UNIVERSITY OF SOUTH FLORIDA

Civil & Environmental Engineering

INVESTIGATION AND DEVELOPMENT OF AN EFFECTIVE, ECONOMICAL AND EFFICIENT CONCRETE PILE SPLICE

BDU79

FINAL REPORT

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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 yard	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)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC

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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
kip	kilopound	4.45	kilonewtons	kN

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2
kN	kilonewtons	0.225	kilopound	kip

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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16. Abstract

Structures such as bridges or tall buildings often require deep foundations in order to reach soil or rock strata capable of resisting the associated high loads. In Florida, concrete elements such as driven piles, drilled shafts or other cast-in-place alternatives provide a natural choice. This is somewhat in response to the economy of concrete in Florida, but for marine structures, concrete provides excellent durability via the concrete cover that protects the reinforcing steel. In some cases, however, bearing layers are too deep for precast piles due to limitations on trucking lengths and lifting weights. In such applications, drilled shafts have an advantage, but drilled shafts are not well-suited for all soil conditions. As a result, longer precast concrete piles must be spliced from shorter, more easily transported segments. Historically, these splices have presented problems.

The most successful and robust concrete pile splices have been mechanical splices that cast into the ends of the piles some form of steel connection detail where a key, bolts, or pins fasten the two segments together in a fashion more aligned with structural steel connections. These connections, while effective, must transfer tension stresses during driving from one pile segment to the next via reinforced concrete concepts (i.e. development length of large rebar cast into the ends of each pile segment). As a result, prudent state specifications restrict tension stresses to less than half that allowed an unspliced prestress pile. This study investigated the use of an alternative approach that incorporated post tensioning pile segments together to form a splice. The concept eliminates the limitations on tension stresses during driving.

This report presents the findings from numerical modeling, laboratory testing, full scale bending tests, and a driving demonstration of prestressed concrete piles spliced using the developed concept.

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Executive Summary

Splicing precast / prestressed concrete piles has historically been difficult as the attachment detail either requires preplanned considerations and cast-in connection details or requires on-site coring and doweling when unplanned pile extensions are needed. When the pile must be driven after splicing, the splice connection is prone to tensile failures due to the inability to transfer driving stresses through the connection and into the other pile segment. Focusing on preplanned splices, the Florida Department of Transportation (FDOT) limits tension stresses during driving to 250psi or 500psi for epoxy dowel splices or mechanical splices, respectively. This can limit the ability to efficiently drive the pile to the point that it may even be impossible. In response to the need for a more robust splicing methodology, FDOT sought an alternative concept which formed the basis of this study. To this end, an alternative approach that incorporated post tensioning was developed. The concept eliminates the limitations on tension stresses during driving.

Four basic tasks were undertaken over the progression of this study: (1) modeling, design and development of a splice concept, (2) laboratory scale testing of components and prototype spliced piles, (3) full scale bending tests of a finalized design, and (4) a pile driving demonstration using a spliced pile.

The design concept used embedded anchorages cast into the ends of splice pile segments where the stress imposed by post tensioning was localized to only that portion of the concrete pile halves. The net effect was the superposition of post tensioning stresses with the existing stresses caused by normal prestress construction practice. Therein, the concrete near the end of the pile segments that normally would have little to no prestress would then gain the tensile capacity that accompanied the post tensioning process.

Laboratory testing included testing individual components to be used in the splice and flexural testing of 14in square prestress piles spliced using the new method. In all four, 20ft long 14in square piles were cast, spliced and tested in four point bending. Two piles served as controls, while the other two were spliced from shorter 10ft segments. Information gained from the first splice was used to refine the second pile splice efforts. Test results showed that the cracking moment was virtually unaffected by the splice demonstrating tensile capacity has been transferred through the splice. The stiffness of the second splice pile was noted to be greater and the permanent deformation was substantially less when compared to the control piles. Lessons learned from the 14in pile tests were then applied to subsequent full scale tests.

Full-scale tests involved casting three, 40ft long piles: two control piles and one splice pile comprised of two, 20ft segments. After splicing, all three piles were tested in four point bending which again showed the cracking moment to be the same and the ultimate moment was 96% of the unspliced control and safely over the minimum required 600k-ft capacity at 636k-ft. Stiffness of the spliced was again higher and permanent deformation signicantly less. The upper half of the spliced pile and one half of a control pile were salvaged from the tested piles and then retested in three point (at approximate half the span length). When compared to the control pile, the upper portion of the splice pile showed approximately 20% increase in bending capacity. This is useful as the extra, unstressed strand in the upper pile can bolster moment capacity at the top of the pile where most foundations suffer the highest applied bending stresses.

The study culminated in a driving demonstration of a 100ft spliced pile specimen. Therein, both the maximum allowable tension and compression stresses were exceeded with no adverse effects to the splice.

The findings of this study suggest that the concept splice design can effectively restore full capacity for the purposes of withstanding pile driving installation and structural loading. Therein, full prestress levels were transferred through the splice zone which no other commercial method provides. Additionally, the splice design provides full corrosion protection as no components of the splice breach the concrete cover.

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Chapter 1: Introduction

1.1 Background

Prestressed concrete piles are widely used in bridge foundations where the most common has a solid square cross-section (Figure 1.1). Other section shapes including circular or octagonal are also available and all can be optionally voided to reduce weight, material waste, and enhance portability [1]. Whereas in many parts of the world the term pile encompasses both precast and cast-in-place deep foundation elements, in the US, pile implies a driven substructural element that is pre-fabricated and transported to the project site. For concrete piles, most are constructed in a prestress casting yard, transported to the site, and driven with a diesel hammer. Alternate hammer technologies might be used that include drop hammers or those where the lift is hydraulically or pneumatically activated. In some countries, such as Japan, prestressed concrete piles are grouted into preformed holes to abide by strict vibration and noise ordinances. Nevertheless, the design of the pile involves consideration not only of relevant geotechnical conditions and loading requirements but also transportation, handling and installation procedures.

Figure 1.1. Thirty inch square solid prestressed piles driven into a bridge bent layout.

A comprehensive guide on the design, manufacture and installation of concrete piles is available [2] wherein the most critical requirements in design are: (1) pile driving stresses must not damage the pile, (2) the installed pile should withstand imposed loads, and (3) the pile-soil interface must support the pile loads.

Like concrete girders, columns, and decks, the application of prestressing technology to precast concrete piles has had a dramatic effect on tensile/cracking load, installation efficiency, and cost. Comparatively, the 1954 plans for the Gandy Bridge provided options for both reinforced concrete and prestressed 20in square piles [3]. Therein, 8 – No. 8, Grade 40 bars (1.6% steel) could be optionally replaced with 16 – 7/16in, Grade 250 stress relieved strands (0.4% steel) which provided 618 psi of effective prestress. The same size pile by today's standards uses 16 – 1/2in strands to provide higher levels of prestress (e.g. 1045 psi) [1].

Despite the advances in precast piles and prestress technology, size limitations still exist due to the required transportation from the casting yard to the site. This is further complicated in cases where deep water or pockets of unsuitable soils demand piles of lengths greater than that which can be practically handled. While accommodations for these circumstances are easily addressed when using steel piles (welded connections of shorter sections), for concrete piles reliable means to fuse together pile segments are also needed.

1.2 Need for Splices

Given the above limitations of concrete piles, they are preferably designed for construction in one piece. However, field splices may be needed where pile lengths are too great to be transported economically or unexpected soil conditions require longer pile lengths than anticipated. To eliminate trucking-related limitations, piles can be cast on-site. Figure 1.2 shows a temporary casting yard on the banks of the Albemarle Sound in North Carolina where 30in piles (shown in Figure 1.1) were cast in a self-stressing bed to better manage the long lengths and heavy trucking loads.

Figure 1.2. Self-stressing, temporary casting beds used for the Haughton Road Bridge in North Carolina to cast solid 30in square piles.

When splices are required, they may be driven until below ground or may remain above. In either case, they must be designed to withstand imposed loading and driving forces if they are driven after splicing. Splice designs must prevent discontinuity in force transfer at the splice point since this can initiate failure.

Splicing concrete piles, the topic of this report and study, has plagued construction for decades and out of hundreds of concepts very few have been shown successful. A post tensioned option for splicing prestressed concrete piles is presented herein that has the potential of providing the same level of performance gain to pile splicing that was brought about by prestressing over reinforced concrete.

1.3 Report Organization

This report discusses the development of a post tensioned pile splice option that uses embedded anchorages that in turn transfer full prestress through the splice region. This concept therefore allows spliced piles to be driven to full permissible tensile stresses just as if the pile were not spliced at all. Additionally, as the anchorages are all within the reinforcing steel cage, no adverse effects are introduced with regards to long-term corrosion durability.

The organization of the report is broken into the five ensuing chapters. Chapter 2 provides a background into various splicing options historically used throughout the world and in the state of Florida. Chapter 3 is discusses the steps taken to design and develop a new splicing technique along with component testing. Chapter 4 deals with casting 14in square prototype splice piles along with unspliced controls, and testing them in four-point bend. Chapter 5 discusses the efforts that parallel Chapter 4 but for full scale 24in square prestressed piles which are more in line with the normally used piles by FDOT. And finally, Chapter 6 provides a discussion and summary of the results as well as recommendations for the use of the newly developed splicing system.

Chapter 2: Literature Review

This chapter provides a brief history of pile splices, and the importance of / need for a more robust, seamless splice alternative.

2.1 Prestressed Piles

In short, the concept of prestressing is simple: by holding the reinforcing steel in tension before the concrete is poured around it, waiting for the concrete to cure to a sufficient strength around the pre-tensioned steel, and then releasing the pre-tensioning force in the steel, the concrete is then pre-compressed. This shifts the origin of the concrete stress / strain diagram and gives the concrete an apparent tensile strength in its unloaded state. When applied to piles, this allows the concrete to stay in compression when tensile stresses that normally occur during driving are imposed. Without it, far more steel is required to control tensile cracking.

The FDOT specifications for piles address strength and construction details that include: permissible driving stresses, axial and bending capacity and considers the location and type of splices that might be encountered.

2.1.1 Pile Specifications for Driving Stresses

The FDOT Specifications 455 Section 5.11.2 limits the stresses developed in prestressed concrete piles during driving based on the Wave Equation [4] wherein the following equations are used to determine the maximum allowed pile stresses measured during driving:

$$s_{apc} = 0.7\, f'_c - 0.75\, f_{pe} \qquad (1)$$

$$s_{apt} = 6.5\, (f'_c)^{0.5} + 1.05\, f_{pe} \qquad \text{(2a) for piles less than 50 feet long}$$

$$s_{apt} = 3.25\, (f'_c)^{0.5} + 1.05\, f_{pe} \qquad \text{(2b) for piles 50 feet long and greater}$$

$$s_{apt} = 500 \qquad \text{(2c) within 20 feet of a mechanical splice}$$

where:

s_{apc}= maximum allowed pile compressive stress, psi
s_{apt}= maximum allowed pile tensile stress, psi
f'_c= specified minimum compressive strength of concrete, psi
f_{pe}= effective prestress (after all losses) at the time of driving, psi, taken as 0.8 times the initial prestress force (fpe= 0 for dowel spliced piles).

Drivability must be considered for pile splices within the proposed study. Piles without splices are permitted to develop between 1200 and 1500 psi of tensile stresses during driving (*f'c* = 6000 psi) based on various FDOT approved 24in pile strand configurations.

Once a splice is made, maximum allowable tensile stresses are significantly reduced to either 250 or 500 psi for dowel or mechanical splices, respectively. This causes a 58 to 83% reduction in the allowable tensile stresses during driving depending on which initial level is assumed (1200

or 1500psi). As a result, the contractor must reduce the driving energy which increases the time required to install the piles. The 500 psi limit applies to the only FDOT-approved mechanical splice (Sure-Lock) which in other states is driven successfully to 1000 psi tensile stress. As more mechanical splice options become available, this may warrant a manufacturer specified rating.
If a spliced pile can maintain full prestress through the spliced region, the driving efficiency will not be affected and associated delays can be avoided. This was a primary goal of this study.

2.1.2 Splice Specifications

Pile splices are categorized into three types by FDOT Section 455-7.7 and Standard Index No. 20601 as (1) Unforeseen, non-drivable, (2) Unforeseen drivable and (3) Preplanned drivable. The first case is used after capacity has been satisfied during driving but the top of pile is too low to meet the cutoff elevation. The upper half of this splice is a cast-in-place build-up and not intended to be driven. The second case addresses the situation where capacity has not been met unexpectedly and it may or may not be known how much farther it will need to be driven; or, in cases where a build-up exceeds the maximum permissible length of 21ft. Epoxy-doweled splices are essentially the only option available where the bottom pile must be cored through a template to assure proper dowel rod placement and alignment. The upper pile is cast with embedded dowel bars (Figures 2.6 and 2.8). The third case is perhaps the most desirable of the three categories where splices are planned in advance and necessary provisions are met. This case can use either an epoxy-doweled splice with preformed holes in the lower segment or the only mechanical splice on the QPL which is the Sure-Lock / Kie-Lock system. Use of the Sure-Lock requires a waiver from the Buy America Provisions from FHWA. The latter two cases apply to this study wherein the third has the greatest potential for immediate improvement. However, developing means to address unforeseen pile splices will be thoroughly reviewed.

Although driving stresses are limited to 250 or 500 psi in tension, the strength of a pile splice must meet the following criteria for compression, tension or bending (Table 2.1) once in place [4]:

Compressive strength = (Pile Cross sectional area) x (28 day concrete strength)

Tensile Strength = (Pile Cross sectional area) x 900 psi

Table 2.1. Bending strength of FDOT prestressed concrete piles.

Pile Size (inches)	Bending Strength (kip-feet)
18	245
20	325
24	600
30	950

2.2 Pile Splices

The ACI guide [2] states that splices may need to be designed to resist the same compression, tension, bending and shear as the pile cross-section; FDOT has similar limitations discussed above [4]. The potential for corrosion must also be addressed and torsional loading can arise if the pile helmet (that transfers the load to the pile) fits too tightly on the pile preventing it from rotating relative to the pile segment already embedded in the soil. Some of these requirements can be minimized by appropriately locating the splice when possible.

A number of proprietary splicing systems are commercially available [5-14]. The connection between the two segments can be made in a variety of different ways each with particular advantages and disadvantages. Splices are generally considered to be a weak link that most drastically affects driving. Failure of a pile splice can occur either in the splicing component itself, or in the connection between the splicing component and the pile segments. Because of these two independent, but equally important, possible modes of failure, pile splices can be categorized in two ways: (1) by the mechanism in which the splicing component joins the pile segments together, and (2) by the way in which the splicing component is anchored to the pile segments (Figure 2.1).

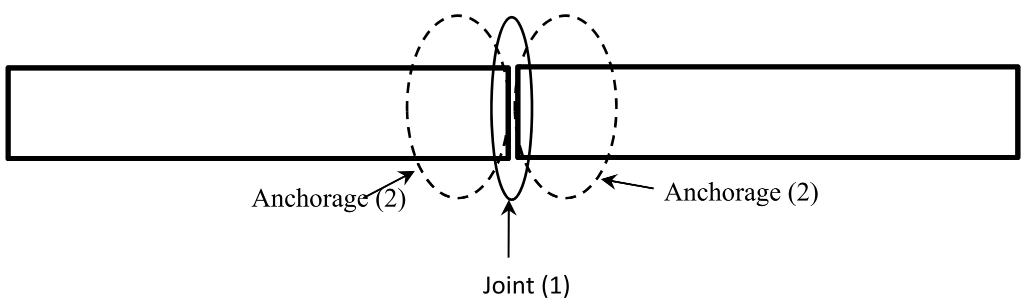


Figure 2.1. Pile splice components used to categorize splice types.

Both joining and anchorage approaches are presented herein.

2.2.1 Joining Mechanism Categories

In 1974, Bruce and Hebert categorized concrete pile joints using the connection types listed below. Surprisingly little has changed since then; however, newer approaches are also discussed.

- Welded – As the name implies the ends of each pile segment are welded together for form this type of splice. This applies to both steel and concrete piles. The latter requires steel end caps, plates or similar. Figure 2.2 shows a type of welded pile splice tested by FDOT [15].
    - *Advantages*
        - Good flexural and tensile capacity.
    - *Disadvantages*
        - High equipment and labor costs.
        - Susceptible to fatigue.

Figure 2.2. Welded pile splice

- Sleeved – This method uses a steel sleeve with a center stop / divider that is slid over the top of the bottom pile segment, and the bottom of the top segment is inserted into the upper portion of the sleeve (Figure 2.3). Bruce and Hebert presented this as an un-bonded connection.
    - *Advantages*
        - Low cost and very little labor.
    - *Disadvantages*
        - Very little flexural capacity and no tensile capacity.

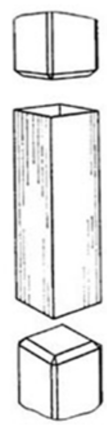


Figure 2.3. Sleeved pile splice

- Mechanical – Steel end caps, usually one male and one female, with specially designed matching holes or grooves, are cast onto the end of each pile segment. In the field, the steel caps are mated and mechanically connected, typically by high strength steel pins or wedges which are designed to fit into the holes or grooves (Figure 2.4).
    - *Advantages*
        - Minimal installation time.
        - Good flexural and tensile capacity (500-1000 psi tensile).
        - Can develop full bending capacity.

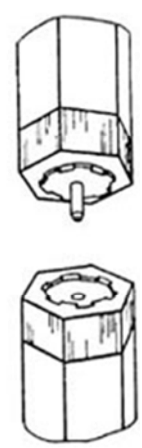


Figure 2.4. Mechanical pile splice

- *Disadvantages*
    - The majority of mechanical splice systems are proprietary due to the specialty design needed for the interlocking pieces.
    - Many are manufactured by foreign companies.

- Connector Ring – This method is similar to the "sleeve" but a short length of steel pipe is additionally cast into the end of each pile segment, protruding from the ends. In the field, the pipe sections are slid into the lower and upper portions of a steel collar. The pipes and collar may be bonded (welded, wedged, etc.) or left un-bonded acting like a sleeve splice (Figure 2.5). In this way, this could be also classified as a sleeve, welded and/or a mechanical splice.

    - *Advantages*
        - Versatile – may be chosen in the field whether or not to be bonded or unbonded, depending on the specific needs of strength, time and cost.
        - Strength can be considerable depending on the anchorage to the pile and the connection detail.

Figure 2.5. Connector ring-type splice

- *Disadvantages*
    - When bonded or unbonded, strength is limited, and time must be allowed for grout or epoxy to cure.
    - When welded, susceptible to fatigue.
    - Material cost is relatively high.

- Doweled – Rebar dowels cast into one segment, and protruding from its end, are inserted into epoxy or grout filled holes drilled or preformed in the end of the lower segment (Figure 2.6).

    - *Advantages*
        - Low material cost and little labor.

    - *Disadvantages*
        - Long idle time while epoxy or grout cures.
        - Reduced flexural and tensile capacity e.g. 250 psi maximum tensile driving stress, [4].

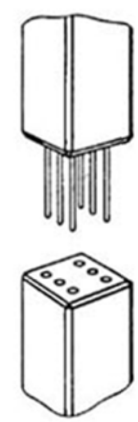


Figure 2.6. Doweled pile splice.

- Post-tensioned – Post-tensioning ducts are cast into the pile segments. The splice is achieved by aligning the two pile segments, running post-tensioning strands or bars through the ducts followed by stressing and anchoring the strands or bars (Figure 2.7). The system shown only requires the first pile to provide an anchorage. Subsequent sections are coupled to the first anchor bar. Dywidag and Williams Form bar systems are well suited for this application [16, 17].

- *Advantages*
    - Excellent flexural and tensile capacity.
    - Does not rely on external steel components, therefore more durable in marine environments.
    - Can be spliced repeatedly.

- *Disadvantages*
    - Constructability is difficult where post tensioning is performed high above ground.
    - Material cost of bars can be high.
    - Foreign provider.

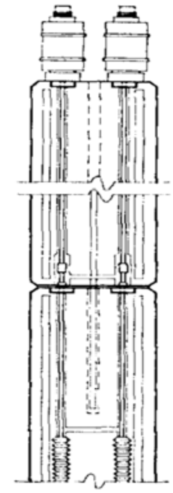


Figure 2.7. Post-tensioned pile splice.

2.2.2 Anchorage Categories

Alternatively, concrete pile splices may be categorized by anchorage state as either stressed or unstressed ends.

- Unstressed End Anchorage – This is the most common prestressed concrete pile where the strand does not develop full prestress without a sufficient transfer length from the ends (e.g. transfer length = 60 times the diameter of the strand). Transfer length differs from development length in that development refers to embedment required to resist ultimate bar or strand capacity whereas transfer length only requires enough embedment to resist the jacking load (≈ 0.7 ultimate). At the ends of spliced piles with unstressed ends, both modes of bar and strand embedment superimpose stresses. At the end of the pile, the splice element is exerting full tensile loads where the concrete has the least amount of prestress to resist tension.

    - *Advantages* - Simple and can be inexpensive in regard to constructability.

    - *Disadvantages* - Capacity limited by development length of reinforcing steel as well as by prestress losses in the ends of the piles.

      - Congestion between reinforcing steel and prestressing strands.

- Stressed End Anchorage – This stress state is typically equated to post tensioned elements where the concrete is precompressed all the way to the ends of the pile. The stress may be imposed by prestressing or by post tensioning. True post tensioning procedures have fewer losses as the elastic shortening occurs during the tensioning. Systems that stress the strands with end plates and release after concrete cures cannot develop the same level of initial prestress. The splicing component can be strands passed through the splice or coupling plates which are often the same components providing prestress to concrete pile. Once the strands are stressed and the pile is cast, the splicing component serves as an anchor for the stressed strands and, likewise, the stressed strands serve to anchor the splicing component.

- *Advantages* - Excellent capacity.

- *Disadvantages* - may require specialized casting equipment
  - some applications have complicated stressing configurations
  - post tensioning in the field may have special safety concerns

2.2.3 Concrete Pile Splicing Systems

The need for pile splicing systems which are strong, cost effective, constructable, and durable has produced a vast array of proprietary and non-proprietary systems which utilize different combinations of the joining and anchoring mechanisms mentioned above. Splicing systems are summarized below which are commonly used, in development and those under consideration, as well as those which are relevant to this study.

- **Epoxy Doweled Splice** – *Doweled splice with unstressed end anchorage.*

  This is the predominant method currently used in the state of Florida. It is a doweled splice, as described previously, which is bonded by epoxy (Figure 2.8). Although this splice type has the disadvantage of waiting for the epoxy to cure to develop tensile capacity, this can be circumnavigated by simply providing a cushion between the pile segments and driving the spliced piles with wet epoxy. This dampens the impact force that may otherwise damage the spliced ends of the piles when allowed to separate during driving. The in-place flexural strength of the pile splice is still limited and the tensile stressed during driving are restricted to about 250 psi [4]. Figure 2.8 shows the basic steps for constructing an epoxy dowel splice. Note: at the completion of this process, the crane must hold the upper segment in correct alignment until the epoxy cures to sufficient strength.

Figure 2.8. Epoxy-doweled splice: bottom pile cored (top left); upper segment cast with rebar hoisted for insertion (top right); aligning rebar while lowering (bottom left); pouring epoxy prior to final descent (bottom right).

- **Kie-Lock (formerly Sure-Lock) Splice** – *Mechanical splice with unstressed ends.*

  This is a mechanical splice with unstressed end anchorages. It uses two steel caps, one male and one female, with matching grooves that line up when the caps are joined and are bonded together by high strength steel pins which are inserted into the alignment grooves (Figure 2.9). The steel caps are anchored to the piles by rebar, extending about 7 feet into each segment. A disadvantage of the system is that, on the female side of the splice, the rebar anchorage must be bent in order to have enough cover to develop a bond [5]. In FDOT load tests, the splice exceeded the expected pile capacities but produced a significant amount of cracking in the concrete around the bent rebar. Cracking under design loads is a concern regarding durability in marine environments. Otherwise the Kie-Lock splice is a strong and economical system and has thus been extensively used in Florida as well as other parts of the country in the past. The recent purchase of Sure-Lock by a Canadian company has rendered this tried product less desirable on transportation projects in the U.S. due to the Buy America Provisions [18].

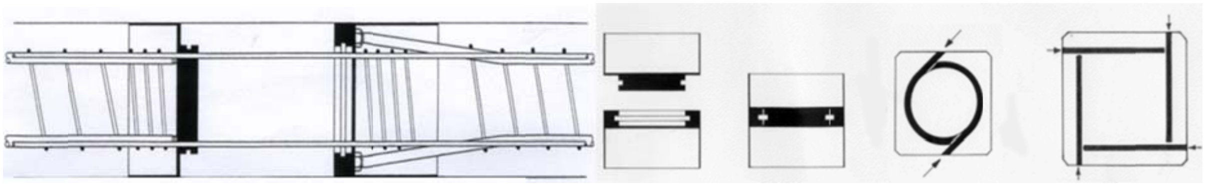


Figure 2.9. Sure-Lock mechanical splice reinforcing detail (left); key mechanisms (right).

- **ICP Piles –** *Welded splice with stressed ends.*

  These specially made spun-cast cylindrical piles incorporate weld-able end plate splices with stressed ends, which give them excellent flexural and tensile capacity [9]. The level of effective stress is subject to losses from elastic shortening but is uniform throughout the pile. Although these piles are currently produced in Canada and Malaysia, and thus not preferred on domestic transportation projects, FDOT has done both laboratory and field testing on these piles and their splices. Capacities, in both service and driving, were found to be satisfactory [15]. Primary concerns included durability issues, due to cracking under service loads, limited concrete cover, and brittle failure, possibly attributed to spot welding of the reinforcement [6]. Figure 2.10 shows the basic reinforcing scheme; Figure 2.1 shows this brand of pile being welded together.

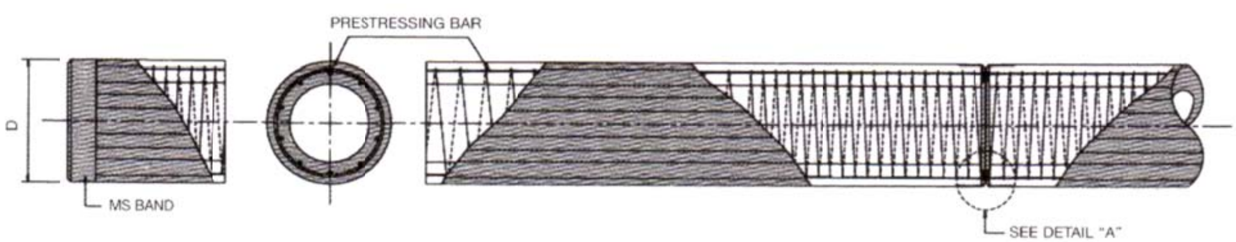


Figure 2.10. ICP Post tensioned segments with steel end plates welded together to provide splice connection (See also Figure 2.2).

- **GCP, Post-Tensioned Concrete Cylinder Piles (FDOT Index 20654) –** *Post-tensioned splice with stressed intermediate segment ends.*

  These are spun cast cylindrical piles which are cast as relatively short reinforced concrete pile segments (e.g.16ft), then spliced together by full-length post-tensioning (Figure 2.11). In this splice, the joining mechanism and anchorage mechanism are one in the same, falling into the categories of a post-tensioned splice with stressed ends for all intermediate splice locations. Therefore the main body of the pile and the intermediate splices are not subject to losses from elastic shortening. Spun cast, post-tensioned cylindrical piles are produced by Gulf Coast Pre-stress, Inc. and used extensively in the Gulf Coast area. These piles have been shown to be highly durable in marine environments [8 and 19]. The major downfall to this type of splicing system is that subsequent splices after the initial assembly require less effective means which fall back into the unstressed end anchorage as shown in Standard Index 20654 [4].

Figure 2.11. Cylinder pile segments aligned with post tension strands in preparation for stressing.

- **NU Chuck Splice –** *Bolted splice with stressed ends.*

This splicing method stems from a recent design from the University of Nebraska and is currently not yet in use (Tadros et al, 2011; U of N, 2012). This system is similar to the ICP system except with a bolted connection detail. Where ICP uses high strength bars through end plates with buttoned ends, this concept uses strands through end plates with “one-time use” chucks. Therefore it can be categorizes as stressed ends with bolted joints. The end plates provide recesses in the concrete to provide access to tighten four hex nuts on threaded rods to approximately 1000 ft-lbs (Figure 2.12). Like the ICP system the effective stress is subject to losses from elastic shortening.

Load tests performed by both University of Nebraska and FDOT, independently, showed that the splice could be used to satisfy capacity requirements. However, in both tests, brittle failure was observed, most likely attributed to the large recesses required at the edges of the pile cross section [20]. Also, in the tests performed by University of Nebraska, strand slippage occurred, likely resulting from the specially designed “one-time use” chucks used to seat the strands. Another limitation of this splice design is its restriction on strand configurations, as strands cannot be placed in the areas of the cross section which must be voided for tensioning of the rod. This was noted in the study done by FDOT resulting in an alternate design, in which the voids were placed at the corners rather than the edges. The latter configuration, however, was less successful [20].

Although not noted in either study, this splice is entirely dependent on the chuck anchorages. Despite numerous wedges provide some redundancy, if wedge slippage occurs there is no secondary load path to maintain connection between the end plates and the pile segments. Durability of the splice would therefore be similarly degraded by exposure of the wedges.

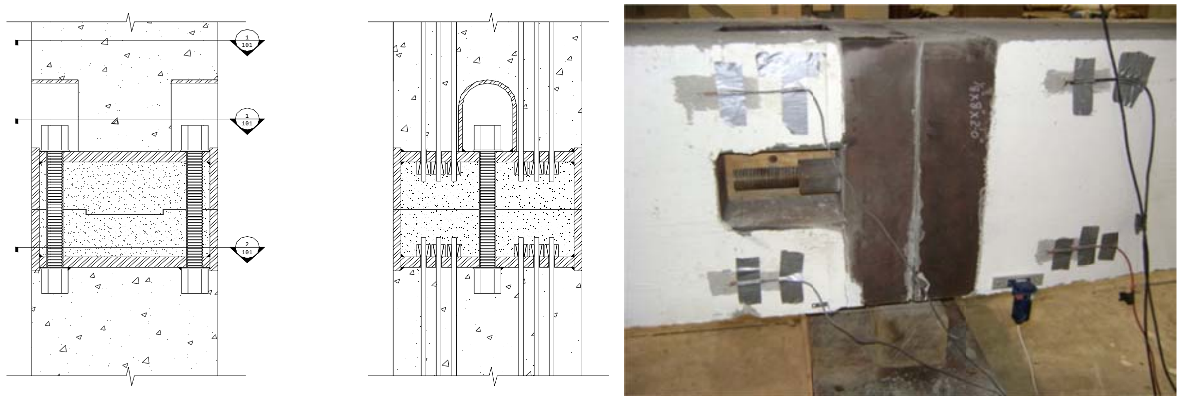


Figure 2.12. University of Nebraska splice concept also tested by FDOT.

- **Voided Pile Splice –** *Sleeve splice with unstressed ends.*

This splicing method was designed by the University of Florida in 2003 for splicing prestressed piles cast with voids. In this system a steel tube is inserted in the voided segment where the pile is to be spliced [21]. The exterior of the steel tube is bonded to the interior of the voided pile with grout. In both laboratory and field testing, capacity requirements were satisfied. The disadvantage noted by the study was that driving cannot resume until the grout has reached its design strength (about 20-24 hours after placement). If the piles are driven before the grout has achieved proper strength, the compression and tension waves will force the steel tube out of place, possibly away from the joint completely. Figure 2.13 shows a schematic of this splice detail.

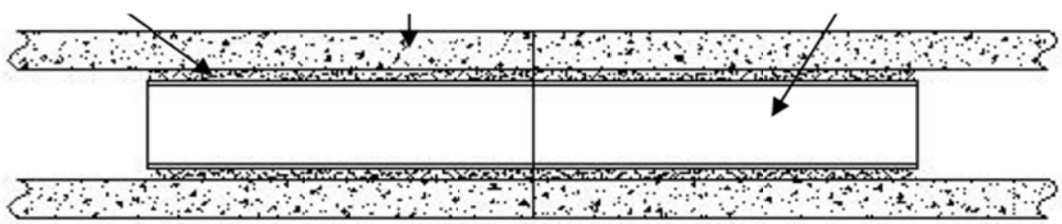


Figure 2.13. Internal collar-type splice detail for voided precast piles.

- **GYA Mechanical Splicer –** *Mechanical splice with unstressed end anchorage.*

This splice was developed and patented in Israel in the late 1990's by SADNAT GYA Kalman Ltd. It is a variation of the Kie-Lock splice which uses two steel caps, each with four grooved pins and four holes in an alternating pattern (Figure 2.14). The caps are anchored to the pile segments by 40 inches of rebar welded to the backside of the caps. When splicing, the pins and holes pieces are aligned and joined. Eight slots in the side of the steel caps allow forks to be inserted which lock into the grooves of the pins. This splice has performed well in load tests in Israel and is currently in use on several projects [11].

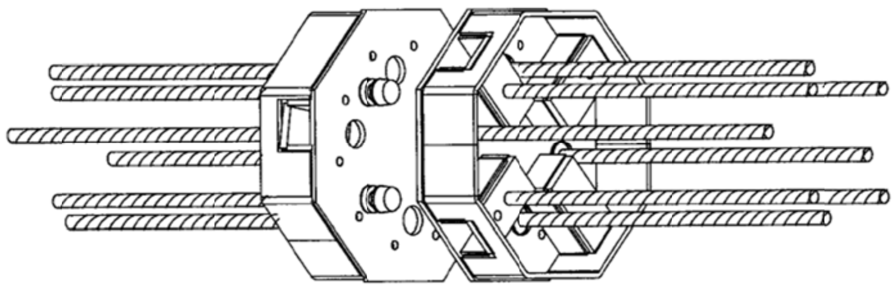


Figure 2.14. Mechanical splicer developed by GYA Kalman, Ltd used in Israel.

- **Macalloy Splice –** *Post-tensioned splice with stressed ends.*

  Although this splice has been used in England where it was originally patented (now expired), there are no published case studies of its use in the US. Other than the GCP Piles it is the only truly post-tensioned pile splice to be tested or used. This splice requires post-tensioning bars to be anchored at one end in the first pile segment by an embedded anchoring system and protruding from the end of the pile segment at the other end. The second pile segment is cast with ducts to allow for another set of post tensioning bars (Figure 2.15). The first pile segment is driven using a driving helmet to protect the protruding bars. The second set of bars is run through the ducts in the second pile segment and threaded into the first pile while the pile segment is being lowered down to the first. Once the post-tensioning bars are coupled, the second pile lowered completely down and the bars are tensioned by a hydraulic jack at the top of the pile, and then anchored [7]. A disadvantage to this method is that post tensioning is conducted at the top of the second pile which may require special considerations to provide safe access.

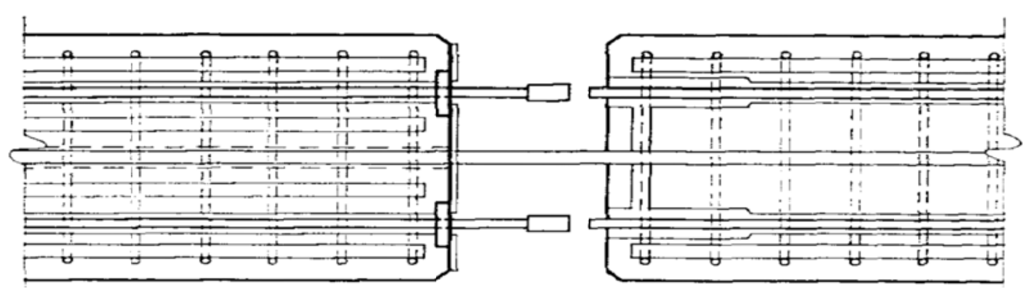


Figure 2.15. Macalloy splice details that uses full length post tensioning rods to provide splice continuity.

All of the above systems have proven to be viable alternatives for splicing precast concrete piles. However, they all have shortcomings which complicate their use for every application. The following table (Table 2.2) describes the important aspects of the splicing systems and provides a summary of how each system rates in these aspects.

Capacity Flexural and tensile strength required for both driving forces and design loads. It is the goal of most splicing systems to match the capacity of the un-spliced pile.

Failure Type Ductile failure is always preferred, but is especially desired in areas where ship impact is considered.

Durability This is of utmost concern in marine environments. Durability of a splice pertains to its own likelihood of corrosion as well as its tendency cause cracks in the pile which allow for corrosion of reinforcement and stressed strands. This is only of concern when the splice comes to rest above the mudline.

Installation This is a combination of the time and labor involved in achieving the splice in the field.

Production US made Yes / No. Only domestically produced products may be used on transportation projects in the U.S. without special exceptions.

Table 2.2. Summary of splicing systems.

	Capacity	*Failure Type*	*Durability*	*Installation*	*Production*
Epoxy Dowel	Poor	Ductile	Good	Moderate / Poor	Yes
Kie-Lock	Good	Ductile	Moderate	Good	No
ICP Piles	Good	Brittle	Moderate	Moderate	No
GCP Piles	Good	Ductile	Good	Poor	Yes
NU Chuck	Good	Brittle	Moderate	Un-tested / Moderate	Yes
UF Tube	Good	Ductile	Good	Poor	Yes
GYA Mechanical	Good	Ductile	Moderate	Good	No
Macalloy	Good	Ductile	Good	Moderate / Poor	No

2.3 FHWA Buy America

The FHWA statutory provisions for Buy America [18] directly apply to all steel or iron products used on roadway projects which include pile splicing materials. This provision can be found in Title 23 United States Code, Section 313; subparagraph (a) states:

> *Notwithstanding any other provision of law, the Secretary of Transportation shall not obligate any funds authorized to be appropriated to carry out the Surface Transportation Assistance Act of 1982 (96 Stat. 2097) or this title and administered by the Department of Transportation, unless steel, iron, and manufactured products used in such project are produced in the United States*

Despite a wide range of pile splicing technologies, there is still need to provide an efficient, cost-effective, and structurally robust concrete pile splice. Presently, only one such splice has been approved by FDOT and it is restricted due to foreign supplier rules.

Further considerations should also address corrosion resistance for applications in harsh marine environments.

Chapter 3: Design of Concept Splicing System and Component Testing

This chapter discusses the design, fabrication and testing of splicing components required for the concept splicing system.

3.1 Concept Pile Splice

At the onset of this study, four basic concepts were envisioned that dealt with embedded and/or external anchorages (Figure 3.1). These basically included: (1) intermediate post tensioning anchorages (along the pile sides) on both the upper and lower pile segment, (2) embedded anchorages cast into the lower pile with full length post tensioning of the upper segment, (3) a combination of first two where embedded anchorages are used on the lower pile and intermediate anchorages are used on the upper pile (or vice versa) and (4) unstressed strands embedded into a standard pile section (upper) with extensions that can be threaded into a lower pile with intermediate anchorages.

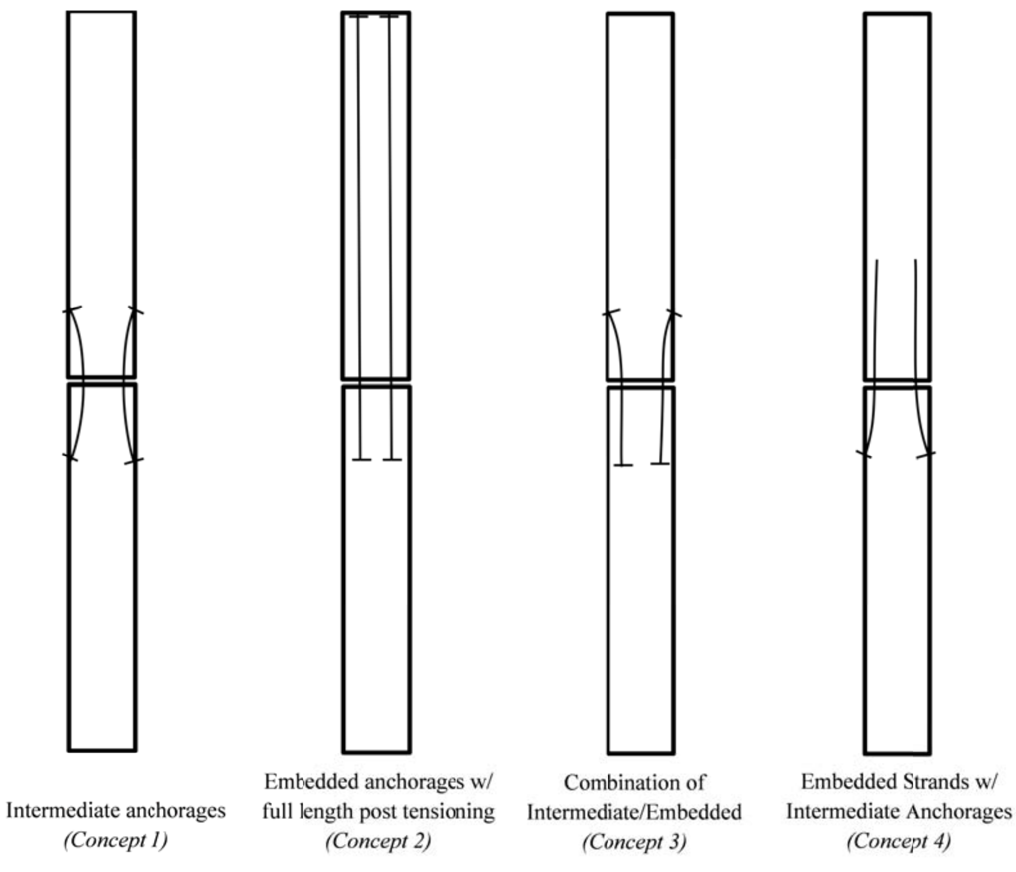


Figure 3.1. Basic concepts for post tensioned pile splices.

Concept 1 used all externally accessed intermediate anchorages. Concept 2 used standard prestressing chucks embedded with individual ducts in the lower segment and full length ducts throughout the upper segment. This is a variation of the Macalloy splice discussed earlier. Concept 3 combined embedded chucks and intermediate anchorages while Concept 4 simply embedded unstressed strands into the upper segment that extend out of the pile with sufficient length to thread through ducts in the lower segment to intermediate anchorages. Figure 3.2 shows renderings of intermediate anchorages used in a Concept 1 configuration (intermediate anchorages in both segments).

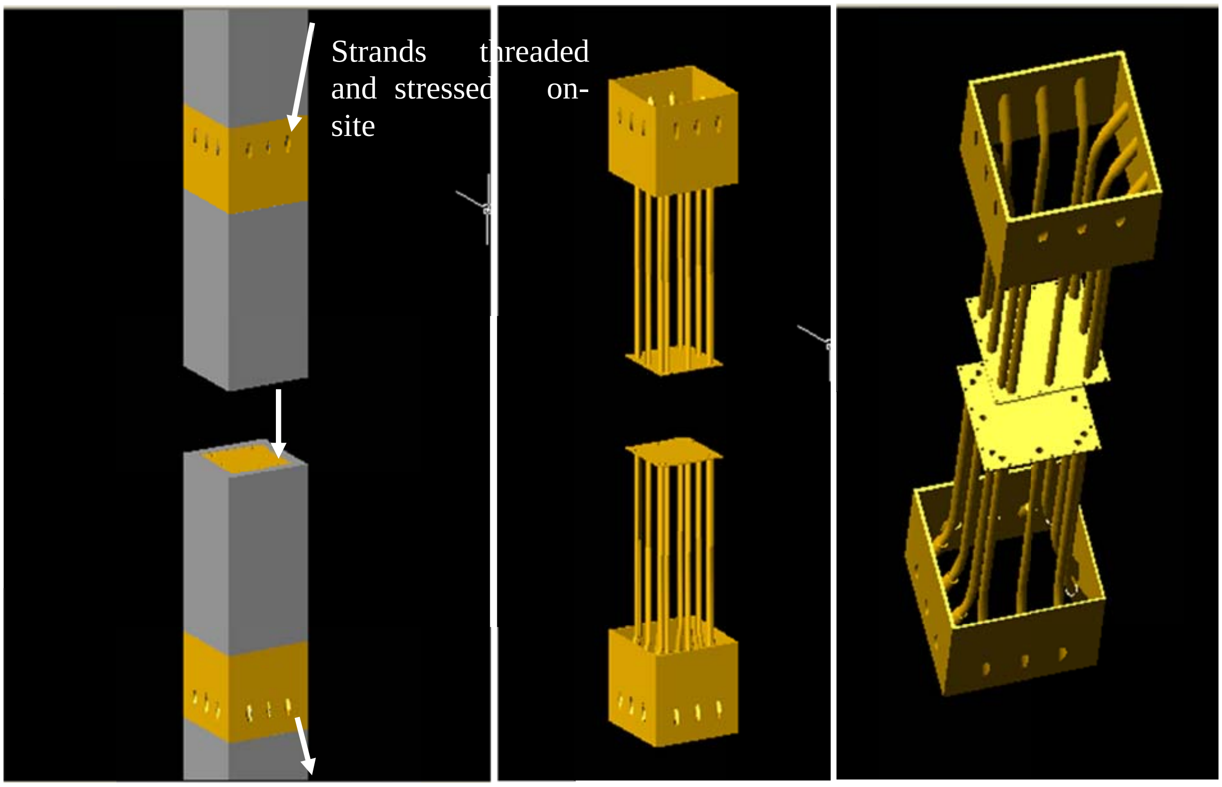


Figure 3.2. Concept 1 splice cast into pile (left), same view shown without concrete (center), down-looking view of curved ducts and end plates (right); spirals removed for clarity.

Is such a concept were to be possible, an important consideration in its design is the superposition of longitudinal stresses in the concrete. In prestressed piles, the transfer of force between strands and concrete is not present at the ends. Therefore the stress profile of the pile is not completely uniform but rather has regions at the ends where the prestress decreases to zero. This region can be anywhere up to 3 feet, depending on strand size, concrete strength, bond, etc. In traditionally spliced piles, this results in zero prestress at the splice joint and a relatively large region of reduced prestress around it. The inherent advantage of a post-tensioned splice is that precompression of the concrete can be maintained through this region. However, as post-tensioning strands extend away from the splice joint, the superposition of initial prestressing and post-tensioning forces can result in a state of excessive precompression in the pile, as shown in Figure 3.3. To combat this, the concept of staggered post-tensioning anchorages was explored for the prototype splice design. By staggering the intermediate anchorages of the post-tensioning strands, the additional compression in the pile can be gradually increased in a stepwise manner so that it offsets the gradual decrease of the initial prestress, like that shown in Figure 3.4. The number of staggered points can be as many as half the number of strands used. Therein, strands on opposite sides of the center point of the pile cross section can be paired at the same elevation so as not to put the pile in a state of permanent eccentric loading.

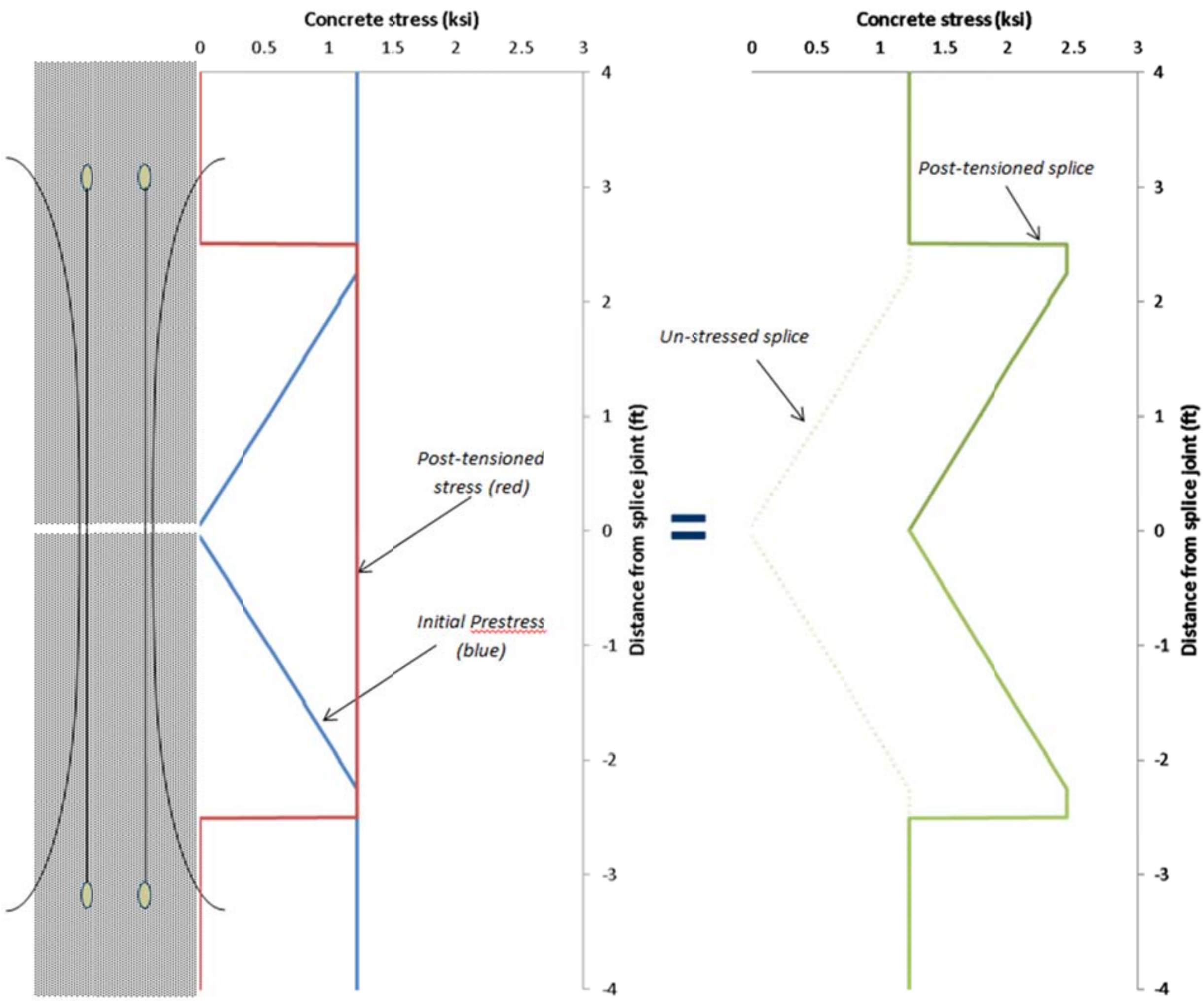


Figure 3.3. Superposition of initial prestressing and stresses from a post-tensioned splice.

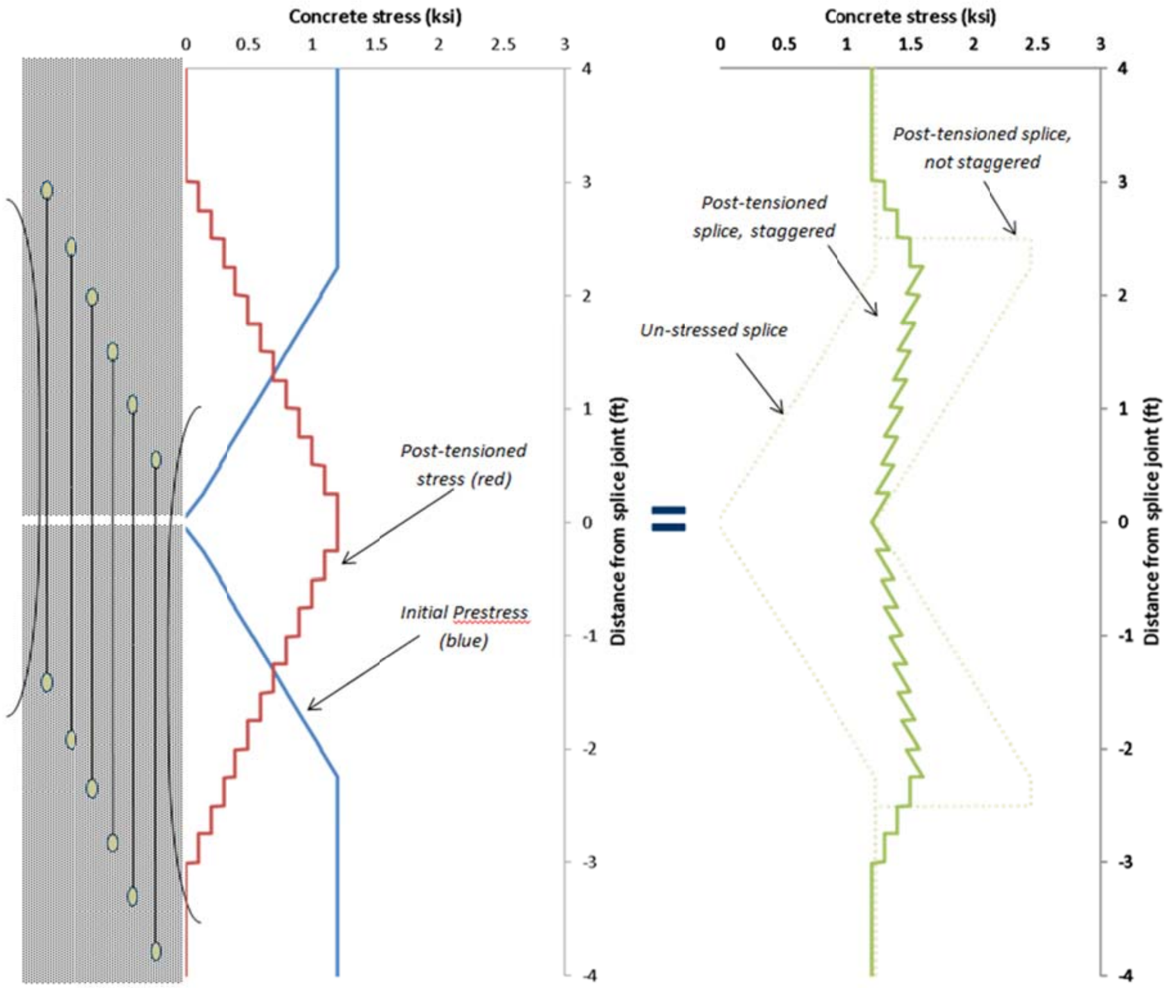


Figure 3.4. Superposition of initial prestressing and a post-tensioned splice with staggered anchorages (24 strands).

Additional stress considerations include lateral tension and compression in the concrete at the location where splice strands deviate outwards and bearing stresses at the anchorage locations (Figure 3.5). Figure 3.6 illustrates how these problems can be mitigated. Lateral stresses at the deviations can be reduced by providing transverse reinforcement steel. This reinforcement can be in the form of steel hoops that encircle the ducts and act as confinement or bars that link between opposing pairs of ducts. Bearing stresses can be reduced by using intermediate anchorages that have adequate surface area and stiffness to distribute the load. Many products are commercially available, like the one shown in Figure 3.6 (from DSI), but it is possible that an anchorage could be custom made to better serve this application. This was the approach taken and discussed later.

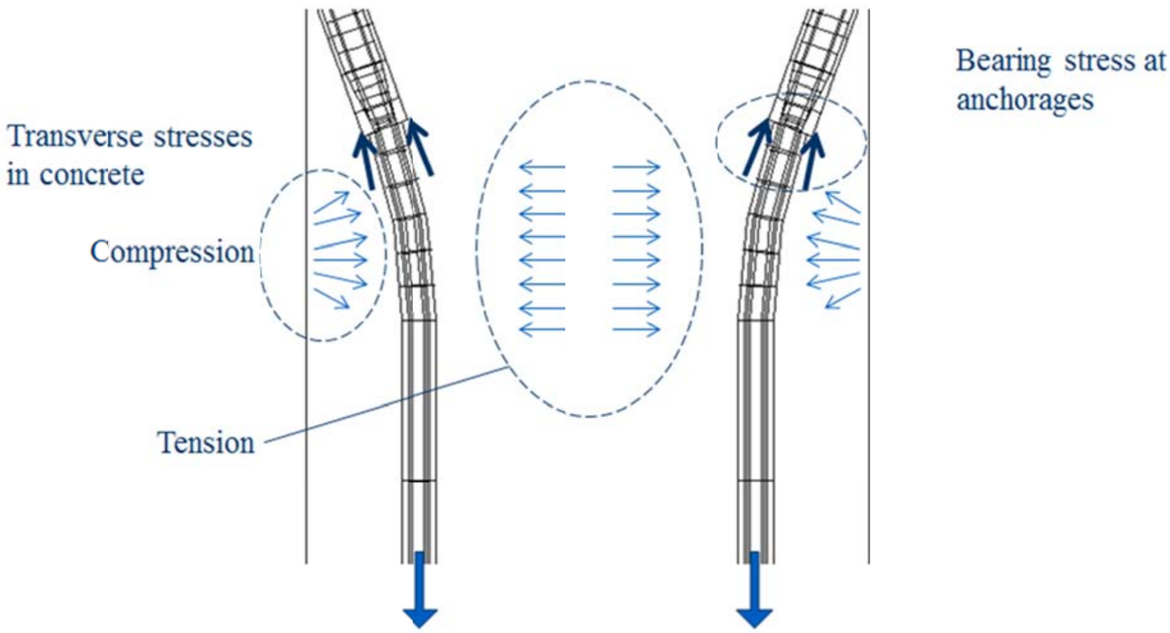


Figure 3.5. Local stresses at deviations and anchorages.

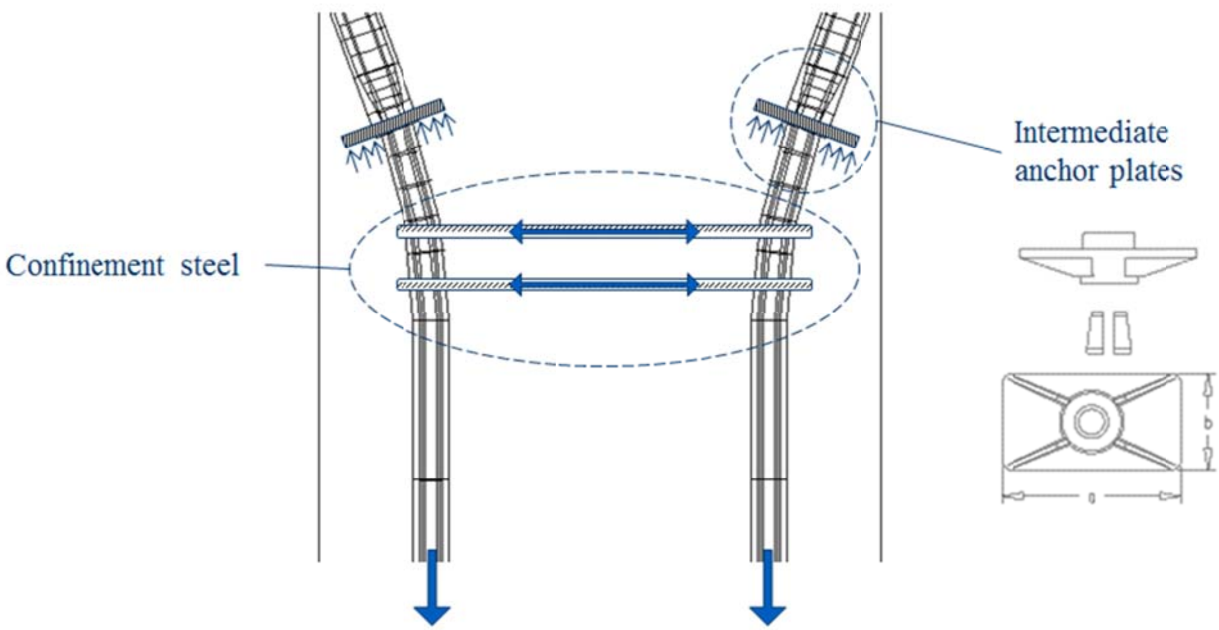


Figure 3.6. Transverse reinforcement and anchor plates used to mitigate stresses.

In an effort to move forward from concepts to prototyping, both numerical and physical models were developed.

3.2 Numerical Modeling

Moving from the concept designs to a prototype design requires an up-front analysis of the stresses that that will be expected. The effects of curved tendons, interior anchorages, additional confinement, and the overlapping of prestressed and post-tensioned sections must all be investigated. This requires rigorous numerical modeling which was undertaken using COMSOL, a finite element method program. For more typical prestressing applications however, analytical methods are readily available and are used herein to help ensure the validity of computer model results.

Table 3.1 shows the specifications, as well as derived dimensions and material properties, for a FDOT standard 24" square prestressed concrete pile with a 24 strand configuration. Table 3.2 provides calculated values based on AASHTO and ACI estimation methods for transfer stresses, prestress losses, and transfer length. Figures 3.7 - 3.11 show model results for the same pile immediately after transfer. The results are comparable to analytical methods.

Table 3.1. 24" square prestressed concrete pile specifications and derived values

B	24	in	Square pile size
N_s	24		Number of strands
d_{ps}	0.5	in	Diameter of strands
A_{ps}	0.153	in^2	Area of single strand
F_{pJ1}	31	k	Jacking force
f_{pJ1}	202.6	ksi	Jacking stress
f_{pu}	270	ksi	Ultimate strength, prestressing steel
f_{py}	243	ksi	Yield strength, prestressing steel
f'_{ci}	6000	psi	Compressive strength, concrete at transfer
E_{ps}	27000	ksi	Elastic modulus, prestressing steel
E_{ci}	4415	ksi	Elastic modulus, concrete at transfer
t_{tr}	7	days	Time between jacking and transfer

Table 3.2. Estimation of losses, resulting stresses, and transfer length

f_{pJ1}	202.6	ksi	Jacking stress
Df_{pR1}	3.199	ksi	Loss due to strand relaxation before transfer (AASHTO)
Df_{pES}	7.109	ksi	Loss due to elastic shortening immediately after transfer (AASHTO)
f_{pi}	192.3	ksi	Initial stress in strands immediately after transfer
f_{ci}	1.226	ksi	Initial stress in concrete immediately after transfer
Df_{pTD}	30.0	ksi	Lump-sum estimate of time-dependent losses (AASHTO)
f_{pe}	162.3	ksi	Effective stress in strands after all losses
f_{ce}	1.035	ksi	Effective stress in concrete after all losses
h	0.84		Ratio of effective stress to initial stress
l_t	27.1	in	Transfer length (Naaman & ACI)

In COMSOL, a 12ft long pile was modeled with all parameters as they are in Table 3.1. Strands were located according FDOT design standards – one strand in each corner and the remaining equally spaced with a cover of 3in and a tie diameter of 0.208in. The loss due to strand relaxation prior to transfer was input as it is in Table 3.2, but the loss due to elastic shortening was left to be computed by the model. In doing this, the validity of the model could be evaluated by a comparison of the stresses in the strands and concrete immediately after transfer (f_{pi} and f_{ci}) as well as the transfer length in the strands (l_t). In order to get realistic results, an additional property characterizing the bond relationship between concrete and strands had to be estimated. Work done by Balazs (1992) defines a non-linear bond stress-slip relationship for ½ inch strands and 5.8ksi concrete. For modeling purposes, a simplified linear relationship was derived from Balazs' work, resulting in a bond-slip stiffness of 30ksi/in. Figures 3.7 - 3.11 show the setup of the model and results for the state of the pile immediately after transfer.

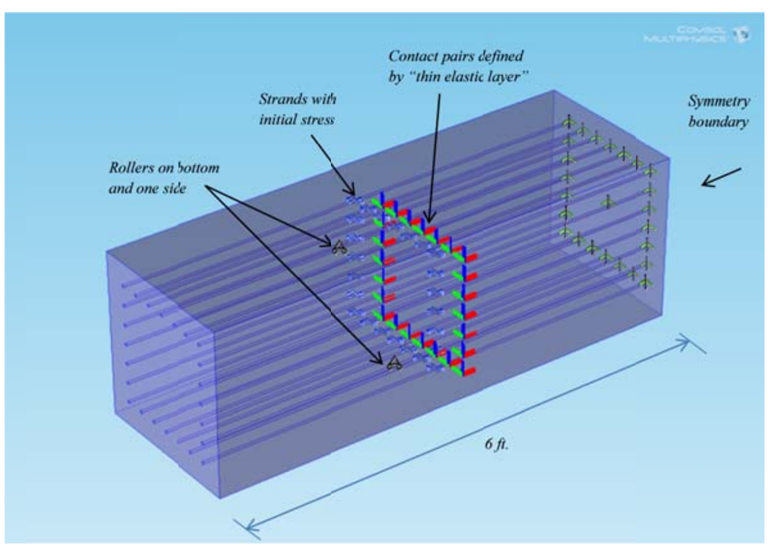


Figure 3.7. Model geometry and boundary conditions

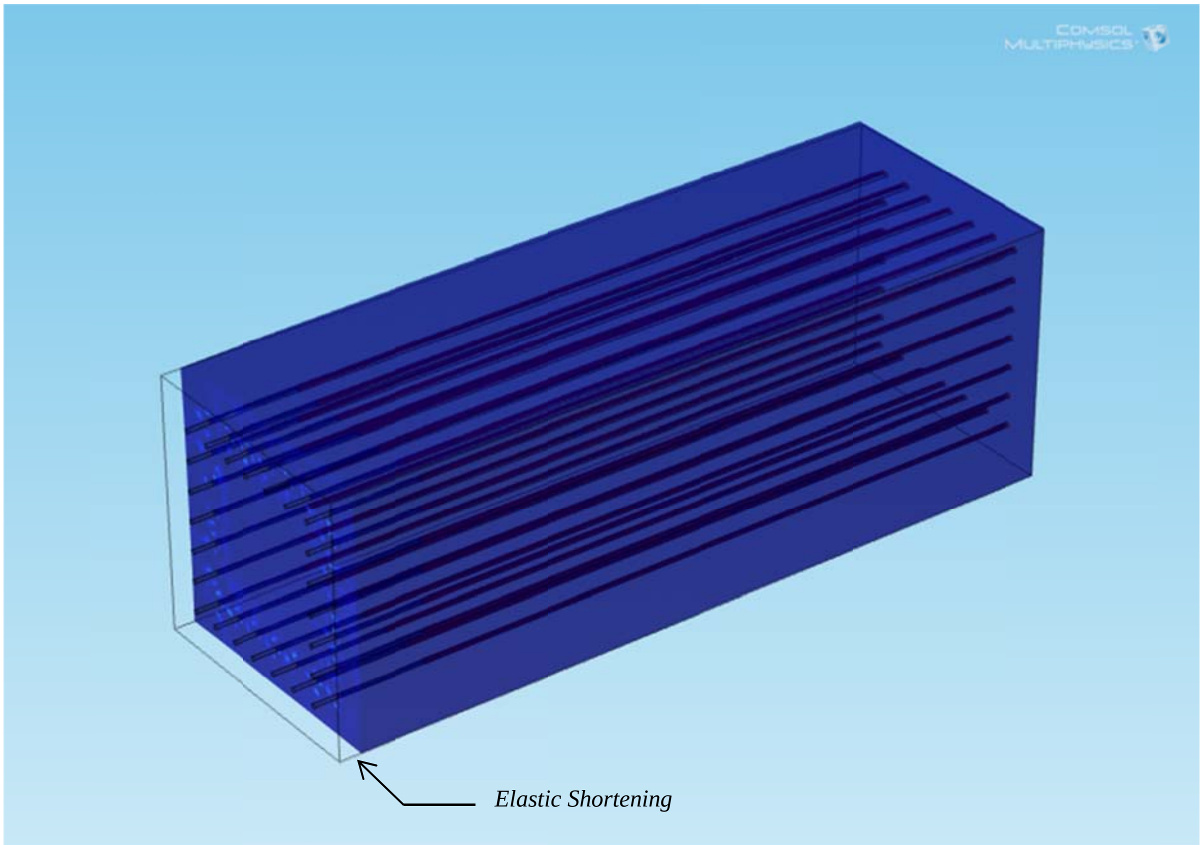


Figure 3.8. Model solution, 3D rendering

Figure 3.9. Model solution, longitudinal stress variation along mid-plane of pile (blue = compression, red = tension)

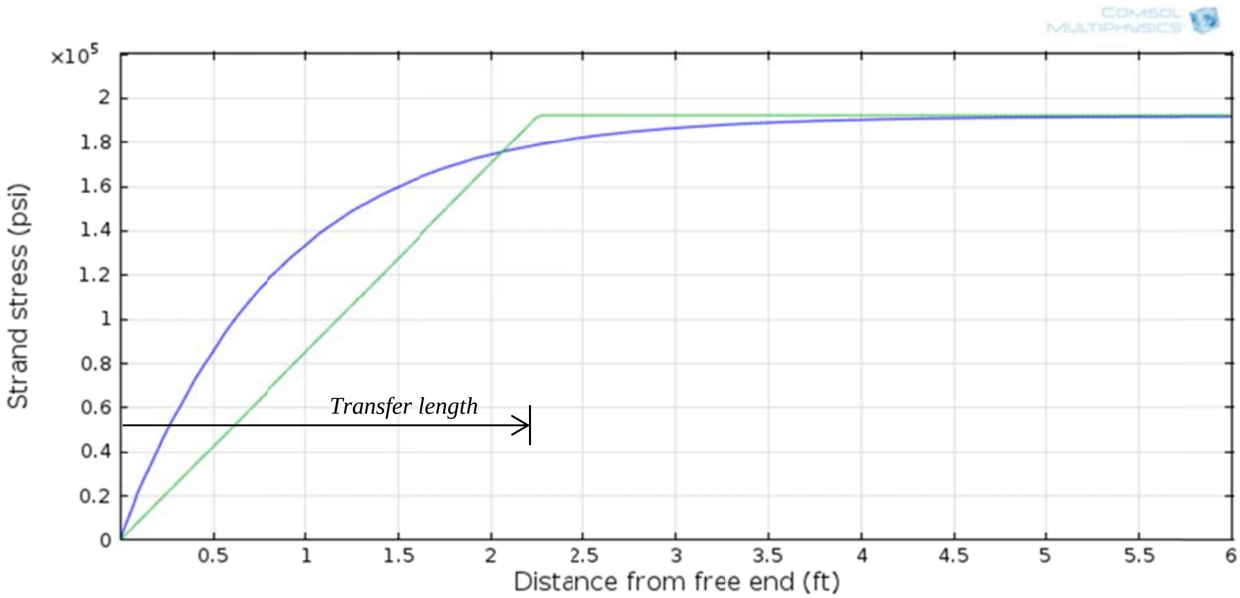


Figure 3.10. Model solution (blue) and analytical approximation (green) of stress variation along a single strand

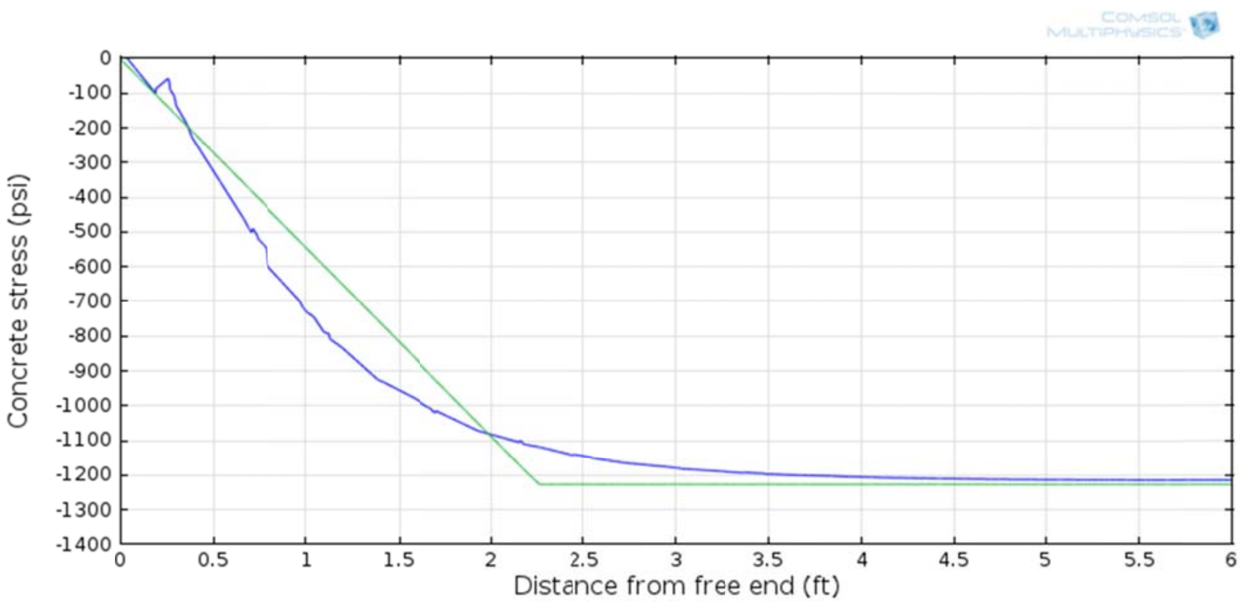


Figure 3.11. Model solution (blue) and analytical approximation (green) of stress variation along centerline of concrete

To gain a better understanding of the distribution of these stresses in the pile, finite element modeling was used to analyze different configurations. Table 3.3 shows the input parameters used for two 14in pile models that were used to compare the effects of confinement steel and anchor plate size. The 14in size was selected as a convenient dimension for the first round of prototypes. Model 1 showed the effects when no specialized deviator reinforcement was used and Model 2 included deviator bars with a larger area anchorage.

Table 3.3. Model parameters for 14in prototype splice piles

	Model 1	**Model 2**
Pile size	14 in.	14 in.
Prestressing	(8) ½" strands @ 31 kips	(8) ½" strands @ 31 kips
Post-tensioning	(8) ½" strands @ 31 kips	(8) ½" strands @ 31 kips
Anchorage location	2 ft.	2 ft.
Deviation angle, radius	20^o, 5"	20^o, 5"
Anchor plate	1.8"x1.8"x0.5"	2.5"x3"x0.75"
Confinement steel	None	(8) #3 ties

Figures 3.12 – 3.14 show comparisons of results from these models. Both the longitudinal stress profiles (Figure 3.12) and the contact pressure on the surface of the anchors (Figure 3.14) show that the anchor plate dimensions used in Model 1 cause bearing stresses in the concrete in excess of 6000 psi. The anchor plate dimensions used in Model 2 however, bring these stresses to a more acceptable level. While the modeled anchor plate is 0.75", a thinner plate with stiffeners, like that shown in Figure 3.6, could provide equivalent results. The effects of confinement steel are seen in the transverse stress profiles (Figure 3.13), where the tension in the concrete at the center of the pile due to the deviated strands is significantly reduced when reinforcement is present. However the model does not show a reduction in the compressive stresses on the concave side of the deviation. This is due to a simplification in the geometry of the model wherein a gap exists between the steel ties and the ducts. In practice, the transverse steel would be in contact with, or possibly fastened to, the ducts.

Two primary findings were taken from these numerical models: (1) increased compression stress in front of the anchorages would need to be addressed in the form of increased confinement steel and, (2) deviator stresses would require tension struts to eliminate the possibility of internal concrete cracking.

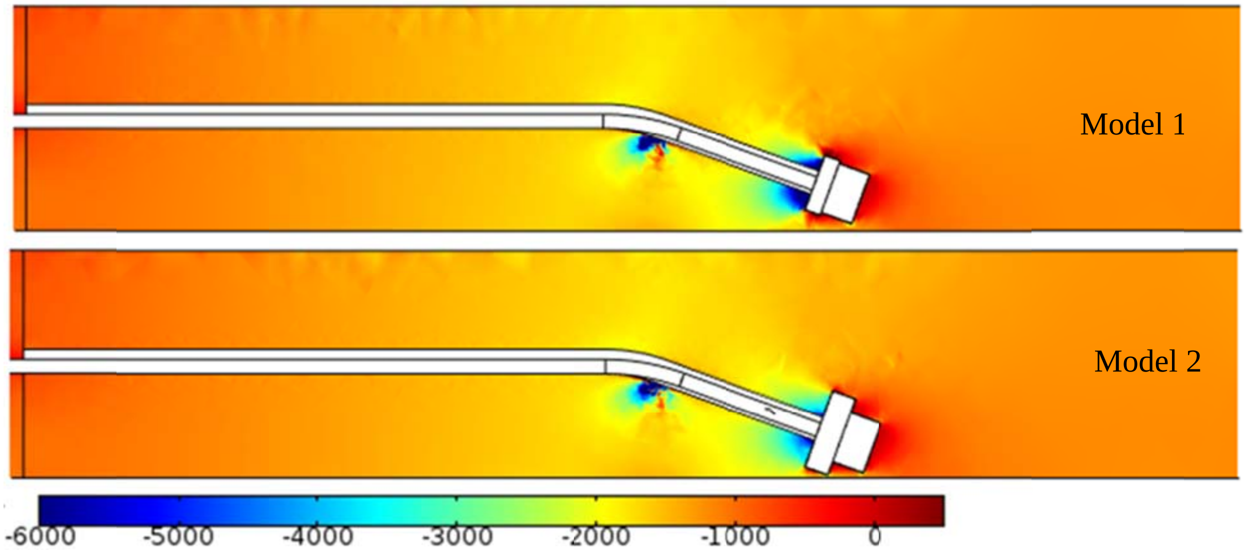


Figure 3.12. Longitudinal stresses in concrete (psi).

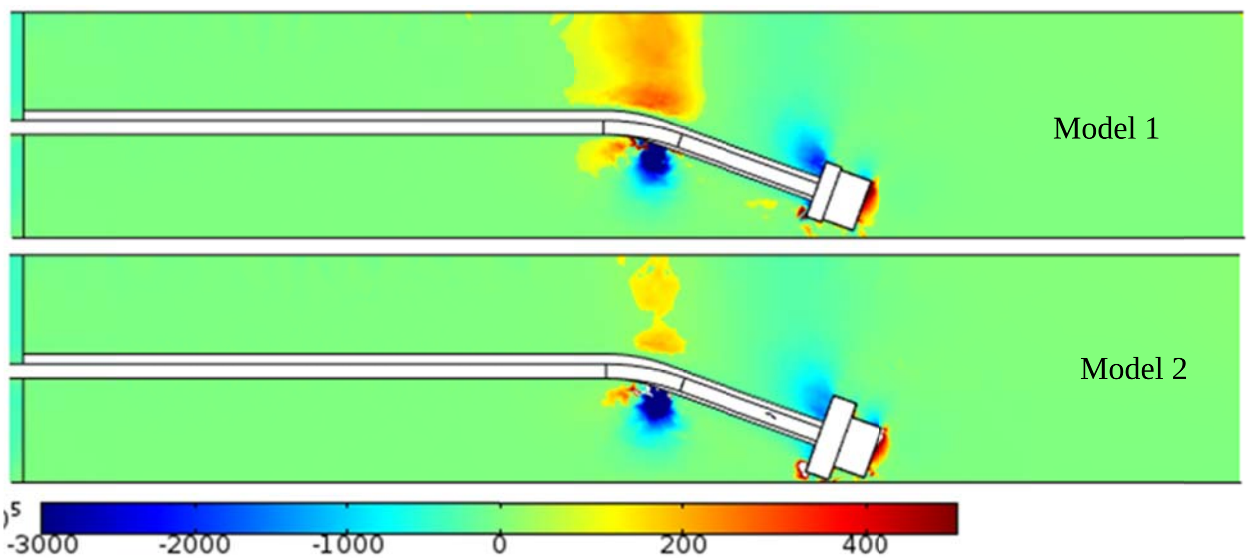


Figure 3.13. Transverse stresses in concrete (psi).

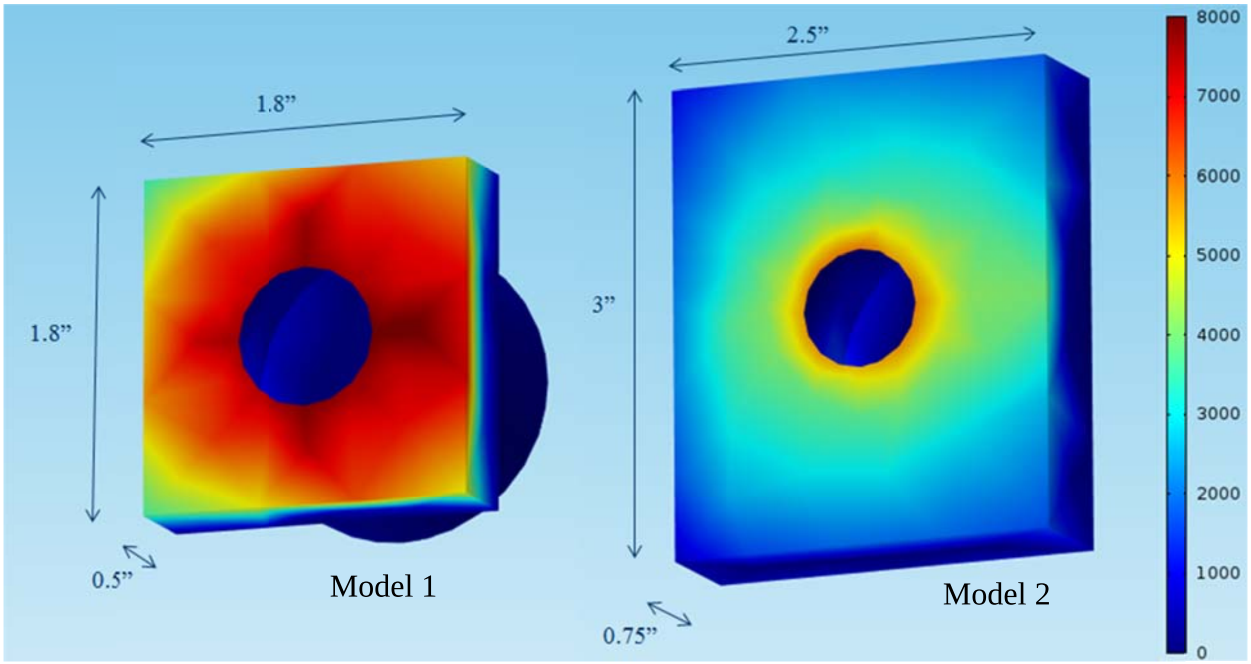


Figure 3.14. Contact pressure on anchor plates (in psi) for two different anchor plate dimensions.

3.3 Physical Modeling

Physical models were developed to serve as practicality checks which could quickly reveal oversights not easily detected in CAD based design drawings (Figure 3.15). These were based on 14in square pile prototypes. Original drawings placed the strands as far from the neutral axis as possible at the splice location (pile segment ends) that would then deviate slightly toward the middle of the pile as the duct got farther from the ends. The intent was to preserve as much geometric advantage (higher moment of inertia) at the splice region where the only reinforcing steel would be the post tensioning strands. Physical models showed this to be impractical.

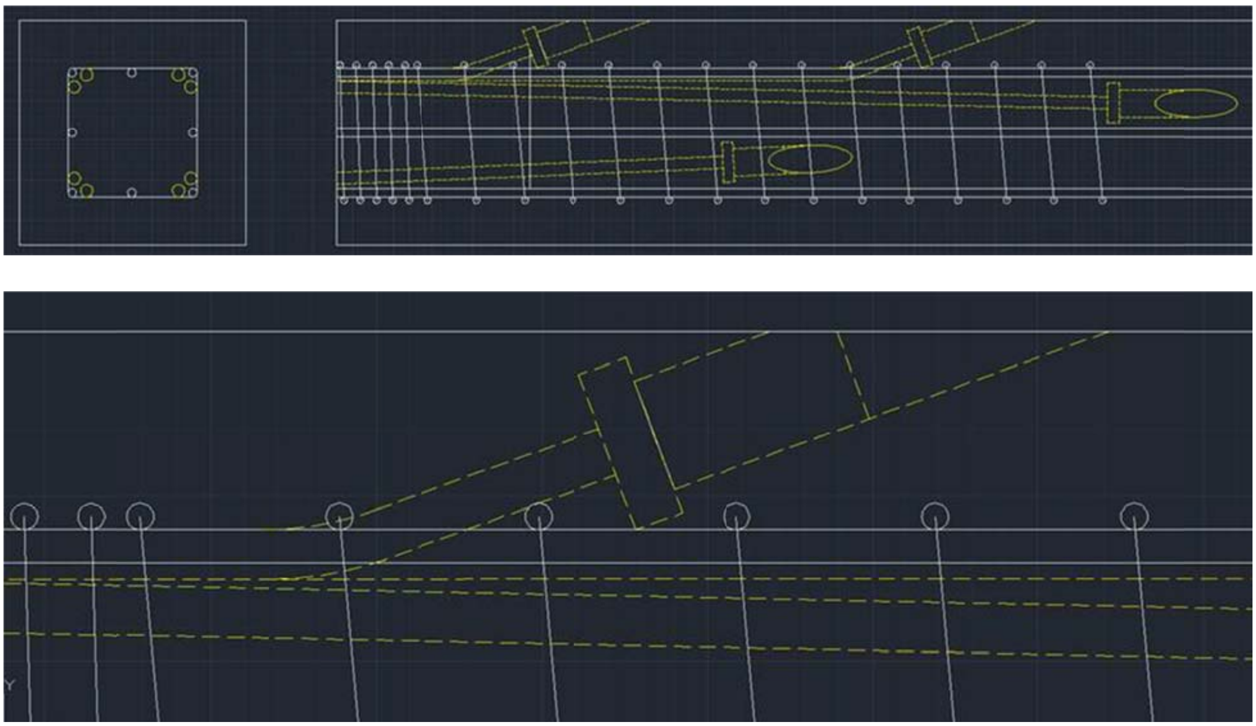


Figure 3.15. Anchorage layout shows minimal interference with stirrups.

Using a refined version of Figure 3.15 where the ducts did not deviate from the corners, a header plate assembly was fabricated where the surfaces of the outer faces were perfectly parallel. This would provide for a near perfect mating of the two spliced pile segments. A mock casting bed layout was then setup in the laboratory to show how stirrup and duct interferences could be minimized. Where possible, off the shelf (OTS) components were used or modified for use.

3.3.1 Component Fabrication / Modification

Chuck Assembly: Standard ½in spring loaded, quarter-turn back chucks were selected for use in this study (Figure 3.16) with minor modifications.

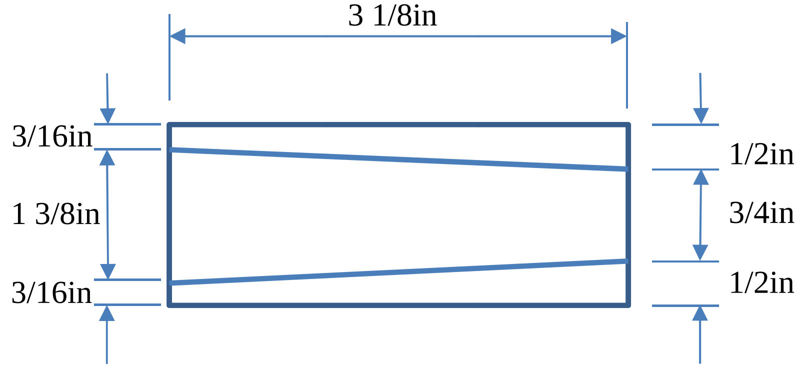


Figure 3.16. Chuck dimensions (as-received).

The OTS chucks were machined to fit the outer dimension of 3/4in metal electrical conduit (EMT) which had an outer diameter of 15/16in as shown in Figure 3.17.

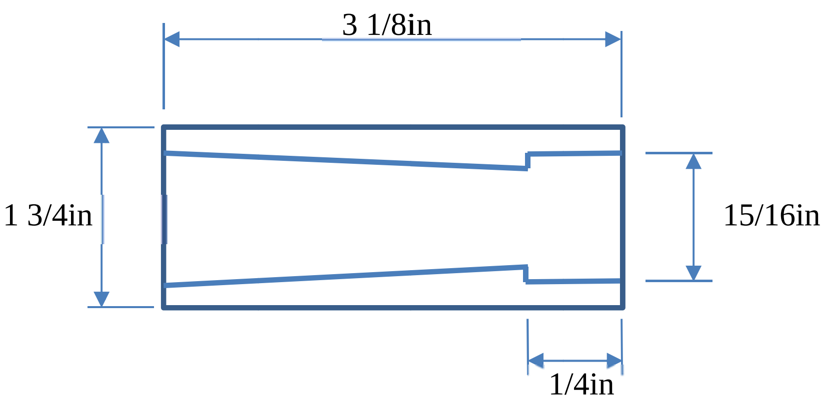


Figure 3.17. Modification of chuck in lathe (left) finished dimensions (right).

The modified chuck was then welded to a 3” x 2 ½” x ¾” steel bearing plate with a matching centered 15/16in hole. The finished chuck assembly is shown in Figure 3.18.

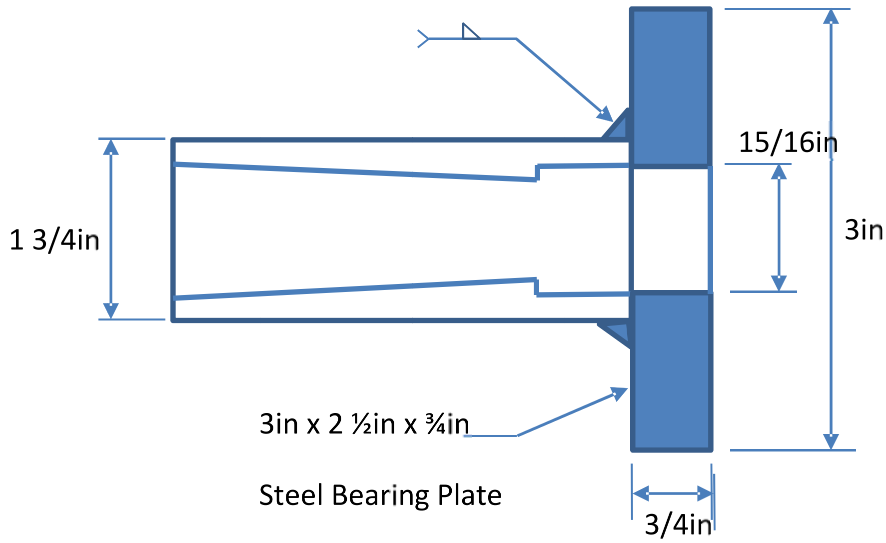


Figure 3.18. Finished chuck assembly.

Splice / Casting Header: A special intermediate header was fabricated to accommodate the additional ductwork associated with embedded chucks as well as the standard strand locations. Figures 3.19 through 3.21 show the fabrication of the splice header.

Figure 3.19. Drilling locations for strands and ductwork into the splicing header.

Figure 3.20. Milling header plates to provide smooth, parallel contact surfaces.

Figure 3.21. Finished splice header assembly.

Fabrication of Alignment Plates: To ensure uniform and proper position of the ducts throughout the length of the splice, special alignment plates were made which fixed the location of the ducts relative to the strands. They were fabricated from 20ga sheet metal and cut to allow adequate concrete flow and impose minimal conflict with shear reinforcement stirrups. Figure 3.22 shows a single alignment plate.

Figure 3.22. Alignment plate.

To test the overall component layout and clearances, an initial assembly was performed in the laboratory in a mock casting bed. This provided the luxury of a controlled environment in which the assembly pattern could be optimized with all splice components, strands, and stirrups included. This process is shown in Figures 3.23 and 3.24.

Figure 3.23. Mock casting bed assembly – placement of components.

Figure 3.24. Mock assembly – components in place (left) and stirrups being tied (right & bottom).

3.4 Concept Redirection

Physical modeling revealed a conceptual / design flaw that could not be eliminated and redirected the entire concept. Two conflicting needs could not be met: (1) the smallest radius of curvature that a single strand can easily negotiate and be pushed by hand through the ducts is approximately 4 to 5ft and (2) the largest radius of the duct so that it can pass through the stirrups without drastically affecting the stirrup spacing is closer to 4 to 5in. Even if the strand could be pushed through the tight turn, the tight radius in the duct would make losses in the strand too high to achieve a usable level of prestress.

This led to an adaptation incorporating dually embedded anchorages where the lower pile segment is exactly as shown in Concept 2 or 3 (Figure 3.1), but the upper pile segment is equipped with similarly embedded anchorages within close proximity to the spliced end. The strands, however, extend through the upper pile segment anchorages to the top of pile where the strands can be stressed, but the locked in stress after the jack is released resides only in the regions of the pile adjacent the splice. All anchorages would still be the same commonly available prestressing chucks with spring loaded end caps and wedges adapted with a bearing plate. Figure 3.25 shows the two phases of the splice during stressing: initial forces applied via jacking at the upper pile end, and the final force applied only in the splicing region between the embedded chucks.

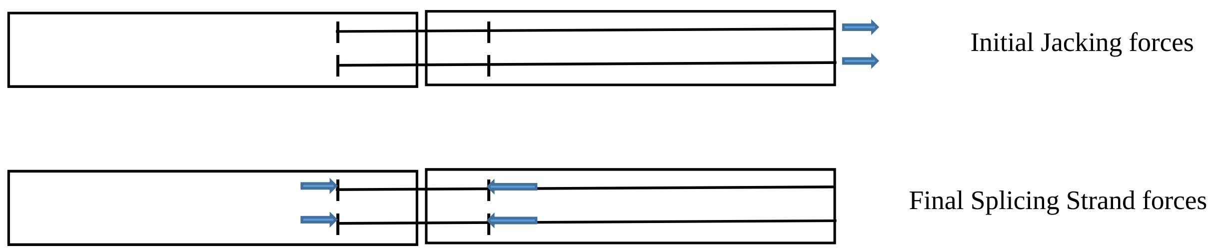


Figure 3.25. Dually embedded anchorage concept.

3.5 Component Testing

Using the input from numerical and physical modeling as well as the redirected concept that stemmed from these efforts, a series of component tests were performed as a final step before committing to specimen casting.

As with all short strand post tensioning applications, wedge setting losses can be significant where the overall strand elongation is similar in magnitude to the wedge setting movement. These losses can be offset through the use of hydraulically actuated wedge setting systems (power wedge setters). For a power wedge set system to be effective it must be able to push directly on the wedges and for the concept shown in Figure 3.18, this distance may be very long (e.g. 50-100ft). As a result, a series of chuck improvement experiments were conducted to first determine the magnitude of loss associated with normal spring loaded wedges and then to what extent these losses could be reduced.

The strand length selected for splicing was 50in based on transfer length estimates and using the staggered anchorage concept (gradually transition and superimpose post tensioning stresses with the linearly increasing prestress). Therefore, all splice strands were set to be the same length and were staggered whereby one strand terminated 10in from the end of one pile segment and extended 40in into the end of the other pile segment. For the eight strand prototype, strands were in pairs starting a 10, 20, 30, and 40in from the splice end of one pile segment and terminated at 40, 30, 20, and 10in, respectively, from the splice end of the other pile segment. By placing them in opposing pairs, no unusual torsional or bending forces would result (Figure 3.26).

Figure 3.26. Splice header with staggered ducts and embedded chuck assemblies; all ducts are the same length.

Using a 50in spacing between chucks, a series of wedge setting tests were conducted that measured the losses from varying wedge setting methodologies. This included normal chucks with spring-loaded back caps, a power wedge setting jack that directly pressed on the wedges, and two variations of shims placed between spring and wedges in standard spring-loaded back caps.

When jacking the 50in long, 1/2in strands (270ksi) to 80% of ultimate (33 kips) and releasing where only the spring loaded caps pushed the wedges into the locked position, the load fell to 20kips. This corresponded to 0.15in of wedge travel before seating (PL/AE). For a 14in square pile with 8 strands, that equates to 816 psi concrete prestress (target design level is 1000psi minimum). Using a power wedge setting jack the final load after release was 25.5 kips (or 1041 psi concrete prestress). While this level of prestress is reasonable, it is not a practical solution for embedded intermediate anchorages with the wedges so far from the jacking surface. Therefore, shims were inserted between the spring and wedges to reduce the 0.15in of wedge travel to a more acceptable level of movement and loss. Figure 3.27 shows the effect of one and two shim washers relative to the other two methods. The net result from using two 0.060 inch shim washers was a final load after release of 26.5 kips (1080 ksi). This corresponds to a 0.11in wedge set movement.

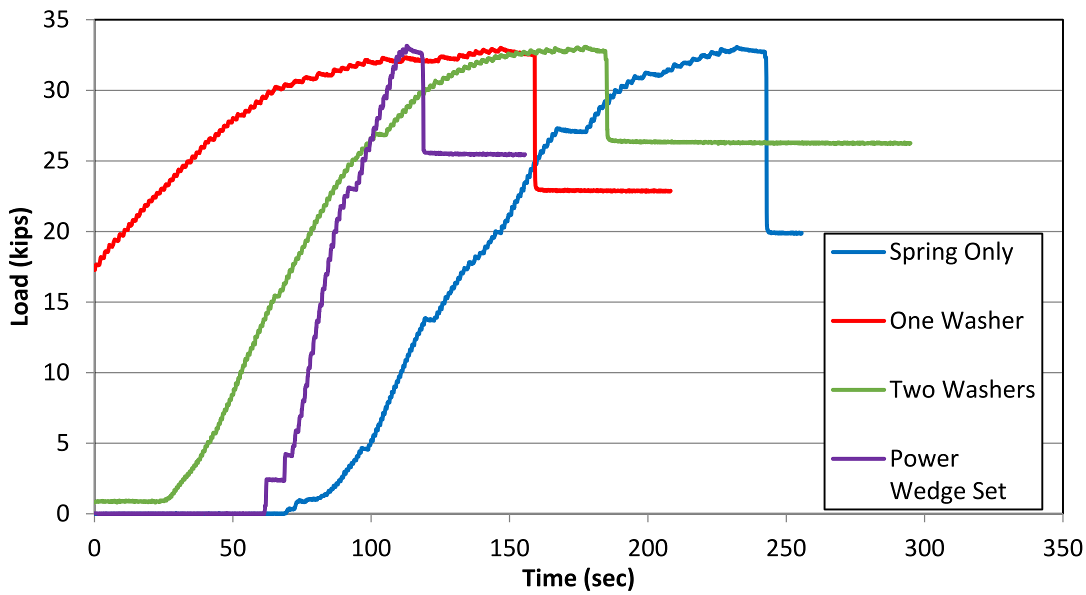


Figure 3.27. Effect of varied wedge setting techniques on final posttensioning force.

While, the increased final load was directly proportional to the thickness of the shim washers, an upper limit also existed where too many shims restricted the ability of the wedges to open enough for the strand to pass through. Likewise, there is always some movement required to full seat the wedges which is accompanied by depressions of the wedge teeth cutting into / imprinting the strands. Therefore, for the prototype splice pile system, the lower pile was equipped with spring only chucks while the upper pile segment was equipped with two, 0.060in shim washers in each chuck. Recall, the lower pile chucks will seat immediately during jacking; only the upper pile will experience wedge setting losses.

Chapter 4: Prototype Splice Pile Tests (14in)

This chapter discusses the steps taken to cast, splice and load test 14in square prestressed piles.

4.1 Laboratory Scale Prestressing Bed

To fully monitor the prestressing process, a prestressing bed / casting facility was designed and constructed on campus that could cast 14in square pile specimens up to 25ft long. The bed was designed as a self-stressing assembly that incorporated previously cast 14in concrete piles as the three compression struts obtained from a local casting yard (Figure 4.1). The design therefore provided the opportunity to cast two piles at a time so that with each spliced pile cast (two 10 ft. segments), an unspliced control pile (one 20 ft. segment) could also be cast and tested. Figure 4.2 shows the components for the casting bed roughly laid out. As square piles are typically constructed with a slight side taper to facilitate removal from the bed, the commercially produced piles that were purchased for the laboratory casting were oriented to make use of the tapered sides. Therefore, by installing the piles flipped over from their orientation in the original casting bed (up-side-down), the tapered sides produced the same taper in the laboratory casting bed so that the test piles could be easily removed from the laboratory bed as well.

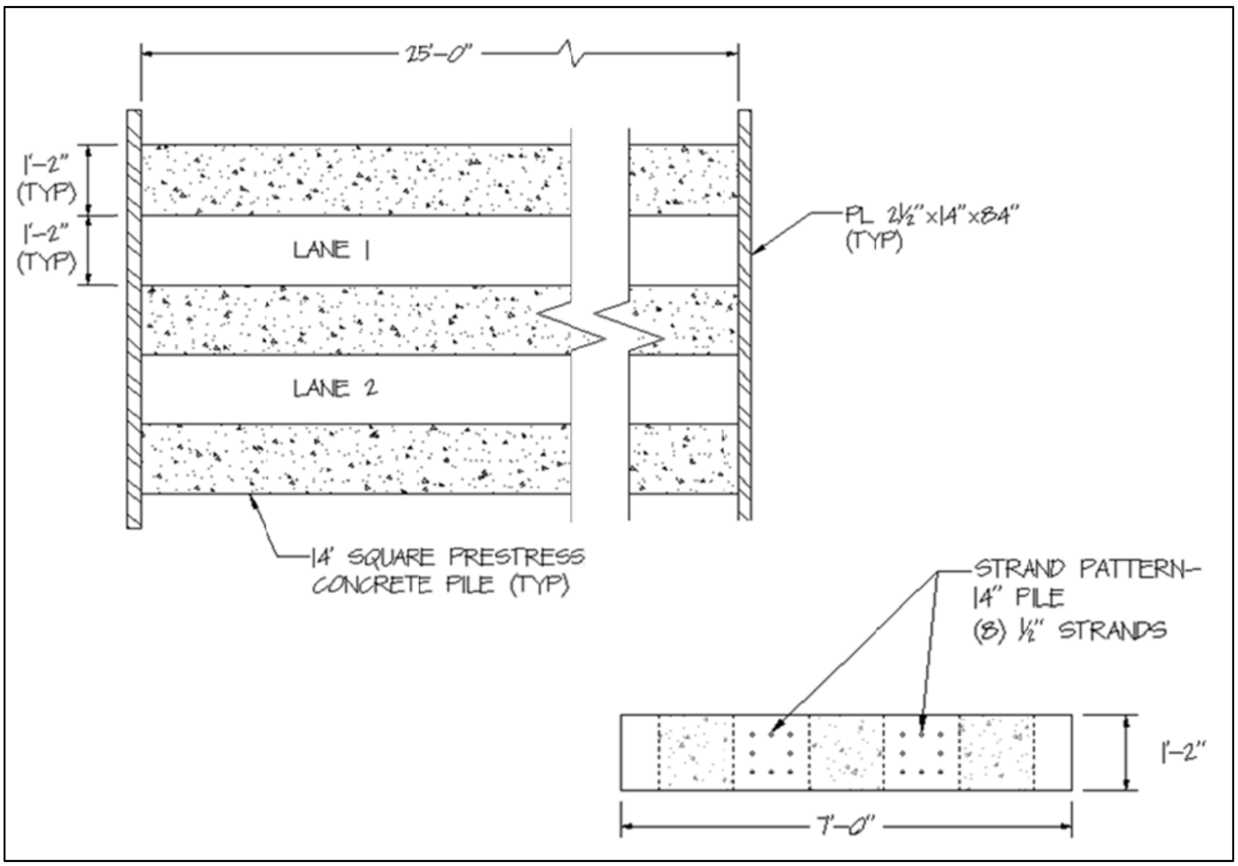


Figure 4.1. Drawing of casting bed for test piles.

Figure 4.2. Casting bed components.

4.1.1 Casting Bed Modeling

Design of a self-stressing casting bed is not a trivial matter in that the stressing forces must be safely withstood while also imposing no appreciable deformations. To this end, finite element analyses were used to analyze the performance of the bed under jacking loads. Figures 4.3 and 4.4 show that stresses in the header plate and contact pressure on the piles were within acceptable limits. However, bending in the header plate caused higher stresses on the inside of the outer compression struts and an associated eccentric load on those piles. Figure 4.5 shows lateral displacement of the outer piles was estimated be just under 0.25in. To reduce or completely eliminate this lateral distortion, the distribution of contact pressure on these piles was tailored to reduce the level of eccentric loading. This was achieved by inserting thin bearing plates between the header plates and the piles and sizing them so that the concentration of contact pressure moved away from the edge of the piles. The exact location of these plates was determined using additional finite element modeling.

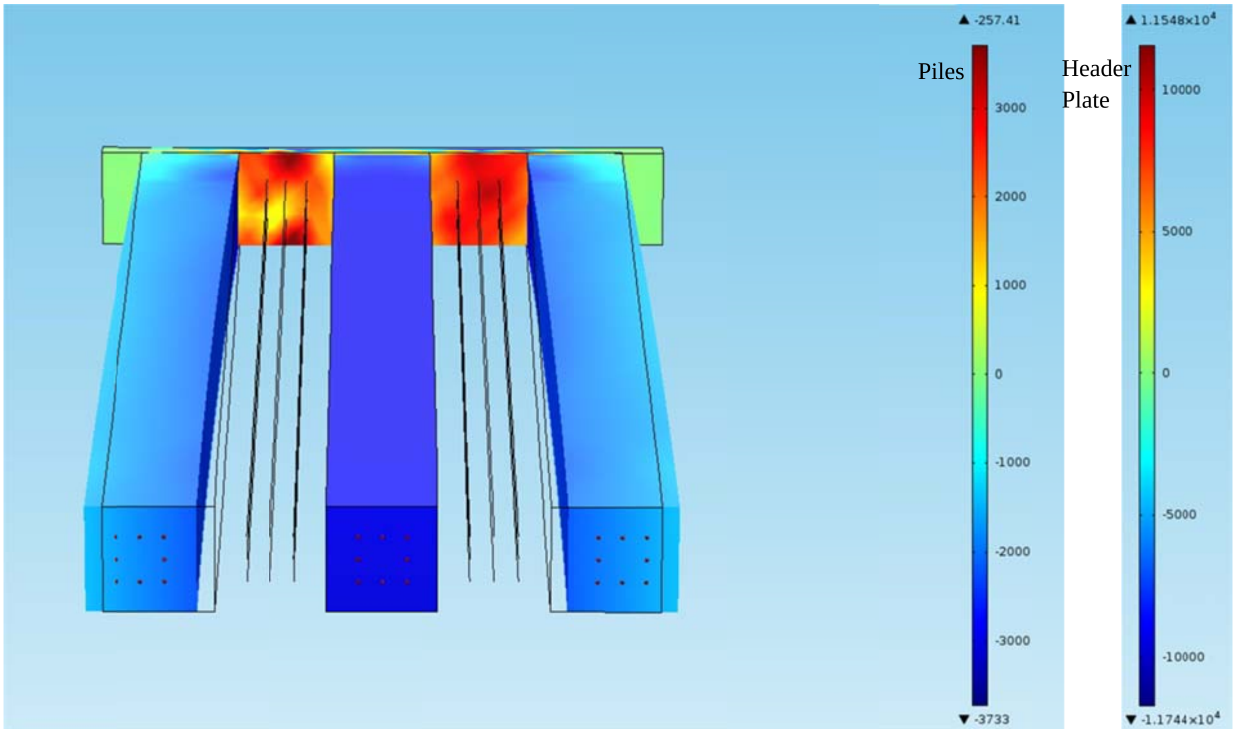


Figure 4.3. Stresses from flexure in the header plate, and from combined flexure and compression in the piles (psi).

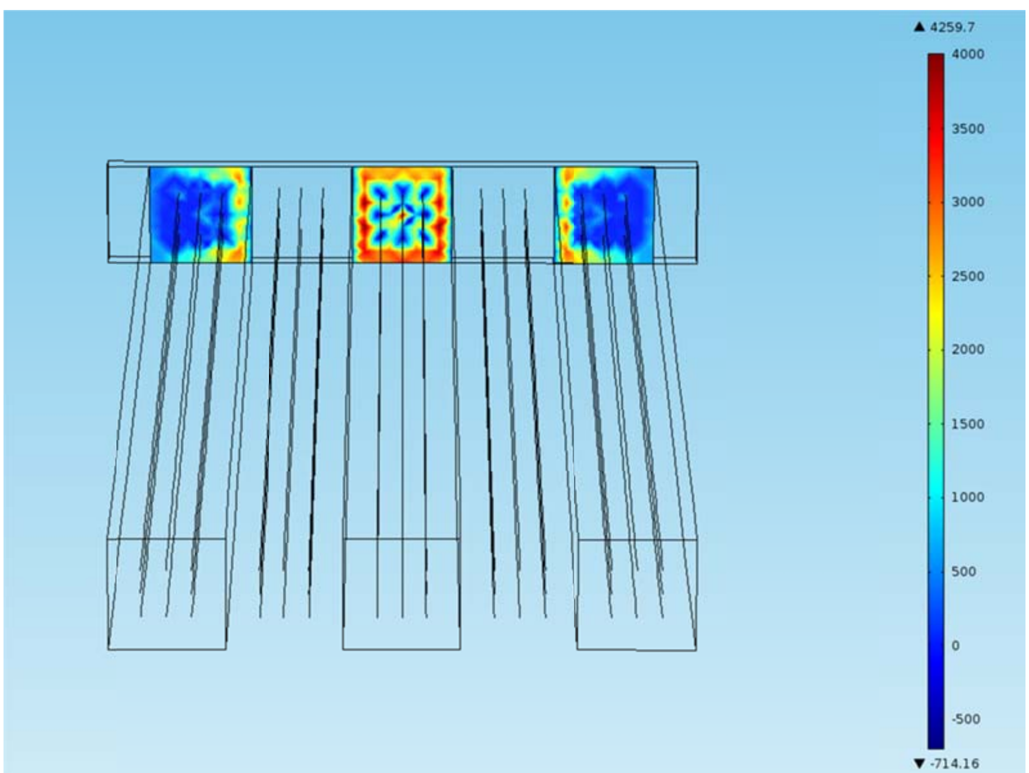


Figure 4.4. Contact pressure between header plate and piles (psi).

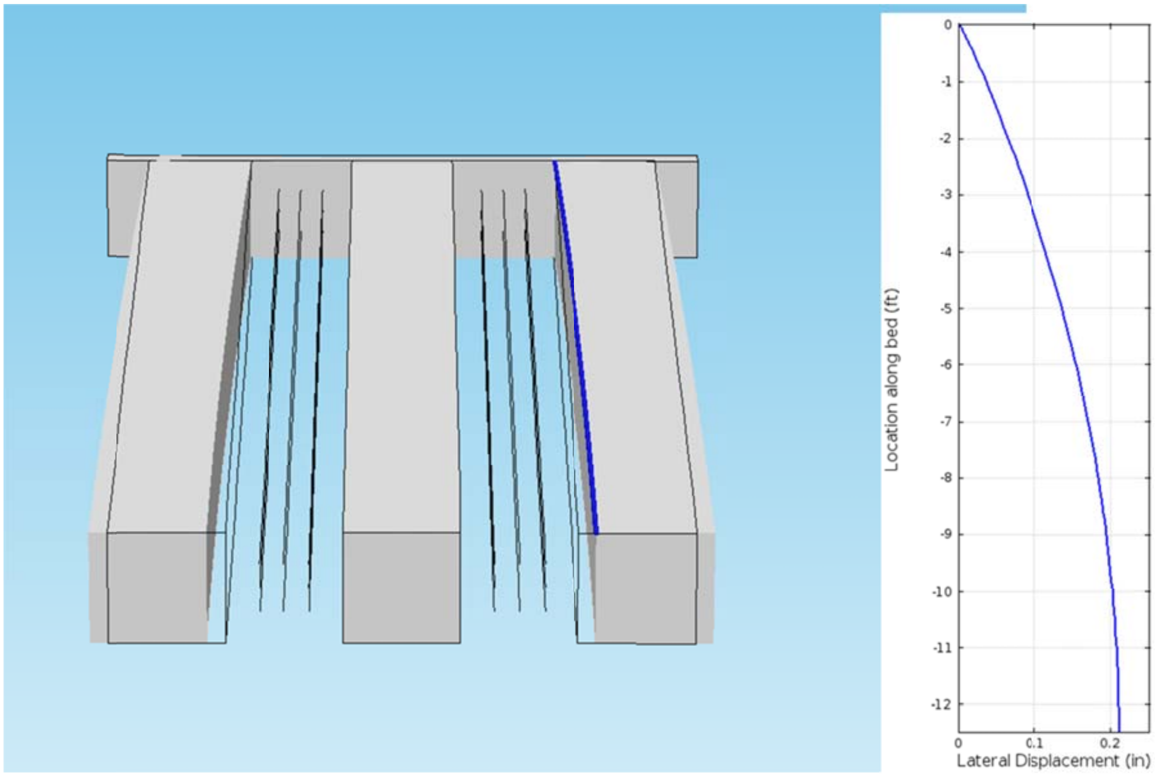


Figure 4.5. Lateral displacement of outer piles.

A second series of models was run to show what amount of plate offset would be required to reduce the lateral distortion to minimal levels. A lead plate with a thickness of 1/8 inches was used between the header plate and placed off-center with respect to the end of the outer concrete piles.

This series of models was analyzed using a quarter model with the corresponding boundary conditions constrained to save the computational time. The quarter model was established as shown in Figure 4.6, using ANSYS.

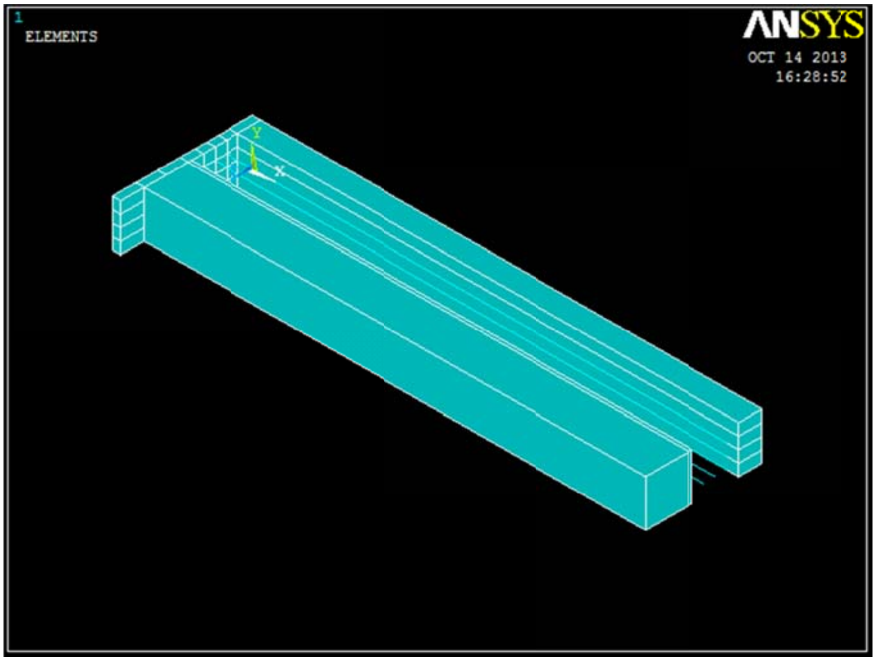


Figure 4.6. Established Quarter Model

Since the steel plate is subjected to in-plane tension, bending and out-of-plane compression, it cannot be simply modeled as a plane stress element. Therefore, a more sophisticated 3D 8-node shell element 281 was used (Figure 4.7). With this element, boundary continuity was ensured along the interface.

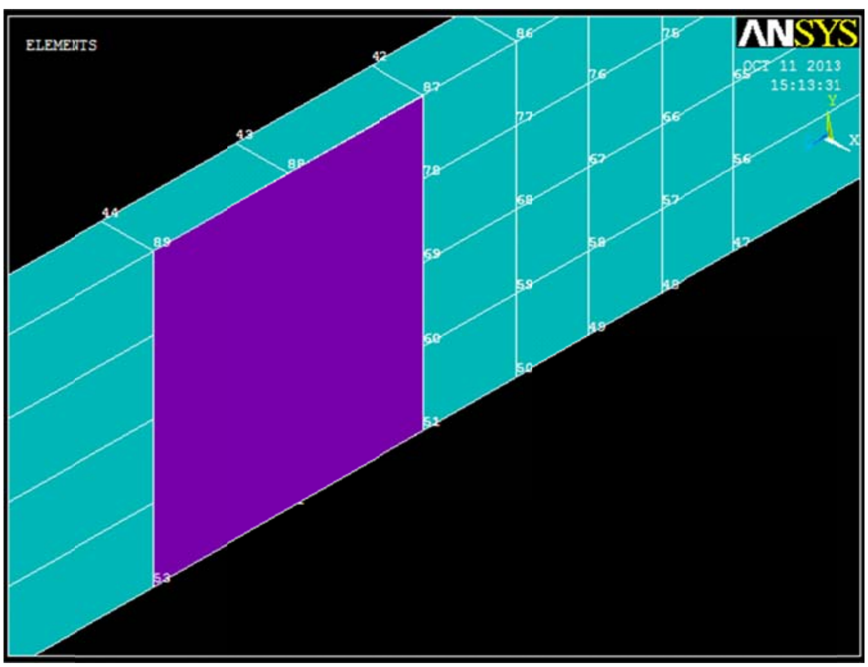


Figure 4.7. Shell Element Model

The prestressing load of 31kips was applied at each strand location. After several loading steps, the computational results converge as shown in Figure 4.8. This shows the typical deformed shapes of the concrete pile.

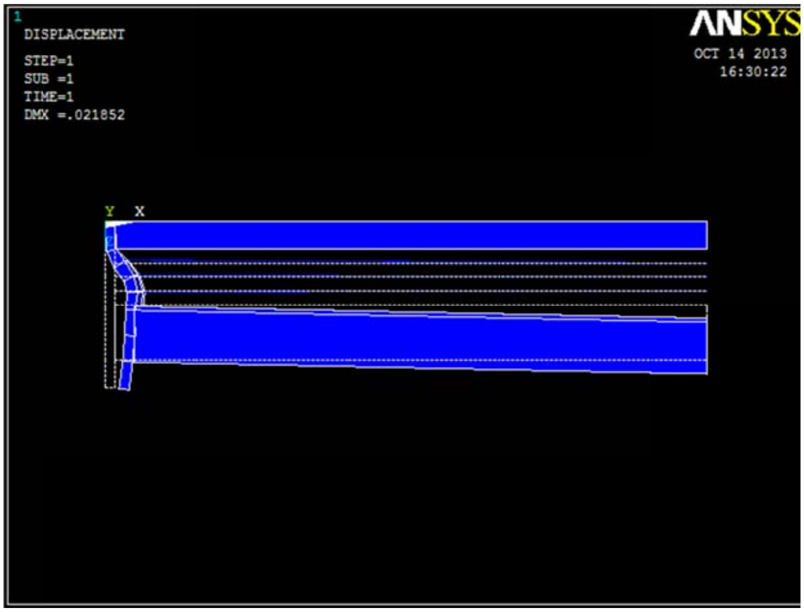


Figure 4.8. Deformed Shapes

Using the more sophisticated header plate element, the outer piles were estimated to move 0.327in outward at the mid length locations without making any eccentricity adjustments. This was slightly more than predicted in the first set of models. Based on the model outputs, a 1in offset of the lead plate, the lateral deflection reduced to 0.136in; a 2in offset reduced the lateral deflection to 0.0101in; a 3in offset caused an inward deflection of -0.0768in. Using these findings, the lateral deflection versus lead shim offset is plotted in Figure 4.9.

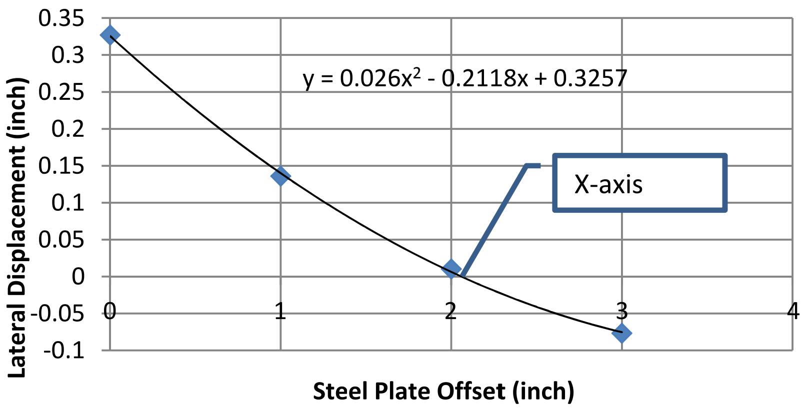


Figure 4.9. Lateral midpoint displacement vs lead plate offset.

Using this information, a 2 1/8in offset was used in the trial casting of the first prototype pile specimens.

4.1.2 Casting Bed Fabrication and Assembly

Aside from purchasing the 14in piles used for the casting bed, several steps were taken to ensure the bed could safely withstand the 500kip compression load. These included: cutting each pile to the same length (as near as possible), cutting the lifting hooks, drilling the 2.5in header plate to match the strand pattern, anchoring the ends of bed so out of plane distortion would not result, pouring a capping compound between pile ends and head plate to remove irregularities, and instrumenting the compression strut piles to monitor bed performance.

After cutting off the hooks from the top of the piles, a cursory evaluation of the top of pile planeness was conducted and high spots were ground down level. The piles were then flipped over (bottom of original casing side up) to provide the correct side slope/tapper and set into their final position. This revealed further variations in the surface planeness that required further fine tuning in the form of grinding and shimming.

Given the overall force in the casting bed was expected to be near 500 kips (16 strands at 30 kips), an assumed alignment error of as much as 1 degree, a worst case scenario, was estimated where lateral out of bed plane forces may develop on the order of 8.7kips at one or both ends, [500 sine(1°) = 8.7kips]. As a result, ground anchors embedment lengths were determined based on a 10kip load and then installed at each of the four corners of the bed to counteract these forces and maintain bed stability. The ground anchors were installed to a depth of 9ft using threaded bars and load tested to 20kips to verify capacity.

Identical header plates were used on both ends. Precise location of the strand holes were determined to accommodate the side-by-side bed. Figure 4.10 shows the 2.5in plate being drilled with this pattern. Figures 4.11 through 4.16 show a progression of the casting bed setup.

Figure 4.10. 2.5in thick header plates drilled for standard 14in strand pattern.


Figure 4.11. Placement of header plates at the ends of the casting beds.

Figure 4.12. Drilled header plates in place.

Figure 4.13. Coring slab to install ground anchors for header plates.

Figure 4.14. Mixing mortar for ground anchors.

Figure 4.15. Placement of 1in high strength threaded bar, 9ft into grout filled hole.

Figure 4.16. Header plate restraints.

Figure 4.17. *Rockite* capping compound used to provide intimate contact between piles and header plates.

4.2 Casting and Splicing Prototype Piles (14in)

Two 14in prototype splice piles were cast along with a companion control pile cast in the adjacent bed. This section describes the steps taken to cast each of the piles and the lessons learned that were incorporated into the subsequent specimens. All piles were tested in four-point bending to assess performance of the splices relative to the unspliced controls.

4.2.1 Splice System Assembly and Bed Setting (casting first prototype pile)

Each of 8 ducts used to provide the splice were cut to length (57in), fed through the 7in wide splice header and welded to the modified chucks making the ducts all one piece. For this first pile, the one-piece duct was used to prevent inadvertent contamination of the ducts from concrete falling into the header. Recall, the splicing header was machined flat as parallel set of plates to ensure the two splice pile segment ends are matched. Even if the header is slightly out of square in the bed, the two faces of the pile would still perfectly align to maintain the correct longitudinal alignment of the overall spliced pile (pile must be spliced in same orientation as the original casting bed). Figure 4.18 shows the fully assembled duct and header assembly test fitted outside the bed.

Figure 4.18. Splice header with splicing ducts and extension ducts to the top of pile jacking plate.

Confinement Coils: Numerical modeling (Figure 3.12) showed that locally high concrete stresses result in the regions directly in front of the anchorage bearing plates and those stresses extended approximately 4 to 6in until distributed to uniform stress. In this region, confinement coils made of 80 ksi steel were fabricated and attached to the anchorages. These confinement coils were designed to increase the concrete strength by 4ksi (the worst case stress increase noted in the models). Figure 4.19 shows one of these coils.

Figure 4.19. Confinement coils used to bolster local concrete stresses in the field identified by modeling.

Grout Flow Tests: A post tensioning duct grout (approved by the qualified product list, QPL) was selected and tested to ensure it could pass through the wedges without special passages. Figure 4.20 shows the grout being pumped from bottom up with the strand fully seated in a chuck assembly. As ducts are intended to be grouted after stressing, all ducts were plumbed together in the lower pile segment and a grout port was provided out the side of the pile. Ducts were also extended to the top of the upper pile segment in order to facilitate jacking, and also afforded visual confirmation of grouting effectiveness.

Figure 4.20. QPL approved grout pumped through chuck assembly.

Setting the Bed: The process of setting the bed involved: placing the splice header, ducts, and spirals; threading the strands through the spirals and header assemblies (ends and splice); jacking the strands, and tying the spirals into position. Figures 4.21 - 4.29 show the overall setting of the bed, stressing and concrete placement.

Bed stressing order shown in Figure 4.25 was selected such that loading on the two-bed / three-compression-strut system would minimize eccentric loading of the center strut and the system as a whole.

Figure 4.21. Threading strand into casting bed.

Figure 4.22. Strands in place and load cells on center two strands of each pile (dead end).

Figure 4.23. Live end prior to jacking.

Figure 4.24. Stressing jack engaged.

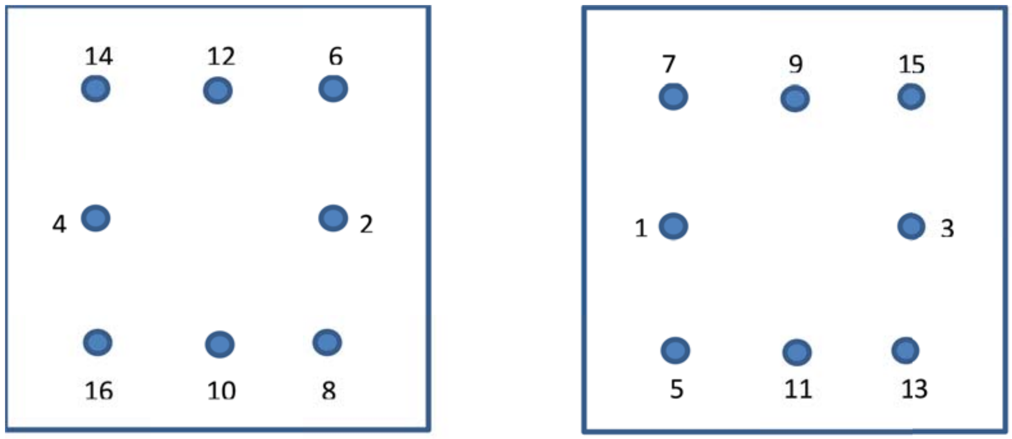


Figure 4.25. Bed stressing order (looking north from live end).

Figure 4.26. Fully stressed bed with spiral reinforcement in place; splice specimen left, control pile right.

Figure 4.27. Grouting manifold beneath lower pile embedded anchorages.

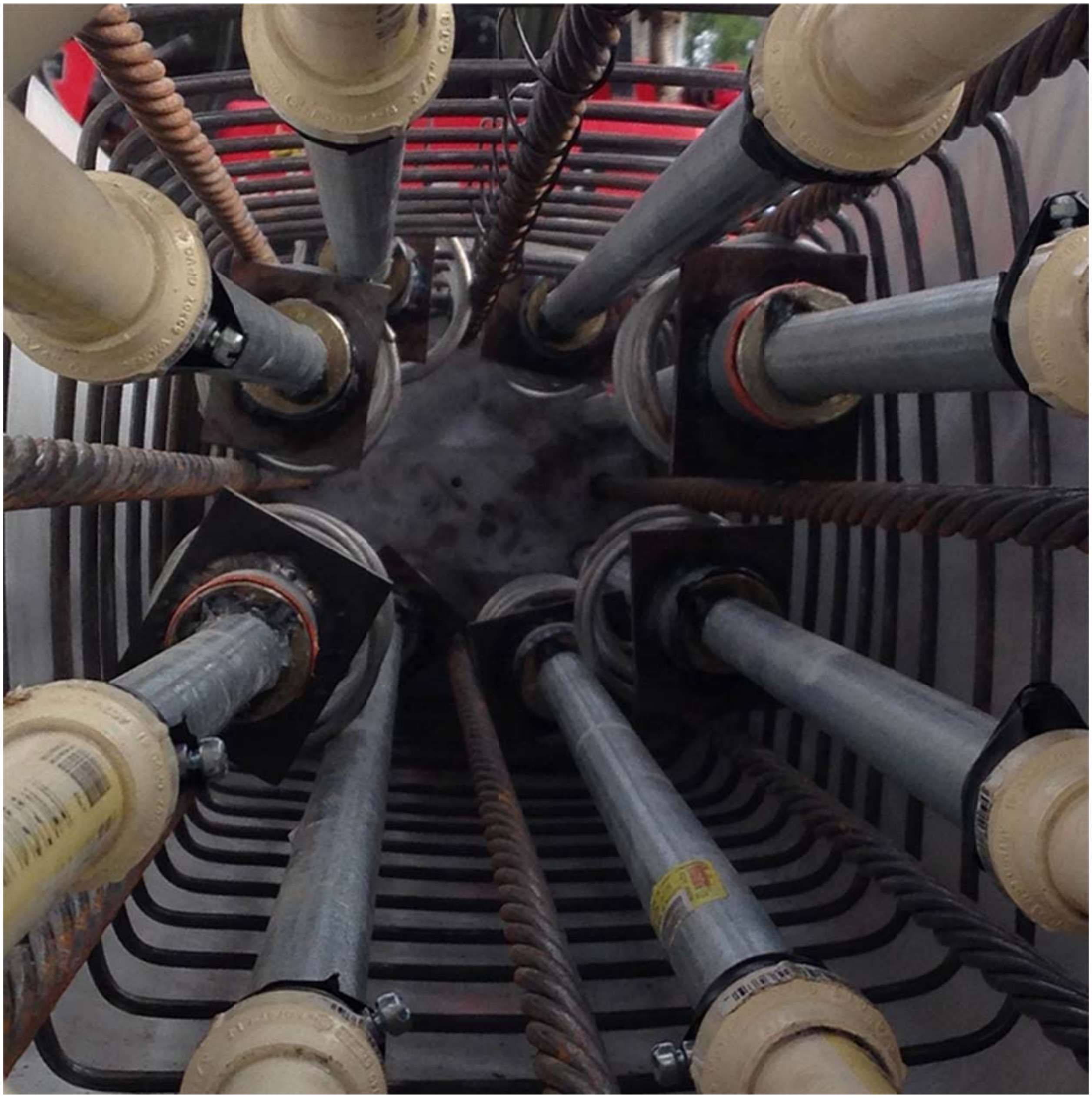


Figure 4.28. Close-up view of lower pile embedded anchorages staggered in opposing pairs at 10inch spacing/offsets.

Figure 4.29. Concrete placement with 4.5in slump; east pile (control) to the right.

Evaluation of Bed Performance. Using the strain gages installed on each casting bed compression strut, the force in the bed was monitored throughout the stressing, curing, and detensioning timeframes. Stressing was performed using the pattern shown in Figure 4.25 where the center strut maintained a balanced load. While only four strands were instrumented with load cells, those strands were also the first four stressed which provided the ability to confirm bed shortening via the load lost during subsequently stressed strands. Figures 4.30 and 4.31 show the loads in the bed and the four load cells. Load cells were colored for wire tracking, but were also named based on Pile side and strand side (e.g. EW denotes East pile and the West strand). Not to be confusing, but the compression struts were named East, West and Center bed piles in those figures. Therefore, from east to west, the bed layout started with (1) the east compression strut (east bed pile), (2) east pile specimen (with EE load cell and EW load cell), (3) center compression strut (center bed pile), (4) west pile specimen (with WE load cell and WW load cell), and (5) the west compression strut (west bed pile). As a further point of reference, Figure 4.29 was taken standing on the south side of the bed looking north. The south side was also the live end (jacking end). Load cells were on the north side (dead end).

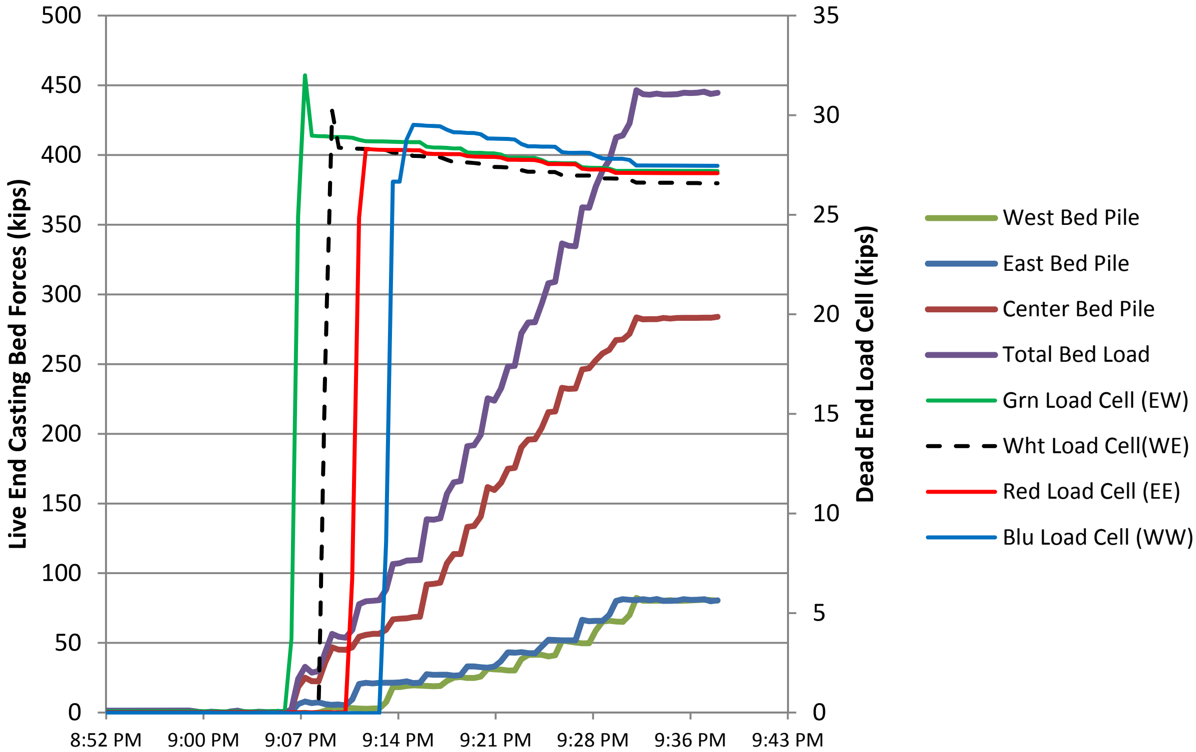


Figure 4.30. Casting bed response to stressing (Live end bed force; dead end load cells).

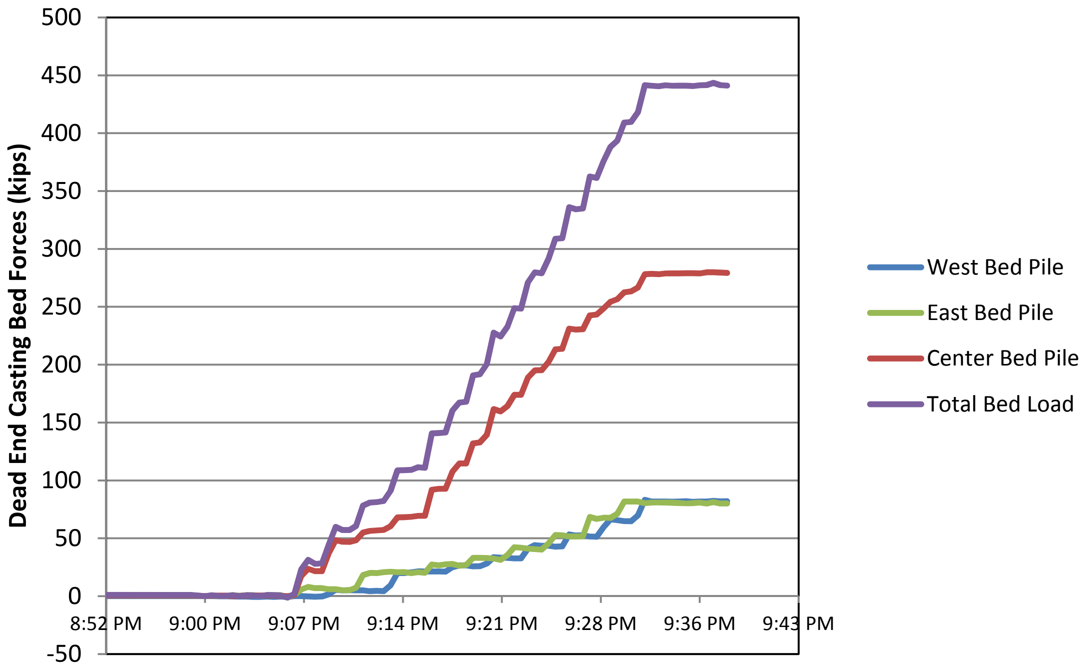


Figure 4.31. Dead end casting bed forces from strain gages.

Figure 4.32 shows how the jacking pattern controlled the center compression strut eccentricity and kept the force in the casting bed balanced. Also, the outer two compression struts show the eccentricity ended at about 4in off-center which corresponds well to the 2 inch void/spacer placed there to minimize the outer strut bending and mid-length lateral movement (2in void stopped 5in from centerline). While numerical modeling showed the 2 inch spacer would eliminate lateral movement, 0.013inches of outward movement was recorded in both outer bed piles (compression struts). This was determined to be due to the increased modulus in the bed piles originally assumed to be 6ksi concrete. The actual strength was calibrated to be closer to 9ksi based on the recorded bed strains.

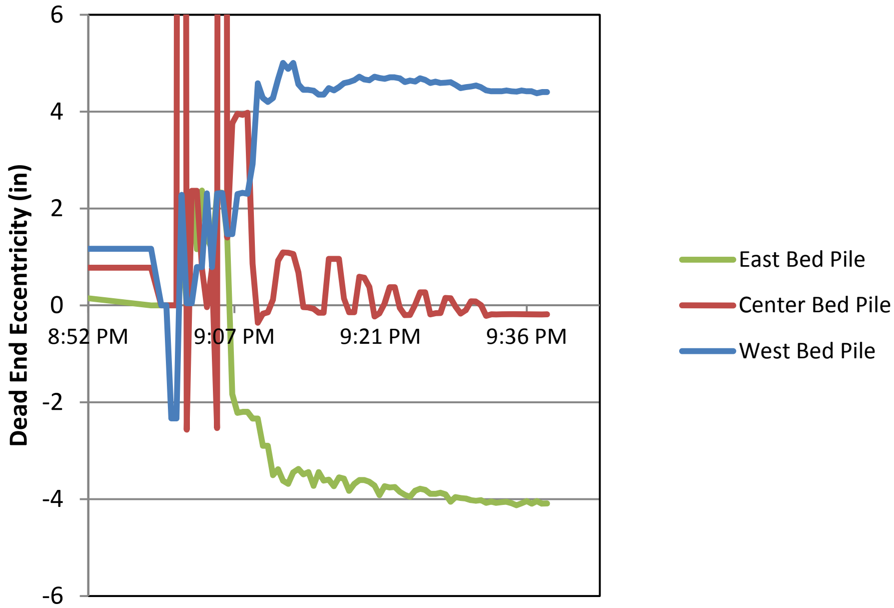


Figure 4.32. Casting bed eccentricities during stressing.

Detensioning was performed 3 days after casting at an average concrete break strength of 6350 psi. A detensioning order was adopted from standard practices for the west pile, but the east pile order was rotated 180 degrees to maintain bed concentricity. Figure 4.33 shows the detensioning order with the east/right side rotated 180degrees. Figure 4.34 shows the strands being cut and Figures 4.35 – 4.37 show the force in the bed and load cells during strand cutting.

Strain gages mounted to the concrete surface of the pile specimens were used to measure the strains locked in by the prestressing and subsequently to be used to show superimposed strains from splicing. A 5in interval between strain gages was selected to coincide with the midpoint between anchorages. Figure 4.38 shows the strains recorded from the ends and splice zone all referenced to the dead end of the pile.

Figures 4.39 – 4.43 show the pile removal and end preparations before splicing.

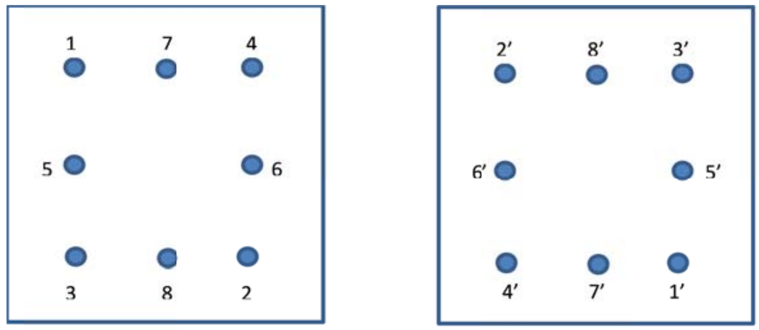


Figure 4.33. Detension order (looking north from live end).

Figure 4.34. Strand cutting live end (TL), middle splice header (TR), and dead end (bot).

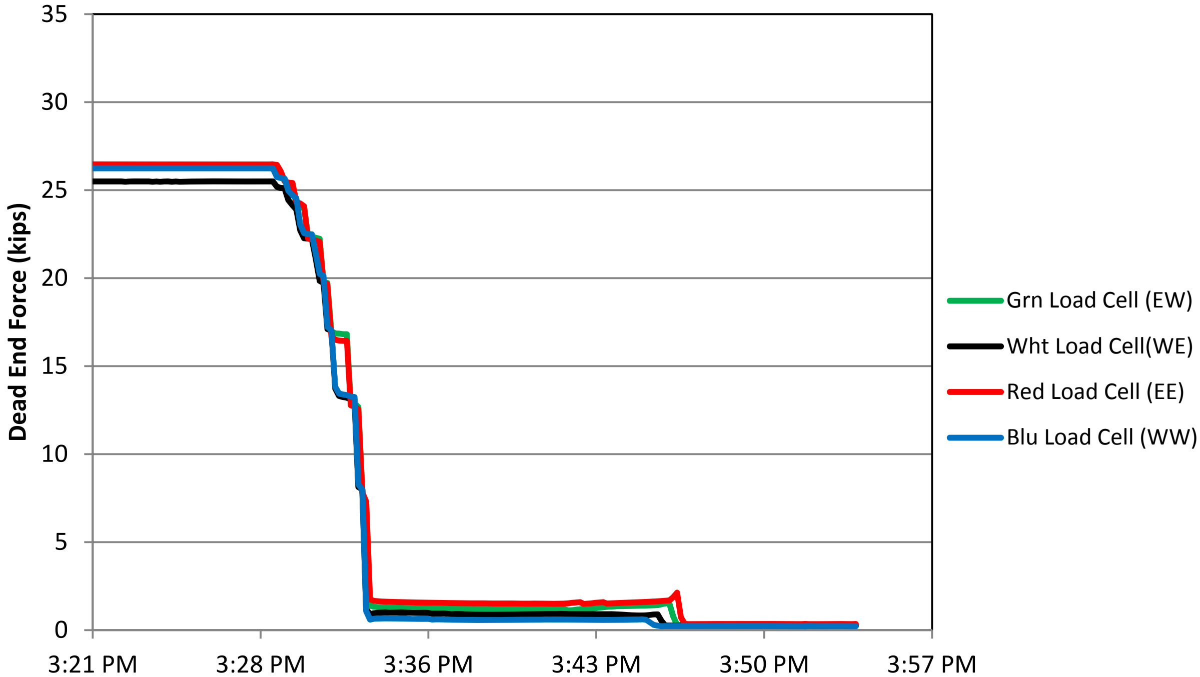


Figure 4.35. Load cell measurements during detensioning.

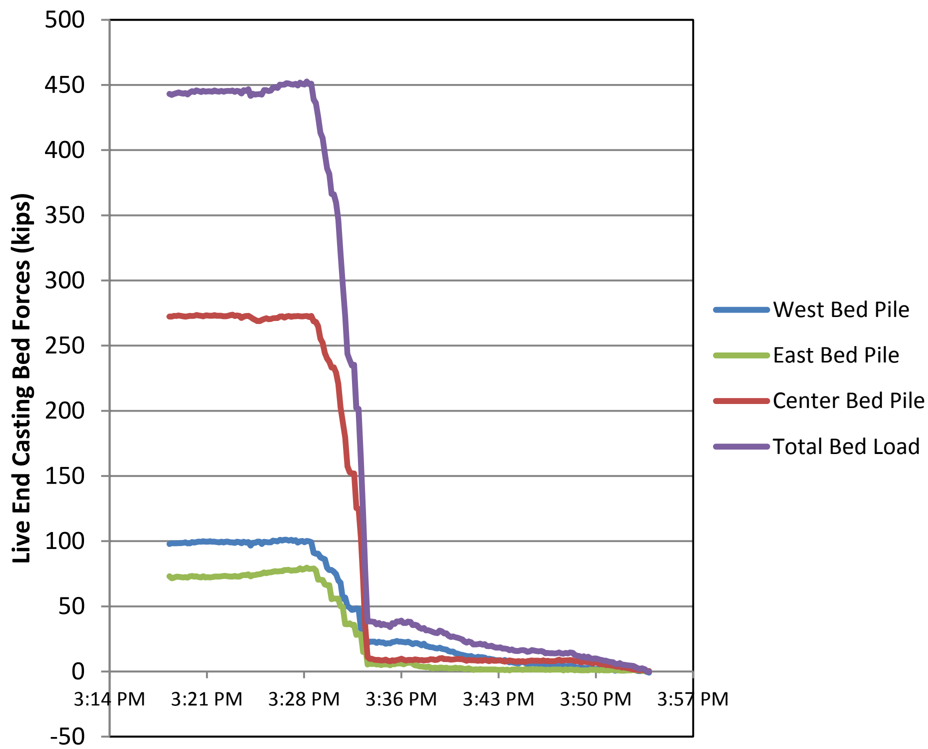


Figure 4.36. Load in casting bed struts during detensioning (live / south end).

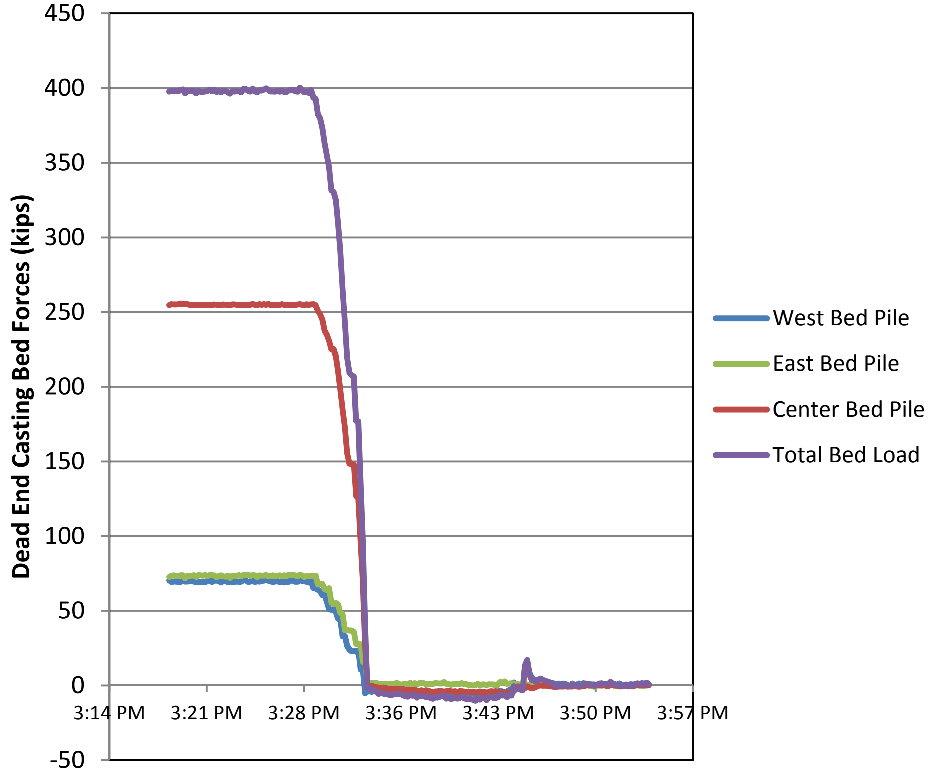


Figure 4.37. Load in casting bed struts during detensioning (dead / north end).

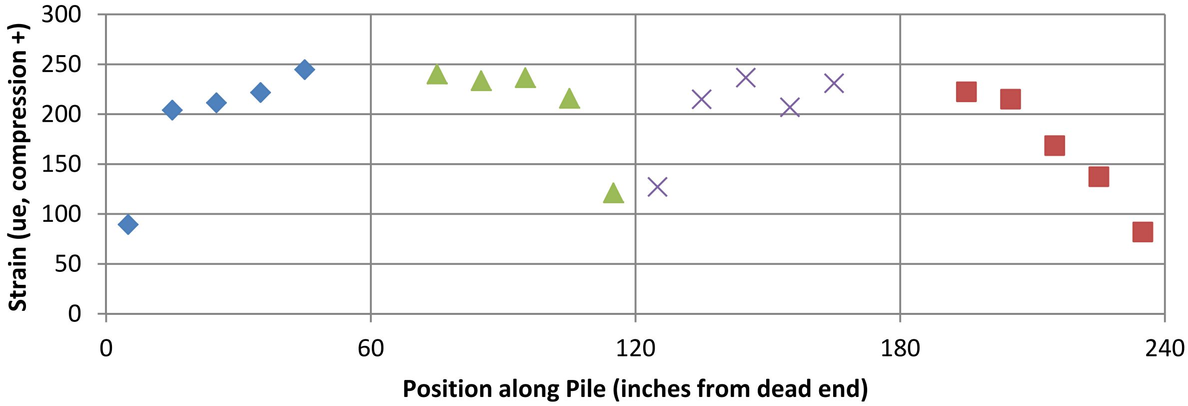


Figure 4.38. Concrete surface strain in piles; transfer length longer on cutting end (also live end).

Figure 4.39. Un-spliced control pile extracted from casting bed; plastic bed lining peeled off.

Figure 4.40. Splice pile specimen: lower segment breaking loose (TL), splice header separation (TR), lower segment with splice header (BR), upper segment with end of bed header (BR).

Figure 4.41. Extracted piles before grinding off duct and strand stubs.

Figure 4.42. Strands cut from jacking (upper) end of upper pile segment.

Figure 4.43. Mated ends of splice after grinding off strand and duct stubs.

4.2.2 Splicing First Prototype Pile (14in)

The procedure for splicing was tailored to be as time efficient as possible in the field. To this end and in preparation, all splicing strands were fed into the upper pile segment leaving 60 inches exposed on the bottom end and clamped to ensure no slippage (upward movement into upper pile segment) during assembly with the lower pile segment. Three feet of exposed strand was provided at the upper end of the upper pile segment for jacking. With these preparations completed, the strands could slide up or down in the upper pile segment ducts. *Clamps at the base of the pile on the strand prevent strands from moving into the pile while the wedges in the anchorages prevented movement out of the pile.* Splicing proceeded using the following instruction:

- inspect ducts for debris and check spring movement of embedded wedges.
- apply an epoxy sealant to the splicing surface of lower pile.

Note: if pile splice is performed horizontally, epoxy will require a medium body paste consistency. Vertically spliced piles can use more fluid material provided side forms are provided. However, care should be taken to keep epoxy from filling the ducts that are later grouted.

- bring the piles together close enough to align each strand.

Note: strands can be cut with ½ to 1in staggers to aid in alignment without necessitating simultaneous alignment of all strands at once.

- lower the upper pile until in close proximity to lower pile and where all strands have penetrated the lower pile anchor wedges (approx. 2 - 3in).

Note: for horizontal splicing, epoxy can be applied at this time but additional clearance will be required to facilitate access. This was the case for the splice performed for the first prototype pile.

- remove strand clamps
- mate pile segments
- while epoxy is still wet, stress the strands in crossing pairs
- cut off excess strand from top of pile with torch or grinder.
- with pile in upright position, inject grout into lower pile grout ports until fluid grout is observed in upper segment ducts (top surface)

Note: grout can be installed at any time and is not necessary for pile driving. Pile driving can proceed immediately after grout has been placed or if grouting port is still accessible, grouting can be performed later.

An overview of these steps as performed for the first prototype pile is provided.

Pre-Splice Checks: The prototype pile splice specimen did not contain sealing caps that were later added to subsequent piles (or production piles). As a result, extra care was observed to keep rain water, insects, lizards, etc. from getting into the ducts. After grinding, each duct was taped shut and the specimens were put under shelter.

Prior to splicing, checks of each spring-loaded wedge assembly were performed by sliding a ¾in diameter solid rod in the duct and pushing on the wedge set thus activating the spring behind the wedges (Figure 4.44). The size of the push rod was previously established such that it was large enough that it could not become ensnared in the wedges (yet small enough to pass through the duct). Recall, the chucks/bearing plate assemblies were drilled out to accommodate the O.D. of the duct to allow smooth passage of a strand; this also permitted the push rod to clear easily. Lower segment anchorages (without washers) were expected to compress approximately 3/8in, while the upper segment anchorages with the washers should have only compressed 1/4in (difference caused by two 0.060 shim washers).

Figure 4.44. 48in long 3/4in diameter rod used to check anchorages by compressing wedge assembly spring.

For the upper pile segment, an 18ft length of strand (splice strand length = segment length + 5ft + 3ft) was also passed through the assembly and pulled entirely through to demonstrate the wedge sets were working properly (one way passage from bottom to top).

Prior to splicing, quality control measures were used to ensure no unknown blockages in the lower pile segment would cause difficulties. These first included the wedge spring movement (approximately 3/8 in) as shown previously in Figure 4.44. Second, a borescope was used to inspect the ducts for cleanliness and again the presence of foreign objects. Figures 4.45 - 4.47 show this process and the internal images. Figures 4.48 - 4.58 show the overall splicing process after inspection.

Figure 4.45. Borescope with 360 degree articulating viewing head.

Figure 4.46. Real-time imaging of duct cleanliness.

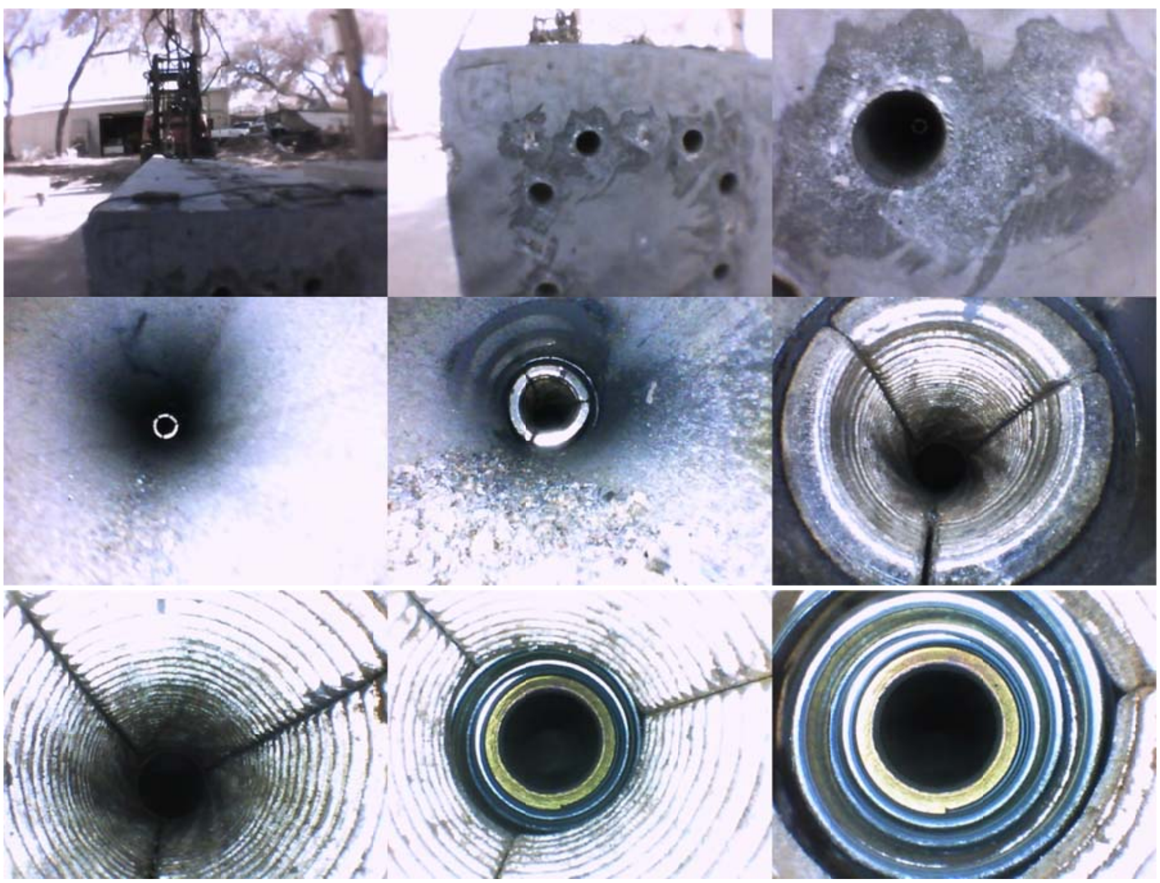


Figure 4.47. Borescope images of duct 8 showing debris near wedges from grinding / cutting off ducts (center image).

All ducts were blown out and debris was successfully removed. An adaptation for subsequent specimens will include threaded end caps and ducts will not pass through splice header assembly.

Figure 4.48. Threading strands into upper pile segment. Each strand measured to be ½ in longer than the previous (shortest strand stubbed out 60 in; longest 63.5 in).

Figure 4.49. All strands in upper pile segment and clamped to prevent inward slippage. Paint stripes show 1.5 in unpainted region to detect whether strands move inward instead of being locked and to show movement was only into lower segment during splice assembly.

Figure 4.50. Lower pile segment aligned with upper segment on splicing rail.

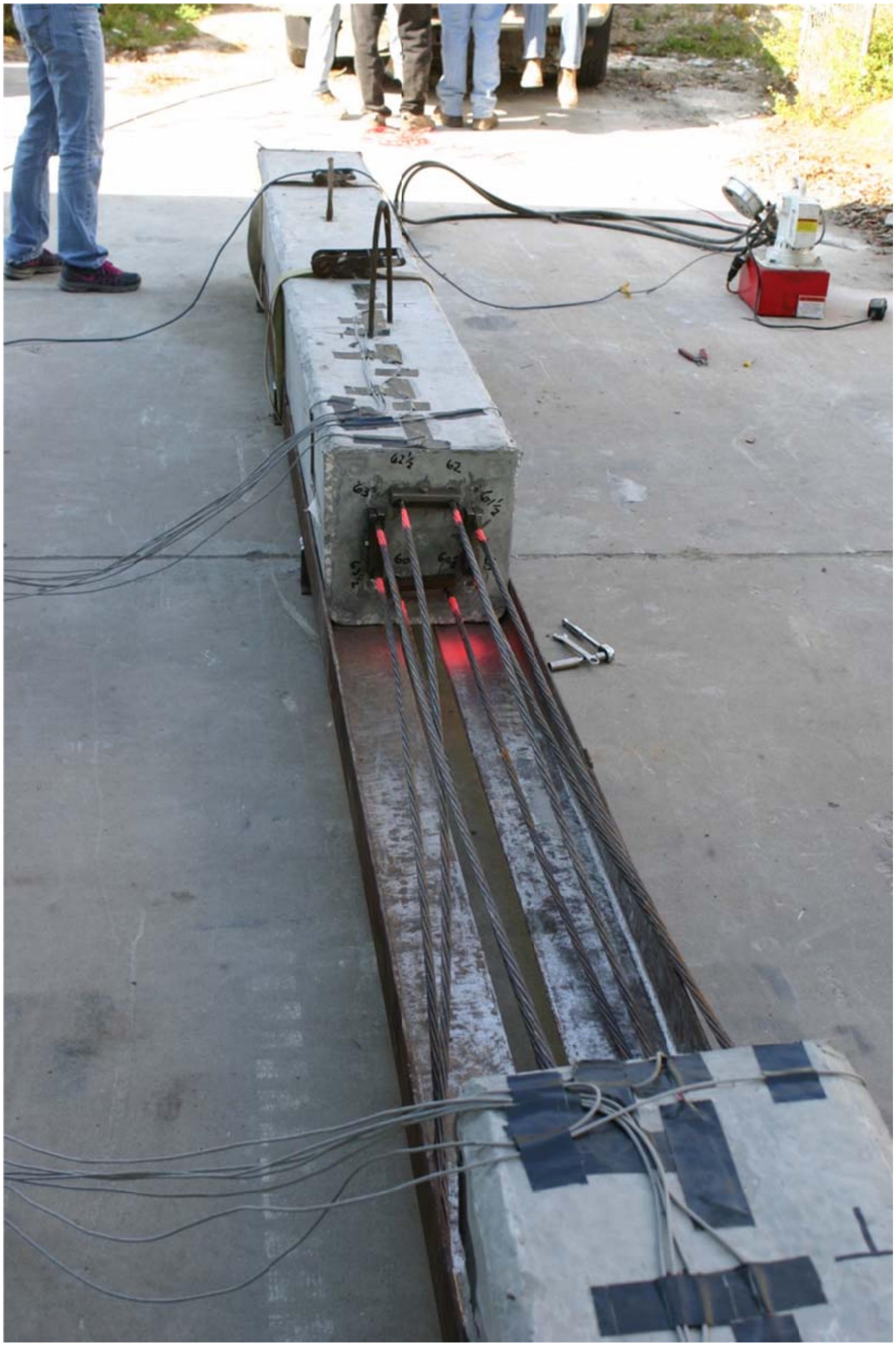


Figure 4.51. Both pile segments on assembly/alignment rail. Upper segment top; lower segment bottom.

Figure 4.52. Aligning first of 8 strands as pile segments are pushed together. A second paint stripe indicates when each strand will come in contact with the embedded anchorage wedges.

Figure 4.53. Weight of moving segment held by overhead crane while fork lift pushed the segments together and while each strand was sequentially inserted into the lower segment ducts.

Figure 4.54. All strands inserted. Flexibility of the 5ft strands allows additional options to insert strands at will even if all strands were the same length. However, staggering also meant that no two anchorages were encountered at the same time.

Figure 4.55. All strands embedded past anchorage wedges (lower pile right).


Figure 4.56. With all strands in, strand clamps need to be removed (upper segment right).

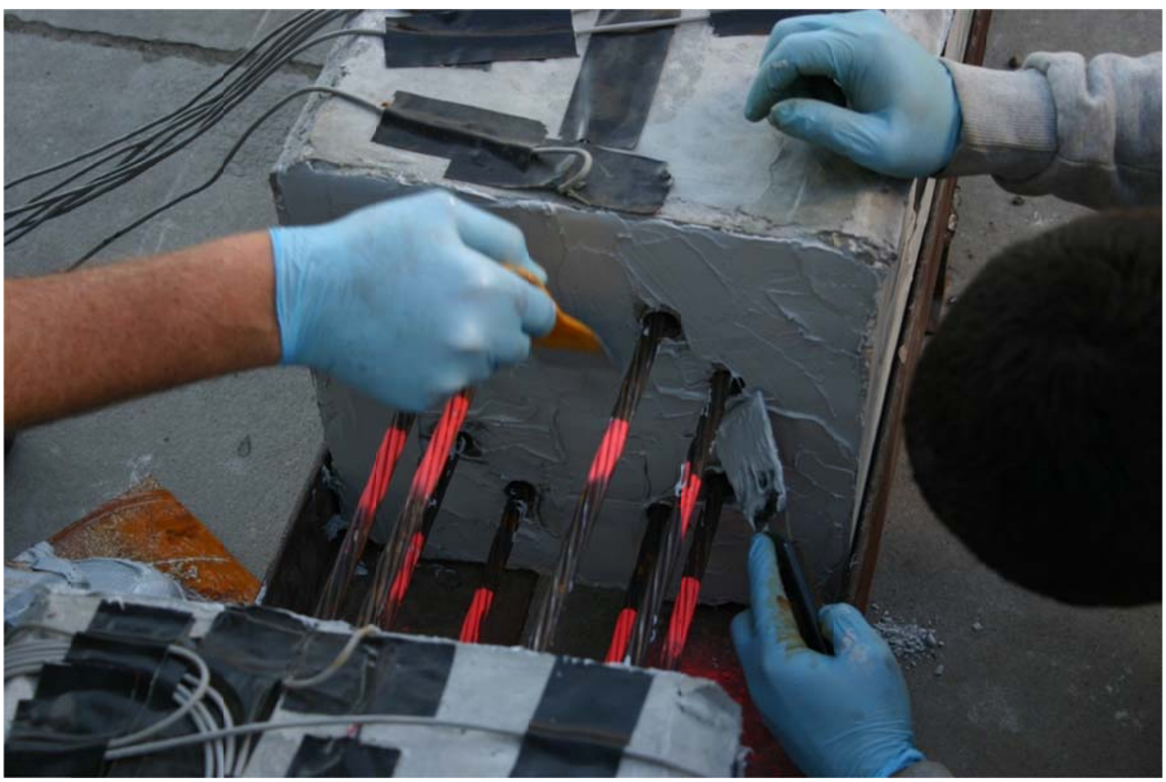


Figure 4.57. QPL-approved epoxy applied to both sides per manufacturer’s recommendations.

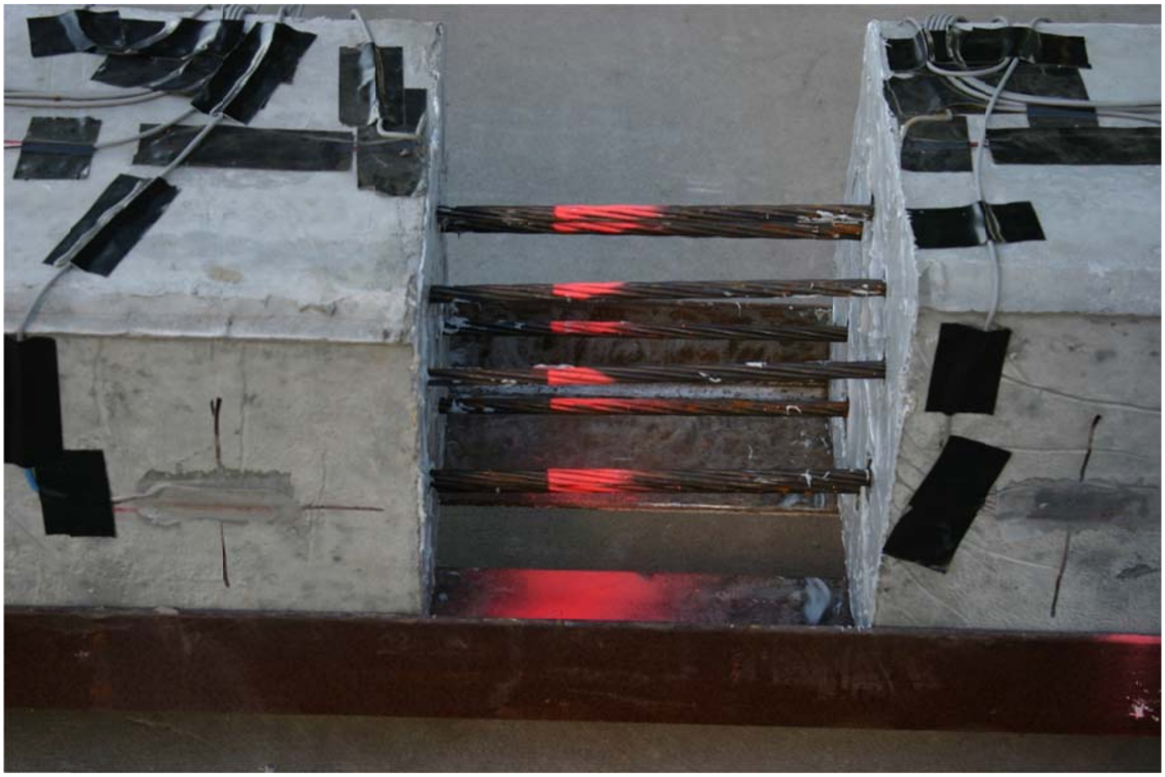


Figure 4.58. Strand clamps removed, epoxy applied to both sides, ready for final assembly. Note no movement of strands into upper segment (left) during assembly. The more restrictive wedge assemblies in upper segment (from double washer inserts) provided more resistance to sliding and therefore all movement from this point forward was into the lower pile segment (to the right).

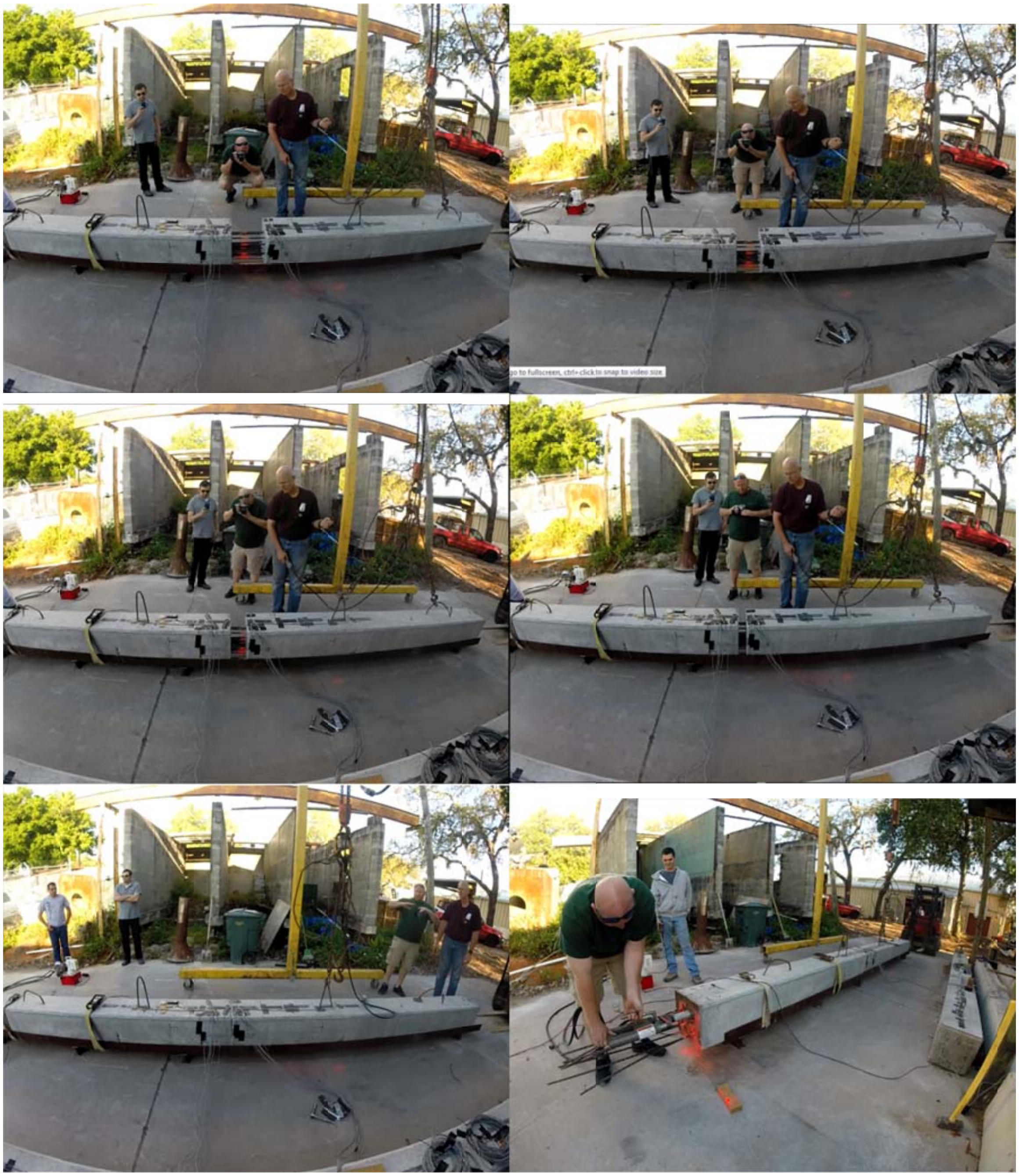


Figure 4.59. Final joining and stressing the splice connection.

The stressing order was selected such that pairs of opposing strands with identical anchorage locations were jacked to predetermined loads in three stages. Similar to tightening cylinder heads on combustion engines, progressively increasing staged loading provided two benefits: maintained balanced strain and reduced losses in the first strands stressed. Without staging, the first strand was calculated to lose approximately 0.8 kips from the subsequently loaded strands and the associated elastic compression in the concrete.

In theory, the more stages used the better where lower losses result as the concrete compression stabilizes with more uniform strand forces. Conversely, fewer stages are preferred when addressing field efficiency. As an upper limit there is a practical limit to the amount of force increase in the strand that can be attained with subsequent pulls due to the imprints made on the strand by the wedges. If a subsequent strand pull does not cause enough elongation to move a given strand imprint to the next wedge tooth then the wedge will reseat in the original imprint. Therefore, it is desirable to ensure that subsequent strand pulls produce enough elongation to at least exceed the tooth pitch in the wedge (25 teeth/inch). Balancing these considerations, two or three stages might be considered; three stages were chosen for the first splicing specimen. Selection of jacking forces was iterative such that each stage would just overcome an integer number of wedge teeth (e.g. 4.1 teeth preferred over 3.9 teeth). Table 4.1 and 4.2 shows the loss in the first strand from subsequent pulls for both the two and three stage options. Table 4.3 and 4.4 likewise shows the number of wedge teeth overcome with each stage of stressing for both options. The 3 stage option resulted in lower losses but still develops enough elongation to engage a different set of imprints.

Table 4.1. Losses in first stressed strand from subsequent strands (3 stage option).

Force in 1st Strand Stressed (kips)			
Strand Loaded	Stage 1	Stage 2	Stage 3
1	7	18.5	26.5
2	6.940056	18.44894	26.46448
3	6.880112	18.39787	26.42896
4	6.820167	18.34681	26.39343
5	6.760223	18.29575	26.35791
6	6.700279	18.24468	26.32239
7	6.640335	18.19362	26.28687
8	6.58039	18.14255	26.25134

Table 4.2. Losses in first stressed strand from subsequent strands (2 stage option).

Strand Loaded	Stage 1	Stage 2
1	8	26.5
2	7.935615	26.38899
3	7.871231	26.27798
4	7.806846	26.16698
5	7.742462	26.05597
6	7.678077	25.94496
7	7.613693	25.83395
8	7.549308	25.72295

Table 4.3. Number of teeth overcome in subsequent stages (3 stage option).

Stage	Force (kips)		Concrete compression (in)	Strand Elongation (in)	Relative Elongation (in)	Number of Teeth
	Jacking	After release				
1	13.5	7	0.000685504	0.15213	0.152805	N/A
2	25	18.5	0.000900672	0.20284	0.20374	5.093511
3	33	26.5	0.000725541	0.163399	0.164124	4.103106

Table 4.4. Number of teeth overcome in subsequent stages (2 stage option).

Stage	Force (kips)		Concrete compression (in)	Strand Elongation (in)	Relative Elongation (in)	Number of Teeth
	Jacking	After release				
1	14.5	8	0.000725541	0.163399	0.164124	N/A
2	33	26.5	0.001250933	0.281722	0.282973	7.07432

Strain gages mounted to the four sides of the pile segments (5 inches on either side of the splice joint) provided a means to track the eccentric load from the stressing sequence. The final state of stress in the pile is shown in Figure 4.60 where some eccentricity can be seen, but the average concrete stress was approximately 1040 psi. Figure 4.61 shows the stressing order of the splicing strands.

Using strain data from the splicing along with that from the detensioning (Figure 4.38), the combined effects from both detensioning (transfer length effects) and splicing are shown in Figure 4.62. For this specimen, strain gages were only mounted on the top surface in an array that coincided with the midpoint between anchorages (e.g. 5, 15, 25, and 35 inches). A fifth gage was also mounted at 45 inches behind the last anchorage, farthest from the splice. As expected, no appreciable strain was observed behind the last anchorage (slight tension). The combined / superimposed strains from splicing and initial prestress amount to only a 20% increase above the norm noted at the time of detensioning.

Note: as the strains are measured between anchorages, higher strains are likely to exist just at the face of the anchorage as shown in the assumed saw tooth strain distribution. These will vary depending on whether it is on the live or dead end of the splice due to anchor set losses.

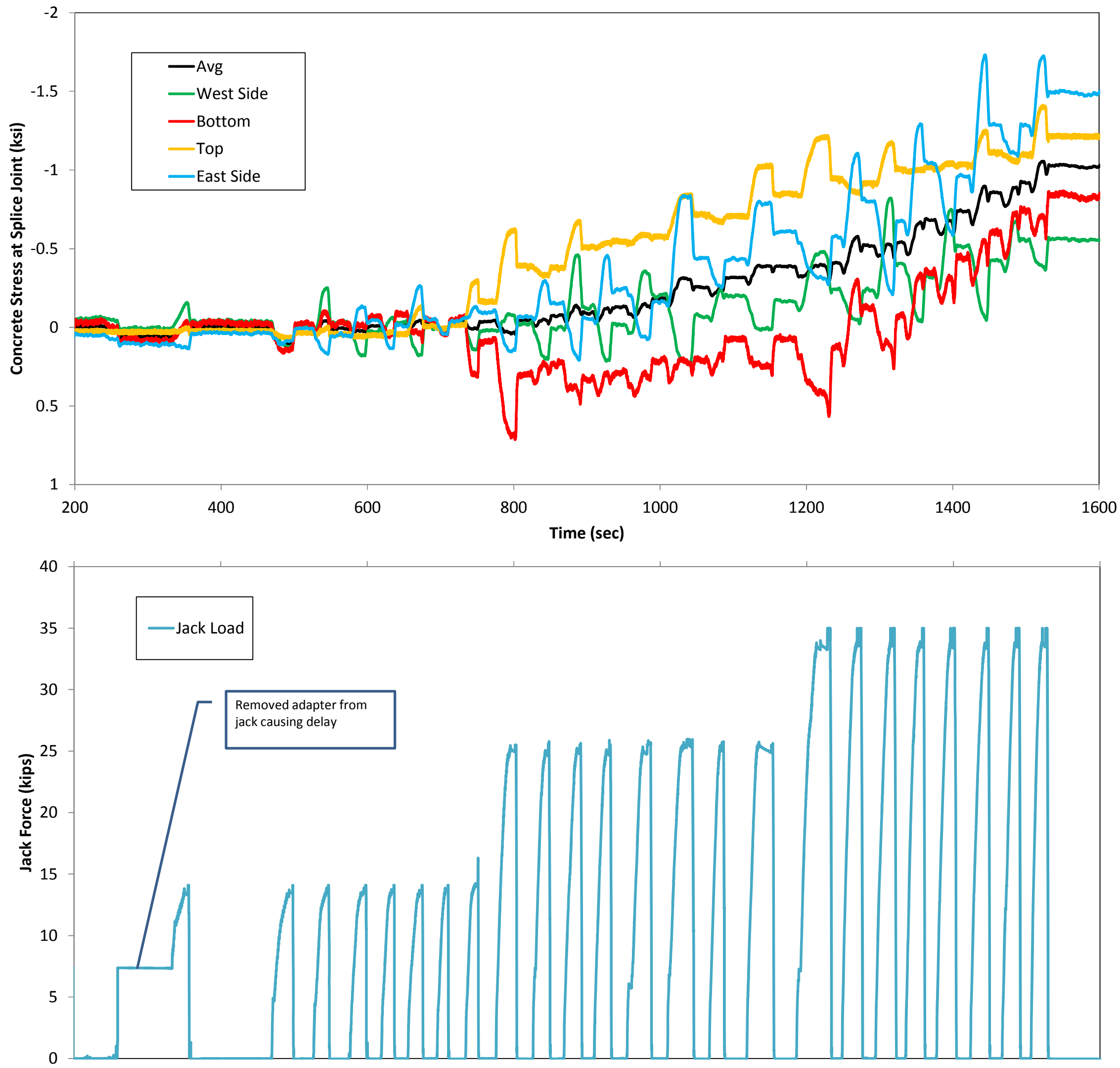


Figure 4.60. Concrete stress at splice joint during splicing (top); jack force for three stages of stressing (bottom). Each peak represents a stressed strand.

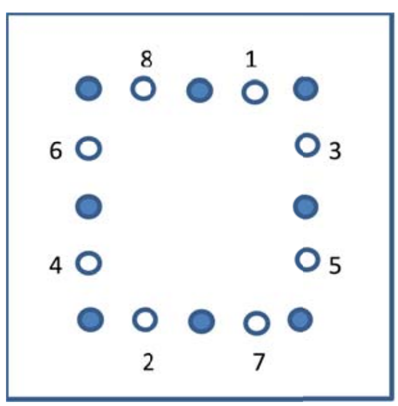


Figure 4.61. Splice stressing order of matched pairs (open circles denote splice strands; closed circles denote existing pile strands).

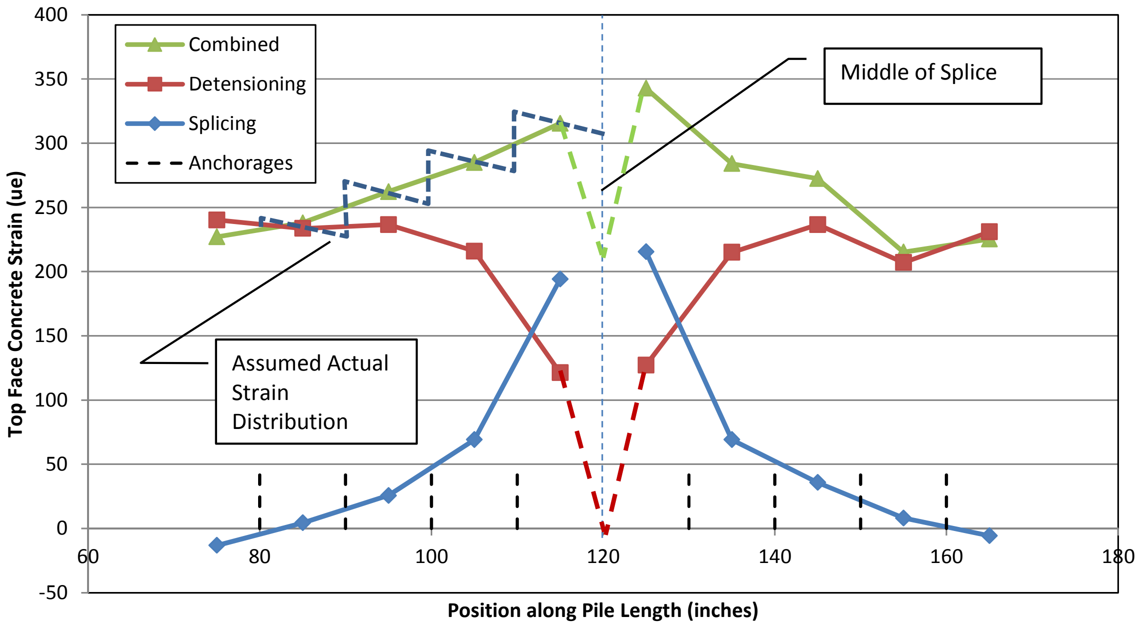


Figure 4.62. Concrete strain from splicing and detensioning.

Several modifications were noted which were incorporated into the second splice prototype specimen:

Design / Fabrication changes:

- Provide threaded duct ends to prevent contamination from getting in ducts during strand stub cutting, pile driving, shipping, or storage.
- Isolation sleeves to prevent epoxy from filling grouting ducts (threaded into the same duct end cap threads)
- Change clamping assemblies to individual strand clamps (strand clamps used during shipping and assembly need to be individual and not grouped to allow spinning during final strand insertion)
- Use longer embedment in lower pile segment (longer strand length between piles provides more flexibility in the strand during splicing and can be more easily manipulated to align in lower pile embedded wedges).
- Full weld on all ducts to caps
- ½ in anchorage plates (discontinue ¾ in thick option)
- Pre-weld anchorage plates to chucks before drilling and reaming; this eliminates the need of precisely aligning during welding
- Washers need larger ID / OD must match chuck I.D.; this keeps washers from falling into the path of the strand during insertion

- 7/8” I.D. ducts for all locations (discontinue use of larger 1.25in, upper segment ducts as they are no longer needed; smaller ducts will suffice).

Procedural Changes

- Seal spring caps before threading strands in bed (sealing after strands in place provided poor access)
- De-burr and test fit ducts to grout manifold before installation in bed
- Vacuum test all ducts before threading strand
- Inspect all ducts with borehole scope
- Continue push rod spring displacement test

4.2.3 Casting Second Prototype Pile (14in)

While the basic procedure of casting was the same as the first, preparations for the second pile set added the deformation of the ducts to increase bond (Figure 4.63) and the fabrication of threaded duct inserts (specialized bolts) to hold the ducts to the splicing header assembly (Figure 4.64). Figure 4.65 shows the deformed ducts attached to anchorages complete with confinement coils. Figures 4.66 – 4.79 show the casting process.

Figure 4.63. Duct deformation in purpose-built hydraulic stamping press.

Figure 4.64. Duct inserts were fabricated to prevent debris from getting into the ducts.

Figure 4.65. Confinement coils used to bolster local concrete strength welded to anchorages in regions of high anticipated stresses.

Figure 4.66. Splice header assembly fitted with inserts awaiting installation of ducts (TL); attaching ducts (TR); fully assembled (BR); and jacking end plate at top of upper pile segment (BR).

Figure 4.67 Assembly prior to pulling strand and spirals (looking toward lower segment).

Figure 4.68. Assembly prior to pulling strand and spirals (looking toward upper segment).

Figure 4.69. Aligment plate inserted into the lower and upper segment of the splice pile to better locate ducts.

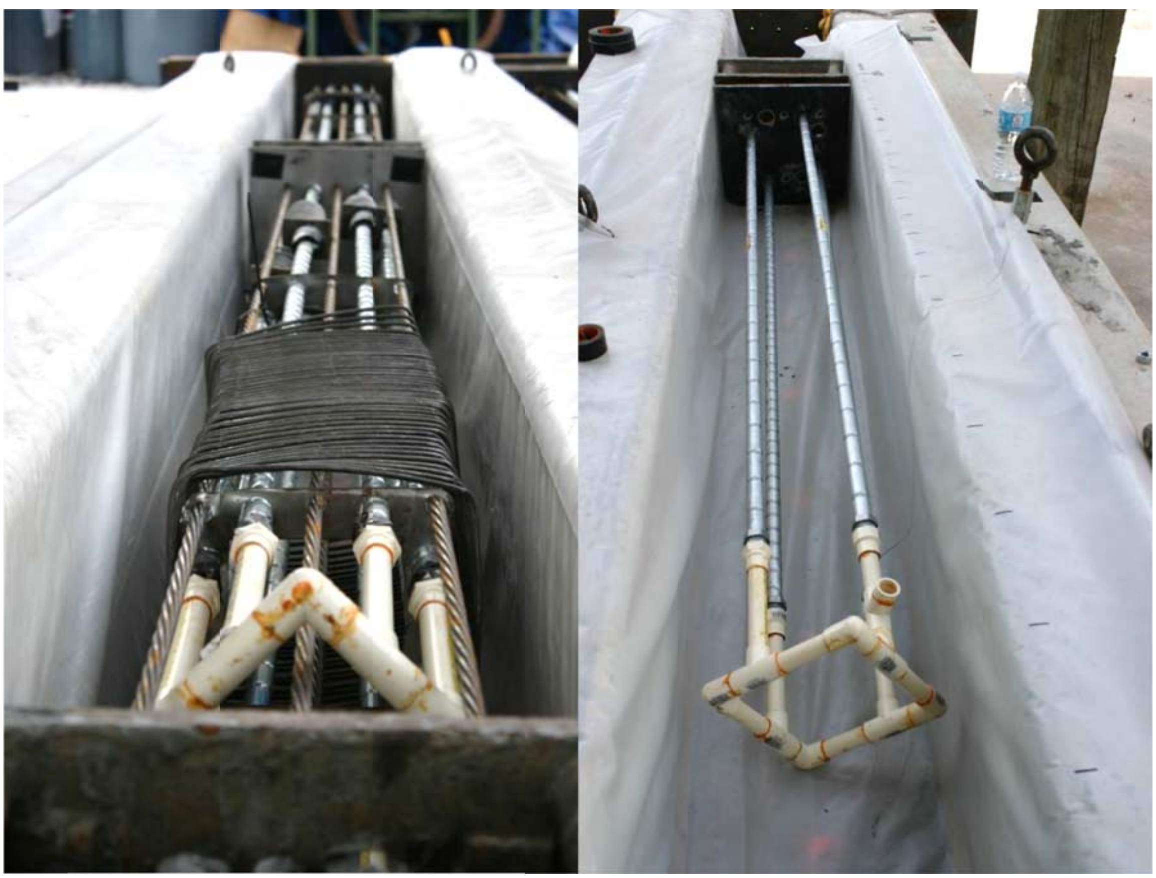


Figure 4.70. Grout manifold in lower pile segment with slight modification, no side access grout port (left); four duct grout duct and manifold in control pile for pull-out tests assessing bond enhancement from duct deformation (right).

Figure 4.71. Threading strand into casting bed.

Figure 4.72. Strands in place and load cells on center two strands of each pile on dead end; control pile left and splice pile right.

Figure 4.73. Live end prior to jacking.

Figure 4.74. Prestressing strands in bed (hydraulic wedge setter used to minimize short bed losses).

Figure 4.75. Fully stressed bed with spiral reinforcement in place; control pile left, splice specimen right.

Figure 4.76. Concrete placement with 5.5in slump; control pile 2 left, splice pile 2 right.

Figure 4.77. Strand cutting after concrete achieved strength; middle splice header (top), and dead end (bot).

Figure 4.78. Control pile extracted from casting bed.

Figure 4.79. Splice pile specimen: lower segment (left), splice header still attached to upper segment (right).

The strand load cells and compression strut strain gages were again analyzed to verify jacking loads for the second set of pile specimens. The total load in the bed (16 strands jacked to 31.8k; ending at 28k) was corroborated from strain gages (dead end Figure 19; live end Figure 20).

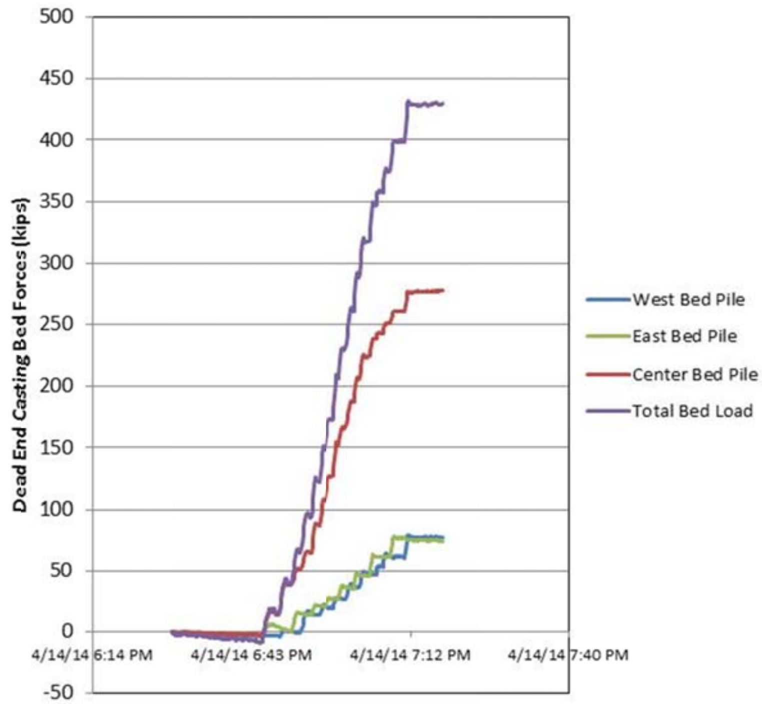


Figure 4.80. Stressing load in each compression strut of the prestress bed (dead end).

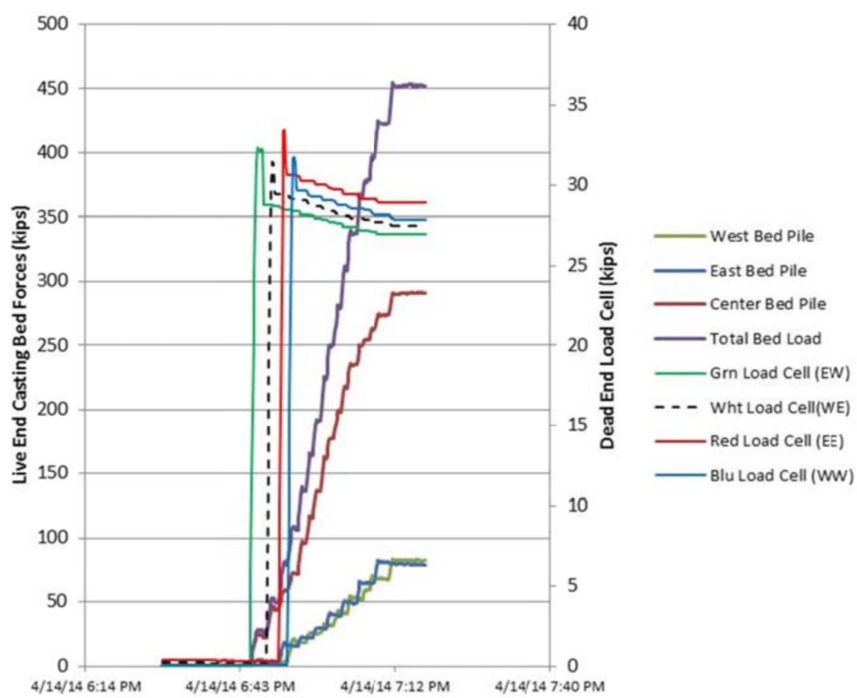


Figure 4.81. Stressing load in each compression strut of bed (live end) and strand load cells.

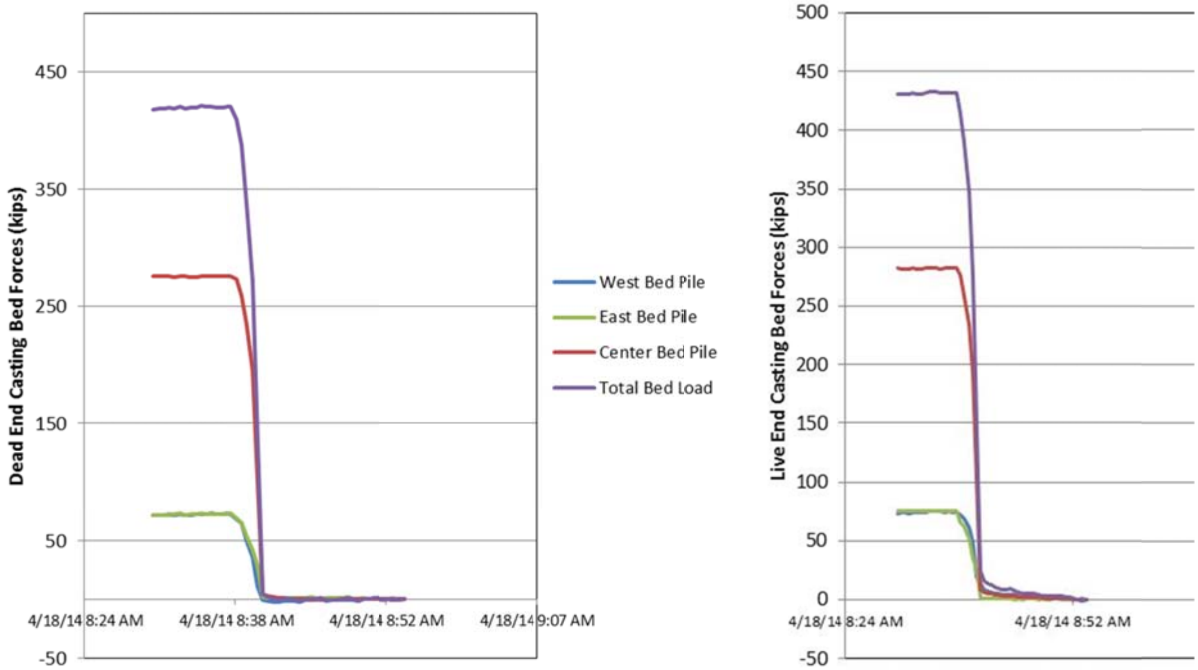


Figure 4.82. De-tensioning forces recorded from the bed strain gages.

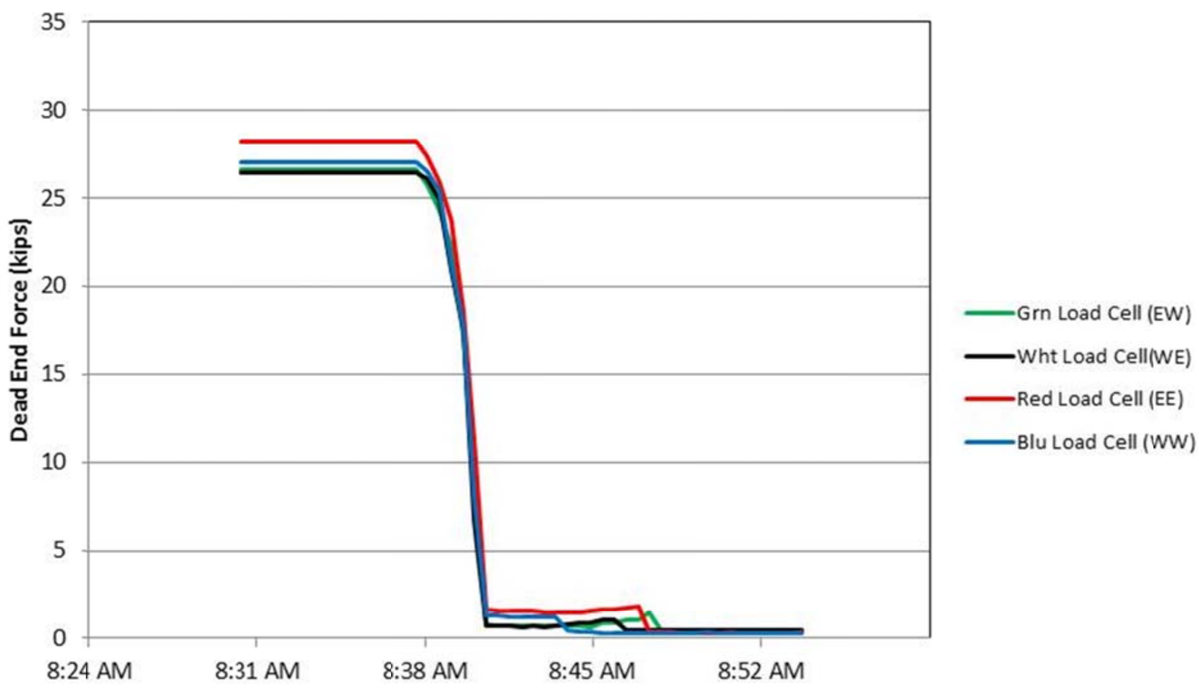


Figure 4.83. Load cell responses during strand cutting.

Subtle adjustments were made in the outer compression strut bearing locations which were confirmed by the eccentricity detected in the strain gages. Figure 4.84 shows the eccentricity in the outer compression struts was approximately 3.1 inches which corresponded well to the location of the plate contact.

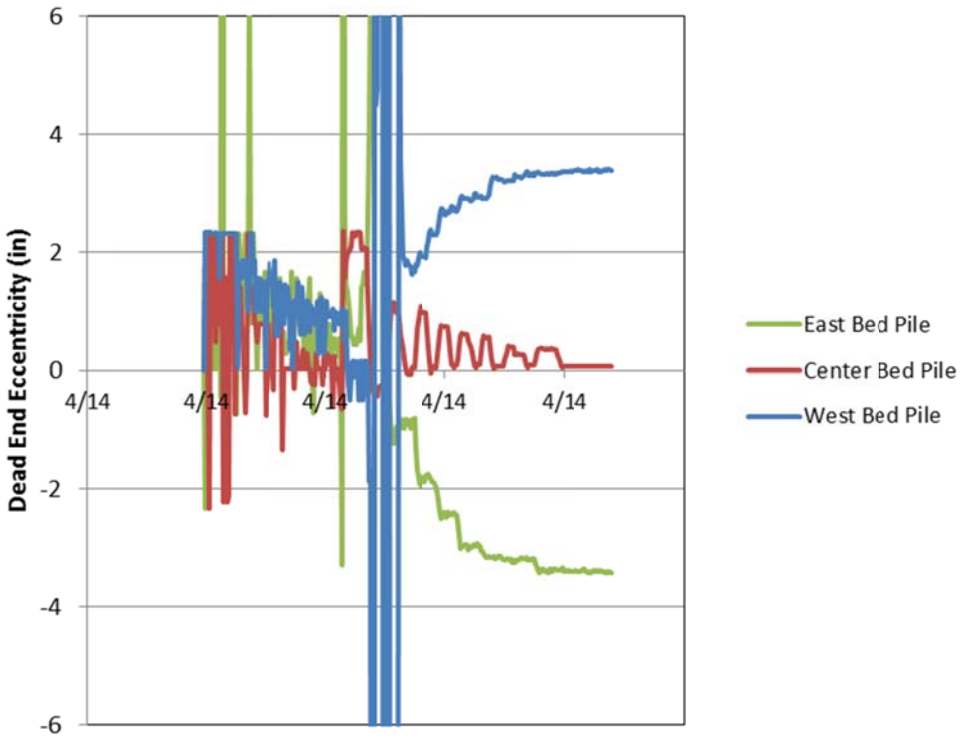


Figure 4.84. Eccentricity shown as the bed was stressed progressively from side to side.

In short, all strain gage and load cell values were consistent and verified the bed performance.

4.2.4 Splicing Second Prototype Pile (14in)

The second pile splice was again performed similarly to the first with the exception that the stressing stages were reduced to two stages instead of three. Also, instead of using the assumed value of ultimate strength for the strand (e.g. 270ksi), the exact strength was determined prior to stressing and post tensioning was taken to 75% of the tested value. In this case, the tested value was closer to 300 ksi using the embedded chuck anchorages, but the this value was recognized to be conservatively lower than that traditionally obtained when using carbide impregnated samples in specially designed grips (Table 4.5 and Figure 4.85).

Table 4.5. Results of ultimate capacity testing of ½ inch strands.

Ultimate Capacity				75% Ultimate (Design Jacking Force)
Sample 1	Sample 2	Sample 3	Average	
45.4 k	45.5 k	45.6 k	45.5 k	34.1 k

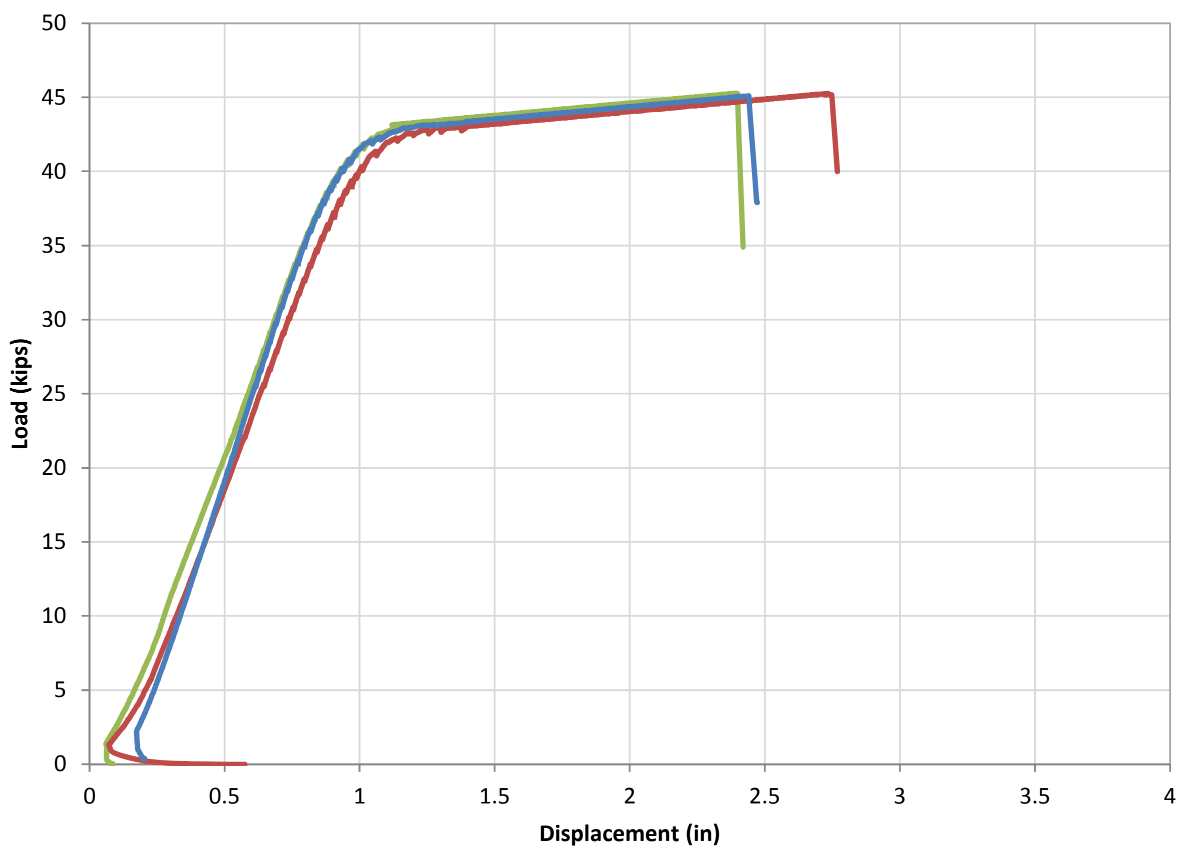


Figure 4.85. Load-Displacement curves for ½ inch strands, 3 samples tested.

Splicing Procedure: In preparation, all splicing strands were fed into the upper pile segment leaving 96 inches (this was increased from 60" in Prototype Splice Pile 1) exposed on the bottom end and clamped to ensure no slippage (upward movement into upper pile segment) during assembly with the lower pile segment. Three feet of exposed strand was provided at the upper end of the upper pile segment for jacking. With these preparations complete, the strands could not slide up or down in the upper pile segment ducts and the splicing proceeded similarly to the first pile splice where the following steps were taken:

- inspect ducts for debris and check spring movement of embedded wedges.
- apply an epoxy sealant to the splicing surface of lower pile.
  Note: if pile splice is performed horizontally, epoxy will require a medium body paste consistency. Vertically spliced piles can use more fluid material provided side forms are provided. However, care should be taken to keep epoxy from filling the ducts that are later grouted.
- bring the piles together close enough to align each strand.
  Note: strands should be cut with ½ to 1in staggers to aid in alignment without necessitating simultaneous alignment of all strands at once.
- lower the upper pile until in close proximity to lower pile and where all strands have penetrated the anchor wedges (approx. 2 - 3in).
  Note: for horizontal splicing, epoxy can be applied at this time but additional clearance will be required to facilitate access. This was the case for the splice performed in this progress report.
- remove strand clamps
- mate pile segments
- while epoxy is still wet, stress the strands in crossing pairs
- cut off excess strand from top of pile with torch or grinder.
- with pile in upright position, inject grout into lower pile grout ports until fluid grout is observed in upper segment ducts (top surface)
  Note: grout can be installed at any time and is not necessary for pile driving. Pile driving can proceed immediately after grout has been placed or if grouting port is still accessible, grouting can be performed later.

An overview of these steps as performed for this specimen is provided.

Pre-Splice Checks: Prior to splicing, checks of each spring-loaded wedge assembly were performed (Figure 4.86). Lower segment anchorages (without washers) should compress approximately 3/8 inch, while the upper segment anchorages with the reduced wedge travel should only compress 1/4 of an inch (difference caused by inserts).

Figure 4.86. 48in long rod used to check anchorages by compression wedge assembly spring.

For the upper pile segment, a 21ft length of strand (splice strand length = segment length + 8ft + 3ft) was also passed through the assembly and pulled entirely through to demonstrate the wedge sets were working properly (one way passage from bottom to top).

Prior to splicing, additional quality control measures were used to ensure no unknown blockages in the lower pile segment would cause difficulties. These first included the wedge spring movement (approximately 3/8 in) as shown previously in Figure 4.86. Second, a borescope was used to inspect the ducts for cleanliness and again the presence of foreign objects. Figures 4.87 – 4.89 show this process and the internal images. Figures 90-103 show the overall splicing process after inspection.

Figure 4.87. Borescope with 360 degree articulating viewing head.

Figure 4.88. Real-time imaging of duct cleanliness.

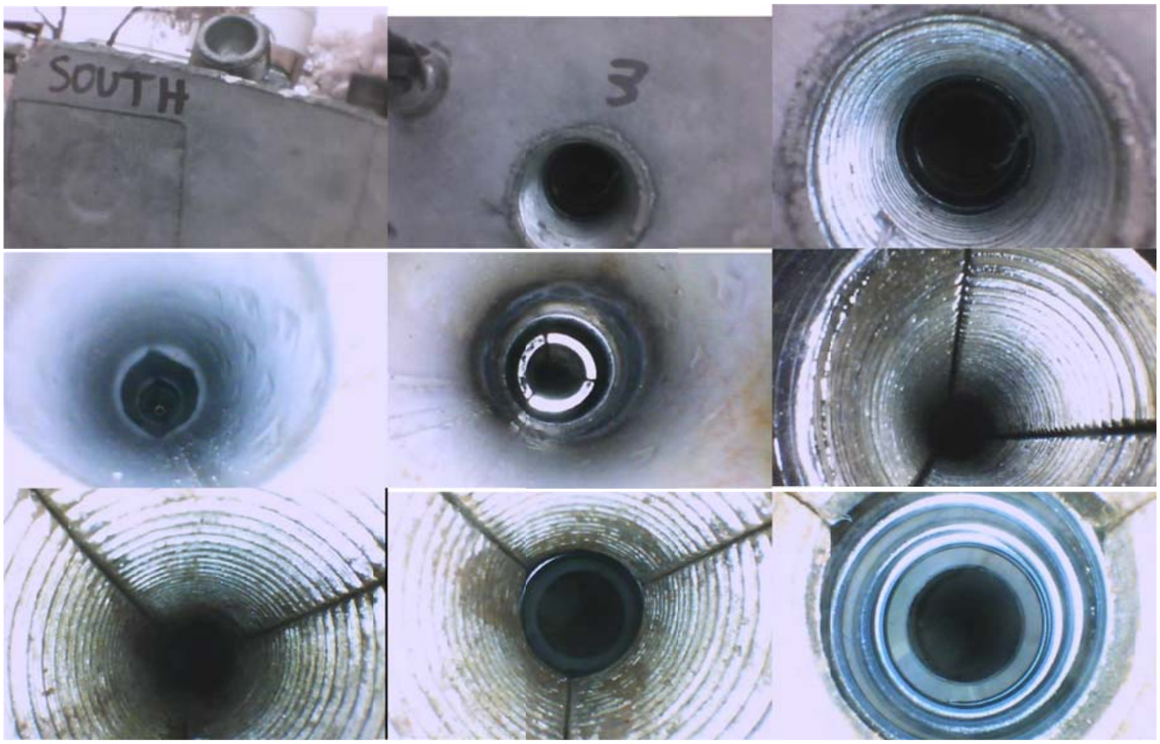


Figure 4.89. Borescope images of duct 3 showing marked reduction in debris near wedges compared to the first splice pile. This reduction in debris is due to the use of threaded duct caps which do not pass through splice header assembly thus eliminating the need to grinding / cutting off the ducts. All ducts were blown out and all debris was removed.

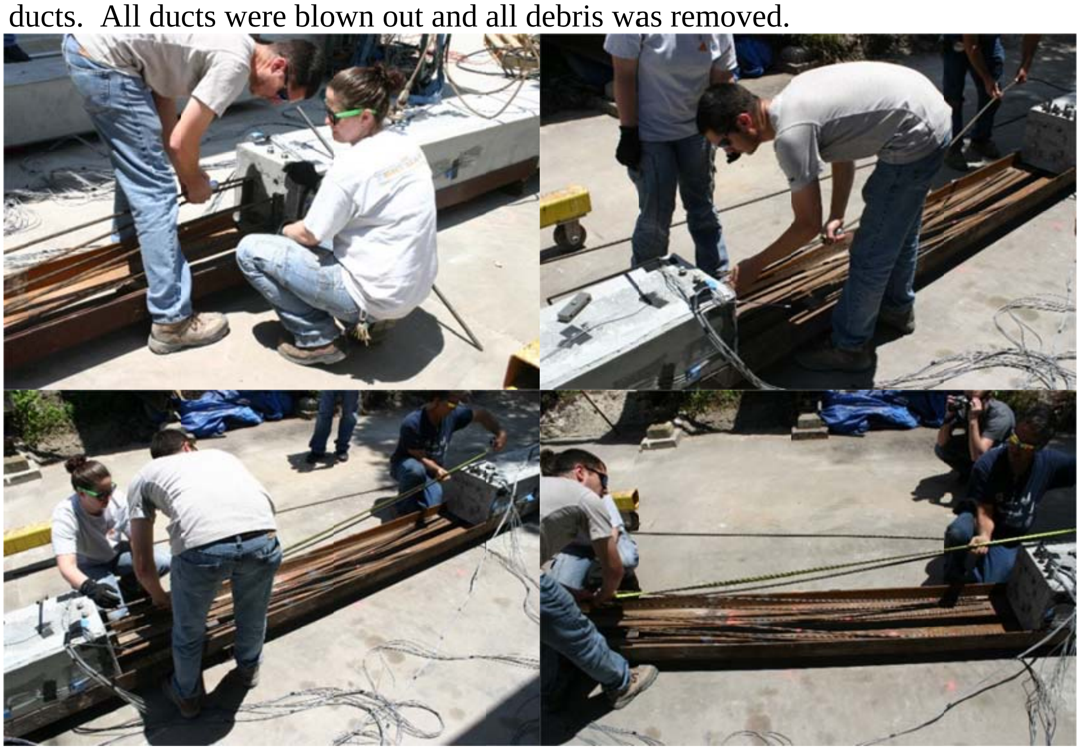


Figure 4.90. Threading strands through upper pile segment. Each strand measured to be ½ in longer than the previous (shortest strand stubbed out 96 in; longest 99.5 in).

Figure 4.91. All strands in upper pile segment and clamped to prevent inward slippage.

Figure 4.92. Lower pile segment aligned with upper segment on splicing rail.

Figure 4.93. Both pile segments on assembly/alignment rail. Upper segment top; lower segment bottom. The paint stripes on strands indicate when each strand will come in contact with the embedded anchorage in bottom pile segment.

Figure 4.94. Aligning first of 8 strands as pile segments are pushed together.

Figure 4.95. Weight of moving segment held by overhead crane while fork lift pushed the segments together and while each strand was sequentially inserted into the lower segment ducts.

Figure 4.96. Placement of threaded isolation sleeves on strands before insertion into south ducts. Sleeves are designed to prevent epoxy from filling grouting ducts during final contact.

Figure 4.97. All strands inserted. Flexibility of the 8 ft strands allows additional options to insert strands at will even if all strands were encountered at the same time. However, staggering also meant that no two anchorages were encountered at the same time.

Figure 4.98. Installation of isolation sleeves into bottom pile segment duct ends

Figure 4.99. All strands embedded past anchorage wedges (lower pile left).

Figure 4.100. With all strands in, strand clamps need to be removed (upper segment right).

Figure 4.101. QPL-approved epoxy applied to both sides per manufacturer's recommendations.

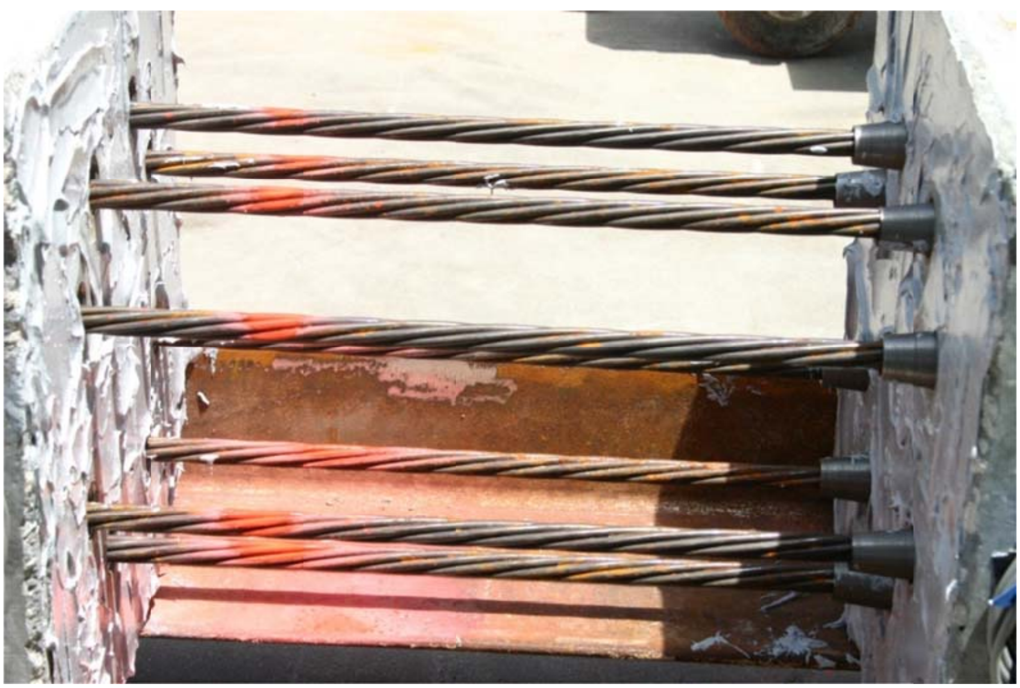


Figure 4.102. Strand clamps removed from upper segment (left), epoxy applied to both sides, ready for final assembly. Note no movement of strands into upper segment during assembly. The more restrictive wedge assemblies in upper segment (from double washer inserts) provide more resistance to sliding and therefore all movement from this point forward is into the lower pile segment (to the right).

Figure 4.103. Final joining and stressing the splice connection.

The stressing pattern was the same as the first pile specimen but where the load was imposed in two stages instead of three (Figure 4.104 and 4.105). Recall, the rationale for staging the load is to progressively compress the concrete and to remove a substantial portion of the elastic shortening and associated losses in the first strand stressed. This is especially important given the short splicing strand length.

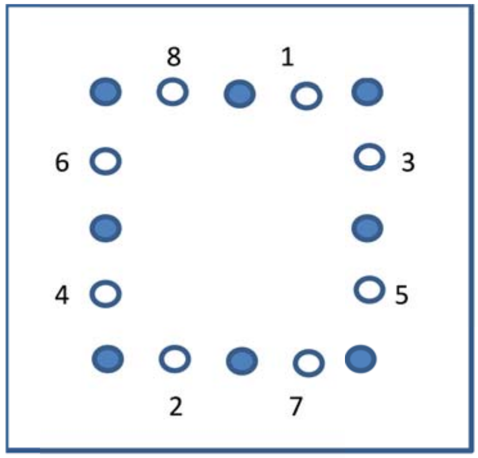


Figure 4.104. Strand stressing order of matched pairs (open circles denote splice strands; closed circles denote existing pile strands).

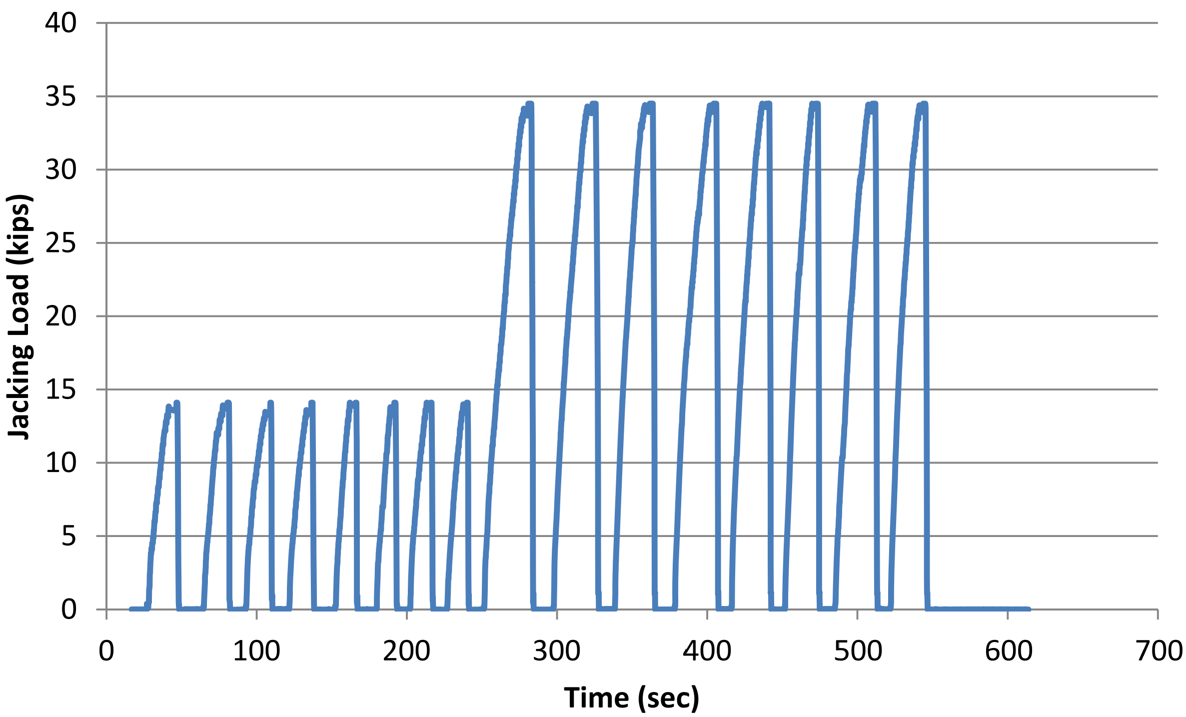


Figure 4.105. Two stages of jacking loads applied to the eight splicing strands.

4.3 Four-Point Bending Tests

While pile splicing was performed 14 and 10 days after casting for prototype pile specimens one and two, respectively, the bending tests were performed over a two day period that corresponded to 90 and 56 days after casting the piles, again respectively. Compressive strength of the concrete and grout strength were monitored throughout the casting, splicing and bending test timeline. These values are shown in Table 4.6 and Figure 4.106.

Table 4.6. Compressive strengths of concrete and cable grout along with timeline of events.

		1st Specimen Set		2nd Specimen Set		Cable Grout	
		Concrete Age (days)	Strength (psi)	Concrete Age (days)	Strength (psi)	Age (days)	Strength (psi)
11-Mar	1st casting	0	0				
14-Mar	1st transfer	3	6396				
25-Mar	1st splice	14	7790				
15-Apr	2nd casting			0	0		
18-Apr	2nd transfer			3	5836		
25-Apr	2nd splice			10	7699		
12-May	Grouting (1 & 2)					0	0
15-May						3	2934
19-May						7	3377
9-Jun	1st Bend test						
10-Jun	2nd Bend test	91	9377	56	9104		
17-Jun						36	4820

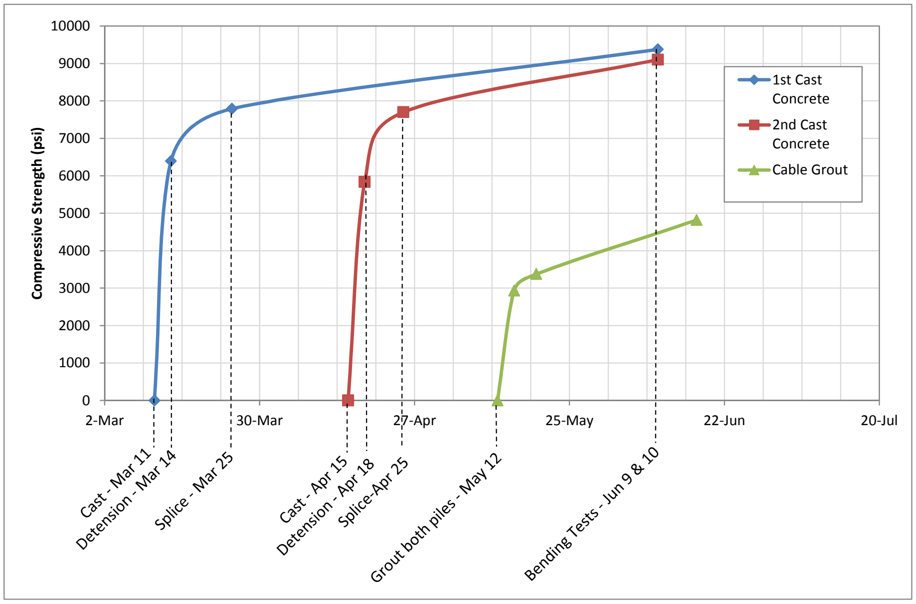


Figure 4.106. Compressive strengths of concrete and cable grout.

4.3.1 Bending Test Setup

Bending tests were performed using a four-point loading configuration (Figure 4.107) where a spreader beam was used to split the load from a single hydraulic jack through a hemi-spherical bearing to two loads 2ft off the midspan location. Surface mounted strain gages were used to verify the two loading points produced the same force at all times.

The flexural experiments were conducted at the Structures Lab of University of South Florida. These tests included two splice specimens and two control specimens. The dimensions of the experimental setup are shown in Figure 4.107.

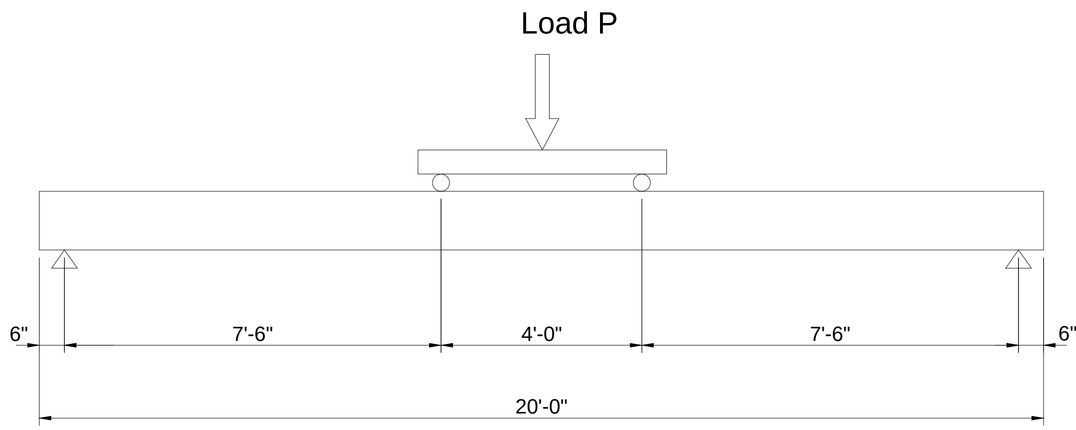


Figure 4.107. Dimensions of the Flexural Experiment

Three LVDTs were positioned as shown in Figure 4.108:

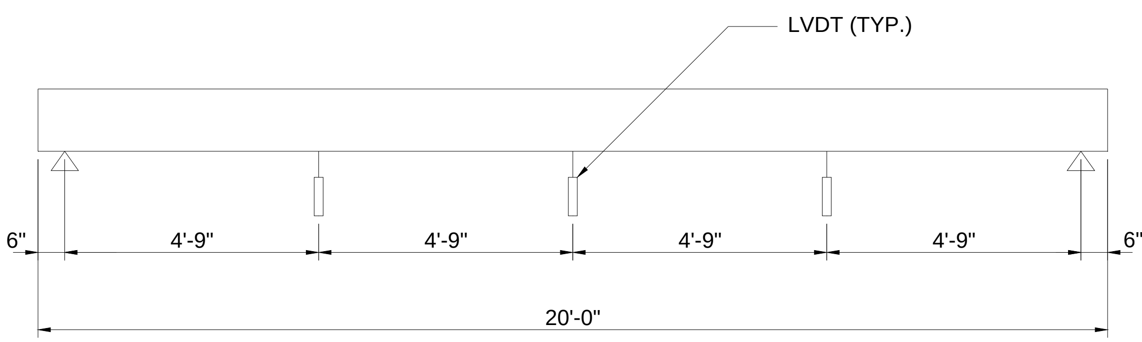


Figure 4.108. Layout of LVDTs

One LVDT was located at the center of the specimen and the other two were located at the quarter positions. The finished experimental setup is shown in Figure 4.109. The specimen was supported on roller supports at each end. The positions of the supports were carefully adjusted to ensure correct alignment of the loading. The roller support is shown in Figure 4.110.

Figure 4.109. Four-point bending set up (control top; splice bottom).

Figure 4.110. Roller support

For control piles, the strain gage layout is shown in Figure 4.111.

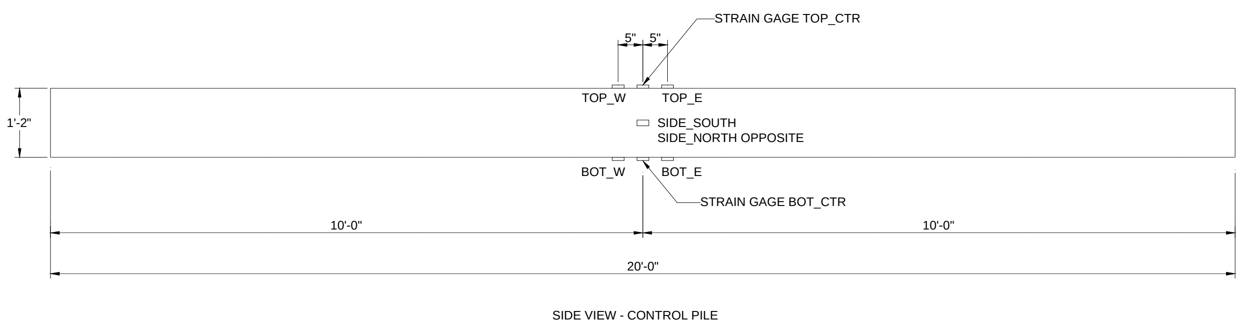


Figure 4.111. Strain Gage Layout for Control Specimens

For spliced specimen, the top view of the strain gage layout is shown in Figure 4.112.

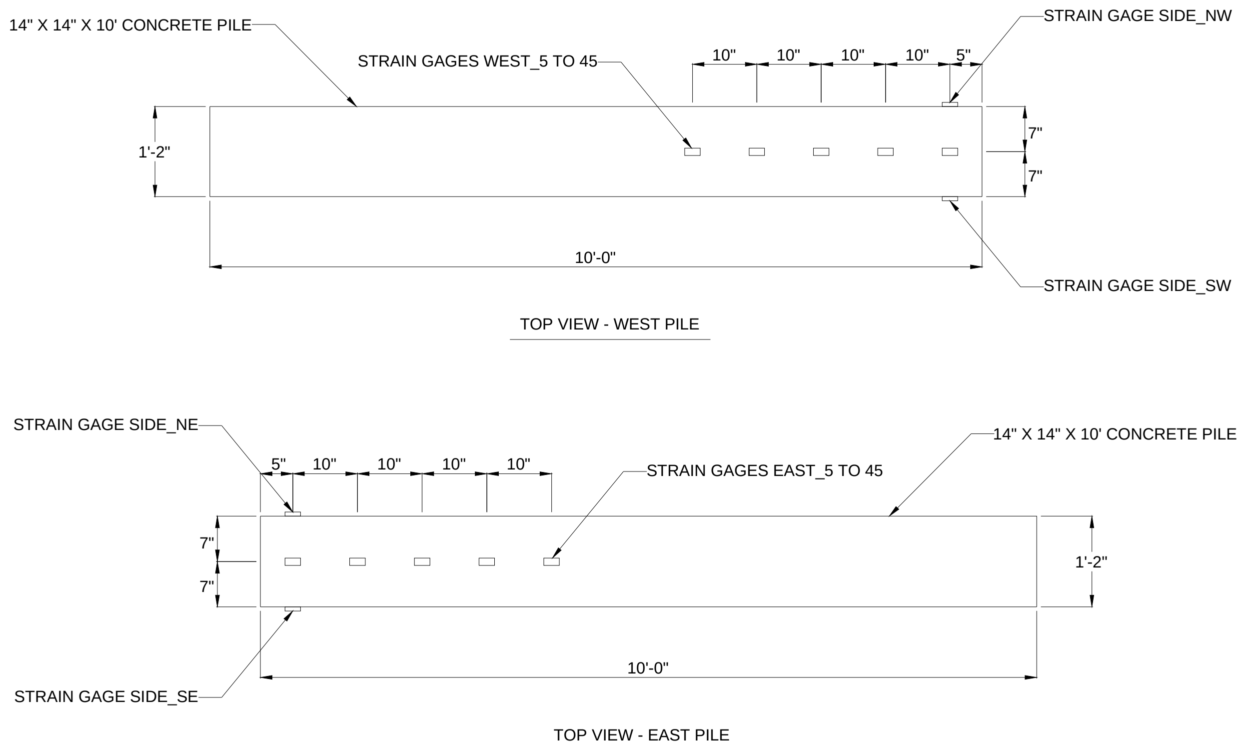


Figure 4.112. Strain Gage Layout Top View for Spliced Specimens

The side view of strain gage layout for spliced specimen is shown in Figure 4.113.

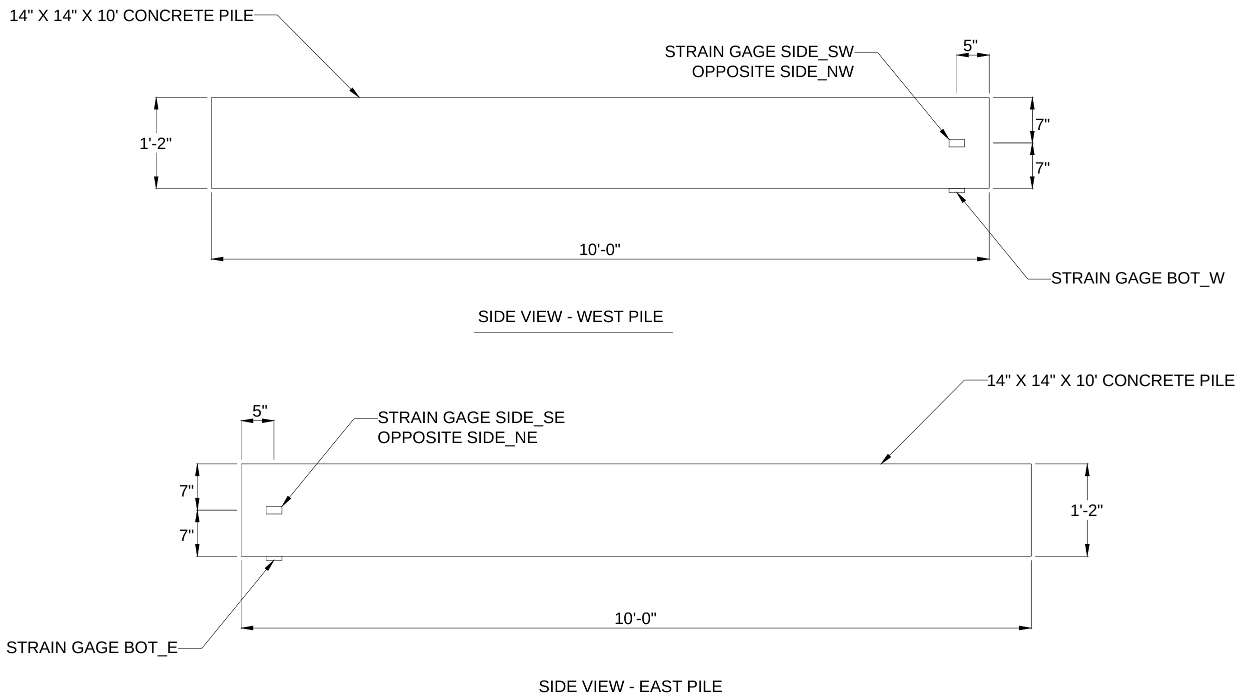


Figure 4.113. Strain Gage Layout Side View for Spliced Specimens

In the side view, top strain gages were not shown for clarity.

Figures 4.114-4.117 show the four piles both before and after failure and in order of testing: Control 1, Splice 1, Control 2, and Splice 2.

Figure 4.114. Control Pile 1 before and after testing. Compression block extends below spiral level (approx. 5in from top). Failure load 39kips.

Figure 4.115. Splice Pile 1 before and after testing. Compression block defined by spiral location. In this region, the splice pile has spiral confinement on 1” pitch; control pile is on 6” pitch. Pile rebounded almost fully. Failure load 33kips.

Figure 4.116. Control Pile 2 before and after testing. Again compression block well below spiral level. Failure load 39kips.

Figure 4.117. Splice Pile 2 before and after testing. Again compression block only in cover. Failure 36kips.

4.3.2 Four-Point Bending Test Results

The load was applied incrementally until pile failure occurred. All test data were recorded via computer for full analysis. The specimens experienced concrete crushing at the top and exhibited several flexural cracks which propagated up from the bottom in the constant moment region between the spreader beam supports. The longest flexural crack was about 8~9 inches. Figure 4.118 shows the failure pattern in Control Pile 1. The load versus deflection response is presented in Figure 4.119.

Figure 4.118. Specimen failure pattern (Control Pile 1 also shown in Figure 4.114)

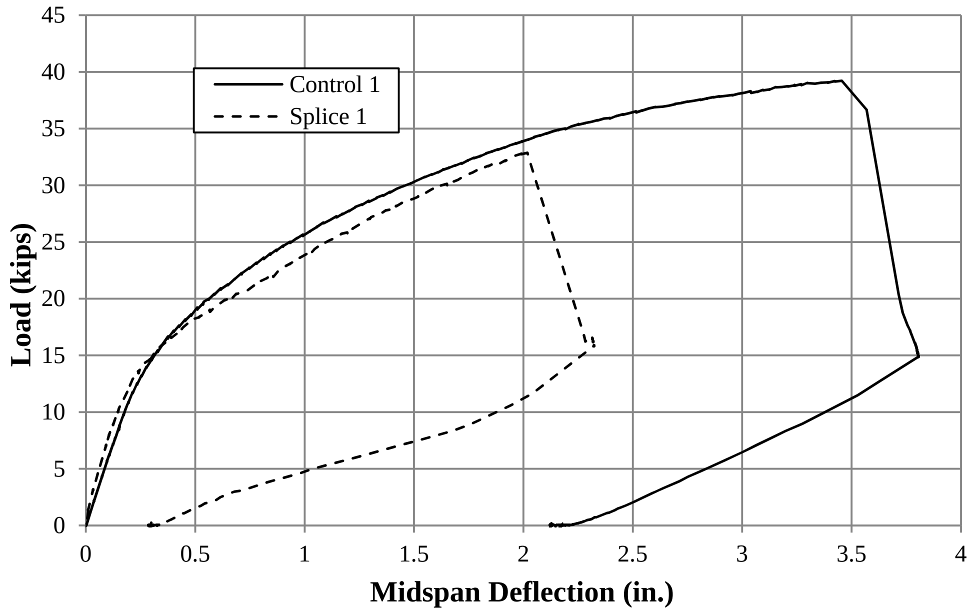


Figure 4.119. Load vs Mid-Span Deflection for Test 1.

After initial failure, the load was fully removed and the splice pile rebounded from 2.25in to 0.25in while the control pile showed 2.25in of permanent deformation.

The load versus deflection diagram for experiment series 2 is shown in Figure 4.120.

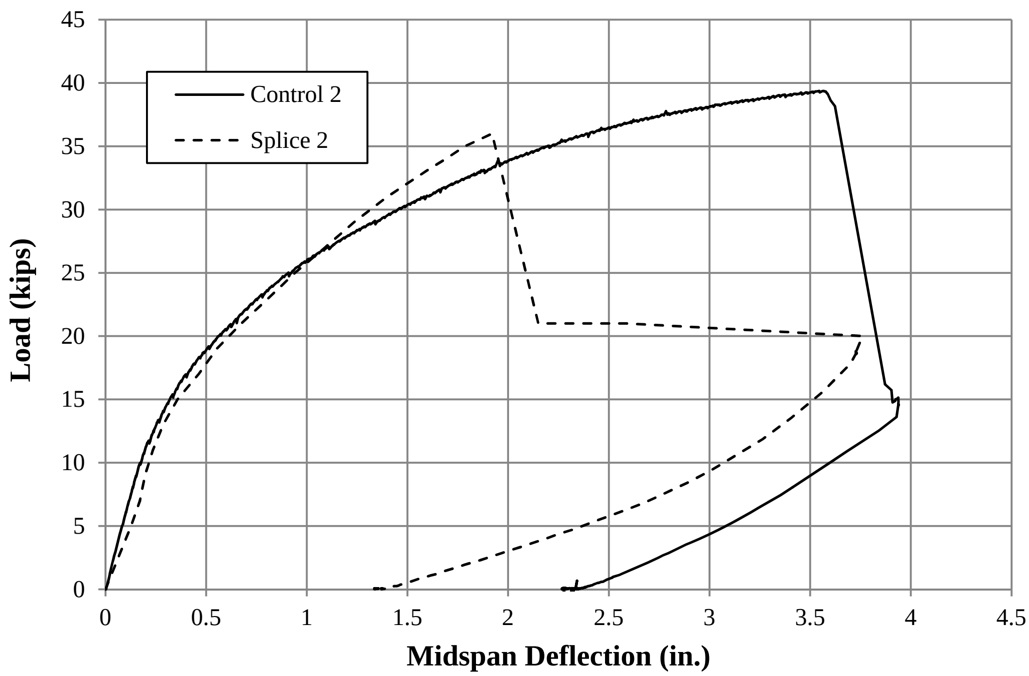


Figure 4.120. Load vs Mid-Span Deflection for Test 2

After initial failure, the splice pile was loaded further to demonstrate ductility but with no additional capacity. In essence, a fulcrum point formed at the top of the reinforcing cage (cover was now missing) and the splice crack opened to that height; with the strands yielding; no additional moment could be developed. However, even after displacing it to the same level as the control pile, the rebound was over an inch more than the control pile. This can be attributed to the crack formation along the splice line which could close more easily than the irregular cracking of the control piles.

In general, the spliced specimens followed the same behavior as that of the control specimens until the failure occurred. The average of the failure load of the control specimens was 39.3 kips and the average failure load of the spliced specimens was 34.4 kips. The second pile was thought to have performed better due to better bond (deformed ducts) and more development length behind the anchorages. Given the variation in strand pattern between the control and splice regions (i.e. 3 prestress strands in bottom layer versus 2 post tensioning strands in bottom layer), the results were as expected. Cracking loads were similar for all piles: average control pile cracking load was 10.9kips; splice piles 10.8kips.

While the spliced piles could not achieve the same moment due to strand layout, the stiffness of the second prototype splice pile was notably stiffer than either control or the first prototype. The only difference was the duct deformations that more effectively transferred the load from the splice strands into the concrete immediately surrounding the splice region. This engaged the unstressed strands at the ends of each pile segment. In essence, the sections on either side of the splice contained twice as much strand steel as the controls and locally did not take on the same level of curvature. The undeformed ducts from the first prototype could not effectively engage the strands and had no advantage.

4.4 Strand Pull-out Tests

While the exact advantage of deforming the ducts was not known at the time of the four-point bending tests, it was speculated to have a positive benefit. To this end, the second control pile was cast with four empty ducts on one end deformed in different configurations. All ducts were 5ft long, one was undeformed, two were deformed on 2in spacing, and one was deformed on 1in spacing. Figure 4.121 shows the duct assemblies connected to a grouting manifold in the casting bed.

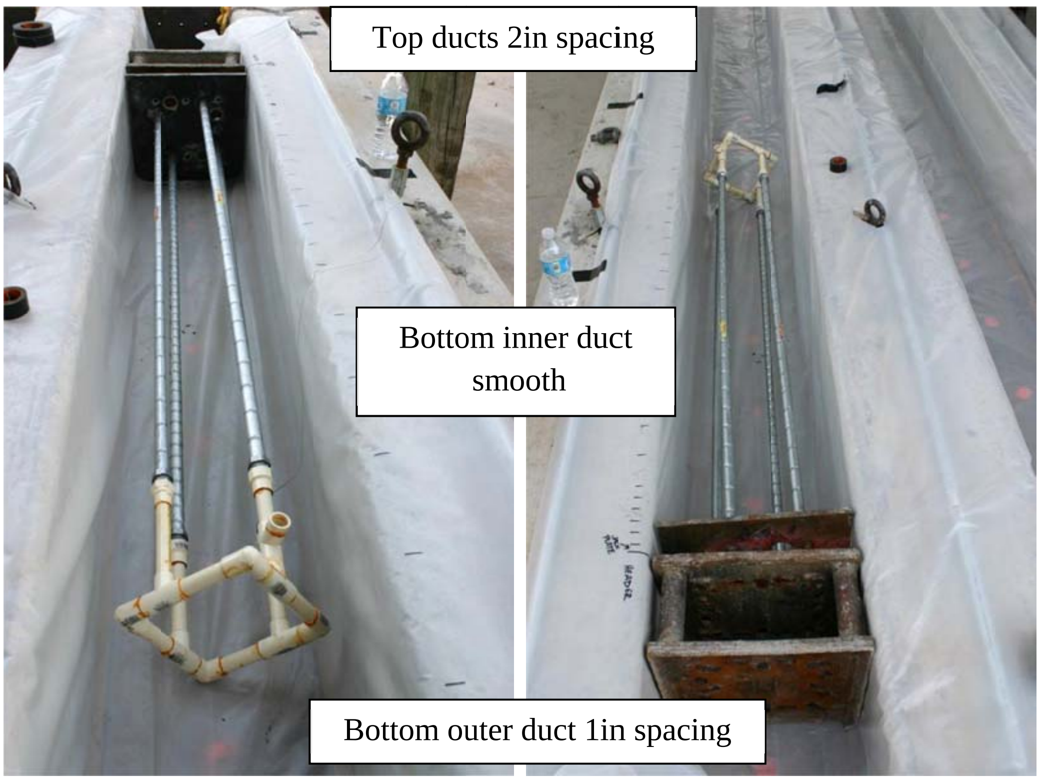


Figure 4.121. Empty ducts cast into control pile for pull-out tests.

At the conclusion of the four point bending tests, 6ft of the end of the second control containing the embedded ducts was cut off and held vertically simulating the top of a spliced pile (duct openings aiming up). Strands were inserted from the top 40in except for one of the ducts with 2in spacing deformation was only inserted 30in. A clamp was used to maintain the embedment length. Figures 4.122 and 4.123 show the process of grouting the pull-out test strands in place.

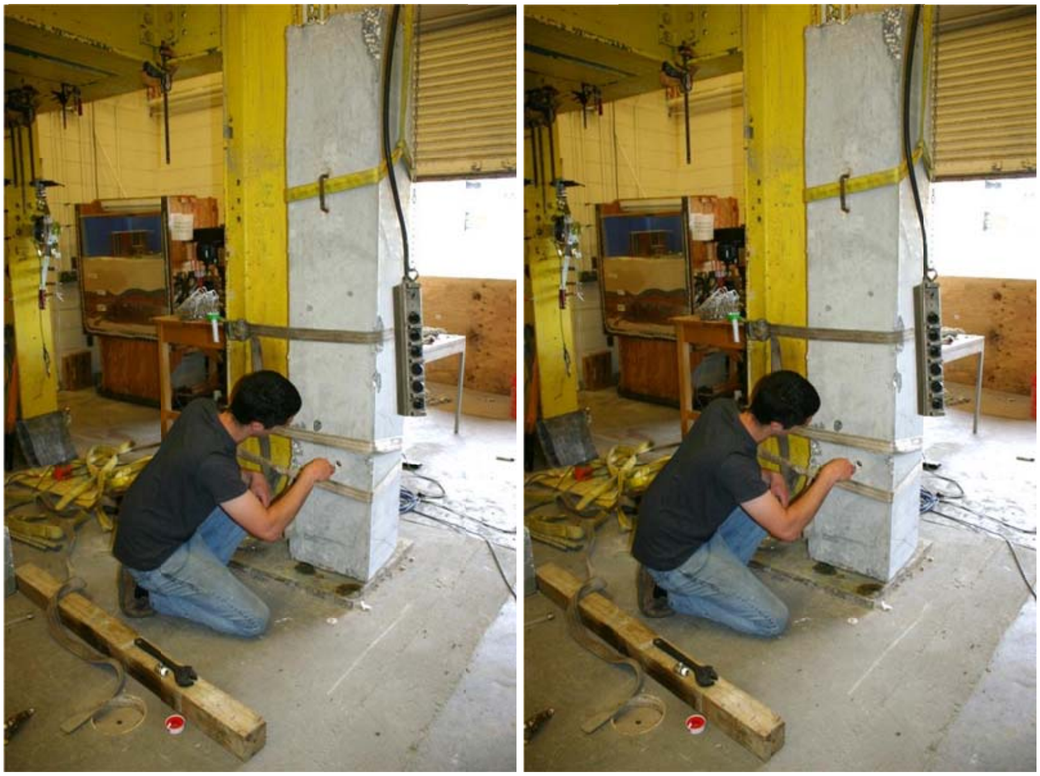


Figure 4.122. Removing casting plug and attached grout pressure pot.

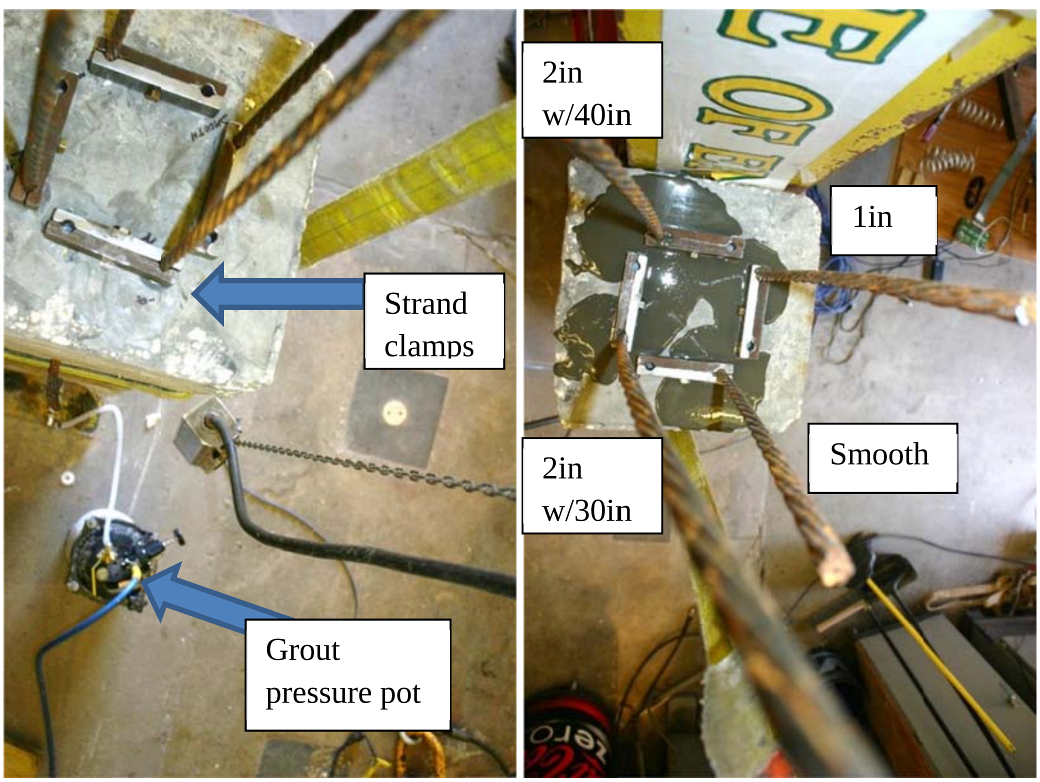


Figure 4.123. Grout pumped in bottom of the duct assemblies and out the top.

Test results from the pullout tests showed significant increase in bond/development capability from the deformed ducts where all strands in the deformed ducts yielded and failed. The strand in the smooth duct, pulled out cleanly with as much as 87% reduction in capacity (Figure 2.124).

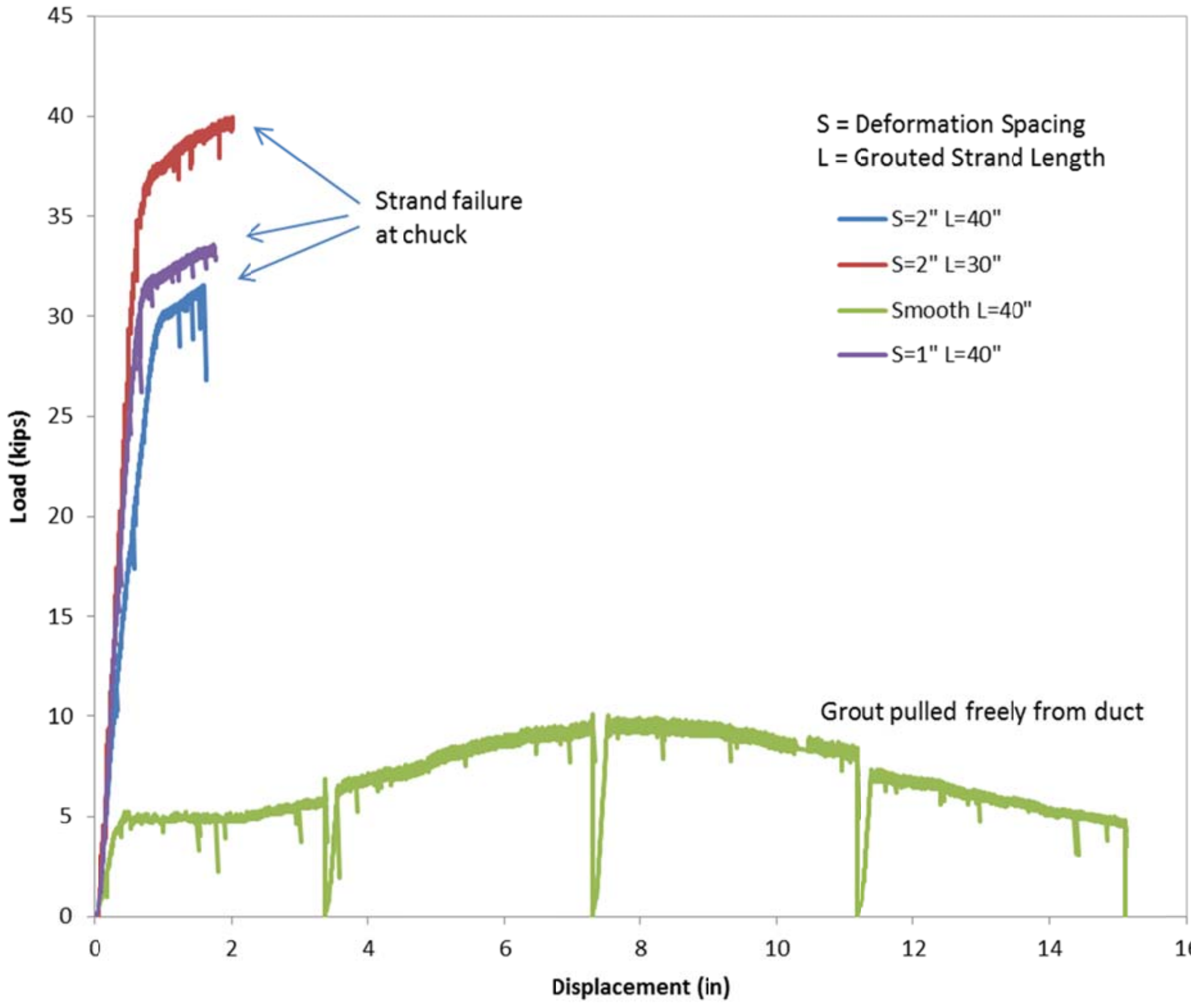


Figure 4.124. Pullout capacity of the embedded strands.

Figure 4.125 shows the 3 failed strands and one from the smooth duct after it was extracted 15in. The grout is shown as a smooth grey encasing around the strand.

Figure 4.125. Strand pullout specimens: 1in x 40in (back); 2in x 40in (left); smooth (right); and2in x 30in (front).

4.5 Lessons Learned

Each splice pile tested to date provided usable feedback to further the advancement of the overall splice concept.

- The 14in pile sets gave rationale for using deformed ducts wherein the smooth duct splice pile was less stiff than that with the deformed ducts. The deformed duct pile also showed a stiffer response than the unspliced control.
- Duct deformations using a 2in spacing performed as well if not better than 1in spacing; use 2in spacing for deformations.
- Grouting port can be optionally removed from the side of pile to the top of pile to enhance corrosion resistance.
- Wedge set losses can be minimized by inserting a tolerance reducing shims behind the wedges.
- Ducts can be grouped into more convenient panels for quicker field installation.

Chapter 5: Full Scale Testing (24in piles)

Even though 14in piles are used commercially, they are rarely if ever used for FDOT projects. In this study the 14in piles were considered lab-scale specimens. Larger 18, 24 and 30in piles are more commonly used and for this study 24in piles were cast to demonstrate full scale applicability of the concept pile splice.

Three 24in square, 40ft long piles were cast for the purposes of demonstrating the bending capacity of the splicing system. One pile was spliced from two 20ft segments and the other two were cast full length as control piles. The only variation in the control piles was to increase the spiral spacing near the midpoint of the pile which would mimic the spiral spacing of the spliced pile (discussed later).

5.1 Fabrication of Components

Much of the same steps were taken in the preparation for the 24in full scale pile casting as that discussed earlier for the 14in specimens. The preparations included fabrication of: chuck assemblies, confinement coils, deformed ducts, splice header, and grout manifolds. The bearing plates welded to the chucks are shown in Figure 5.1, confinement coils attached to the bearing plate and duct in Figure 5.2 and machining of the splice header in Figure 5.3. In all, 40 chuck assemblies were needed for the 20 splicing strands configuration. Therein, twenty 1/2in special strands were used for both the pile and splice.

Figure 5.1. Chucks welded to bearing plates.

Figure 5.2. Forty confinement coil, chuck and duct assemblies.

Figure 5.3. CNC machining of header plates.

Eight, 2in diameter solid steel spacer rods were cut to 12in lengths and faced to be both perfectly flat and precisely the same length. This was an increase in length from the 14in piles which were found to be too tight to easily access the duct fastening bolts. The pattern for the duct layout and strands was obtained from the casting yard drawings which were in turned based on FDOT standard specifications for precast piles (Appendix B). After the individual plates were drilled with the strand and duct pattern, the plates were machine to ensure the all surfaces were perfectly parallel. The finished header plate assembly is shown in Figure 5.4; the strand and duct pattern is shown in Figure 5.5.

Figure 5.4. Finished header plate assembly.

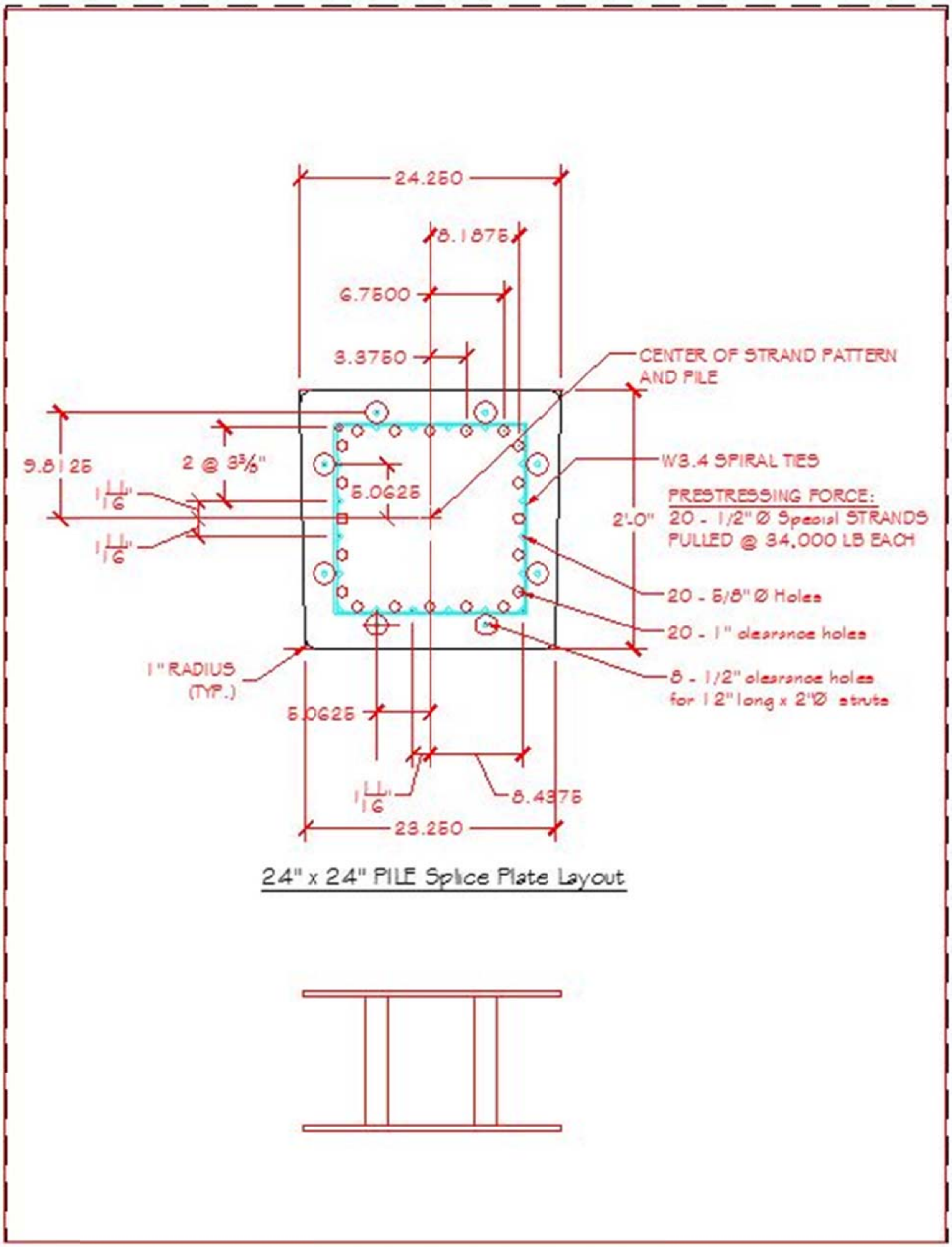


Figure 5.5. Header plate schematic showing strand and duct pattern.

5.2 Casting of 24-inch Pile Specimens

The 24in pile specimens were cast at CDS Manufacturing Inc. in Gretna, Florida. There were in total three 40ft specimens including two control piles and one spliced pile (two 20ft segments) cast. The prefabricated internal components of the splicing system were transported to the Gretna casting plant. In the earlier 14in piles, each duct assembly was installed and tied separately to the spirals. With only 8 ducts, it was reasonably achievable, but with 20 duct assemblies an alternate approach was taken to minimize confusion from 20 loose ducts. As each side of the pile was slated for 5 ducts, the 20 overall required ducts were grouped in panels of 5 representing that required for each of the four sides of the splicing reinforcing pattern. The predetermined duct spacing was set based on the strand configuration of the bed (Figure 5.5). Figure 5.6 shows the internal components as delivered to the casting yard. Duct extensions were installed separately (also shown). All end caps were taped shut to prevent inadvertent introduction of foreign objects.

Figure 5.6. Internal components of splicing system.

Additionally, the anchorage locations were staggered into five steps which coincided with the five ducts on each side of the pile. Therefore, all panels of five were interchangeable. Figure 5.7 shows one panel being taken over to the bed.

Casting the splice pile followed standard procedures where the splice header was inserted in the bed like any normal pile separation header (Figure 5.8) and strands were progressively installed in order from bottom to top prior to installing ducts and pulling the spiral reinforcement.

Figure 5.7. Five-duct, interchangeable panel complete with staggered anchorages and deformed ducts.

Installation of the duct assemblies required that all long lengths of duct extensions and duct panels (groups of 5) be placed after the strands were tensioned but before the spirals were pulled to the ends of the pile. Two top strands were left slack to ease installation and tensioned only after all splicing fixtures were installed. While the panel concept did preset and maintain the proper spacing, it also introduced space restrictions and did not provide enough flexibility to easily manipulate the panels into proper position. This was due to the rigidity of the cross straps welded to the ducts. An alternate panel concept was devised for future applications using more flexible sheet metal straps/spacers. Figure 5.8 shows the finished splicing system in cast bed. Figures 5.9 and 5.10 show the finished bed.

Figure 5.8. Strands threaded through header plates in casting bed.

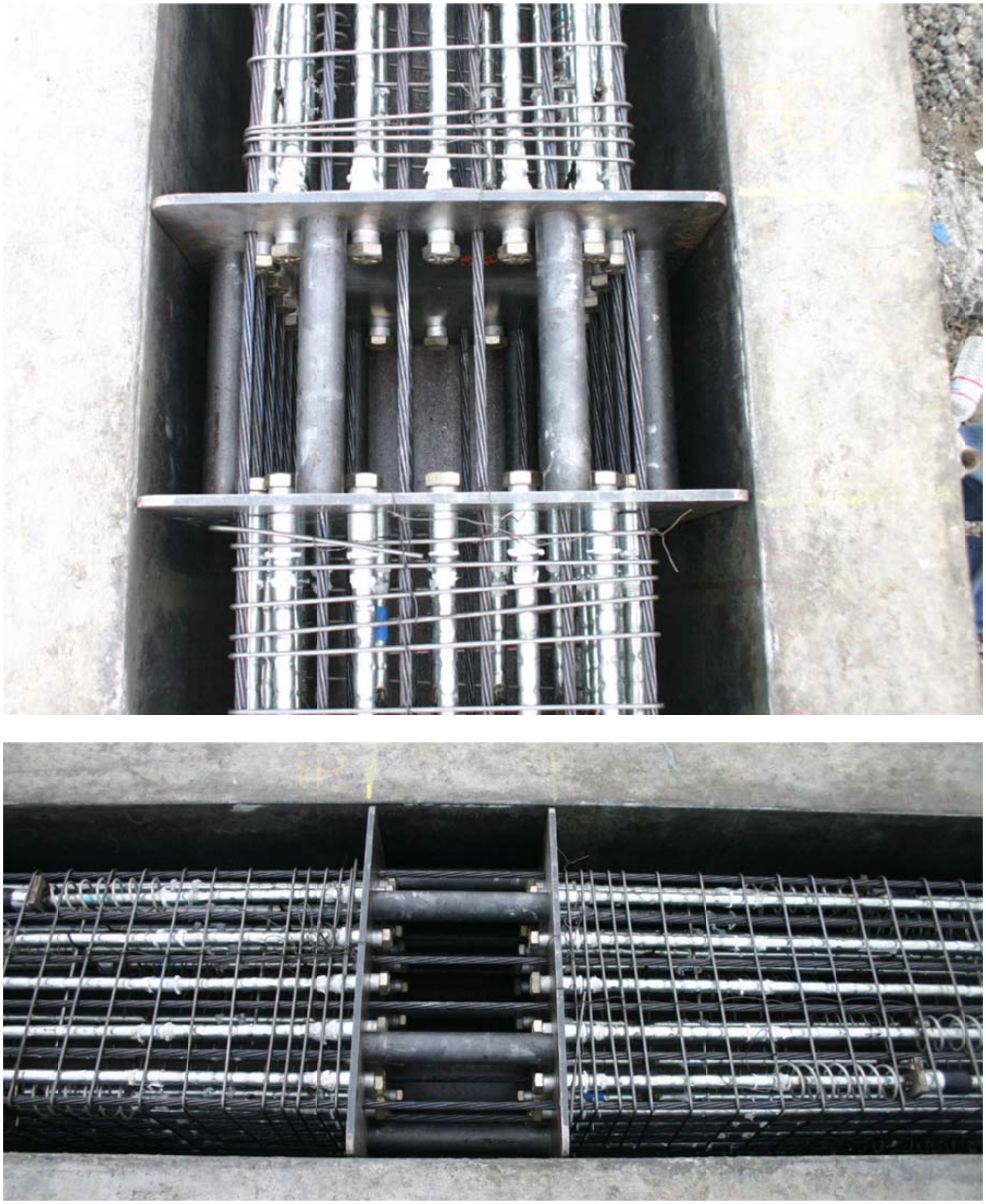


Figure 5.9. Splicing system in place.

Figure 5.10. Fully stressed bed with lifting loops in-place (note staggered anchorages complete with confinement coils).

The spirals in the control piles had two layouts identified by specimen identifications C-1 or C-2. The C-1 control had a spiral layout identical to the spliced pile, i.e., 5 turns @ 1in followed by 16 turns @ 3in at the ends. At the mid-span splice location there were 10 turns @ 1in followed by 32 turns @ 3in symmetrically placed about the mid-span (16 turn on each side of center). The remaining portion of pile had a 6in pitch. This layout is shown in Figure 5.11.

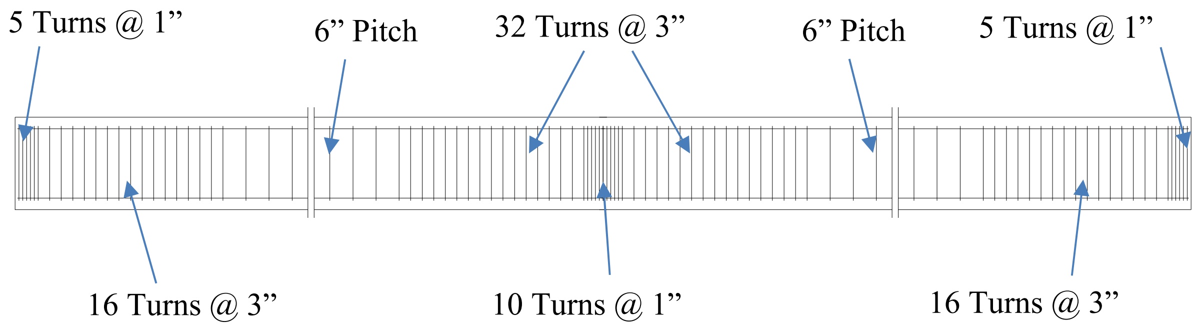


Figure 5.11. C-1 control pile spiral layout (same as spliced pile).

The C-2 specimen used a traditional layout consisting of 5 turns @ 1in CTC spacing followed by 16 turns @ 3in at each end. The remaining portion of the pile had a 6in spiral pitch. This layout is shown in Figure 5.12.

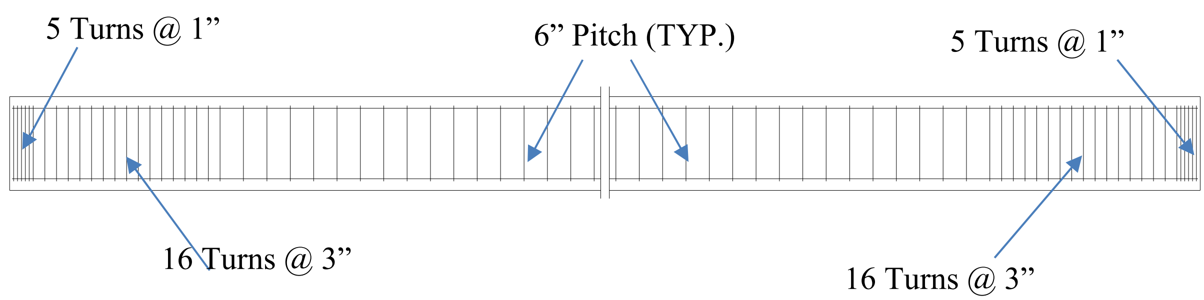


Figure 5.12. C-2 control pile spiral layout (same as FDOT standard specifications).

Figure 5.13 shows concrete placement of the 24in test specimens. All test piles were cast in the same bed, with the same concrete and prestress levels. Concrete test cylinders were made for compressive strength tests which were conducted at the Structures Lab at the University of South Florida.

Figure 5.13. Concrete placement.

5.3 Splicing of the 24in Pile

The cured pile specimens were transported to FDOT Structures Research Center in Tallahassee and made ready for splicing.

5.3.1 Preparations for Splicing

During preparation work, all the ducts were inspected for debris with a borescope and spring movement of embedded wedges was checked. Splicing strands (approximately 33ft) were fed into the upper pile segment with exposed strands being clamped to ensure no slippage during assembly (Figure 5.14). Ten feet was left at the base of the upper pile segment and 3ft was exposed out the top of the upper pile segment. For convenience, the lower exposed strand lengths were staggered in one inch increments so only one strand at a time would come into contact with the lower pile segment. However, the flexibility of the long exposed lengths (10ft) permits manual bending and reinserting strands that miss the corresponding duct location.

Epoxy sealant was applied to the splicing surfaces prior to rolling the upper pile segment into close contact with the lower pile segment. Horizontal splicing somewhat complicates the procedure as alignment can only be adjusted via overhead crane and optical alignment. This is important as it removes needless bending stress at the splice interface while the individual splicing strands are stressed. Vertical splicing processes are less prone to these types of alignment induced stresses. In this case, a forklift was used to push the upper pile which was placed on rollers.

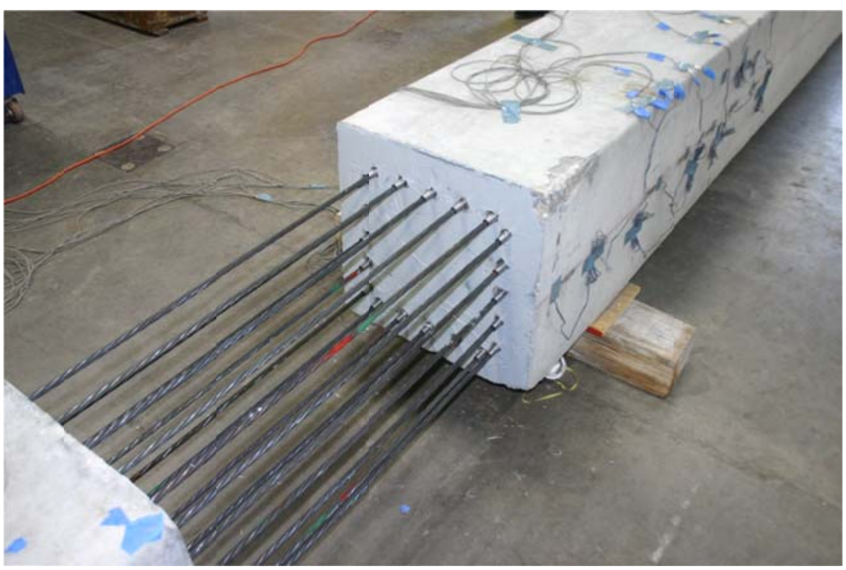


Figure 5.14. All strands inserted passed the lower pile wedges and segments were pushed together.

Figure 5.15 shows the final stages just before joining the splice connection. Strand clamps (not shown) were removed once the two pile segments were in close proximity and the proper length of strand had been pushed into the lower pile.

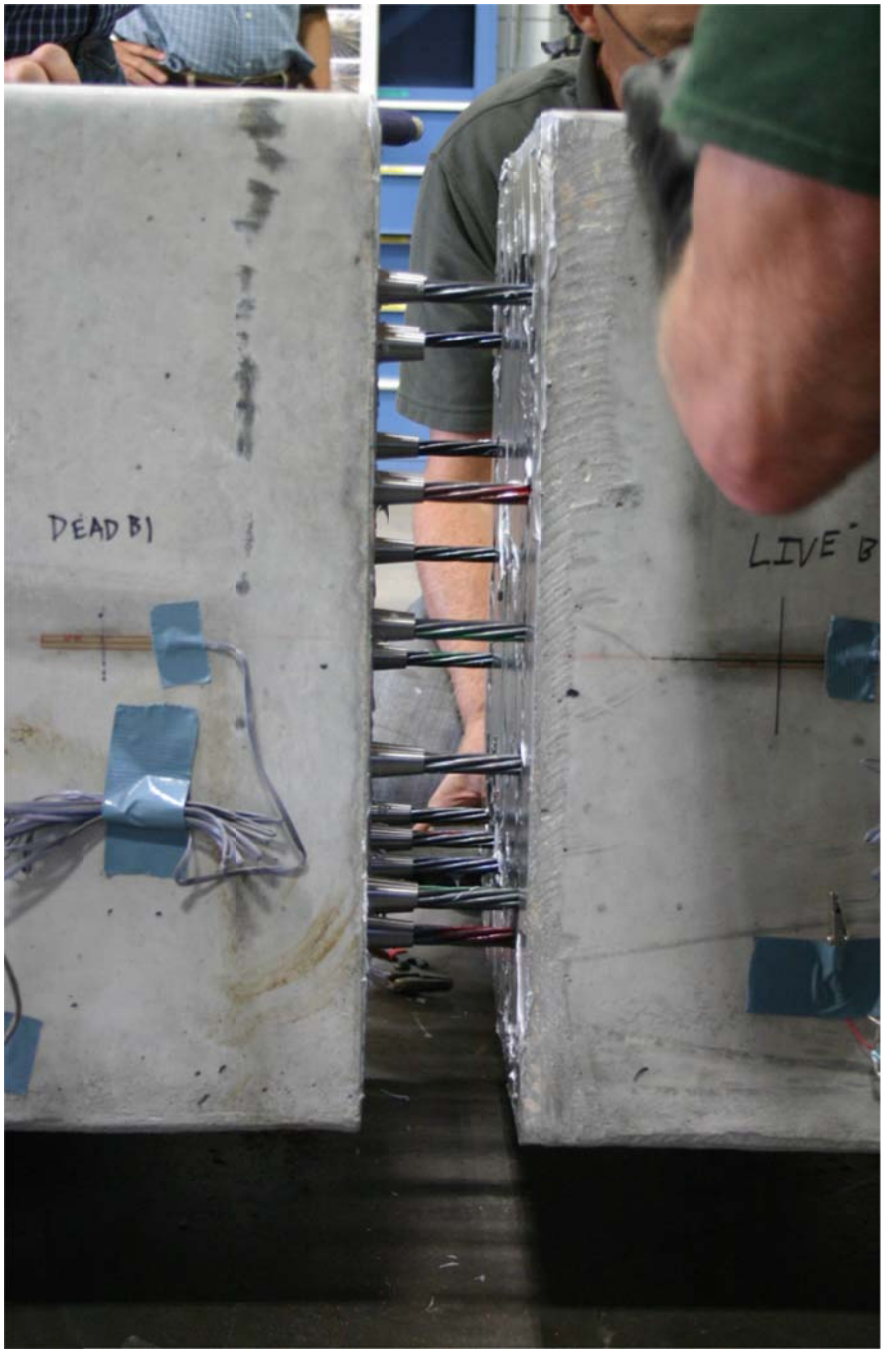


Figure 5.15. Final splicing stage removes the clamps prior to mating.

5.3.2 Post-Tensioning / Splicing

The post-tension stressing order was determined so that each pair of opposite strands were jacked to predetermined loads in three stages, which as noted earlier maintained balanced strain and reduced losses from elastic compression of the concrete. The splice stressing order of matched pairs was marked on to the pile face for convenience as shown in Figure 5.16. Figure 5.17 shows the stressing jack while loading.

Figure 5.16. Splice stressing order of matched pairs

Figure 5.17. Post tensioning of splicing strands.

Concrete strain was measured near the splice interface during post tensioning via six strain gages attached to either side of both pile segments (24 gages total). Gages were located along the mid-height / neutral axis of the pile at distances of 6, 27, 37, 47, 57, and 67in from the splice interface (Figures 5.18 and 5.19). Figure 5.18 shows the strain traces of data averaged from either side of the upper pile segment; Figure 5.19 shows the same data for the lower pile segment. The strain oscillation shown in the traces coincide with the applied jack loadings.

During the post tensioning, the strain gages closest to the interface registered lower than expected strain until about half way through the second pass of stressing. This was caused by the gradual expulsion of the fluid epoxy and indicated the pile was not in full contact throughout until that time. The possibility of strand relaxation associated with squeezing out the epoxy was remedied by performing a third pass of stressing to the same target level. Figure 5.20 shows the load cycles applied in three passes.

The strain data was converted to stress using an assumed concrete modulus based on concrete break strength. At the conclusion of the splicing process, the final stress distribution reflected the linear distribution created by staggering the anchors (five staggered anchorages shown in Figure 5.10). Figure 5.21 show the stress distribution.

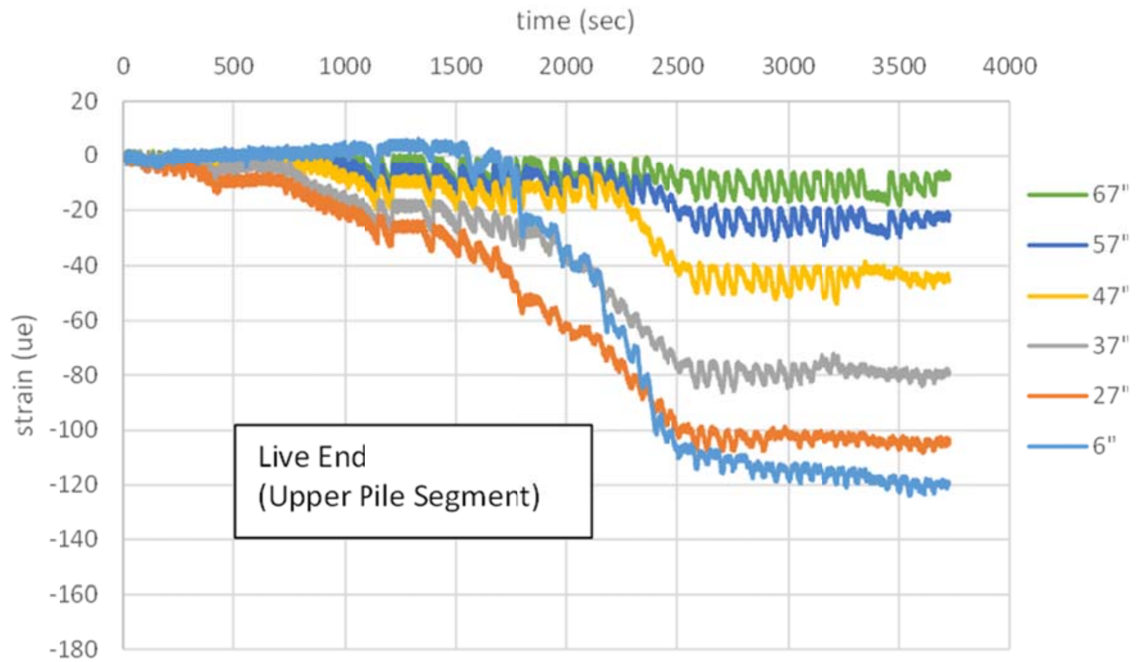


Figure 5.18. Average strain vs time for the upper pile segment.

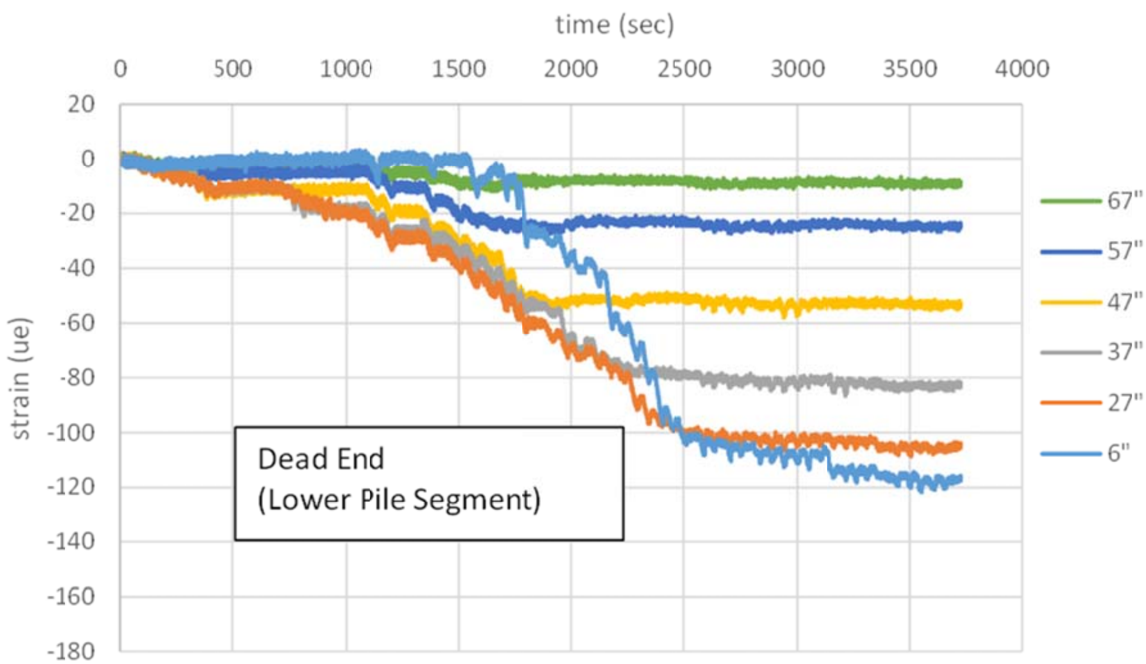


Figure 5.19. Average strain vs time for the lower pile segment.

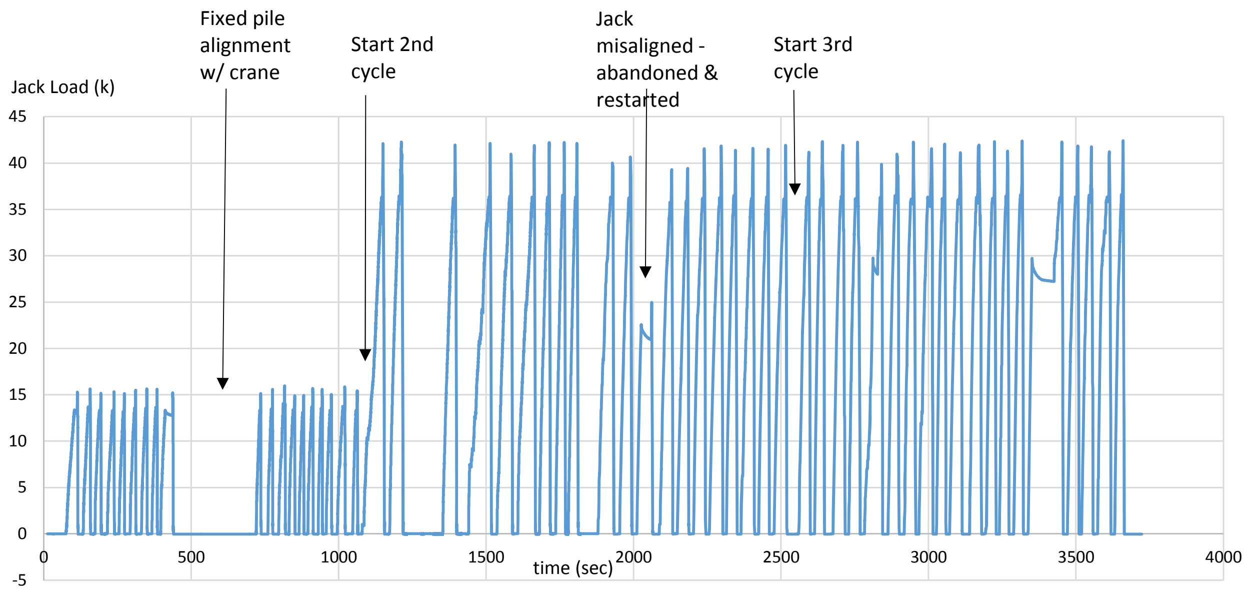


Figure 5.20. Timeline showing the stressing progression for three stressing passes.

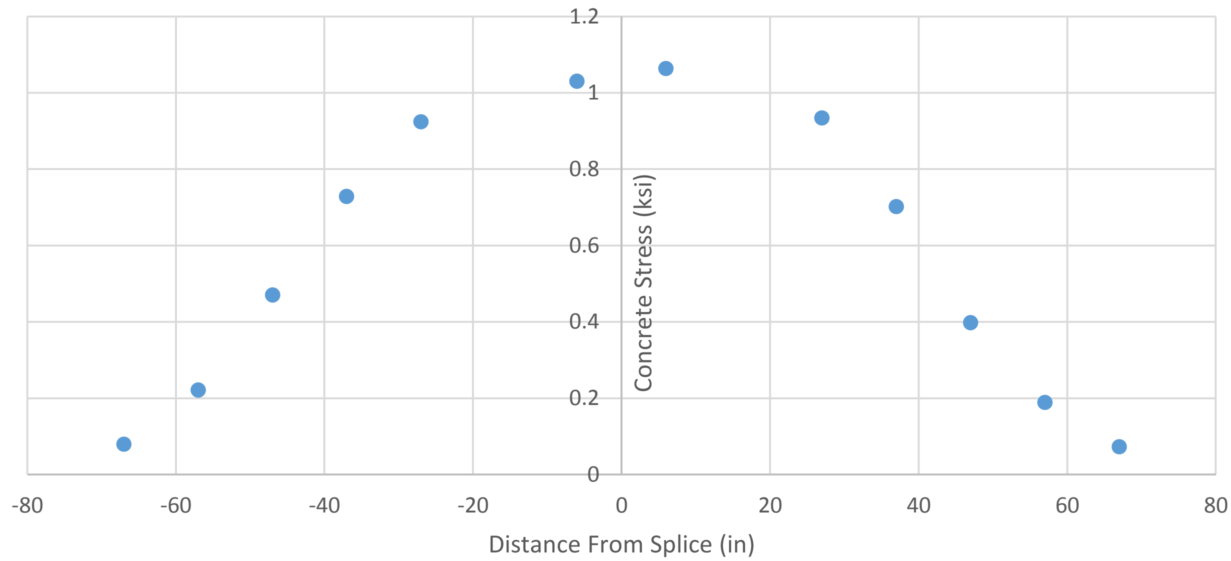


Figure 5.21. Stress distribution at the conclusion of splicing on both sides of the splice interface computed from average strains.

The strands extending from the jacking end (top of spliced pile) were cut which was necessary to permit unobstructed access to the threaded duct ends for grouting (Figure 5.22).

Figure 5.22. Cutting excess strand length before attaching grout fittings.

Each duct was fitted with a 3/4in pipe nipple. One duct from each side was additionally fitted with a ball valve through which the grout was pumped (Figure 5.23). A post tension tendon cable grout (product information provided in Appendix A) was pumped into each of the four side panels. Recall that two different grout plumbing configurations were used in the earlier 14in piles: one had a side-of-pile grout port which was connected to all 8 ducts, the other had all 8 ducts connected together but grouting was performed from the top (down one and back the other 7). In this case, each panel of 5 ducts (one panel each side) was piped together to provide 4 individual grouting circuits. This meant that grout could be injected into any one of the ducts and returned out the other four of that circuit.

Figure 5.23. Duct ends fitted with 3/4in pipe nipple and/or ball valve.

Grouting was performed using two different systems: an air over fluid grout pressure pot and a manually operated positive displacement grout pump. Three of the four panels were grouted using the pressure pot and one using the grout pump. Grouting was slow due to restrictions in the wedge assemblies. However, once a steady flow of grout was observed coming from a given duct, that duct was capped / sealed shut. Injection was then continued until all ducts on that circuit exhibited good grout return (Figure 5.24).

In order to decrease the grouting time, the third and fourth panels were grouted using pressure pot and grout pump simultaneously.

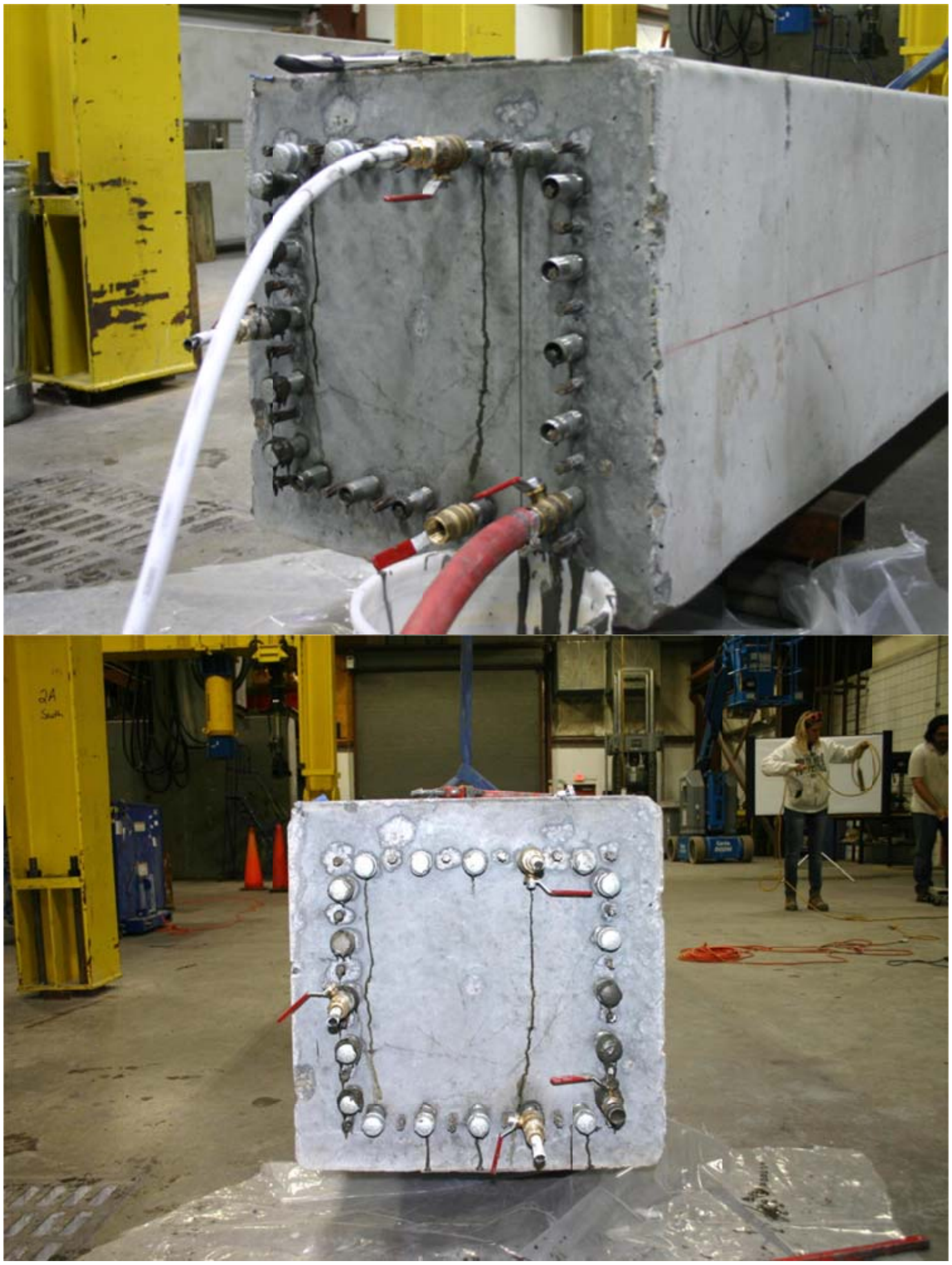


Figure 5.24. Grouting (top); grouting complete with all ducts capped or valves shut (bottom).

5.4 Flexural Testing

Two types of flexural testing were conducted to compare the performance of the splice system relative to unspliced control piles. The first set of tests used the full 40ft piles with a span of 38ft and two load points creating a constant bending moment within the center of the pile. This tested the effectiveness of the splice itself (compared to unspliced controls). Second, the upper portion of the splice pile (with double strand steel) was cut from the bottom half and tested in 3-point bending with a span of 18ft.

5.4.1 Four-Point Bending Tests

Four-point bending tests were conducted on November 19, 2014 at the FDOT Structures Research Center in Tallahassee to compare the performance of the splicing system relative to unspliced control specimens. The dimensions of the experimental setup were set in coordination with the FDOT / USF research team. Test setup scheme is shown in Figure 5.25.

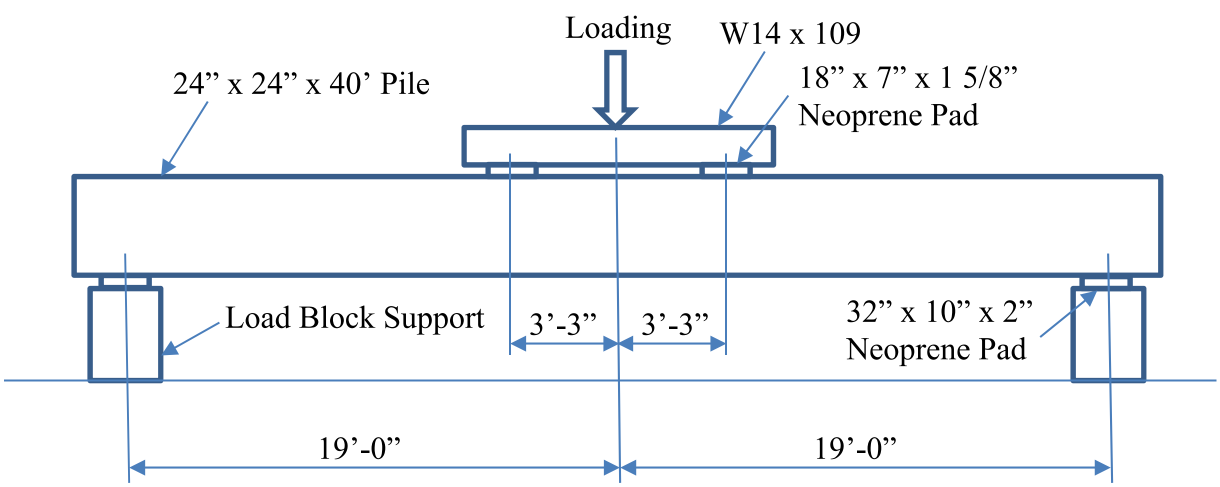


Figure 5.25. Four-point flexural test setup.

The spreader beam assembly used for all four-point bending tests is shown in Figure 5.26.

Figure 5.26. Spreader beam with neoprene loading points set at 6.5 ft.

The initial load rate for all tests was 250lb/s through the elastic portion of the test and trailed off with respect to the amount of damage induced into the specimen due to hydraulic flow rate shortfalls. The test data was recorded via computer for further analysis. The failure modes (Figure 5.27) included crushing of concrete on top and flexural cracks propagating from bottom. Visual inspection showed a stark difference in permanent deformation after failure where the splice pile showed less permanent deformation (Figures 5.27 and 5.28).

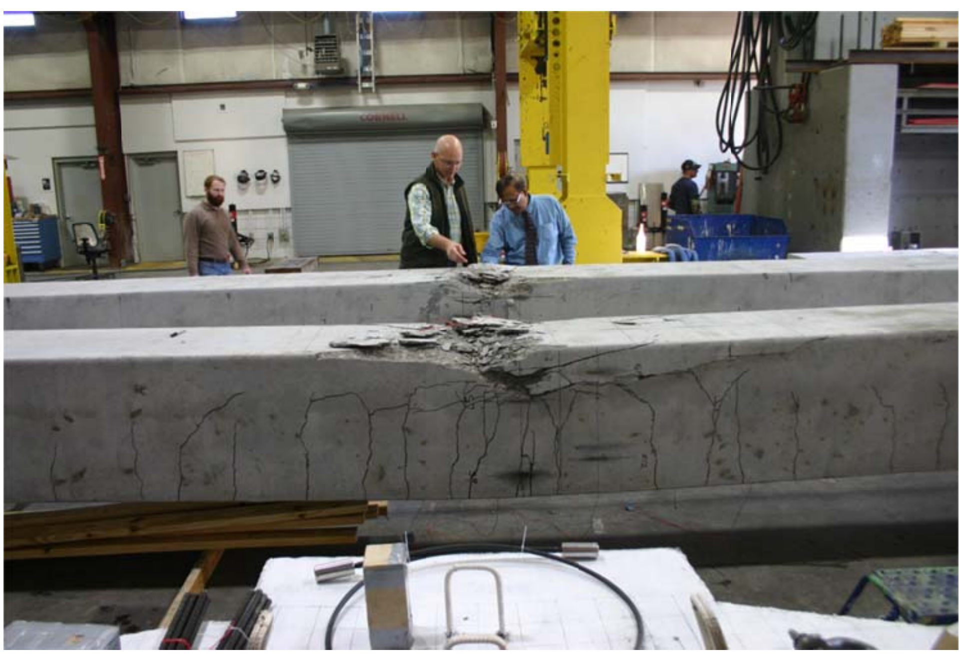


Figure 5.27. Control (foreground) and splice pile (back) failure patterns.

Figure 5.28. Smaller permanent deformation of spliced pile (right); control (left) after testing.

Diagonal cracks were noted that emanated from the base of the splice joint in both halves (Figure 5.29).

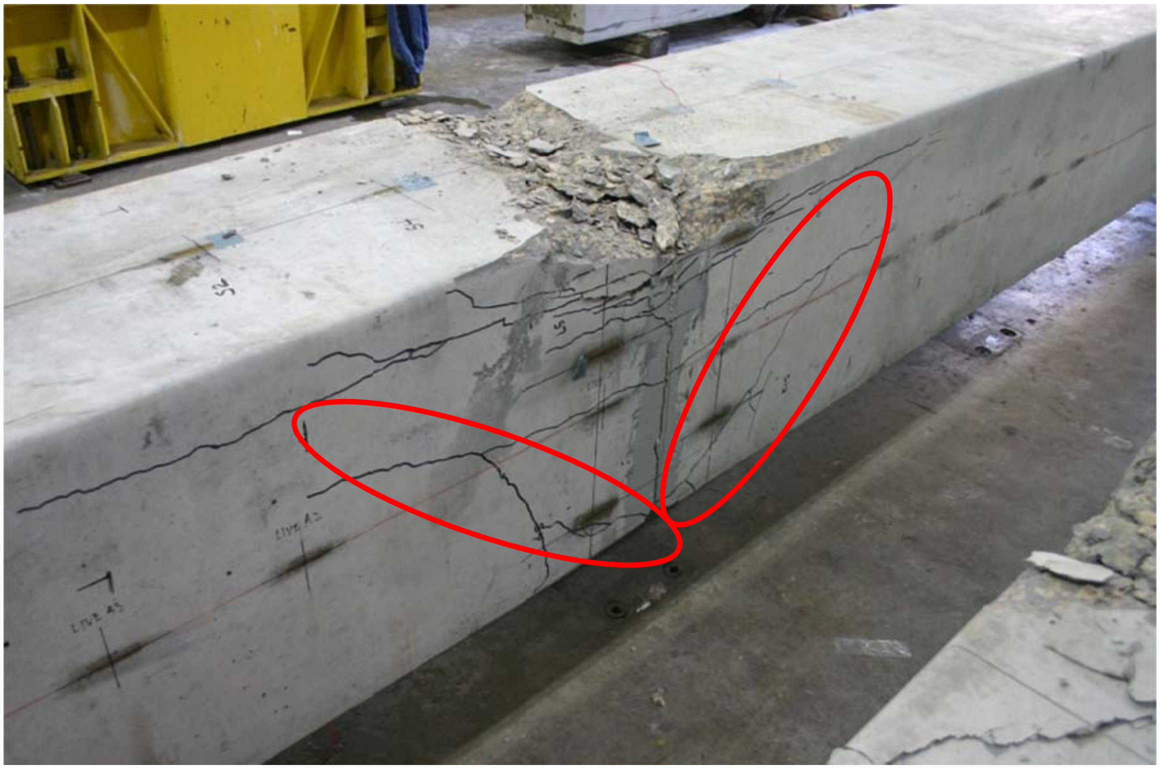


Figure 5.29. Highlighted crack pattern of spliced pile.

The inclined cracks are thought to have formed where the post-tensioning strands and ducts formed a transient support due to their larger flexural stiffness compared with other locations. The stress state of the inclined cracked element is shown in Figure 5.30.

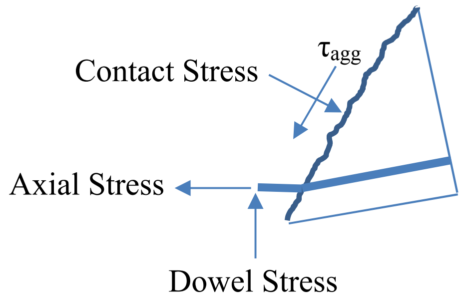


Figure 5.30. Stress state of inclined cracked element.

The aggregate interlocking stress τ_{agg} and contact stress of inclined crack can resist the dowel stress.

5.4.2 Three-point bending

The three-point bending tests (Figure 5.31 and 5.32) were conducted on the segments from the upper half of the spliced pile (with double steel) and one half of the control pile that had the extra spirals at the mid-length (Control-1). Control 1 was selected as it exhibited less distress and provided a viable specimen for retesting at half the original span length. Figure 5.33 shows three-point specimens after testing.

In essence, these unplanned tests gave insight into the true benefit of extra reinforcing steel at the top of spliced piles. This region of a pile in a bridge foundation is often exposed to the highest bending moments.

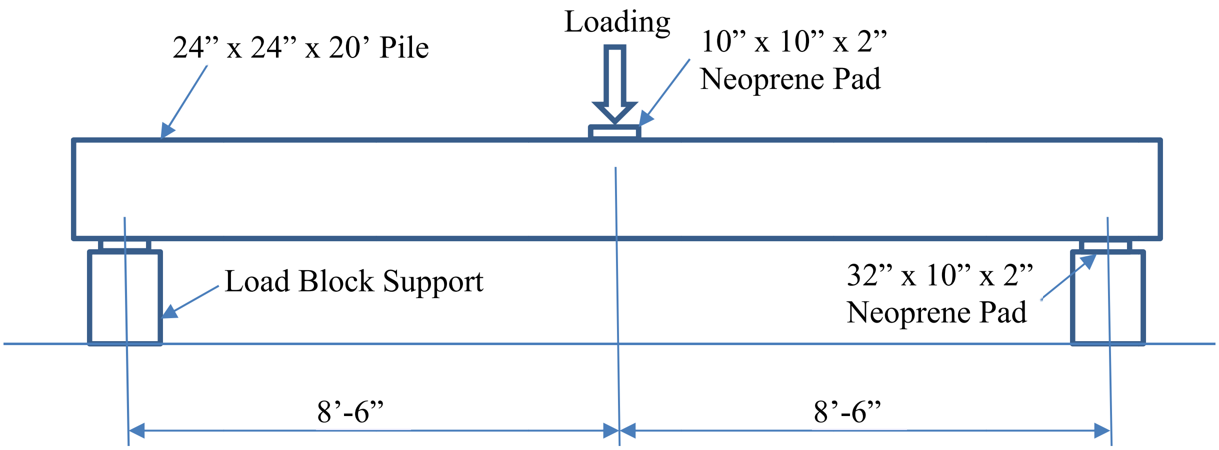


Figure 5.31. Three-point flexural test setup.

Figure 5.32. Three-point bending tests

Figure 5.33. Three-point specimens after testing (control foreground; splice background).

5.5 Test Results

5.5.1 Four-point bending

The load deflection plots for the spliced and the two control piles are shown in Figure 5.34. Similar to the second 14in pile with deformed ducts, the 24in spliced pile (solid line) had a stiffer load displacement response after cracking and exhibited smaller permanent deformation than the controls. The cracking loads were approximately same for control and spliced piles around 36 to 37 kips. Flexural capacities were 635.9 kip-ft for spliced pile and 670.2 kip-ft for averaged value of control piles. The ultimate capacity of the spliced pile was approximately 96% that of the control and exceeded the 600k-ft minimum requirement for 24in piles (Table 2.1).

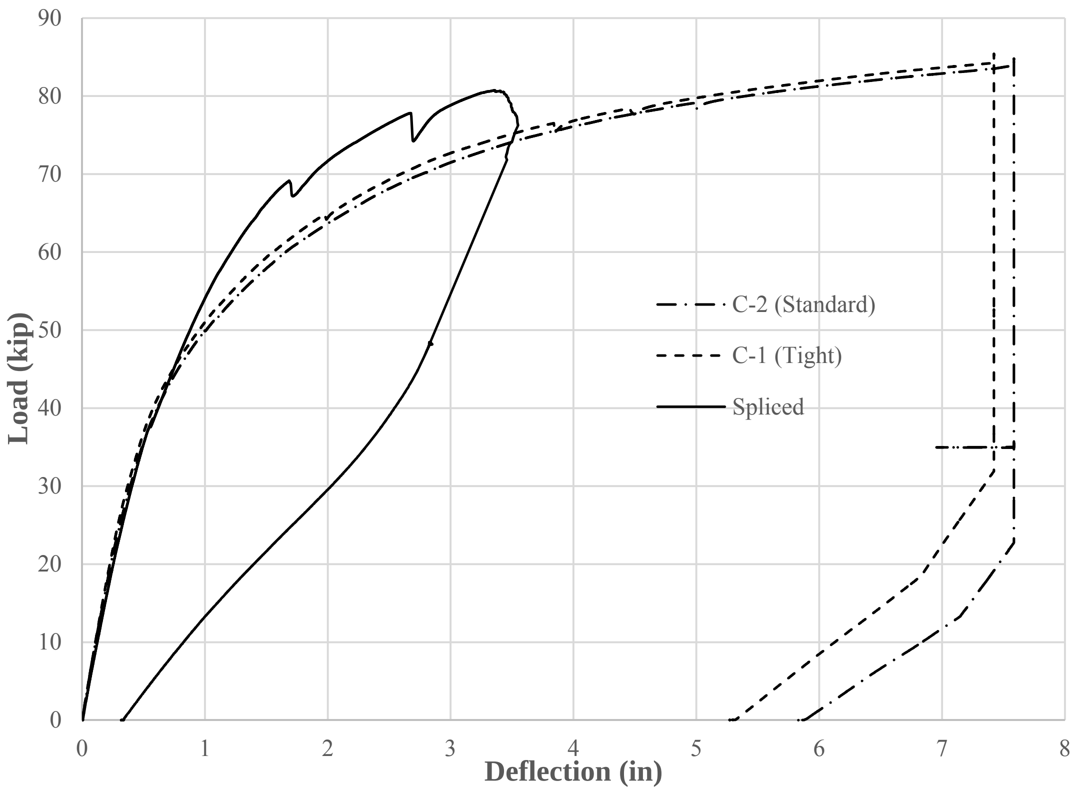


Figure 5.34. Load vs deflection for different pile specimens of four-point bending tests.

Strain gages were attached to all the four point bending tests per the general schematic shown in Figure 5.35 where centerline denotes mid-span (side gages are only shown for west side). The numbers in the bracket refer to the strain gages on the east side.

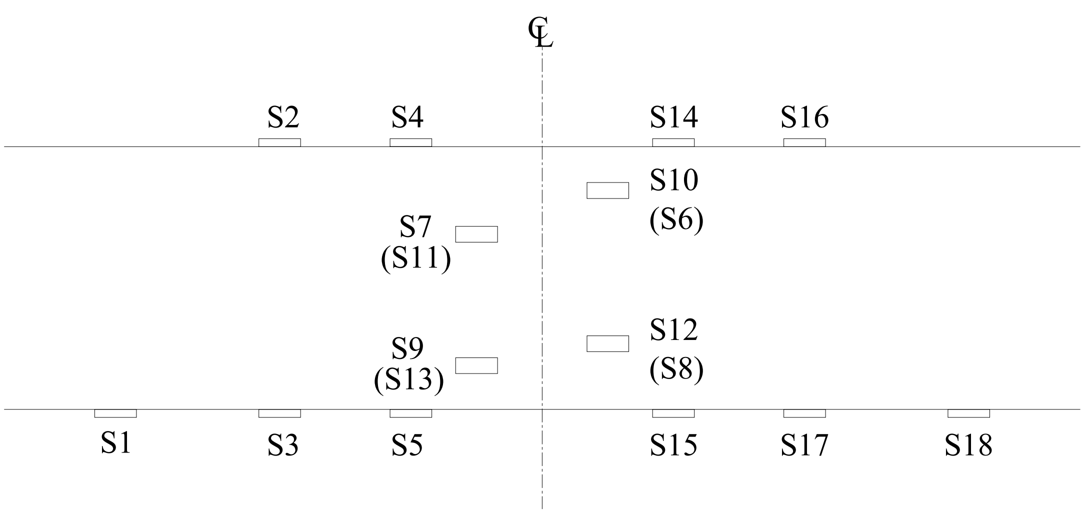


Figure 5.35. Overall strain gage layout.

The detailed strain gage layouts for different views and load versus strain relationships are shown in Figures 5.36 to 5.39. Compression strain is shown as positive and tension strain as negative.

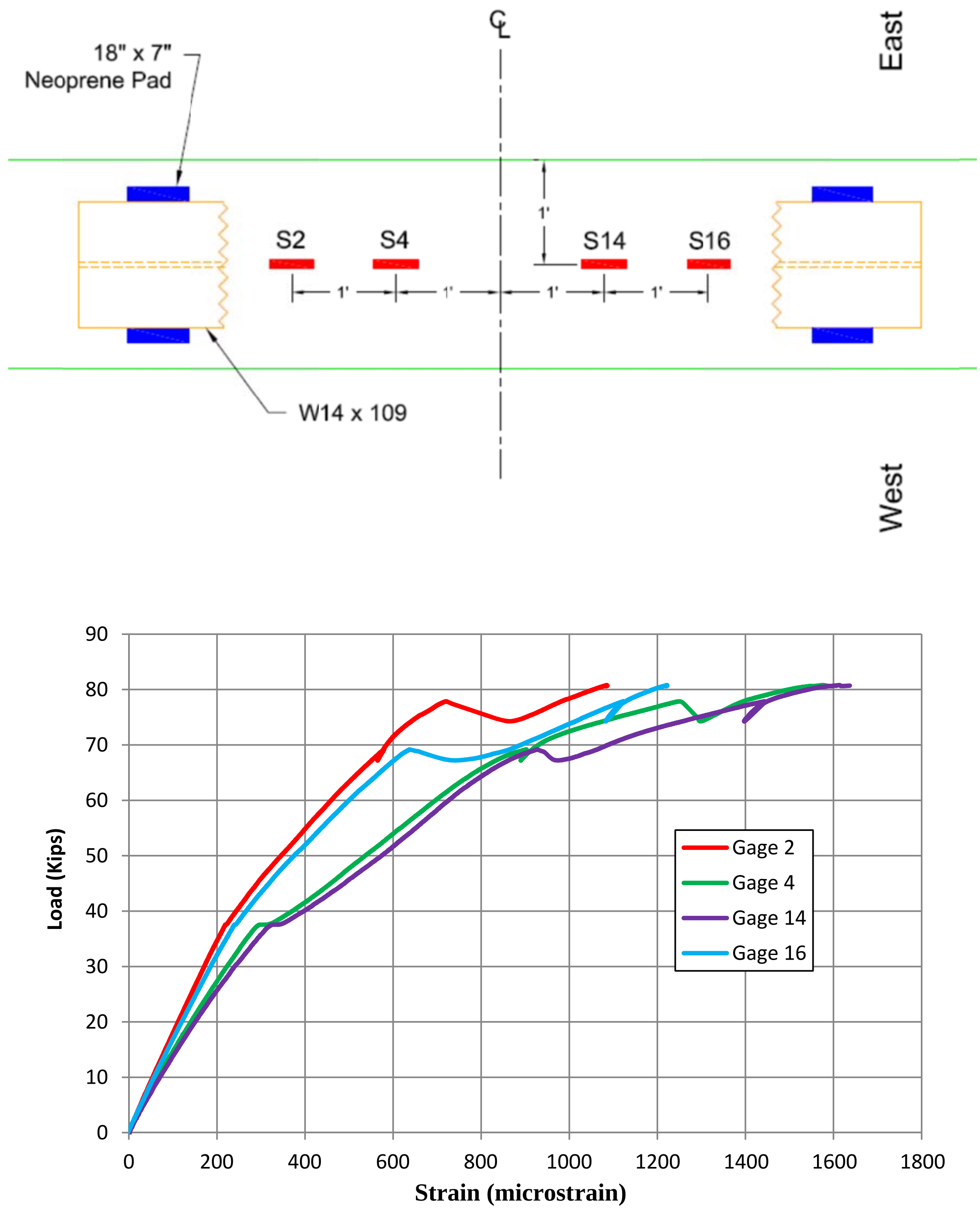


Figure 5.36. Gage layout and load vs strain for top gages 2, 4, 14, and 16 (splice pile).

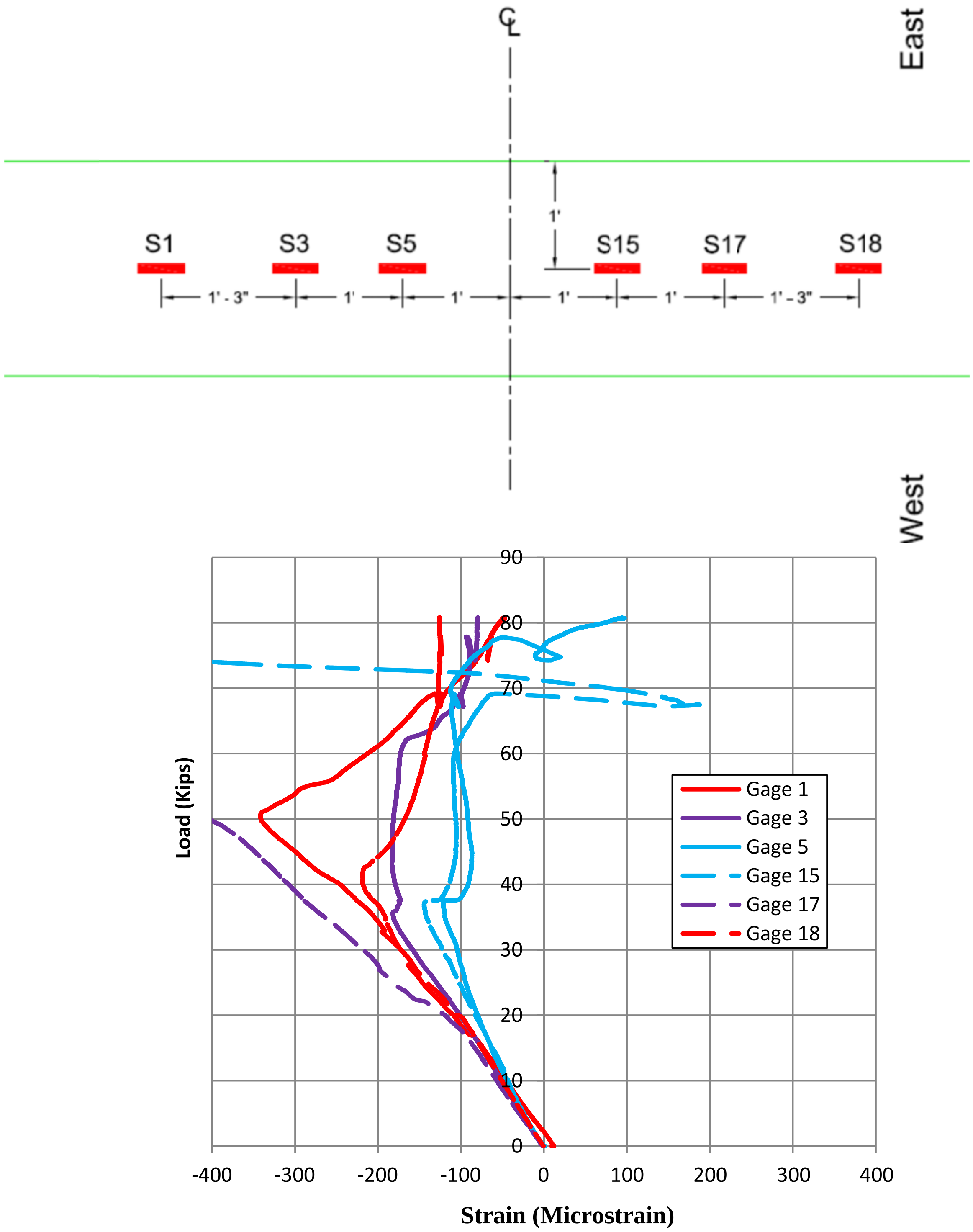
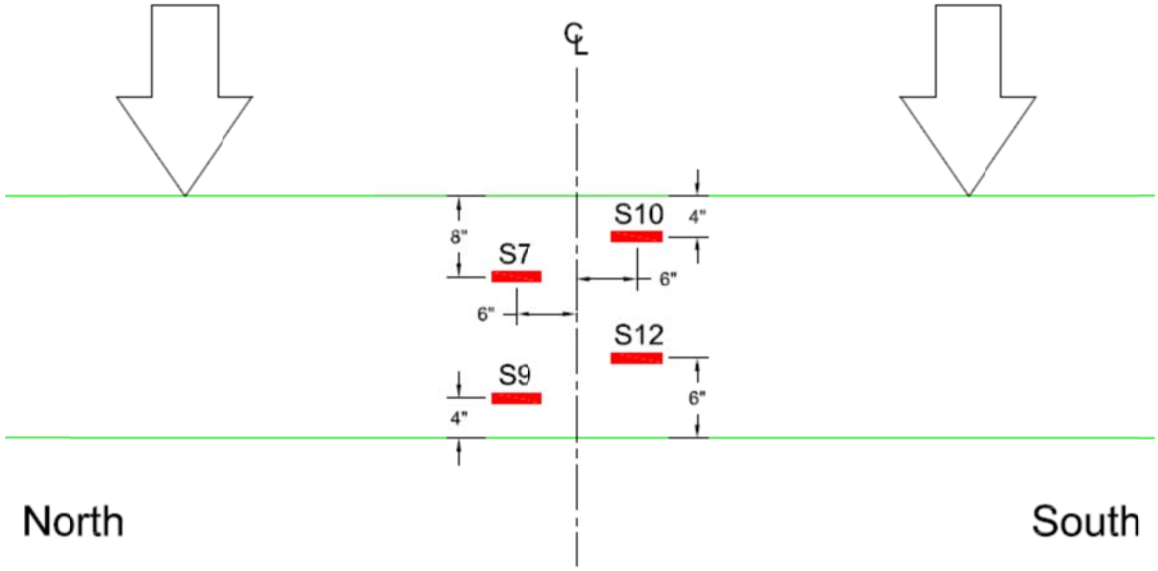


Figure 5.37. Gage layout and load vs strain for bottom gages 1, 3, 5, 15, 17, and 18 (splice pile).



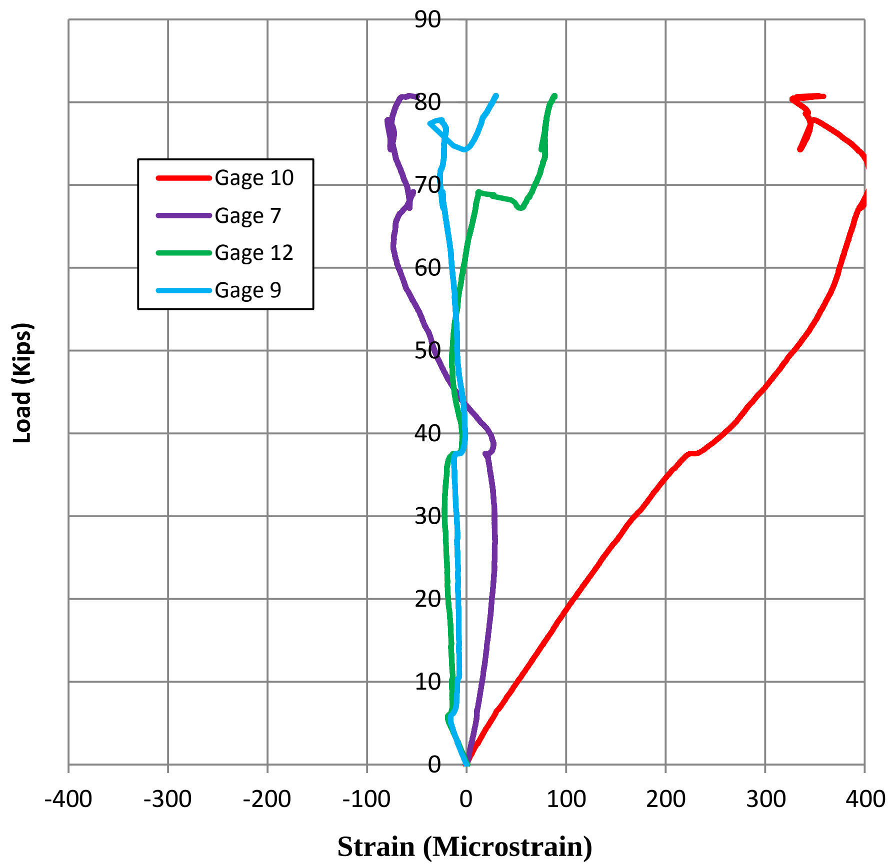


Figure 5.38. Load vs strain for west side gages 7, 9, 10, and 12 (splice pile).

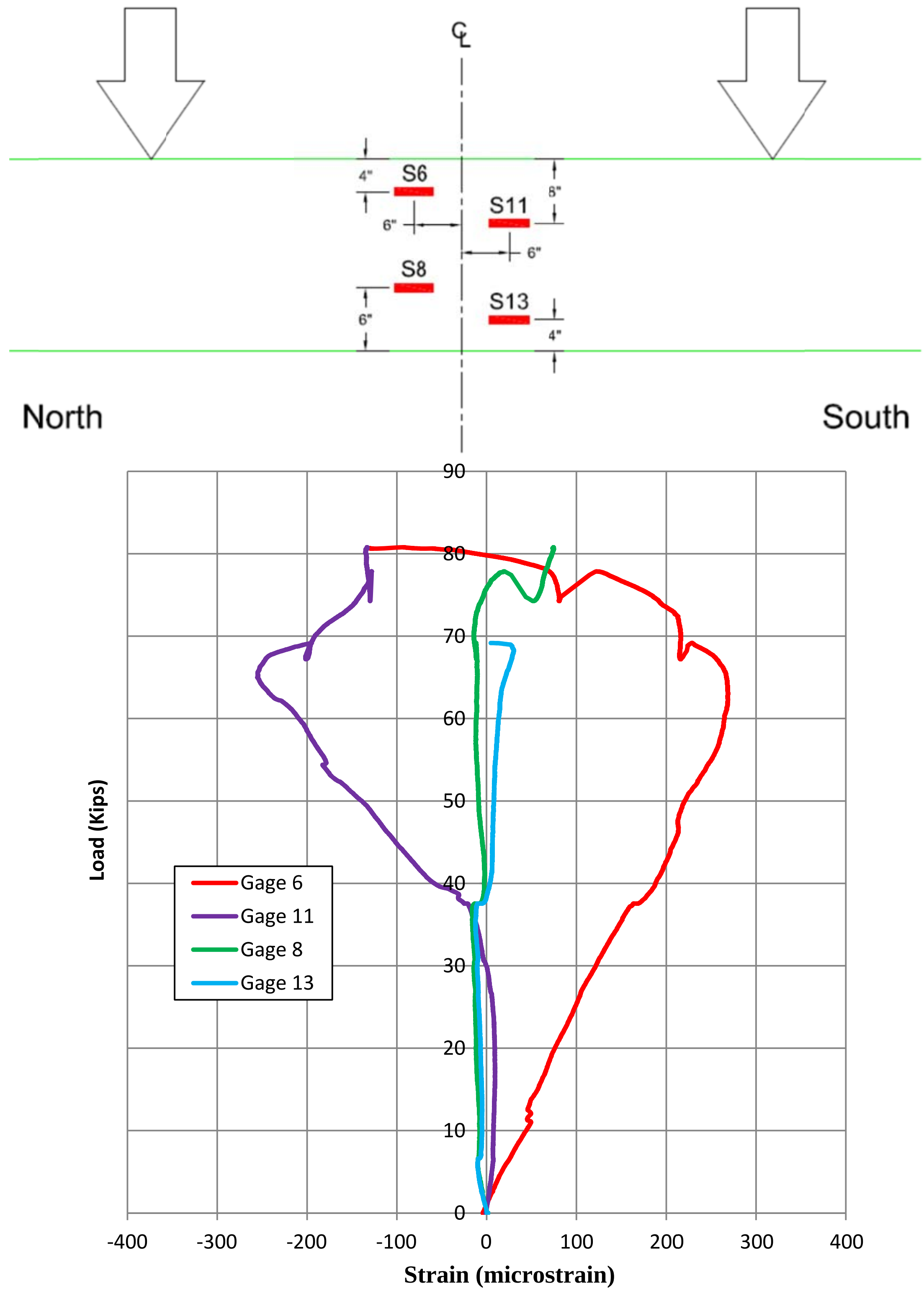


Figure 5.39. Load vs strain for east side gages 6, 8, 11, and 13 (splice pile).

For comparison, the load versus strain relationships of control pile (C-1 with tight spiral spacing) are shown in the Figures 5.40 – 5.44.

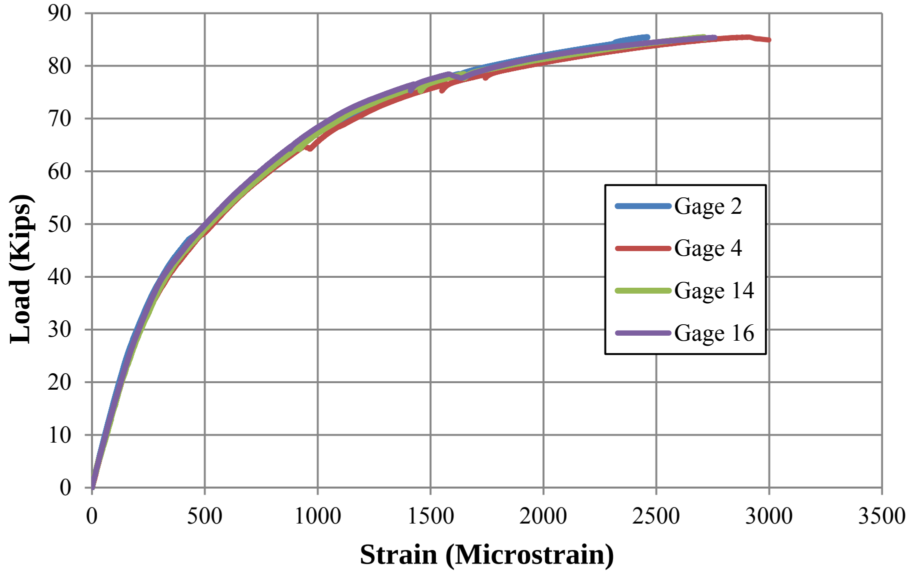


Figure 5.40. Load vs strain for top gages 2, 4, 14, and 16 (tight control pile).

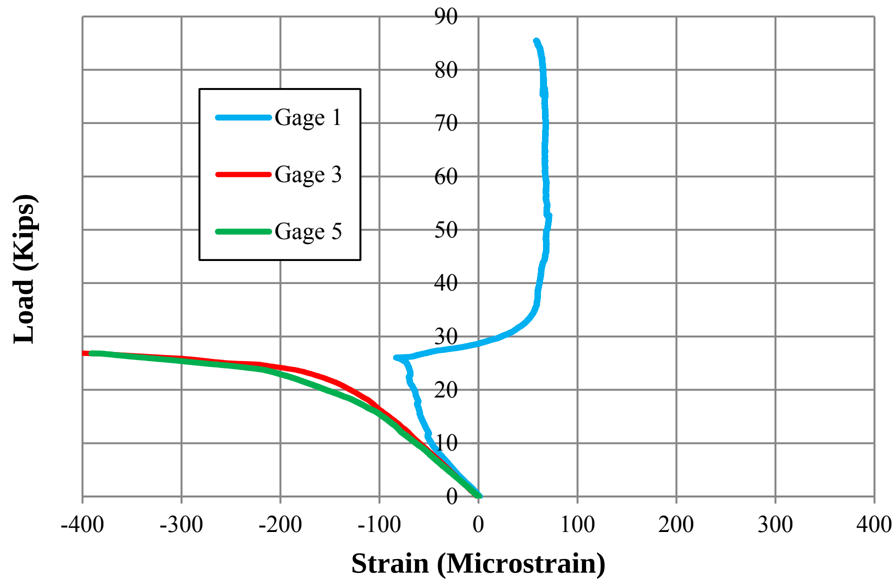


Figure 5.41. Load vs strain for north bottom gages 1, 3, and 5 (tight control pile).

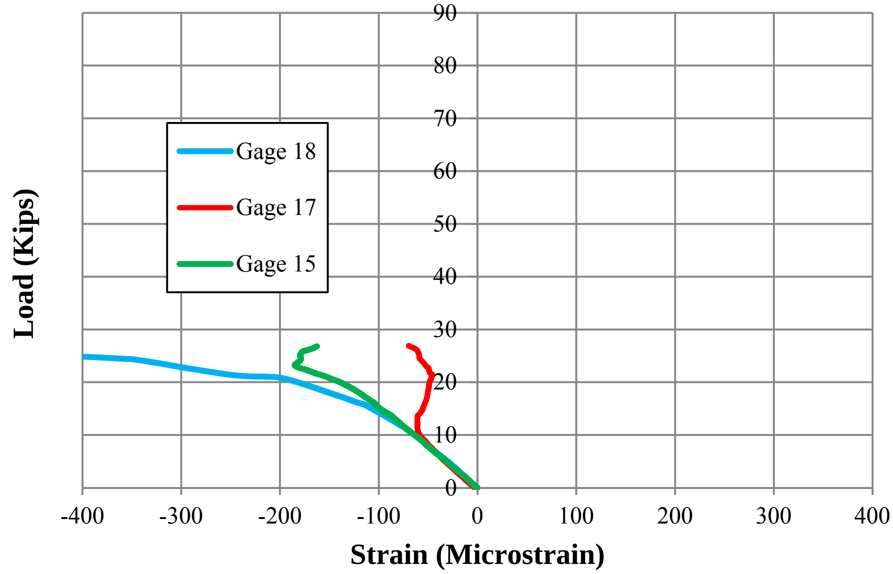


Figure 5.42. Load vs strain for South bottom gages 15, 17, and 18 (tight control pile).

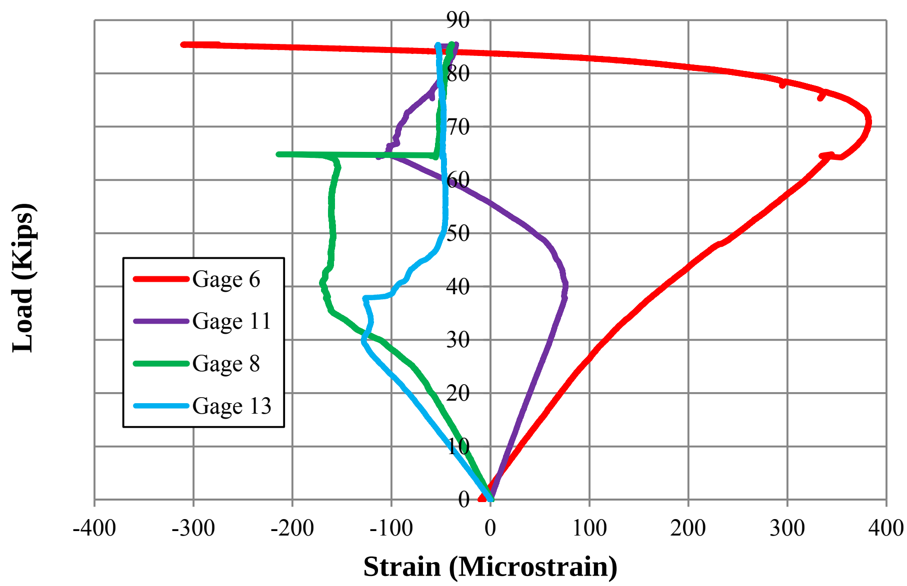


Figure 5.43. Load vs strain for east side gages 6, 8, 11, and 13 (tight control pile).

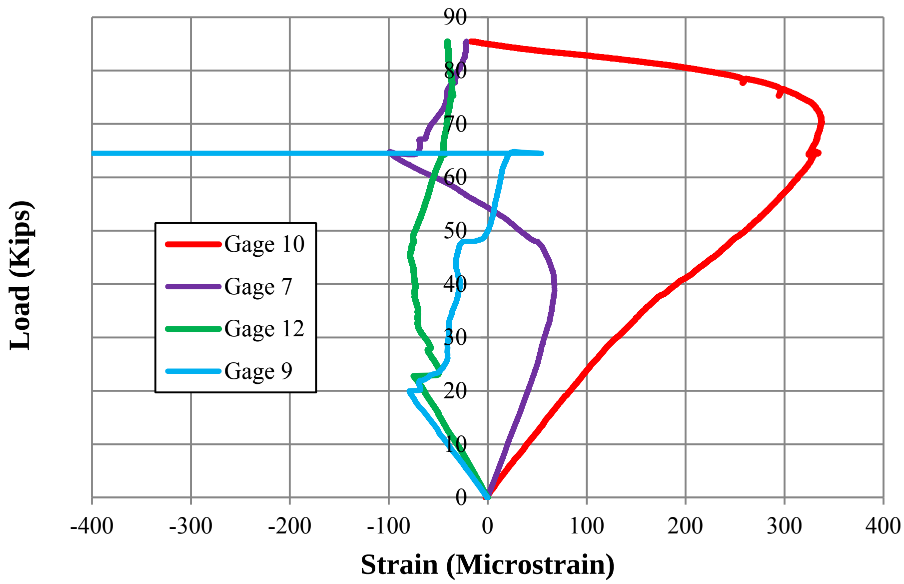


Figure 5.44. Load vs strain for west side gages 7, 9, 10, and 12 (tight control pile).

The comparisons of load versus top strain relationships between spliced pile and control piles for four-point bending tests are shown in Figure 5.45.

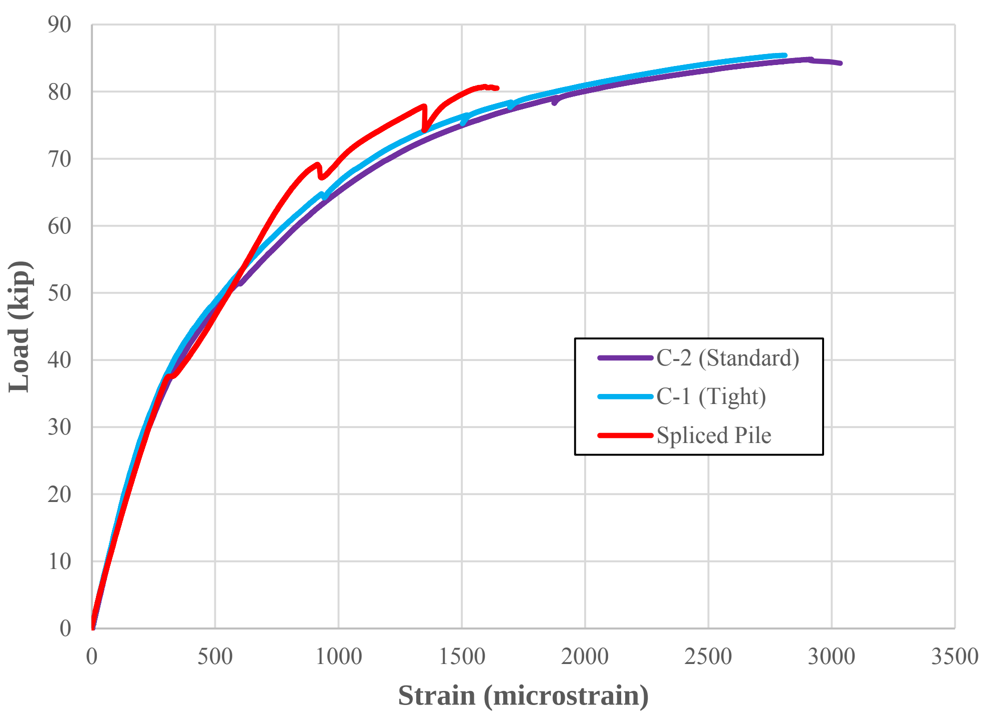


Figure 5.45. Load vs top strain for all three pile specimens in four-point bending.

All strain data was plotted up to peak load unless the gage failed earlier.

The highest compressive strain registered in the splice pile was about 1650ue, whereas the control pile went up to the normal concrete crushing limit of 3000ue. Tension gages in the splice pile fared better than the control pile as all cracking was confined to the splice interface region. This is due to the resident strands of the pile that are unstressed and capable of resisting cracking. All but one strain gage on bottom of the control pile failed by 30kips due to the uniformly distributed cracking in the constant moment region. However, concrete crushing was also observed to be confined to the splice interface (spliced pile) whereas the control piles showed a more widely distributed crushing region (especially in the standard C-2 control). Closer inspection of Figure 5.36 indicates that the compressive strains at the top of the splice pile were not uniform and decreased with distance away from the splice interface; gages 2ft way were 1100ue at failure while gages 1ft away were higher at 1600ue. Knowing crushing was confined well inside the 1 ft gages (Figure 5.27), strain at the interface must have risen to 3000ue as with the other piles tested. Figure 5.46 shows the suspected strain distribution across the top of the splice pile based on Figure 5.36 strain data at failure and the observed crushing zones.

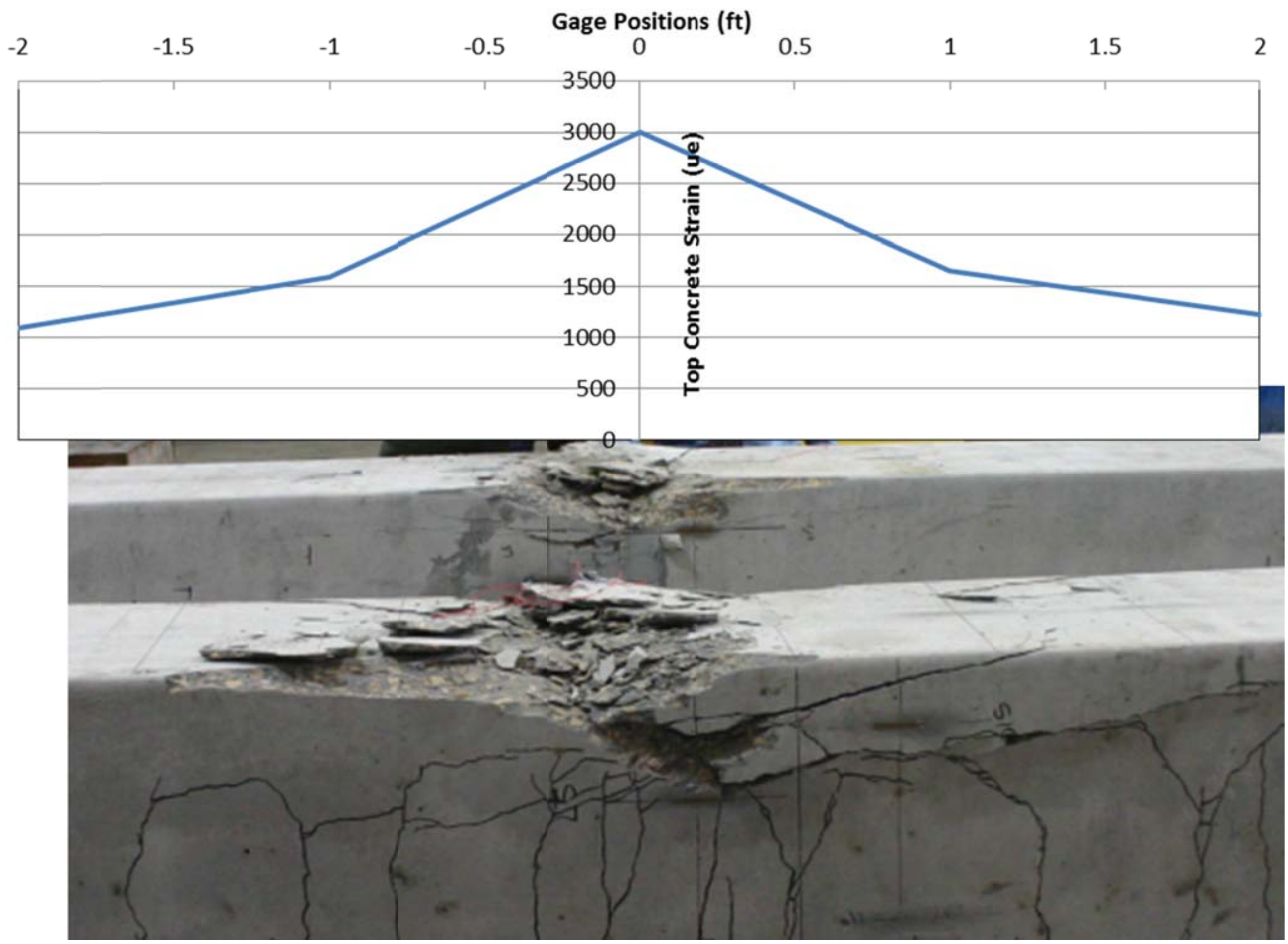


Figure 5.46. Concrete crushing regions: control (foreground); splice (background).

The strain distribution along the side of the piles varied considerably between the splice and control piles where the control pile followed expected beam bending theory: gages 8, 9, 12, and 13 in the bottom half of the pile were in tension and gages 6, 7, 10 and 11 in the top half were in compression. At approximately 55kips the neutral axis crossed over gages 7 and 11 (8in from top) where those gages transitioned from compression to tension.

In contrast, the strain distribution along the side of the splice pile showed very little longitudinal strain in the bottom three gages; the neutral axis did appear to cross over gages 7 and 11 but at a lower load (45kips west side 32kips east side). The bottom two gages on each side show little to no strain which indicated the orientation of the gages was aligned in a zero stress direction (45 degrees from any principal stress). This was confirmed by the crack that propagated from the bottom of the splice interface at a 45 degree angle (shown in Figure 5.29) essentially aligning with one of the principal stress directions (tension perpendicular the crack).

5.5.2 Three-point bending

The presence of double steel in the upper splice pile segment has the potential to be beneficial especially in cases where higher bending moments are experienced near the top of pile (beneath the pile cap). In such cases additional reinforcing steel is often added. However, the splice pile already has unused / abandoned steel strand in this region due to the process used for the splice. Therefore, the upper portion of the spliced four-point bending specimen was cut above the splice and retested with a shorter span in 3-point bending. Likewise, Control pile 1 with tighter spacing at mid span was also cut similarly and tested for comparison. Experimental results of load versus mid-span deflections for the two pile segments are shown in Figure 5.47.

The spliced segment failed at 216.4 kips (919.7k-ft) while the control segment only failed at 181.5 kips (771.4k-ft). Not surprisingly, this was lower than that of spliced segment. When the uniformly distributed dead load moment is added to the test load moments, validation of the 3 point test can be shown where both tests of the control pile (3 or 4 point) resulted in the same failure moment (Table 5.1). The comparison also shows the spliced pile developed smaller permanent deformation but also exhibited higher flexural capacity in the upper segment than the regular (control) segment.

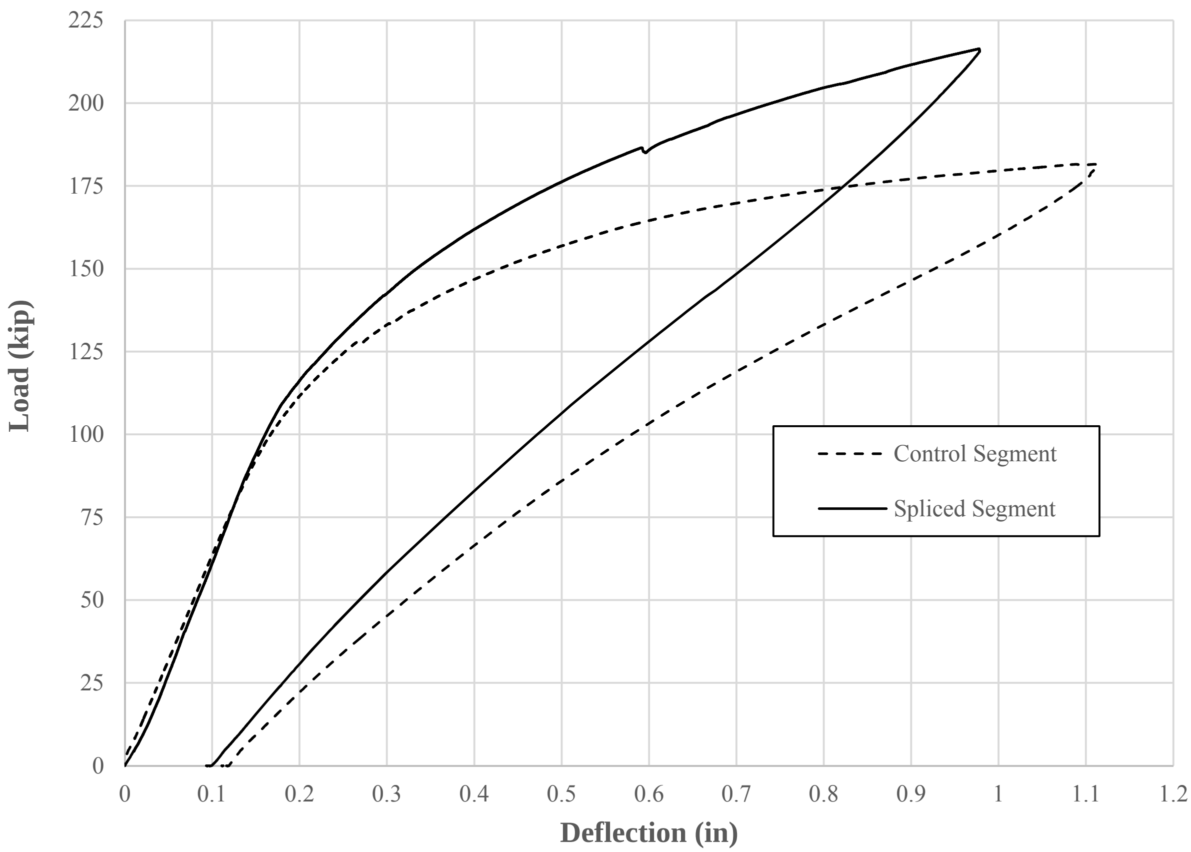


Figure 5.47. Loadings vs deflections for pile segments of three-point bending tests.

Table 5.1. Comparison of bending moment at failure

Tests	**Test Moment (k-ft)**	**DL Moment* (k-ft)**	**Ultimate Moment (k-ft)**	**Specimens**
3 point	771.4	21.7	793.1	20ft Control
4 point	670.2	108.3	778.5	40ft Control
3 point	919.7	21.7	941.4	20ft Upper Splice Section
4 point	635.9	108.3	744.2	40ft Spliced Pile

**Dead load moments for both control and spliced piles were based on a unit weight of 150pcf. The true unit weight of the splice pile would be slightly higher than the control piles due to the additional weight of the splicing strands and components.*

The load versus top strain relationships for the pile segments in three-point bending tests are shown in Figure 5.48. Both segments failed at a compressive strain of 0.003 (3000ue shown in graph); the spliced segment again at a higher capacity than that of the control segment.

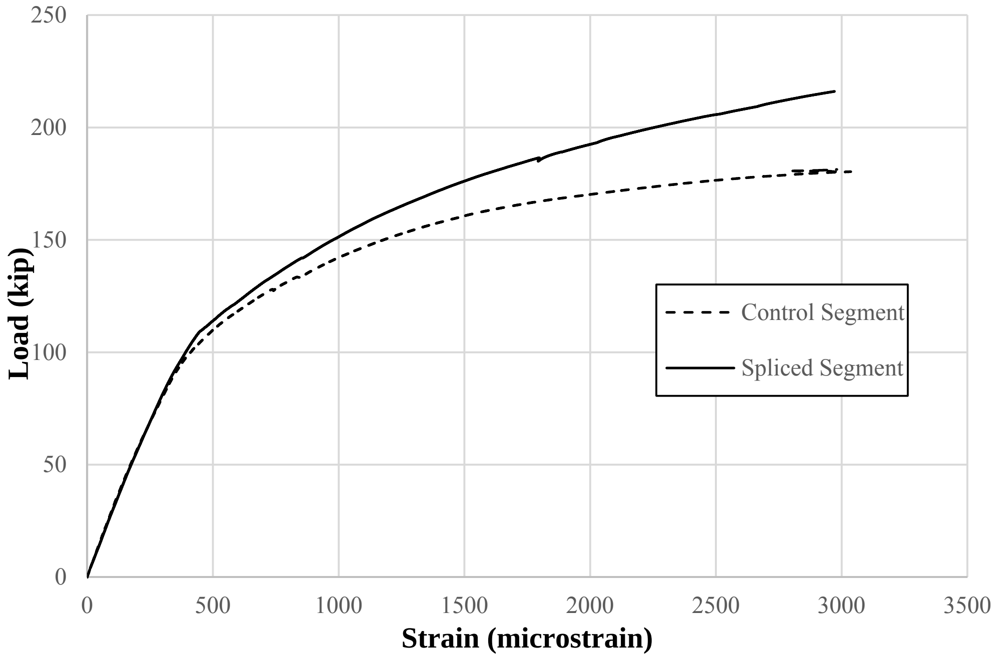


Figure 5.48. Load vs top Strains for pile segments in three-point bending.

5.6 Lessons Learned

Each splice pile tested provided usable feedback to further the advancement of the overall splice concept. For the 24in pile tests the following was concluded:

- Casting yard installation highlighted the need to keep preassembled duct panel more flexible or use fewer ducts per assembly.
- Concreting the splice pile could be made easier by reducing the number of splicing strands thereby using the next larger size strand (e.g. 16 – 0.6 splice strands instead of 20 – 1/2in special, etc.). This also would reduce splicing time. However, this is only more cost effective and efficient if the reduced number of strands is divisible by 4.
- The washer insert used to reduce wedge set losses may also restrict strand pass-through if the spring cap on the chuck is inadvertently left in the compressed state in the casting bed. Screw cap chucks with no spring back lash should be used to fully control final in-bed tolerances. This complication manifested itself in difficult strand installation when preparing the upper pile segment prior to splicing. *Recall the lower pile segment does not use the tolerance reducing washers.*
- Grouting was successful but slow due to restrictions in and around the wedges. Recall, grout was pumped through the ducts and passed through the three slits that form around the three wedges. Enhanced passageways should be considered to speed the grouting process or an even more fluid grout may be considered.

- High pressure used to force grout through the wedges also forced some grout to find alternate pathways out of the pile in the splice region. No adverse effects were noted as a result of the grout paths through the epoxy at the time of bending tests, but alternate couplers should be considered to provide the contractor the option to grout immediately while the epoxy is uncured.

Chapter 6: Field Pile Driving Demonstration (24in pile)

As an extension to the full scale bending tests, another 24in splice pile specimen was cast and driven on a production bridge project where similarly sized piles had been already driven. This chapter outlines the steps taken in preparing the components, casting the pile segments, splicing the pile, driving the pile and an analysis of the collected data.

The field demonstration specimen was cast from two 24in square pile segments where the lower segment was 70ft and the upper segment was 30ft. The location of the splice was set based on discussions with state engineers and practitioners that noted that tension stresses are often highest in the upper 1/3 of the pile length.

6.1 Fabrication of Components

Using recommended upgrades from the previous experiences (Chapters 4 and 5), the field demonstration spliced pile specimen incorporated the following modifications:

(1) Screw back chucks were used to eliminate changes in wedge tolerances in the bed.
(2) The number of splicing strands was reduced from 20-1/2in special strands to 16-0.6in strands. The motivation was to reduce splicing time and cage congestion during concreting.
(3) The alignment dowels (previously conical in shape, Figure 5.15) were fabricated in a straight cylindrical fashion that could accommodate 1/8in thick neoprene compression gaskets to form a seal during grouting.
(4) Recesses for neoprene seal/gaskets were produced on each face of the splice pile faces around each alignment dowel using 1/16in thick neoprene washers compressed between the splice header plate and the duct assemblies.
(5) Ducts were fitted to the anchorages using thermal expansion/contraction to eliminate mismatched steel thicknesses that were previously welded.
(6) Spatial conflicts between the anchor bearing plates and the adjacent duct were eliminated by tailoring the contact area and plate shape to fit around the adjacent strands and ducts. All bearing plates were laser cut by a computerized burn table.
(7) Grout manifolds were maintained one per side of pile, but an offset was used to reduce the conflicts with the pile prestressing strands.
(8) Finally, all ducts were deformed over the full length of duct and not just those portions immediately adjacent the splice interface (previous specimen was limited to 10ft on either side of the splice)

As with the previous splice specimens, the preparations included fabrication of: chuck assemblies, confinement coils, deformed ducts, splice header, and grout manifolds. The laser cut bearing plates welded to the chucks are shown in Figure 6.1, ducts thermally press fitted to the anchorage and threaded chuck cap in Figure 6.2 and 6.3, confinement coils and finished assemblies Figure 6.4 and the resurfaced splice header plate Figure 6.5. In all, 32 chuck assemblies were needed for the 16 splicing strand configuration. The pile however still used twenty 1/2in special strands.

Figure 6.1. Laser cut plates (top left), welding chuck/plate anchorage assembly (top right), machined close tolerance anchorage (bottom left), chucks ready for duct connections (bottom right).

Figure 6.2. Thermal press fitted duct to anchorage process (screw back cap, top four images; bearing plate, bottom).

Figure 6.3. Ducts fitted to bearing plates (left) and screw back caps (right)

Figure 6.4. 32 confinement coils before and after assembling into completed anchorages.

The same 24in splice header was used even though the center duct holes on each side were not needed. The faces of the header were resurfaced to ensure no deviations from parallel occurred during the previous usage (Figure 6.5). The assembled header was used to pre-set the duct spacing where the ducts were welded together in pairs. Therefore, 8 pairs would then be used on each side of the splice (in each pile segment). This also meant that there would be four stagger lengths instead of five used previously. Before each duct was assembled and welded into the pairs, the wedges and/or wedge spacers were installed. Spacers were only used in the upper pile side of the splice where wedge set losses would occur.

Figure 6.5. Blanchard grinding header plate faces (top left); wedge loss spacers (top right) reconditioned header (bottom) used to pre-fit duct assemblies welded into pairs.

6.2 Casting of 24-inch Driving Demonstration Specimen

The 24in driving demonstration specimen was cast at DuraStress Inc. in Leesburg, Florida. This included two pile segments 70ft and 30ft complete with internal splicing components. Figure 6.6 shows the internal components as delivered to the casting yard. Duct extensions were installed as separate pieces, but all ducts were deformed.

Figure 6.6. Internal components of splicing system as delivered to casting yard; extension ducts (left), paired anchorages (right).

Anchorage locations were staggered into four steps which corresponded to four ducts on each side of the pile. Therefore, two pairs were installed on each side, one pair with the two shorter lengths to the anchorage and the other pair with the longer lengths. In all, there were two types of interchangeable panels which were matched with a corresponding panel in the other side of the pile splice. This meant that some care was required to ensure proper matching to other side of the pile splice. Figure 6.7 shows two pairs of ducts that would form one pile side (during fabrication).

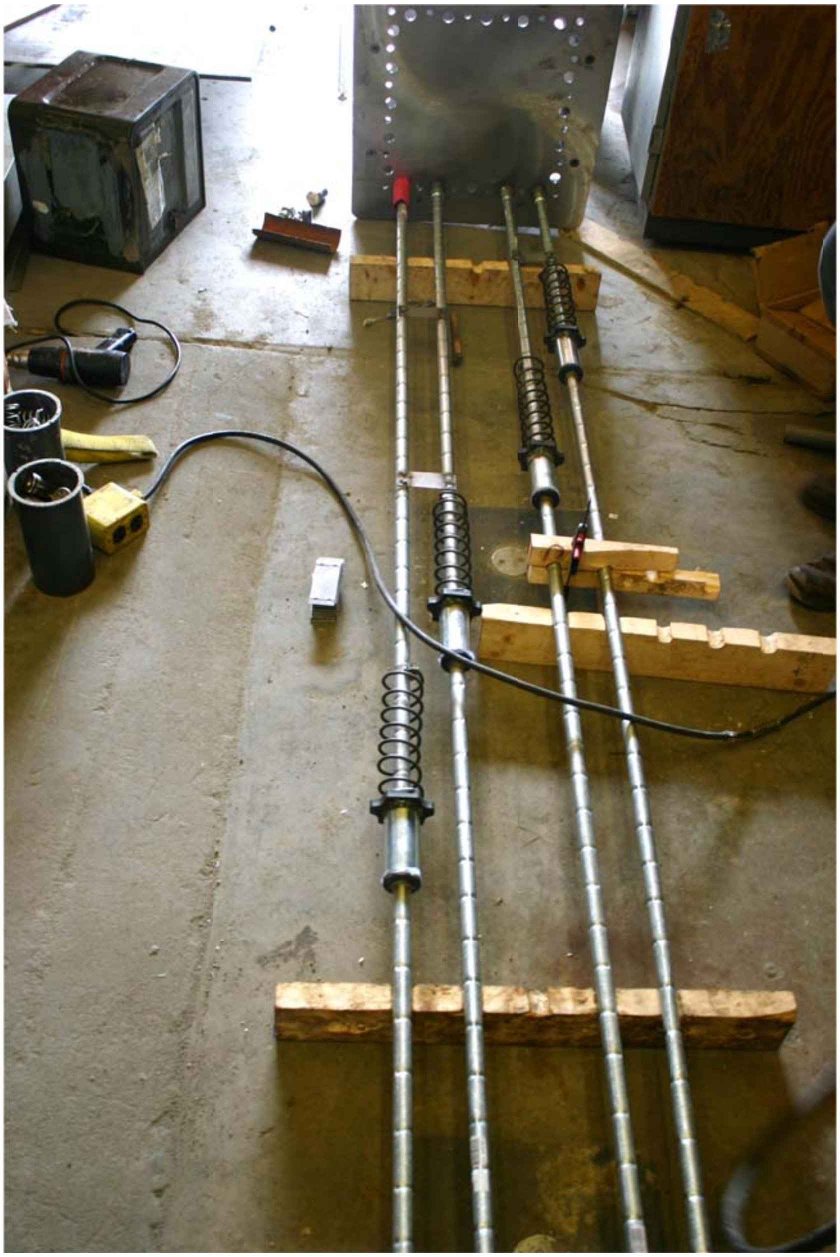


Figure 6.7. Four-duct, interchangeable two pair set with staggered anchorages and deformed ducts.

Casting the splice pile followed standard procedures where the splice header was inserted in the bed like any normal pile separation header (Figure 6.8) and strands were progressively installed in order from bottom to top prior to installing ducts and pulling the spiral reinforcement.

Figure 6.8. Strands threaded through header plates in casting bed, splice header foreground; chamfered jacking plate header (background).

Installation of the duct assemblies required that the long lengths of duct extensions and duct pairs be placed in the bed after the strands were tensioned but before the spirals were pulled to the ends of the pile. Two top strands were left slack to ease installation and tensioned only after all splicing fixtures were installed. The two, pair duct option eased installation over the previously used 5 duct panel which was awkward to handle. Figure 6.9 shows the ducts bundled inside the strands so that the spirals could be easily pulled over and spaced prior to installation.

Figure 6.9. Duct extensions and anchorage assemblies suspended with tie wire while spirals were set at prescribed spacings of 1, 3 or 6inches per standard specifications (upper 30ft pile, left; lower 70ft pile, right).

Figure 6.10 shows the finished assemblies. The black and red heat shrink sealant denoted whether or not the duct assembly incorporated the wedge spacer (black left were in upper pile segment and did contain the spacer as discussed earlier). Figure 6.11 shows the bearing plate shape and how it conforms to the strand configuration.

Figure 6.10. Splicing system in place

Figure 6.11. Four duct adaptation increased concrete access/flow (right), previously used 5 duct version (left), sculpted bearing plate shape (top right).

The spirals in the pile segments conformed to FDOT standard specifications as shown in Figure 6.12, (i.e. 5 turns @ 1in followed by 16 turns @ 3in at each end). The remaining portion of pile had a 6in pitch.

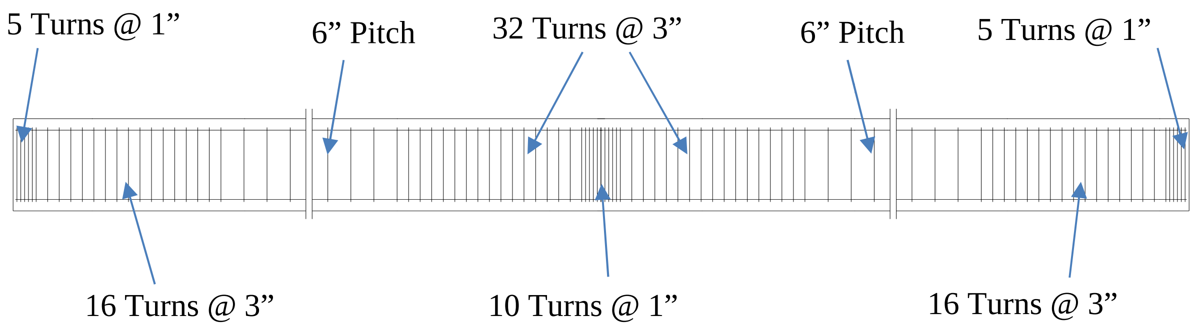


Figure 6.12. Pile spiral layout (FDOT specifications).

A self-consolidation concrete mix was used which removed the need for vibrating (Figure 6.13). Concrete test cylinders were made for compressive strength tests which were conducted both at DuraStress and the Structures Lab at the University of South Florida.

Figure 6.13. Concrete placement.

Concrete strength at the time of de-tensioning was 7900psi (3 days after casting) which increased to 8400psi at the time the pile segments were spliced (4 days after casting).

6.3 Splicing the 24in Demonstration Pile

The cured pile specimens were removed from the bed and set aside in a low traffic region of the DuraStress yard where they were made ready for splicing. During extraction the bottom corners of the upper pile segment were chipped leaving two large triangular gaps in the cover region (Figure 6.14). Some honeycombing was also noted on the lower pile.

Figure 6.14. Upper pile splicing was face was chipped along bottom side during removal from bed (duct bolts retained to keep ducts clean and free from debris).

Figure 6.15. Lower pile segment splicing face showing some honeycombing on right side.

6.3.1 Preparations for Splicing

During preparation work, all the duct bolts were removed and each duct was inspected for debris with a borescope (Figure 6.16). Figure 6.16 also shows the 1/16in thick neoprene washers used to form the grout seal recesses. After inspection, the mating surfaces of the piles were checked for flatness where strand stubs were ground below the surface and the edges of the pile were slightly rounded (Figure 6.17).

Figure 6.16. Borescope inspection showed no debris in wedges.

Figure 6.17. Prepping splicing faces by removing high spots from steel strands or concrete edges. Note recessed formed by 1/16in rubber washers.

The 0.6in splicing strands were cut to length (approximately 45ft), ends were ground to remove burrs or sharp edges (Figure 6.18) and then fed into the upper pile segment where the exposed strands were clamped to ensure no slippage into the upper pile segment occurred during splicing (Figure 6.19).

Figure 6.18. Splicing strands ground to removed burrs and sharp edges facilitating easier insertion into embedded anchorages.

Approximately ten feet of strand was left exposed at the base of the upper pile segment (for insertion in the lower pile) and 5ft was exposed out the top of the upper pile segment for jacking (only 1 to 2ft was necessary). For convenience, the lower exposed strand lengths were staggered in one inch increments so only one strand at a time would come into contact with the lower pile segment. Figure 6.19 shows all strands in place ready for splicing complete with strand clamps. Readily available, easy-to-use mechanical vice grips were used. A quick check of hardness showed the teeth of the clamps made no indentations in the strand that could cause stress concentrations. Also, note the alignment dowels and the 1/8in thick neoprene compression gasket were installed at the same time. Recall the recess around the ducts was formed with a 1/16in neoprene washer compressed between the header plate and the duct at the time of casting. Detailed schematics of the dowel and seal design can be found in Appendix E.

Figure 6.19. Upper pile segment with all splicing strands inserted and clamped complete with alignment dowels and neoprene gaskets.

The pile segments were spliced horizontally using a mobile gantry-type cast yard crane (Figure 6.20). Figure 6.21shows the pile segments being brought together as the strands were aligned and fed into the corresponding ducts in the lower pile. Even though the strands had been staggered in 1in increments, the flexibility of the long exposed lengths (10ft) permitted manual bending and reinserting strands that missed the sequence of strand insertion.

Figure 6.20. Gantry crane used to lift upper pile segment (left) and bring into close proximity with lower pile segment (right).

Figure 6.21. Feeding splicing strands into lower pile (background) as upper pile (foreground) was brought closer.

Strand clamps were removed, epoxy sealant was applied to both splicing surfaces and the upper pile was brought within 1in of contact (Figure 6.22). As the weight was independently held by the crane in two positions, the vertical and horizontal alignment was controlled. Minor adjustments could then easily be made by hand.

Figure 6.22. Final steps in aligning the splice.

6.3.2 Post-Tensioning

The post-tension stressing order was determined so that each pair of opposite strands were jacked to predetermined loads in three stages, which as noted earlier maintained balanced strain and reduced losses from elastic compression of the concrete. The splice stressing order of matched pairs was marked on to the pile for convenience as shown in Figure 6.23. All strands with the same anchorage locations were stressed in further sets of four (i.e. 1, 2, 3, and 4 all had anchorages 59in into lower pile and 43in into upper pile). Three jacking cycles were used starting at 8kips and increasing in 20kip intervals up to 80 percent of 270ksi in the 0.6in strands: 1st cycle, 8kips; 2nd cycle, 28kips; and 3rd cycle, 48kips. Figure 6.24 shows the stressing jack while loading.

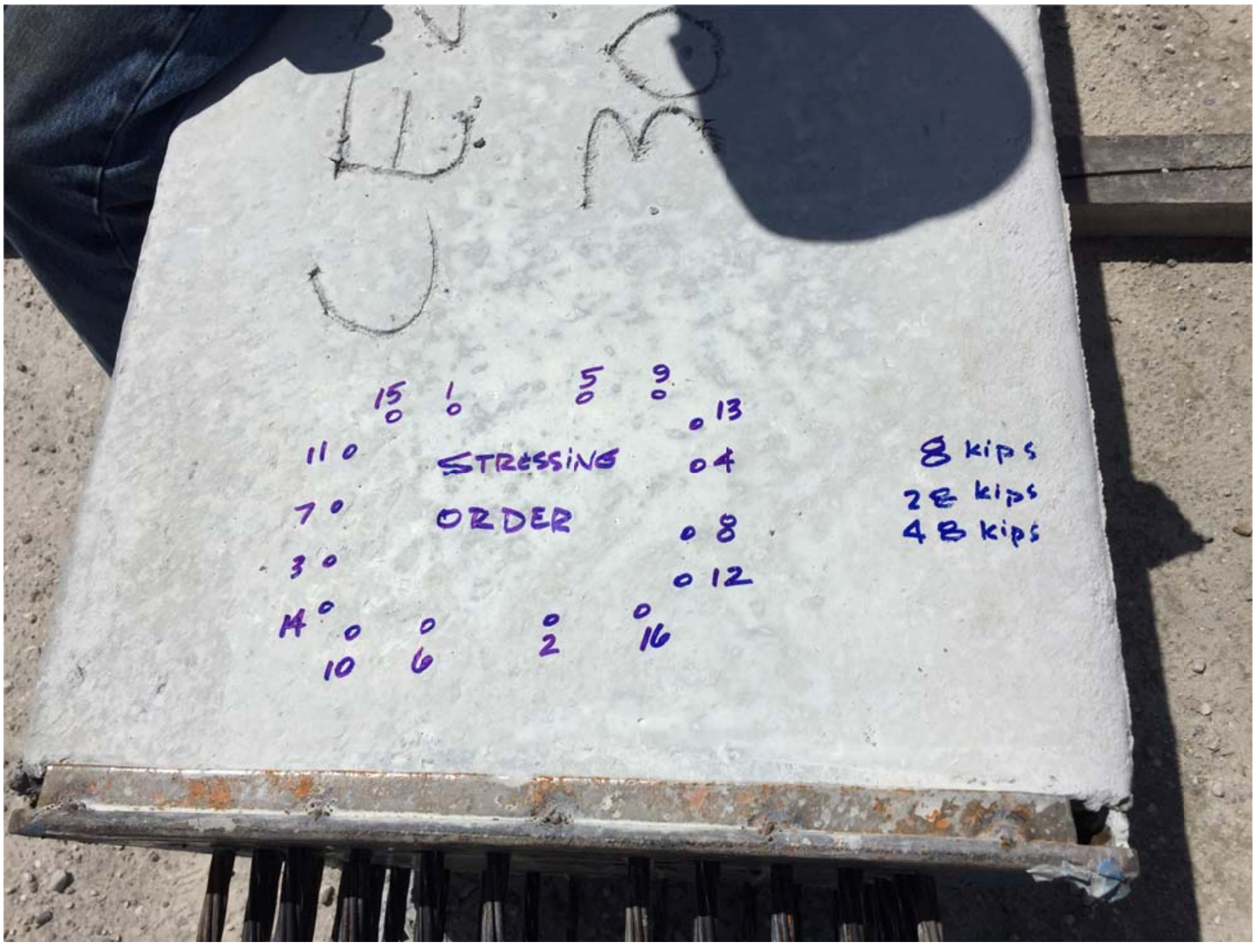


Figure 6.23. Splice stressing order of matched pairs

Figure 6.24. Post-tensioning of splicing strands.

Strain gages mounted to both sides of each pile segment were positioned at four locations spaced away from the splice interface at 13.5, 35, 51, and 67in. This corresponded to the midpoint between anchorages or the anchorage and the splice interface (Figure 6.25). Anchorage locations were at 27, 43, 59, and 75in away from the splice interface.

All strain gages as well as the jacking force was recorded during the post tensioning. Delays were experienced due to a faulty generator. Figures 6.26-6.29 show the jacking force, average strains from each pair of similarly positioned gages, the eccentricity imposed on the pile during jacking at the splice interface and the final strain distribution in the pile splice region.

No appreciable strain or load was imposed during the first cycle of loading as the wedge seating losses exceeded the 8 kip applied load (Figure 6.27). This was an oversight; a two stage loading would have been more efficient. However, soft contact was achieved. Similar to the previously spliced pile (Chapter 5), the strain gages closest to the splice did not register the full load due to the epoxy in the center taking load which progressively moved to the outer edges when load was then registered.

The “left” and “right” denoted in Figure 6.28 are arbitrary definitions whereby they represent the eccentricity computed from strain gages on opposite sides of the pile. No appreciable eccentricity remained after the splicing was completed.

Figure 6.25. Strain gages mounted on both sides of each pile 13.5, 35, 51, and 67in from splice.

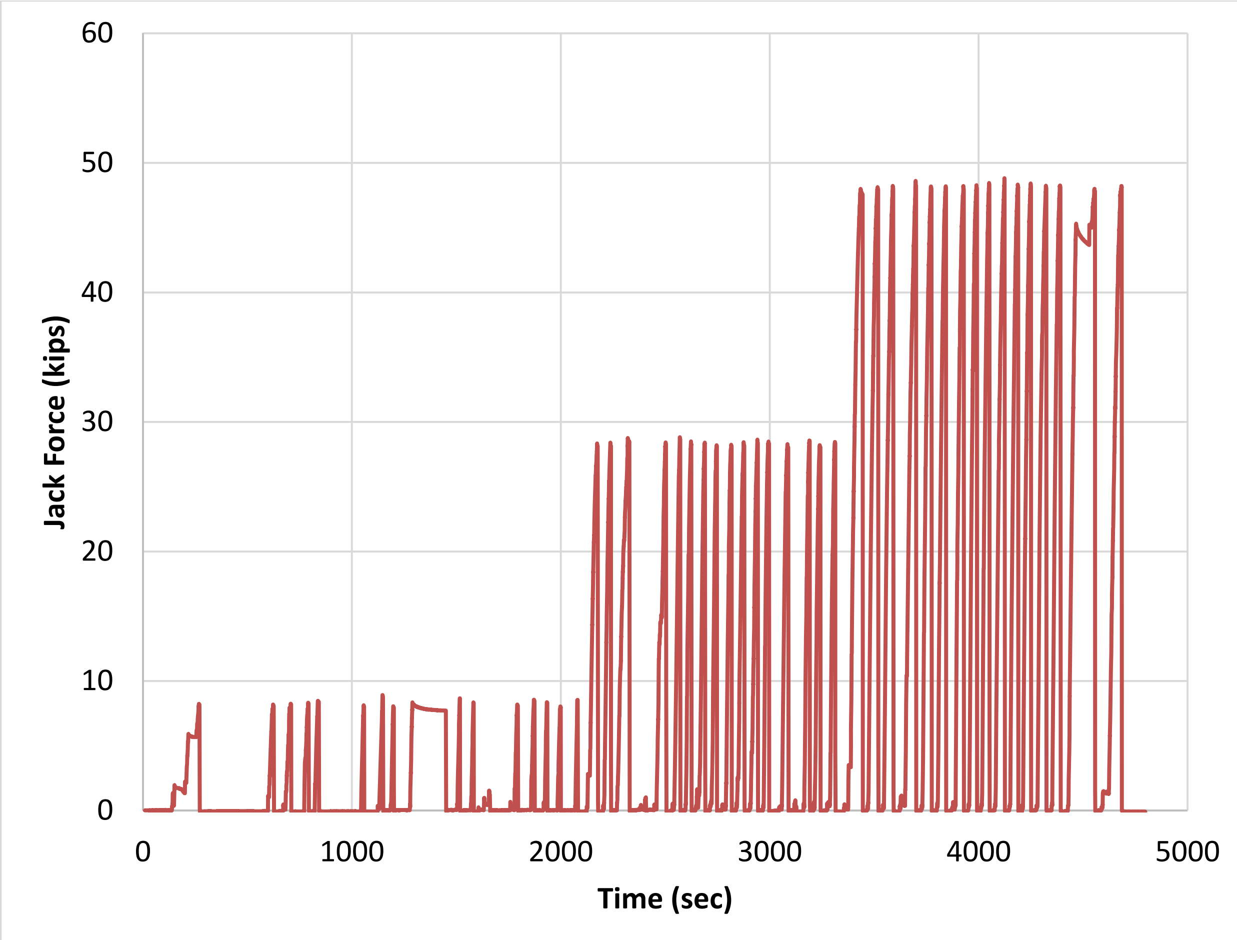


Figure 6.26. Jack force history showing increasing load cycles.

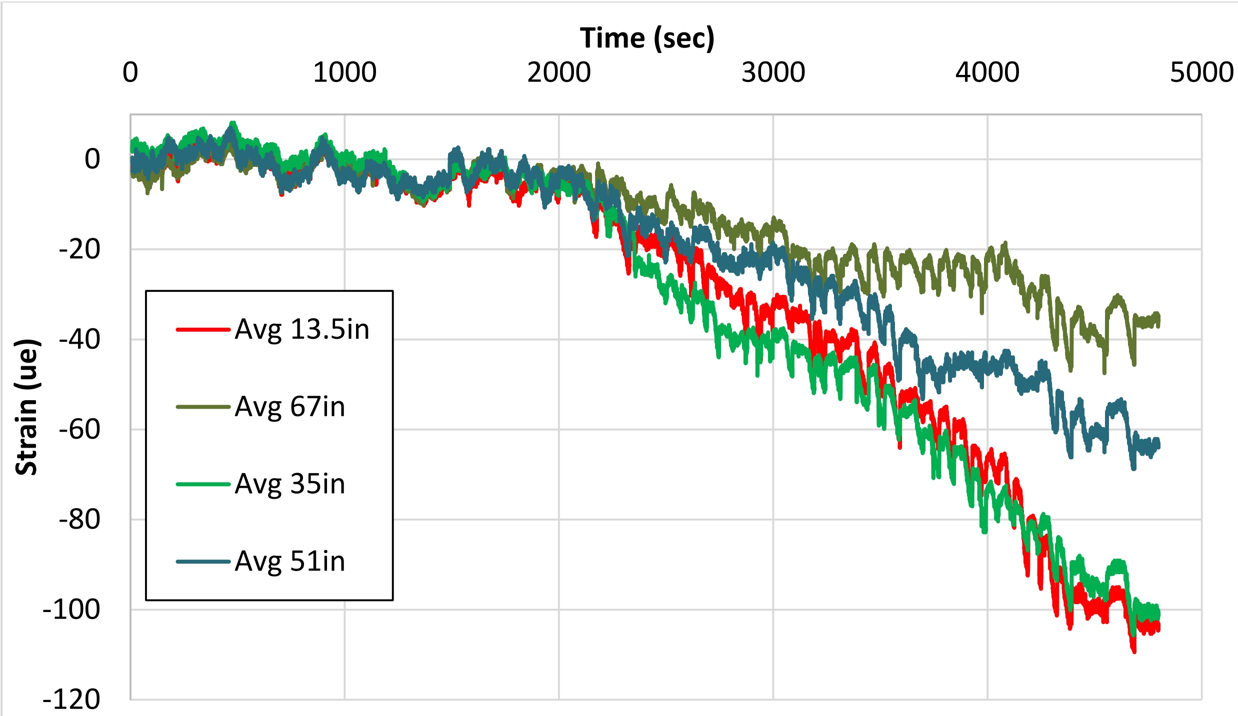


Figure 6.27. Average strain history in the pile during stressing.

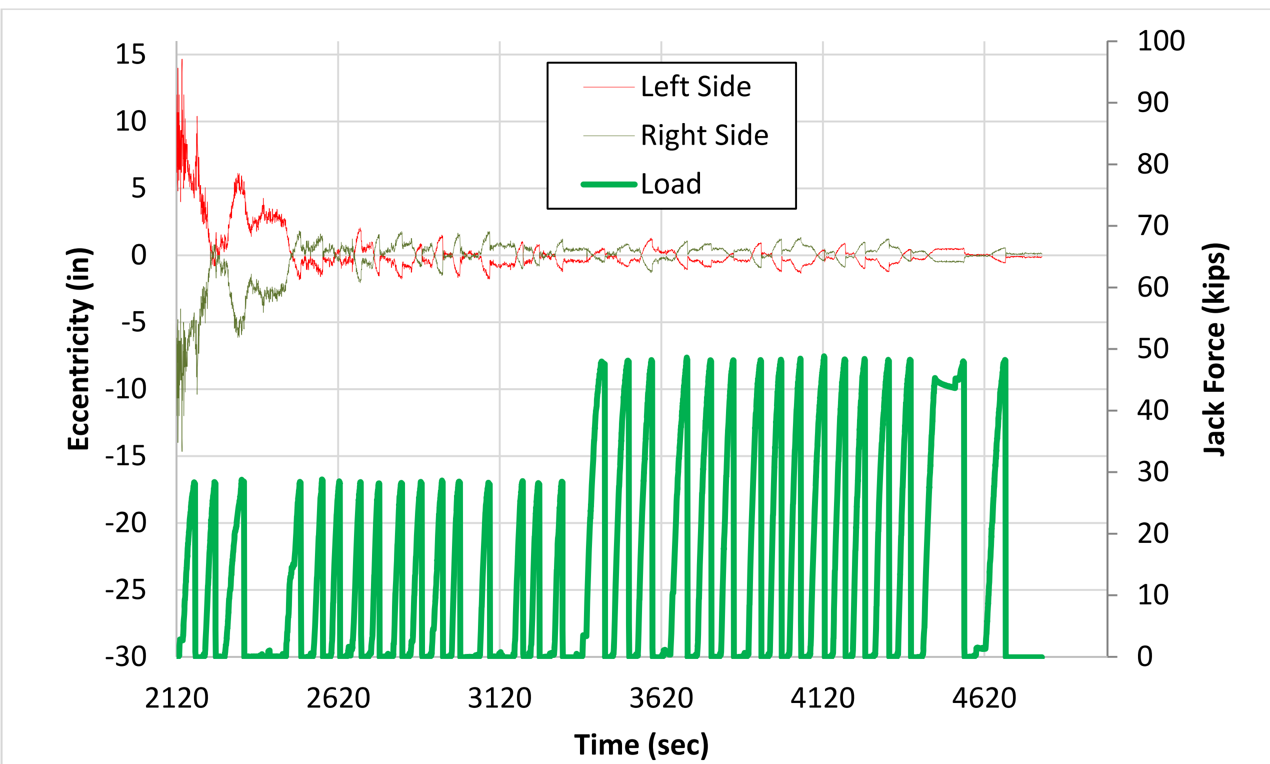


Figure 6.28. Eccentricity induced during stressing minimized by stressing order.

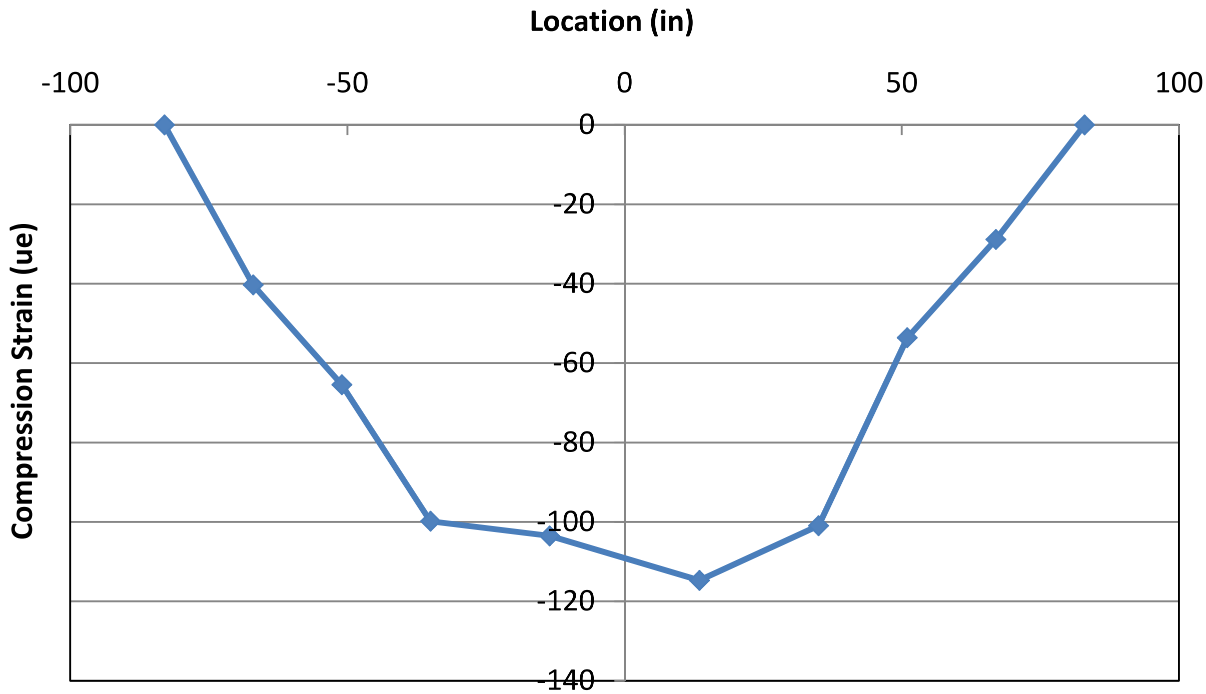


Figure 6.29. Final strain distribution in the pile near the splice.

After stressing, the strands extending from the jacking end (top of spliced pile) were cut which was necessary to permit unobstructed access to the threaded duct ends for grouting (Figure 6.30). Alternately, strands would be cut prior to driving and grouting would then be performed after driving from a lower elevation.

The jacking plate/header plate was also removed at that time to further facilitate access.

Figure 6.30. Cutting excess strand length before attaching grout fittings.

Each duct was fitted with a 3/4in pipe nipple. One duct from each side was additionally fitted with a ball valve through which the grout was pumped (Figure 6.31). The same post tension tendon cable grout previously used (product information provided in Appendix A) was pumped into each of the four side panels. As with the previous 24in spliced pile specimen, all four ducts from each side of the pile were piped together to serve as one grout circuit (four circuits total). This meant that grout could be injected into any one of the ducts in the circuit and returned out the other three.

As each of the other three ducts confirmed return flow of grout (Figure 6.31) it was then capped to direct flow to the remaining ducts. Grouting was performed using a ChemGrout model CG-550P grout mixing and pumping unit. The fully grouted pile is shown in Figure 6.32.

The following week, the grout fittings were removed and the strands were all cut flush for pile driving (Figure 6.33).

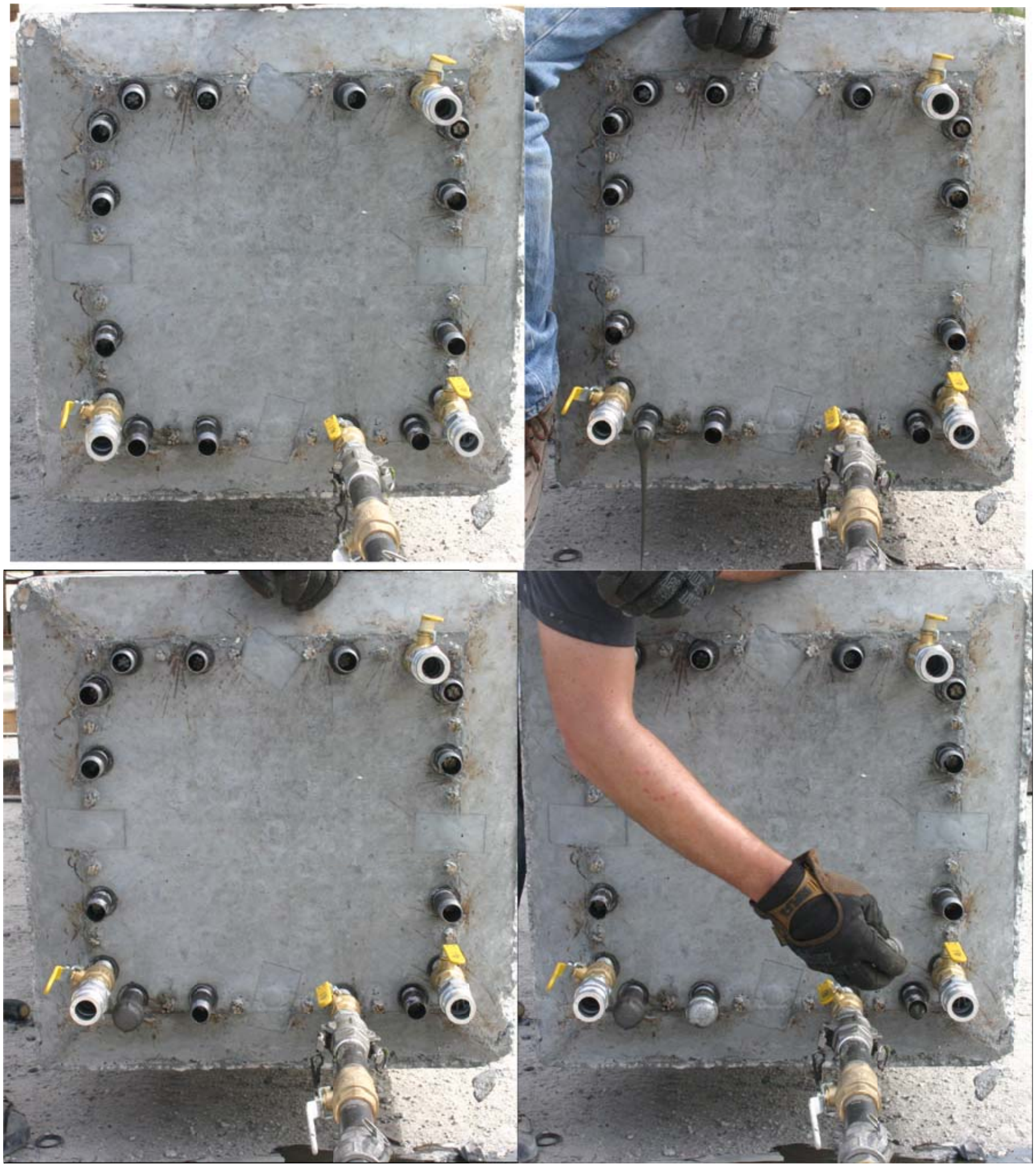


Figure 6.31. Grout pumped in one of the four ducts in the circuit until it flowed from the other three and capped.

Figure 6.32. Grouting fourth circuit (top) as last duct shows grout return; all others capped.

Figure 6.33. Upper pile end with all grout fittings removed and strands cut flush for driving.

6.4 Pile Driving Demonstration

The performance of the concept splice was demonstrated via a side-by-side comparison with a one piece pile of similar length. The one piece control pile was driven as part of a routine test pile program at the I-4 / Deer Bridge wildlife crossing near Deland, Florida using an APE Model 46-32 single acting diesel hammer. The Plans sheets showing the pier layout, soil borings, and pile driving logs can be found in Appendix E.

6.4.1 Test Pile Installation

The closest of the test piles to the demonstration splice specimen was Pile 1 in End Bent 3-3. This pile was 115ft in length and was driven on April 21, 2015 through 85ft of fine sand and silt terminating will refusal conditions in limestone. Test piles and production piles were driven with a 13.75in plywood pile cushion to control tension stresses in the early stages of driving. Figure 6.34 shows the compression and tension stresses recorded by pile driving analyzer (PDA). Tension stresses are shown from two PDA outputs: TSX denotes the worst case tension stress that occurred anywhere in the pile and TLS indicates the stresses specifically extracted from a position in the pile 30ft from the top which corresponded to the splice location for comparison. Note tension stress never exceeded 1ksi nor the maximum allowable tensile stress (1.25ksi for 6ksi concrete) and the maximum tension stress TLS was the same as TSX for most of the driving. This confirmed that a splice in the upper third of the pile would experience the worst cast tension stresses and provided the rationale for selecting that location for the splice in the demonstration splice pile specimen.

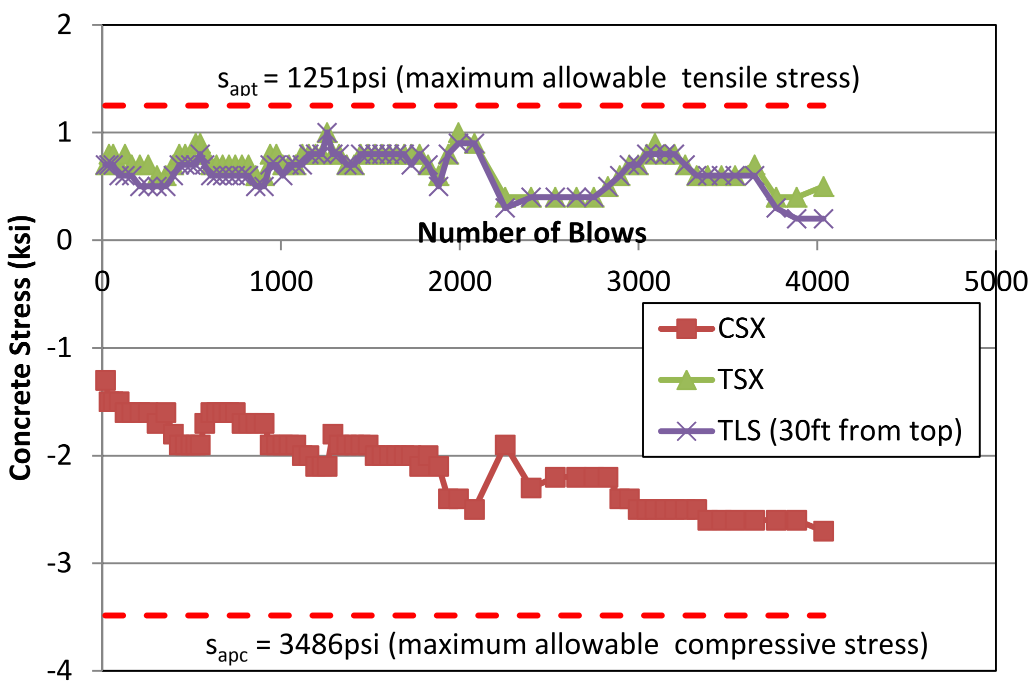


Figure 6.34. Driving stresses in test pile used as comparative control (End Bent 3-3, Pile 1).

6.4.2 Splice Pile Installation

The demonstration splice pile was designated as Pile 1-1 as it was directly adjacent Pile 1 in End Bent 3-3 opposite Pile 2 and was not used as part of the bridge support. The pile length was 100ft as noted earlier, 15ft shorter than the adjacent control pile (Pile 1). Figure 6.35 shows the as-delivered splice pile. No attempts were made to patch the chipped portion of the lower pile segment.

Figure 6.35. Splice pile specimen delivered on-site (left); damaged corners (right).

All piles on this bridge were placed into a 25ft preformed hole which expedited the driving process while minimizing tensile stresses required to penetrate the upper soils. Figure 6.36 shows the pile after placement in the preformed hole prior to driving. Figure 6.37 shows the entire bridge pile groupings.

Figure 6.36. Pile in template prior to driving (left); close-up of splice prior to driving (middle and right).

Figure 6.37. Deer Bridge piles looking northeast (End Bent 3-1 in foreground, End Bent 3-3 in background, splice pile in leads driven to the left / northwest of Pile 1, EB 3-3).

To introduce tensile stresses that would approach or even exceed the maximum allowable stress, a thinner plywood pile cushion (11.75in) was used at the onset. Recall, 13.75in was used on the test piles which balanced driving efficiency and controlled excessive tension stress. The hammer used to drive the splice pile specimen was the same as that used for the test pile and had four fuel settings to control stroke height and the associated energy. A chronology of the driving progression is presented for the splice pile specimen:

- Starting with fuel setting one, the splice pile was driven from 25 to 44ft of embedment (286 blows) with tensile stresses as high as 1.34ksi which occurred at the splice location (TLS). The average tensile stress for this portion of the drive was 1.25ksi. Average compressive stress was 1.96ksi.
- To increase driving efficiency, an additional 7.5in of cushion was added (lengthens load pulse and lowers tension stress) which allowed the hammer to be run at higher energy levels. An additional 1195 blows were imparted starting with fuel setting 1 (from 44 to 52ft), fuel setting 2 from 52 to 61ft, fuel setting 3 from 61 to 62ft, and fuel setting 4 from 62 to 76ft. Average tensile stresses were 0.72, 0.80, 0.92, and 1.12ksi for fuel settings 1, 2, 3, and 4, respectively. Compressive stresses, however, were only 1.86, 2.12, 2.21, and 2.36ksi, again respectively, which were not likely to achieve pile capacity without another cushion change to thereby increase compression stress (and tension stress). Note as the pile became more embedded, tensile stresses were expected to decrease.
- To further increase energy transfer into the pile, the pile cushion was removed and replaced with a thinner 9in cushion. The 19in (11.75in + 7.5in) original cushion thickness was found to have reduced to 14.5in. An additional 2036 blows were applied to the pile immediately using fuel setting 4. The pile was driven from 76 to 95ft when driving was interrupted to remove the template allowing it to be driven an additional 4ft. Driving stresses for this portion of the drive started at 0.9ksi tension and 2.6ksi compression and concluded at 0.3 and 3.6ksi, respectively. Pile driving records for both the test and splice piles can be found in Appendix E.
- At the end of drive, the PDA estimated pile capacity was 1660kips with 131blows/ft. In comparison, the test pile was driven to 1400kips with 150blows/ft. In all, the pile withstood 3231 blows with no detrimental effects.

The PDA recorded tension and compression stresses are shown in Figure 6.38 for the splice pile specimen. Note the allowable tensile and compressive stresses were exceeded for the experimental pile; under normal pile driving conditions these limits would not have been intentionally ignored.

During each pile cushion adjustment, the pile was visually inspected for obvious damage; none was noted. Figure 6.39 shows the pile at each cushion change (286 and 1766 blows).

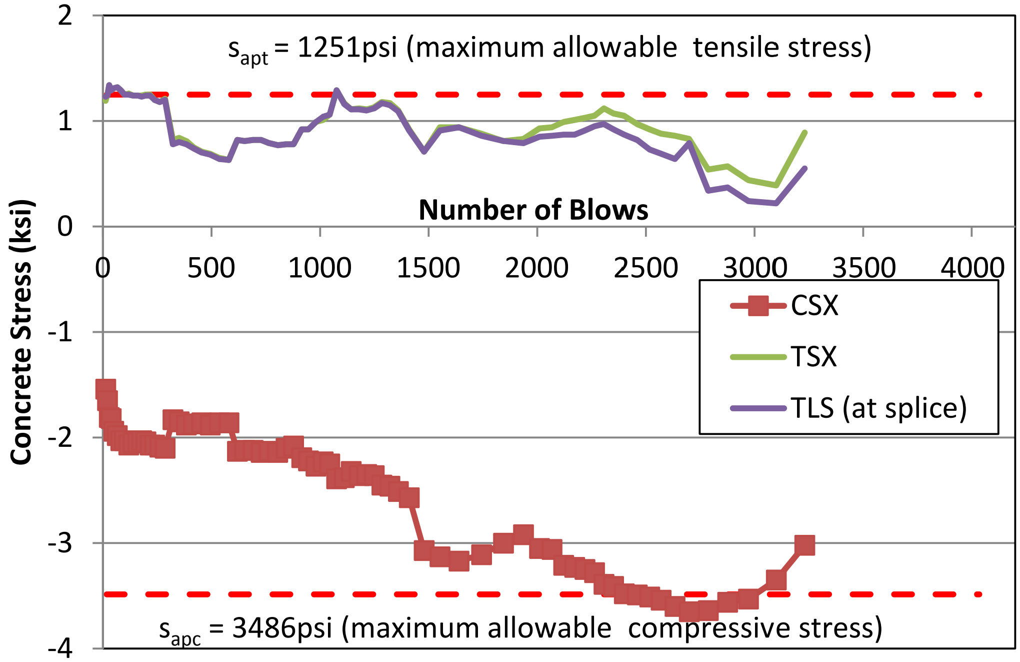


Figure 6.38. Driving stresses in splice pile specimen (End Bent 3-3, Pile 1-1).

Figure 6.39. Visual inspection of splice at each cushion change (added 7.5in at 44ft, left; replaced with 9in at 76ft, right).

The integrity of the pile was continuously monitored via the BTA integrity factor which registered an average value of 97.4 out of 100 (min 90; max 100). In comparison the one piece control pile registered an average of 99.9 (min 89.4; max 100). The last visual inspection (prior to going underground) showed a hairline crack in the epoxy (Figure 6.40). While the crack was completely closed, it appeared to have initiated from the chipped edge which was expected given the notched shape and the associated stress concentrations with such a shape.

Figure 6.40. Visual inspection of all sides of the pile showed hairline crack in epoxy.

6.5 Lessons Learned

Each splice pile tested provided usable feedback to further the advancement of the overall splice concept. For the 24in pile driving demonstration specimen the following was concluded:

- Fewer ducts did reduce the number of components, the casting yard installation time, splice preparation time, and the number of strands stressed during the splice process.
- Self-consolidating concrete was used to cast pile so no direct comparison could be made with regards to the reduced congestion in the cage. However, the cage appeared to be more open and any traditionally used concrete vibrator would have been applicable.
- The use of screw cap chucks removed all tolerance variations and the strands were easily inserted in both the upper pile segment (with shim washers) and the lower pile. *Recall the lower pile segment does not use the tolerance reducing shims.*
- Grouting was still slow even though larger 0.6in chucks were used. No dedicated pathways around or through the wedges were provided which should be considered.
- The use of neoprene gaskets around the alignment dowels (in the preformed recesses) proved to withstand the 500psi grout pressure that was applied by the pump. No grout leaks were apparent.
- The new alignment dowels had much tighter tolerance than earlier versions which meant alignment needed to be closely watched to prevent the dowels from catching the face of the lower pile face. The lower pile face threaded duct connectors should be internally tapered (e.g. ¼in chamfer) to prevent this concern and automatically align the pile halves.
- The additional stiffness of the 0.6in strands more easily overcame the wedge spring resistance making the splice process progress smoothly.
- Simple vice-grip clamps were used on each strand in lieu of specialized clamps which sped the upper pile preparation process (however not recommended for shipping).
- Inadvertent damage at the splice face is inevitable. The type of damage experienced was perhaps the worst type of stress concentrator for the epoxy. Even though, the pile drove without notable concern from the PDA operators. With this experience a small chamfer should be considered to lessen the likelihood of chips recognizing that the lower pile segment would first be driven. However, the standard chamfer is not recommended as it nearly replicates the chip in the tested pile. Suggested pile extraction techniques are outlined in the procedure below to minimize pile damage.
- A two-piece, separable splicing header could be considered that can be disassembled prior to pile removal from the bed to minimize / prevent the damage experienced.
- Driving the pile with wet epoxy would eliminate cracking and increase long term durability by forming the seal after drive. The driven demonstration pile was essentially cracked after the first 283 blows due to the higher than allowable tensile stresses. For production applications, however, staying within FDOT allowable tension limits would also prevent cured epoxy from cracking.

- While the research team was well versed in the steps to perform the splice, slip-ups made it apparent that a comprehensive checklist should be used especially as each progressive experimental splice employed more considerations and refinements.

6.6 Splicing Checklist / Procedure

- Detensioning (Pile removal)
    - When removing pile from bed, remove all duct fastening bolts from both sides of the splice header plate prior to detensioning.
    - After the strand have been cut roughly in the middle between pile halves, tap the strands with a hammer in all directions (up, down, left, right) to loosen concrete bond between the strands and header.
    - During pile removal lift one end of one pile half that is away from the splice header first and then set back down one or two times to systematically loosen the splice header from the pile halves.
    - Do not allow / discourage dragging the pile down the bed in an attempt to separate the pile halves; lifting and lowering will usually be sufficient (above).

      Note a separating header plate is being considered that may reduce damage associated with separation problems.

- Prepare splicing faces and jacking end
    - Grind off all prestressing strand stubs from the ends of each side of the splice faces.
    - There are three faces to consider: the two splice faces and the upper jacking end face (top of upper pile). *The bottom end of the lower pile segment should be prepared using standard pile preparation techniques and is not considered as part of this procedure.*
    - Use a straight edge to ensure that no high points exist on either of the splice faces. Any high spot or bump will attract load and potential cause a local failure or spall. Strands should be ground to be slightly below the slice surface plane.
    - The edges of the splice faces will always have excess concrete that also will need to be removed making sure that the entire edge also causes no obstruction to the straight edge. *Use of a small chamfer ¾ by ¾in (discussed earlier) will minimize this effort.*
    - The top face of the upper pile has a jacking plate that serves two functions: (1) during casting it aligns the ducts relative to the strands and (2) during splicing it can provide a bearing plate. This plate can be removed prior to splicing while still on the ground to removed prestressing strand stubs completely and the plate can then be reinstalled for jacking. Or, the plate can be abandoned and a smaller

bearing plate can be incorporated into the stressing jack. In either case, cleaning the pile face on the ground prior to stressing is preferred.

- Preparing upper pile half
  - The upper pile segment can be prepared prior to shipping to the project site or can be prepared on site.
  - Prepare splicing strands involves cutting them to length and softening / grinding the sharp edges caused by cutting. This can done either individually in advance, or by directly pulling strand from spool.
  - Before inserting strand in upper pile segment soften the leading edge of the strand with a grinder to minimize the potential of snagging in the wedges during insertion.
  - The length of strand should extend sufficiently out the top of pile and provide enough length to fully penetrate into the lower pile ducts. The length can be conservatively set to be the upper pile segment length plus 15ft.
  - Install alignment dowel with 1/8in neoprene gasket into the duct openings and make sure no threads are exposed. The strand can be inserted through the alignment dowel after it is installed or the alignment dowel can be slid onto the strand and installed after strand is inserted.
  - With the strand exposed the correct length out the bottom of the pile, clamp the strand with an external fixture to prevent upward/inward movement of the strand during shipping or splicing operations. For shipping, vice-grips are not recommended as they may be inadvertently dislodged or stolen. A bolt-on clamp should be used for shipping but the more quickly removable vice-grips (or similar) should be used during splicing for ease of removal.
  - Recall the strand can only go one direction in the upper pile segment. Therefore, predetermine the length of strand that will be exposed out the bottom end. Staggering the strand lengths by an inch is a reasonable approach as no two strand will touch at the same time. However, the 10ft exposed length provides enough flexibility to bend and insert by hand. If the strand is mistakenly pushed in too far, then fully remove the strand out the top of the pile and reinstall with the correct exposed length and affix the clamp.
- Splicing
  - Epoxy can be applied to the splice faces prior to hoisting the upper pile half to prevent unnecessary workplace harm to the field technicians. For horizontally spliced piles epoxy can be applied later.
  - Lift the upper pile and suspend over lower pile segment oriented with the top of bed faces aligned.
  - Lower the upper pile while aligning each strand with the corresponding duct in the lower pile. Take care to prevent crossing the strands.

    - As the pile comes within a couple inches of making contact, adjust alignment to ensure the dowels do not catch the lower pile face (a slight redesign of the lower pile threads will eliminate this concern as discussed earlier). Small movements ca be performed by hand.
    - Lower until contact is achieved but maintain tension in the crane connection.
- Post tensioning
    - The order of strand stressing should be balanced such that eccentricity is minimized / controlled.

      The orders used for the specimens in this study also stressed all strands with the same anchor locations. For example, the last pile had four staggers (positions) and four strands with each location (one per side). Therefore, opposite sides of the pile were stressed for each position and then systematically went to the next set of four with the same location.

    - Stress strands in at least two stages (preferably three). The first stage should be 1/3 the final jacking load and must exceed 10kips.
    - After all strands have been stressed, cut all post tension strands flush for pile driving.
- Grouting
    - Depending on the specific grout duct configuration / circuit design, install 3/4in NPT pipe nipples in each of the ducts.
    - Pipe caps should be available to stop grout flow once return has been demonstrated. One fewer than the number in the grout circuit is required.
    - Connect the grout inflow hose to one of the circuit ducts and pump grout until flow return is confirmed from the rest of the ducts in that circuit.
    - Cap each duct as grout flow is observed.
    - When all ducts are grouted, disconnect grout hose and move to the next circuit.
    - Caps can be removed and re-used for the next circuit (vertical pile case).
    - When all circuits have been grouted, remove all grout fixtures (pipe nipples, caps, etc.). Splice is complete.

While the design of the splice does not depend on the grout bond between the ducts and the strand, there is benefit received after the grout cures and strands become fully bonded. However, grouting can be performed after driving when the pile top is nearer the ground surface and easier to access. Similarly, it is envisioned that pile driving can commence directly after splicing while grout is still fluid (or ungrouted) and epoxy is uncured. These conditions were not tested within the timeframe of this study.

Chapter 7: Conclusions

Structures such as bridges or tall buildings often require deep foundations in order to reach soil or rock strata capable of resisting the associated high loads. In Florida, concrete elements such as driven piles, drilled shafts or other cast-in-place alternatives provide a natural choice. This is somewhat in response to the economy of concrete in Florida, but for marine structures, concrete provides excellent durability via the concrete cover that protects the reinforcing steel. In some cases, however, bearing layers are too deep for precast piles due to limitations on trucking lengths and lifting weights. In such applications, drilled shafts have an advantage, but drilled shafts are not well-suited for all soil conditions. As a result, longer precast concrete piles must be spliced from shorter, more easily transported segments. Historically, these splices have presented problems.

The most successful and robust concrete pile splice has been mechanical splices that cast into the ends of the piles some form of steel connection detail where a key, bolts, or pins fasten the two segments together in a fashion more aligned with structural steel connections. These connections, while effective, must transfer tension stresses during driving from one pile segment to the next via reinforced concrete concepts (i.e. development length of large rebar cast into the ends of each pile segment). As a result, prudent state specifications restrict tension stresses to less than half that allowed an unspliced prestress pile. This study investigated the use of an alternative approach that incorporated post tensioning the two splice pile segments together. The concept eliminates the limitations on tension stresses during driving.

In the process of developing the concept splice design, numerical modeling, laboratory testing, and full scale tests were performed which culminated with a pile driving demonstration of a spliced pile specimen.

7.1 Laboratory Scale Testing

Laboratory tests both verified acceptability and helped refine the design for the concept splice. These included both individual component testing and evaluation of prototype splices of 14in square, 20ft long prestressed piles. In all, four 14in piles were cast and tested in four point bending; two were spliced piles comprised of two, 10ft sections and the other two were full length (20ft) unspliced piles used as controls. The results of these tests showed that concept works and could restore the full cracking moment capacity in the region around the splice. Hence, the tension capacity of the splice was shown to be comparable to the unspliced control piles.

Lessons learned from the laboratory scale testing included:

- Use of deformed ducts was shown to have a positive effect on bending capacity and stiffness where the spliced pile was stiffer than the unspliced control.
- Duct deformations using a 2in spacing performed as well if not better than 1in spacing and far better than smooth duct conditions
- Grouting can be performed from the top side of the pile which eliminates the need for anything in the cover portion of the pile thereby maintaining corrosion resistance.
- Wedge set losses can be minimized by inserting a tolerance reducing shims behind the wedges.

- Ducts can be grouped into more convenient panels for quicker field installation.

7.2 Full Scale Testing

While the size of the piles used for laboratory trials is in fact a production pile size, larger 24in piles were used for the full scale testing portion of this study. Three 40ft long, 24in square prestressed piles were cast in a commercial casting yard and transported to the FDOT Structures Research Center in Tallahassee for bending test evaluation. Of the three piles, two were full length (40ft) control piles and one was a spliced pile comprised of two, 20ft segments.

Three types of monitoring/testing were performed: (1) monitor jacking load and surface strains in the pile during splicing, (2) four point bending of the three 40 piles, and (3) three point bending of undamaged 20ft segments of one control pile half and the upper half of the spliced pile that contained twice as many strands.

Test results from the four point bending showed that the cracking moment for the spliced and unspliced piles were the same and the ultimate bending moment of the splice pile was 96% of the unspliced controls (Table 5.1). Stiffness of the splice was, however, noticeably higher where an 80kip applied was load achieved at 3.3in of midspan displacement in the spliced pile compared to 5.5in in the control piles (Figure 5.30). Additionally, permanent deformation of the spliced pile upon unloading was shown to be minimal compared to the control piles (0.4in vs 6in).

Test results from the three point bending of the upper splice pile segment showed the presence of the additional unstressed strand left in the pile increased the ultimate bending capacity by approximately 20% and increased the post cracking stiffness (Figure 5.42).

Lessons learned from the full scale testing included:

- Spliced pile bending strength of 636k-ft met the 600k-ft required strength (Table 2.1)
- Casting yard complications highlighted the need to keep preassembled duct panels more flexible or use fewer ducts per assembly.
- Concreting the splice pile could be made easier by reducing the number of splicing strands thereby using the next larger size strand (e.g. 16 – 0.6 splice strands instead of 20 – 1/2in special, etc.). This also would reduce splicing time. However, this is only more cost effective and efficient if the reduced number of strands is divisible by 4.
- Tolerances from spring back chuck movement require the use of fixed / threaded back chucks.
- Enhanced passageways should be considered to speed the grouting process or an even more fluid grout may be considered.
- For both the 2nd 14in lab scale and the 24in full scale spliced piles, a tapered insert was threaded into the bottom pile segment which helped align the pile segments. This insert did not provide full isolation from the grout ducts. A revised duct coupler should be used when grouting is to be performed while the epoxy is uncured.

7.3 Pile Driving Demonstration

The experimental components of this study culminated in a driving demonstration of a 100ft spliced pile specimen. Many of the suggested refinements from the full scale bending specimen were incorporated making the pile both easier to cast and splice. The location of the splice was selected to be in the upper 1/3 of the pile length as tension stresses are usually highest in this region. The segments were therefore 30ft (upper) and 70ft (lower).

The demonstration pile was driven immediately adjacent a production test pile for the Deer Bridge wildlife crossing on I-4 near Deland, FL. Both the test pile and demonstration pile were instrumented with PDA to monitor tension and compression stresses incurred by a single acting diesel pile driving hammer. A thinner than production pile cushion was used to exacerbate tension stresses in the demonstration pile (11.75in vs 13.75in used in the production piles). This caused tension stresses to exceed the allowable 1.25ksi reaching a maximum tension of 1.34ksi at the splice location. Trying variations in pile cushion thickness, a maximum compression stress of 3.6ksi was ultimately attained (exceeding the maximum allowable compression stress of 3.5ksi) after 3231 hammer blows.

At the end of drive, the PDA estimated pile capacity was 1660kips with 131blows/ft. In comparison, the test pile was driven to 1400kips with 150blows/ft. No detrimental effects were noted from driving despite two rather large chips on one face of the splice interface caused at the time of pile removal from the casting bed. The post tension strand connection proved to be a viable and durable mechanism for splicing concrete piles and withstanding driving stresses.

7.4 Splice Designs for FDOT Pile Sizes

Prestressed elements exhibit a transition zone at the ends where prestress is gradually increased from zero at the ends to full precompression in the concrete at some transfer length into the element. This effect is the byproduct of strand slippage near the ends during detensioning where eventually the slippage is full restrained and the concrete bond to the strand is unbroken. While the length of this transition/transfer length is known to exist, the magnitude is dependent on factors such as concrete strength, strand diameter, surface roughness, etc. The concept splice design incorporates a series of embedded post tensioning anchorages within the two pile segment ends; their positions are staggered to tailor the superposition of post tensioning stresses and the pre-existing prestress in the concrete such that the target level of prestress (e.g. 1000psi) is not exceeded.

To accommodate standard FDOT pile specifications for common pile sizes, the concept design incorporates post tensioning ducts between existing strands. The number of ducts is at most the same as the number of strands in the pile, but can be reduced in multiples of 4 where larger post tensioning strands are used (i.e. the same force is provided but with fewer, larger strands). Figures 6.1-6.3 show possible duct configurations for standard pile sections. Strand patterns using 1/2in special or 0.6in strands are preferred as this reduces congestion in the pile. Standard 1/2in strand configurations are shown only where there is no advantage to using larger splicing strands.

The spacing between anchorages can be selected on the basis of losses that are accompanied by a 0.11in wedge setting movement while maintaining a required post tensioning level (e.g. 1045 psi minimum). Table 6.1 shows an example of minimum lengths between splice anchorages and recommended values for the number of staggered locations and positions. In general, the number of staggered locations was set to be a minimum of 4 but no more than 6. Additionally, the distribution of staggers was set to be multiples of 2 or 4. Therefore, when used in pairs, the stagger locations wrap around two sides of the pile like the 14in piles discussed in Chapter 4; when used in groups of 4 (no. of strands/4), all stagger locations exist on each side like the 24in piles discussed in Chapter 5.

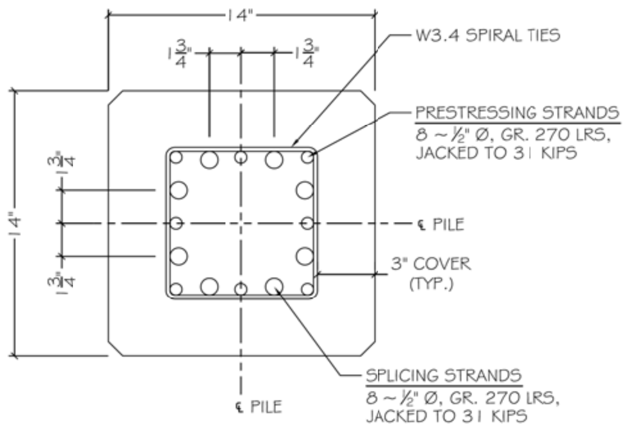


Figure 7.1. Splice configuration for 14in pile using standard 1/2in strand.

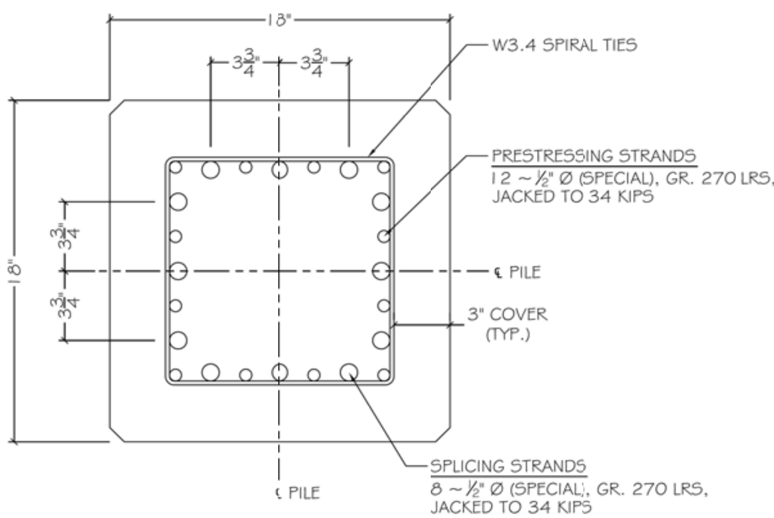
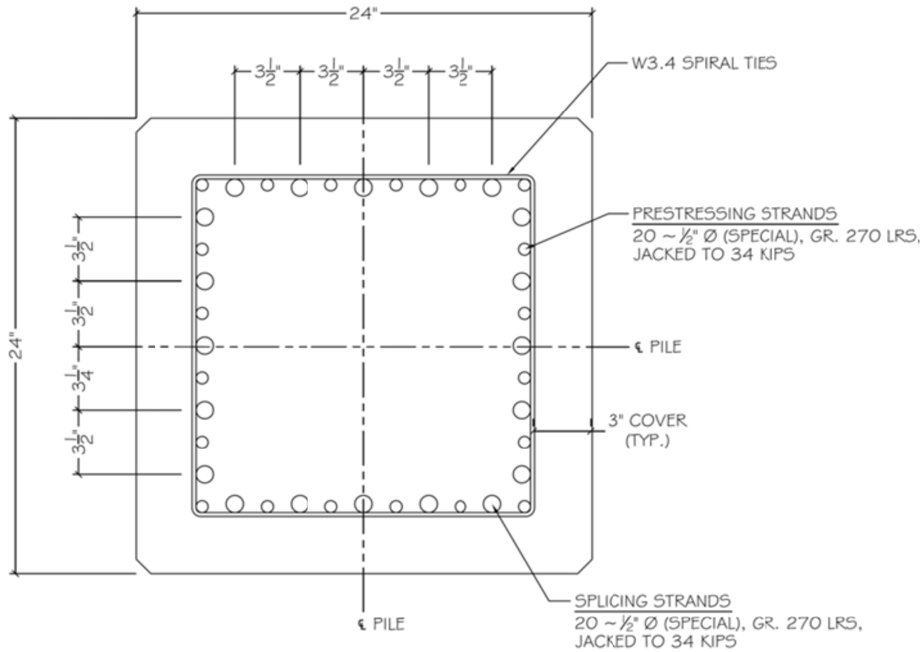


Figure 7.2. Splice configuration for 18in pile using 1/2in special strand.



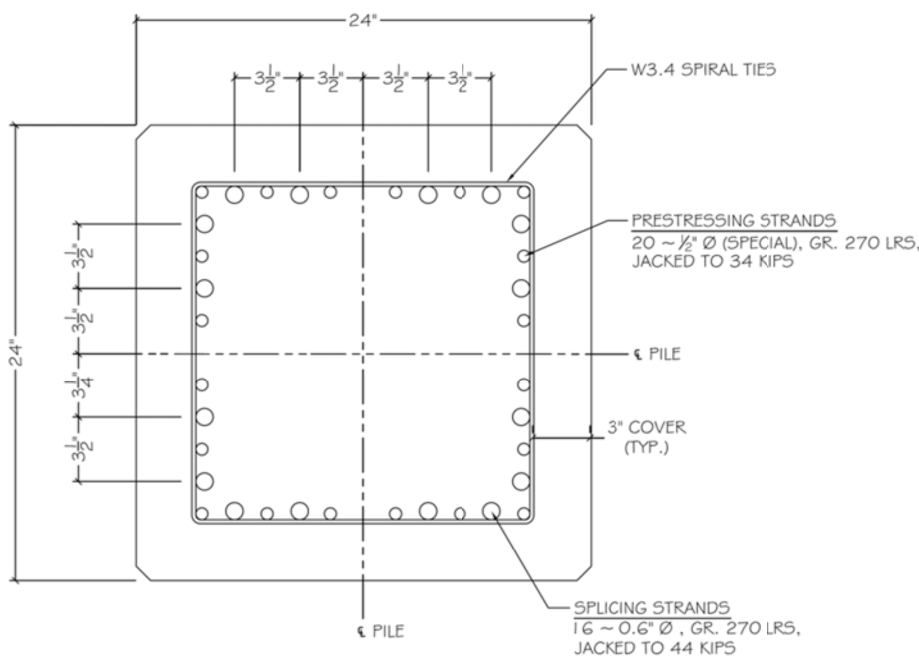


Figure 7.3. Splice configuration for 24in pile using 1/2in special strand (20 splicing strands, top; 16 splicing strands, bottom).

Table 7.1. Example determination of splice strand and anchor layouts.

Pile Size (in)	14	18	24	24
Strand Pattern	8 - 1/2in	12 - 1/2in sp	20 - 1/2in sp	16 - 0.6in
No. Strands	8	12	20	16
Fu (ksi)	270	270	270	270
A str (in2)	0.153	0.167	0.167	0.22
Jacking Force (kips)	33.048	36.072	36.072	47.52
Target Prestress (psi)	1045	1045	1045	1045
Reqd Strand Force (kips)	25.6025	28.215	30.096	37.62
Wedge Travel (in)	0.11	0.11	0.11	0.11
Min length (in)	65.6	67.8	89.1	70.9
Recommended length (in)	72	72	96	96
No. Staggers	4	6	5	4
Anchor 1	14	18	24	24
Anchor 2	29	25	36	40
Anchor 3	43	32	48	56
Anchor 4	58	40	60	72
Anchor 5		47	72	
Anchor 6		54		

7.4 Summary

The findings of this study suggest that the concept splice design can effectively restore full capacity for the purposes of withstanding pile driving installation and structural loading. Therein, full prestress levels were transferred through the splice zone which no other commercial method provides. Although the originally proposed splicing scheme considered anchorages that breached the concrete cover, the finalized version preserves the integrity of the cover and maintains the same corrosion resistance/durability of unspliced, one-piece piles.

Components used to provide the post tension anchorages for this study were fabricated at USF on a case-by-case basis depending on the casting bed, pile size, and strand layout/configurations. However, the recommended splicing schemes (Table 7.1 and Figures 7.1 – 7.3) were tailored to accommodate standard FDOT pile dimensions and strand configurations and therefore should be readily adaptable to most commercial casting yards. It is envisioned that post tension component suppliers will be able to quickly develop a line of components for the express purpose of supporting the concept splice design presented herein.

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Appendix A

Properties of Materials (QPL Approved).

- *Precast Segmental Epoxy Adhesive*
- *High Performance Cable Grout*
- *Delivery Tickets for Class V Pile Concrete*
- *½" Diameter, Grade 270, LRS Prestressing Strand*
- *Multiple Use Prestressing Strand Chucks*

DURAL 106

Slow-Setting Precast Segmental Epoxy Adhesive

Description

DURAL 106 is a two-component, 100% solids, moisture insensitive epoxy adhesive for use as a bonding agent for precast segmental box girders, bridge and other segmental construction. DURAL 106 is a non-sag paste which provides a 6 hour contact time before joining.

Class	Temperature Application Range
D	40 - 65°F (4 - 18°C)
E	60 - 90°F (16 - 32°C)
F	85 - 115°F (29 - 46°C)

Technical Information

Property	Class D	Class E	Class F
Temperature Range	40 - 65°F (4 - 18°C)	60 - 90°F (16 - 32°C)	85 - 115°F (29 - 46°C)
Sag Resistance at High Temperature	Non-Sag	Non-Sag	Non-Sag
Gel Time at High Temperature	4 hours	3 hours 40 minutes	5 hours
Compressive Yield ASTM D 695	7 days: 11,390 psi (79 MPa) 14 days: 12,350 psi (85 MPa)	7 days: 6,023 psi (42 MPa) 14 days: 8,510 psi (59 MPa)	7 days: 3,220 psi (22 MPa) 14 days: 6,150 psi (42 MPa)
Heat Deflection Temperature, 14 days ASTM D 648	>140°F (60°C)	>140°F (60°C)	>154°F (66°C)
Open Contact Time at High Temperature	6 hours	6 hours	6 hours
Bond Strength, 14 days ASTM C 882	> 3,710 psi (26 MPa)	> 5,065 psi (35 MPa)	>3,810 psi (26 MPa)
Contact Strength ASTM C 882	1,100 psi (8 MPa) at 2 days	3,623 psi (25 MPa) at 14 days	1,380 psi (6 MPa) at 14 days

Properties shown were determined at laboratory conditions.

Packaging

DURAL 106 (Class D, E, and F) is available in 3 gal (11.4 L) pre-measured kits.

Shelf Life

2 years in original, unopened container

Specifications/Compliances

Complies with ASTM C 881 Type VI, Grade 3, Classes D, E and F
Meets the requirements of Florida DOT Section 453.

Coverage Rates

12 - 13 ft²/gal (0.29 - 0.32 m²/L) at 1/8 inch (3 mm) thickness

Coverage rates are approximate and for estimating purposes only.

Surface temperature, texture and porosity will determine actual material requirements.

BONDING AGENTS AND ADHESIVES

DURAL 106

MASTER FORMAT #: 03 05 00

The Euclid Chemical Company
19218 Redwood Rd. • Cleveland, OH 44110
Phone: [216] 531-9222 • Toll-free: [800] 321-7628 • Fax: [216] 531-9596
www.euclidchemical.com

An RPM Company

Figure A.1. Dural 106 Precast Segmental Epoxy Adhesive

Directions for Use

Surface Preparation: The surface must be dry and structurally sound. The substrate must also be free of all dust, dirt grease, oil, coatings, laitance and other contaminants that would interfere with proper adhesion. The surface should be lightly sand blasted, shot blasted or water blasted with a minimum pressure of 5,000 psi (34.5 MPa). Wet surfaces must be dried. Remove all visible water with a heater and/or an oil-free air compressor. Any dust that may have accumulated between cleaning and application of DURAL 106 should be removed by an oil-free air compressor.

Mixing: Do not begin mixing until the segment is prepared for installation. Add all of Part B to Part A and mix thoroughly with a slow speed motor and "jiffy" mixer. Scrape the sides and the bottom while mixing. Do not aerate the mix. Mix for a minimum of 3 minutes or until a uniform color with no streaks is achieved. Mix complete units only and do not thin.

Application: Use a trowel, brush, mop or gloved hand to apply DURAL 106 on both segments to be joined. Apply at minimum and uniform thickness of 1/16 inch (1.6 mm). A visible bead line must be observed on all exposed contact areas. DURAL 106 should be applied completely around the pre-stressing ducts but not within 3/8 inch (9.5 mm) of the ducts. DURAL 106 should be applied within the first half of its gel time (approx. 1 hour 45 minutes). Erection, assembly and temporary post tensioning must be completed within the contact time of DURAL 106, which is approximately 6 hours from the time the epoxy is mixed. The segments should be joined with a minimum provisional stress of 30 psi (0.21 MPa) across the entire cross section. If the segments have not been joined within 70% of the contact (open) time, the operation should be discontinued, the DURAL 106 removed and fresh DURAL 106 applied. After the segments have been joined, excess DURAL 106 should be removed from the joints, where accessible. Tendon ducts should be swabbed immediately after stressing to remove or smooth out any epoxy in the conduit and to seal any pockets or air bubble holes that may have formed at the joint.

Clean-Up

Clean tools and equipment immediately following use with EUCO SOLVENT, xylene or acetone. Clean drips and spills while still wet with the same solvent. Dried DURAL 106 requires mechanical abrasion for removal.

Precautions/Limitations

- Do not thin DURAL 106.
- Apply to dry concrete surfaces.
- Do not store at temperatures below 50°F (10°C) or above 90°F (32°C).
- Do not apply to frozen or frost filled substrates or when the temperature is below 40°F (4°C) or expected to fall below that temperature within 24 hours.
- Not intended for areas subjected to prolonged or strong chemical attack.
- Protect from moisture.
- In all cases, consult the Material Safety Data Sheet before use.

Rev. 03.13

Figure A.2. Dural 106 Precast Segmental Epoxy Adhesive (continued)

EUCO CABLE GROUT PTX

HIGH PERFORMANCE CABLE GROUT

GROUTS

DESCRIPTION

EUCO CABLE GROUT PTX is formulated to produce a pumpable, non-shrink, high strength grout. It provides unparalleled corrosion protection for steel cables, anchorages and rods. EUCO CABLE GROUT PTX is extremely fluid, and cured grout is similar in appearance to concrete. EUCO CABLE GROUT PTX exhibits thixotropic properties defined in PTI specifications, and can be used to repair previously grouted cables.

PRIMARY APPLICATIONS

- Post-tensioned cables and ducts
- Grouting of tight clearances

FEATURES/BENEFITS

- Superior corrosion protection
- High fluidity for easy placement
- Non-shrink
- Exceptional strength
- Aggregate free
- Pumpable for a minimum of 2 hrs @ 90˚F (32˚C)

TECHNICAL INFORMATION

PROPERTY	RESULT @ 1.5 gal/50 lb (5.7 L/22.7 kg) mix water (unless otherwise noted)
Flow Rate ASTM C 939 modified	9 to 20 seconds initial flow *9 to 30 seconds at 30 minutes
Initial Setting Time at 70°F (21°C) ASTM C 953	8 to 12 hours (depending on material and ambient temperature)
Compressive Strength ASTM C 942	7 days: > 3,000 psi (20.7 MPa) 28 days: > 7,000 psi (48.3 MPa)
Hardened Height Change ASTM C 1090	24 hours: 0.0% to 0.1% 28 days: 0.0% to 0.2%
Plastic Expansion ASTM C 940	0.0% to 2.0% for up to 3 hours
Wick Induced Bleed ASTM C 940 Modified according to C4.4.6.1 of the PTI Guide Specification	0.0% at 5 minutes 0.0% at 3 hours
Schupack Pressure Bleed Test @1.5 gal/50 lb (5.7 L/22.7 kg) mix water	0.0% (5 minutes @ 100 psi)
Schupack Pressure Bleed Test @1.7 gal/50 lb (6.4 L/22.7 kg) mix water	0.0% (5 minutes @ 50 psi)
Chloride Permeability ASTM C 1202	28 days (30V for 6 hrs): < 2,500 coulombs

*Tested under laboratory conditions with a 30 second re-mix prior to measuring the flow.

EUCO CABLE GROUT PTX is a free flowing powder designed to be mixed with water. After mixing and placing, the color may initially appear much darker than the surrounding concrete. While this color will lighten up substantially as the grout cures, the grout may always appear somewhat darker than the surrounding concrete.

EUCO CABLE GROUT PTX

Master Format #: 03 62 13

The Euclid Chemical Company

19218 Redwood Rd. • Cleveland, OH 44110
Phone: [216] 531-9222 • Toll-free: [800] 321-7628 • Fax: [216] 531-9596
www.euclidchemical.com

An RPM Company

Figure A.3. Euco Cable Grout PTX

Availability & Shelf Life

****EUCO CABLE GROUT PTX is Made To Order (MTO) and has a 10 day lead time.** It has a 6 month shelf life in the original, properly stored, unopened package.

Packaging/Yield

EUCO CABLE GROUT PTX is packaged in 50 lb (22.7 kg) bags or pails and yields 0.54 ft³ (0.015 m³) of fluid grout when mixed with 1.5 gal (5.7 L) of potable water. Yield is 0.56 ft³ (0.016 m³) when mixed with 1.7 gal (6.4 L) of potable water.

Directions for Use

If the contractor is not familiar with standard grout placement techniques, a pre-job meeting is suggested to review the project details unique to the particular job. Refer to the PTI Guide Specification for Post-Tensioned Structures for proper mixing, pumping and placement practices.

Mixing:

Consistency	Estimated Water Content*
Fluid	1.5 to 1.7 gal/50 lb (5.7 to 6.4 L/22.7kg)
Flowable	1.3 to 1.5 gal/50 lb (4.9 to 5.7 L/22.7kg)

* Do not add water in an amount that will cause bleeding. Do not add aggregate or cement to the grout since this action will change its precision grouting characteristics.

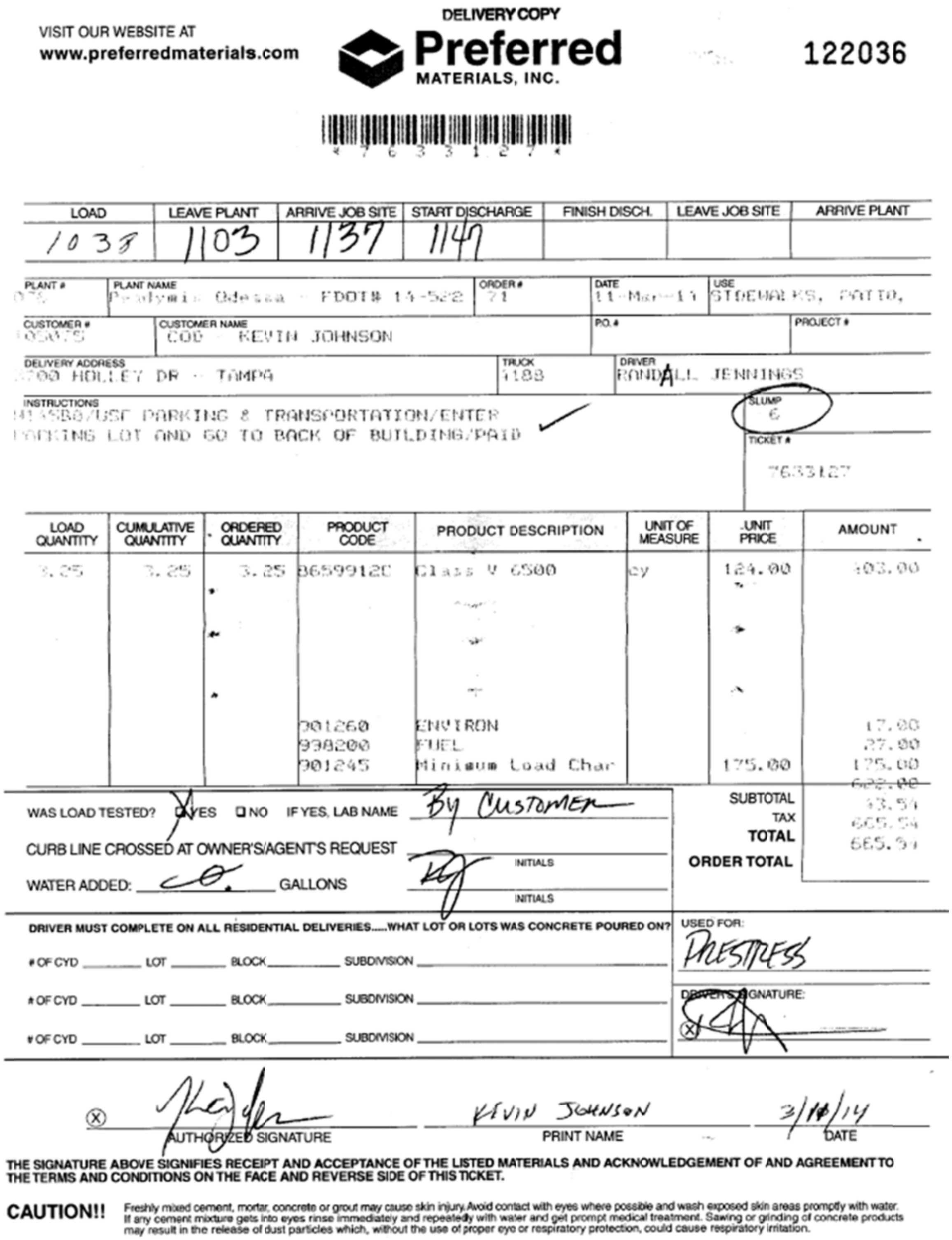
Curing and Sealing: Cure all exposed grout by wet curing for 24 hours, then with a high solids curing and sealing compound, such as Super Diamond Clear or Super Diamond Clear VOX.

Precautions/Limitations

- To minimize bleeding in vertical applications greater than twenty feet, The Euclid Chemical Company recommends a water dosage no greater than 1.50 gal/50 lb (5.7 L/22.7 kg).
- Clean tools and equipment with water before the material hardens.
- Do not add any admixture or fluidifiers.
- Do not use any more or less water than what is specified above.
- Store materials in a dry place.
- Application temperature must be 40° F (4° C) or above and remain so for 24 hours after placement.
- Employ cold weather or hot weather grouting practices as the temperature dictates.
- Rate of strength gain and setting times are significantly affected at temperature extremes.
- The Euclid Chemical Company is not responsible for stress corrosion caused by ingredients in the flushout, saturation, or mixing water, or for contaminants either in the space being grouted or from other materials used in the system.
- In all cases, consult the Material Safety Data Sheet before use.

Rev. 11.12

Figure A.4. Euco Cable Grout PTX (continued)



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www.preferredmaterials.com

DELIVERY COPY

Preferred MATERIALS, INC.

122036

* 7 6 3 3 1 2 7 *

LOAD	LEAVE PLANT	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCH.	LEAVE JOB SITE	ARRIVE PLANT
1038	1103	1137	1147			

PLANT #: 07? | PLANT NAME: Prodymix Odessa - FDOT# 14-522 | ORDER #: 71 | DATE: 11-Mar-14 | USE: SIDEWALKS, PATIO,

CUSTOMER #: 105075 | CUSTOMER NAME: COD - KEVIN JOHNSON | P.O. # | PROJECT #

DELIVERY ADDRESS: 3700 HOLLEY DR - TAMPA | TRUCK: 1188 | DRIVER: RANDALL JENNINGS

INSTRUCTIONS: #135B0/USF PARKING & TRANSPORTATION/ENTER PARKING LOT AND GO TO BACK OF BUILDING/PAID

SLUMP: 6

TICKET #: 7633127

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UNIT OF MEASURE	UNIT PRICE	AMOUNT
3.25	3.25	3.25	B6599120	Class V 6500	cy	124.00	403.00
			001260	ENVIRON			17.00
			938200	FUEL			27.00
			001245	Minimum Load Char		175.00	175.00

SUBTOTAL	622.00
TAX	43.54
TOTAL	665.54
ORDER TOTAL	665.54

WAS LOAD TESTED? ☒ YES ☐ NO IF YES, LAB NAME By Customer

CURB LINE CROSSED AT OWNER'S/AGENT'S REQUEST ______ INITIALS

WATER ADDED: 0. GALLONS ______ INITIALS

DRIVER MUST COMPLETE ON ALL RESIDENTIAL DELIVERIES.....WHAT LOT OR LOTS WAS CONCRETE POURED ON?

\# OF CYD ______ LOT ______ BLOCK ______ SUBDIVISION ______

\# OF CYD ______ LOT ______ BLOCK ______ SUBDIVISION ______

\# OF CYD ______ LOT ______ BLOCK ______ SUBDIVISION ______

USED FOR: Prestress

DRIVER'S SIGNATURE:

Ⓧ ______ AUTHORIZED SIGNATURE | Kevin Johnson PRINT NAME | 3/11/14 DATE

THE SIGNATURE ABOVE SIGNIFIES RECEIPT AND ACCEPTANCE OF THE LISTED MATERIALS AND ACKNOWLEDGEMENT OF AND AGREEMENT TO THE TERMS AND CONDITIONS ON THE FACE AND REVERSE SIDE OF THIS TICKET.

CAUTION!! Freshly mixed cement, mortar, concrete or grout may cause skin injury. Avoid contact with eyes where possible and wash exposed skin areas promptly with water. If any cement mixture gets into eyes rinse immediately and repeatedly with water and get prompt medical treatment. Sawing or grinding of concrete products may result in the release of dust particles which, without the use of proper eye or respiratory protection, could cause respiratory irritation.

Figure A.5. Concrete delivery ticket from first lab scale (14 in.) pile casting.

Preferred
MATERIALS, INC.

Delivery Ticket for Structural Concrete

Financial Project Number	N/A	Serial #	7633127
DOT Plant Number	14-522	Date	March 12, 2014
Concrete Supplier	Oldcastle Southern Group / Preferred Materials, Inc.	Delivered to	KEVIN JOHNSON
		Phone #	
Phone Number	800-331-3375	Address;	3700 HOLLEY DR
Address	11913 S.R. 54 Odessa, FL 33556		TAMPA

Truck # 4188	DOT class CL V 6500 W/HRWR	DOT mix ID 01-1025-01	Cubic yards this load 3.25
allowable jobsite Water 21.97	Time loaded 2:00 PM	Mixing revolutions	Cubic yards total today 3.25
Chloride Test Results: 0.102		Chloride Test Date: 11/12/2013	

Cement			Flyash / Slag			
American	TYPEI/ II	2440	ProAsh	F		590
source	Type	amount-lbs	source	Type		amount-lbs
Coarse agg			Air admixture			
87-089	1.50	5030	Euclid	AEA-92S		1.5
Pit num.	%moisture	amount-lbs	source	brand	Type	amount-oz.
Fine agg.			Admixture			
16-659	3.50	3300	Euclid	WR	D	303
Pit num.	% moisture	amount-lbs	source	brand	Type	amount-oz.
		0.00				
ICE	Lbs.	Gal.	Admixture			
Batch water			Euclid	6200EXT	F	75
Amount	659.7	79	source	brand	Type	amount
	Lbs.	Gal.				

Issuance of this ticket constitutes certification that the concrete batched was produced and information recorded in compliance with Department specifications for Structural Concrete

Y232481644230

CTQP Technician Identification number

Signature of batch plant operator

Arrival on jobsite		Number of revolutions upon arrival at job site	
Water added at job site(gal or lbs)		Additional mixing revs. With added water	
Time concrete completely discharged		Total number of revolutions	
Initial slump	Initial air	Initial concrete temp	Initial W/C ratio
Accept. Slump	Accept. Air	Accept. Concrete temp	Accept W/C ratio

Issuance of this ticket constitutes certification that the maximum specified water cementitious ratio was not exceeded and the batch was delivered and plaved in compliance with Department specification requirements

CTQP Technician Identification number

Signature of contractors representative

Figure A.6. Concrete mix design from first lab scale (14 in.) pile casting.

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MATERIALS, INC.

04761

* 7 5 3 6 9 4 7 *

LOAD	LEAVE PLANT	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCH.	LEAVE JOB SITE	ARRIVE PLAN
11:29						

PLANT #	PLANT NAME	ORDER #	DATE	USE
07_	R/M 57th St-Tampa FDOT#10-410	82	15-Apr-14	

CUSTOMER #	CUSTOMER NAME	P.O. #	PROJECT #
105075	COD - KEVIN JOHNSON	KEVIN	

DELIVERY ADDRESS	TRUCK	DRIVER
USF - PARKING AND TRANSPORTATION BLDG	3911	FRANK STEWARD

INSTRUCTIONS
USF HOLLY AND USF PLUM 275 to Fletcher Right to magnolia Right to Holly left 1st left Pass Laurel

SLUMP: 6
TICKET #: 7536947

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	PRODUCT CODE	PRODUCT DESCRIPTION	UNIT OF MEASURE	UNIT PRICE	AMOUNT
3.25	3.25	3.25	86599120	Class V 6500	cy	120.00	390.
			901260	ENVIRON			[illegible]
			998200	FUEL			28.
			901245	Minimum Load Char		175.00	175.

SUBTOTAL	610.
TAX	42.
TOTAL	[illegible]
ORDER TOTAL	[illegible]

WAS LOAD TESTED? ☐ YES ☐ NO IF YES, LAB NAME ____________

CURB LINE CROSSED AT OWNER'S/AGENT'S REQUEST ____________ INITIALS

WATER ADDED: ____________ GALLONS ____________ INITIALS

DRIVER MUST COMPLETE ON ALL RESIDENTIAL DELIVERIES....WHAT LOT OR LOTS WAS CONCRETE POURED ON?

\# OF CYD ________ LOT ________ BLOCK ________ SUBDIVISION ________

\# OF CYD ________ LOT ________ BLOCK ________ SUBDIVISION ________

\# OF CYD ________ LOT ________ BLOCK ________ SUBDIVISION ________

USED FOR:

DRIVER'S SIGNATURE: ⓧ

ⓧ ____________ AUTHORIZED SIGNATURE — KEVIN JOHNSON PRINT NAME — 4/15/14 DATE

THE SIGNATURE ABOVE SIGNIFIES RECEIPT AND ACCEPTANCE OF THE LISTED MATERIALS AND ACKNOWLEDGEMENT OF AND AGREEMENT TO THE TERMS AND CONDITIONS ON THE FACE AND REVERSE SIDE OF THIS TICKET.

CAUTION!! Freshly mixed cement, mortar, concrete or grout may cause skin injury. Avoid contact with eyes where possible and wash exposed skin areas promptly with water. If any cement mixture gets into eyes rinse immediately and repeatedly with water and get prompt medical treatment. Sawing or grinding of concrete products may result in the release of dust particles which, without the use of proper eye or respiratory protection, could cause respiratory irritation.

Figure A.7. Concrete delivery ticket from second lab scale (14 in.) pile casting.

Preferred
MATERIALS, INC.

Delivery Ticket for Structural Concrete

Financial Project Number	N/A	Serial #	7536947
DOT Plant Number	10-410	Date	April 15, 2014
Concrete Supplier	Oldcastle Southern Group / Preferred Materials, Inc.	Delivered to	KEVIN JOHNSON
		Phone #	
Phone Number	800-331-3375	Address;	USF PARKING
Address	1811 N. 57th Street		TAMPA
	Tampa, FL 33619		

Truck # 3911	DOT class CL V 6500 W/HRWR	DOT mix ID 01-1025-01	Cubic yards this load 3.25
allowable jobsite Water 16.80	Time loaded 11:29 AM	Mixing revolutions	Cubic yards total today 3.25
Chloride Test Results:		Chloride Test Date:	

Cement			Flyash / Slag			
American	TYPEI/ II	2425	ProAsh	F		570
source	Type	amount-lbs	source	Type		amount-lbs
Coarse agg			Air admixture			
87-089	2.40	5020	Euclid	AEA-92S		2
Pit num.	%moisture	amount-lbs	source	brand	Type	amount-oz.
Fine agg.			Admixture			
16-659	2.50	3280	Euclid	WR	D	240
Pit num.	% moisture	amount-lbs	source	brand	Type	amount-oz.
		0.00				
ICE	Lbs.	Gal.	Admixture			
Batch water			Euclid	6200EXT	F	75
Amount	691.0	83	source	brand	Type	amount
	Lbs.	Gal.				

Issuance of this ticket constitutes certification that the concrete batched was produced and information recorded in compliance with Department specifications for Structural Concrete

G 363-620-53-391-0
CTQP Technician Identification number

Signature of batch plant operator

Arrival on jobsite		Number of revolutions upon arrival at job site	
Water added at job site(gal or lbs)		Additional mixing revs. With added water	
Time concrete completely discharged		Total number of revolutions	
Initial slump	Initial air	Initial concrete temp	Initial W/C ratio
Accept. Slump	Accept. Air	Accept. Concrete temp	Accept W/C ratio

Issuance of this ticket constitutes certification that the maximum specified water cementitious ratio was not exceeded and the batch was delivered and plaved in compliance with Department specification requirements

CTQP Technician Identification number

Signature of contractors representative

Figure A.8. Concrete mix design from second lab scale (14 in.) pile casting.

Table A.1. Concrete cylinder break strengths.

Concrete	Concrete Age (days)	Cylinder 1			Cylinder 2			Average
		Force (k)	Area (sq. in.)	*f'c* (psi)	Force (k)	Area (sq. in.)	*f'c* (psi)	***f'c* (psi)**
1st lab scale casting	3	79.68	12.56	6344	80.98	12.56	6448	**6396**
	14	98.89	12.56	7874	96.79	12.56	7706	**7790**
	91	117.11	12.56	9324	118.45	12.56	9431	**9377**
2nd lab scale casting	3	71.24	12.56	5672	75.36	12.56	6000	**5836**
	10	96.01	12.56	7644	97.39	12.56	7754	**7699**
	56	113.85	12.56	9064	114.84	12.56	9143	**9104**
Full scale casting	65	150.65	12.56	11994	155.63	12.56	12391	**12193**

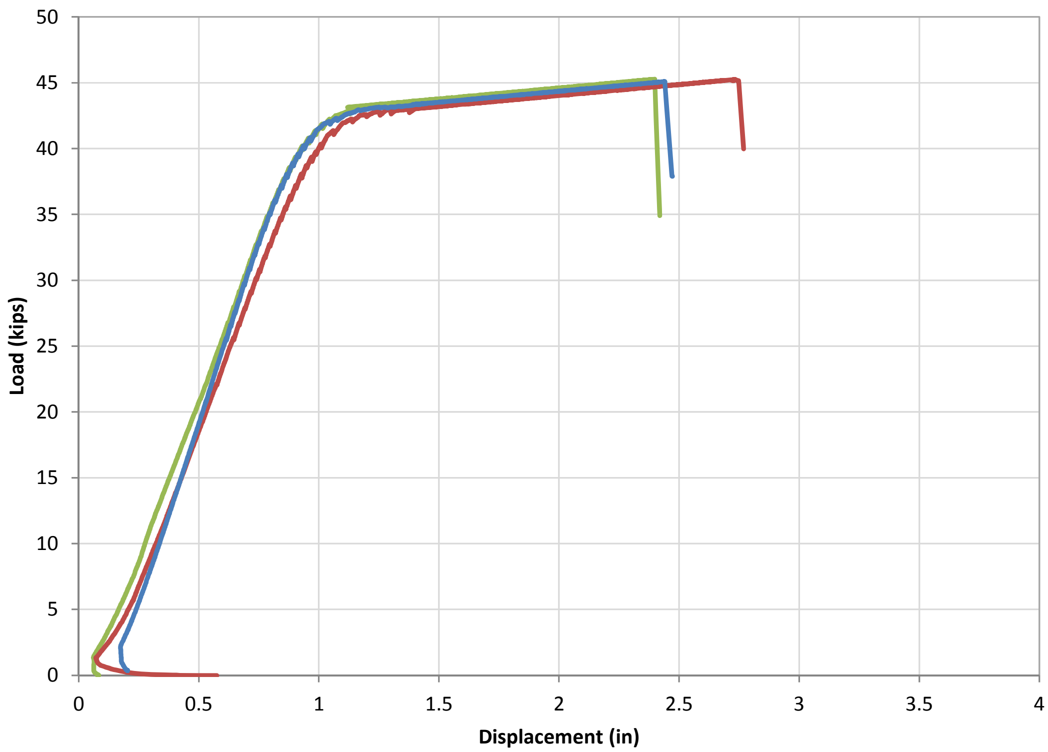

Figure A.9. Load vs. displacement for ½” LRS strands.

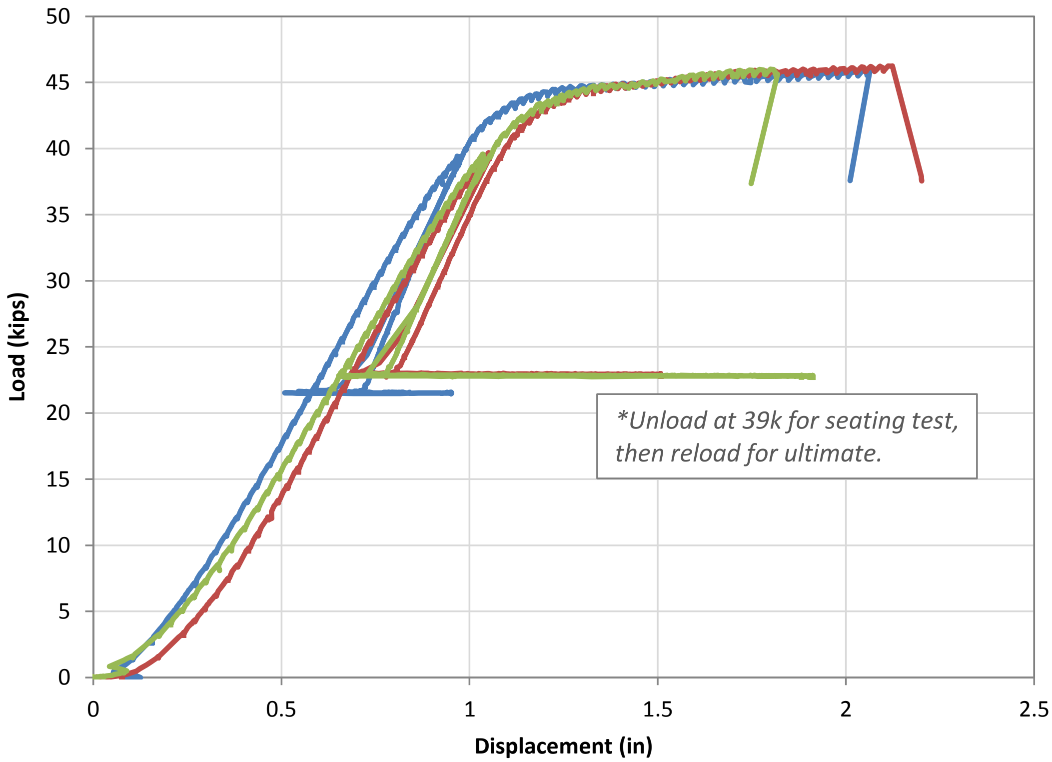

Figure A.10. Load vs. displacement for ½” special LRS strands.

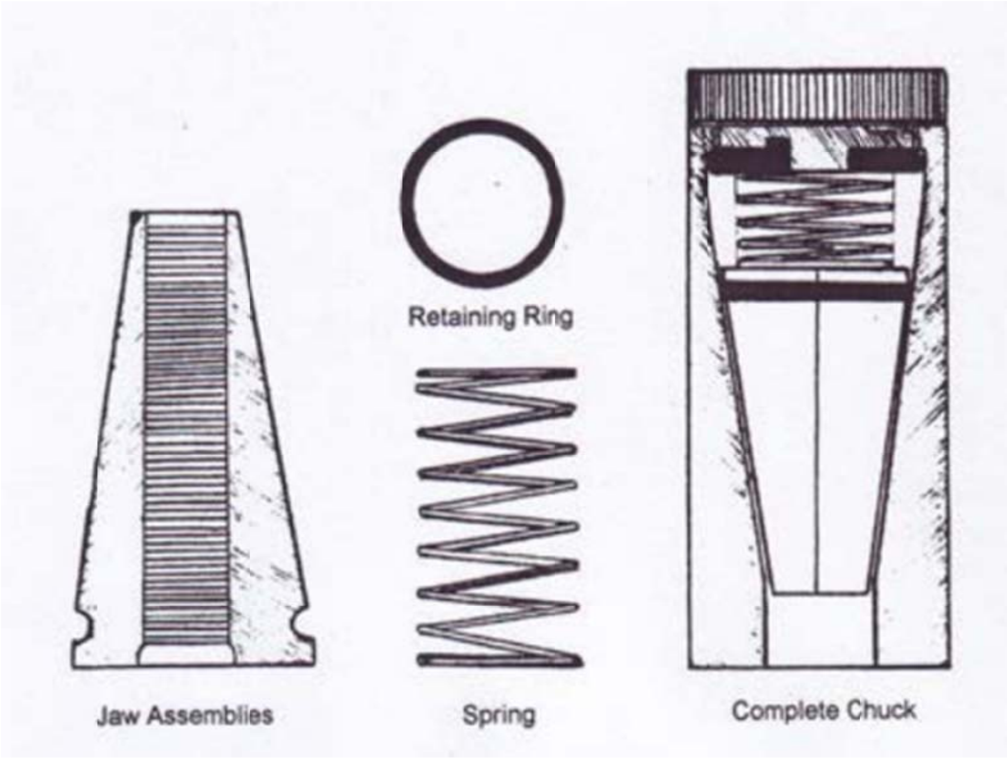

Figure A.11. PSI/PAUL strand chuck assembly.

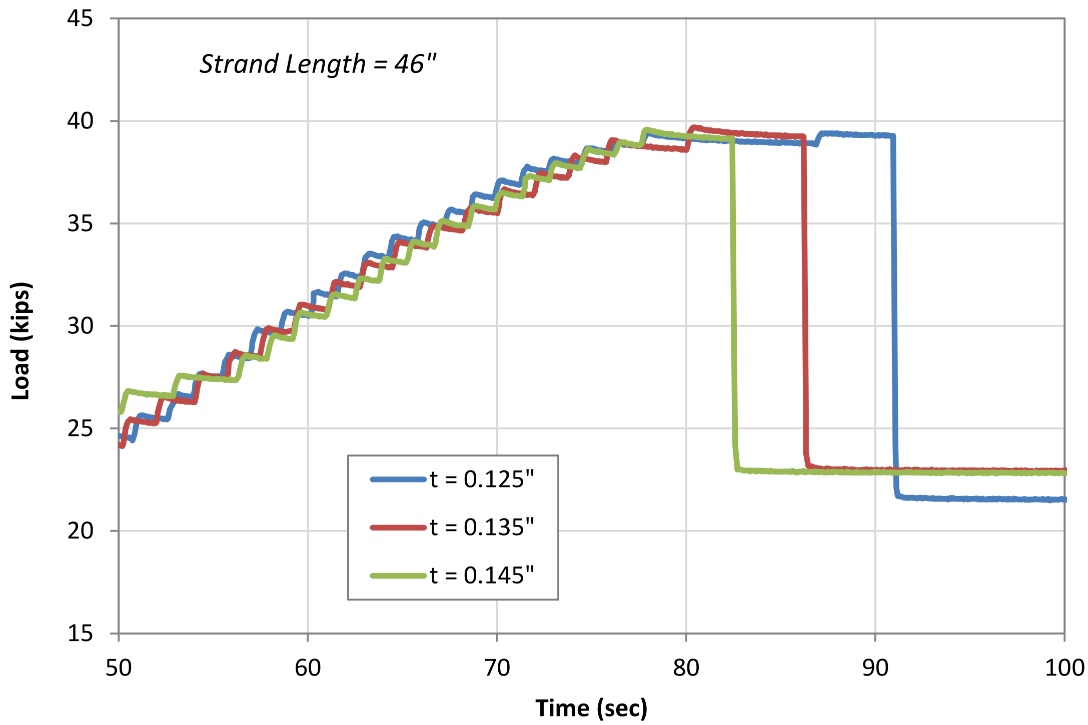

Figure A.12. Seating losses using PSI/PAUL strand chucks modified with spacers of variable thickness.

Appendix B

FDOT Specifications for square concrete prestressed piles

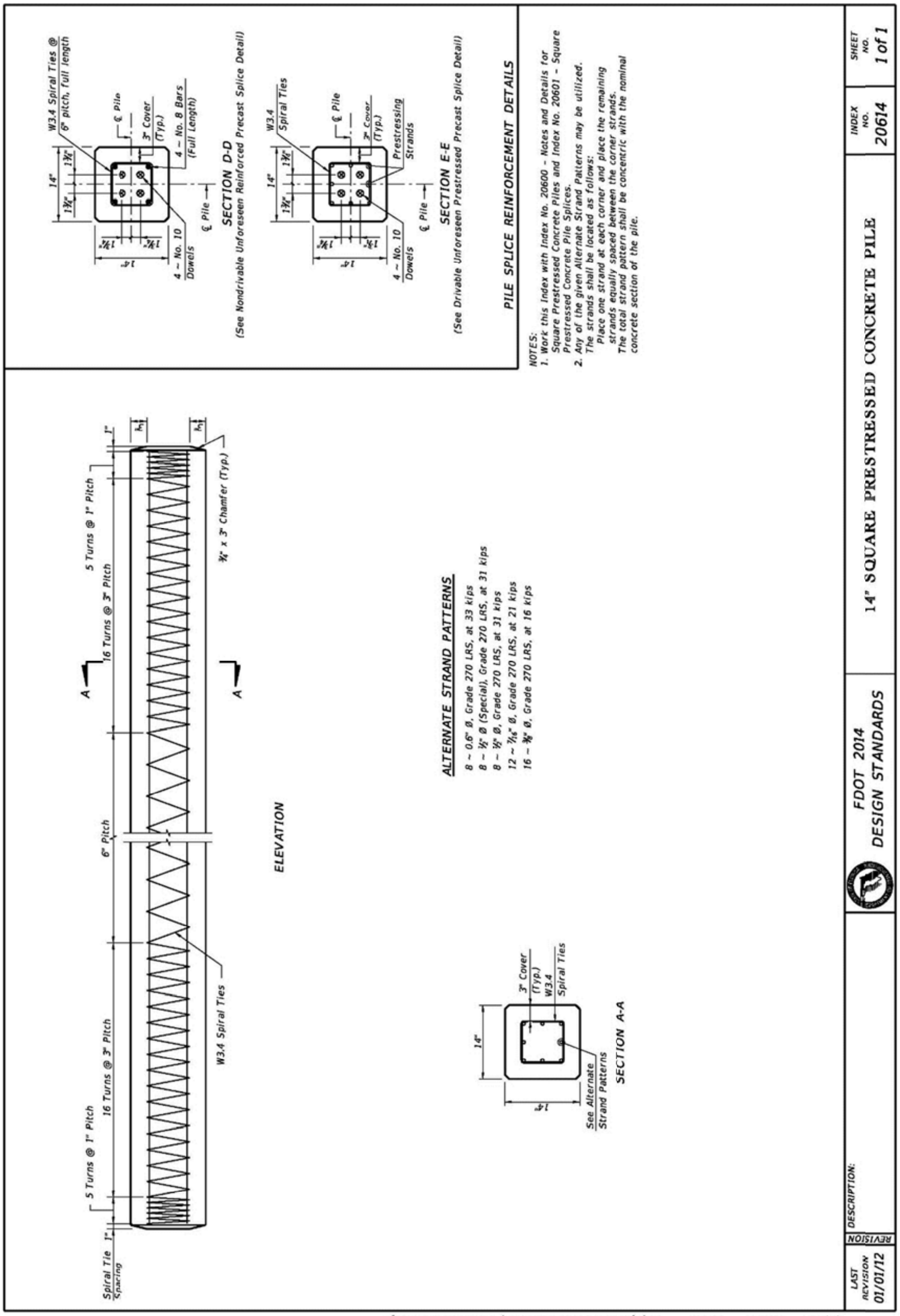

Figure B.1. 14 in. square pile (FDOT index 20614)

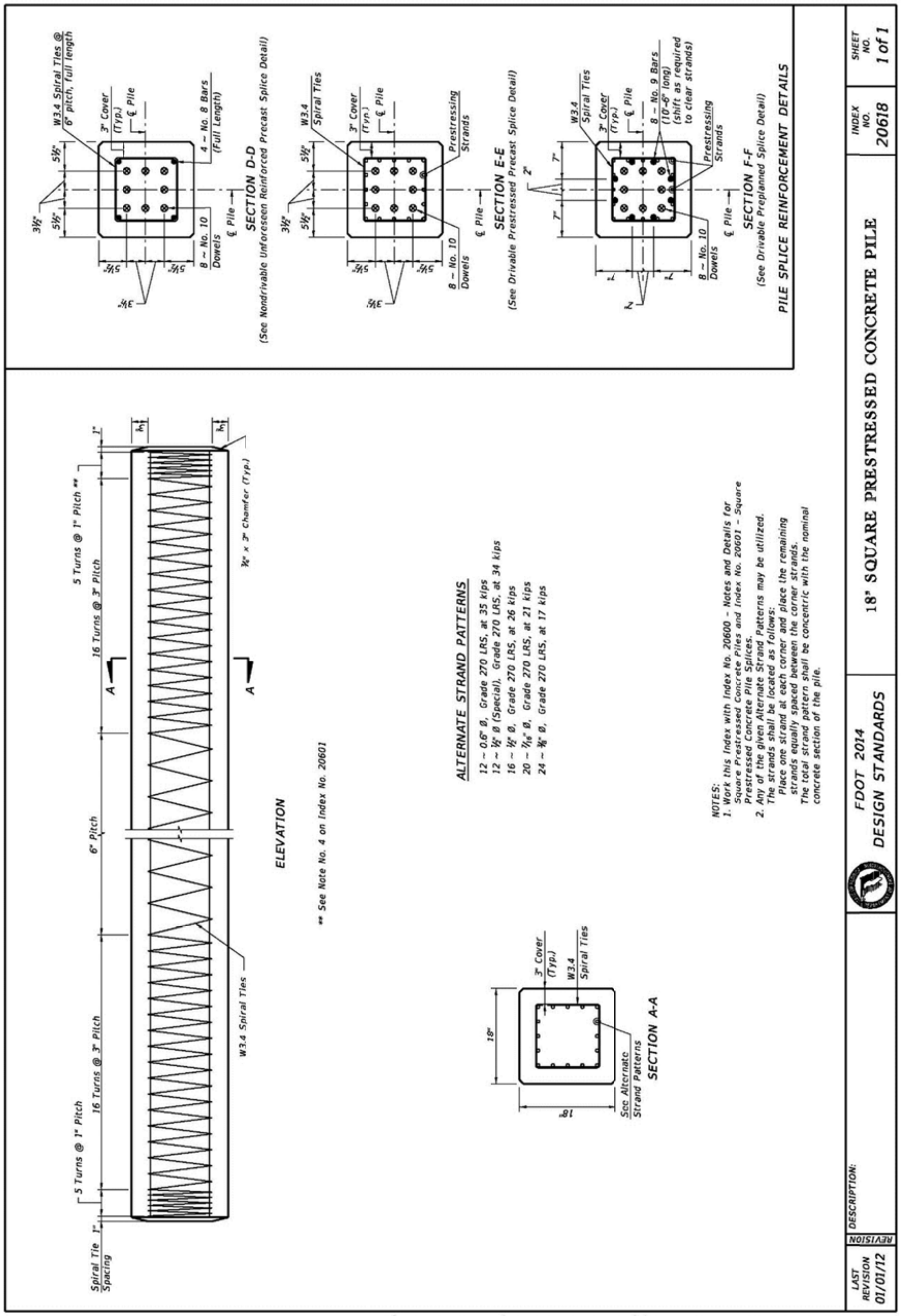

Figure B.2. 18 in. square pile (FDOT index 20618)

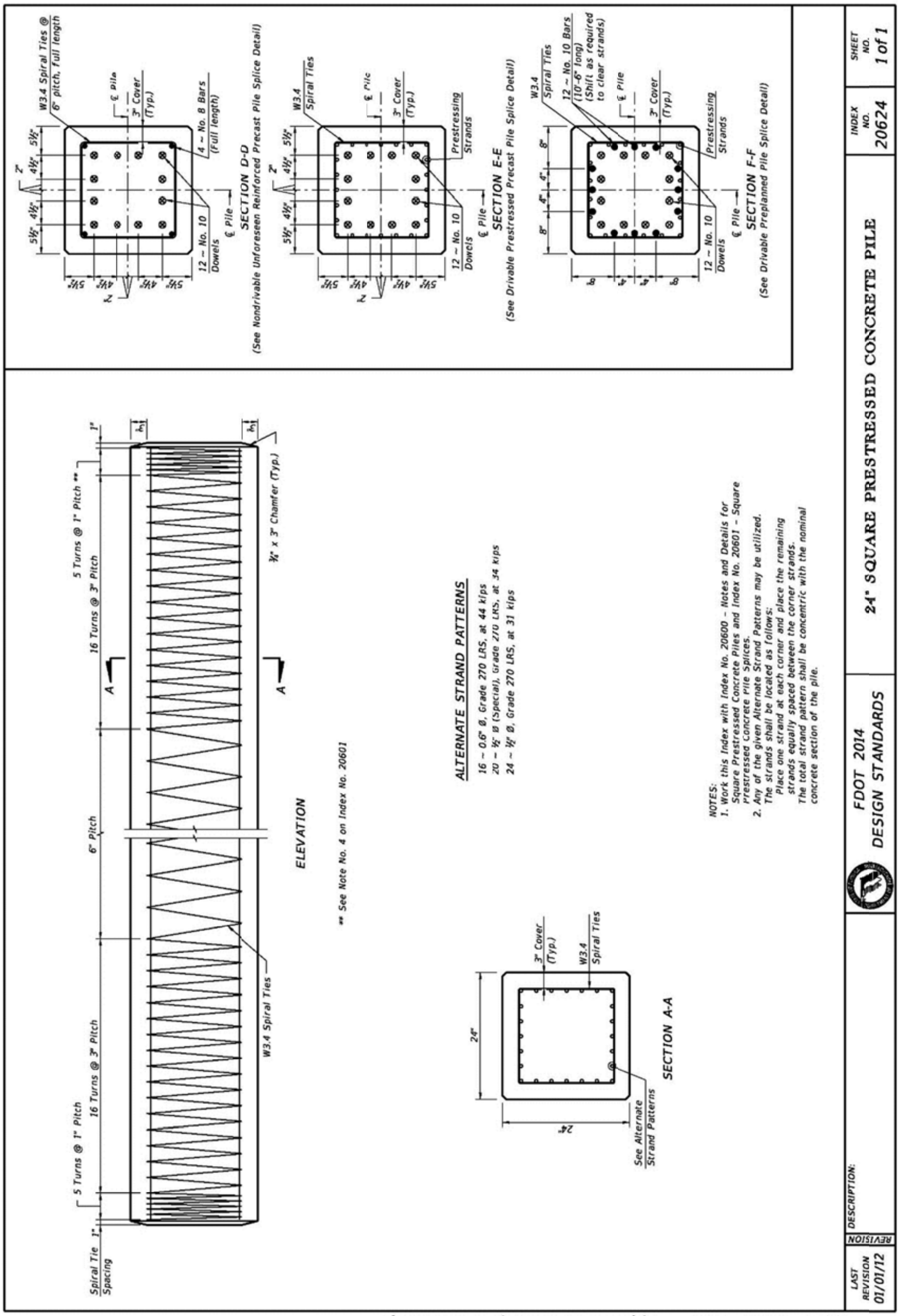

Figure B.3. 24 in. square pile (FDOT index 20624)

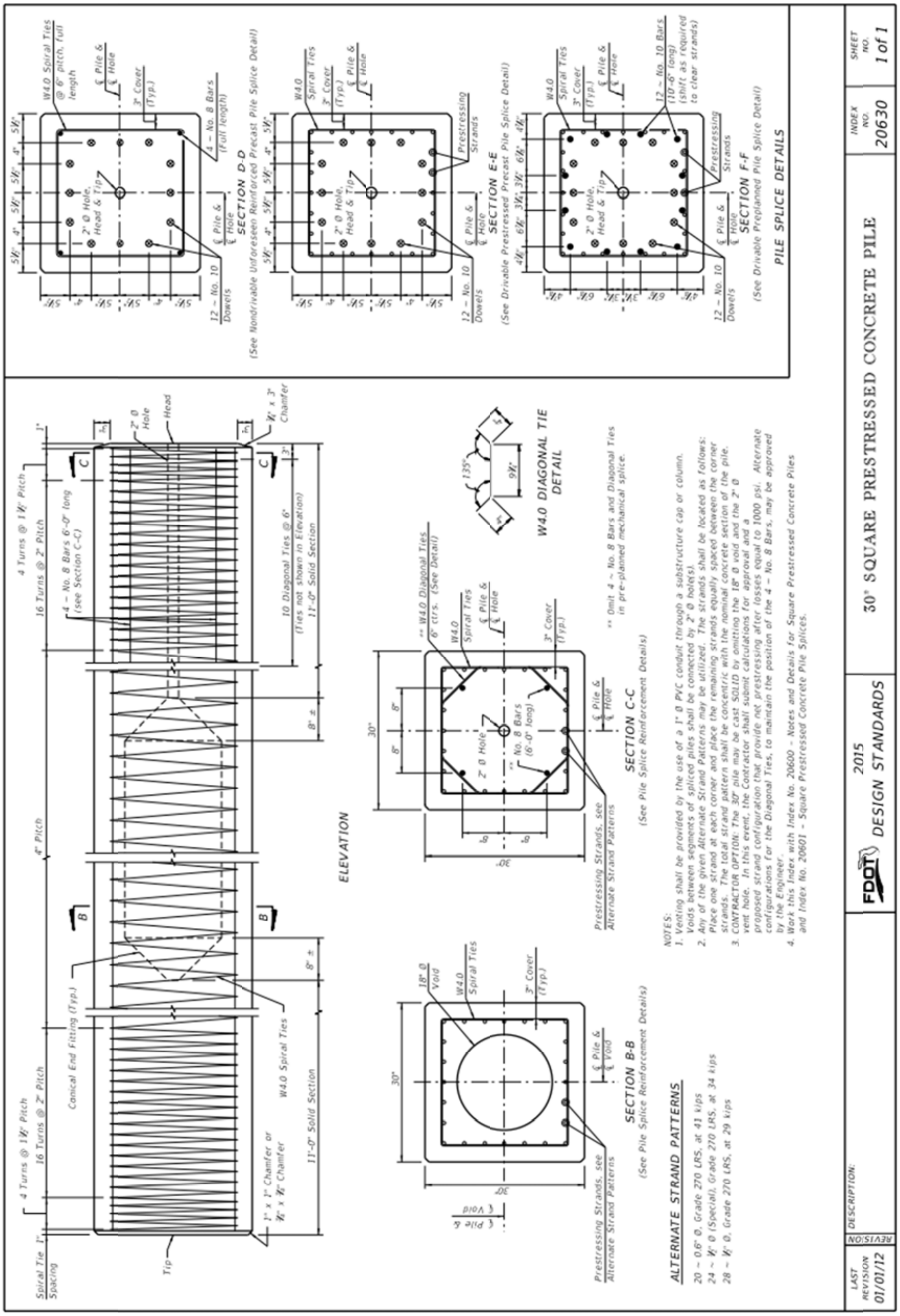

Figure B.4. 30 in. square pile (FDOT index 20630)

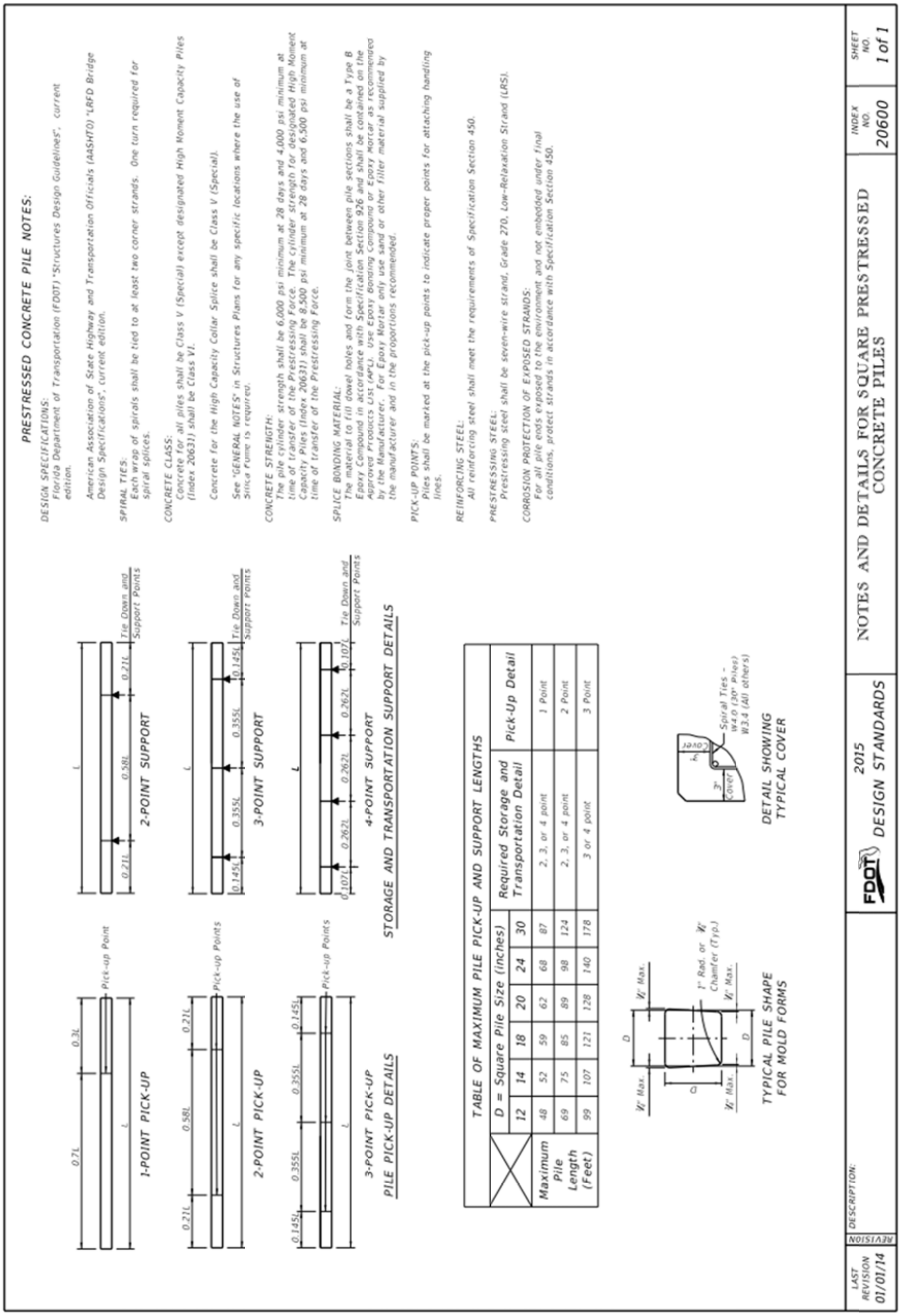

Figure B.5. Notes and details for square prestressed concrete piles (FDOT index 20600)

Appendix C

Load versus Strain Relationships for Standard Control Pile

(24in full scale 4-point bending test)

The load versus strain relationships for gages of standard control piles are shown in this appendix C from Figure C.1 to figure C.4.

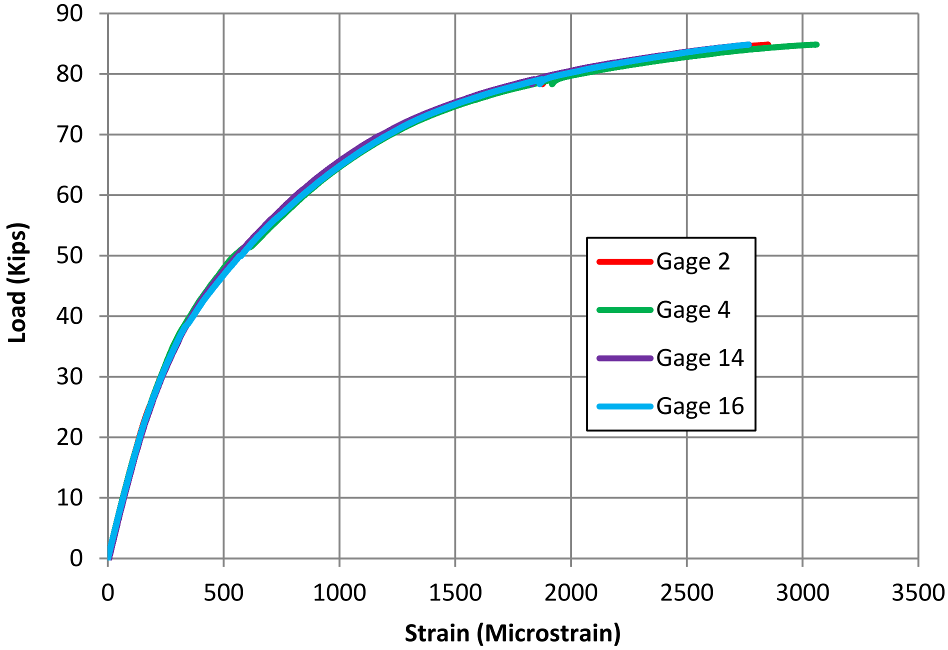

Figure C.1. Load vs strain for top gages 2, 4, 14, and 16.

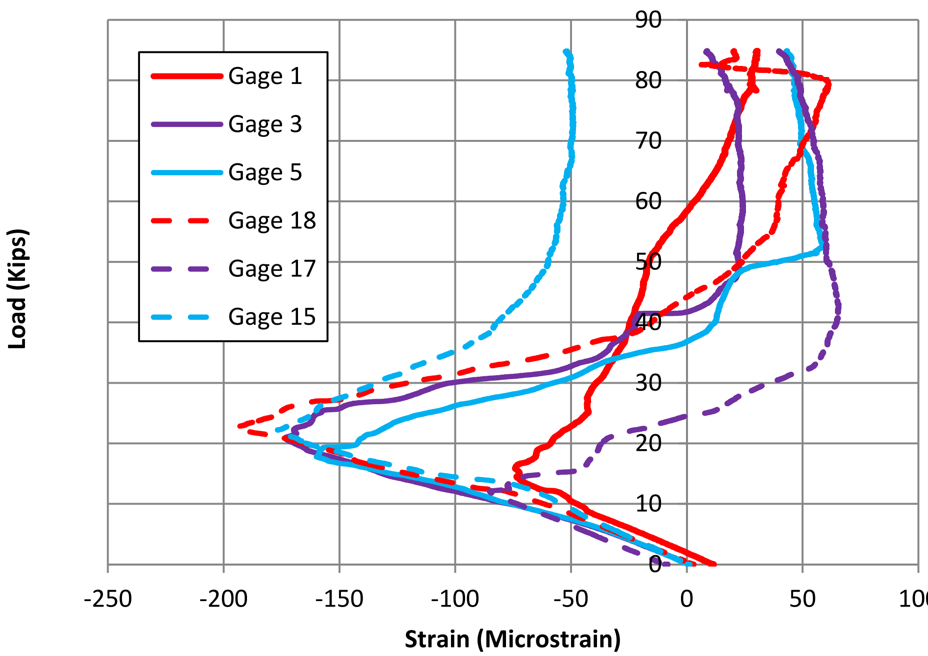

Figure C.2. Load vs strain for bottom gages 1, 3, 5, 15, 17, and 18.

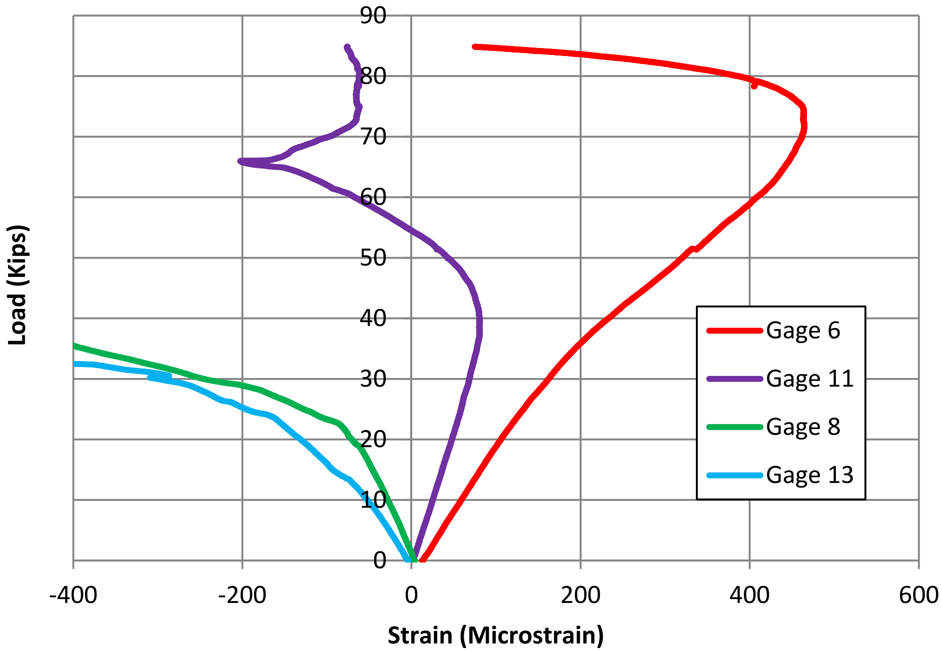

Figure C.3. Load vs strain for east side gages 6, 8, 11, and 13.

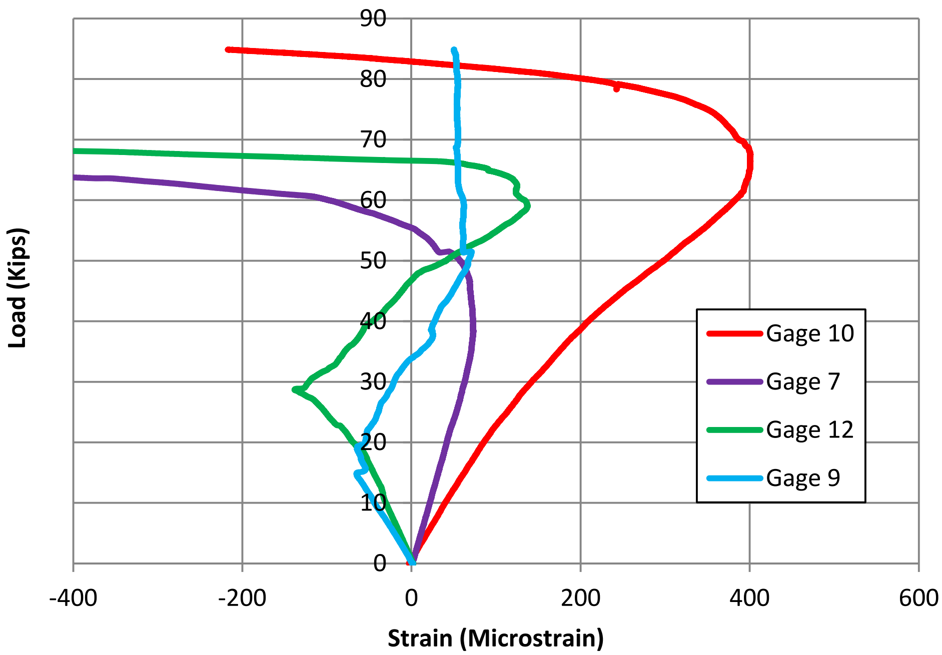

Figure C.4. Load vs strain for west side gages 7, 9, 10, and 12.

Appendix D

Load versus Strain Relationships for Three-Point Bending Tests

(24in full scale 3-point bending test)

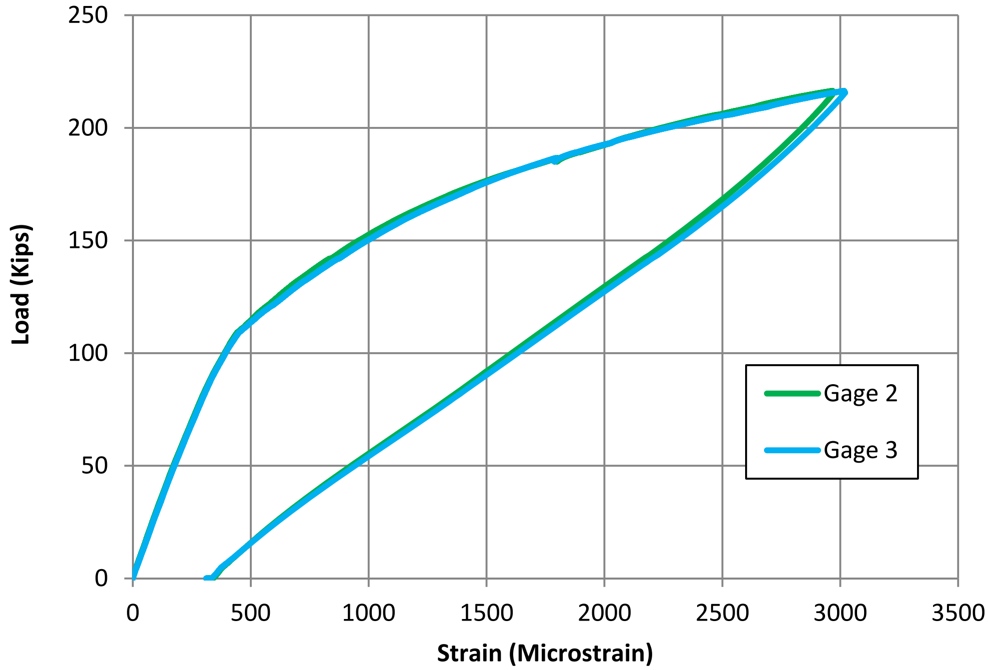

Figure D.1. Load vs strain for top gages 2 and 3 for spliced segment

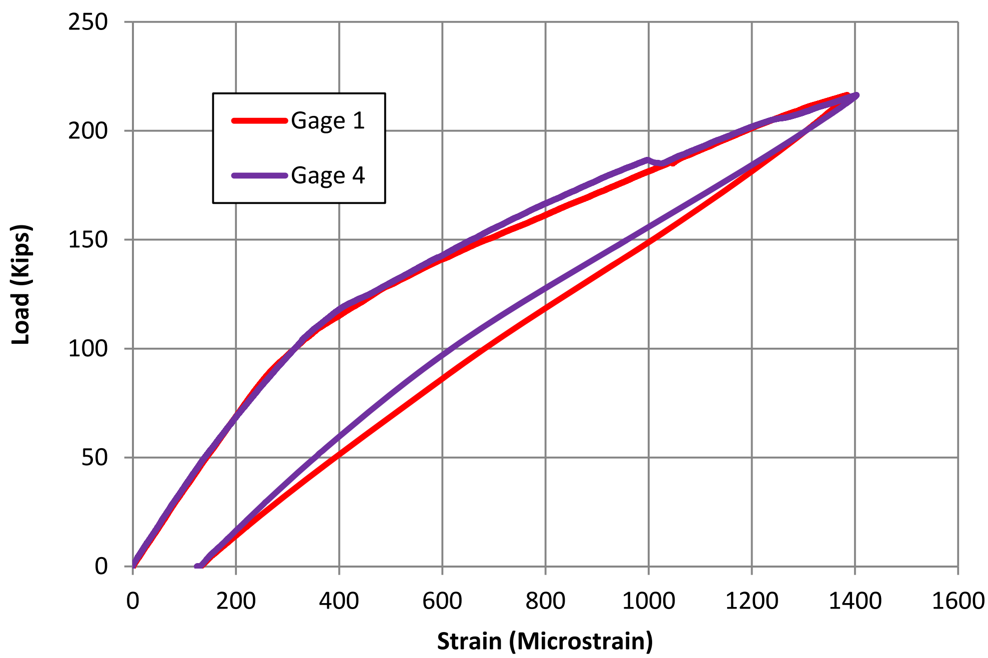

Figure D.2. Load vs strain for top gages 1 and 4 for spliced segment

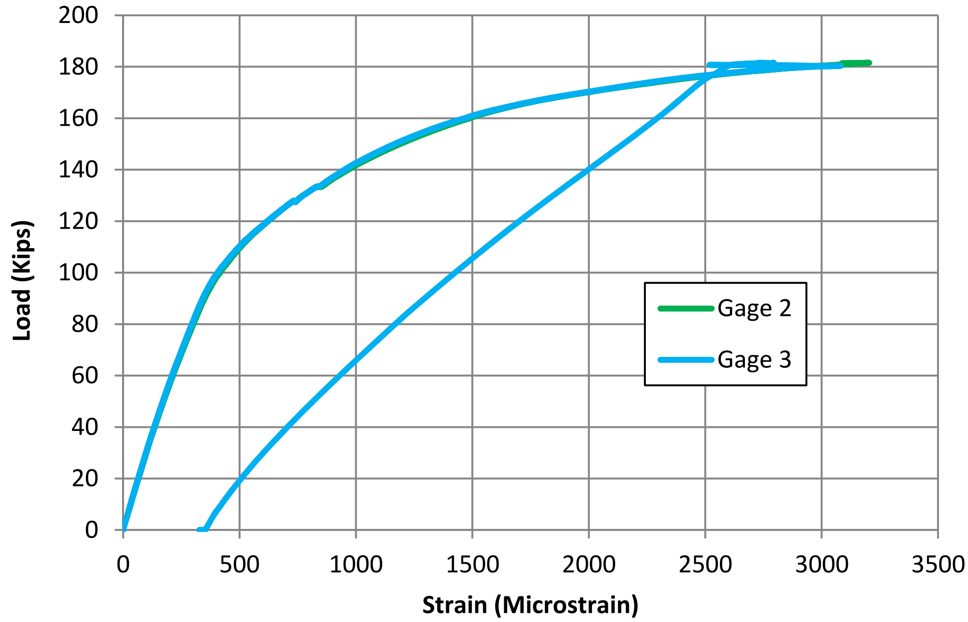

Figure D.3. Load vs strain for top gages 2 and 3 for control segment

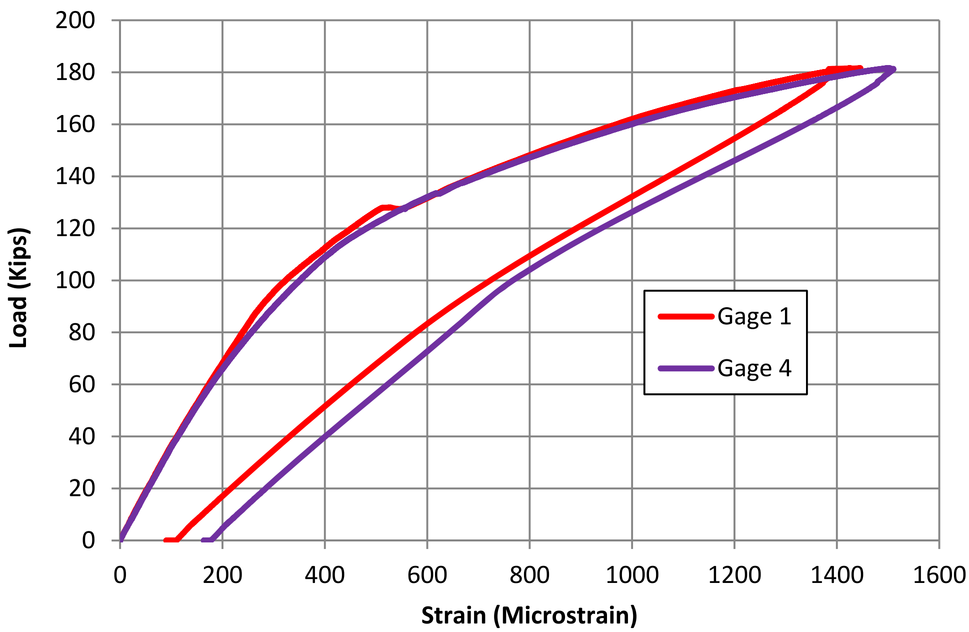

Figure D.4. Load vs strain for top gages 1 and 4 for control segment

Appendix E

Demonstration Pile Driving Specimen

- *Revised Bearing Plate*
- *Sixteen 0.6in strand configuration*
- *Revised alignment dowel and seal*
- *I-4 Deer Crossing Plan Sheets and Soil Borings*
- *End Bent 3-3 Pile 1 PDA Driving Summary*
- *End Bent 3-3 Pile 1 Field*
- *End Bent 3-3 Pile 1-1 PDA Driving Summary*
- *End Bent 3-3 Pile 1-1 Field*

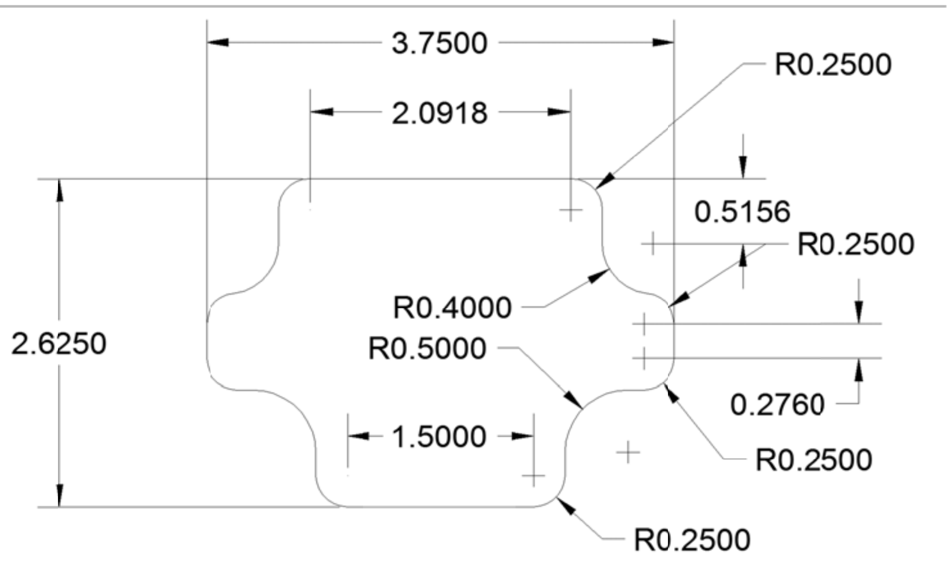

Figure E.1. Revised Bearing Plate Design for 0.6in or 0.5in strand chucks.

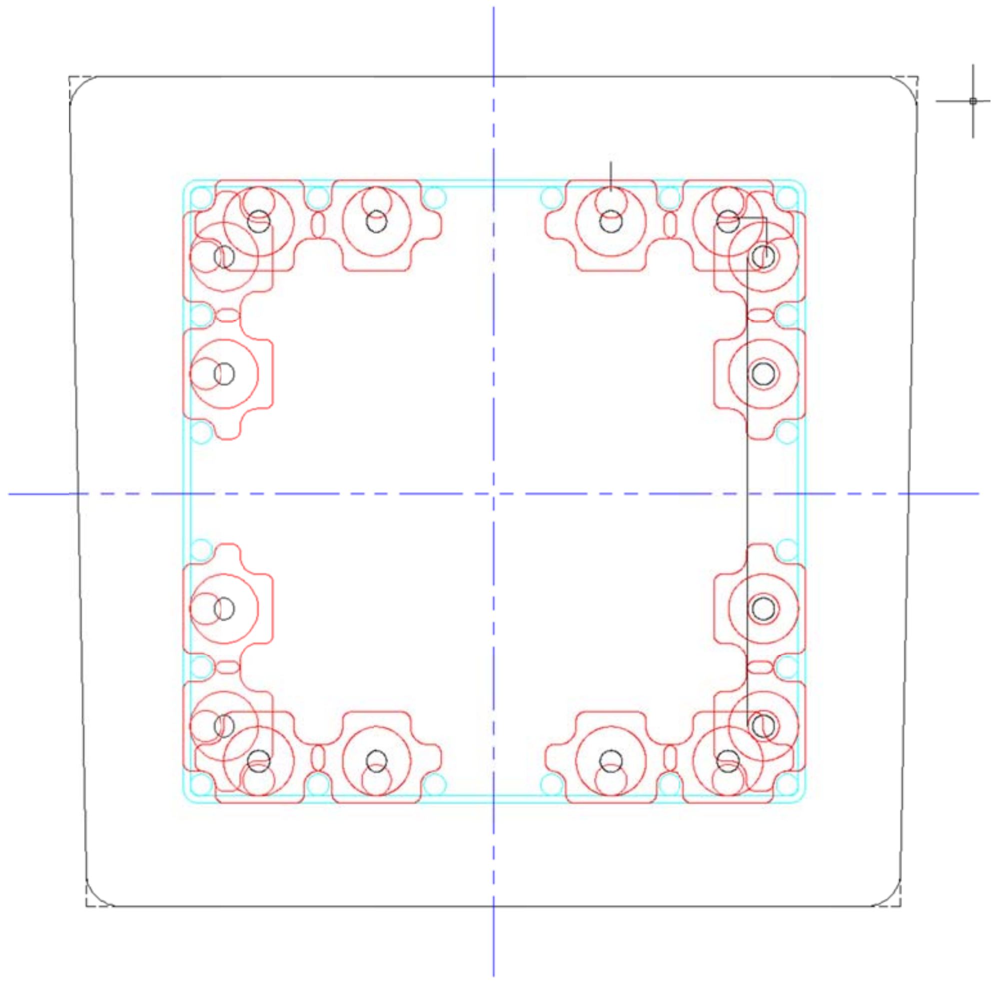

Figure E.2. Sixteen 0.6in strand configuration with revised bearing plates.

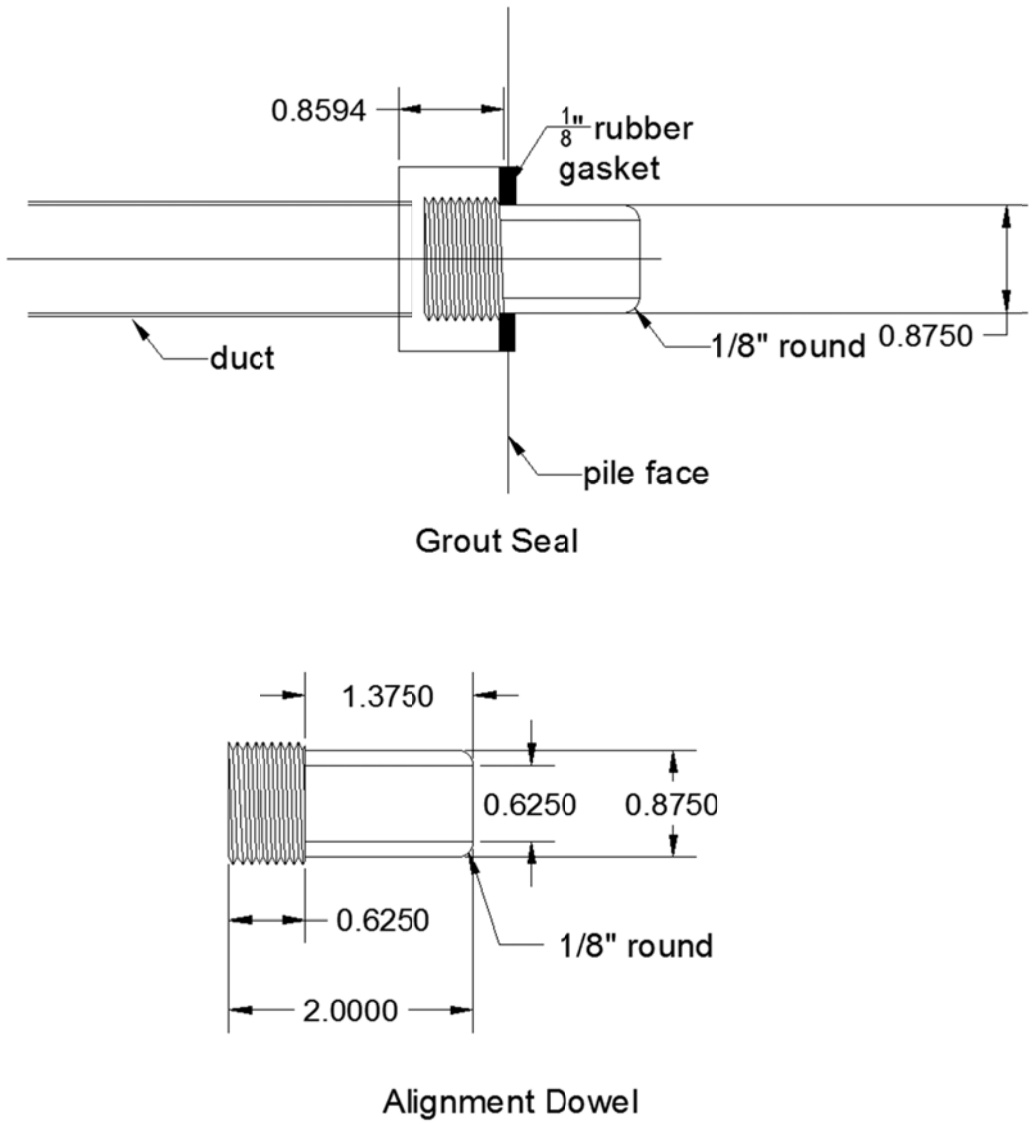

Figure E.3. Revised alignment dowel and grout seal.

.

Table E.1. Concrete cylinder break strengths for 24 in. full scale tests.

Concrete Age (days)	Cylinder 1 $f'c$ (psi)	Cylinder 2 $f'c$ (psi)	Average $f'c$ (psi)	Note
3	7710	8275	7993	Bed detensioned at 3 days
5	8375	8513	8444	Pile spliced at 4 days
32	10804	10473	10639	Pile driven at 27 days

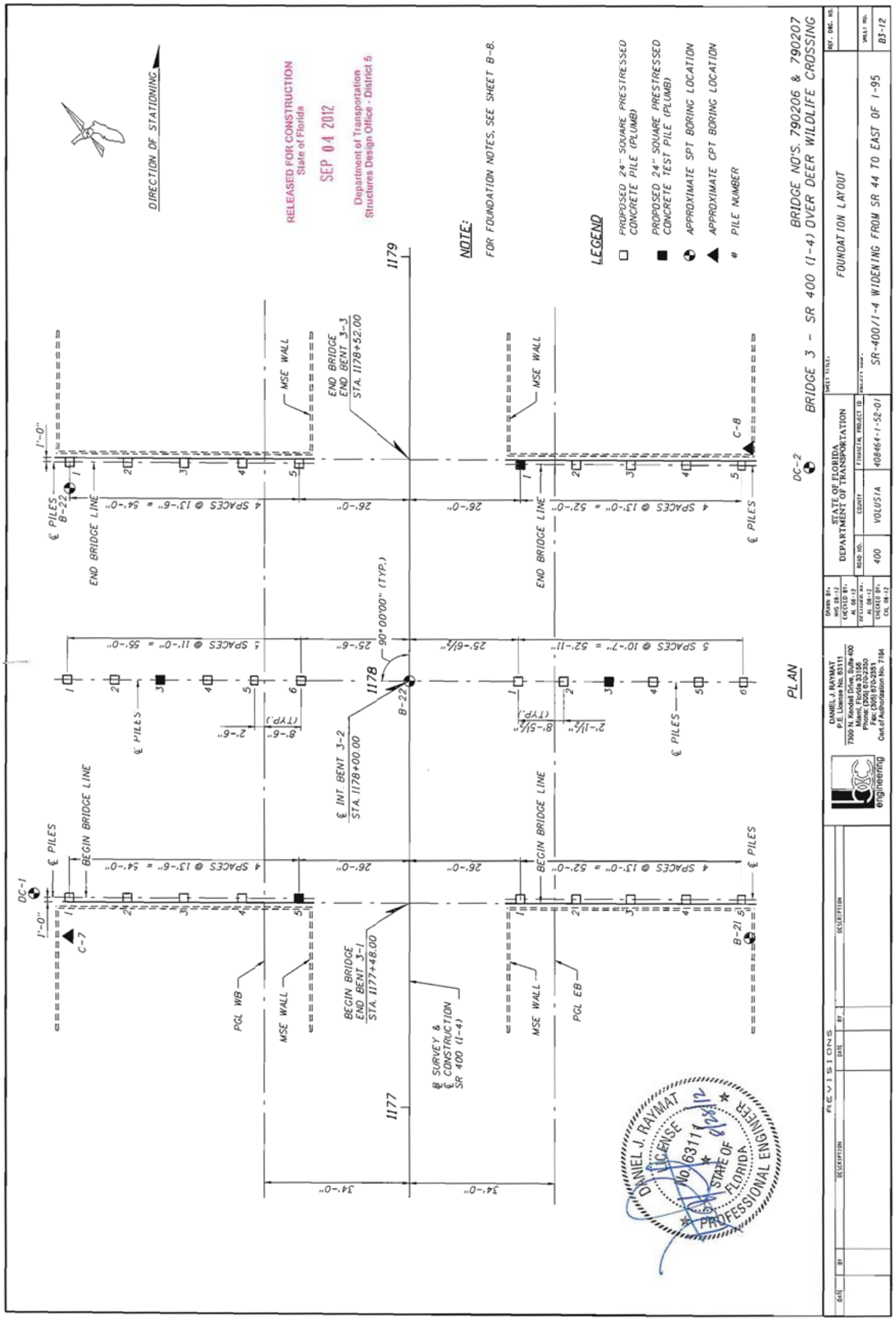

Figure E.4. Deer Bridge Pile Layout (plans sheet).

PILE DATA TABLE

PIER OR BENT NUMBER	PILE SIZE (IN.)	NOMINAL BEARING RESISTANCE (TONS)	TENSION RESISTANCE (TONS)	MINIMUM TIP ELEVATION (FT.)	TEST PILE LENGTH (FT.)	REQUIRED JET ELEVATION (FT.)	REQUIRED PREFORM ELEVATION (FT.)	FACTORED DESIGN LOAD (TONS)	DOWN DRAG (TONS)	TOTAL SCOUR RESISTANCE (TONS)	NET SCOUR RESISTANCE (TONS)	100-YEAR SCOUR ELEVATION (FT.)	LONG TERM SCOUR ELEVATION (FT.)	RESISTANCE FACTOR-Ø	PILE CUT-OFF ELEVATIONS
		INSTALLATION CRITERIA						DESIGN CRITERIA							
END BENT 3-1	24	247	N/A	*	115	N/A	N/A	160	N/A	N/A	N/A	N/A	N/A	0.65	SEE TABLE BELOW
INT. BENT 3-2	24	377	N/A	-3 **	120	N/A	N/A	245	N/A	N/A	N/A	N/A	N/A	0.65	SEE TABLE BELOW
END BENT 3-3	24	247	N/A	*	115	N/A	N/A	160	N/A	N/A	N/A	N/A	N/A	0.65	SEE TABLE BELOW

* MINIMUM TIP ELEVATION SHALL BE DETERMINED IN ACCORDANCE WITH SECTION 455-5.8 OF THE SPECIFICATIONS.
** MINIMUM TIP ELEVATION REQUIRED FOR LATERAL STABILITY.

PILE CUT-OFF ELEVATIONS TABLE

	LOCATION	PILE 1	PILE 2	PILE 3	PILE 4	PILE 5	PILE 6
WESTBOUND BRIDGE	END BENT 3-1	50.5	50.8	51.1	51.3	51.6	-
WESTBOUND BRIDGE	INT. BENT 3-2	50.6	50.8	51.1	51.3	51.5	51.7
WESTBOUND BRIDGE	END BENT 3-3	50.5	50.8	51.1	51.3	51.6	-
EASTBOUND BRIDGE	END BENT 3-1	51.6	51.4	51.1	50.8	50.6	-
EASTBOUND BRIDGE	INT. BENT 3-2	51.7	51.5	51.3	51.1	50.9	50.7
EASTBOUND BRIDGE	END BENT 3-3	51.6	51.4	51.1	50.8	50.6	-

$$\frac{\text{FACTORED DESIGN LOAD} + \text{NET SCOUR RESISTANCE} + \text{DOWN DRAG}}{\phi} \leq \text{NOMINAL BEARING RESISTANCE}$$

TENSION RESISTANCE - THE ULTIMATE SIDE FRICTION CAPACITY THAT MUST BE OBTAINED BELOW THE 100 YEAR SCOUR ELEVATION TO RESIST PULLOUT OF THE PILE (SPECIFY ONLY WHEN DESIGN REQUIRES TENSION CAPACITY).

TOTAL SCOUR RESISTANCE - AN ESTIMATE OF THE ULTIMATE STATIC SIDE FRICTION RESISTANCE PROVIDED BY THE SCOURABLE SOIL.

NET SCOUR RESISTANCE - AN ESTIMATE OF THE ULTIMATE STATIC SIDE FRICTION RESISTANCE PROVIDED BY THE SOIL FROM THE REQUIRED PREFORMED OR JETTING ELEVATION TO THE SCOUR ELEVATION.

100-YEAR SCOUR ELEVATION - ESTIMATED ELEVATION OF SCOUR DUE TO THE 100 YEAR STORM EVENT.

LONG TERM SCOUR ELEVATION - ESTIMATED ELEVATION OF SCOUR USED IN DESIGN FOR EXTREME EVENT LOADING.

RELEASED FOR CONSTRUCTION
State of Florida
SEP 04 2012
Department of Transportation
Structures Design Office - District 5

DANIEL J. RAYMAT
LICENSE
No. 63111
STATE OF FLORIDA
PROFESSIONAL ENGINEER
8/28/12

BRIDGE NO'S. 790206 & 790207
BRIDGE 3 - SR 400 (I-4) OVER DEER WILDLIFE CROSSING

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

engineering

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CHECKED BY: AL 08-12
DESIGNED BY: AL 08-12
CHECKED BY: CRL 08-12

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
400	VOLUSIA	408464-1-52-01

SHEET TITLE: PILE DATA TABLE

PROJECT NAME: SR-400/I-4 WIDENING FROM SR 44 TO EAST OF I-95

REF. DWG. NO.

SHEET NO. B3-13

Figure E.5. Deer Bridge Pile Design Criteria (plans sheet).

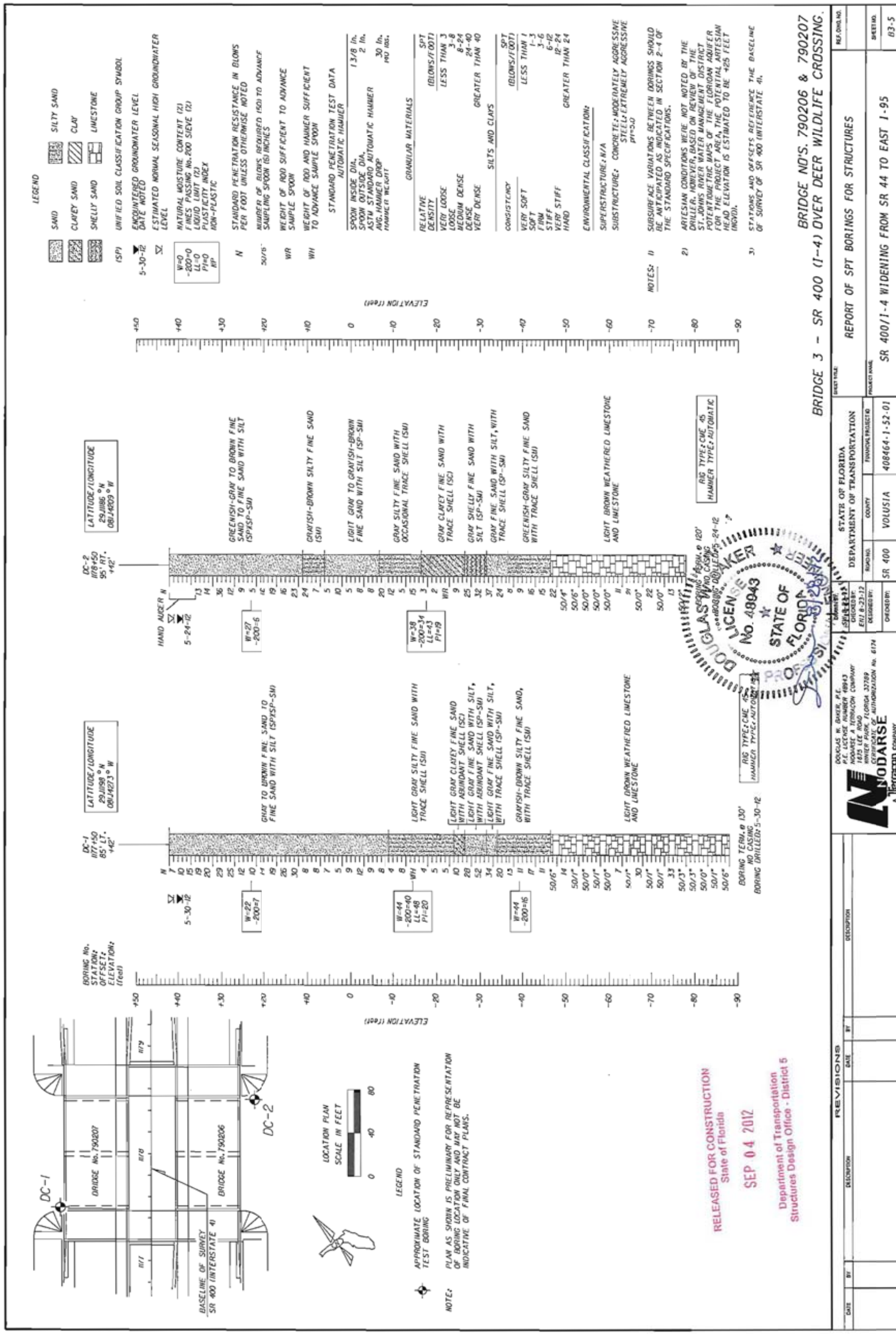

Figure E.6. Deer Bridge boring logs (plans sheet).

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Case Method & iCAP® Results — PDIPLOT2 2015.1.49.4 - Printed 05-May-2015

I-4 DEER CROSSING - EB 3-3, P1 — RE 49.26, GE 42.77, CE51.60
OP: TERRACON — Date: 21-April-2015

AR: 576.00 in² — SP: 0.150 k/ft³
LE: 111.00 ft — EM: 6,907 ksi
WS: 14,606.0 f/s — JC: 0.60 []

RX6: Max Case Method Capacity (JC=0.6)
RX7: Max Case Method Capacity (JC=0.7)
CSX: Max Measured Compr. Stress
TSX: Tension Stress Maximum
STK: O.E. Diesel Hammer Stroke
EMX: Max Transferred Energy
BTA: BETA Integrity Factor
CSB: Compression Stress at Bottom
TLS: Tension Stress at Splice

BL#	Depth ft	TYPE	RX6 kips	RX7 kips	CSX ksi	TSX ksi	STK ft	EMX k-ft	BTA (%)	CSB ksi	TLS ksi
18	25.00	AV18	157	153	1.3	0.7	5.95	14.7	99.2	0.5	0.7
38	26.00	AV20	238	235	1.5	0.8	6.15	16.1	100.0	0.6	0.7
61	27.00	AV23	264	263	1.5	0.8	6.07	15.6	100.0	0.7	0.7
94	28.00	AV33	301	301	1.5	0.7	6.09	15.3	100.0	0.8	0.6
127	29.00	AV33	308	307	1.6	0.8	6.15	15.4	100.0	0.8	0.6
165	30.00	AV38	326	326	1.6	0.7	6.24	15.6	100.0	0.8	0.6
211	31.00	AV46	344	344	1.6	0.7	6.31	15.8	100.0	0.9	0.5
257	32.00	AV46	356	356	1.6	0.7	6.36	15.8	100.0	0.9	0.5
306	33.00	AV49	359	359	1.7	0.6	6.37	15.9	100.0	0.9	0.5
356	34.00	AV50	353	353	1.6	0.6	6.42	15.8	100.0	0.9	0.5
398	35.00	AV42	350	345	1.8	0.7	7.03	18.7	100.0	0.9	0.6
431	36.00	AV33	359	343	1.9	0.8	7.54	21.1	100.0	1.0	0.7
464	37.00	AV33	360	343	1.9	0.8	7.51	21.1	100.0	1.0	0.7
495	38.00	AV31	351	334	1.9	0.8	7.53	21.3	100.0	0.9	0.7
523	39.00	AV28	332	318	1.9	0.9	7.57	21.7	100.0	0.9	0.7
548	40.00	AV25	318	309	1.9	0.9	7.52	21.5	100.0	0.8	0.8
573	41.00	AV25	311	305	1.7	0.8	7.02	18.6	100.0	0.8	0.7
607	42.00	AV34	310	305	1.6	0.7	6.47	15.8	100.0	0.7	0.6
639	43.00	AV32	317	309	1.6	0.7	6.55	16.2	100.0	0.7	0.6
673	44.00	AV34	325	317	1.6	0.7	6.62	16.3	100.0	0.7	0.6
709	45.00	AV36	334	327	1.6	0.7	6.64	16.1	100.0	0.7	0.6
744	46.00	AV35	336	331	1.6	0.7	6.71	16.3	100.0	0.7	0.6
782	47.00	AV38	342	337	1.7	0.7	6.83	16.6	100.0	0.7	0.6

Figure E.7. Test Pile PDA pile driving summary (Page 1 of 5).

Foundation & Geotechnical Engineering
Case Method & iCAP® Results

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PDIPLOT2 2015.1.49.4 - Printed 05-May-2015

I-4 DEER CROSSING - EB 3-3, P1
OP: TERRACON

RE 49.26, GE 42.77, CE51.60
Date: 21-April-2015

BL#	Depth ft	TYPE	RX6 kips	RX7 kips	CSX ksi	TSX ksi	STK ft	EMX k-ft	BTA (%)	CSB ksi	TLS ksi
818	48.00	AV36	348	341	1.7	0.7	6.83	16.6	100.0	0.7	0.6
859	49.00	AV41	364	358	1.7	0.6	6.92	17.0	100.0	0.7	0.5
905	50.00	AV46	366	361	1.7	0.6	6.97	17.2	100.0	0.8	0.5
939	51.00	AV34	414	401	1.9	0.8	8.42	23.1	100.0	0.9	0.7
974	52.00	AV35	414	403	1.9	0.8	8.40	23.0	100.0	0.9	0.7
1010	53.00	AV36	417	408	1.9	0.7	8.33	22.9	100.0	0.8	0.6
1045	54.00	AV35	415	407	1.9	0.7	8.34	23.0	100.0	0.9	0.7
1082	55.00	AV37	408	401	1.9	0.7	8.20	22.4	100.0	0.9	0.7
1119	56.00	AV37	393	384	2.0	0.8	8.15	22.9	100.0	0.9	0.7
1156	57.00	AV37	389	378	2.0	0.8	8.18	23.1	100.0	0.9	0.8
1193	58.00	AV37	384	373	2.1	0.8	8.19	23.7	100.0	0.9	0.8
1229	59.00	AV36	379	366	2.1	0.8	8.23	24.0	100.0	0.9	0.8
1258	60.00	AV29	351	340	2.1	1.0	8.22	24.3	100.0	0.8	1.0
1290	61.00	AV32	322	320	1.8	0.8	7.32	19.4	100.0	0.8	0.8
1325	62.00	AV35	336	331	1.9	0.8	7.36	20.0	100.0	0.8	0.8
1368	63.00	AV43	355	345	1.9	0.7	7.35	20.4	100.0	0.9	0.7
1409	64.00	AV41	335	322	1.9	0.7	7.34	20.5	100.0	0.9	0.7
1446	65.00	AV37	328	313	1.9	0.8	7.25	20.2	100.0	0.8	0.8
1487	66.00	AV41	338	327	1.9	0.8	7.27	19.5	100.0	0.8	0.8
1526	67.00	AV39	344	333	2.0	0.8	7.50	20.5	100.0	0.8	0.8
1565	68.00	AV39	337	325	2.0	0.8	7.52	20.7	100.0	0.8	0.8
1606	69.00	AV41	334	324	2.0	0.8	7.58	21.1	100.0	0.8	0.8
1646	70.00	AV40	328	321	2.0	0.8	7.68	21.7	100.0	0.8	0.8
1689	71.00	AV43	322	317	2.0	0.8	7.66	21.3	100.0	0.8	0.8
1729	72.00	AV40	313	310	2.0	0.8	7.65	21.5	100.0	0.8	0.7
1775	73.00	AV46	318	316	2.1	0.8	7.76	22.3	100.0	0.9	0.8

Figure E.8. Test Pile PDA pile driving summary (Page 2 of 5).

I-4 DEER CROSSING - EB 3-3, P1
OP: TERRACON

RE 49.26, GE 42.77, CE51.60
Date: 21-April-2015

BL#	Depth ft	TYPE	RX6 kips	RX7 kips	CSX ksi	TSX ksi	STK ft	EMX k-ft	BTA (%)	CSB ksi	TLS ksi
1823	74.00	AV48	297	295	2.0	0.7	7.53	21.1	100.0	0.9	0.7
1882	75.00	AV59	384	377	2.1	0.6	7.82	23.0	100.0	1.1	0.5
1936	76.00	AV54	506	504	2.4	0.8	8.80	29.9	100.0	1.4	0.8
1993	77.00	AV57	605	605	2.4	1.0	8.84	30.2	100.0	1.5	0.9
2082	78.00	AV89	751	750	2.5	0.9	9.16	31.7	100.0	1.7	0.9
2254	79.00	AV172	732	732	1.9	0.4	8.00	20.9	89.4	1.5	0.3
2399	80.00	AV145	815	815	2.3	0.4	8.63	27.5	100.0	1.7	0.4
2534	81.00	AV135	780	778	2.2	0.4	8.89	27.4	100.0	1.7	0.4
2653	82.00	AV119	744	742	2.2	0.4	9.07	27.2	100.0	1.6	0.4
2750	83.00	AV97	701	701	2.2	0.4	9.11	27.3	100.0	1.5	0.4
2829	84.00	AV79	660	660	2.2	0.5	9.09	27.7	100.0	1.5	0.5
2895	85.00	AV66	631	631	2.4	0.6	9.33	29.4	100.0	1.5	0.6
2949	86.00	AV54	605	605	2.4	0.7	9.44	30.4	100.0	1.5	0.7
2998	87.00	AV49	578	578	2.5	0.7	9.48	31.0	100.0	1.5	0.7
3046	88.00	AV48	555	555	2.5	0.8	9.41	30.8	100.0	1.5	0.8
3092	89.00	AV46	556	555	2.5	0.9	9.50	31.6	100.0	1.5	0.8
3148	90.00	AV56	561	561	2.5	0.8	9.06	29.5	100.0	1.5	0.8
3201	91.00	AV53	593	592	2.5	0.8	8.97	29.4	100.0	1.6	0.8
3261	92.00	AV60	625	625	2.5	0.7	9.08	29.9	100.0	1.7	0.7
3325	93.00	AV64	645	643	2.5	0.6	9.13	30.2	100.0	1.8	0.6
3387	94.00	AV62	687	685	2.6	0.6	9.65	32.1	100.0	1.9	0.6
3464	95.00	AV77	753	747	2.6	0.6	9.40	31.0	100.0	2.0	0.6
3540	96.00	AV76	835	824	2.6	0.6	9.08	30.1	100.0	2.0	0.6
3649	97.00	AV109	1,060	1,058	2.6	0.7	9.31	31.4	100.0	2.2	0.6
3770	98.00	AV121	1,143	1,141	2.6	0.4	9.27	30.9	100.0	2.3	0.3
3885	99.00	AV115	1,122	1,120	2.6	0.4	9.29	30.6	100.0	2.3	0.2
4035	99.83	AV150	1,412	1,410	2.7	0.5	9.91	32.5	100.0	2.8	0.2

Figure E.9. Test Pile PDA pile driving summary (Page 3 of 5).

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Case Method & iCAP® Results | PDIPLOT2 2015.1.49.4 - Printed 05-May-2015

I-4 DEER CROSSING - EB 3-3, P1 | RE 49.26, GE 42.77, CE51.60
OP: TERRACON | Date: 21-April-2015

BL#	Depth	TYPE	RX6	RX7	CSX	TSX	STK	EMX	BTA	CSB	TLS
	ft		kips	kips	ksi	ksi	ft	k-ft	(%)	ksi	ksi
		Average	593	589	2.1	0.7	8.25	24.7	99.5	1.4	0.6

Total number of blows analyzed: 4035

BL#	Sensors
1-2194	F3: [9487] 94.3 (1.05); F4: [8291] 96.7 (1.05); A3: [K4153] 355.0 (0.95); A4: [K4154] 395.0 (0.95)
2195-4035	F3: [9487] 94.3 (1.05); F4: [F269] 91.4 (1.05); A3: [K4153] 355.0 (0.95); A4: [K4154] 395.0 (0.95)

BL#	Comments
2082	
2194	CHANGE STRAIN F4
4035	99 ft 10 in, high CSB

Time Summary

Drive 57 minutes 32 seconds 3:09 PM - 4:07 PM (4/21/2015) BN 1 - 2082
Stop 52 minutes 23 seconds 4:07 PM - 4:59 PM
Drive 8 minutes 39 seconds 4:59 PM - 5:08 PM BN 2083 - 2194
Stop 14 minutes 5 seconds 5:08 PM - 5:22 PM
Drive 47 minutes 0 second 5:22 PM - 6:09 PM BN 2195 - 4035

Total time [02:59:41] = (Driving [01:53:11] + Stop [01:06:29])

Figure E.10. Test Pile PDA pile driving summary (Page 4 of 5).

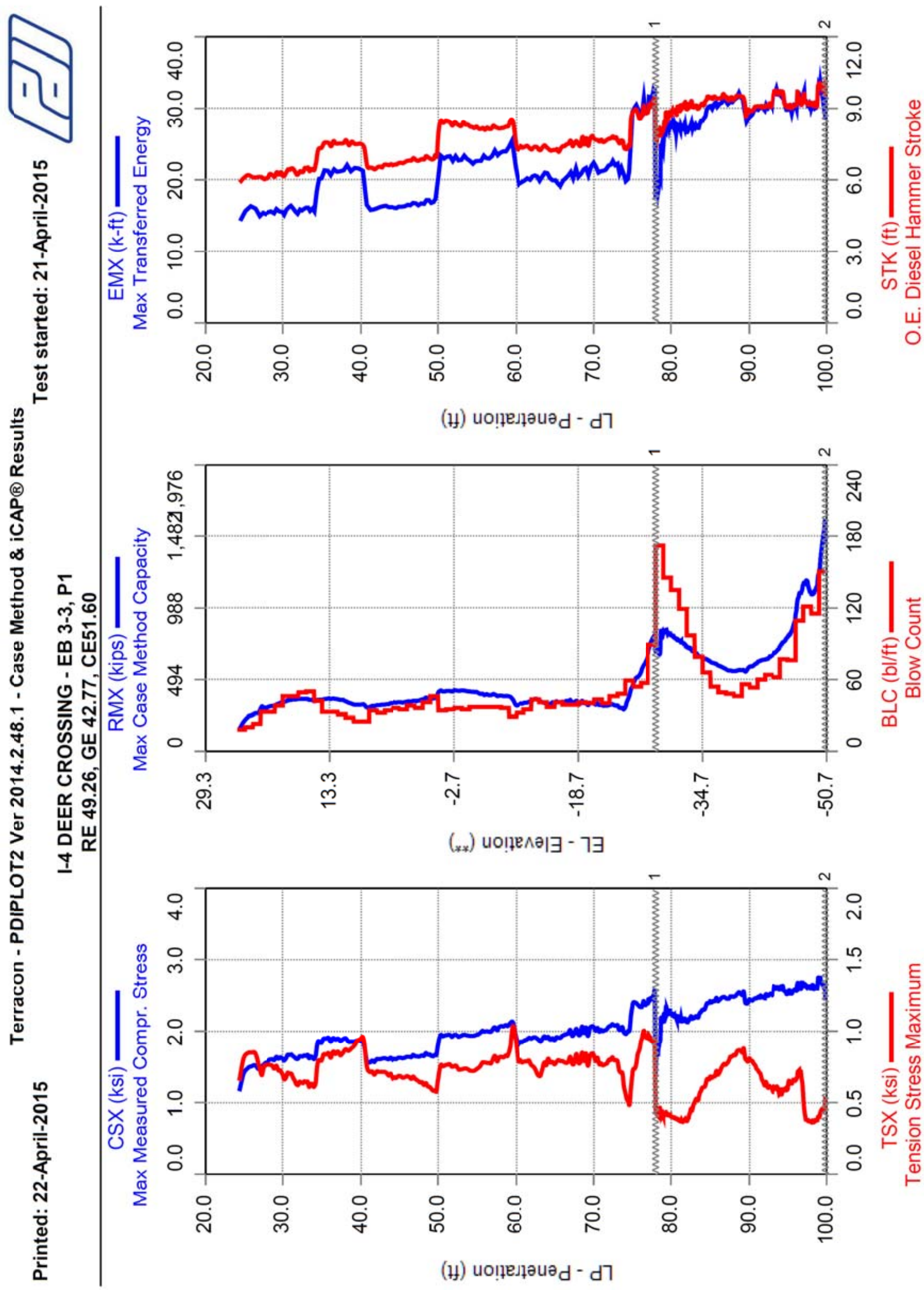

Figure E.11. Test Pile PDA pile driving summary (Page 5 of 5).

Page No. 1/2

700-010-
Construct
11/

PILE DRIVING INFORMATION

Structure Number: DEER CROSSING 790206

FIN PROJ. ID # 408464-1-52-01 DATE 4-21-15 STATION NO. 1178+52.00

PILE SIZE 24"SQ ACTUAL/AUTH LENGTH 115.00' BENT/PIER NO. END BENT 3-3 PILE NO. 1

HAMMER TYPE APE 46-32 SERIAL 991046 RATED ENERGY 107,280 ft/lbs OPERATING RATE VARIES

REF. ELEV +49.26 MIN. TIP ELEV N/A PILE CUTOFF ELEV +51.60

DRIVING CRITERIA TEST PiLE

PILE CUSHION THICKNESS AND MATERIAL PINE PLYWOOD ① 14" 2080 BLOWS COMPRESSED TO 10" ② 13¾ 1957 BLOWS COMPRESSED TO

HAMMER CUSHION THICKNESS AND MATERIAL 1½" ALUMINUM (3x½") & 2" MICARTA (2x1")

WEATHER CLEAR TEMP 84° START TIME 3:09PM STOP TIME 6:10PM

PILE DATA

PAY ITEM NO. N/A WORK ORDER NO. N/A

MANUFACTURED BY DURASTRESS T.B.M./B.M. ELEV N/A GROUND ROD READ N/A

DATE CAST 2-24-15 ROD READ N/A PILE HEAD ROD READ N/A

MANUFACTURER'S PILE NO. B1577 H=179 H.I. N/A PILE HEAD ELEV. ~~+64.26~~ +64.42

PILE HEAD CHAMFER ¾"X3" PILE TIP ELEV. ~~-50.74~~ -50.573 MH

PILE TIP CHAMFER ¾"X3" GROUND ELEV. +42.77

QUALIFED INSPECTOR'S NAME: Michael Harrington TIN #: H65255860

SPLICE / EACH	PREFORMED HOLE	DYNAMIC LOAD TEST	PAY SET CHECK	NO PAY SET CHECK	REDRIVE	EXTRACTION	DRIVING OF SPLICE	PILE TYPE CODE	BATTER	PILE LENGTH: ORIGINAL FURNISHED	PILE LENGTH: TOTAL LENGTH WITH EXTENSION	PENETRATION BELOW GROUND	EXTENSION / BUILD UP: AUTHORIZED	EXTENSION / BUILD UP: ACTUAL
0.000	0.000	1.000	0.000	0.000	0.000	0.00	0.00	1.00	0.000	115.000	115.000	~~93.510~~ 93.343	0.000	0.000

NOTES: ① FUEL SETTING 1 ⑥ STOPPED DRIVING @ 100' MARK ON PILE 99'10"

② FUEL SETTING 2

③ FUEL SETTING 3

④ FUEL SETTING 4

⑤ STOPPED DRIVING @ 78' MARK ON PILE TO CHANGE PILE CUSHION

For Trainee experience evidence only:
Name of CTQP Trainee being supervised by the Qualified Inspector: ______

CTQP Trainee

I certify the Pile Driving Record accuracy and that the named above Trainee has observed the full pile installation:

Qualified Inspector (Signature)

Figure E.12. Test Pile Driving Logs (Page 1 of 2).

Page No. 2/2

700-010-60
Construction
11/11

PILE DRIVING LOG

DOER CROSSINGS
Structure No. 790206 | Bent/Pier No. END BENT 3-3 | Pile No. 1

Depth	Blows	Stroke/ Pressure	Note No.	Depth	Blows	Stroke/ Pressure	Note No.	Depth	Blows	Stroke/ Pressure	Note No.	Depth	Blows	Stroke/ Pressure	Note No.
1				34	50	6.36	1	67	39	7.41	1	~~100~~	~~150~~	~~9.89~~	~~4/6~~
2				35	42	7.20	1/2	68	39	7.35	1	99'10"	150	9.89	4/6
3				36	33	7.45	2	69	41	7.51	1				
4				37	33	7.46	2	70	40	7.61	1				
5				38	31	7.47	2	71	43	7.44	1				
6				39	28	7.47	2	72	40	7.64	1				
7				40	25	7.51	2	73	46	7.67	1				
8				41	25	7.11	2/1	74	48	7.36	1				
9				42	34	6.29	1	75	59	7.65	1				
10				43	32	6.58	1	76	54	8.15	1/2				
11				44	34	6.56	1	77	57	8.83	2				
12				45	36	6.60	1	78	69	8.89	2/5				
13				46	35	6.60	1	79	177	7.85	2				
14				47	38	6.70	1	80	145	8.48	2/3/4				
15				48	36	6.78	1	81	135	8.80	4				
16				49	41	6.81	1	82	119	9.01	4				
17				50	46	6.87	1	83	97	9.03	4				
18				51	34	8.10	1/2	84	79	9.03	4				
19				52	35	8.31	2	85	66	9.17	4				
20	SEE BELOW	SEE BELOW	SEE BELOW	53	36	8.42	2	86	54	9.03	4				
21				54	35	8.29	2	87	49	9.50	4				
22				55	37	8.11	2	88	48	9.01	4				
23				56	37	8.12	2	89	46	9.57	4/3				
24				57	37	7.91	2	90	56	9.00	3				
25	13	6.08	1	58	37	8.14	2	91	53	8.91	3				
26	20	6.11	1	59	36	8.19	2	92	60	8.94	3				
27	23	6.03	1	60	29	8.28	2	93	64	8.86	3				
28	33	5.93	1	61	32	7.28	2/1	94	62	9.58	3				
29	33	6.12	1	62	35	7.27	1	95	77	9.31	3/4				
30	38	6.14	1	63	43	7.11	1	96	76	9.04	4				
31	46	6.24	1	64	41	7.25	1	97	109	9.12	4				
32	46	6.32	1	65	37	7.12	1	98	121	9.21	4				
33	49	6.31	1	66	41	7.19	1	99	115	9.07	4				

3:09 PM 3:21 5:01 5:02 PDA? 5:22 6:10 1' ABOVE CUT OFF. 111.6 MARK ON P/S

3:11 PM PDA? 4:07 5:04 5:03 PDA?

5:07 5:08 PDA CHANGE LEADS 2080 10"

10"

Figure E.13. Test Pile Driving Logs (Page 2 of 2).

Terracon
Case Method & iCAP® Results

Page 1
PDIPLOT2 2014.2.48.1 - Printed 15-May-2015

I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1
OP: TERRACON

24" sq. PSC Spliced (30' top + 70' bottom)
Date: 14-May-2015

AR: 576.00 in²
LE: 96.00 ft
WS: 14,015.0 f/s

SP: 0.150 k/ft³
EM: 6,359 ksi
JC: 0.50 []

TLS: Tension Stress at Splice
CSX: Max Measured Compr. Stress
TSX: Tension Stress Maximum
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke

RMX: Max Case Method Capacity
RX6: Max Case Method Capacity (JC=0.6)
CSB: Compression Stress at Bottom
BTA: BETA Integrity Factor

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
13	28.0	7	AV13	1.03	1.34	0.97	18.2	5.57	75	2	0.48	88
			STD	0.17	0.18	0.23	1.4	0.20	19	6	0.14	10
			MAX	1.23	1.54	1.19	20.3	5.86	88	22	0.64	100
			MIN	0.63	0.99	0.41	14.9	5.12	21	0	0.23	59
21	29.0	8	AV8	1.20	1.60	1.21	19.8	5.70	97	38	0.70	97
			STD	0.05	0.04	0.04	0.9	0.15	11	20	0.04	6
			MAX	1.24	1.65	1.25	20.6	5.88	114	62	0.75	100
			MIN	1.08	1.51	1.13	18.3	5.38	84	2	0.62	86
29	30.0	8	AV8	1.24	1.71	1.25	20.3	5.97	192	152	0.80	96
			STD	0.06	0.07	0.05	1.8	0.33	29	13	0.05	5
			MAX	1.34	1.81	1.34	23.9	6.63	217	169	0.88	100
			MIN	1.16	1.63	1.16	17.8	5.63	121	125	0.73	88
38	31.0	9	AV9	1.27	1.79	1.27	20.6	5.98	259	218	0.89	100
			STD	0.01	0.02	0.01	0.3	0.05	23	25	0.02	0
			MAX	1.29	1.82	1.29	21.2	6.06	288	258	0.92	100
			MIN	1.25	1.76	1.26	20.3	5.91	222	176	0.85	100
50	32.0	12	AV12	1.28	1.89	1.29	21.8	6.19	327	286	0.97	99
			STD	0.03	0.03	0.02	0.6	0.10	19	21	0.02	2
			MAX	1.31	1.94	1.31	22.5	6.31	362	318	1.01	100
			MIN	1.23	1.82	1.24	20.7	6.01	293	245	0.93	95
66	33.0	16	AV16	1.27	1.94	1.27	22.1	6.27	384	347	1.02	97
			STD	0.03	0.03	0.03	0.6	0.08	14	15	0.02	2
			MAX	1.32	1.98	1.32	23.3	6.40	409	371	1.05	100
			MIN	1.22	1.88	1.22	21.0	6.12	359	323	0.99	95
82	34.0	16	AV16	1.24	1.96	1.25	22.1	6.30	413	377	1.03	97
			STD	0.03	0.04	0.03	0.7	0.11	7	7	0.02	2
			MAX	1.29	2.03	1.29	23.4	6.45	428	390	1.07	100
			MIN	1.17	1.88	1.18	20.6	6.01	399	366	0.98	95
99	35.0	17	AV17	1.21	1.98	1.22	22.1	6.32	426	393	1.04	97
			STD	0.03	0.05	0.03	1.0	0.14	10	9	0.03	2
			MAX	1.25	2.03	1.25	23.3	6.54	437	402	1.07	100
			MIN	1.12	1.84	1.13	19.8	5.98	398	368	0.96	95
119	36.0	20	AV20	1.21	2.00	1.22	22.3	6.36	438	406	1.05	99
			STD	0.03	0.04	0.03	0.7	0.09	6	5	0.02	2
			MAX	1.25	2.07	1.26	23.8	6.57	448	414	1.09	100
			MIN	1.16	1.94	1.17	20.8	6.23	428	396	1.01	95

Figure E.14. Test Pile PDA pile driving summary (Page 1 of 9).

Terracon — Page 2

Case Method & iCAP® Results — PDIPLOT2 2014.2.48.1 - Printed 15-May-2015

I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1 — 24" sq. PSC Spliced (30' top + 70' bottom)

OP: TERRACON — Date: 14-May-2015

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
140	37.0	21	AV21	1.21	2.01	1.21	22.5	6.40	446	415	1.06	100
			STD	0.02	0.03	0.02	0.5	0.08	8	7	0.02	1
			MAX	1.24	2.05	1.24	23.2	6.54	458	430	1.09	100
			MIN	1.15	1.94	1.16	21.3	6.23	429	399	1.02	96
161	38.0	21	AV21	1.21	2.01	1.22	22.5	6.42	459	428	1.08	100
			STD	0.02	0.03	0.02	0.4	0.07	8	8	0.02	0
			MAX	1.24	2.05	1.24	23.4	6.51	472	440	1.11	100
			MIN	1.15	1.95	1.16	21.7	6.31	436	407	1.03	100
179	39.0	18	AV18	1.20	1.98	1.21	22.2	6.38	451	420	1.07	100
			STD	0.02	0.03	0.02	0.5	0.06	9	9	0.02	0
			MAX	1.23	2.03	1.24	23.0	6.48	464	434	1.10	100
			MIN	1.14	1.92	1.16	21.2	6.23	430	400	1.02	100
198	40.0	19	AV19	1.21	2.00	1.22	22.5	6.40	455	424	1.08	100
			STD	0.02	0.03	0.02	0.5	0.07	8	7	0.02	0
			MAX	1.24	2.04	1.25	23.3	6.51	469	437	1.10	100
			MIN	1.17	1.93	1.18	21.4	6.20	437	408	1.03	100
217	41.0	19	AV19	1.19	2.00	1.20	22.3	6.44	463	437	1.06	100
			STD	0.03	0.04	0.03	0.7	0.09	11	11	0.03	0
			MAX	1.24	2.07	1.25	23.6	6.60	479	453	1.11	100
			MIN	1.13	1.94	1.13	21.2	6.25	439	412	1.00	100
238	42.0	21	AV21	1.13	2.01	1.14	22.1	6.47	479	458	1.05	99
			STD	0.03	0.03	0.03	0.4	0.07	11	11	0.02	2
			MAX	1.20	2.07	1.20	23.1	6.63	504	482	1.09	100
			MIN	1.05	1.95	1.10	21.2	6.31	465	443	1.00	96
262	43.0	24	AV24	1.13	2.02	1.13	22.2	6.55	502	486	1.07	98
			STD	0.03	0.04	0.03	0.7	0.10	12	10	0.03	2
			MAX	1.18	2.09	1.18	23.6	6.69	521	502	1.12	100
			MIN	1.07	1.94	1.06	20.9	6.31	469	468	1.01	96
286	44.0	24	AV24	0.96	1.91	1.00	21.0	6.55	494	474	1.03	97
			STD	0.35	0.29	0.32	3.0	0.19	49	49	0.09	2
			MAX	1.20	2.10	1.21	23.5	6.79	542	531	1.12	100
			MIN	0.00	1.02	0.11	10.9	5.83	319	302	0.77	93
322	45.0	36	AV36	0.68	1.71	0.70	20.6	6.75	434	420	0.85	96
			STD	0.10	0.11	0.11	1.6	0.16	16	14	0.03	2
			MAX	0.78	1.83	0.82	22.8	7.28	450	435	0.90	100
			MIN	0.28	1.38	0.32	15.3	6.45	363	356	0.77	95
351	46.0	29	AV29	0.74	1.79	0.78	21.4	6.67	438	425	0.85	95
			STD	0.04	0.04	0.04	0.7	0.13	5	5	0.02	1
			MAX	0.80	1.85	0.84	22.6	6.91	447	434	0.89	100
			MIN	0.65	1.69	0.68	19.9	6.37	427	416	0.80	95
384	47.0	33	AV33	0.72	1.80	0.75	21.6	6.70	443	430	0.87	95
			STD	0.04	0.04	0.03	0.7	0.13	6	6	0.02	0
			MAX	0.78	1.88	0.81	22.6	6.98	454	441	0.92	96

Figure E.15. Test Pile PDA pile driving summary (Page 2 of 9).

Terracon
Case Method & iCAP® Results

Page 3
PDIPLOT2 2014.2.48.1 - Printed 15-May-2015

I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1
OP: TERRACON

24" sq. PSC Spliced (30' top + 70' bottom)
Date: 14-May-2015

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
			MIN	0.64	1.71	0.68	20.0	6.37	430	419	0.81	95
417	48.0	33	AV33	0.69	1.80	0.70	21.6	6.77	448	438	0.90	96
			STD	0.02	0.03	0.02	0.5	0.09	5	6	0.02	0
			MAX	0.74	1.87	0.75	23.1	6.95	462	453	0.94	96
			MIN	0.64	1.72	0.65	20.6	6.54	438	427	0.84	95
455	49.0	38	AV38	0.66	1.82	0.67	21.9	6.84	458	448	0.94	96
			STD	0.03	0.03	0.03	0.6	0.08	5	5	0.02	0
			MAX	0.70	1.86	0.71	23.1	7.01	467	458	0.99	96
			MIN	0.59	1.75	0.61	20.3	6.66	445	435	0.91	95
494	50.0	39	AV39	0.64	1.82	0.65	22.1	6.91	466	458	0.98	96
			STD	0.02	0.03	0.02	0.5	0.06	4	4	0.01	0
			MAX	0.68	1.88	0.69	23.0	7.04	476	468	1.01	96
			MIN	0.59	1.77	0.59	21.1	6.79	457	449	0.96	96
536	51.0	42	AV42	0.60	1.80	0.61	21.8	6.89	468	460	1.00	96
			STD	0.02	0.03	0.02	0.5	0.08	5	5	0.01	0
			MAX	0.64	1.86	0.65	22.8	7.08	477	469	1.03	96
			MIN	0.55	1.73	0.55	20.6	6.69	460	448	0.96	96
580	52.0	44	AV44	0.57	1.79	0.58	21.7	6.87	475	464	1.01	96
			STD	0.03	0.03	0.03	0.5	0.08	4	5	0.01	0
			MAX	0.63	1.86	0.63	23.0	7.08	482	474	1.03	96
			MIN	0.52	1.74	0.53	20.8	6.76	468	455	0.98	96
619	53.0	39	AV39	0.71	1.99	0.71	26.5	7.77	520	509	1.11	96
			STD	0.10	0.13	0.10	3.0	0.56	25	24	0.06	1
			MAX	0.82	2.13	0.82	29.5	8.31	544	531	1.17	100
			MIN	0.50	1.73	0.51	20.8	6.72	472	461	1.01	95
652	54.0	33	AV33	0.75	2.05	0.76	27.8	8.00	524	512	1.12	98
			STD	0.03	0.04	0.03	0.8	0.15	8	8	0.03	2
			MAX	0.81	2.12	0.81	29.2	8.19	535	523	1.17	100
			MIN	0.67	1.94	0.68	25.3	7.57	504	494	1.04	95
691	55.0	39	AV39	0.74	2.04	0.74	27.5	7.96	520	508	1.13	98
			STD	0.04	0.04	0.04	0.8	0.14	6	6	0.03	2
			MAX	0.82	2.12	0.82	29.1	8.23	530	518	1.19	100
			MIN	0.63	1.91	0.63	25.4	7.64	504	493	1.06	95
728	56.0	37	AV37	0.73	2.05	0.74	27.9	8.06	518	506	1.13	99
			STD	0.04	0.05	0.04	0.9	0.15	7	7	0.03	2
			MAX	0.82	2.14	0.82	29.7	8.36	532	520	1.19	100
			MIN	0.59	1.89	0.59	24.9	7.57	498	488	1.04	95
765	57.0	37	AV37	0.72	2.04	0.72	27.9	8.07	508	495	1.12	99
			STD	0.03	0.04	0.03	1.0	0.14	8	7	0.02	2
			MAX	0.79	2.13	0.79	29.4	8.31	522	509	1.18	100
			MIN	0.63	1.94	0.63	25.6	7.72	490	479	1.07	95
805	58.0	40	AV40	0.71	2.06	0.72	28.6	8.23	490	475	1.13	100

Figure E.16. Test Pile PDA pile driving summary (Page 3 of 9).

I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1
OP: TERRACON

24" sq. PSC Spliced (30' top + 70' bottom)
Date: 14-May-2015

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
			STD	0.03	0.04	0.03	0.7	0.14	12	13	0.02	1
			MAX	0.77	2.14	0.77	30.1	8.44	513	500	1.18	100
			MIN	0.63	1.96	0.64	26.9	7.79	470	451	1.09	96
843	59.0	38	AV38	0.71	2.01	0.71	27.4	8.04	443	426	1.09	100
			STD	0.04	0.05	0.04	1.0	0.17	10	10	0.03	1
			MAX	0.78	2.10	0.78	29.8	8.36	469	454	1.15	100
			MIN	0.61	1.90	0.62	25.0	7.64	427	412	1.02	96
878	60.0	35	AV35	0.71	2.00	0.71	26.8	7.99	430	416	1.04	100
			STD	0.03	0.05	0.03	0.8	0.14	6	6	0.03	1
			MAX	0.78	2.08	0.78	28.1	8.23	441	429	1.11	100
			MIN	0.64	1.91	0.64	25.3	7.72	419	405	0.98	96
915	61.0	37	AV37	0.80	2.09	0.80	29.3	8.55	453	445	1.04	99
			STD	0.06	0.06	0.06	1.2	0.23	13	12	0.03	1
			MAX	0.92	2.19	0.92	31.2	8.84	479	471	1.12	100
			MIN	0.68	1.93	0.68	25.8	7.83	422	416	0.98	96
946	62.0	31	AV31	0.84	2.13	0.84	30.0	8.69	467	460	1.04	98
			STD	0.05	0.05	0.05	0.9	0.18	10	9	0.03	2
			MAX	0.92	2.22	0.92	31.8	8.98	483	475	1.12	100
			MIN	0.68	1.96	0.68	26.8	8.23	442	434	0.95	96
981	63.0	35	AV35	0.83	2.14	0.83	30.2	8.69	461	451	1.07	100
			STD	0.07	0.06	0.07	1.2	0.20	14	16	0.05	1
			MAX	0.99	2.27	0.99	33.1	9.07	501	495	1.16	100
			MIN	0.71	2.02	0.71	28.1	8.36	435	424	0.99	96
1014	64.0	33	AV33	0.92	2.15	0.91	30.3	8.75	467	465	1.02	99
			STD	0.07	0.05	0.06	1.1	0.17	17	19	0.04	2
			MAX	1.04	2.23	1.01	32.2	9.07	502	511	1.12	100
			MIN	0.76	2.05	0.76	28.3	8.48	440	430	0.93	96
1044	65.0	30	AV30	0.99	2.16	0.99	29.8	8.66	482	479	0.96	96
			STD	0.04	0.06	0.04	1.6	0.28	22	22	0.06	1
			MAX	1.06	2.25	1.06	32.5	9.07	514	513	1.06	100
			MIN	0.85	2.01	0.85	26.4	8.15	428	428	0.83	96
1076	66.0	32	AV32	1.01	2.15	1.01	28.5	8.48	471	468	0.93	96
			STD	0.13	0.12	0.14	2.5	0.14	32	39	0.07	1
			MAX	1.29	2.39	1.29	32.0	8.84	544	543	1.08	100
			MIN	0.34	1.56	0.34	16.3	8.19	339	290	0.68	94
1111	67.0	35	AV35	1.08	2.29	1.08	30.3	8.35	472	472	1.00	96
			STD	0.05	0.05	0.05	1.0	0.17	26	26	0.04	0
			MAX	1.16	2.38	1.17	32.0	8.66	521	521	1.09	96
			MIN	0.97	2.18	1.00	28.2	7.99	431	434	0.89	95
1143	68.0	32	AV32	1.05	2.25	1.06	29.3	8.26	446	445	0.94	95
			STD	0.04	0.04	0.03	1.0	0.16	20	21	0.04	0
			MAX	1.11	2.32	1.11	30.9	8.57	481	481	1.03	96
			MIN	0.95	2.16	0.99	26.9	7.87	393	393	0.86	95

Figure E.17. Test Pile PDA pile driving summary (Page 4 of 9).

Terracon
Case Method & iCAP® Results

Page 5
PDIPLOT2 2014.2.48.1 - Printed 15-May-2015

I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1
OP: TERRACON

24" sq. PSC Spliced (30' top + 70' bottom)
Date: 14-May-2015

BL#	depth ft	BLC bl/ft	TYPE	TLS ksi	CSX ksi	TSX ksi	EMX k-ft	STK ft	RMX kips	RX6 kips	CSB ksi	BTA (%)
1180	69.0	37	AV37	1.05	2.26	1.05	29.5	8.36	442	442	0.93	95
			STD	0.03	0.04	0.04	0.9	0.15	19	18	0.04	0
			MAX	1.11	2.36	1.12	31.8	8.79	489	489	1.05	96
			MIN	0.97	2.15	0.95	27.3	7.99	404	403	0.84	95
1214	70.0	34	AV34	1.03	2.25	1.04	29.4	8.41	426	426	0.93	95
			STD	0.05	0.05	0.05	1.1	0.16	23	23	0.03	0
			MAX	1.10	2.35	1.11	31.5	8.70	468	468	1.00	96
			MIN	0.92	2.15	0.93	27.2	8.11	382	382	0.85	95
1249	71.0	35	AV35	1.01	2.25	1.02	29.0	8.41	408	406	0.93	96
			STD	0.05	0.05	0.05	1.0	0.14	23	23	0.04	0
			MAX	1.12	2.36	1.13	31.1	8.70	463	463	1.02	96
			MIN	0.91	2.16	0.92	27.1	8.15	369	369	0.87	95
1284	72.0	35	AV35	1.08	2.35	1.09	31.4	8.95	452	453	0.98	96
			STD	0.06	0.07	0.06	1.8	0.37	39	36	0.05	0
			MAX	1.17	2.45	1.18	33.7	9.42	508	508	1.05	96
			MIN	0.92	2.18	0.94	27.6	8.15	369	369	0.86	95
1321	73.0	37	AV37	1.05	2.37	1.07	31.7	9.05	429	427	0.98	96
			STD	0.06	0.05	0.06	1.1	0.20	33	35	0.04	0
			MAX	1.15	2.46	1.17	33.6	9.37	483	483	1.06	96
			MIN	0.88	2.25	0.89	28.8	8.57	340	340	0.91	95
1360	74.0	39	AV39	0.97	2.43	0.98	32.7	9.24	385	382	1.01	96
			STD	0.07	0.05	0.07	1.0	0.18	23	26	0.04	0
			MAX	1.09	2.51	1.10	34.2	9.62	442	442	1.10	96
			MIN	0.81	2.26	0.83	29.3	8.61	350	336	0.90	95
1410	75.0	50	AV50	0.63	2.46	0.65	33.2	9.35	517	502	1.19	96
			STD	0.12	0.06	0.12	1.4	0.22	67	64	0.09	0
			MAX	0.91	2.57	0.92	35.8	9.83	633	618	1.37	96
			MIN	0.35	2.25	0.37	28.9	8.66	402	394	1.02	95
1479	76.0	69	AV69	0.44	2.42	0.46	34.3	9.18	708	687	1.52	94
			STD	0.10	0.20	0.08	12.6	0.58	96	72	0.27	8
			MAX	0.71	3.07	0.71	134.5	9.73	1,413	1,134	3.55	96
			MIN	0.00	1.01	0.00	6.5	4.65	419	409	0.79	25
1552	77.0	73	AV73	0.73	2.84	0.76	35.4	8.84	728	722	1.80	98
			STD	0.21	0.42	0.18	5.0	0.31	22	17	0.17	2
			MAX	0.91	3.13	0.94	40.2	9.42	763	759	1.99	100
			MIN	0.00	1.05	0.24	10.9	7.21	603	693	1.09	92
1639	78.0	87	AV87	0.85	3.06	0.90	38.4	9.10	796	795	1.91	96
			STD	0.04	0.06	0.04	1.3	0.20	15	15	0.04	1
			MAX	0.94	3.17	0.94	41.2	9.52	817	816	2.01	100
			MIN	0.71	2.82	0.76	33.8	8.53	759	756	1.77	95
1743	79.0	104	AV104	0.75	2.95	0.80	36.8	8.99	776	774	1.87	96
			STD	0.04	0.06	0.03	1.4	0.22	17	17	0.05	1

Figure E.18. Test Pile PDA pile driving summary (Page 5 of 9).

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I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1 | 24" sq. PSC Spliced (30' top + 70' bottom)
OP: TERRACON | Date: 14-May-2015

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
			MAX	0.86	3.11	0.88	40.2	9.47	812	812	1.99	100
			MIN	0.70	2.81	0.73	33.7	8.48	743	743	1.79	96
1844	80.0	101	AV101	0.72	2.87	0.75	36.4	9.15	723	721	1.82	96
			STD	0.03	0.07	0.03	1.3	0.21	16	16	0.04	1
			MAX	0.81	3.00	0.81	39.9	9.73	756	755	1.91	100
			MIN	0.65	2.69	0.69	32.7	8.61	692	690	1.76	95
1936	81.0	92	AV92	0.74	2.81	0.77	36.7	9.26	665	663	1.75	96
			STD	0.02	0.06	0.03	1.3	0.21	17	16	0.04	0
			MAX	0.79	2.92	0.83	39.4	9.78	697	696	1.83	96
			MIN	0.68	2.69	0.68	34.1	8.84	637	635	1.66	96
2010	82.0	74	AV74	0.77	2.82	0.82	37.0	9.30	621	619	1.73	96
			STD	0.03	0.07	0.03	1.6	0.23	11	11	0.06	1
			MAX	0.85	3.05	0.93	41.2	9.83	641	640	1.92	100
			MIN	0.70	2.57	0.74	31.6	8.57	593	587	1.57	95
2067	83.0	57	AV57	0.80	2.92	0.88	37.6	9.37	574	572	1.79	96
			STD	0.03	0.07	0.03	1.4	0.20	14	14	0.06	1
			MAX	0.86	3.06	0.94	40.9	9.78	611	611	1.92	100
			MIN	0.74	2.75	0.79	34.8	8.93	551	549	1.64	95
2121	84.0	54	AV54	0.80	3.00	0.90	37.7	9.35	539	538	1.84	96
			STD	0.03	0.08	0.03	1.4	0.19	12	13	0.07	0
			MAX	0.87	3.21	0.99	41.3	9.83	578	578	2.01	96
			MIN	0.75	2.86	0.83	34.8	8.89	518	516	1.73	95
2172	85.0	51	AV51	0.81	3.08	0.93	38.2	9.43	523	522	1.88	95
			STD	0.03	0.08	0.04	1.6	0.22	14	15	0.07	0
			MAX	0.87	3.23	1.01	41.9	9.89	565	565	2.01	96
			MIN	0.74	2.92	0.85	35.6	9.03	500	499	1.73	95
2219	86.0	47	AV47	0.83	3.09	0.95	37.4	9.33	505	504	1.87	95
			STD	0.04	0.09	0.04	1.7	0.25	12	13	0.08	0
			MAX	0.91	3.25	1.03	40.4	9.78	543	543	2.03	95
			MIN	0.73	2.87	0.84	32.8	8.70	485	485	1.67	94
2263	87.0	44	AV44	0.85	3.11	0.97	37.0	9.31	497	498	1.87	94
			STD	0.04	0.08	0.04	1.6	0.22	15	18	0.07	0
			MAX	0.95	3.28	1.05	40.2	9.73	544	562	2.01	95
			MIN	0.77	2.92	0.89	33.3	8.79	480	480	1.72	94
2307	88.0	44	AV44	0.87	3.21	1.01	38.3	9.47	503	501	1.94	94
			STD	0.04	0.08	0.04	1.6	0.19	22	20	0.07	1
			MAX	0.97	3.39	1.12	42.2	9.94	578	578	2.11	94
			MIN	0.77	2.99	0.92	34.2	9.03	479	479	1.76	92
2351	89.0	44	AV44	0.83	3.24	0.98	38.0	9.46	510	510	1.97	93
			STD	0.04	0.10	0.04	1.9	0.24	10	10	0.09	0
			MAX	0.92	3.41	1.07	41.5	9.89	532	529	2.11	94
			MIN	0.75	3.03	0.88	34.2	8.89	488	492	1.78	93

Figure E.19. Test Pile PDA pile driving summary (Page 6 of 9).

I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1
OP: TERRACON

24" sq. PSC Spliced (30' top + 70' bottom)
Date: 14-May-2015

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
2400	90.0	49	AV49	0.79	3.29	0.94	38.1	9.50	535	534	2.02	94
			STD	0.04	0.10	0.04	1.9	0.26	7	7	0.08	0
			MAX	0.87	3.48	1.05	41.8	10.05	550	548	2.19	94
			MIN	0.71	3.00	0.84	32.6	8.84	523	521	1.85	93
2460	91.0	60	AV60	0.73	3.30	0.88	37.8	9.53	560	558	2.06	94
			STD	0.05	0.09	0.05	1.9	0.23	6	6	0.06	0
			MAX	0.82	3.49	0.97	42.1	10.05	573	571	2.21	94
			MIN	0.58	2.98	0.71	31.9	8.79	545	545	1.93	93
2517	92.0	57	AV57	0.63	3.35	0.81	38.4	9.65	565	563	2.19	93
			STD	0.06	0.09	0.06	1.8	0.23	5	5	0.05	0
			MAX	0.73	3.51	0.92	41.8	10.05	575	574	2.29	93
			MIN	0.47	3.15	0.63	34.8	9.12	555	552	2.07	93
2570	93.0	53	AV53	0.58	3.38	0.77	38.8	9.73	581	580	2.26	92
			STD	0.05	0.08	0.06	1.6	0.22	10	10	0.04	0
			MAX	0.69	3.54	0.88	42.2	10.16	603	603	2.38	93
			MIN	0.45	3.19	0.62	35.1	9.22	555	553	2.15	92
2634	94.0	64	AV64	0.52	3.41	0.71	38.9	9.75	645	637	2.34	92
			STD	0.06	0.09	0.07	1.7	0.24	28	25	0.05	0
			MAX	0.64	3.60	0.86	42.7	10.28	698	685	2.47	92
			MIN	0.38	3.17	0.55	34.3	9.07	606	601	2.19	91
2699	95.0	65	AV65	0.43	3.40	0.63	38.6	9.69	757	737	2.36	90
			STD	0.07	0.10	0.06	1.8	0.25	27	46	0.06	1
			MAX	0.79	3.65	0.83	42.0	10.22	800	783	2.64	91
			MIN	0.33	3.17	0.50	34.3	9.07	693	431	2.23	88
2787	96.0	88	AV88	0.18	3.41	0.36	39.8	9.37	992	976	2.65	90
			STD	0.09	0.29	0.08	4.7	0.57	74	55	0.24	1
			MAX	0.34	3.64	0.54	44.0	10.00	1,054	1,038	2.85	92
			MIN	0.00	1.50	0.09	8.3	4.70	439	606	0.97	88
2876	97.0	89	AV89	0.24	3.41	0.42	39.7	9.46	1,047	1,035	2.59	89
			STD	0.07	0.08	0.07	1.5	0.20	10	9	0.05	0
			MAX	0.37	3.56	0.57	43.2	10.00	1,067	1,055	2.74	90
			MIN	0.03	3.10	0.27	34.1	8.79	1,016	1,008	2.48	88
2972	98.0	96	AV96	0.14	3.33	0.32	38.9	9.56	1,044	1,036	2.61	89
			STD	0.04	0.08	0.05	1.5	0.20	16	17	0.04	0
			MAX	0.24	3.53	0.44	42.7	10.05	1,083	1,082	2.72	90
			MIN	0.03	3.12	0.24	34.8	8.93	1,010	1,003	2.49	89
3100	99.0	128	AV128	0.12	3.09	0.31	36.2	9.40	1,197	1,192	2.52	89
			STD	0.04	0.12	0.03	1.8	0.27	79	80	0.06	1
			MAX	0.22	3.35	0.39	40.9	10.11	1,345	1,339	2.65	90
			MIN	0.00	2.80	0.24	32.0	8.75	1,069	1,062	2.37	88
3231	99.4	314	AV131	0.10	2.81	0.40	33.9	9.23	1,482	1,474	2.80	88
			STD	0.05	0.16	0.05	2.8	0.48	105	123	0.19	1
			MAX	0.55	3.02	0.89	39.2	10.05	1,660	1,657	3.15	90

Figure E.20. Test Pile PDA pile driving summary (Page 7 of 9).

Terracon
Case Method & iCAP® Results

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I4 OVER DEER BRIDGE - EXPERIMENTAL- PILE 1-1
OP: TERRACON

24" sq. PSC Spliced (30' top + 70' bottom)
Date: 14-May-2015

BL#	depth	BLC	TYPE	TLS	CSX	TSX	EMX	STK	RMX	RX6	CSB	BTA
	ft	bl/ft		ksi	ksi	ksi	k-ft	ft	kips	kips	ksi	(%)
			MIN	0.00	1.32	0.33	8.9	4.97	727	727	1.29	86
			Average	0.70	2.63	0.77	32.5	8.62	647	638	1.64	95
			Std. Dev.	0.31	0.58	0.26	6.8	1.10	276	278	0.63	4
			Maximum	1.34	3.65	1.34	134.5	10.28	1,660	1,657	3.55	100
			Minimum	0.00	0.99	0.00	6.5	4.65	21	0	0.23	25

Total number of blows analyzed: 3231

BL# Sensors

1-3231 F3: [9487] 94.3 (1.02); F4: [F269] 91.4 (1.02); A3: [K4153] 355.0 (0.98); A4: [K4154] 395.0 (0.98)

BL# Comments

283 ADDING CUSHION
1478 REPLACE CUSHION FOR A 9"
2700 REMOVING TEMPLATE

Time Summary

Drive 8 minutes 51 seconds 2:45 PM - 2:54 PM (5/14/2015) BN 1 - 283
Stop 47 minutes 43 seconds 2:54 PM - 3:42 PM
Drive 38 minutes 58 seconds 3:42 PM - 4:21 PM BN 284 - 1478
Stop 25 minutes 34 seconds 4:21 PM - 4:46 PM
Drive 34 minutes 5 seconds 4:46 PM - 5:21 PM BN 1479 - 2700
Stop 24 minutes 1 second 5:21 PM - 5:45 PM
Drive 13 minutes 41 seconds 5:45 PM - 5:58 PM BN 2701 - 3231

Total time [03:12:56] = (Driving [01:35:36] + Stop [01:37:19])

Figure E.21. Test Pile PDA pile driving summary (Page 8 of 9).

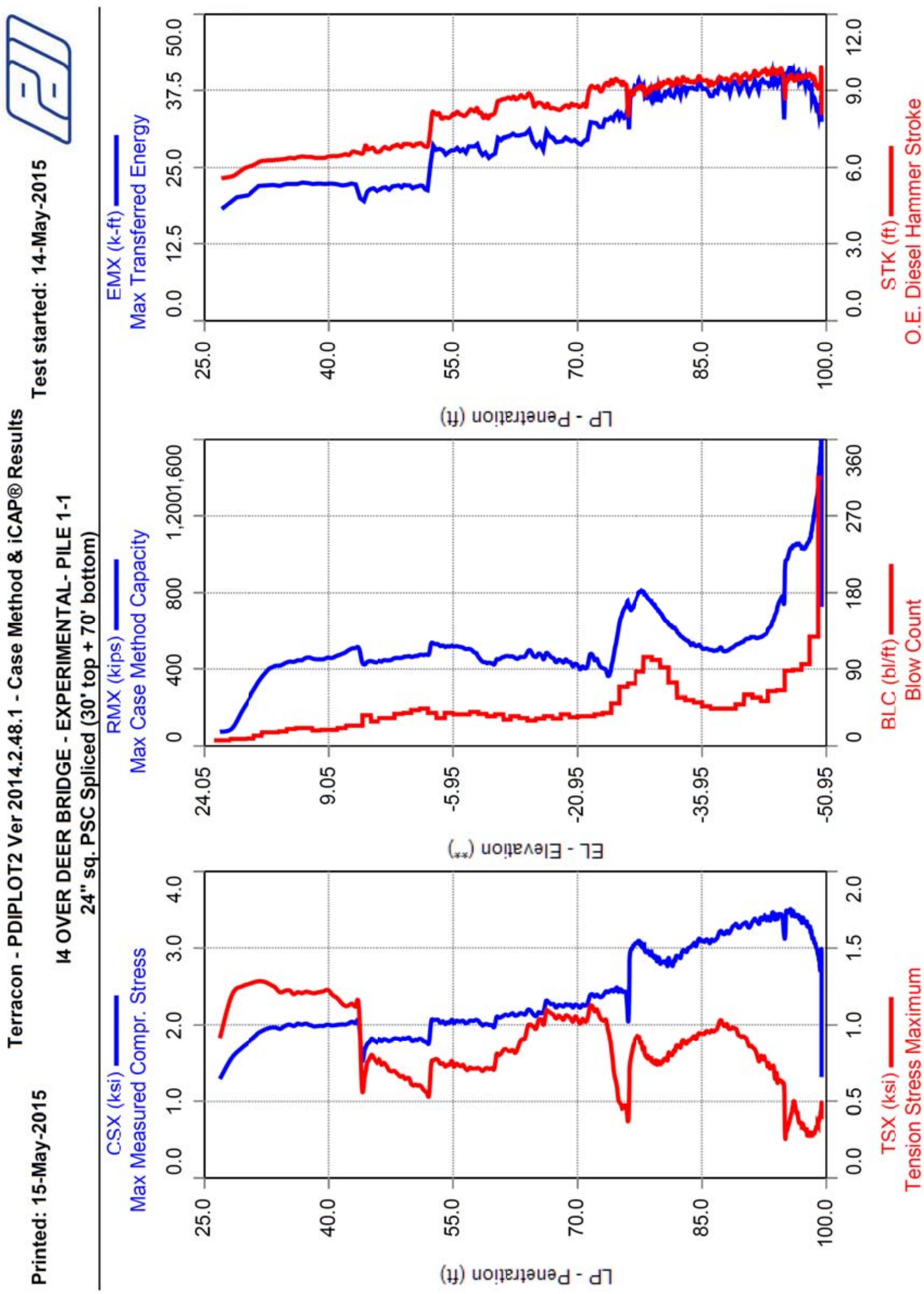

Figure E.22. Test Pile PDA pile driving summary (Page 9 of 9).

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700-010-60
Construction
11/11

PILE DRIVING INFORMATION

Structure Number: DEER CROSSING 790206

FIN PROJ. ID # 408464-1-52-01 DATE MAY 14 2015 STATION NO. 1178+52 13' LT OF ℄ PILE EXPERIMENTAL

PILE SIZE 24" SQ ACTUAL/AUTH LENGTH Bottom 70 / Top 30' BENT/PIER NO. END BENT 3-3 PILE NO. 1-1

HAMMER TYPE APE 46-32 SERIAL # 1991046 RATED ENERGY 107,280 ft/lbs OPERATING RATE VARIES

REF. ELEV +49.05 MIN. TIP ELEV PILE CUTOFF ELEV N/A

DRIVING CRITERIA INSTRUMENTED EXPERIMENTAL PILE

PILE CUSHION THICKNESS AND MATERIAL PINE PLYWOOD compressed to 14½" ① 11¾" 286 Blows ADDED 7½ to Pile Cushion 1480 Blows ⑨ 9" 1755 compressed to 7"

HAMMER CUSHION THICKNESS AND MATERIAL 1½" Aluminum (3 x ½") & 2" Micarta (2 x 1")

WEATHER CLEAR TEMP 87° START TIME 2:45 PM STOP TIME 6:00 PM

PILE DATA

PAY ITEM NO. N/A WORK ORDER NO. N/A

MANUFACTURED BY DURASTRESS T.B.M./B.M. ELEV N/A GROUND ROD READ N/A

DATE CAST 4-17-15 ROD READ N/A PILE HEAD ROD READ N/A

MANUFACTURER'S PILE NO. B1668 CE-1 Top 30' CE-2 Bottom 70' H.I. N/A PILE HEAD ELEV. +49.63

PILE HEAD CHAMFER 3/4" x 3" PILE TIP ELEV. -50.37

PILE TIP CHAMFER 3/4" x 3" GROUND ELEV. +42.76

QUALIFED INSPECTOR'S NAME: Michael HARRINGTON TIN #: H65255860

SPLICE / EACH	PREFORMED HOLE	DYNAMIC LOAD TEST	PAY SET CHECK	NO PAY SET CHECK	REDRIVE	EXTRACTION	DRIVING OF SPLICE	PILE TYPE CODE	BATTER	PILE LENGTH		PENETRATION BELOW GROUND	EXTENSION / BUILD UP	
										ORIGINAL FURNISHED	TOTAL LENGTH WITH EXTENSION		AUTHORIZED	ACTUAL
1.00	0.00	1.00	0.00	0.00	0.00	0.00	1.00	1.00	0.000	Bottom 70' Top 30' 100.000	100.00	93.130	0.000	0.000

NOTES: ① Fuel setting 1
② Fuel setting 2
③ Fuel setting 3
④ Fuel setting 4
⑤ STOPPED DRIVING @ 44' MARK ON PILE ADDED 7½" OF PLYWOOD TO PILE CUSHION
⑥ STOPPED DRIVING @ 76' MARK ON PILE TO CHANGE PILE CUSHION
⑦ STOPPED DRIVING @ 95' MARK ON PILE TO REMOVE TEMPLATE
⑧ STOPPED DRIVING @ 99'5" MARK ON PILE

For Trainee experience evidence only:
Name of CTQP Trainee being supervised by the Qualified Inspector: ______

CTQP Trainee

I certify the Pile Driving Record accuracy and that the named above Trainee has observed the full pile installation:

Qualified Inspector (Signature)

Figure E.23. Splice Pile Driving Logs (Page 1 of 2).

Page No. 2/2

PILE DRIVING LOG

700-010-60
Construction
05/13

DEER CROSSING Structure No. 790206	END BENT Bent/Pier No. 3-3	EXPERIMENTAL PILE Pile No. 1-1

Depth	Blows	Stroke/ Pressure	Note No.	Depth	Blows	Stroke/ Pressure	Note No.	Depth	Blows	Stroke/ Pressure	Note No.	Depth	Blows	Stroke/ Pressure	Note No.
1				34	16	6.39	1	67	35	8.18	4	99'5"	131	9.13	4
2				35	17	6.27	1	68	32	8.15	4				
3				36	20	6.30	1	69	37	8.28	4				
4				37	21	6.36	1	70	34	8.31	4	Splice			
5				38	21	6.31	1	71	35	8.45	4				
6				39	18	6.40	1	72	35	8.53	4				
7				40	19	6.33	1	73	37	8.99	4				
8				41	19	6.41	1	74	39	9.17	4				
9				42	21	6.41	1	75	50	9.04	4				
10				43	24	6.49	1	76	69	9.12	4/6				
11				44	24	6.41	1/5	77	73	8.84	4				
12				45	36	6.46	1	78	87	9.04	4				
13				46	29	6.61	1	79	104	8.92	4				
14				47	33	6.65	1	80	101	9.09	4				
15				48	33	6.72	1	81	92	9.20	4				
16				49	38	6.87	1	82	74	9.22	4				
17				50	39	6.83	1	83	57	9.30	4				
18				51	42	6.84	1	84	54	9.28	4				
19				52	44	6.79	1	85	51	8.99	4				
20				53	39	7.62	1/2	86	47	9.03	4				
21				54	33	7.96	2	87	44	9.24	4				
22				55	39	7.82	2	88	44	9.20	4				
23				56	37	8.08	2	89	44	9.49	4				
24				57	37	8.00	2	90	49	9.43	4				
25				58	40	8.09	2	91	60	9.44	4				
26				59	38	7.99	2	92	57	9.50	4				
27				60	35	7.94	2	93	53	9.71	4				
28	6	5.90	1	61	37	7.62	2/3	94	64	9.69	4				
29	8	5.09	1	62	31	8.68	3/4	95	65	9.62	4/5				
30	8	5.76	1	63	35	8.62	4	96	88	9.27	4				
31	9	6.10	1	64	33	8.68	4	97	89	9.24	4				
32	12	6.10	1	65	30	8.44	4	98	96	9.49	4				
33	16	5.86	1	66	32	8.39	4	99	128	9.32	4				

49.0

2:45 2:47 3:42 4:02 5:45 6:00
2:49 2:55 4:11 4:21 Pile Cushion
ADD PILE CUSHION 4:50 5:21

Figure E.24. Splice Pile Driving Logs (Page 2 of 2).