shorter region. Finite elements models in ADINA were used to analysis this modified test apparatus (Figure 16). The test apparatus has a concrete collar installed around each pile two feet from the beginning of the splice. The distance of two feet was determined by examining the field loading model results from ADINA and noting that the bond stress drops to basically zero after 24 inches. The normal stresses in the tube and bond stresses between the tube and the grout from the test apparatus were compared with the results from the original models (Figure 17). The comparison shows that the test apparatus places the splice under the same peak stresses as the field loading model. Beyond the peak bond stresses, the test apparatus places the splice under a more conservative loading then field loading would. Conservative loading means that the test apparatus places higher stresses on the splice then the field loading model. These results demonstrate that the test apparatus will be effective and safe for laboratory testing.

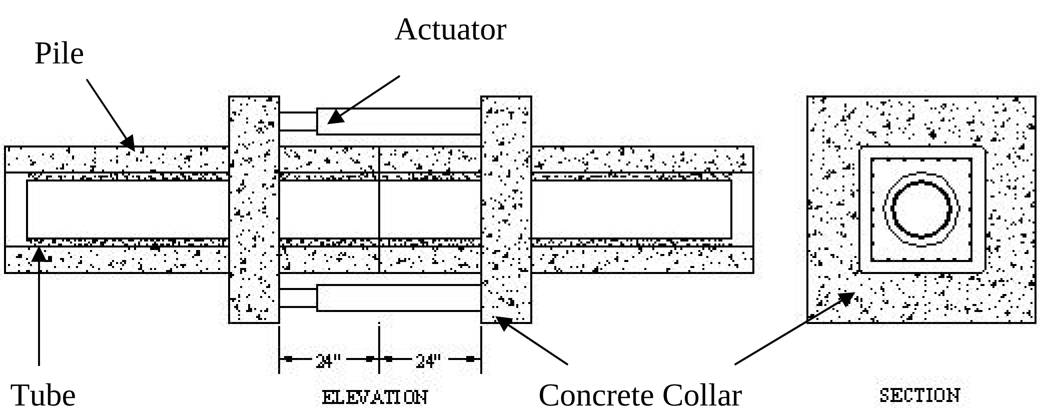


Fig 16. Collar Test Apparatus Detail

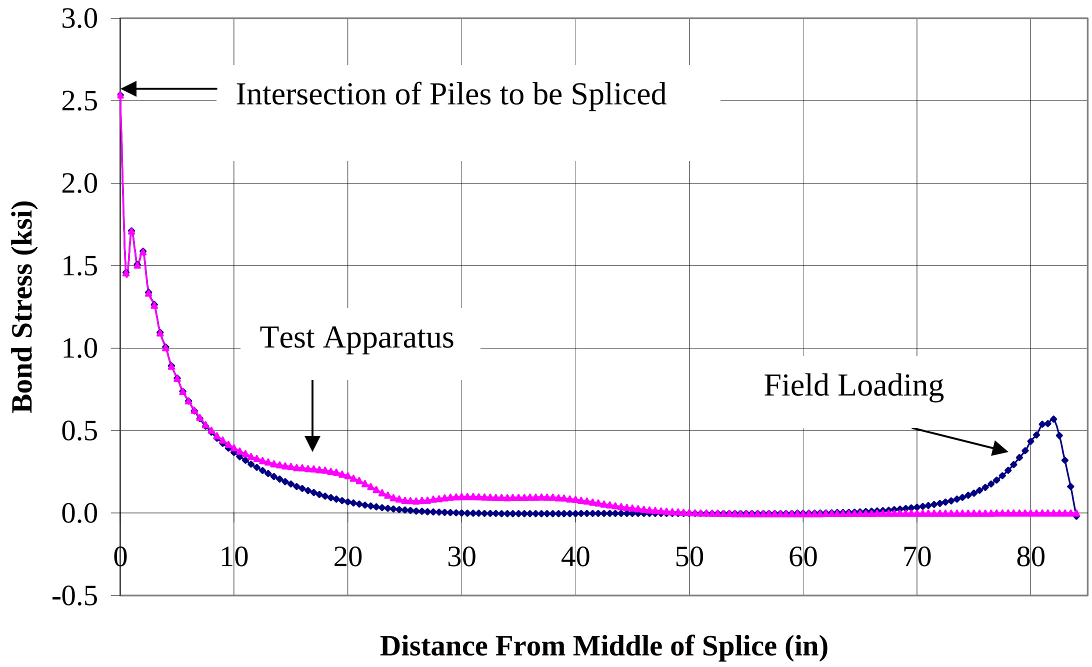


Fig 17a. Bond Stress at the Steel/Grout Interface for Seven Foot Splice

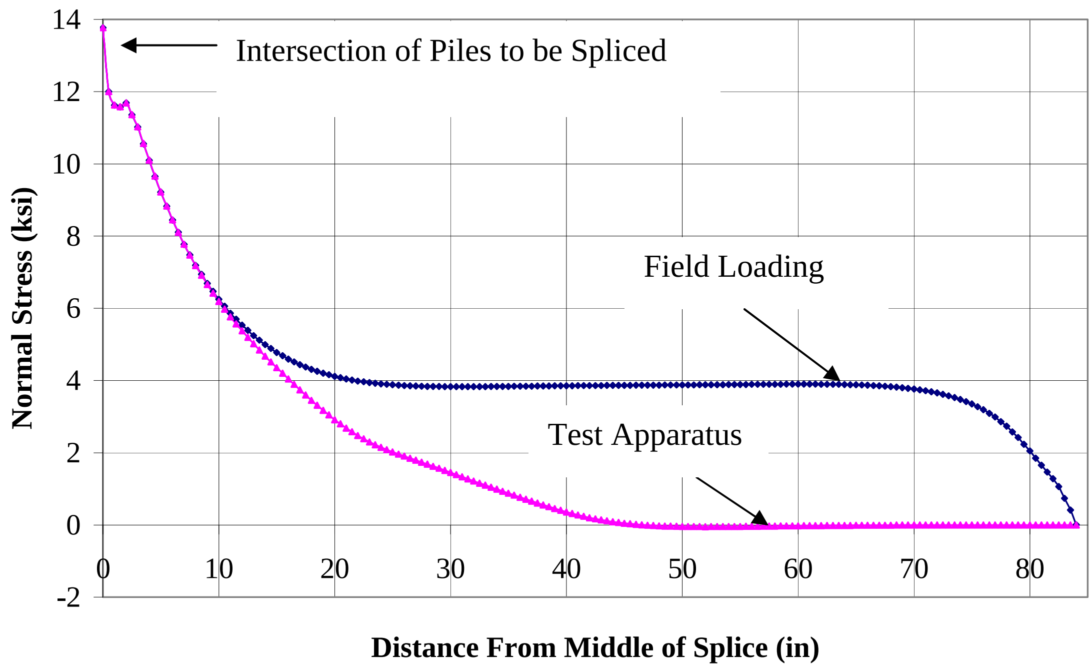


Fig 17b. Normal Stress in the Steel Tube for Seven Foot Splice

The collars will extend 18 inches from the face of the pile. Moment and shear friction calculations were done to compute the thickness of the collar and the amount of steel reinforcement needed. Calculations were performed assuming the collar thickness was 18 inches and a concrete strength of 4,000 psi. The load that the collar was designed to withstand is 850 kips. This load was chosen based on the loads that the tubing and prestressing strands would begin to yield. The tubing will yield at a load of 832 kips and the prestressing strands will yield at a load of 892 kips as described in Section 3.1. Load factors will not be used on this loading. The loading is assumed to be applied as two point loads on opposite sides of the pile.

Shear friction was considered according to ACI 318 [4] Section 11.7. The coefficient of friction was taken as 0.6 assuming that the surface of the pile where the collar will be attached has not been intentionally roughened and that normal weight concrete will be used in the collar. The strength reduction factor of 0.75 was used per ACI 318 [4] Section 9.3.2.3. A steel area of 15.7 in^2 was determined to be required and it will be laid out so that half of the area of steel is in the collar on opposite sides of the pile.

$$A_{vf} = \frac{V_u}{\phi f_y \mu} = 15.7\ \text{in}^2$$

Where: Vu = 425 kip

$\phi = 0.75$

f_y = 60 ksi

$\mu = 0.6$

The upper limits on shear strength as provided by ACI 318 [4] Section 11.7.5 were checked. The area of concrete section resisting shear transfer was taken as the interface between the pile and the collar. The shear strength provided was below the upper limits.

$$V_n \le 0.2f'_c A_c = 432 \text{ kip} \quad \text{or}$$

$$V_n \le 800A_c = 432 \text{ kip}$$

Where: Vn = 425 kip

f'_c = 4,000 psi

A_c = 30 in x 18 in = 540 in^2

The final step in designing the collar was to act as if the edge of the collar were a cantilevered beam. The 425 kip load was placed on the cantilever nine inches from the fixed end, inducing a moment of 319 kip-ft. A strength reduction factor of 0.9 was used assuming the section will be tension controlled [4]. A steel area of 5.28 in^2 is required to resist the moment in the cantilever. This amount of steel is required in the collar on both sides of the pile.

The moment resisting steel will be nine #7 bars. The bars will be hooked on the end in order to develop the bars in 18 inches. ACI 318 [4] Section 12.5.2 requires 16.6 inches to development a #7 standard hook.

$$l_{dh} = (.02\beta\lambda f_y/\sqrt{f'_c})d_b = 16.6 \text{ in}$$

$$\beta = 1.0$$

$$\lambda = 1.0$$

f_y = 60,000 psi

f'_c = 4,000 psi

d_b = 0.875 in

The steel will be placed at the top of the section. The shear friction steel will be 14 #7 bars placed in two layers below the moment resisting steel. These bars will also be hooked at the end. ACI 318 [4] Section 7.7.1 specifies that the bars require ¾ inch cover and this has been provided. The minimum clear spacing between #7 bars is one inch [4]. Figure 18 displays the reinforcement detail for the collar.

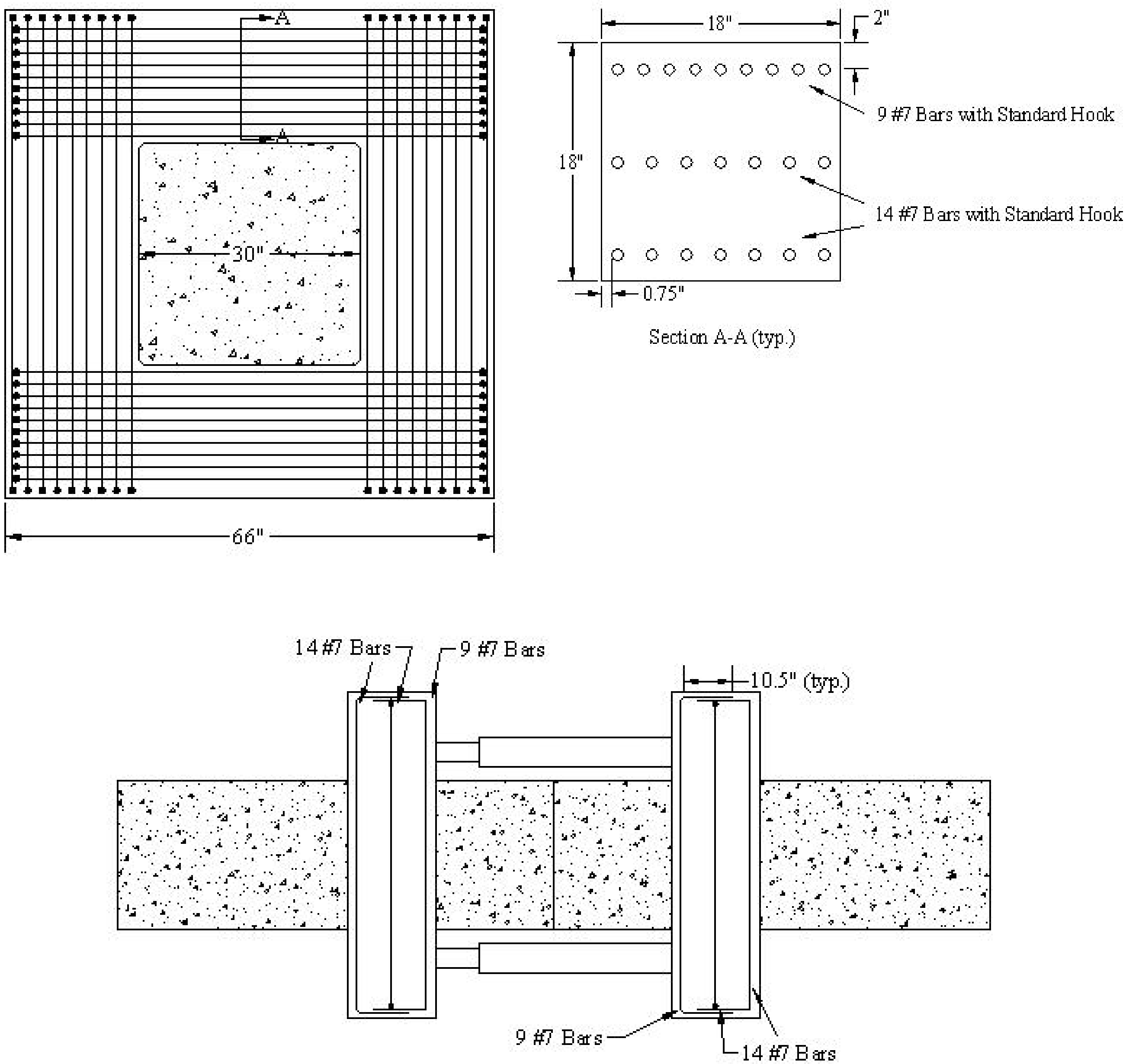


Fig 18. Collar Reinforcement Detail

4.2 Flexural Test Apparatus

Several concerns arise in regards to the flexural testing apparatus for the splice. The splice should be tested under a constant moment loading condition. Loading at a constant moment will ensure that the test is only testing the flexural capacity of the splice. A change in moment along the length of the splice would introduce a shear force onto the splice resulting in a test which is not purely flexural. The load points will be placed at least 30 inches from the ends of the steel tubing based on St. Venant's Principle as described in Section 3.2. A distance of 30 inches is far enough away so that the splice will have a linear stress distribution. The supports will be placed 60 inches away from the load points based on St. Venant's Principle. Figure 19 displays the loading locations and the moment and shear distributions for that loading.

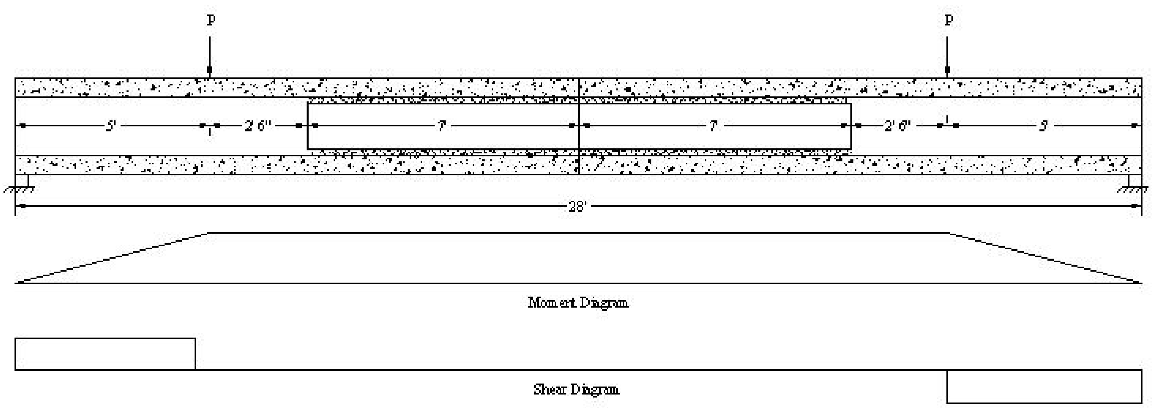


Fig 19. Moment Test Apparatus Detail

5.0 Field Assembly

Field assembly of any splice is crucial to its performance [3]. There are several key issues in the field assembly process.

5.1 Wind Loads

The piles must be perfectly aligned. This will be accomplished by using steel channel sections to clamp the piles into alignment. The channels have been designed to withstand a thunderstorm with a wind speed of 70 mph according to ASCE 7 [13] Section 6.5.13. The design was performed with the channels being a chimney structure in exposure category C. The pile section is assumed to extend 70 feet into the air.

ASCE 7-98 Section 6.5.13

$F = q_z G C_f A_f = 3390$ lb

Where: $q_z = .00256 K_z K_{zt} K_d V^2 I = 11.4$ psf

$K_z = 1.01$

$K_{zt} = 1.0$

$K_d = 0.9$

$V = 70$ mph

$I = 1.0$

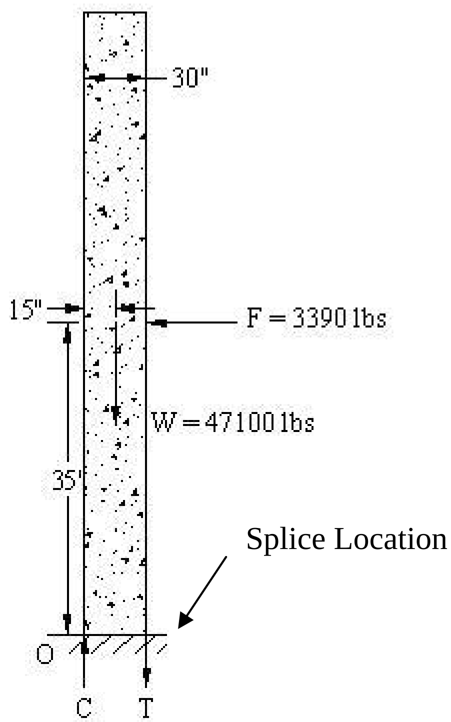
$z = 35$ ft

$G = 0.85$

$C_f = 2.0$

$A_f = 175$ ft^2

A wind force of 3,390 lbs will be acting on the pile at a distance of 35 feet above the ground. Summing the overturning and resisting forces acting on the pile will determine the magnitude of tension load to be carried by the channels. The weight of the concrete was taken as 150 pcf giving a weight per linear foot for the pile of 672 lb/ft. Assuming the pile is 70 feet long, the weight is 47,100 lbs. Figure 18 shows the forces on the pile during a wind loading. Summing moments about point "O" will give a tension force of 31,800 lbs to be carried by the channels. Two-thirds of the total weight of the pile was used in this calculation to be conservative. A steel channel section of C15 x 33.9 was chosen.



$$30''T + \frac{2}{3}(15''W) - 35'F = 0$$

$$T = \frac{35'F - \frac{2}{3}(15''W)}{30''} = 31.8 \text{ kips}$$

Fig 20. Wind Loading on Pile

The tension load will be transferred to the channels by threaded bars. The bars have been calculated to transfer a shear force of 21.2 kips each. Two bars of one inch diameter will be satisfactory in transferring the total tension load of 31.8 kips. Since the bars will be threaded, the effective cross sectional area was taken as 75% of the nominal area of the bar. ACI 318 [4] Appendix D was used to determine the capacity of the bolts.

ACI 318 Equation D-3:

$N_s = \phi n A_{se} f_{ut} = 42.4 \text{ kips}$

Where: $\phi = 0.6$

$n = 2$

$$A_{se} = .75\left(\frac{\pi d^2}{4}\right) = .589\,in^2$$

f_{ut} = 60 ksi

5.2 Installation Guidelines

1. The steel tube will be filled with concrete or grout and a three inch steel pipe. One half inch dowels will be welded on to the tube at a pitch of eight inches as shown in Figure 6. These steps may be performed off site before the time of splicing.
2. Holes will be drilled into the pile so that two bars which will hold the tubing and grout skirt in place can be inserted into the pile. The holes are to be drilled far enough down so that the bottom of the tubing is seven feet down from the splice. Two one inch diameter threaded bar inserted through the pile on adjacent sides (Figure 21). The grout skirt should be a rubber ring attached to the end of the tubing. Other type of grout skirt may be used as long as it prohibits the grout from filling the pile past the end of the splice.
3. The steel tube is lowered into the bottom pile until it rests on the bars (Figure 22). The tubing will be extending up seven feet past the end of the bottom pile.
4. The bonding material described in Section 3.4 is applied to the top of the bottom pile. Concrete shims will be used to ensure that a half an inch of the material will be at the mating surface. Threaded bars as described in Step 2 will be installed in the top pile so that the tubing can extend into the top pile

for seven feet past the splice. The upper pile is lowered down onto the steel tubing (Figure 23). The bonding material is then given time to cure.

5. Once the bonding material has cured, steel channels are placed on all four sides of the pile extending past the bars installed earlier (Figure 24). The channels are secured to the pile by bolting to the bars.
6. Once the steel channels have properly aligned the piles, the top pile may be released from the crane.
7. Holes for pumping in grout are drilled into the pile at the bottom of the splice. Two holes are drilled so that the grout may be pumped on two sides (Figure 25). Two holes are drilled in the upper pile a distance of seven feet above the splice to let air escape during pumping of the grout and to monitor the grout level.
8. Grout is pumped into the two holes that were drilled into the driven pile. Pumping continues until the grout starts to flow out of the upper holes. The holes are plugged up after pumping is completed.
9. Driving may continue once the grout has come up to adequate strength and the channels have been removed.

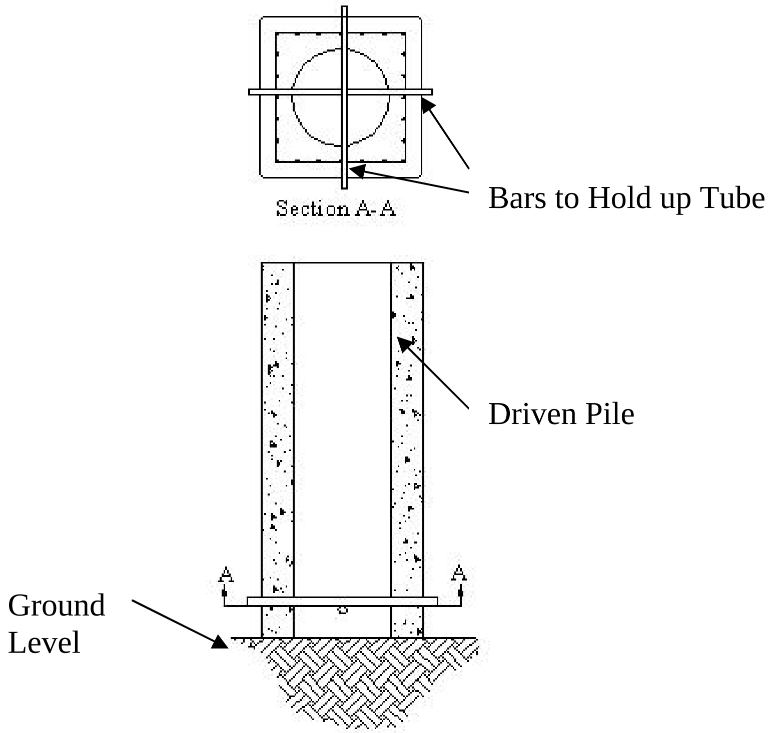


Fig 21. Step 2

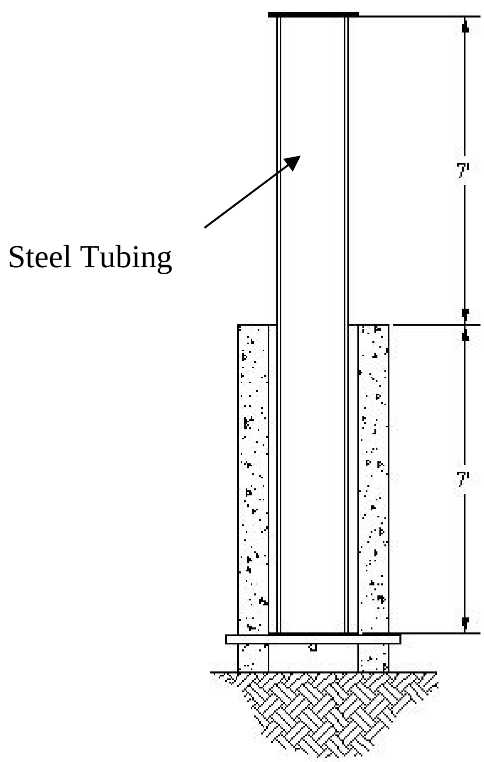


Fig 22. Step 3

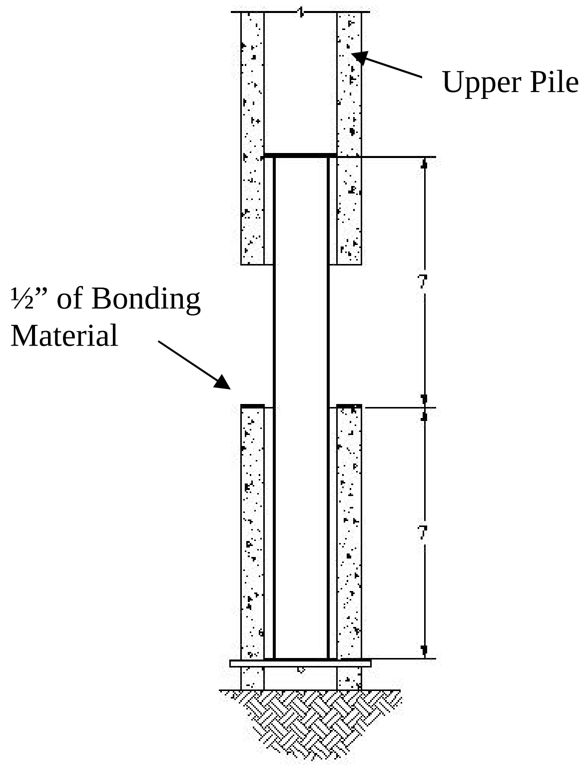


Fig 23. Step 4

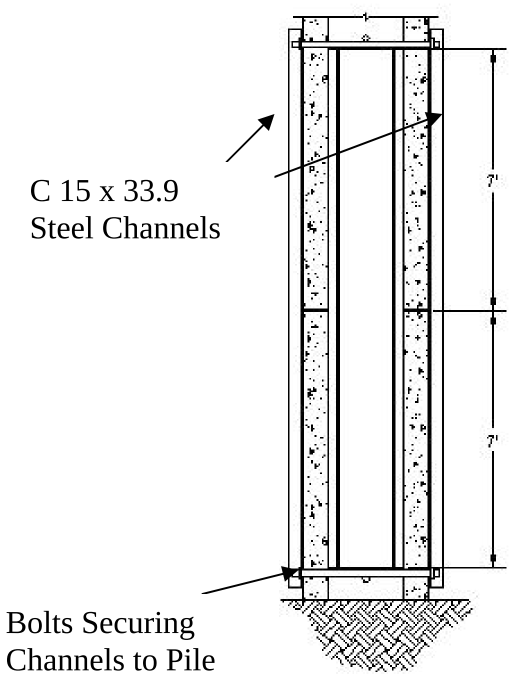


Fig 24. Step 5

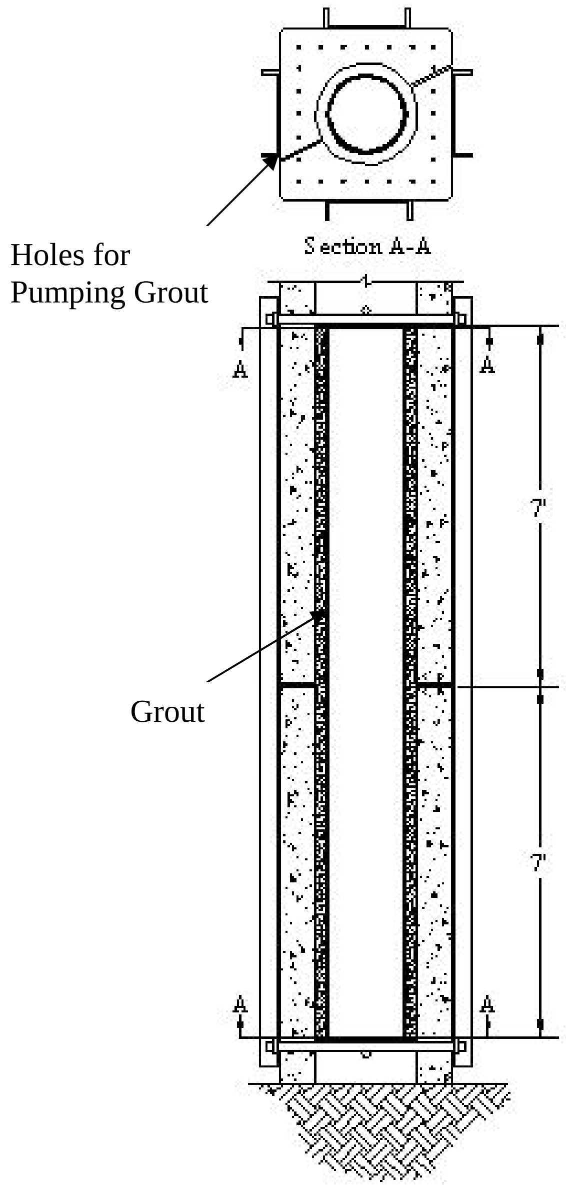


Fig 25. Step 7

An alternative for installing the splice would involve grouting the steel tube into the upper pile prior to the time of splicing (Figure 26). This could be done on site while the bottom pile is being driven. After the grout in the upper pile has been thoroughly cured, the tube will guide the upper pile onto the lower pile. Once in place grout is placed into the lower pile and allowed to cure before driving continues.

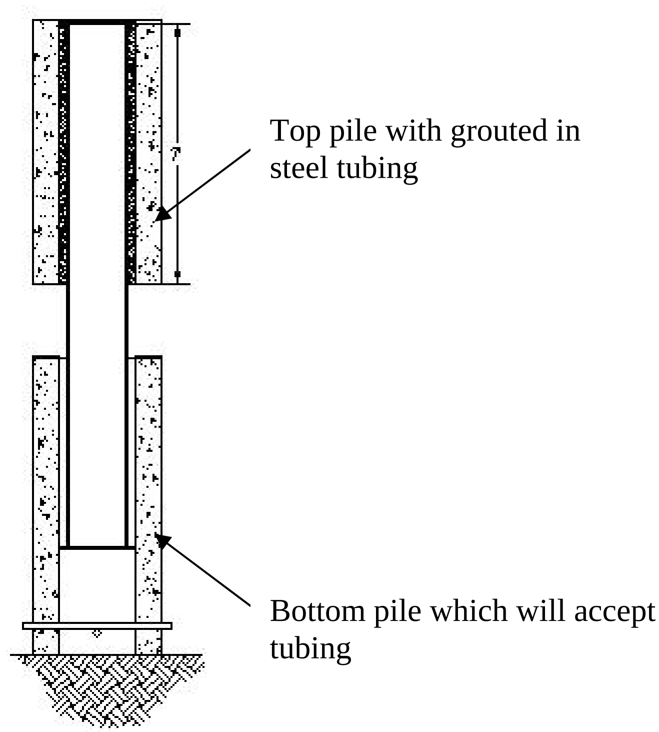


Fig 26. Alternative Installation Detail

6.0 Summary

The splice designed in this report will develop adequate strength in tension, compression, and flexure. Steel tubing with a 14 inch diameter and wall thickness of a half inch will be used. The steel tubing must extend at least seven feet from the splice on each side. This length ensures that the strands and the tubing have adequate length to develop. The length also provides enough distance for bond transfer of tension, compression, and flexural loads to occur. The tubing will have plain bars welded to the outside in order to establish bond between the tubing and the grout. The inside of the tubing will be filled with either grout or concrete to prevent slenderness problems in the tube. A three inch hole will be provided inside the filled tube to allow gases to escape.

The spiral reinforcing which is presently used in the pile will need to be changed. This will ensure proper performance during flexural loading of the splice if the grout were to become debonded from the tubing or pile. If debonding was to occur and proper spiral reinforcing was not present in the pile, concentrated loads would develop on the pile and splitting failures could occur as exhibited in previous tests [8].

The testing apparatus for the tensile test of the splice consists of two concrete collars being constructed around the piles. Each collar is placed two feet back from the splice. Actuators will apply the tensile load onto the collars. This test apparatus is being used to avoid instability of a long actuator assembly. The actuator assembly would be at least 14 feet long if the load were applied past the ends of the tubing. Finite element modeling of the test apparatus was performed to ensure the test apparatus is not significantly changing the stress distribution in the splice.

Proper installation of the splice will insure satisfactory performance of the splice. The grout must fill the entire void area between the pile and the tubing to ensure proper bond. Using the corrugated steel form in the pile is also important for establishing adequate bond. During installation, the two piles must be perfectly aligned to ensure a good bond at the mating surface. Steel channels will provide stability to the piles during wind loadings. Driving can not continue until the grout has come up to adequate strength. Figures 21 through 25 display the steps in the installation process.

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