Report No. BC354 RPWO #80 – Part 2 August 2005
FINAL REPORT – Part 2

Contract Title: Alternatives for Precast Pile Splices
UF Project No. 4910 4504 960 12 (00026867)
Contract No. BC354 RPWO #80

# ALTERNATIVES FOR PRECAST PILE SPLICES

# Part 2


Principal Investigators: Ronald A. Cook
Michael C. McVay

Graduate Research Assistant: Isaac Canner

Project Manager: Marcus H. Ansley

Department of Civil & Coastal Engineering
College of Engineering
University of Florida
Gainesville, Florida 32611

Engineering and Industrial Experiment Station

Technical Report Documentation Page

| 1. Report No. | 2. Government Accession No. | 3. Recipient's Catalog No. |
|---|---|---|
| BC354 RPWO #80 – Part 2 | | |

4. Title and Subtitle

Alternatives for Precast Pile Splices – Part 2

5. Report Date

August 2005

6. Performing Organization Code

7. Author(s)

R. A. Cook, M. C. McVay and I. Canner

8. Performing Organization Report No.

4910 4504 960 12

9. Performing Organization Name and Address

University of Florida
Department of Civil Engineering
345 Weil Hall / P.O. Box 116580
Gainesville, FL  32611-6580

10. Work Unit No. (TRAIS)

11. Contract or Grant No.

BC354 RPWO #80

12. Sponsoring Agency Name and Address

Florida Department of Transportation
Research Management Center
605 Suwannee Street, MS 30
Tallahassee, FL  32301-8064

13. Type of Report and Period Covered

Final Report

Part 2

14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

This project involved the design and field testing of a splice for square precast prestressed concrete piles containing a cylindrical void.  The pile splice incorporates a steel pipe grouted into the cylindrical void.  Part 2 of the report deals with field testing the splice and contains the results of the field testing along with recommended construction and installations guidelines. Part 1 contains the design of the pile splice for tension, compression, flexure, and shear.

17. Key Words

pile splice, precast piles, prestressed piles

18. Distribution Statement

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| 19. Security Classif. (of this report) | 20. Security Classif. (of this page) | 21. No. of Pages | 22. Price |
|---|---|---|---|
| Unclassified | Unclassified | 159 | |

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ACKNOWLEDGMENTS

The field testing for this project would not have been possible without the gracious donations of time, equipment and materials from those involved. We express thanks to the following individuals and companies for their time and resources.

Paul Gilbert and Frank Woods of Wood Hopkins Contracting, LLC allowed piles to be driven in their equipment yard and provided useful input on the assembly process.

Mike Elliott of Pile Equipment Inc. was very gracious in the donation of a Delmag D46-32 diesel hammer and a set of leads for the two-week long testing period. Pile Equipment also provided a hammer operator to assist with the pile driving.

Don Robertson and Chris Kohlhof of Applied Foundation Testing, Inc. monitored both the top and bottom set of accelerometers and strain gages for both pile driving events. Applied Foundation Testing also lent the software (CAPWAP) for the analysis the pile driving data.

Brian Bixler of FDOT performed cone penetration tests at the field site to determine the depth of the rock layer. He facilitated the FDOT's donation of strain transducers and accelerometers that were sacrificed because they went below ground.

Walt Hanford of Degussa Building Systems was very helpful in the selection of grouts for the steel pipe splice.

John Newton of Dywidag Systems International performed the grout mixing and pumping for both pile splices with a consistent grout mix and the correct flow cone time.

Kathy Grey of District 5, FDOT, also lent a PDA unit for one of the pile drive tests.

John Farrell of District 2, FDOT, lent a set of accelerometers, and a PDA unit for one of the pile drives, as well as monitored one of the spliced piles during driving, and provided valuable insight into the analysis of pile driving data.

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# ABSTRACT

## ALTERNATIVES FOR PRECAST PILE SPLICES - PART 2

## FIELD TESTING OF PRESTRESSED CONCRETE PILES SPLICED WITH STEEL PIPES

This project involved the design and field testing of a splice for square precast prestressed concrete piles containing a cylindrical void. The pile splice incorporates a 20 foot long 14 inch diameter steel pipe grouted into the 18 inch diameter cylindrical void of a 30 inch square pile. The material specifications and a description of the construction process are included.

Two spliced piles were driven using a diesel hammer. The forces propagating through the piles during installation were measured using dynamic load testing equipment. The maximum forces were used to calculate the maximum tensile and compressive stresses in the pile to compare these with the allowable pile driving stress limits. The maximum measured tensile stresses exceeded the allowable limit. The maximum measured compressive stress was comparable to the allowable limit. Field observations and review of data acquired during installation indicated no signs of splice deterioration or pile damage.

# CHAPTER 1
# INTRODUCTION

Currently, the Florida Department of Transportation [FDOT] uses a dowel bar splice for prestressed concrete piles (FDOT 2005). The details consist of steel dowels and epoxy mortar. The size and number of dowels depend on the cross sectional area of the pile. There are no standard national guidelines on how to splice together piles; however guidelines suggest that a pile splice should be of equal strength and performance of the unspliced pile (Issa 1999). The steel pipe splice method presented in this thesis is an alternative method to be used for an unplanned splice of a voided 30 inch square prestressed concrete pile.

## 1.1 Problem Statement

An alternative pile splice method was needed for prestressed concrete piles. The alternative method investigated incorporates a steel pipe grouted into the void of the pile. The flexural strength of the steel pipe splice method was verified by laboratory testing (Issa 1999); however the axial capacity of the splice needed to be checked to verify that the stresses caused during pile driving would not cause the splice to fail. Furthermore, the construction method and construction materials needed to be tested in the field environment to determine if the means and methods were adequate to be specified by the Florida Department of Transportation.

## 1.2 Goals and Objectives

The goal of this research was to test the steel pipe splice design, by selecting the best materials and construction method, to determine the axial capacity of the splice. The reason for conducting a full scale pile driving test on the pile splice design was that the stresses caused by pile driving are the largest axial load the pile will be subjected to during its design life. The best way to verify that the steel pipe splice design could withstand the allowable stresses was to drive it in the ground and use dynamic load testing equipment to measure the axial load applied to the pile for each hammer impact. The dynamic load test results would provide the maximum forces carried by the splice, which can be converted to an equivalent stress to compare with the allowable pile driving stress limits from Section 455 of the FDOT Standard Specifications for Road and Bridge Construction (FDOT 2004a) and the computed axial design strength of the splice from the Alternatives for Precast Pile Splices report (Britt, Cook, and McVay 2003).

After proving the minimum axial strength of the splice was greater than the maximum allowable pile driving load, the objective was to create the first draft of the FDOT specification for the steel pipe splice method. This would include:

- Detailed material specifications used in the splice.
- Outline of the construction process to follow for a successful splice.
- Design drawings to illustrate the materials and construction process.

## 1.3 Background

Previous research on the alternative pile splice method in the state of Florida includes both laboratory and field testing. The steel pipe splice method was first tested in the laboratory to determine the flexural capacity of a spliced 30 inch square prestressed concrete pile (Issa 1999). Success in the laboratory was followed by the testing of three splices being constructed at an FDOT site (Goble Rauche Likens and Associates [GRL], Inc. 2000). However, due to problems during construction with assembly of the splice, the pile driving was not successful because of failure of the splice region. The next step was Part 1 of the Alternatives for Precast Pile Splices report (Britt, Cook, and McVay 2003) which calculated the design capacity of the splice, developed a lab test setup to determine the static axial strength, and outlined field assembly guidelines. Details of these projects are presented in the following sections.

### 1.3.1 FDOT Structures Laboratory Flexural Tests

At the FDOT Structures Laboratory in Tallahassee, the splice was tested in flexure with 10 foot and 15 foot long steel pipe splices, to provide 5 feet and 7.5 feet embedment on either side of the joint. A report was written by Issa (1999) on the results of the testing. For both tests, the pipe was a HSS 14.00 x 0.500 and made of grade 42 steel. Rebar was welded to the outside of the pipe at a 6 inch pitch.

The 10 foot long steel pipe splice was tested by simply supporting the ends of the 22 foot long pile, and placing hydraulic jacks at a distance of 2.5 feet from either side of the splice interface to provide a region of uniform moment. The 10 foot long steel pipe splice did not work because horizontal cracks occurred in the splice region at a moment of 255 kip-ft with a failure moment of 581 kip-ft.

The second specimen's steel pipe was a total of 15 feet long and was filled with concrete to prevent buckling of the steel pipe. The 30 foot long pile was simply supported at each end and hydraulic jacks were placed at a distance of 5 feet from either side of the splice interface. The ultimate test moment capacity was observed to be 840 kip-ft.

The unspliced pile had a calculated nominal moment capacity of 1000 kip-ft and the steel pipe spliced pile section had a calculated nominal moment capacity of 878 kip-ft. Therefore, the pile developed 84% of the calculated unspliced pile capacity and 96% of the calculated spliced pile capacity (Issa 1999).

### 1.3.2 Field Testing at St. Johns River Bridge

After completion of the laboratory flexural test of the splice, a minimum splice length of 12 feet was recommended, with 6 feet on either side of the joint (Issa 1999). The splices tested at St. Johns River Bridge were constructed using 20 foot long steel pipes. The steel pipe splice design was tested in the field by driving three 75 foot long piles, splicing a 75 foot long section on top of each, and re-driving the spliced 150 foot long piles. All three spliced piles experienced failure of the splice and the spliced piles would not drive (GRL, Inc. 2003).

Several issues may have contributed to the spliced pile failure. The 75 foot long upper pile section was not released from the crane while the grout in the annulus cured. This may have resulted in the annulus grout not setting properly because of small sway movements of the crane. Secondly, the steel pipe was smooth; a ½ inch diameter steel bar was not welded to the pipe to add deformations to create a mechanical bond. Lastly, an epoxy mortar bed between pile ends was created by placing steel shims at the joint. These steel shims were not removed prior to driving and therefore created four stiff points at the joint. One possible cause of the mating surface to fail during pile driving was stress concentrations in the epoxy grout caused by the difference in elastic modulus between the epoxy grout and the steel shims. It is not known if the splice interface at the pile ends, or the grout in the annulus failed first. If the grout in the annulus had cured properly, the tension stresses caused during driving would have been transferred to the steel pipe through shear and carried across the splice. However, if the epoxy mortar bed and the concrete at the splice mating surface deteriorated, a large discontinuity in cross-section properties would be created. The large decrease in pile impedance at the joint would result in smaller refracted compression waves and larger reflected tension waves at the splice. The reflected tension waves would act to pull the piles apart, which could only be transferred across the splice by the annulus grout through shear transfer.

The problems in the prior splice tests were considered during the design of the new splice and the development of the construction guidelines utilized. For example, the steel pipe was deformed with a ½ inch diameter bar spirally wound at an 8 inch pitch. Also, the steel shims were removed from the splice interface to create a more homogenous transition between pile end materials. Additionally, the pile was released from the crane and supported by an external rigid frame while the annulus grout cured overnight.

### 1.3.3 Previous Steel Pipe Splice Research at the University of Florida

The Alternatives for Precast Pile Splices report by Britt, Cook, and McVay (2003) provides the design of the steel pipe splice for tension, flexure, and compression. The load path for each loading was considered and then designed in order to provide adequate capacity. The minimum length of steel pipe was determined to provide a capacity equal to a continuous unspliced 30 inch square prestressed concrete pile. The minimum length of steel pipe included the development and transfer lengths of the steel pipe and strands in the concrete. The required length of steel pipe embedment was determined to be 7 feet, for a 14 foot long pipe as shown in Figure 1-1.

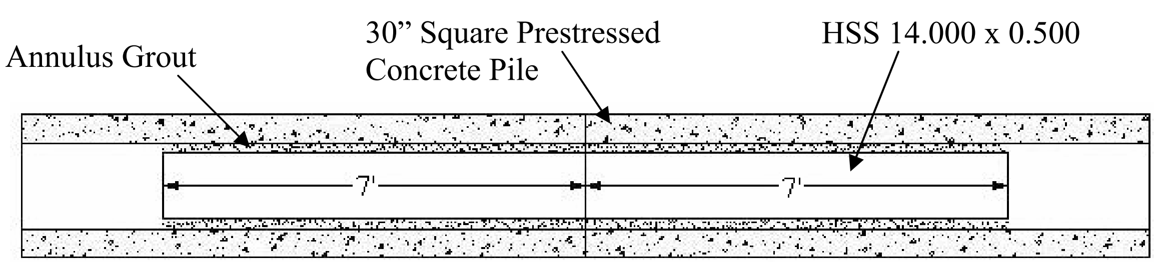



Figure 1-1 The steel pipe splice components and minimum splice length.

After the splice failures during pile driving at the St. Johns River Bridge (GRL, Inc. 2003), the axial design of the splice was investigated. The splice was designed to resist the pile driving load. The load from the hammer was transferred from the pile to the steel pipe through the grout in the annulus. A mechanical bond was provided between the inside of the pile, the grout, and the deformed steel pipe. In tension, the steel pipe carries the entire load across the splice mating surface. The steel pipe has a cross sectional area of 19.8 $in^2$ and is Grade 42 steel; therefore the pipe can resist a tensile load of 832 kips before yielding.

The nominal moment capacity of an unspliced 30 inch pile was determined to be 966 kip-ft. The nominal moment capacity of the steel pipe spliced section was determined to be 855 kip-ft (Britt, Cook, and McVay 2003).

# CHAPTER 2
# PILE SPLICE TEST SPECIMEN MATERIALS

This chapter presents information on the materials that were used to construct the splice. Two steel pipe splices were constructed using the same prestressed concrete piles, hollow structural steel pipes, cementitious annulus grout, and mating surface grout.

## 2.1 Prestressed Concrete Piles

The prestressed concrete piles tested were constructed by Standard Concrete Products of Tampa, FL. The FDOT standard drawing Index No. 630 (FDOT 2005) was used to specify the two 40 feet long 30 inch square prestressed concrete piles with a strand pattern of twenty 0.6 inch diameter, 270 Low Relaxation Strands, at 41 kips each. The solid ends of the pile were 4 feet long and the middle 32 feet section was hollow with a mean diameter of 18 inches as shown in Figure 2-1.

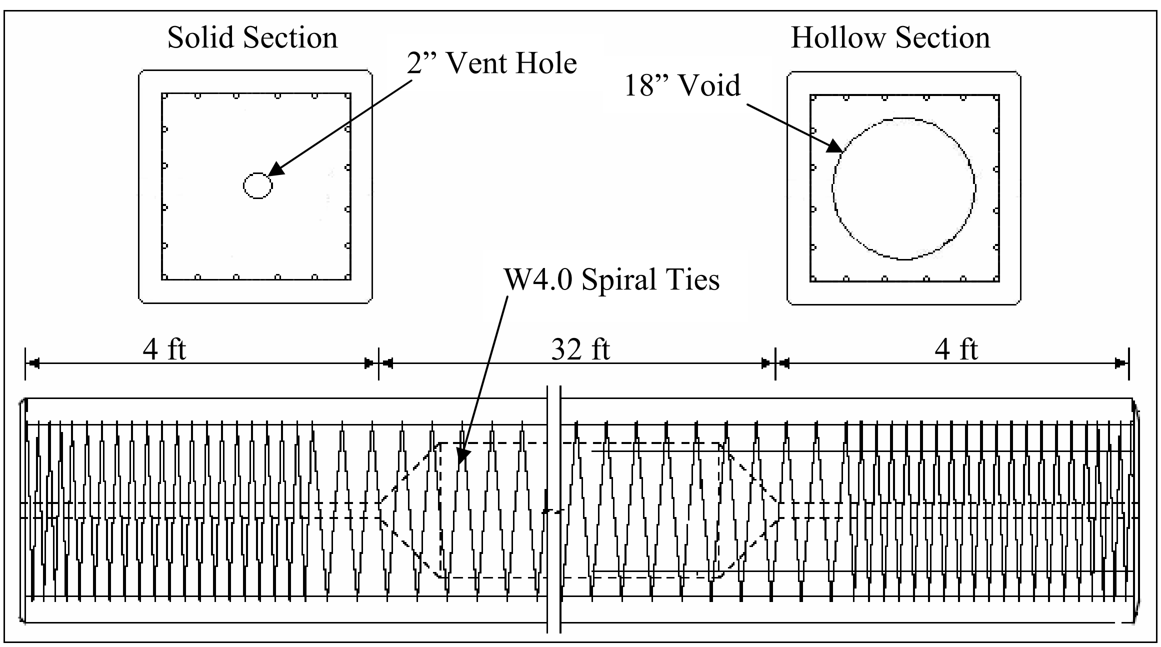



Figure 2-1 Details of 30 inch square prestressed concrete pile as constructed.

The form used to construct the void was requested to be corrugated metal for the entire length as shown in Figure 2-2. The depth of the corrugation was 0.5 inches, measured as the vertical distance from a straight edge resting on the corrugation crests to the bottom of the intervening valley (ASTM A760 1994).

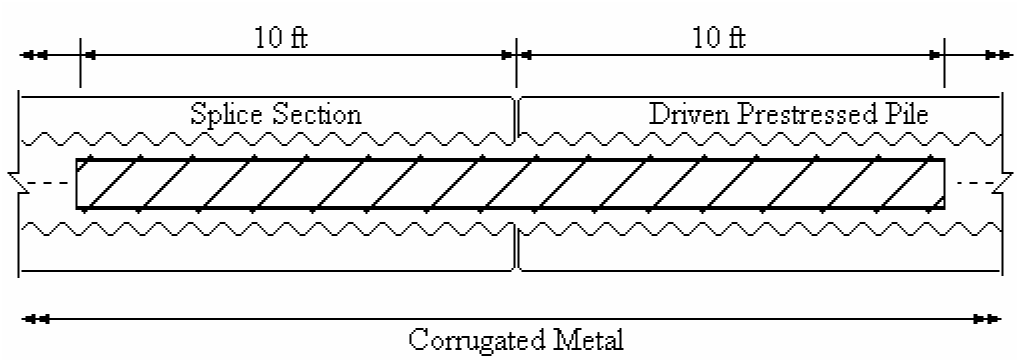



Figure 2-2 Corrugated metal for the entire length of void is required.

After driving both piles and cutting them in half, it was discovered that corrugated metal was used to form 20 feet of the 32 foot void length, with the remaining 12 feet being cardboard sonotube. The top half was entirely corrugated metal. The bottom half of pile in the ground had 4.5 feet of corrugated metal below the cutoff elevation, and the remaining 7.5 feet below were cardboard sonotube. Figure 2-3 shows the corrugated metal liner in the splice section on the left side, and the cutoff driven pile on the right side with both corrugated metal and cardboard sonotube.

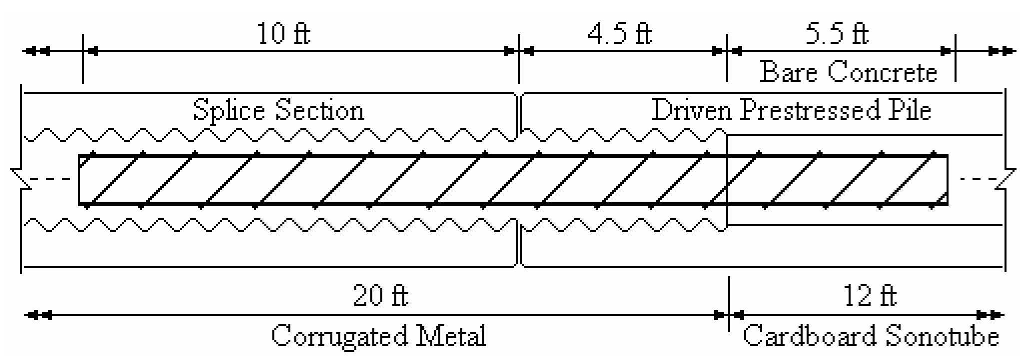



Figure 2-3 Pile void material location for piles used in pipe splice test.

In future applications of the steel pipe splice, the piles should be required to have a corrugated metal pipe to form the void. Metal void liner was requested for the entire void, but was not provided for the entire void, the only option was to remove the cardboard and continue the splice construction. To strip the cardboard, the void in the pile was filled with water and allowed to soak overnight. The next morning the cardboard was stripped using a variety of tools to expose smooth bare concrete.

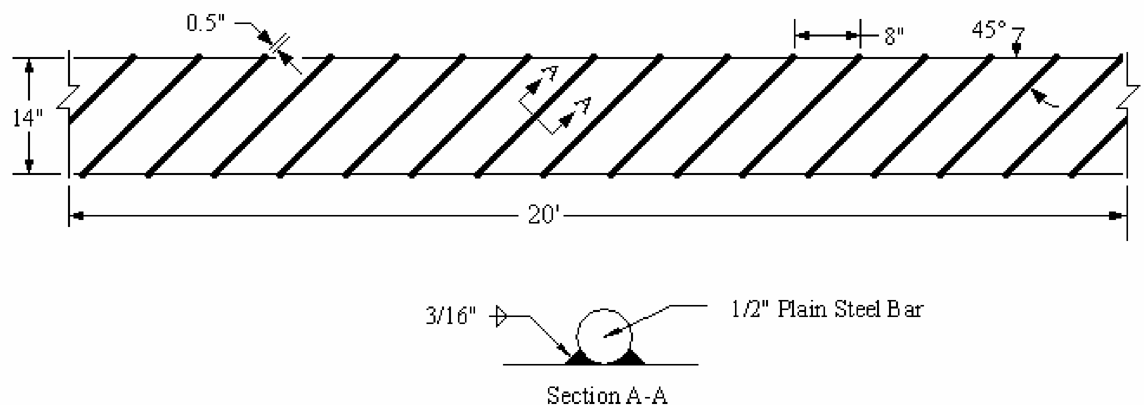
Galvanized steel pipe will no longer be used to form the void of prestressed concrete piles, because the potentials developed upon the steel strands is of sufficient magnitude and duration to cause hydrogen embrittlement of the strands (Hartt and Suarez 2004).

Acceptable alternatives to galvanized steel pipe would be either bare steel corrugated pipe or two options provided by Contech are Aluminized Steel Type 2, which is bare steel hot-dipped in commercially pure aluminum, or a polymer coated steel pipe, such as Trenchcoat (Contech Products 2005).

## 2.2 HSS Steel Pipe with Shear Transfer Mechanism

The steel pipe used to splice the piles was a 20 foot long HSS 14.000 x 0.500. The preferred material specification for round Hollow Structural Sections [HSS] is ASTM A500 grade B with minimum yield stress of 42 ksi (AISC 2001). The minimum design length of the steel pipe recommended in the Alternatives for Precast Pile Splices report (Britt, Cook and McVay 2003) was increased from 14 feet to 20 feet, providing 10 feet of bond length on both sides of the splice.

Prior to testing, the steel pipe was prepared with ½ inch diameter plain steel bar welded to the pipe to provide deformations at 8 inch spacing. The bar was spirally wound and fillet welded in position with two inches of 3/16 inch fillet weld per foot of steel bar as shown in Figure 2-4.




A

B

Figure 2-4 HSS steel pipes. A) Details of pipe with welded bars, B) HSS steel pipes with bars as-built.

Steel hoops could also be used and would likely be more cost effective than the spirally wound bar. After forming them to 14 inch diameter hoops, they would be welded to the pipe at 8 inch spacing with two inches of 3/16 inch fillet weld per foot of steel bar.

The steel pipe was filled with concrete to prevent local buckling when loaded in bending. To allow gasses to escape through the spliced section, a 3 inch diameter pipe was provided inside of the 14 inch diameter pipe. To accomplish this, a 14 inch diameter steel plate with a 3 inch diameter center hole was welded to the bottom end of the 14 inch diameter steel pipe. The 3 inch diameter steel pipe was welded in place, and the 14 inch diameter steel pipe was filled with normal weight concrete. The steel pipe filled with concrete weighed approximately 2 tons.

### 2.3 Annulus Cementitious Grout

One of the most critical parts of the splice was the grout in the annulus that bonded the HSS steel pipe to the inside of the pile. The grout provided a mechanical bond because of the deformations on the steel pipe and the corrugation on the inside of the pile. Degussa Building Systems's product Masterflow 928, a high-precision mineral-aggregate grout with extended working time was chosen as the best option. The Masterflow 928 product specification sheet is attached in Appendix A. The extended working time was essential because 14 cubic feet or 30 bags of grout had to be mixed and pumped continuously into the splice. This requirement eliminated the possibility of using a polymer epoxy grout or a rapid setting cementitious product such as Master Builders 747 Rapid Setting Grout. Another requirement of the grout was that it be designated a non-shrink grout and reach 3800 psi within 20 hours.

The products on the FDOT list of approved post-tensioning grouts were fluid and could be pumped into the annulus, but did not have the required 24 hour compressive strength for this type of dynamic loading. No prior FDOT specification existed for this type of grouting application.

The fluid grout consistency was used to ensure good consolidation in the small crevices in the annulus of the splice and to fill the 20 foot grout head. According to the product specification sheet, at a fluid consistency, the unit weight of Masterflow 928 was approximately 135 pounds per cubic foot and the flow cone time was between 25-30 seconds per ASTM C939. The compressive strength for the fluid consistency was 3500 psi after 1 day, and 7500 psi after 28 days.

Dywidag Systems International performed the grout mixing and pumping using their colloidal mixer with an agitator holding tank. Two large air compressors were used to power the mixers and pump. The mixer had a water tank with a volume measurement so that the mixing process could be consistently repeated, after a trial batch was mixed with the correct water volume to achieve the required flow time. The first batch of grout was mixed and the flow cone time was measured at 44 seconds for Pile #1. The product specification sheet specified a flow time between 25 and 30 seconds for a fluid grout consistency. A longer flow time corresponded to a more plastic grout; therefore water was added to decrease the flow time to 30 seconds for Pile #1, before pumping continued. For Pile #2, the first flow time was measured at 22 seconds; the grout mix was adjusted to a flow time of 35 seconds before pumping continued.

During the grouting process, grout cubes were cast for testing in accordance with ASTM C942. Before driving the spliced piles, the grout cubes were tested to measure the compressive strength.

Pile #1 was spliced and driven 24 hours after the grout pumping was completed when the annulus grout cube compressive strength was 4500 psi. Pile #2 was spliced and driven 20 hours after the grout pumping was completed. The minimum grout compressive strength required was set at 3800 psi because spliced Pile #2 was driven successfully when the grout cube compressive strength was equal to 3800 psi. Figure 2-5 is a plot of the average compressive strength of the grout cubes. Each point represents the average of three cubes tested.

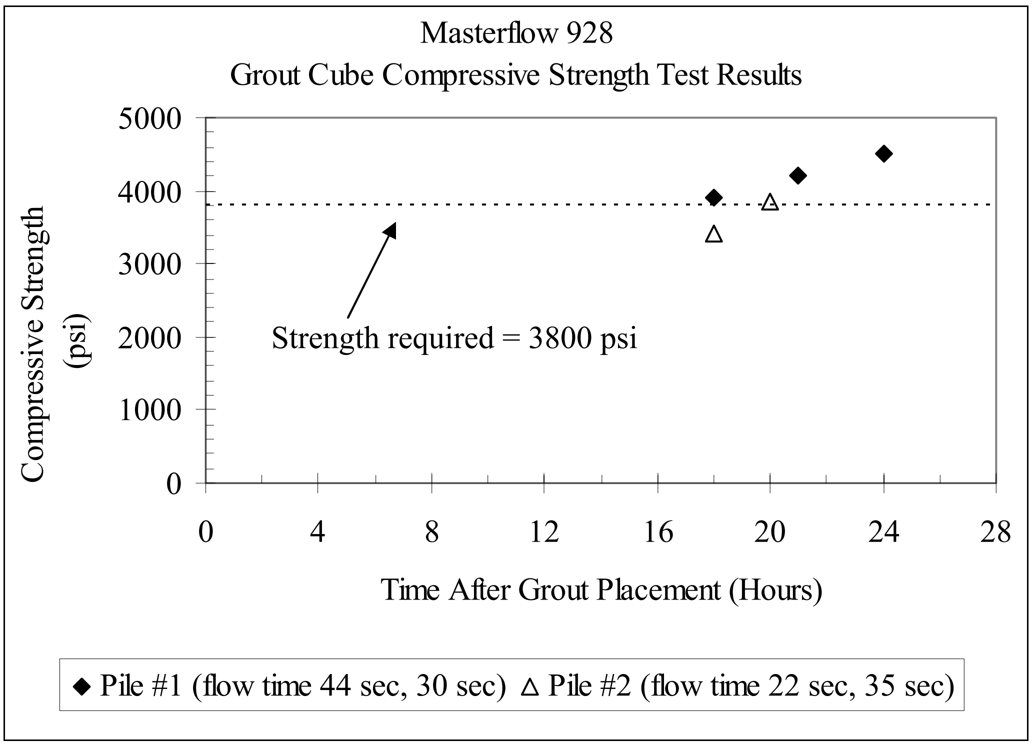



Figure 2-5 Masterflow 928 annulus grout cube compressive strength test results.

The characteristics of the Masterflow 928 annulus grout are outlined below. An equivalent product could be used in the annulus of the splice, provided that it meets the requirements outlined below:

- Designated as a non-shrink grout.
- Extended working time to allow continuous placement of 14 cubic feet.
- Fluid consistency pumpable into the 2 inch wide by 20 feet vertical splice annulus.
- High early compressive strength: minimum 3800 psi.

## 2.4 Mating Surface Grout

At the mating surface between the two piles a rapid setting mortar was needed to fill and seal the gap between the piles. The fluid Masterflow 928 grout would leak if the mating surface was not sealed. The other purpose of the mating surface grout was to provide compressive force transfer between the pile ends. The characteristics of the mating surface grout are outlined below:

- High compressive strength with a cure time less than one hour.
- Easy to trowel onto the mating surface in a mortar bed.
- Good workability so the contractor has time to align the piles plumb.
- Provide a seal at the mating surface for the grout to be pumped into the annulus.

The pile head was removed using an air powered diamond blade circular saw and a choker cable from the crane. After the saw cut through the prestressing strands the crane slowly bent the pile until it broke. When the splice section was lowered into position, the gap at the mating surface was measured at the outer edge and ranged from 0.5 to 1 inch depending on the side of the pile.

Initially for the splice mating surface, Concresive 1420 general purpose gel epoxy adhesive seemed like the best product because of its high strength and ability to seal the mating surface.

While in the field on the day of the splice assembly, the plan to use Concresive 1420 general purpose gel epoxy adhesive changed because the product was supplied in two-part tubes with a mixing gun to apply it. If the product were supplied in a gallon bucket, the volume required could have been mixed at once and applied to the mating surface. However, for the supply on hand, the volume required to fill the gap was too large to dispense using tubes. Also, after mixing a trial batch, the product setup too quickly and would not give the contractor enough time to align the piles plumb. The

FDOT dowel splice method had a similar problem of short setup time with an epoxy adhesive.

The Degussa Building Systems product Set 45 was used because it had sufficient working time with a quick setup and high strength. Two bags were enough to spread a bed of mortar on the mating surface as shown in Figure 2-6. The Set 45 was mixed with the minimum recommended water volume. The extra mortar was pushed out when the top pile was lowered into position. A plywood form was not used because it was not needed for the mortar consistency. However, a plywood form should be required for FDOT jobs for quality control, and to ensure the gap is entirely filled no matter what the water content. The Set 45 product specification sheet is attached in Appendix A.

A B

Figure 2-6 The Set 45 mating surface grout. A) Apply mating surface grout, B) ready to lower the top pile into position.

At this point during construction it was important for the spliced pile section to be braced from moving while the grout cured. For this test, the top pile was braced in position by the template with wood wedges holding it plumb when the crane cable was released as shown in Figure 4-8. After about 45 minutes, the mortar was solid and the grout could be pumped into the annulus without leaking as shown in Figure 2-7 below.

Figure 2-7 Set 45 grout used to seal mating surface after curing 45 minutes.

# CHAPTER 3
# ANALYSIS OF DRIVING A PRESTRESSED CONCRETE PILE

This chapter discusses the methods used to analyze the soil profile and the prestressed concrete pile driving at the site where the steel pipe splice tests were conducted. The pile driving hammer was selected for the pile size and soil profile at the site. The goal of this analysis was to determine the effect of the weak layers and stiff layers in the soil profile on the pile capacity and maximum stresses in the pile during driving.

## 3.1 Pile Driving Test Site Selection

The pile splice test site was selected based on several factors. An initial goal was to find a test site that had a layered soil stratum with Florida limestone approximately 40 feet below grade. A shallow limestone rock layer was desired because a shorter pile length would be less expensive and more easily handled by the contractor.

A soil profile consisting of both strong and weak layers was preferred to test the splice design under the most strenuous pile driving conditions. The pile resistance is a combination of side friction along the length of the pile and end bearing at the tip. The relative magnitude of side friction to end bearing will cause different magnitudes of stresses in the pile during driving. Layers of sand, silt, and clay would provide the type of pile driving conditions necessary to stress the pile in both tension and compression.

## 3.2 Cone Penetration Test from Field Site

The University of Florida Cone Penetration Test [CPT] truck was used to determine the soil profile at the test site in Jacksonville. The cone was continuously pushed into the soil at a rate of about 20 mm/sec powered by hydraulics in the truck. The electronic cone penetrometer measured end resistance and sleeve friction on the steel cone as a function of depth. The friction ratio, Rf, was equal to the sleeve friction divided by the tip resistance on the cone. The friction ratio was used to classify the soil into cohesive and cohesionless layers based on Table 3-1.

Table 3-1 Soil classification based on friction ratio.

| Soil Type | Rf |
|---|---|
| Sand | $0 < Rf < 1.5$ |
| Silt | $1.5 < Rf < 3.0$ |
| Clay | $3.0 < Rf < 6.0$ |

At the test site in Jacksonville, two cones were pushed into the ground, approximately 130 feet apart, numbered 9604 and 9606, on either side of the proposed pile driving location. The sleeve friction and end bearing on the electronic cone penetrometer was measured from ground elevation to the impenetrable rock layer, possibly limestone. Both cone tests showed similar soil profile layer data and the impenetrable rock layer at a depth of 31 feet below grade. The truck moved when the cone was pushed into the rock layer. The pressure was released to avoid bending the steel rod. Figure 3-1 is a plot of the sleeve friction, end bearing, and friction ratio recorded from each cone sounding with soil layer divisions of cohesive and cohesionless.

The piles were driven 30 feet away from the cone penetration test hole. During driving of spliced pile #1, the rock layer was not encountered at 31 feet below grade as predicted by both CPT results. Two additional cones were pushed adjacent to the piles to determine the depth of limestone rock. The CPT test performed 15 feet east of pile #1 showed the rock layer at elevation -36 feet. The CPT test performed 20 feet west of pile #2 showed the rock layer at elevation -39 feet.

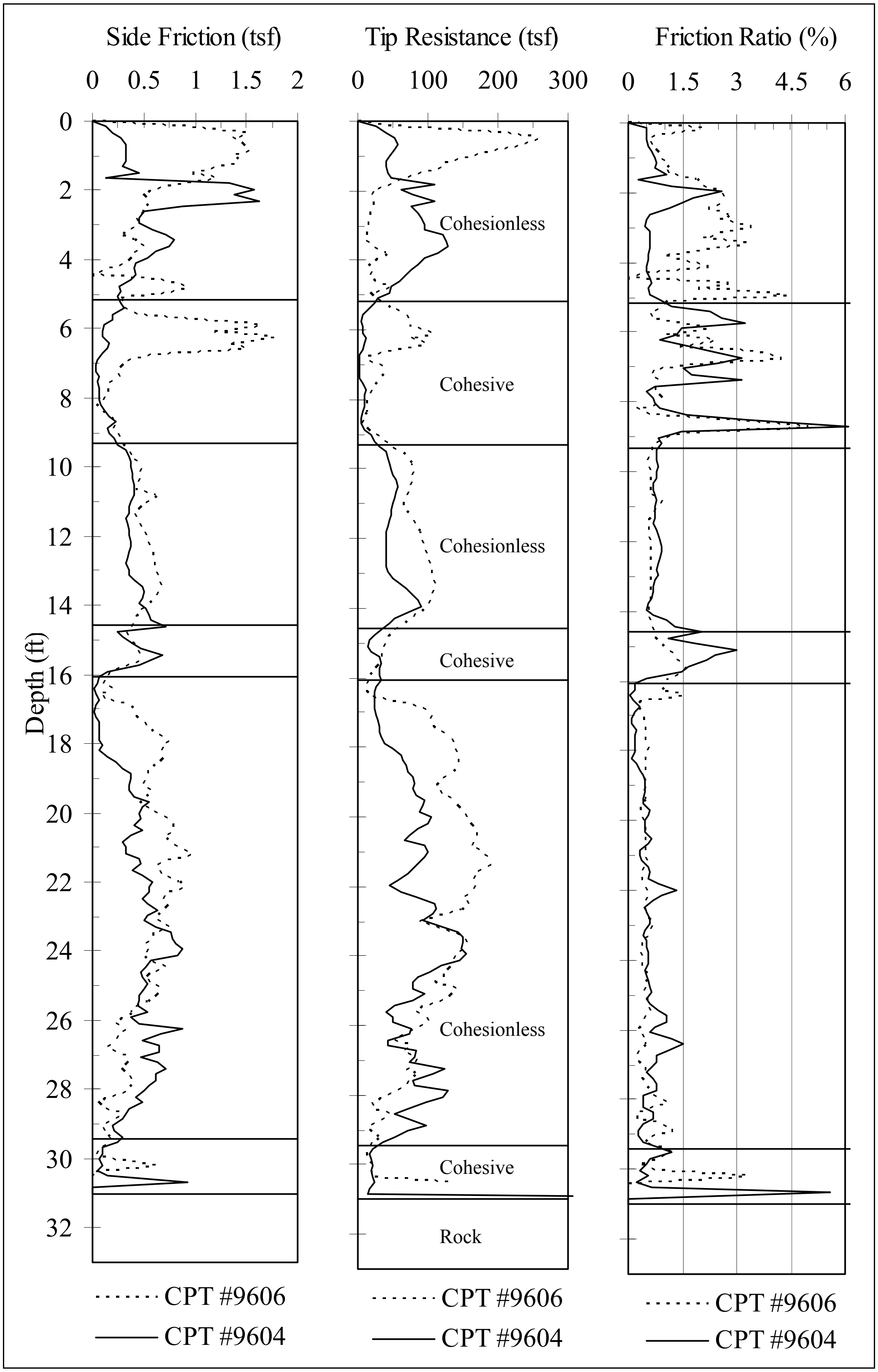



Figure 3-1 CPT results with soil divided into layers of cohesive and cohesionless.

## 3.3 Software Analysis of Pile Driving at the Test Site

Geotechnical engineering software was used to estimate the side friction and end bearing on a 30 inch square prestressed concrete pile from the CPT data recorded at the test site. The side friction and end bearing was used to model the soil profile in GRL, Inc. software titled GRLWEAP, which was used to simulate the proposed pile driving hammer system.

### 3.3.1 Static Pile Capacity Assessment with PL-AID

The PL-AID software was used to estimate the static pile capacity, which was a combination of side friction and end bearing. The data recorded by the cone penetrometer was input into PL-AID with the pile material, cross section, and length to determine the unit side friction and unit end bearing on a 30 inch square prestressed concrete pile as a function of depth. The PL-AID software output the design side friction and end bearing in tons at one foot depth increments. PL-AID used the minimum path rule (AASHTO 2004a) considering the soil 8 diameters above the tip and 0.7 to 4 diameters below the tip to determine the tip resistance. The output from PL-AID was a table of the estimated static pile capacity versus tip elevation as shown in Table 3-2.

The ultimate unit side friction was calculated by multiplying the average side friction for a layer by two to get an ultimate value and dividing by the surface area of pile in the layer. The ultimate end bearing was calculated by multiplying the design value by three to get an ultimate value. The side friction on a prestressed concrete pile can also be estimated as 40% of the side friction recorded on the cone penetrometer. The output of these calculations was shown in Figure 3-2 as a plot of side friction and end bearing on a 30 inch square prestressed concrete pile versus depth. The shape of the plot was similar to the CPT results highlighting both strong and weak layers. The ultimate unit side friction and ultimate end bearing on a 30 inch square concrete pile were used to model the soil profile at the test site for GRLWEAP software analysis.

Table 3-2 PL-AID static pile capacity analysis output.

| Test Pile Length feet | Pile Tip Elevation feet | Design Side Friction tons | Design End Bearing tons | Design Pile Capacity tons | Ultimate Pile Capacity tons | Factor of Safety |
|---|---|---|---|---|---|---|
| 2 | -2 | 0.64 | 24.1 | 24.7 | 73.5 | 0.24 |
| 3 | -3 | 1.62 | 20.1 | 21.7 | 63.5 | 0.21 |
| 4 | -4 | 2.65 | 13.3 | 16.0 | 45.3 | 0.15 |
| 5 | -5 | 3.31 | 7.30 | 10.6 | 28.5 | 0.09 |
| 6 | -6 | 4.24 | 6.40 | 10.7 | 27.7 | 0.09 |
| 7 | -7 | 4.81 | 6.70 | 11.5 | 29.6 | 0.10 |
| 8 | -8 | 5.07 | 20.4 | 25.4 | 71.2 | 0.24 |
| 9 | -9 | 5.73 | 32.3 | 38.0 | 108 | 0.36 |
| 10 | -10 | 7.7 | 34.2 | 41.9 | 117 | 0.39 |
| 11 | -11 | 11.2 | 34.6 | 45.7 | 126 | 0.42 |
| 12 | -12 | 13.6 | 35.3 | 48.9 | 133 | 0.44 |
| 13 | -13 | 16.0 | 36.4 | 52.4 | 141 | 0.47 |
| 14 | -14 | 19.0 | 32.7 | 51.7 | 136 | 0.45 |
| 15 | -15 | 21.5 | 32.2 | 53.6 | 139 | 0.46 |
| 16 | -16 | 22.8 | 36.0 | 58.8 | 153 | 0.51 |
| 17 | -17 | 22.7 | 56.7 | 79.4 | 215 | 0.71 |
| 18 | -18 | 22.7 | 73.3 | 95.9 | 265 | 0.87 |
| 19 | -19 | 23.6 | 75.9 | 99.5 | 274 | 0.91 |
| 20 | -20 | 25.9 | 73.3 | 99.2 | 271 | 0.89 |
| 21 | -21 | 28.4 | 88.4 | 117 | 322 | 1.06 |
| 22 | -22 | 30.6 | 115 | 145 | 405 | 1.33 |
| 23 | -23 | 33.3 | 96.3 | 129 | 355 | 1.17 |
| 24 | -24 | 36.8 | 81.6 | 118 | 318 | 1.04 |
| 25 | -25 | 39.2 | 79.9 | 119 | 318 | 1.04 |
| 26 | -26 | 41.3 | 98.5 | 139 | 378 | 1.24 |
| 27 | -27 | 44.3 | 84.7 | 128 | 342 | 1.12 |
| 28 | -28 | 47.0 | 51.8 | 98.7 | 249 | 0.82 |

### 3.3.2 GRLWEAP Software Analysis

The Wave Equation Analysis for Piles (WEAP) is the standard method to evaluate the suitability of the Contractor’s proposed pile driving system, as well as to estimate the driving resistance, in blows per 12 inches, to achieve the pile bearing requirements, and to evaluate pile driving stresses (FDOT 2004a).

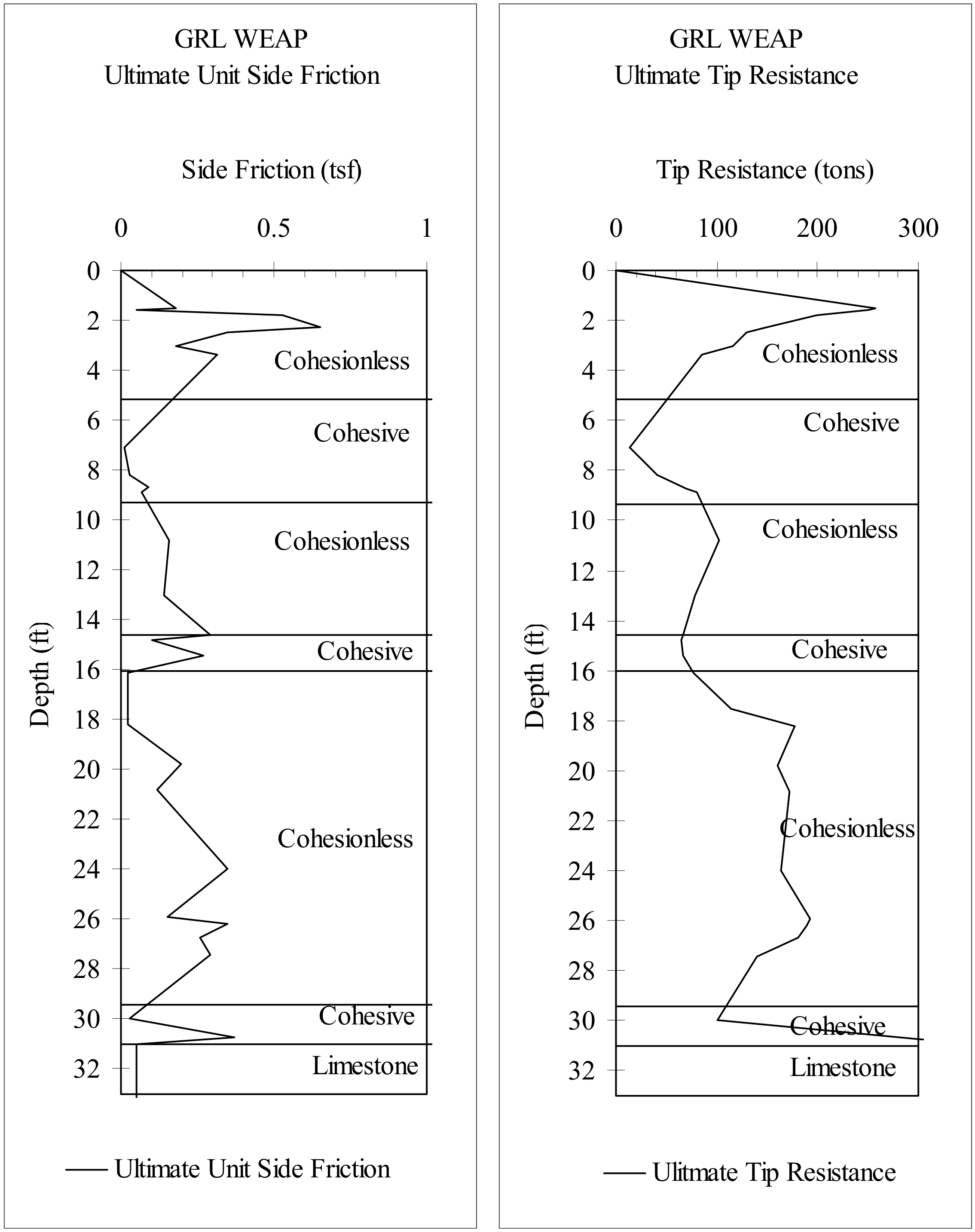



Figure 3-2 Side friction and tip resistance on a 30 inch pile at the test site, used to describe the soil profile in GRLWEAP.

For this project, the University of Florida proposed the pile driving system and evaluated it based on the soil profile at the test site. The proposed pile driving system was simulated using GRLWEAP software. It was necessary to simulate the soil profile at the site, the spliced pile geometry, the pile cushion thickness, and different pile driving hammer types to estimate the pile capacity, and stresses during driving.

The spliced pile was modeled in GRLWEAP by inputting the cross section properties as a function of length, as shown below in Table 3-3.

Table 3-3 Spliced pile model used in GRLWEAP software.

| Distance Below Top feet | Cross Sectional Area $in^2$ | Elastic Modulus ksi | Unit Weight $lb / ft^3$ |
|---|---|---|---|
| 0 | 900 | 4,000 | 150 |
| 4 | 900 | 4,000 | 150 |
| 4 | 646 | 4,170 | 150 |
| 10 | 646 | 4,170 | 150 |
| 10 | 891 | 4,680 | 150 |
| 30 | 891 | 4,680 | 150 |
| 30 | 646 | 4,170 | 150 |
| 36 | 646 | 4,170 | 150 |
| 36 | 900 | 4,000 | 150 |
| 40 | 900 | 4,000 | 150 |

The soil profile data of skin friction and end bearing on a 30 inch pile shown in Figure 3-2 was input. A drivability analysis was used to estimate the maximum stresses in the pile, the pile capacity, and the blow count log.

To choose the correct size hammer for the field site and pile size, the cushion thickness and fuel settings were adjusted for different Open End Diesel [OED] hammers. The optimal hammer would cause stresses in the pile equivalent to the allowable limits set by Section 455 of the FDOT Standard Specification for Road and Bridge Construction (FDOT 2004a).

### 3.3.3 Results of GRLWEAP Software

A Delmag D46-32 single-acting OED hammer was donated by Pile Equipment, Inc. of Green Cove Springs. The output shown below is for the Delmag D46-32 hammer with a 3 inch thick plywood pile cushion. The properties of the hammer are included in GRLWEAP and are summarized below. The hammer piston weighed approximately 5 tons, the operating weight of the hammer was 10 tons, and the pile cap weighed 7.5 tons. The hammer had four fuel settings which were all used during pile driving, with the majority being fuel settings 2 and 3. The energy per blow delivered to the pile ranged from 52.26 ft-kips to 122.14 ft-kips for a D46-32 hammer.

The output from GRLWEAP was provided at one foot depth increments as shown below in Table 3-4, which included the estimated ultimate pile capacity, side friction, end bearing, blow count, compressive stress, and tension stress. The stroke height was a function of pile resistance which would be useful to control tensile stresses in concrete piles during easy driving. The pile capacity increased near the rock layer. The compressive stresses were consistent until the rock layer at elevation -31 feet when they increased. The tension stresses were high, but could be controlled by using a lower fuel setting or increasing the plywood pile cushion thickness from 3 to 6 inches.

The maximum stresses in the pile were compared with the maximum allowable stresses specified in Section 455 of the FDOT Standard Specifications for Road and Bridge Construction. The estimated pile capacity was compared with the design capacity of 200 to 450 tons for a 30 inch pile. The estimated blow counts were compared with the recommended range of 20 to 120 blows per foot for a correct sized hammer (FDOT 2004a).

Table 3-4 GRLWEAP output for spliced pile with Delmag D46-32 OED hammer.

| Depth<br><br>feet | Ultimate Capacity kips | Side Friction kips | End Bearing kips | Blow Count Blows/ft | Compressive Stress ksi | Tensile Stress ksi |
|---|---|---|---|---|---|---|
| 1 | 345 | 1.2 | 344 | 17.1 | 2.55 | -0.56 |
| 2 | 366 | 6.3 | 360 | 18.6 | 2.56 | -0.52 |
| 3 | 245 | 14.6 | 231 | 10.6 | 2.47 | -0.74 |
| 4 | 167 | 20.1 | 147 | 6.2 | 2.40 | -0.87 |
| 5 | 133 | 24.6 | 108 | 4.6 | 2.33 | -0.90 |
| 6 | 97.0 | 27.5 | 69.5 | 3.2 | 2.27 | -0.93 |
| 7 | 59.6 | 28.7 | 30.9 | 2.3 | 2.15 | -0.90 |
| 8 | 101 | 29.1 | 71.6 | 3.3 | 2.28 | -0.93 |
| 9 | 193 | 30.2 | 162 | 7.7 | 2.43 | -0.84 |
| 10 | 218 | 32.1 | 186 | 9.1 | 2.45 | -0.80 |
| 11 | 235 | 35.0 | 200 | 10.2 | 2.47 | -0.77 |
| 12 | 217 | 38.1 | 178 | 9.1 | 2.45 | -0.80 |
| 13 | 198 | 41.0 | 157 | 8.1 | 2.43 | -0.83 |
| 14 | 186 | 44.7 | 141 | 7.4 | 2.42 | -0.86 |
| 15 | 179 | 49.1 | 129 | 7 | 2.41 | -0.87 |
| 16 | 202 | 52.8 | 149 | 8.4 | 2.44 | -0.83 |
| 17 | 254 | 53.3 | 201 | 11.5 | 2.49 | -0.74 |
| 18 | 373 | 53.8 | 319 | 19.6 | 2.56 | -0.53 |
| 19 | 393 | 55.0 | 338 | 21.2 | 2.58 | -0.50 |
| 20 | 383 | 58.2 | 325 | 20.4 | 2.57 | -0.52 |
| 21 | 404 | 61.1 | 343 | 22.1 | 2.59 | -0.49 |
| 22 | 402 | 64.5 | 337 | 22 | 2.59 | -0.49 |
| 23 | 401 | 69.3 | 332 | 22 | 2.59 | -0.50 |
| 24 | 402 | 75.6 | 326 | 22.1 | 2.59 | -0.50 |
| 25 | 438 | 81.6 | 357 | 25 | 2.61 | -0.44 |
| 26 | 468 | 85.5 | 383 | 27.5 | 2.63 | -0.40 |
| 27 | 417 | 91.3 | 326 | 23.5 | 2.61 | -0.48 |
| 28 | 358 | 96.6 | 262 | 18.9 | 2.59 | -0.59 |
| 29 | 331 | 100 | 231 | 17 | 2.58 | -0.64 |
| 30 | 302 | 102 | 200 | 15.2 | 2.55 | -0.70 |
| 31 | 906 | 106 | 800 | 66.6 | 2.74 | -0.22 |
| 32 | 907 | 107 | 800 | 66.9 | 2.74 | -0.22 |

## 3.4 FDOT Standard Specifications for Road and Bridge Construction

Section 455 of the FDOT Standard Specifications for Road and Bridge Construction (FDOT 2004a) provided requirements to properly install foundation structures including piling, drilled shafts and spread footings. This section was used as a guideline for determining the pile capacity and the maximum allowable stresses in prestressed concrete piles.

The maximum allowable stresses in the pile are a function of the specified minimum compressive strength of concrete, f`c, and the effective prestress, $f_{pe}$, on the cross section at the time of driving, taken as 0.8 times the initial prestress force, after all losses. The calculation of $f_{pe}$ for a 30 inch square prestressed concrete pile with twenty 0.6 inch diameter prestressing strands, at 41 kips each, is summarized below in Table 3-5.

Table 3-5 Variables for calculation of maximum allowable pile driving stresses.

| | | | |
|---|---|---|---|
| $f`_c$ | 6,000 | psi | Specified minimum compressive strength of concrete |
| $A_{conc}$ | 646 | $in^2$ | Cross sectional area of voided pile |
| $A_{strand}$ | 0.217 | $in^2$ | Area of 0.6 inch diameter strand |
| $f_{pu}$ | 270 | ksi | Ultimate prestress |
| $f_{pi} = f_{pu} * 0.70$ | 189 | ksi | Initial prestress, specified at 41 kips |
| $f_{eff} = 0.90 * f_{pi}$ | 170 | ksi | Effective prestress, assume 10% losses. |
| $F_{strand} = f_{eff} * A_{strand}$ | 37 | kips | Force per strand after losses. |
| $F_{total} = 20 * F_{strand}$ | 738 | kips | Total force on cross section |
| $f_{pe} = 0.8 * F_{total} / A_{conc}$ | 920 | psi | Effective prestress on the cross section for a continuous pile |
| $f_{pe} = 0$ | 0 | psi | Zero effective prestress at the splice. |

The equations provided in Section 455 of the FDOT Standard Specifications for Road and Bridge Construction (FDOT 2004a) in non SI units are provided below. The maximum allowable compressive stress was computed in equation (1), and the maximum allowable tensile stress was computed in equation (2).

$$S_{apc} = 0.7\ f`_c - 0.75\ f_{pe} \qquad \text{(psi)} \qquad \text{Eqn. (1)}$$

$$S_{apt} = 3.25\ (f`_c)^{0.5} + 1.05\ f_{pe} \qquad \text{(psi)} \qquad \text{Eqn. (2)}$$

For a continuous unspliced 30 inch square prestressed concrete pile, the prestressing strands contribute an effective prestress, $f_{pe}$, to the concrete of about 920 psi. This net compression in the section helps the concrete to survive the tensile stresses caused during pile driving. For a continuous unspliced 30 inch square prestressed concrete pile, the maximum allowable compressive stress is equal to 3,500 psi by equation (1), and the maximum allowable tensile stress is equal to 1,200 psi by equation (2).

For a spliced pile, the $f_{pe}$ is equal to zero because the prestress force is transferred to the concrete by bond. For 0.6 inch diameter strands with an effective prestress of 170 ksi, the transfer length is equal to 34 inches (ACI 318 2002). The concrete in this 34 inch zone adjacent to the mating surface is more likely to fail in tension than the fully prestressed portion of the pile during pile driving. For a spliced 30 inch square prestressed concrete pile with twenty 0.6 inch diameter strands the maximum allowable compressive stress in the non-prestressed region is 4,200 psi by equation (1), and the maximum allowable tensile stress is 250 psi by equation (2).

### 3.5 Summary of Analyses

The University of Florida CPT truck determined the depth of the limestone rock layer at the test site to be 31 feet below grade. The piles were driven 30 feet away from the CPT hole location. At the location the piles were driven, the rock depth increased to 36 feet adjacent to pile #1, and 39 feet adjacent to pile #2. Also, layers with high end bearing were located at depths of -15 feet, -23 feet, and -27 feet below grade, these were identified because they would likely generate tension stresses in the pile after the tip punched through the layer. The CPT data from the test site was used to calculate the unit side friction and unit end bearing on a 30 inch square prestressed concrete pile. GRLWEAP software was used to simulate pile driving at the test site with a Delmag D46-32 OED hammer and a soil profile model to estimate the pile capacity and stresses. The D46-32 was determined to be an adequate hammer for the piles and the soil profile.

For a spliced pile the effective prestress is zero in the splice region, thus, does not increase the allowable tensile stress in the pile. The maximum allowable tensile stress was 250 psi for a spliced 30 inch square prestressed concrete pile, and the maximum allowable compressive stress was 4,200 psi in the spliced region or 3,500 in the prestressed region of the pile (FDOT 2004a).

# CHAPTER 4
# CONSTRUCTION PROCESS AND FIELD TESTING METHOD

The construction process, heavy equipment and material details were determined during three project meetings at Wood Hopkins Construction in Jacksonville, FL. For example, the necessary equipment to drive the piles, the splice bracing and template design, the pile cutoff method, the steel pipe vertical support, the grout inlet port location, the foam rubber plug design, and the selection of the annulus cementitious grout were discussed. The project construction schedule was also discussed at the meeting.

## 4.1 Pile Support and Spliced Pile Bracing Method

A steel template was used to support the piles while the crane lifted the pile driving hammer. After splicing, the template was used to secure the top pile section without moving while the grout cured. The contractor’s means and methods were used to support the top half of the splice while the grout cured; the template method effectively braced the splice to prevent movement.

### 4.1.1 Steel Template used to brace Spliced Piles

The template was constructed by driving four steel H-piles as the foundation which extended up to approximately 15 feet as columns. Two steel beams spanned between the columns as the primary frame, and the template rested on the steel beams as shown in Figure 4-1. The template was raised and lowered by changing the welds and bolted connections to the columns. The two openings in the template were approximately 2 inches larger than the pile width and approximately 10 feet apart.

The template was initially set at 9 feet above grade. Each pile was lifted using a double choker with a load stabilizer plate so that it would hang vertically. After being lowered into the opening in the template, wooden wedges were used to secure the pile from moving as shown in Figure 4-1.

A

B

Figure 4-1 Splice testing preparation. A) Template, piles and HSS pipes, B) the piles in the template.

The template supported both piles while the crane lifted the pile driving assembly. After driving the piles to a tip elevation of -14 feet, the template was lowered to the ground. The template had to be lowered to the ground so that it would not interfere with removal of the top half of the pile. The piles were cut in half, at the 20 foot mark, with 14 feet below the ground and 6 feet remaining above ground as shown in Figure 4-3.

Before assembling the pile splice, the template was raised up to its maximum height to support the top half of the spliced pile while the grout was pumped into the annulus and given overnight to cure. After the grout cured, before the pile was driven the template was lifted off the top so it would not interfere with the leads.

### 4.1.2 Steel Channels used to brace Spliced Piles

In the Alternatives for Precast Pile Splices report (Britt, Cook and McVay 2003) a support method was developed using four C15 x 33.9 sections to brace the top half of the splice while the grout cured. The channels would squeeze the pile from four sides using threaded rods. The channels would also force the two halves of the pile into alignment. The attachment method assumed 10 feet of the driven pile was above ground after the head of the pile was cutoff. For this situation, the channels would be bolted on by drilling through the pile above and below the 20 foot long section to insert threaded rods to bolt the channels to the pile as shown in Figure 4-2. The bottom threaded rods would also be used to support the steel pipe in position. If less than 10 feet of the lower half was above ground, the contractor’s means and methods would be used to attach the channels to the bottom pile. For example, additional steel sections, bars, or threaded rods would be bolted together to brace the pile sections from moving.

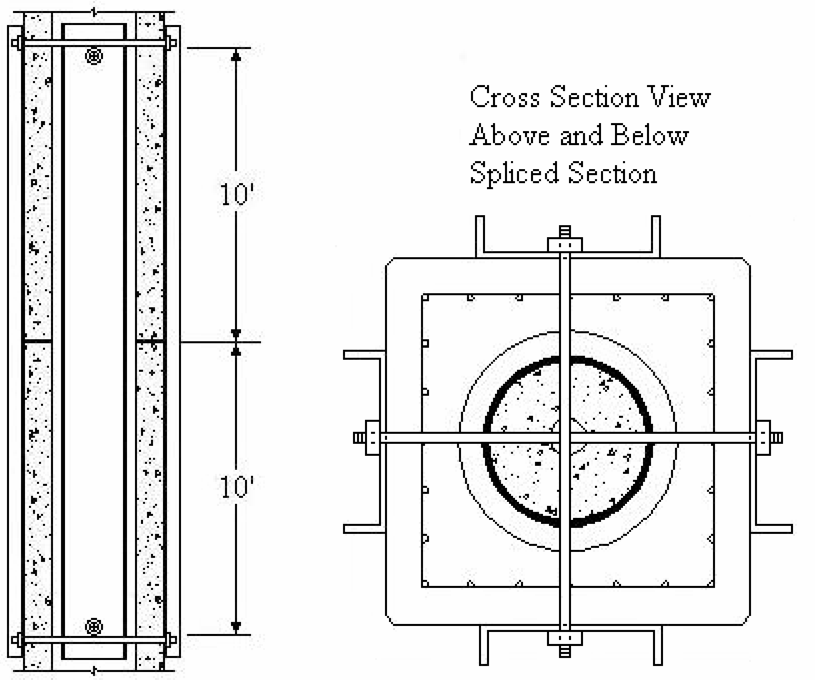



Figure 4-2 Steel C channels to support spliced pile section.

## 4.2 Initial Pile Drive to Cutoff Elevation

The East side pile, or Pile #1, was driven to a tip elevation of -14 feet. The pile began to gain resistance at -10 feet. Before that depth, the blow counts were very low, less than approximately 5 blows per foot. The pile driving blow count record for each pile is shown below in Table 4-1. The final blow count at a tip elevation of -14 feet was 18 blows per foot.

The West side pile, or Pile #2, was driven with similarly low blow counts with an increase in capacity and blow count at a tip elevation of -10 feet. The final blow count at a tip elevation of -14 feet was 19 blows per foot. Pile driving stopped for the day after both piles were driven to a tip elevation of -14 feet.

The CPT test performed 15 feet east of pile #1 showed a local maximum tip resistance at a depth of -15.7 feet. The CPT test performed 20 feet west of pile #2 showed a local maximum tip resistance at a depth of -14.4 feet. Pile driving was stopped before the tip punched through the stiff layer for either pile. Based on the CPT test, after 24 hours of wait time, the spliced piles would begin with an increased capacity and moderate compressive stresses due to the soil setup after the pore water pressure dissipated, and after punching through the stiff layer, the tip resistance would decrease which could cause high tensile stresses for a spliced prestressed concrete pile.

Table 4-1 Blow Count Log for initial pile drive to cutoff elevation.

| Tip Elevation (ft) | Pile #1 | | Pile #2 | |
|---|---|---|---|---|
| | Blow Count ( Blows/ft ) | Total # of Blows | Blow Count (Blows/ft ) | Total # of Blows |
| -1 | 0 | 0 | 0 | 0 |
| -2 | 0 | 0 | 0 | 0 |
| -3 | 9 | 9 | 1 | 1 |
| -4 | 4 | 13 | 2 | 3 |
| -5 | 2 | 15 | 2 | 5 |
| -6 | 3 | 18 | 2 | 7 |
| -7 | 2 | 20 | 2 | 9 |
| -8 | 5 | 25 | 2 | 11 |
| -9 | 2 | 27 | 2 | 13 |
| -10 | 2 | 29 | 4 | 17 |
| -11 | 2 | 31 | 6 | 23 |
| -12 | 4 | 35 | 6 | 29 |
| -13 | 9 | 44 | 7 | 36 |
| -14 | 18 | 62 | 19 | 55 |

### 4.3 Top Half of Piles Cutoff

Both of the piles were cut in half at a tip elevation of -14 feet, thus 6 feet of pile remained above ground surface. An air powered concrete saw with a 14 inch diameter diamond blade was used to cut through the prestressing strands as shown in Figure 4-3. After all of the strands were cut, the crane was used to pull the pile slowly to the side, until it broke. The metal liner was cut with an oxy-acetylene torch to release it from the lower half of the pile as shown in Figure 4-3.


A B

Figure 4-3 Pile cutoff to expose void. A) Concrete pile is cut with diamond blade circular saw; B) metal liner of pile void is cut with an oxyacetylene torch.

In both piles the metal liner only extended 54 inches below the cutoff elevation as shown in Figure 2-3. The cardboard sonotube was spliced to the corrugated metal to form the void below 54 inches.

To test the 10 foot splice bond length in each half of the pile, the cardboard was removed so the annulus grout could bond to the bare concrete inside of the pile to transfer the load. The void in both piles was filled with water to soften the cardboard so that it could be more easily removed the next morning. The water was pumped out using a submersible pump. Figure 4-4 is the inside of the pile after the cardboard sonotube was removed to expose the bare concrete.

Figure 4-4 Void in each pile after removing cardboard sonotube below 54 inches.

For this test, the pile was cut in half and the top half was reattached using the steel pipe splice. In typical field splice conditions, only the top 5 feet of pile would need to be removed to expose the 18 inch diameter void.

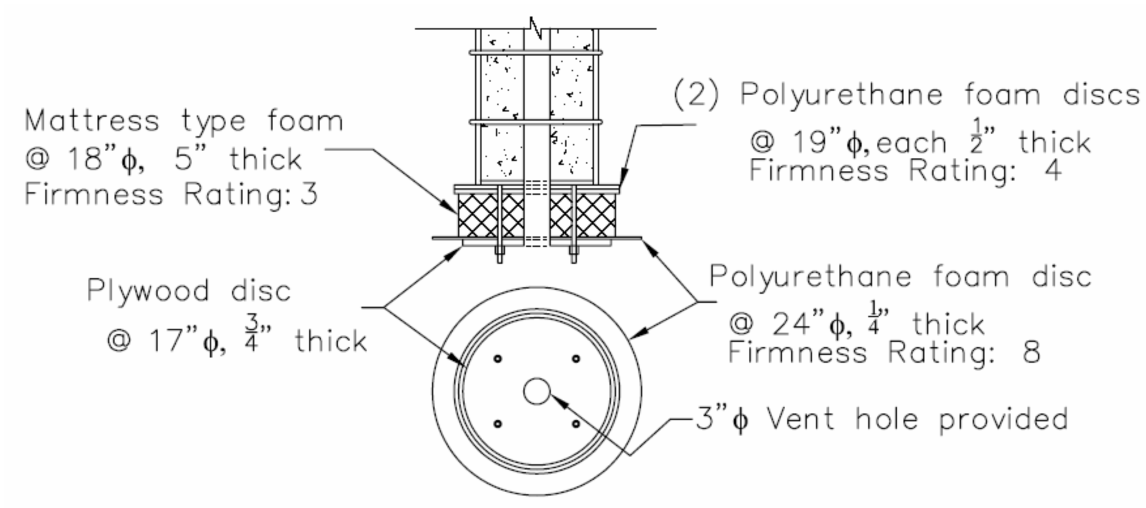
### 4.4 Assembly of the Steel Pipe Splice

To support the steel pipe vertically inside the pile, two 1 inch diameter steel bolts were used. The steel pipe was lowered into the void, and the pile was drilled to receive two 1 inch diameter bolts approximately 12 inches below the mating surface near the centerline of two sides of the pile as shown below in Figure 4-5. The steel pipe was marked so that holes could be cut in the steel pipe in-line with the holes in the pile. The holes were cut in the steel pipe with an oxyacetylene torch and the concrete inside was drilled 4 inches deep. A hole was also drilled in the side of the pile to receive the grout inlet port.

Figure 4-5 Holes drilled to receive bolts to support the steel pipe.

The foam rubber plug was attached to the bottom end of the steel pipe to prevent the annulus grout from leaking out of the spliced section as shown in Figure 4-6. Four 5/8 inch diameter threaded rods were welded to the bottom end plate when the steel pipe was hanging from the crane. The plywood on the bottom layer and the foam rubber plug were drilled to fit the threaded rods. The plywood was used to compress the 5 inch thick piece of polyurethane mattress type foam. The other layers of the plug were Poron Quick Recovery Polyurethane Foam. In the center of the plug a 3 inch diameter hole was cut to allow gasses to escape. The contractor’s means and methods may be used to prevent the annulus grout from leaking out of the spliced section and filling the void of the driven pile below.

The steel pipe was slowly lowered into the void of the pile to avoid damage to the foam rubber plug. After positioning the steel pipe, the steel bolts were greased, so the annulus grout would not bond to them, and were inserted into the predrilled holes as shown in Figure 4-7.




A

B

C

Figure 4-6 Details of the grout plug. A) The dimensions of the grout plug, B) the grout plug is bolted on and compressed with a plywood disc, C) plug in the pile void.

### 4.5 Mating Surface Grouted and Annulus Grout Pumped

The mating surface was clean and ready for the grout. The spliced section of pile was lowered down to observe the gap and to align the template. The spliced section was one foot above the mating surface as shown in Figure 4-7.

Figure 4-7 Steel bolts greased and inserted to support HSS pipe, annulus grout globe valve was attached with epoxy, and mating surface grout was applied.

For Pile #1, Set 45 Hot Weather was used instead of regular Set 45, because it was a very hot day and the grout sets in a shorter amount of time in warmer weather. For Pile #2 regular Set 45 was used. For both piles the grout was mixed with the minimum recommended water content and the required volume was applied to the mating surface to fill the gap. The spliced pile section was lowered into contact, wooden wedges were at the template to secure the spliced section of pile plumb, and the pile choker cable was slackened so that it would not disturb the bond of the grout at the splice interface. After about 15 minutes the grout had setup, and the grout had cured after 45 minutes.

After the mating surface grout cured for about 45 minutes mixing began for the Masterflow 928 grout. The mixer had an agitator holding tank so the grout could be pre-mixed and continuously pumped to fill the void. Grout pumping began at 4:00 pm and ended at 4:30 pm. The FDOT State Materials Office personnel were present to measure the fluidity of the grout by recording the flow cone time with the cone type specified in ASTM C939. A flow time of 25 – 30 seconds was specified for a fluid grout consistency. The flow cone time was measured after mixing the first batch of grout, to verify the consistency was fluid. At the end of grout pumping the flow cone times were 30 seconds and 35 seconds for Pile #1, and Pile #2, respectively, as shown in Figure 2-5. A globe valve was used for the top vent for Pile #1 to have a second inlet location ready, if the lower valve became clogged. The vent hole at the top of the splice section was used to monitor the grout level as shown in Figure 4-8.

Figure 4-8 Vent hole active and wooden wedges bracing the spliced pile section.

## 4.6 Driving of Spliced Piles

Spliced Pile #1 was spliced and driven first, and then Pile #2 was spliced and driven. The top set of instruments were 6 feet below the top of the pile, so driving stopped for both piles when the instruments were at ground elevation.

### 4.6.1 Spliced Pile #1 Driven after Grout Cured 24 hours

Driving of spliced Pile #1 resumed 24 hours after the grout was finished pumping, when the grout cube compressive strength was measured at 4500 psi as shown in Figure 2-5. Approximately three-hundred-and-ninety-four hammer impacts were recorded to penetrate the pile from a tip elevation of -14 feet to -34 feet as shown in Table 4-2. For Pile #1, the highest blow count recorded was 56 blows per foot at a tip elevation of -16 feet. Based on the CPT performed at the site, the tip was above a stiff layer. Below a tip elevation of -17 feet, the blow counts averaged 18 blows per foot. The hard layer was not encountered at the predicted depth of -31 feet, and the top sets of gages were at the ground surface, so driving was stopped for the day.

### 4.6.2 Spliced Pile #2 Driven after Grout Cured 20 hours

Driving spliced Pile #2 resumed 20 hours after the grout was finished pumping, when the grout cube compressive strength was measured at 3800 psi as shown in Figure 2-5. Approximately four-hundred-and-three hammer impacts were recorded to penetrate the pile from a tip elevation of -14 feet to -34 feet as shown in Table 4-2. Pile #2 punched through a stiff layer at a tip elevation of -17 feet with the maximum recorded blow count of 40 blows per foot. The pile was driven until the top sets of gages were at ground elevation and would be damaged by continued driving. The rock layer was not penetrated with Pile #2 because the depth of the rock layer was approximately 39 feet below grade.

Table 4-2 Blow Count Log for Driving Spliced Piles #1 and #2

| Tip Elevation (ft) | Pile #1 | | Pile #2 | |
|---|---|---|---|---|
| | Blow Count ( Blows/ft) | Total # of Blows | Blow Count (Blows/ft) | Total # of Blows |
| -15 | 26 | 26 | 17 | 17 |
| -16 | 56 | 82 | 18 | 35 |
| -17 | 6 | 88 | 40 | 75 |
| -18 | 13 | 101 | 15 | 90 |
| -19 | 22 | 123 | 11 | 101 |
| -20 | 11 | 134 | 16 | 117 |
| -21 | 25 | 159 | 28 | 145 |
| -22 | 23 | 182 | 14 | 159 |
| -23 | 23 | 205 | 7 | 166 |
| -24 | 24 | 229 | 8 | 174 |
| -25 | 13 | 242 | 32 | 206 |
| -26 | 11 | 253 | 8 | 214 |
| -27 | 21 | 274 | 18 | 232 |
| -28 | 23 | 297 | 22 | 254 |
| -29 | 23 | 320 | 23 | 277 |
| -30 | 18 | 338 | 21 | 298 |
| -31 | 25 | 363 | 22 | 320 |
| -32 | 18 | 381 | 23 | 343 |
| -33 | 13 | 394 | 21 | 364 |
| -34 | - | - | 19 | 383 |

### 4.6.3 Spliced Pile #1 Re-Driven after 4 days

The CPT test performed 15 feet east of Pile #1 showed that a hard layer, possibly limestone rock was 36 feet below grade. To drive Pile #1 into rock, the top 5 feet of soil was excavated adjacent to the pile so the gages would not be damaged by soil and water. Pile #1 was driven to a tip elevation of -39 feet with a maximum blow count of 35 blows per foot as shown in Table 4-3.

Table 4-3 Blow count log for continued driving of spliced Pile #1

| Tip Elevation (ft) | Pile #1 | |
|---|---|---|
| | Blow Count ( Blows / f t ) | Total # of Blows |
| -34 | 26 | 26 |
| -35 | 34 | 60 |
| -36 | 29 | 89 |
| -37 | 29 | 118 |
| -38 | 30 | 148 |
| -39 | 35 | 183 |

### 4.7 Summary of Splice Construction Process

The detailed summary of the splice construction process is outlined in the order the steps would be performed to construct the splice.

1. Prepare Steel Pipe
   - The HSS pipe was deformed with ½ inch diameter dowel bars at eight inch spacing with 2 inches of 3/16 fillet weld per foot of bar.
   - The HSS pipe was filled with concrete a three inch diameter vent pipe, a plate with a 3 inch diameter center hole was welded to the bottom to accomplish this.
2. Cutoff Pile and Prepare void
   - The pile was cutoff in the hollow section, below the solid driving head to expose the 18 inch diameter void.
   - The corrugated metal liner was cut near the top using an oxyacetylene torch, as the crane slowly broke off the solid driving head.
   - The metal liner was hammered down out of the way, so that the foam rubber plug would not catch the edges when inserted into the void.
3. Drill Holes in Pile
   - Holes were drilled through two opposite sides of the pile approximately 12 inches below the top of the cutoff driven pile to receive 1 inch diameter steel bolts. The HSS pipe was temporarily lowered into the void (with out the foam rubber plug attached), so the hole locations would be marked.
   - A hole for pumping in grout was drilled 8 inches below the top of the cutoff driven pile. Epoxy was used to attach an inlet port compatible with the grout pump hose.
   - A vent hole was drilled in the top pile section, 10 feet above the splice interface to let air escape during pumping of the grout, and to monitor the grout level.

4. Cut Holes in HSS Pipe
    - Holes were cut in the HSS pipe on two sides with a cutting torch at the location marked during drilling in step 6

    - The concrete was drilled 4 inches deep to accept the dowels at the correct angle, based on the holes in the sides of the pile from step 6.

5. Setup Splice Bracing Channels or Template
    - Setup and assemble bracing for the top half of the splice. A template or steel channel system or equivalent must be used to support the pile overnight. The crane choker cable must be loose or removed from the pile while the grout at the mating surface hardens.

6. Attach Foam Rubber Plug
    - Attach the foam rubber plug or equivalent to the end of the HSS pipe. The grout plug shall seal a 2 inch wide gap in the annulus of the splice. An equivalent method may be used to prohibit the grout from filling the pile past the end of the splice. A plastic grout could be placed at the bottom of the splice to seal a poorly designed plug.

7. Insert HSS Pipe into Driven Pile Void
    - Slowly lower the HSS pipe with the foam rubber plug attached into the void of the pile.

    - The two steel bolts are greased and inserted through the holes in the side of the pile and into the holes drilled into the HSS pipe to support it vertically.

8. Attach Mating Surface Formwork
    - A plywood form should be attached around the splice interface so that the mortar completely fills the gap at the interface between the piles. Concrete shims may be used at the mating surface in the gap, but definitely not metal shims.

9. Place Spliced Pile Section
    - The top pile will be lifted into position and dry fit to observe the gap at the splice interface. This helps to identify the size of the necessary formwork at the splice interface. Also, the channel support or template can be adjusted plumb.

10. Mix and Place Mating Surface Mortar
    - With the top pile in position and approximately a one foot gap between the piles, place the mortar, Set 45 or equivalent, to the top of the bottom pile in a 1 to 2 inch thick layer, depending on the gap at the splice interface. The mating surface should be prepared for mortar in accordance the manufacturers recommendations.

11. Release Choker Cable from Spliced Pile Section
    - The top pile shall be checked that it is stable and then shall be released from the crane to prevent disturbing the bond with movement. The bonding material is given time to cure, approximately 45 minutes, so the fluid grout does not leak out at the interface.

12. Mix and Pump Annulus Grout
    - The grout is mixed and the flow cone time is measured to compare with the flow cone time for a fluid consistency. The grout mix should be adjusted to the proper flow cone time.

    - The grout is pumped into the inlet port below the splice interface. Grout shall be placed in a continuous flow. Pumping continues until the grout starts to flow out of the upper vent hole.

    - Cast grout cubes during grout pumping.

13. Test Grout Cube Strength
    - Pile driving may continue once the grout cube strength has reached 3800 psi.

# CHAPTER 5
# COLLECTION AND ANALYSIS OF PILE DRIVING DATA

This chapter discusses the dynamic load testing methods used to determine the maximum stresses in the pile during driving. A Pile Driving Analyzer [PDA] unit was used with strain transducer and accelerometer instruments attached to the top of each pile. A general discussion of the collection of PDA data and the meaning of the output is discussed.

## 5.1 Data Collection with a Pile Driving Analyzer

It is standard practice to monitor spliced prestressed concrete piles during driving so they are not damaged by high stresses. The standard monitoring equipment consists of a PDA unit model PAK, which is a laptop computer that accepts inputs from the strain transducer and accelerometer sensors. For each impact of the hammer to the pile, the sensors acquire acceleration and strain signals at a sampling rate of 0.076 milliseconds and send the signals to the PDA unit. The PDA unit conditions, digitizes, displays, stores, and performs automatic calculations on the input signals based on the pile properties input by the user. For example, the average strain is converted to an equivalent force through the elastic modulus and the cross sectional area, and the acceleration is time integrated to velocity.

Both strain transducers and accelerometers were attached to the top of the pile, the same distance from the top, to be able to separate the waves traveling down from the waves traveling up the pile. The total force and velocity are measured at the top of the pile. The total force at any location in the pile is the sum of the upward and downward traveling waves. The pile impedance, Z, defined in equation (1) is a property of the pile. The particle velocity multiplied by the pile impedance has units of force. The force due to a downward traveling wave is defined in equation (2). The force due to an upward traveling wave is defined in equation (3). The total force is equal to the sum of the upward and downward traveling waves, equation (4). The sign convention used for force was positive for compression and negative for tension. The sign convention for particle velocity was positive for downward and negative for upward particle velocities.

Equations used to separate the upward and downward traveling waves in piles:

| | |
|---|---|
| $Z = \dfrac{EM \cdot AR}{WC}$ | Eqn. (1) |
| $F_{down} = Z \cdot V_{down}$ | Eqn. (2) |
| $F_{down} = -Z \cdot V_{up}$ | Eqn. (3) |
| $F_{total} = F_{down} + F_{up}$ | Eqn. (4) |

The net force was measured by the strain transducers at the top of the pile as shown in Figure 5-1 for blow number [BN] 227 of 383 for Pile #2. The wave down and wave up are automatically calculated by the PDA unit using the velocity at the gage location. The wave up and wave down are used to calculate the maximum compressive and tensile stress in the pile. The large magnitude of the tensile wave up caused the maximum tensile stress in the pile.

Figure 5-2 is a second example of the force at the top gage versus time for BN 116 of 183, when the maximum compressive stress was recorded. The net force at the top gages was greater than the magnitude of the wave down because the wave up was also initially compressive, which was caused by high end bearing at the tip of the pile.

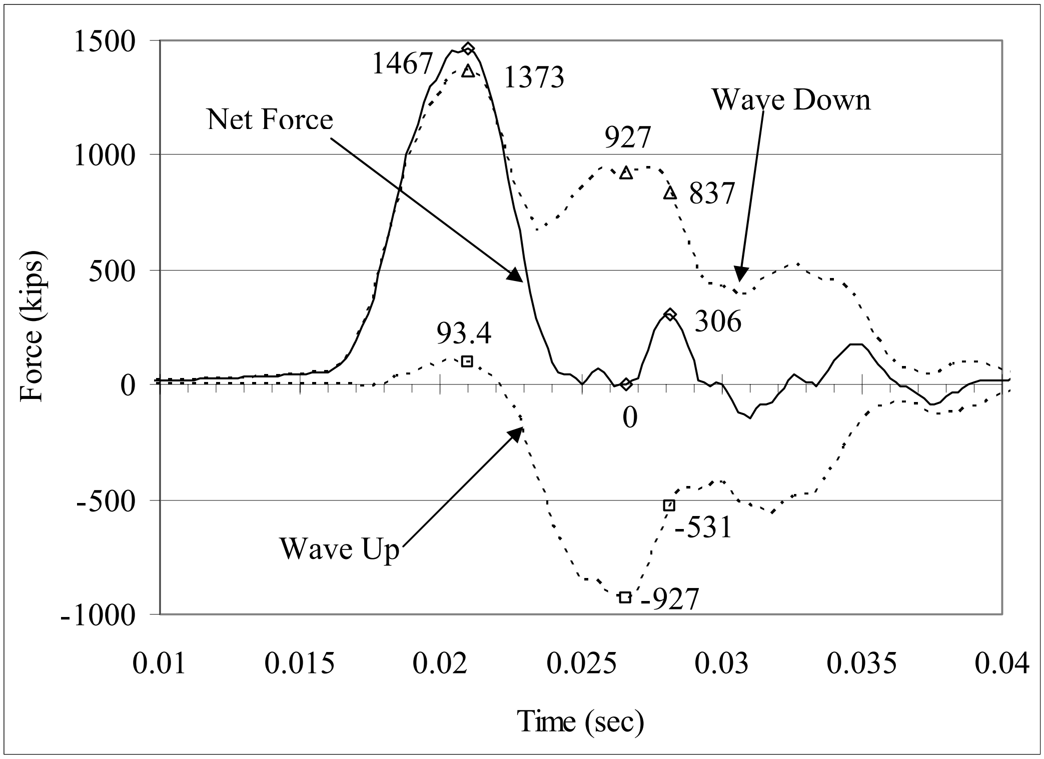



Figure 5-1 Force at the top instruments, Pile #2 BN 227 of 383, high tensile stresses.

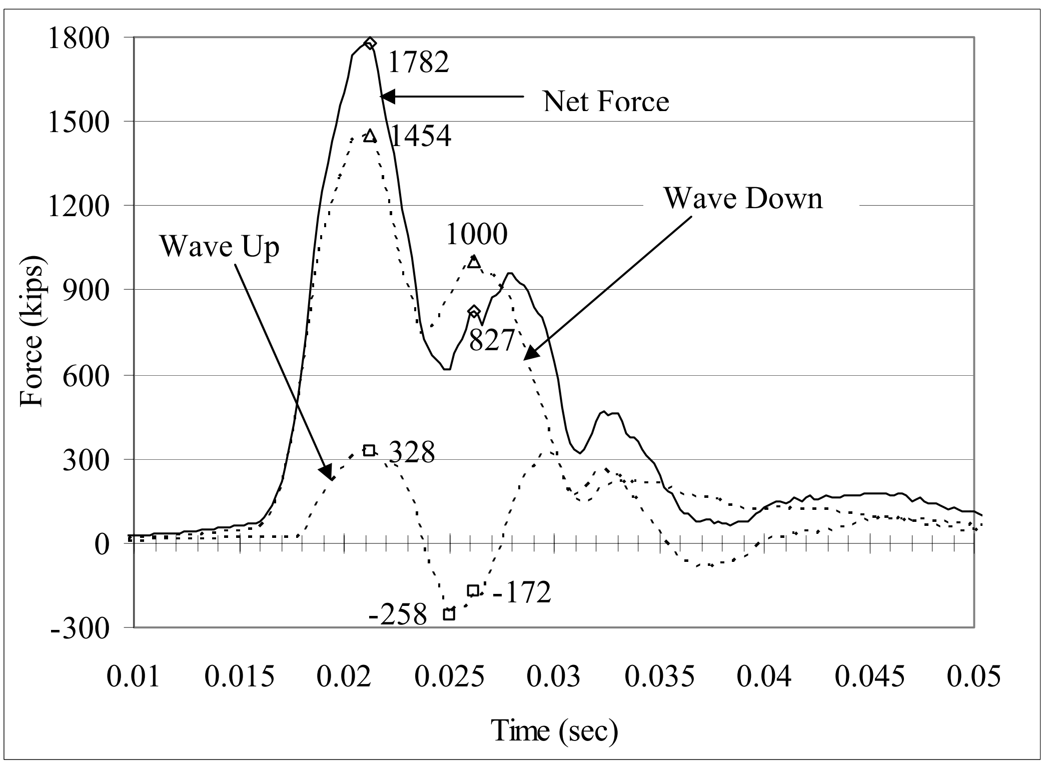



Figure 5-2 Force at the top instruments, Pile #1, BN 116 of 183, high compressive stress.

## 5.2 PDA Input Information

The properties of the 30 inch square prestressed concrete piles were input to the PDA unit, as shown in Table 5-1. The effective length of pile, LE, was the distance from the top gages to the tip of the pile as shown in Figure 5-3. In the PDA unit, the cross section of the pile must be constant over the effective length. The top set of instruments were attached to the pile in the hollow voided section, therefore, the cross sectional area, AR, of the voided pile was used. The elastic modulus, EM, and specific weight, SP, of the pile were input and the wave speed, WS, was calculated as the square root of the elastic modulus divided by the mass density, ρ, of the pile, as shown below in Table 5-1.

Table 5-1 Pile input information used in PDA unit.

| **Input** | **Description of Input** | **Value** | **Units** |
|---|---|---|---|
| LE | Length of Pile Below Gages | 34 | feet |
| EM | Elastic Modulus of Pile | 5,506 | ksi |
| AR | Cross Sectional Area of Pile | 646 | $in^2$ |
| SP | Specific Weight of Pile | 0.151 | $kips/feet^3$ |
| $WS = \sqrt{\frac{EM}{\rho}}$ | Wave Speed Input | 13,000 | feet/sec |
| $WC = \frac{2 \cdot L}{\Delta t}$ | Wave Speed Calculated | 13,080 | feet/sec |

For verification, the wave speed, WC, is automatically calculated by recording the time for the wave to travel down the pile and back up to the instruments. The wave speed, WC, is calculated as twice the effective pile length divided by the time between peak values. During initial hammer impacts, the elastic modulus, EM, of the pile was adjusted so that the wave speed input, WS, would match the wave speed calculated, WC.

The PDA unit accounted for the increase in stiffness of the spliced pile by requiring an increased modulus of elasticity to match the wave speed, WC, in the pile.

For comparison, the static elastic modulus of the pile was computed by AASHTO Section 5.4.2.4 (AASHTO 2004b). The minimum specified unconfined compressive strength, f`c, of the piles was 6000 psi (FDOT 2005). The unit weight of the pile was input to the PDA unit was 151 lb/ft$^3$ to account for the steel pipe splice. The minimum static modulus of the pile was 4,740 ksi, and the modulus used in the PDA unit was 125% greater than the minimum elastic modulus for normal weight concrete. This may be due to a higher value of f`c, or the increased stiffness of the spliced pile with the steel pipe cross section. The elastic modulus of the pile was also calculated for higher strength concrete as shown in Table 5-2.

Table 5-2 AASHTO Elastic Modulus Equations for a range of *f`c* values.

| *f`c* | Unit weight = 151 lb/ft$^3$ |
|---|---|
| | $E = w_c^{1.5} \cdot 33\sqrt{f`c}$ |
| psi | ksi |
| 6,000 | 4,740 |
| 7,000 | 5,120 |
| 8,000 | 5,480 |
| 9,000 | 5,810 |

### 5.3 PDA Instrumentation Attachment Locations

One PDA model PAK unit can accept inputs from eight instruments. Each pile was monitored using four strain transducers and four accelerometers. A set of instruments included two strain transducers and two accelerometers. At the top of the pile a pair of instruments was attached on each of two opposite sides of the pile, exactly 6 feet below the head of the pile. A strain transducer and an accelerometer are attached side by side, 1.5 inches from the centerline, and reversed left and right on the opposite side of the pile as shown in Figure 5-3. The instruments are attached in this manner so that the average strain and acceleration may be used. The top set of instruments was the minimum required for dynamic pile testing. For this project an additional set of instruments was attached to each pile 27 feet below the top sets of gages, or 7 feet above the toe of the pile in the voided section. The purpose of this lower set of instrumentation was to measure the axial strain below the splice section. The measured strain would be plotted versus time and compared with the computed force at the same pile segment as discussed in Section 5.6. The top set of instruments was attached on the face of the pile with the lower strain transducer, not the lower accelerometer as shown in Figure 5-3, so that strain gage measurements would be on the same side of the pile.

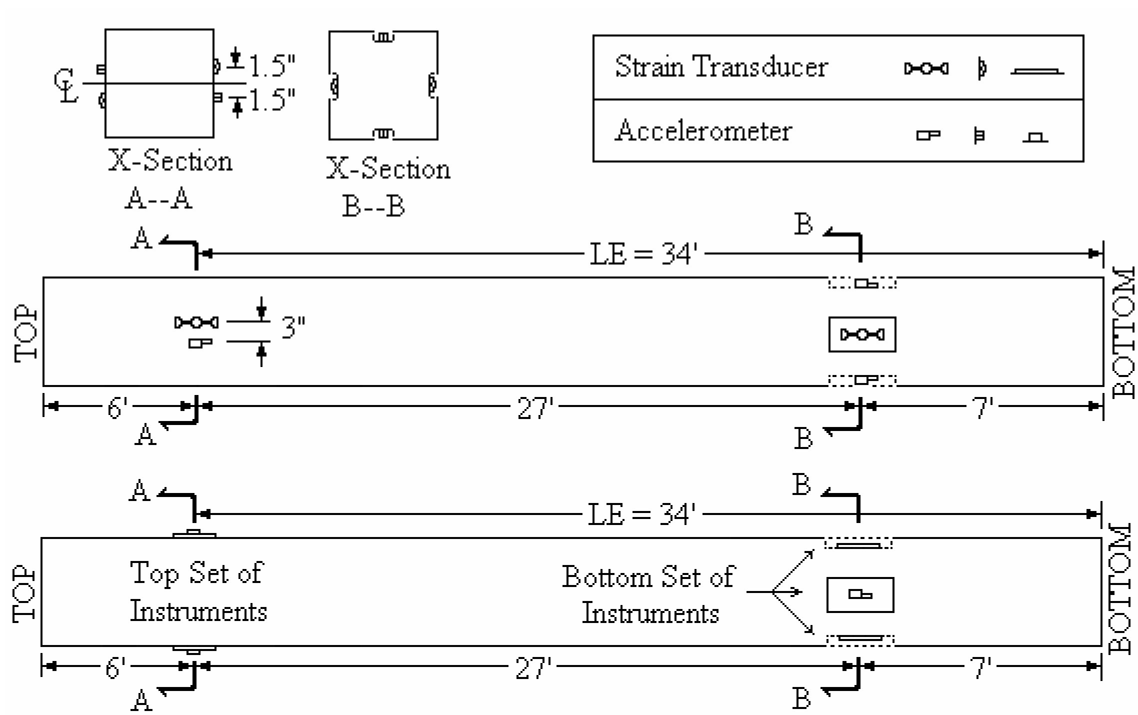



Figure 5-3 PDA instrumentation attached at the top and bottom of the piles.

The lower set of instruments was to be driven 30 feet below grade and had to be sealed and covered to be protected from damage by soil and water. The piles were cast with indentions on the centerline of each side of the pile. The indentions were 3 inches by 6 inches and 1.5 inches deep, to allow clearance for one instrument per indention.

Each instrument was covered with a thick layer of silicone window caulk after being plugged into the PDA unit for a verification of signal. A 1/16 inch thick steel plate was bolted on using six ¼ inch diameter bolts threaded into concrete sleeve anchors. A bead of silicone caulk was applied near the edges of the plate so that it would seal when the plate was tightened down. The bottom set of instruments were sacrificed for the project because they went below ground and would not be recovered. A groove was cut along the centerline of each side of each pile to mount the instrumentation wire. The groove was cut ½ inch deep by ¼ inch wide to allow a 3/16 inch diameter wire to fit below the surface. Hilti HY 150 adhesive was used to glue the wire into the groove. Several figures of the instrumentation are provided in Appendix B.

## 5.4 PDA Unit Output

The PDA unit has the capability to output every variable versus depth or BN. The maximum forces, stresses and pile capacity are summarized below. Additional PDA unit output is presented using PDIPLOT software in Appendix C.

### 5.4.1 Maximum Stress in the Pile from PDA Output

The PDA unit calculated the stress in the pile with a cross sectional area of the hollow section, AR, and an adjusted elastic modulus, EM, to account for the increased stiffness due to the 20 foot long solid section as shown in Table 5-1. For each hammer impact the maximum and minimum net force in the pile was computed. The stress computed by the PDA unit was the force divided by the voided cross sectional area, AR. The PDA unit does not show the force distribution in the pile, only the maximum and minimum are provided, and their location is unknown.

The maximum compressive stress typically occurred when the pile had a high end bearing, for example when the tip of the pile was above a hard soil layer. The maximum tensile stress typically occurred after the pile tip punched through the hard soil layer.

The tensile stresses ranged from zero to 0.39 ksi during driving of spliced Pile #2. For example, hammer impacts or blow numbers [BN] 119 and 227 of 383 had tension stresses of 0.37 and 0.39 ksi, respectively. The hammer impacts with maximum tensile or compressive stresses typically occurred during successive BN. For example, in Pile #2 after splicing the pile at a tip elevation of -14 feet, the tip was above a stiff layer. Table 5-3 below summarizes the PDA output information for BN 14 – 21 when the pile tip was at elevation -15 feet. The PDA estimated the maximum pile capacity to be 180 kips during driving for the BN summarized in Table 5-3.

Table 5-3 High tensile stresses for pile #2, PDA output calculated with voided cross sectional area of 646 in$^2$.

| BN | Max Compressive Force kips | Max Compressive Stress ksi | Max Tensile Force kips | Max Tensile Stress ksi |
|---|---|---|---|---|
| 14 | 1264 | 1.96 | -151 | 0.23 |
| 15 | 1296 | 2.01 | -178 | 0.28 |
| 16 | 1300 | 2.01 | -200 | 0.31 |
| 17 | 1364 | 2.11 | -252 | 0.39 |
| 18 | 1351 | 2.09 | -254 | 0.39 |
| 19 | 1319 | 2.04 | -217 | 0.34 |
| 20 | 1291 | 2.00 | -206 | 0.32 |
| 21 | 1254 | 1.94 | -128 | 0.20 |

The tensile stresses were compared with the maximum allowable tensile stress of 252 psi, for a spliced prestressed concrete pile computed in Section 3.4. The stresses recorded for BN 15 – 20 were greater than the allowable tensile stress of 252 psi for a spliced pile. The maximum allowable tensile stress was exceeded purposefully to test the splice design. The allowable tensile stress was exceeded when the splice mating surface was above ground, yet no degradation of the spliced pile was observed.

The middle 20 feet of the pile had a spliced cross sectional area of 891 $in^2$, not 646 $in^2$, which was used in the calculation of the maximum stress. Thus, if the maximum tensile force for each BN in Table 5-3 was divided by the cross sectional area of the solid pile, then the maximum tensile stress would be less than the value automatically calculated by the PDA unit. The effect of the change in cross sectional area is discussed further in Section 5.5.

The maximum compressive stress in the spliced piles ranged from 1.2 to 2.8 ksi during pile driving. The maximum compressive stress recorded during driving of spliced Pile #2 was 2.4 ksi at a tip elevation of -26 feet for BN 226 of 383. The tip of spliced Pile #2 did not reach the rock layer because the depth of rock was greater than anticipated.

Spliced Pile #1 was driven to a tip elevation of -39 feet, and pile driving was stopped to prevent damage of the top set of instruments from soil and water. The maximum compressive stress recorded during driving of spliced Pile #1 was 2.8 ksi at a tip elevation of -36 feet on BN 116 of 183. The successive blows near BN 116 also had high compressive stresses, and low tension stresses. Table 5-4 summarizes the PDA output information for BN 113 – 121.

The maximum compressive stress from the PDA output was less than the maximum allowable compressive stress of 3.4 for a continuous pile or 4.2 ksi for a spliced pile computed in Section 3.4. However, concrete has a lower stress limit for tension than compression, so even though the maximum compressive stress was not exceeded, the pile splice should be able to carry a higher compressive force than measured by the PDA.

Table 5-4 High compressive stresses for pile #1, PDA output calculated with the voided cross sectional area of 646 $in^2$.

| BN | Max Compressive Force kips | Max Compressive Stress ksi | Max Tensile Force kips | Max Tensile Stress ksi |
|---|---|---|---|---|
| 113 | 1572 | 2.43 | 0 | 0 |
| 114 | 1682 | 2.6 | 0 | 0.03 |
| 115 | 1717 | 2.66 | 0 | 0.06 |
| 116 | 1782 | 2.76 | 0 | 0.05 |
| 117 | 1685 | 2.61 | 0 | 0.02 |
| 118 | 1610 | 2.49 | 0 | 0.02 |
| 119 | 1609 | 2.49 | 0 | 0.05 |
| 120 | 1594 | 2.47 | 0 | 0.05 |
| 121 | 1473 | 2.28 | 0 | 0.07 |

### 5.4.2 Pile Capacity from PDA Output

The pile capacity, or failure load, according to the FDOT Standard Specifications for Road and Bridge Construction (FDOT 2004b) is defined as the load that causes a pile top deflection equal to the calculated elastic compression plus 0.15 inch plus 1/30 of the pile diameter for piles greater than 24 inches in width.

The pile capacity was automatically calculated by the PDA unit based on the measured data at the top set of instruments. Both piles had similar capacities for tip elevations above -34 feet, the capacity of Pile #1 did not exceed 256 kips, and Pile #2 did not exceed 242 kips. Pile #1 had the maximum capacity recorded at a tip elevation of -38 feet on BN 155 of 183. The PDA unit estimated the pile capacity to be 1080 kips with a maximum compressive force of 1540 kips.

## 5.5 CAPWAP Software Analysis of PDA Data

One-dimensional wave propagation through a pile is effected by changes in cross sectional properties. The axial strain and acceleration data recorded by the PDA unit included the effect of the steel pipe splice on wave propagation. The pile properties input to the PDA unit as shown in Table 5-1 did not include the changes in cross sectional area. The PDA automatic calculation of maximum stress used the voided cross sectional area of 646 in$^2$, however, twenty feet of the pile was primarily solid with a cross sectional area of 891 in$^2$. The pile impedance, Eqn. 1, was a function of both the modulus of elasticity and cross sectional area of the pile. The pile impedance, Eqn. 1, would increase in the 20 foot long spliced section because of the increased area and the increased transformed modulus of elasticity due to the steel pipe.

To account for the changes of cross sectional area and elastic modulus, the GRL, Inc. Case Pile Wave Analysis Program [CAPWAP] was used. The advantage of CAPWAP was the ability to model a spliced pile and the detailed output of force versus time for each pile segment. The CAPWAP software modeled the pile – soil interaction by considering equilibrium of forces acting on a short segment of pile. The pile was divided into a finite number of rigid weights, with elastic springs connecting them together to model the elastic compression of the pile. The inertial force of the segment was included to account for the weight of each segment. A nonlinear spring with the force dependent on the displacement was used to model the interaction between the pile tip (end bearing) and the soil, and the surface of the pile (side friction) and the soil.

### 5.5.1 CAPWAP Analysis Method

The strain transducer and accelerometer data for hammer impacts with high magnitudes of stress were imported into the CAPWAP software for more detailed analysis. The steel pipe splice method added a 20 foot long 99% solid section to the pile with a different cross-sectional area, specific weight and elastic modulus than the hollow section. The pile was modeled in CAPWAP by dividing the pile into one foot long segments and inputting the unit weight, cross sectional area, and transformed elastic modulus for each segment. The pile model input to CAPWAP is shown below in Table 5-5 and Figure 5-4.

Table 5-5 Pile model input to CAPWAP Software for effective length of pile.

| Distance Below Top Gages feet | Cross Sectional Area $in^2$ | Elastic Modulus ksi | Specific Weight $lb/ft^3$ | Pile Impedance (Eqn. 1) kips/ft/sec |
|---|---|---|---|---|
| 0 | 646 | 5500 | 151 | 273 |
| 4 | 646 | 5500 | 151 | 273 |
| 4 | 891 | 6180 | 159 | 410 |
| 24 | 891 | 6180 | 159 | 410 |
| 24 | 646 | 5500 | 151 | 273 |
| 30 | 646 | 5500 | 151 | 273 |
| 30 | 900 | 5500 | 151 | 381 |

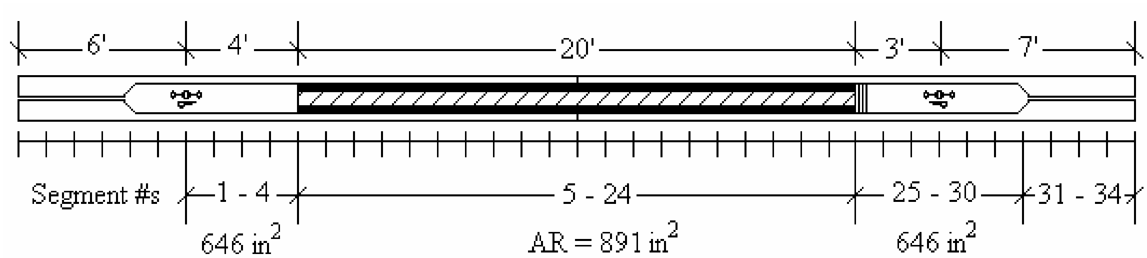



Figure 5-4 Pile divided into 1 foot long segments for CAPWAP software.

The elastic modulus of 5,500 ksi was the approximate value used in the PDA unit for the spliced pile with a uniform cross section. The elastic modulus of 6,180 ksi was calculated based on the ratio of the transformed elastic modulus in the splice to the elastic modulus of the voided pile. The calculation of transformed cross section properties were computed in the MathCAD worksheet in appendix D.

### 5.5.2 Analysis of Hammer Impacts at Critical Tip Elevations

The PDA output was used to identify the hammer impacts with high magnitudes of stress which typically occurred in the tip elevation range of -14 to -18 feet for tension, and between -36 to -39 feet for compression. The data from these hammer impacts was imported into CAPWAP for further analysis and verification. Additional hammer impacts for Pile #2 at other tip elevations, such as BN 119 and 227 were also analyzed in CAPWAP due to high tensile stresses.

The piles were both spliced at a tip elevation of -14 feet which was in a stiff layer. The initial 4 feet of driving of the spliced piles was critical because after punching through the stiff layer with compressive stresses, the tip was unsupported causing tensile stresses. For example, BN 17 and 18 from Table 5-3 were analyzed in CAPWAP.

The range of tip elevations between 36 feet and 39 feet below grade was only reached by Pile #1 because to reach this depth range, the soil adjacent to the pile was excavated so the top set of instruments would not be damaged by soil and water when the instruments went 5 feet below grade. The limestone rock layer was penetrated by Pile #1 at a tip elevation of -36 feet. The blow numbers with the maximum compressive stress and maximum pile capacity from Table 5-4 and sections 5.4.1 and 5.4.2 were analyzed using CAPWAP, for example, BN 116, 117, 154, and 155.

## 5.6 Results of CAPWAP Software Analysis

The results of interest were the maximum tensile and compressive stresses in the steel pipe splice section and the maximum pile capacity. The maximum compressive stress and maximum pile capacity occurred during the same range, thus are discussed together.

### 5.6.1 Maximum Tensile Stress in the Splice Section

The output of maximum force and stress in each one foot long pile segment was used to identify the hammer impact that caused the maximum tensile stress in the 20 foot long spliced pile cross section. The maximum value table was output by CAPWAP for each hammer impact analyzed. Presented below in Table 5-6 is the extreme values table for BN 17 of 383, the hammer impact with the maximum magnitude of tensile stress in the steel pipe splice section.

Table 5-6 Maximum value table for BN 17 of 383 for each segment of Pile #2.

| Pile Segment No. | Distance Below Top Gages feet | Max Compressive Force kips | Max Compressive Stress ksi | Max Tensile Force kips | Max Tensile Stress ksi | Cross Sectional Area $in^2$ |
|---|---|---|---|---|---|---|
| 1 | 1 | 1345 | 2.08 | -149 | -0.23 | 646 |
| 2 | 2 | 1340 | 2.08 | -152 | -0.235 | 646 |
| 4 | 4 | 1324 | 1.51 | -155 | -0.177 | 646 |
| 6 | 6 | 1300 | 1.46 | -159 | -0.178 | 891 |
| 8 | 8 | 1271 | 1.43 | -173 | -0.195 | 891 |
| 10 | 10 | 1237 | 1.39 | -221 | -0.248 | 891 |
| 12 | 12.1 | 1191 | 1.34 | -258 | -0.289 | 891 |
| 14 | 14.1 | 1138 | 1.28 | -291 | -0.326 | 891 |
| 16 | 16.1 | 1074 | 1.21 | -315 | -0.353 | 891 |
| 18 | 18.1 | 1001 | 1.12 | -326 | -0.366 | 891 |
| 20 | 20.1 | 933 | 1.05 | -335 | -0.376 | 891 |
| 22 | 22.2 | 858 | 0.96 | -329 | -0.369 | 891 |
| 24 | 24.2 | 771 | 0.91 | -302 | -0.357 | 891 |
| 26 | 26.2 | 700 | 1.08 | -268 | -0.414 | 646 |
| 27 | 27.1 | 663 | 1.03 | -246 | -0.381 | 646 |
| 28 | 28.1 | 625 | 0.97 | -222 | -0.344 | 646 |
| 29 | 29.1 | 586 | 0.91 | -196 | -0.303 | 646 |
| 30 | 30.1 | 543 | 0.81 | -167 | -0.25 | 646 |
| 31 | 31.1 | 485 | 0.55 | -125 | -0.14 | 891 |
| 32 | 32 | 379 | 0.43 | -165 | -0.185 | 891 |
| 33 | 33 | 322 | 0.36 | -113 | -0.127 | 891 |
| 34 | 34 | 313 | 0.35 | -55 | -0.062 | 891 |

Another output of CAPWAP was a force versus time plot for any pile segment of interest. The pile segments chosen were at the top of the pile, the segment with the maximum tensile force in the spliced section of the pile, and at segment 27 which is the location of the lower set of instruments. Shown below in Figure 5-5 is the force versus time plot which shows the magnitude of tension and compression at three locations in the pile. For this plot, segment 20 was chosen because it had the maximum tensile force. In Figure 5-5, note that the maximum compressive force at time 0.021 seconds was the largest at the top of the pile from the hammer impact, and decreased for each segment down the pile. This trend was also seen in Table 5-6 in the maximum force column.

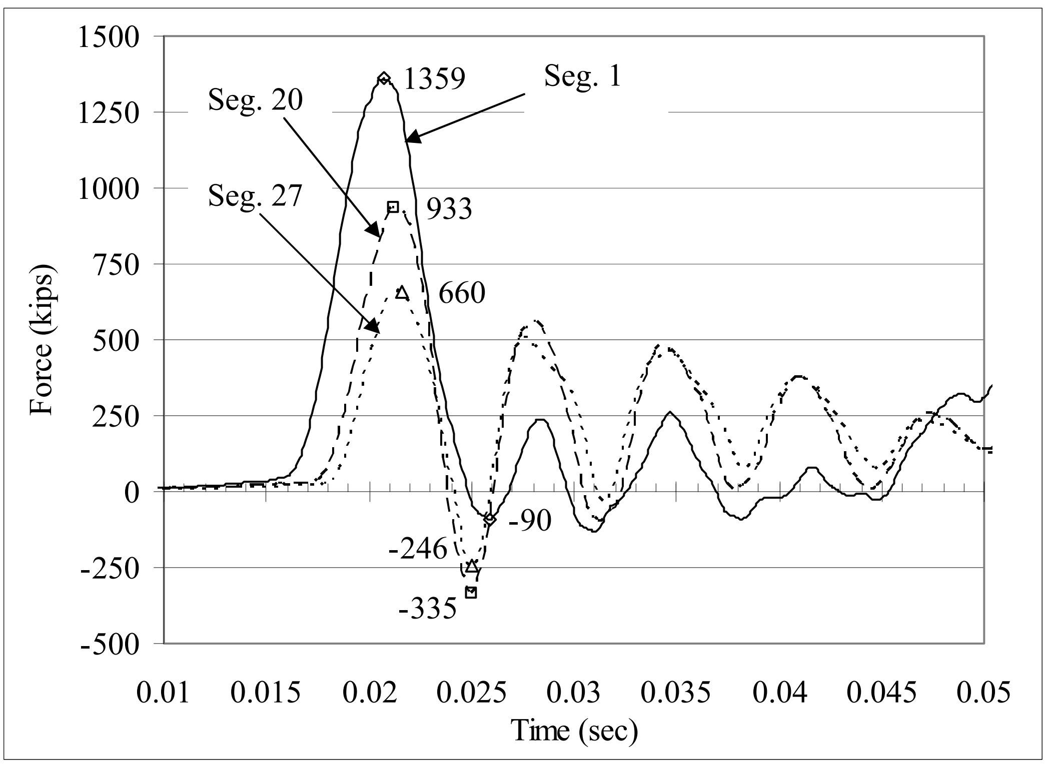



Figure 5-5 CAPWAP output of force at three pile segments for BN 17 of 383 with maximum tensile force for spliced Pile #2.

The maximum tensile force originated at the bottom of the pile and propagated up the pile. The tensile force in the bottom segments of the pile was small because the downward traveling compressive wave was still arriving when the tensile wave was traveling upward. At the middle of the pile the total force was predominantly tensile because the downward force had passed and the upward traveling tensile force controlled the magnitude. The upward traveling tensile wave can also be seen in Figure 5-1.

During several hammer impacts, the tensile stress in the splice was close to the maximum for BN 17 of 383 of Pile #2. Table 5-7 summarizes the output from CAPWAP of several BN with the maximum top force, the maximum tensile force and stress in the splice, and the distance below the top gages to the pile segment.

Table 5-7 Summary of BN with high tensile stresses in the splice of Pile #2 with spliced cross sectional of 891 in$^2$.

| BN | Max Top Force kips | Max Tensile Force in the Splice kips | Max Tension Stress in the Splice ksi | Distance Below Top Gages to Pile Segment feet |
|---|---|---|---|---|
| 17 | 1345 | -335 | -0.376 | 20.1 |
| 18 | 1358 | -305 | -0.342 | 20.1 |
| 119 | 1346 | -319 | -0.358 | 20.1 |
| 227 | 1346 | -321 | -0.36 | 16.1 |

In appendix E, maximum value tables are included for each BN included in Table 5-7, in addition to figures such as, wave up match, force at top, force at middle, and force at segment 27 plotted versus time.

### 5.6.2 Maximum Pile Capacity and Compressive Stress in the Splice Section

High compressive stresses were recorded for several hammer impacts of Pile #1 when the pile tip was above the hard layer. The hammer impact with the maximum magnitude of compressive stress in the pile was BN 116 of 183. During several hammer impacts, the compressive stress in the pile was close to the maximum for BN 116 of 183 of Pile #1. Table 5-8 below summarizes the output from CAPWAP of several BN with a high pile capacity and high compressive stresses. The maximum top force, the maximum compressive stress in the pile, and the distance below the top gages to the pile segment are included in Table 5-8.

Table 5-8 Summary of BN with high pile capacity and compressive stresses in Pile #1 with spliced cross sectional area of 891 $in^2$.

| BN | Length of Penetration feet | Pile Capacity kips | Max Top Force kips | Max Compressive Stress in the Splice ksi | Distance Below Top Gages to Pile Segment feet |
|---|---|---|---|---|---|
| 116 | 36.9 | 782 | 1780 | 2.00 | 6 |
| 117 | 37 | 699 | 1685 | 1.89 | 6 |
| 154 | 38.2 | 951 | 1485 | 1.66 | 6 |
| 155 | 38.2 | 1184 | 1520 | 1.69 | 6 |

Shown below in Figure 5-6 is the force at the top, middle, and segment 27 of the pile versus time for BN 116 of 183. In appendix F, maximum value tables are included for each BN included in Table 5-8, in addition to figures such as wave up match, force at top, force at middle, and force at segment 27 plotted versus time.

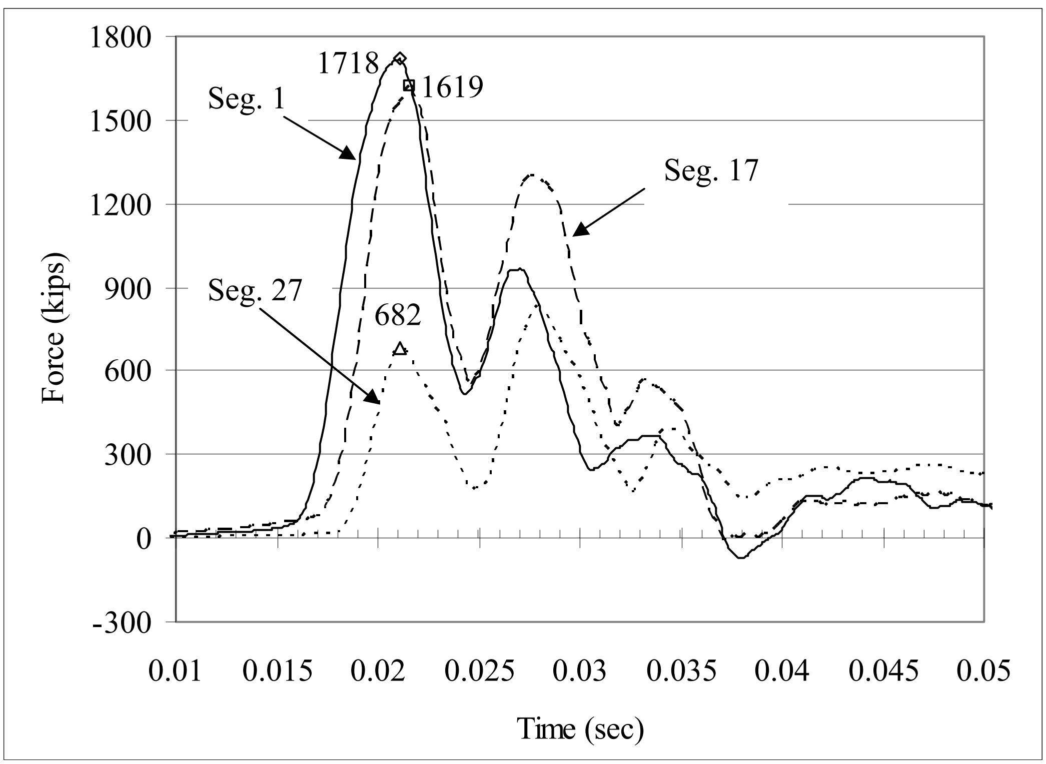



Figure 5-6 CAPWAP output of force at three pile segments for BN 116 of 383 with maximum compressive force for spliced Pile #1.

Presented below in Table 5-9 is the maximum value table for BN 116 of 183. Note that the maximum compressive force is from the initial downward traveling wave because it decreased as the force moved down the pile. The maximum values from Figure 5-6 were included in Table 5-9.

Table 5-9 Maximum value table for BN 116 of 183 for each segment of Pile #1.

| Pile Segment No. | Distance Below Top Gages feet | Max Compressive Force kips | Max Compressive Stress ksi | Max Tensile Force kips | Max Tensile Stress ksi | Cross Sectional Area $in^2$ |
|---|---|---|---|---|---|---|
| 1 | 1 | 1771 | 2.74 | -0.6 | -0.001 | 646 |
| 2 | 2 | 1776 | 2.75 | -0.5 | -0.001 | 646 |
| 4 | 4 | 1779 | 2.02 | -3.3 | -0.004 | 646 |
| 6 | 6 | 1772 | 1.99 | -3.9 | -0.004 | 891 |
| 8 | 8 | 1762 | 1.98 | -1.8 | -0.002 | 891 |
| 10 | 10 | 1749 | 1.96 | -13.2 | -0.015 | 891 |
| 12 | 12.1 | 1723 | 1.93 | -17.9 | -0.02 | 891 |
| 14 | 14.1 | 1683 | 1.89 | -15.3 | -0.017 | 891 |
| 16 | 16.1 | 1642 | 1.84 | -13.8 | -0.015 | 891 |
| 18 | 18.1 | 1590 | 1.78 | -7.6 | -0.009 | 891 |
| 20 | 20.1 | 1224 | 1.37 | 0 | 0 | 891 |
| 22 | 22.2 | 1151 | 1.29 | 0 | 0 | 891 |
| 24 | 24.2 | 943 | 1.11 | 0 | 0 | 891 |
| 26 | 26.2 | 949 | 1.47 | 0 | 0 | 646 |
| 27 | 27.1 | 830 | 1.29 | 0 | 0 | 646 |
| 28 | 28.1 | 828 | 1.28 | 0 | 0 | 646 |
| 29 | 29.1 | 824 | 1.28 | 0 | 0 | 646 |
| 30 | 30.1 | 816 | 1.22 | 0 | 0 | 646 |
| 31 | 31.1 | 725 | 0.81 | -0.2 | 0 | 891 |
| 32 | 32 | 715 | 0.8 | -0.3 | 0 | 891 |
| 33 | 33 | 708 | 0.79 | -0.4 | 0 | 891 |
| 34 | 34 | 696 | 0.78 | -0.3 | 0 | 891 |

The maximum pile capacity occurred during BN 155 of 183 of Pile #1 at a tip elevation of -38 feet. The maximum compressive force in the pile was 1525 kips, the pile capacity was 1184 kips, with 575 kips shaft resistance and 608 tip resistance.

### 5.7 Comparison of PDA Output with CAPWAP Software Output

The match quality was used in CAPWAP to rate the correctness of the computed solution. The match quality was based on a comparison between the PDA measured values and the CAPWAP computed values at the top set of instruments for the items outlined below:

- Blow Count match.
- Wave Up at top gages versus time, as shown in Figure 5-7.
- Force at top gages versus time, as shown in Figure 5-8.
- Velocity at top gages versus time, as shown in Figure 5-9.

Wave up matching was the preferred method of analysis, because it used information from both the strain transducers and the accelerometers, whereas the other two matching methods only used one type of instrument, the average strain or the average acceleration versus time.

The shape of the computed wave versus time as shown in Figures 5-7, 5-8, and 5-9 was adjusted by changing the variables that define the interaction between the soil and the pile below the top set of instruments. For example, the resistance distribution on the shaft and the force at the toe of the pile were adjusted to improve the match quality. The estimated pile capacity and the magnitude of stresses output from PDA unit were used to estimate the shaft resistance and toe force for the iterations. The soil quake and damping values were also adjusted to improve the match quality. The other method of improving the match quality was by using the automatic features of CAPWAP. The soil parameters were optimized by defining the minimum, maximum and tolerance value for each variable, and the software would iterate the parameters. The parameters to be adjusted were chosen all at once, or the unloading related parameters or the toe related parameters. The impedance of each pile segment was also adjusted to the values recommended by

CAPWAP to increase the match quality. The input pile impedance and the adjusted pile impedance are included in appendix E and F for each BN included in Table 5-7 and Table 5-8, respectively.

Iterations were performed until the match quality was less than five, or further improvement was not possible. The match quality for BN 17 was 2.92 without including the input blow count, or 5.85 with the blow count included for matching the measured wave up to the computed wave up versus time as shown in Figure 5-7. The match of the top force measured by the PDA unit and the top force computed using CAPWAP for BN 17 is shown in Figure 5-8. The match of the velocity measured by the PDA unit at the top of the pile and the velocity computed using CAPWAP for BN 18 is shown in Figure 5-9. Figures 5-7, 5-8, 5-9 are for the top set of instruments.

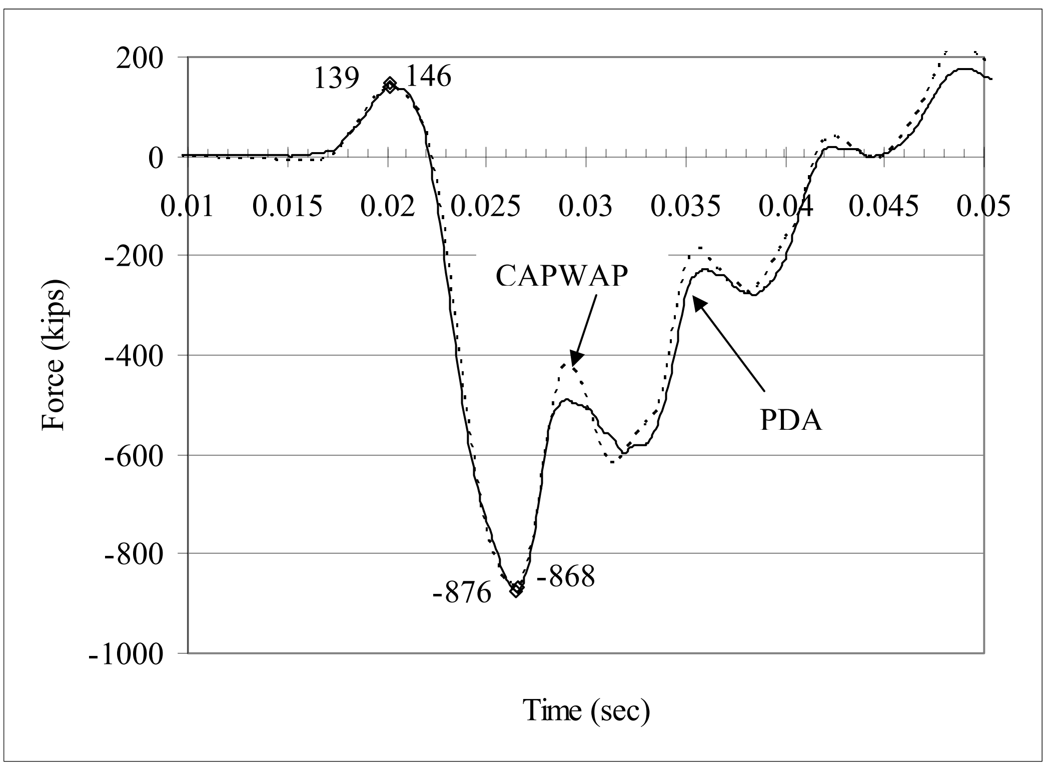



Figure 5-7 Match quality of output of CAPWAP computed wave up and PDA measured wave up at the top of Pile #2 for BN 17 of 383.

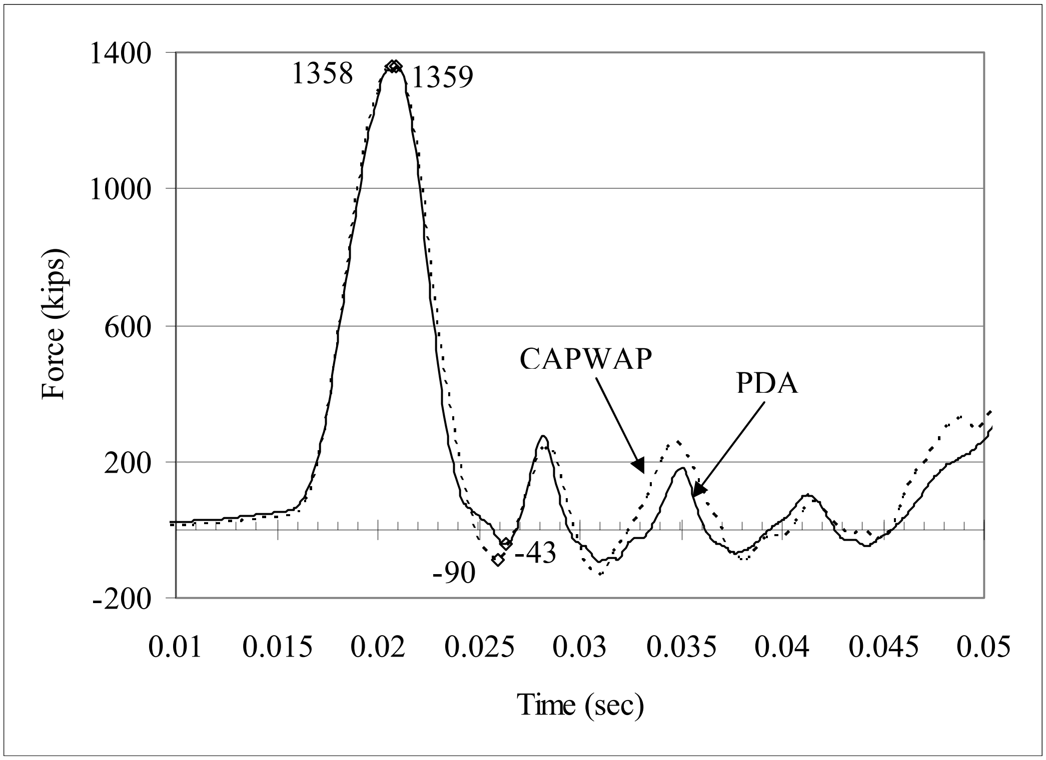



Figure 5-8 Match quality of output of CAPWAP computed force and PDA measured force at the top of Pile #2 for BN 17 of 383.

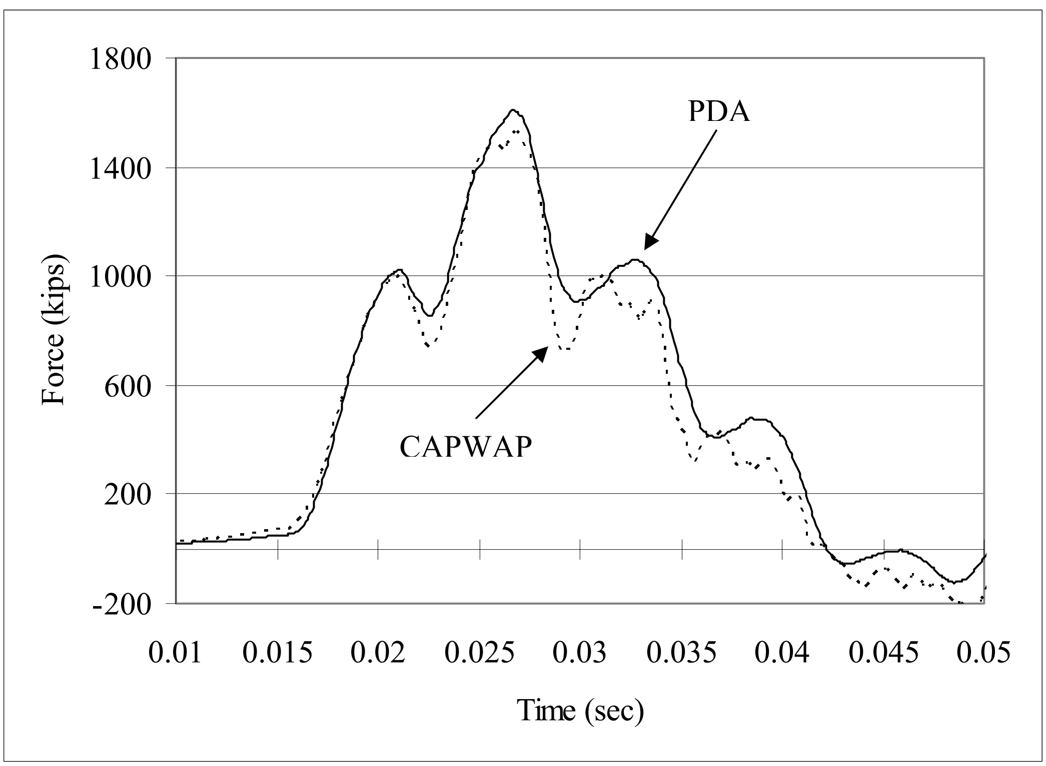



Figure 5-9 Match quality of output of CAPWAP computed velocity and PDA measured velocity at the top of Pile #2 for BN 18 of 383.

The bottom set of instruments were used to verify the CAPWAP software output at the location of the bottom set of instruments. For each hammer impact analyzed in CAPWAP, the computed force at the bottom instrument location was plotted versus time. The average measured strain at the bottom set of instruments was plotted versus time as an equivalent force by multiplying by the cross-sectional area of the voided pile, AR, and the elastic modulus, EM. For example, BN 17 of 383 of Pile #2 was the hammer impact with the maximum tensile stress. Figure 5-10 is the PDA measured and CAPWAP computed force at the lower strain transducers for BN 17 of 383 for Pile #2.

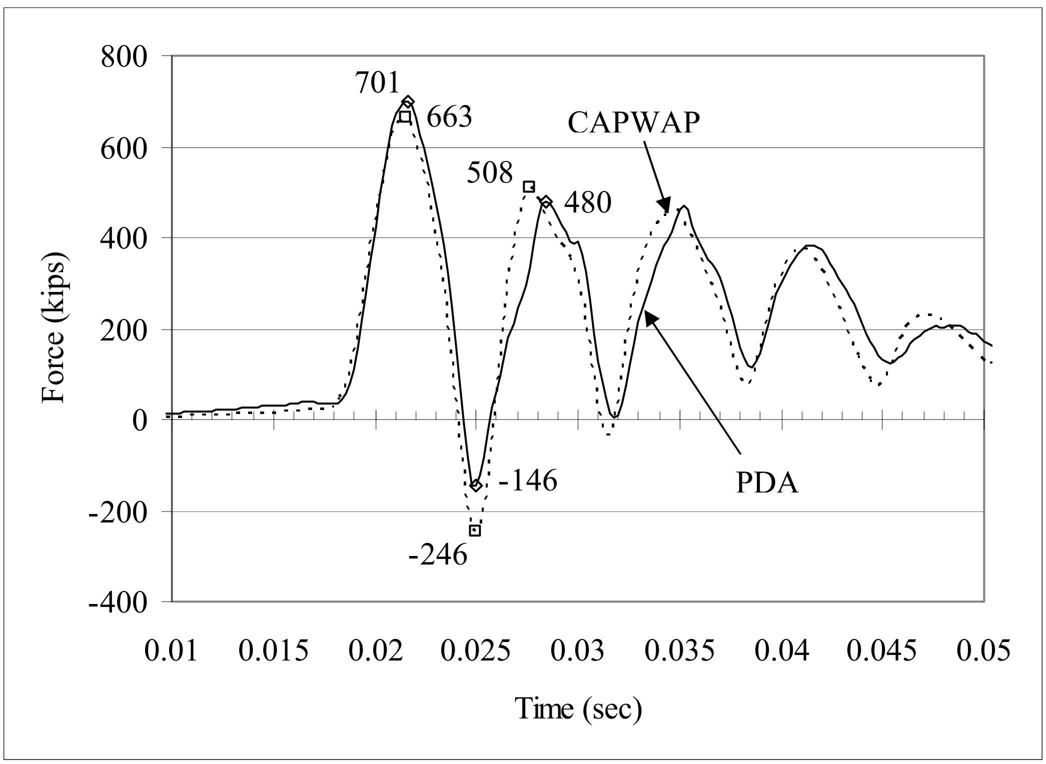



Figure 5-10 Comparison of PDA output and CAPWAP output at the lower gage location.

A comparison of the PDA output and CAPWAP output at the lower gage location, similar to Figure 5-10, is included in appendix E and F for each BN included in Table 5-7 and Table 5-8, respectively.

Another output to compare between the PDA unit and the CAPWAP software was the maximum pile stresses and the maximum pile capacity. Table 5-10 and 5-11 is a comparison between the values of interest from the PDA unit output and the CAPWAP software output. For the percentage difference calculation, the CAPWAP value was the true value. The maximum stresses output by CAPWAP in Table 5-10 are included in the maximum value tables in Appendix E. The goal of the CAPWAP software analysis was not to match the output from the PDA unit. CAPWAP considered pile impedance changes that were not considered in the PDA unit.

Table 5-10 Pile #2 comparisons of PDA and CAPWAP maximum stresses.

| BN | PDA Compressive Stress ksi | CAPWAP Compressive Stress ksi | % Difference | PDA Tensile Stress ksi | CAPWAP Tensile Stress ksi | % Difference |
|---|---|---|---|---|---|---|
| 17 | 2.11 | 2.08 | 1.44 | -0.39 | -0.444 | 12.2 |
| 18 | 2.09 | 2.09 | 0 | -0.393 | -0.408 | 3.68 |
| 119 | 2.11 | 2.08 | 1.44 | -0.377 | -0.401 | 5.96 |
| 227 | 2.27 | 2.27 | 0 | -0.393 | -0.362 | 8.56 |
| Avg. % Difference | | | 0.72 | Avg. % Difference | | 7.6 |

Table 5-11 Pile #1 comparisons of PDA and CAPWAP maximum compressive stresses and pile capacity.

| BN | PDA Compressive Stress ksi | CAPWAP Compressive Stress ksi | % Difference | PDA Pile Capacity kips | CAPWAP Pile Capacity ksi | % Difference |
|---|---|---|---|---|---|---|
| 116 | 2.76 | 2.74 | 0.73 | 891 | 782 | 14 |
| 117 | 2.61 | 2.61 | 0 | 717 | 699 | 2.6 |
| 154 | 2.27 | 2.29 | 0.87 | 1058 | 951 | 11.3 |
| 155 | 2.38 | 2.35 | 1.27 | 1077 | 1184 | 9 |
| Avg. % Difference | | | 0.72 | Avg. % Difference | | 9.3 |

The maximum compressive stress and tensile stress typically occurred for times less than 5.2 milliseconds, which is for the first time the wave traveled down the pile and back up the pile.

The PDA data of force, velocity and wave up versus time from the top set of instruments was compared with the CAPWAP output as shown in Figures 5-7, 5-8, and 5-9. The two traces in each figure are well matched for shape and maximum values. The data recorded at the bottom of the pile was used as a second check to verify the CAPWAP software output of the force in each one foot long pile segment as shown in Figure 5-10. The CAPWAP software output of maximum force in each one foot long pile segment was accurate because it was verified at the top and bottom set of instruments.

## 5.8 Summary of Data Analysis Results

Dynamic load testing was used to assess the pile capacity and maximum forces in the pile during driving. The PDA data was analyzed using CAPWAP software to account for the changes in cross sectional area and elastic modulus. The CAPWAP software modeled the pile – soil interaction by dividing the pile into one foot long segments. This provided the output of maximum force in each segment. The CAPWAP output was verified to be accurate by a comparison of the force, velocity and wave up traces at the top set of instruments. The bottom set of instruments also verified the computed force versus time output below the splice section at segment 27.

The maximum compressive force of 1780 kips was measured at the top of Pile #1 during BN 116. The high force was due to the high pile capacity. The net compressive force was larger than the magnitude of the downward traveling wave, because the reflection from the toe of the pile was compressive. Several other BN had equivalent compressive forces in the splice section in Pile #1, such as BN 116, 117, 154, and 155. The maximum compressive stress measured during pile driving was less than the maximum allowable specified in Section 455 of the FDOT Standard Specifications for Road and Bridge Construction. However, the unconfined compressive strength of the prestressed concrete pile is specified at 6000 psi. Thus the compressive stresses during driving are not as problematic as the tensile stresses which typically cause concrete to fail. Even though the maximum compressive stress was not exceeded, the pile splice should be able to carry a higher compressive force than measured by the PDA.

The maximum net tensile force recorded in the spliced section of Pile #2 was 335 kips, or 0.375 ksi when divided by the spliced cross sectional area of 891 $in^2$. If the largest measured tension load was assumed to be carried only by the steel, the resulting tensile stress in the pipe was 16 ksi during pile driving. Several other BN had equivalent tensile forces in the splice section in Pile #2, such as BN 17, 18, 119, and 227. The magnitude of the upward traveling tensile force wave was 876 kips, for BN 17 as shown in Figure 5-7. The short pile length caused the maximum net force to only be 335 kips tensile, because of the downward traveling compressive force wave.

For the 40 foot long pile with an effective length of 34 feet, the time for the wave to go down the pile and be reflected back to the top set of instruments was 5.2 milliseconds. The duration of the hammer impact was the rise time on the force graph as shown in Figure 5-8. It can be seen in Figure 5-1 that the upward traveling wave was occurring while the downward traveling wave was still occurring. This was because the rise time was approximately equal to the time required for the wave to go down the pile and back up. This was a problem because the maximum tensile force in the wave up was covered up by the initial downward traveling compression wave. If the pile were twice as long, the full tensile wave up could have crossed the splice region and the tensile stresses would have been higher. In actual application, during pile driving, the PDA would alert the field engineer to the high tensile stresses, and the pile cushion thickness would be increased, or the hammer fuel setting decreased to limit the stresses within those specified in Section 455 of the FDOT Standard Specifications for Road and Bridge Construction.

The concrete in the transfer length of the prestressing strands would be more likely to fail in tension than the concrete outside of the transfer length, because of the net compression transferred to the concrete. For this splice design, the tensile load would be redistributed to the steel pipe to be carried across the splice interface (Britt, Cook, and McVay 2003). The steel pipe can resist a tensile load before yielding of 832 kips, so the full magnitude tensile wave up could be carried across the splice by the steel pipe.

The maximum tensile stresses recorded exceeded 350 psi tension within the transfer length of the splice mating surface between pile ends. The maximum allowable tensile stress is limited to 252 psi anywhere in the pile by Section 455 of the FDOT Standard Specifications for Road and Bridge Construction. The splice design was tested with stresses greater than the allowable stresses, for example, in Table 5-6 the maximum tensile stress in the voided pile at segment 26 was 414 psi. Therefore if Section 455 is observed during driving of the steel pipe splice, it should be strong enough to resist the tensile stresses.

# CHAPTER 6
# SUMMARY AND CONCLUSION

## 6.1 Summary

The steel pipe splice method presented in this report is an alternative method for splicing voided 30 inch square prestressed concrete piles. Previous laboratory research (Issa 1999) on the steel pipe splice has shown that a 15 foot long steel pipe splice, with 7.5 feet on either side of the joint, developed an ultimate moment capacity that was 96% of the calculated spliced pile nominal moment capacity, and 84% of the unspliced pile nominal moment capacity.

The goal of this research project was to test the axial capacity of the splice to validate that it could withstand the maximum allowable stress limits specified in Section 455 of the FDOT Standard Specifications (2004). Since the maximum axial load that the pile will undergo occurs during pile driving installation, this project involved the installation of two spliced piles constructed with the same materials and time schedule as in typical field conditions. Basically, the splice utilized a 20 foot long 14 inch diameter steel pipe grouted into the 18 inch diameter void of the pile with 10 feet on either side of the joint. Details on the construction and installation process are provided in Section 4.7 and information on the materials specified is provided in Chapter 2.

During the installation the axial forces propagating through the piles for each hammer impact were measured. Details on the instrumentation and analysis of the field data are provided in Chapter 5. The stresses resulting from these forces were then compared to the maximum allowable stresses.

Section 455 of the FDOT Standard Specifications for Road and Bridge Construction were used to determine the maximum allowable pile driving stresses. For a continuous unspliced 30 inch prestressed concrete pile, the maximum allowable tensile stress is 1,200 psi and the maximum allowable compressive stress is 3,500 psi. For a spliced 30 inch prestressed concrete pile, the maximum allowable tensile stress is 250 psi because the prestressing strands are terminated at the splice. The maximum allowable compressive stress is 3,500 psi in the prestressed portion and 4,200 psi in the non-prestressed splice region.

Based on analysis of the measured field data, the spliced pile withstood a maximum concrete tensile stress of 375 psi in the splice section and 444 psi in the voided section of pile without showing a visible signs of degradation. Although it may not be prudent to permit an increase in the maximum allowable tensile stress of 250 psi for piles spliced using this method, the results certainly show that this type of pile splice can be implemented under the current limits for concrete tensile stress.

The maximum compressive stress determined from analysis of the field data was 2,800 psi in the voided section of pile and 2,000 psi in the splice section (note that there is a larger concrete area at the splice). Although the measured compressive stress was less than the allowable compressive stress (due to the rock layer not being firm enough to cause a higher compressive load), there should be no need to limit the allowable compressive stress for this type of splice since in the area of the splice there is a larger cross-sectional area of concrete to transfer the compression load than that of the currently approved dowel splice system.

Regarding the steel pipe, the minimum specified yield strength of the pipe was 42 ksi and the splice length of 20 feet was designed to ensure that the steel could yield. If the largest measured tension load is assumed to be carried only by the steel, the resulting tensile stress in the pipe was limited to 16 ksi during pile driving.

## 6.2 Conclusion

The results of this research project indicate that an alternative pile splice method using a 20 foot long 14 inch diameter steel pipe section grouted into 30 inch voided piles is a viable method that should be considered for FDOT approval. The recommended materials for the splice are specified in Chapter 2 and details of the construction and installation processes are provided in Section 4.7. For installation, it is recommended to continue with the allowable stress limits currently specified in Section 455 of the FDOT Standard Specifications for Road and Bridge Construction.

## 6.3 Recommended Pile Splice Specifications

The following recommendation includes steel pipe splice construction specifications and detailed drawings of the construction process. Figure 6-1 provides recommended construction specifications for the pile splice. Figure 6-2 is an elevation view showing three stages in the construction process: pre-splice preparation, splice assembly setup for grouting, and grout mix and placement. Figure 6-3 is a mating surface detail showing the steel pipe filled with concrete, the form used to retain the mating surface grout, the grout inlet hole, and the hole for temporary steel bolts. Figure 6-4 is a detail of the foam rubber plug that was used to seal the void below the splice section. Figure 6-5 is a pile cross section view at the location of the steel bolts that support the steel pipe vertically.

CONSTRUCTION SPECIFICATIONS

PRE-SPLICE PREPARATION

1. The HSS 14.00 x 0.500 pipe shall be filled with concrete and a 3 inch diameter vent pipe shall extend 6 inches above top of splice section.
2. ½ inch diameter steel bars shall be formed into hoops and fillet welded (2 inches of 3/16 inch fillet weld per foot) to the HSS pipe at 8 inches on center.
3. The pile shall be cutoff in the voided section, approximately 5 feet below the pile top. The metal liner shall be trimmed and the edges shall be bent smooth after the pile is cutoff, to allow the foam rubber plug to be inserted.
4. Two (2) holes, 1.25 inch diameter shall be drilled on two (2) opposite faces of the pile 1 foot below the cutoff, to receive steel bolts. Before attaching the grout plug, fit the HSS into the pile void to mark the hole location on the HSS pipe to receive steel bolts.
5. One (1) hole, 1 inch diameter shall be drilled 8 inches below the cutoff to attach the grout inlet port.
6. One (1) hole, 1 inch diameter shall be drilled 10 feet from the end of the splice section to monitor the grout level.
7. Cut holes in the HSS pipe to receive temporary steel dowels.

SPLICE ASSEMBLY SETUP FOR GROUTING

1. Setup and assemble bracing for top half of splice. A template, steel channels or equivalent shall be used. The top half shall be supported so the crane choker cable is slackened.
2. Attach foam rubber plug or equivalent to seal the 2 inch wide annulus gap. The grout plug shall prohibit the grout from filing the pile below the splice section.
3. Insert the HSS pipe with grout plug attached into the void, insert steel dowels to support the HSS pipe vertically.
4. Attach mating surface formwork.
5. Lower the spliced section into positon, check bracing alignment and gap between pile ends.

GROUT MIX AND PLACEMENT

1. The mating surface grout shall seal the gap between the pile ends.
2. The mating surface grout shall set quickly and have a high strength. (Masterbuilders Set 45 or equivalent shall be used.)
3. The choker cable shall be slackened and the splice section shall be braced to prevent movement.
4. The annulus grout shall be mixed and continuously pumped to fill the splice annulus. (Masterbuilders Masterflow 928 or equivalent shall be used).
5. Verify flow cone time is in accordance with product specification sheet.
6. Annulus grout cubes shall be made to verify grout strength is greater than 3800 psi, prior to driving spliced piles.

Figure 6-1 Steel pipe splice specifications for construction.

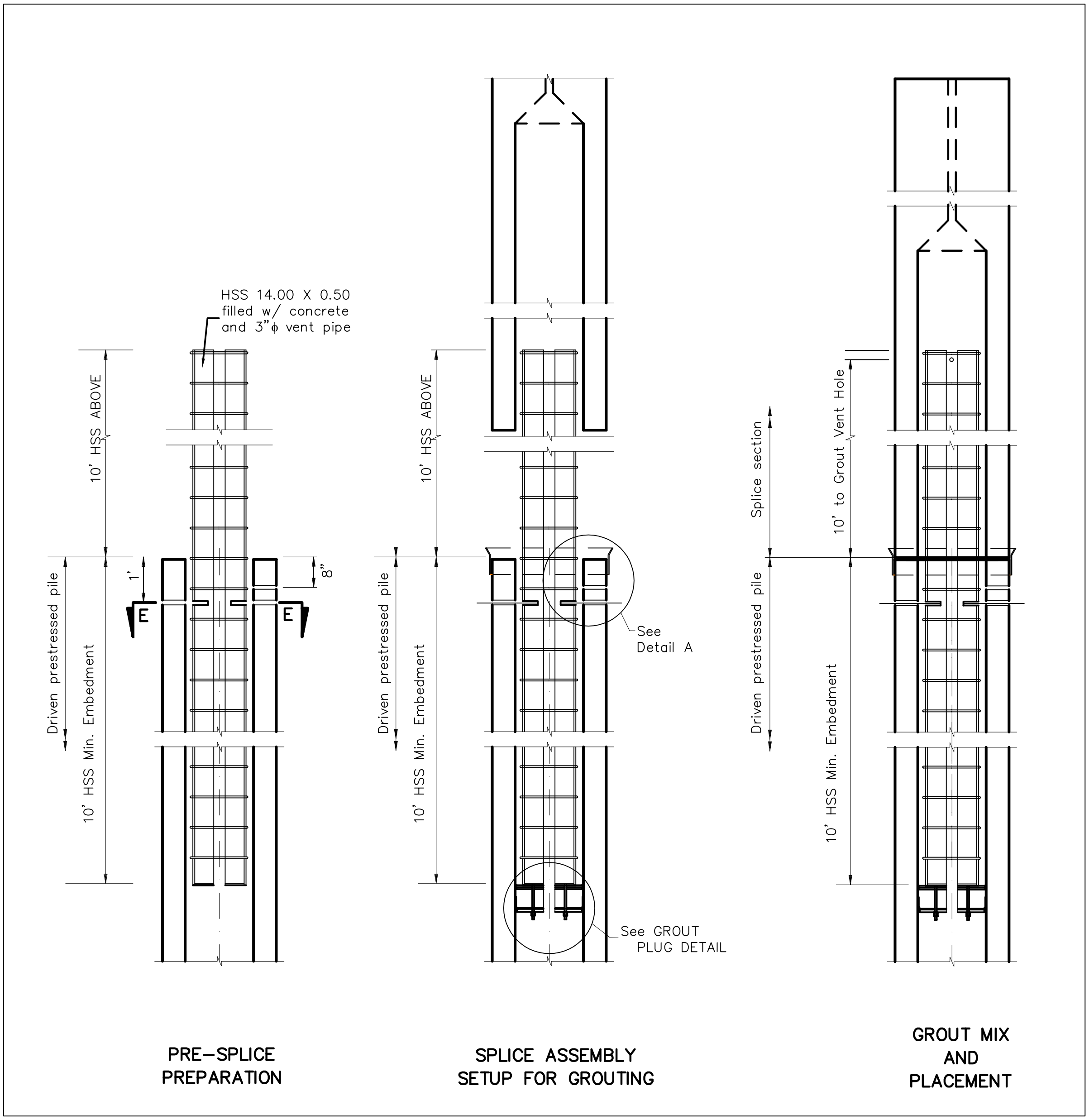



Figure 6-2 Elevation view of splice construction process.

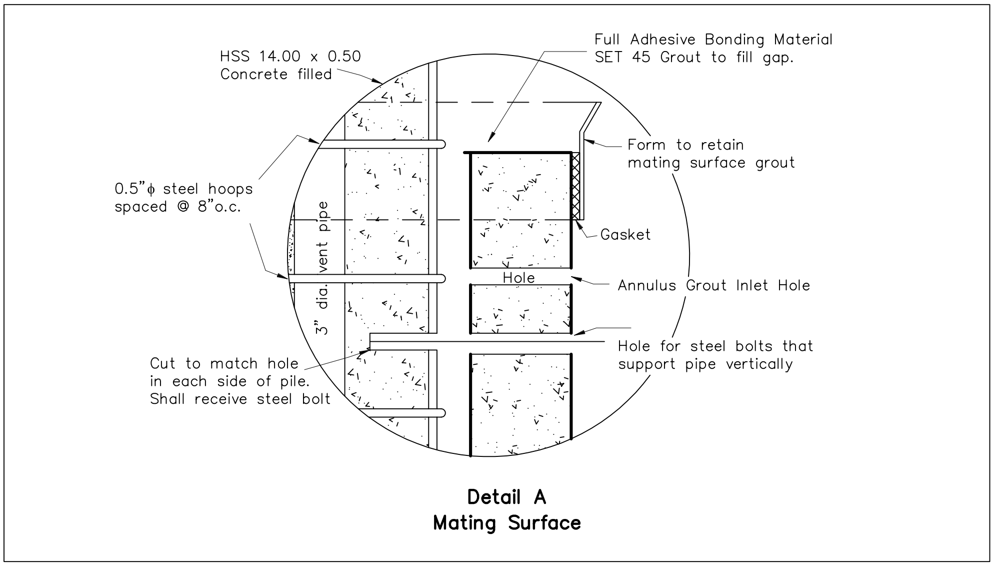



Figure 6-3 Mating surface detail of the steel pipe splice.

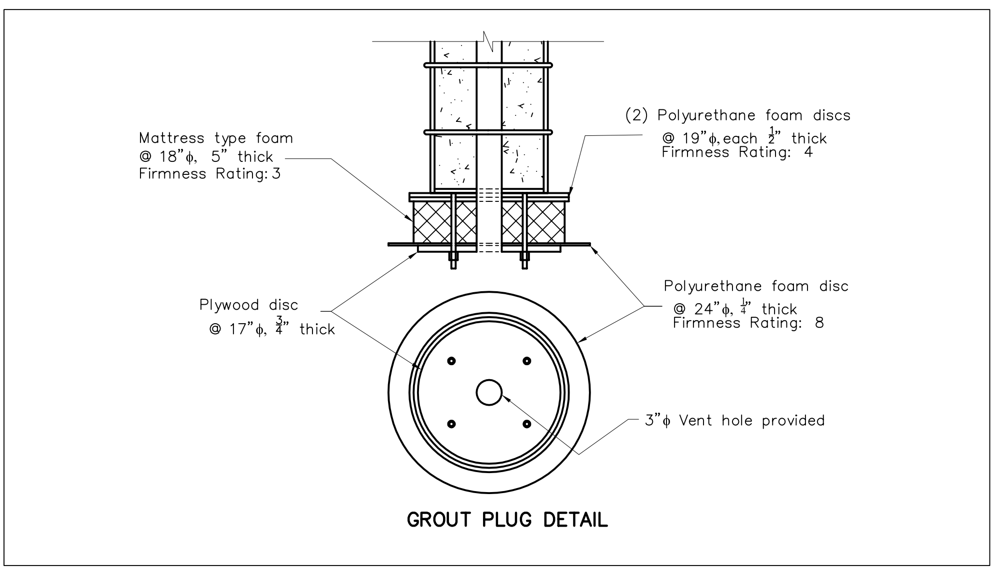



Figure 6-4 Grout plug detail with materials and dimensions.

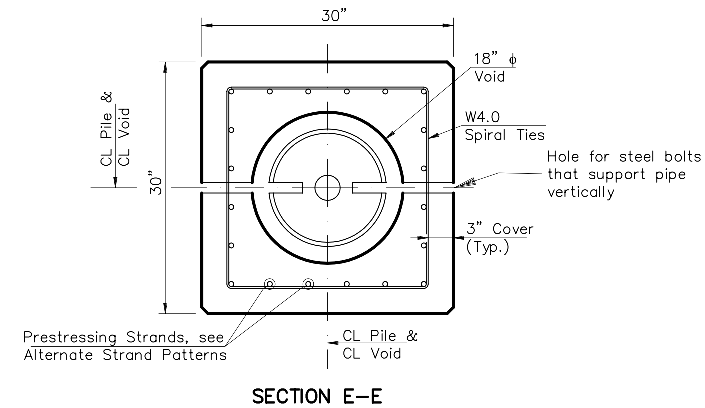



Figure 6-5 Cross section view of the spliced pile at the steel pipe vertical support.

# APPENDIX A
# CEMENTITIOUS GROUTS

This appendix contains the product specification sheets for the grouts used in the annulus and at the mating surface of the splice. Pictures of the grout mixing and pumping machine are also included.

A


B

Figure A-1 Grout mixing operation. A) DSI grout mixer and flow cone time measured by FDOT, B) DSI grout mixer and pump machine.

degussa.

*Construction Chemicals*

# MASTERFLOW® 928

**High-precision mineral-aggregate grout with extended working time**

PRODUCT DATA

3 03600 **Grouts**

## Description

Masterflow® 928 grout is a hydraulic cement-based mineral-aggregate grout with an extended working time. It is ideally suited for grouting machines or plates requiring precision load-bearing support. It can be placed from fluid to damp pack over a temperature range of 45 to 90° F (7 to 32° C). Masterflow® 928 grout meets the requirements of ASTM C 1107, Grades B and C, and the Army Corp of Engineers' CRD C 621, Grades B and C, at a fluid consistency over a 30-minute working time.

## Yield

One 55 lb (25 kg) bag of Masterflow® 928 grout mixed with approximately 10.5 lbs (4.8 kg) or 1.26 gallons (4.8 L) of water, yields approximately 0.50 $ft^3$ (0.014 $m^3$) of grout.

The water requirement may vary due to mixing efficiency, temperature, and other variables.

### Packaging

55 lb (25 kg) multi-wall paper bags

3,300 lb (1,500 kg) bulk bags

### Shelf Life

1 year when properly stored

### Storage

Store in unopened bags in clean, dry conditions.

Protection and Repair

| Features | Benefits |
|---|---|
| • Extended working time | Ensures sufficient time for placement |
| • Can be mixed at a wide range of consistencies | Ensures proper placement under a variety of conditions |
| • Freeze/thaw resistant | Suitable for exterior applications |
| • Hardens free of bleeding, segregation, or settlement shrinkage | Provides a maximum effective bearing area for optimum load transfer |
| • Contains high-quality, well-graded quartz aggregate | Provides optimum strength and workability |
| • Sulfate resistant | For marine, wastewater, and other sulfate-containing environments |

## Where to Use

APPLICATION

- Where a nonshrink grout is required for maximum effective bearing area for optimum load transfer
- Where high one-day and later-age compressive strengths are required
- Nonshrink grouting of machinery and equipment, baseplates, soleplates; precast wall panels, beams, columns; curtain walls, concrete systems, other structural and nonstructural building members; anchor bolts, reinforcing bars, and dowel rods
- Applications requiring a pumpable grout
- Repairing concrete, including grouting voids and rock pockets
- Marine applications
- Freeze/thaw environments

LOCATION

- Interior or exterior

## How to Apply

### Surface Preparation

1. Steel surfaces must be free of dirt, oil, grease, or other contaminants.
2. The surface to be grouted must be clean, SSD, strong, and roughened to a CSP of 5 – 9 following ICRI Guideline 03732 to permit proper bond. For freshly placed concrete, consider using Liquid Surface Etchant (see Form No. 1020198) to achieve the required surface profile.
3. When dynamic, shear or tensile forces are anticipated, concrete surfaces should be chipped with a "chisel-point" hammer, to a roughness of (plus or minus) 3/8" (10 mm). Verify the absence of bruising following ICRI Guideline 03732.
4. Concrete surfaces should be saturated (ponded) with clean water for 24 hours just before grouting.
5. All freestanding water must be removed from the foundation and bolt holes immediately before grouting.
6. Anchor bolt holes must be grouted and sufficiently set before the major portion of the grout is placed.
7. Shade the foundation from sunlight 24 hours before and 24 hours after grouting.

## Technical Data

### Composition

Masterflow® 928 is a hydraulic cement-based mineral-aggregate grout.

### Compliances

- ASTM C 1107, Grades B and C, and CRD 621, Grades B and C, requirements at a fluid consistency over a temperature range of 40 to 90° F (4 to 32° C)
- City of Los Angeles Research Report Number RR 23137

### Test Data

| PROPERTY | RESULTS | | | TEST METHODS |
|---|---|---|---|---|
| **Compressive strengths,** psi (MPa) | | | | ASTM C 942, according to ASTM C 1107 |
| | | **Consistency** | | |
| | **Plastic**[1] | **Flowable**[2] | **Fluid**[3] | |
| 1 day | 4,500 (31) | 4,000 (28) | 3,500 (24) | |
| 3 days | 6,000 (41) | 5,000 (34) | 4,500 (31) | |
| 14 days | 7,500 (52) | 6,700 (46) | 6,500 (45) | |
| 28 days | 9,000 (62) | 8,000 (55) | 7,500 (52) | |
| **Volume change*** | | | | ASTM C 1090 |
| | **% Change** | **% Requirement of ASTM C 1107** | | |
| 1 day | > 0 | 0.0 – 0.30 | | |
| 3 days | 0.04 | 0.0 – 0.30 | | |
| 14 days | 0.05 | 0.0 – 0.30 | | |
| 28 days | 0.06 | 0.0 – 0.30 | | |
| **Setting time,** hr:min | | | | ASTM C 191 |
| | | **Consistency** | | |
| | **Plastic**[1] | **Flowable**[2] | **Fluid**[3] | |
| Initial set | 2:30 | 3:00 | 4:30 | |
| Final set | 4:00 | 5:00 | 6:00 | |
| **Flexural strength,*** psi (MPa) | | | | ASTM C 78 |
| 3 days | 1,000 (6.9) | | | |
| 7 days | 1,050 (7.2) | | | |
| 28 days | 1,150 (7.9) | | | |
| **Modulus of elasticity,*** psi (MPa) | | | | ASTM C 469, modified |
| 3 days | $2.82 \times 10^6$ ($1.94 \times 10^4$) | | | |
| 7 days | $3.02 \times 10^6$ ($2.08 \times 10^4$) | | | |
| 28 days | $3.24 \times 10^6$ ($2.23 \times 10^4$) | | | |
| **Coefficient of thermal expansion,*** in/in/° F (mm/mm/° C) | $6.5 \times 10^{-6}$ ($11.7 \times 10^{-6}$) | | | ASTM C 531 |
| **Split tensile and tensile strength,*** psi (MPa) | | | | ASTM C 496 (splitting tensile) ASTM C 190 (tensile) |
| | **Splitting Tensile** | **Tensile** | | |
| 3 days | 575 (4.0) | 490 (3.4) | | |
| 7 days | 630 (4.3) | 500 (3.4) | | |
| 28 days | 675 (4.7) | 500 (3.4) | | |
| **Punching shear strength,* psi (MPa),** 3 by 3 by 11" (76 by 76 by 279 mm) beam | | | | Degussa Method |
| 3 days | 2,200 (15.2) | | | |
| 7 days | 2,260 (15.6) | | | |
| 28 days | 2,650 (18.3) | | | |
| **Resistance to rapid freezing and thawing** | 300 Cycles RDF 99% | | | ASTM C 666, Procedure A |

[1]100 – 125% flow on flow table per ASTM C 230

[2]125 – 145% flow on flow table per ASTM C 230

[3]25 to 30 seconds through flow cone per ASTM C 939

*Test conducted at a fluid consistency

Test results are averages obtained under laboratory conditions. Expect reasonable variations.

### Test Data, continued

| PROPERTY | | | | RESULTS | TEST METHODS |
|---|---|---|---|---|---|
| **Ultimate tensile strength and bond stress** | | | | | ASTM E 488, tests* |
| **Diameter in (mm)** | **Depth in (mm)** | **Tensile strength lbs (kg)** | **Bond stress psi (MPa)** | | |
| 5/8 (15.9) | 4 (101.6) | 23,500 (10,575) | 2,991 (20.3) | | |
| 3/4 (19.1) | 5 (127.0) | 30,900 (13,905) | 2,623 (18.1) | | |
| 1 (25.4) | 6.75 (171.5) | 65,500 (29,475) | 3,090 (21.3) | | |

*Average of 5 tests in ≥ 4,000 psi (27.6 MPa) concrete using 125 ksi threaded rod in 2" (51mm) diameter, damp, core-drilled holes.

Notes:

1. Grout was mixed to a fluid consistency.
2. Recommended design stress: 2,275 psi (15.7 MPa).
3. Refer to the "Adhesive and Grouted Fastener Capacity Design Guidelines" for more detailed information.
4. Tensile tests with headed fasteners were governed by concrete failure.

### Jobsite Testing

If strength tests must be made at the jobsite, use 2" (51 mm) metal cube molds as specified by ASTM C 942 and ASTM C 1107. DO NOT use cylinder molds. Control field and laboratory tests on the basis of desired placement consistency rather than strictly on water content.

### Forming

1. Forms should be liquid tight and nonabsorbent. Seal forms with putty, sealant, caulk, polyurethane foam.
2. Moderately sized equipment should utilize a head form sloped at 45 degrees to enhance the grout placement. A moveable head box may provide additional head at minimum cost.
3. Side and end forms should be a minimum 1" (25 mm) distant horizontally from the object grouted to permit expulsion of air and any remaining saturation water as the grout is placed.
4. Leave a minimum of 2" between the bearing plate and the form to allow for ease of placement.
5. Use sufficient bracing to prevent the grout from leaking or moving.
6. Eliminate large, nonsupported grout areas wherever possible.
7. Extend forms a minimum of 1" (25 mm) higher than the bottom of the equipment being grouted.
8. Expansion joints may be necessary for both indoor and outdoor installation. Consult your local Degussa field representative for suggestions and recommendations.

### Temperature

1. For precision grouting, store and mix grout to produce the desired mixed-grout temperature. If bagged material is hot, use cold water, and if bagged material is cold, use warm water to achieve a mixed-product temperature as close to 70° F (21°C) as possible.

**Recommended Temperature Guidelines for Precision Grouting**

| | MINIMUM ° F (° C) | PREFERRED ° F (° C) | MAXIMUM ° F (° C) |
|---|---|---|---|
| Foundation and plates | 45 (7) | 50 – 80 (10 – 27) | 90 (32) |
| Mixing water | 45 (7) | 50 – 80 (10 – 27) | 90 (32) |
| Grout at mixed and placed temp | 45 (7) | 50 – 80 (10 – 27) | 90 (32) |

2. If temperature extremes are anticipated or special placement procedures are planned, contact your local Degussa representative for assistance.
3. When grouting at minimum temperatures, see that the foundation, plate, and grout temperatures do not fall below 40° F (7° C) until after final set. Protect the grout from freezing (32° F or 0° C) until it has attained a compressive strength of 3,000 psi (21 MPa).

### Mixing

1. Place estimated water (use potable water only) into the mixer, then slowly add the grout. For a fluid consistency, start with 9 lbs (4 kg) (1.1 gallon [4.2L]) per 55 lb bag.
2. The water demand will depend on mixing efficiency, material, and ambient-temperature conditions. Adjust the water to achieve the desired flow. Recommended flow is 25 – 30 seconds using the ASTM C 939 Flow-Cone Method. Use the minimum amount of water required to achieve the necessary placement consistency.
3. Moderately sized batches of grout are best mixed in one or more clean mortar mixers. For large batches, use ready-mix trucks and 3,300 lb (1,500 kg) bags for maximum efficiency and economy.
4. Mix grout a minimum of 5 minutes after all material and water is in the mixer. Use mechanical mixer only.
5. Do not mix more grout than can be placed in approximately 30 minutes.
6. Transport by wheelbarrow or buckets or pump to the equipment being grouted. Minimize the transporting distance.
7. Do not retemper grout by adding water and remixing after it stiffens.
8. DO NOT VIBRATE GROUT TO FACILITATE PLACEMENT.

9. For aggregate extension guidelines, refer to Appendix MB-10: Guide to Cementitious Grouting.

**Application**

1. Always place grout from only one side of the equipment to prevent air or water entrapment beneath the equipment. Place Masterflow® 928 in a continuous pour. Discard grout that becomes unworkable. Make sure that the material fills the entire space being grouted and that it remains in contact with plate throughout the grouting process.

2. Immediately after placement, trim the surfaces with a trowel and cover the exposed grout with clean wet rags (not burlap). Keep rags moist until grout surface is ready for finishing or until final set.

3. The grout should offer stiff resistance to penetration with a pointed mason's trowel before the grout forms are removed or excessive grout is cut back. After removing the damp rags, immediately coat with a recommended curing compound compliant with ASTM C 309 or preferably ASTM C 1315.

4. Do not vibrate grout. Use steel straps inserted under the plate to help move the grout.

5. Consult your Degussa representative before placing lifts more than 6" (152 mm) in depth.

**Curing**

Cure all exposed grout with an approved membrane curing compound compliant with ASTM C 309 or preferably ASTM C 1315. Apply curing compound immediately after the wet rags are removed to minimize potential moisture loss.

## For Best Performance

- For guidelines on specific anchor-bolt applications, contact Degussa Technical Service.
- Do not add plasticizers, accelerators, retarders, or other additives unless advised in writing by Degussa Technical Service.
- The water requirement may vary with mixing efficiency, temperature, and other variables.
- Hold a pre-job conference with your local representative to plan the installation. Hold conferences as early as possible before the installation of equipment, sole plates, or rail mounts. Conferences are important for applying the recommendations in this product data sheet to a given project, and they help ensure a placement of highest quality and lowest cost.
- The ambient and initial temperature of the grout should be in the range of 45 to 90° F (7 to 32° C) for both mixing and placing. Ideally the amount of mixing water used should be that which is necessary to achieve a 25 – 30 second flow according to ASTM C 939 (CRD C 611). For placement outside of the 45 to 90° F (7 to 32° C) range, contact your local Degussa representative.
- For pours greater than 6" (152 mm) deep, consult your local Degussa representative for special precautions and installation procedures.
- Use Embeco® 885 grout for dynamic load-bearing support and similar application conditions as Masterflow® 928.
- Use Masterflow® 816, Masterflow® 1205, or Masterflow® 1341 post-tensioning cable grouts when the grout will be in contact with steel stressed over 80,000 psi (552 MPa).
- Masterflow® 928 is not intended for use as a floor topping or in large areas with exposed shoulders around baseplates. Where grout has exposed shoulders, occasional hairline cracks may occur. Cracks may also occur near sharp corners of the baseplate and at anchor bolts. These superficial cracks are usually caused by temperature and moisture changes that affect the grout at exposed shoulders at a faster rate than the grout beneath the baseplate. They do not affect the structural, nonshrink, or vertical support provided by the grout if the foundation-preparation, placing, and curing procedures are properly carried out.
- The minimum placement depth is 1" (25 mm).
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current version.
- Proper application is the responsibility of the user. Field visits by Degussa personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

## Health and Safety

MASTERFLOW® 928

**Caution**

**Risks**

Eye irritant. Skin irritant. Causes burns. Lung irritant. May cause delayed lung injury.

**Precautions**

KEEP OUT OF THE REACH OF CHILDREN. Avoid contact with eyes. Wear suitable protective eyewear. Avoid prolonged or repeated contact with skin. Wear suitable gloves. Wear suitable protective clothing. Do not breathe dust. In case of insufficient ventilation, wear suitable respiratory equipment. Wash soiled clothing before reuse.

**First Aid**

Wash exposed skin with soap and water. Flush eyes with large quantities of water. If breathing is difficult, move person to fresh air.

**Waste Disposal Method**

This product when discarded or disposed of, is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

**Proposition 65**

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

**VOC Content**

0 lbs/gal or 0 g/L.

**For medical emergencies only, call ChemTrec (1-800-424-9300).**

**Degussa Building Systems**

889 Valley Park Drive
Shakopee, MN, 55379

www.degussabuildingsystems.com

**Customer Service** 800-433-9517
**Technical Service** 800-243-6739



Form No. 1019303 9/03 (Replaces 1/02)
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Construction Chemicals

# SET® 45 AND SET® 45 HW

Chemical-action repair mortar

PRODUCT DATA

3 03930 **Concrete Rehabilitation**

## Description

Set® 45 is a one-component magnesium phosphate-based patching and repair mortar. This concrete repair and anchoring material sets in approximately 15 minutes and takes rubber-tire traffic in 45 minutes. It comes in two formulations: Set® 45 Regular for ambient temperatures below 85° F (29° C) and Set® 45 Hot Weather for ambient temperatures ranging from 85 to 100° F (29 to 38° C).

### Yield

A 50 lb (22.7 kg) bag of mixed with the required amount of water produces a volume of approximately 0.39 ft³ (0.011 m³); 60% extension using 1/2" (13 mm) rounded, sound aggregate produces approximately 0.58 ft³ (0.016 m³).

### Packaging

50 lb (22.7 kg) multi-wall bags

### Color

Dries to a natural gray color

### Shelf Life

1 year when properly stored

### Storage

Store in unopened containers in a clean, dry area between 45 and 90° F (7 and 32° C).

Protection and Repair

| Features | Benefits |
|---|---|
| • Single component | Just add water and mix |
| • Reaches 2,000 psi compressive strength in 1 hour | Rapidly returns repairs to service |
| • Wide temperature use range | From below freezing to hot weather exposures |
| • Superior bonding | Bonds to concrete and masonry without a bonding agent |
| • Very low drying shrinkage | Improved bond to surrounding concrete |
| • Resistant to freeze/thaw cycles and deicing chemicals | Usable in most environments |
| • Only air curing required | Fast, simple curing process |
| • Thermal expansion and contraction similar to Portland cement concrete | More permanent repairs |
| • Sulfate resistant | Stable where conventional mortars degrade |

## Where to Use

APPLICATION

- Heavy industrial repairs
- Dowel bar replacement
- Concrete pavement joint repairs
- Full-depth structural repairs
- Setting of expansion device nosings
- Bridge deck and highway overlays
- Anchoring iron or steel bridge and balcony railings
- Commercial freezer rooms
- Truck docks
- Parking decks and ramps
- Airport runway-light installations

LOCATION

- Horizontal and formed vertical or overhead surfaces
- Indoor and outdoor applications

## How to Apply

### Surface Preparation

1. A sound substrate is essential for good repairs. Flush the area with clean water to remove all dust.

2. Any surface carbonation in the repair area will inhibit chemical bonding. Apply a pH indicator to the prepared surface to test for carbonation.

3. Air blast with oil-free compressed air to remove all water before placing Set® 45.

### Mixing

1. Set® 45 must be mixed, placed, and finished within 10 minutes in normal temperatures (72° F [22° C]). Only mix quantities that can be placed in 10 minutes or less.

2. Do not deviate from the following sequence; it is important for reducing mixing time and producing a consistent mix. Use a minimum 1/2" slow-speed drill and mixing paddle or an appropriately sized mortar mixer. Do not mix by hand.

3. Pour clean (potable) water into mixer. Water content is critical. Use a maximum of 4 pts (1.9 L) of water per 50 lb (22.7 kg) bag of Set® 45. Do not deviate from the recommended water content.

## Technical Data

### Composition

Set® 45 is a magnesium-phosphate patching and repair mortar.

### Test Data

| PROPERTY | RESULTS | TEST METHODS |
|---|---|---|
| **Typical Compressive Strengths*,** psi (MPa) | | ASTM C 109, modified |

| | **Plain Concrete 72° F (22° C)** | **Set® 45 Regular 72° F (22° C)** | **Set® 45 Regular 36° F (2° C)** | **Set® 45 HW 95° F (35° C)** |
|---|---|---|---|---|
| 1 hour | — | 2,000 (13.8) | — | — |
| 3 hour | — | 5,000 (34.5) | — | 3,000 (20.7) |
| 6 hour | — | 5,000 (34.5) | 1,200 (8.3) | 5,000 (34.5) |
| 1 day | 500 (3.5) | 6,000 (41.4) | 5,000 (34.5) | 6,000 (41.4) |
| 3 day | 1,900 (13.1) | 7,000 (48.3) | 7,000 (48.3) | 7,000 (48.3) |
| 28 day | 4,000 (27.6) | 8,500 (58.6) | 8,500 (58.6) | 8,500 (55.2) |

NOTE: Only Set® 45 Regular formula, tested at 72° F (22° C), obtains 2,000 psi (13.8 MPa) compressive strength in 1 hour.

| **Modulus of Elasticity,** psi (MPa) | **7 days** | **28 days** | ASTM C 469 |
|---|---|---|---|
| Set® 45 Regular | $4.18 \times 10^6$ ($2.88 \times 10^4$) | $4.55 \times 10^6$ ($3.14 \times 10^4$) | |
| Set® 45 Hot Weather | $4.90 \times 10^6$ ($3.38 \times 10^4$) | $5.25 \times 10^6$ ($3.62 \times 10^4$) | |

| PROPERTY | RESULTS | TEST METHODS |
|---|---|---|
| **Freeze/thaw durability test,** % RDM, 300 cycles, for Set® 45 and Set 45® HW | 80 | ASTM C 666, Procedure A (modified**) |
| **Scaling resistance to deicing chemicals,** Set® 45 and Set 45® HW | | ASTM C 672 |
| 5 cycles | 0 | |
| 25 cycles | 0 | |
| 50 cycles | 1.5 (slight scaling) | |
| **Sulfate resistance** | | ASTM C 1012 |
| Set® 45 length change after 52 weeks, % | 0.09 | |
| Type V cement mortar after 52 weeks, % | 0.20 | |
| **Typical setting times,** min, for Set® 45 at 72° F (22° C), and Set® 45 Hot Weather at 95° F (35° C) | | Gilmore ASTM C 266, modified |
| Initial set | 9 – 15 | |
| Final set | 10 – 20 | |
| **Coefficient of thermal expansion,***** both Set® 45 Regular and Set® 45 Hot Weather coefficients | $7.15 \times 10^{-6}$/° F ($12.8 \times 10^{-6}$/° C) | CRD-C 39 |
| **Flexural Strength,** psi (MPa), 3 by 4 by 16" (75 by 100 by 406 mm) prisms, 1 day strength, | | ASTM C 78, modified |
| Set® 45 mortar | 550 (3.8) | |
| Set® 45 mortar with 3/8" (9 mm) pea gravel | 600 (4.2) | |
| Set® 45 mortar with 3/8" (9 mm) crushed angular noncalcareous hard aggregate | 650 (4.5) | |

* All tests were performed with neat material (no aggregate)

**Method discontinues test when 300 cycles or an RDM of 60% is reached.

***Determined using 1 by 1 by 11" (25 mm by 25 mm by 279 mm) bars. Test was run with neat mixes (no aggregate). Extended mixes (with aggregate) produce lower coefficients of thermal expansion.

Test results are averages obtained under laboratory conditions. Expect reasonable variations.

4. Add the powder to the water and mix for approximately 1 – 1-1/2 minutes.

5. Use neat material for patches from 1/2 – 2" (6 – 51 mm) in depth or width. For deeper patches, extend a 50 lb (22.7 kg) bag of Set® 45 HW by adding up to 30 lbs (13.6 kg) of properly graded, dust-free, hard, rounded aggregate or noncalcareous crushed angular aggregate, not exceeding 1/2" (6 mm) in accordance with ASTM C 33, #8. If aggregate is damp, reduce water content accordingly. Special procedures must be followed when angular aggregate is used. Contact your local Degussa representative for more information. (Do not use calcareous aggregate made from soft limestone. Test aggregate for fizzing with 10% HCL).

**Application**

1. Immediately place the mixture onto the properly prepared substrate. Work the material firmly into the bottom and sides of the patch to ensure good bond.

2. Level the Set® 45 and screed to the elevation of the existing concrete. Minimal finishing is required. Match the existing concrete texture.

**Curing**

No curing is required, but protect from rain immediately after placing. Liquid-membrane curing compounds or plastic sheeting may be used to protect the early surface from precipitation, but never wet cure Set® 45.

## For Best Performance

- Color variations are not indicators of abnormal product performance.
- Regular Set® 45 will not freeze at temperatures above -20° F (-29° C) when appropriate precautions are taken.
- Do not add sand, fine aggregate, or Portland cement to Set® 45.
- Do not use Set® 45 for patches less than 1/2" (13 mm) deep. For deep patches, use Set® 45 Hot Weather formula extended with aggregate, regardless of the temperature. Consult your Degussa representative for further instructions.
- Do not use limestone aggregate.
- Water content is critical. Do not deviate from the recommended water content printed on the bag.
- Precondition these materials to approximately 70° F (21° C) for 24 hours before using.
- Protect repairs from direct sunlight, wind, and other conditions that could cause rapid drying of material.
- When mixing or placing Set® 45 in a closed area, provide adequate ventilation.
- Do not use Set® 45 as a precision nonshrink grout.
- Never featheredge Set® 45; for best results, always sawcut the edges of a patch.
- Prevent any moisture loss during the first 3 hours after placement. Protect Set® 45 with plastic sheeting or a curing compound in rapid-evaporation conditions.
- Do not wet cure.
- Do not place Set® 45 on a hot (90° F [32° C]), dry substrate.
- When using Set® 45 in contact with galvanized steel or aluminum, consult your local Degussa sales representative.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current versions.
- Proper application is the responsibility of the user. Field visits by Degussa personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

## Health and Safety

SET® 45

**Caution**

**Risks**

Eye irritant. Skin irritant. Lung irritant. May cause delayed lung injury.

**Precautions**

KEEP OUT OF THE REACH OF CHILDREN. Avoid contact with eyes. Wear suitable protective eyewear. Avoid prolonged or repeated contact with skin. Wear suitable gloves. Wear suitable protective clothing. Do not breathe dust. In case of insufficient ventilation, wear suitable respiratory equipment. Wash soiled clothing before reuse.

**First Aid**

Wash exposed skin with soap and water. Flush eyes with large quantities of water. If breathing is difficult, move person to fresh air.

**Waste Disposal Method**

This product when discarded or disposed of is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

**Proposition 65**

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

**VOC Content**

0 lbs/gal or 0 g/L.

**For medical emergencies only, call ChemTrec (1-800-424-9300).**

# APPENDIX B
# INSTRUMENTATION ATTACHEMENT METHOD

This appendix contains details about the method used to attach the top and bottom sets of instruments. The lower instruments went below ground and had to be sealed and protected from damage by soil and water. The following figures show the indentions provided by Standard Concrete, the groove that was cut to mount the wire flush, the plates bolted on, and the top set of instruments.

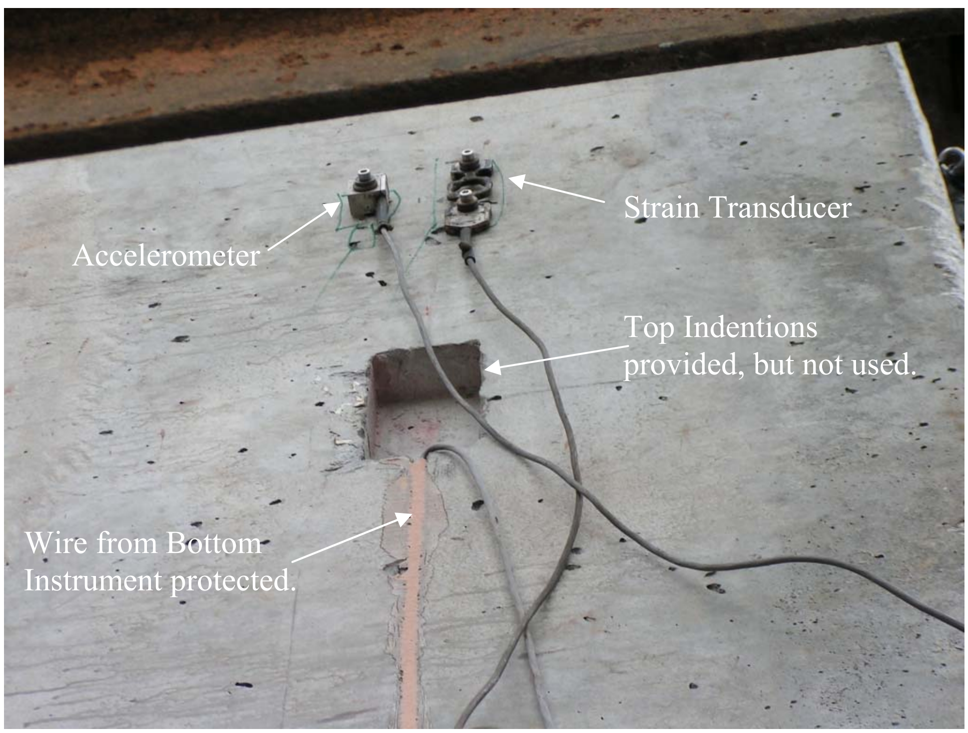



Figure B-1 Top set of instruments; accelerometer on left side and strain transducer on right side.

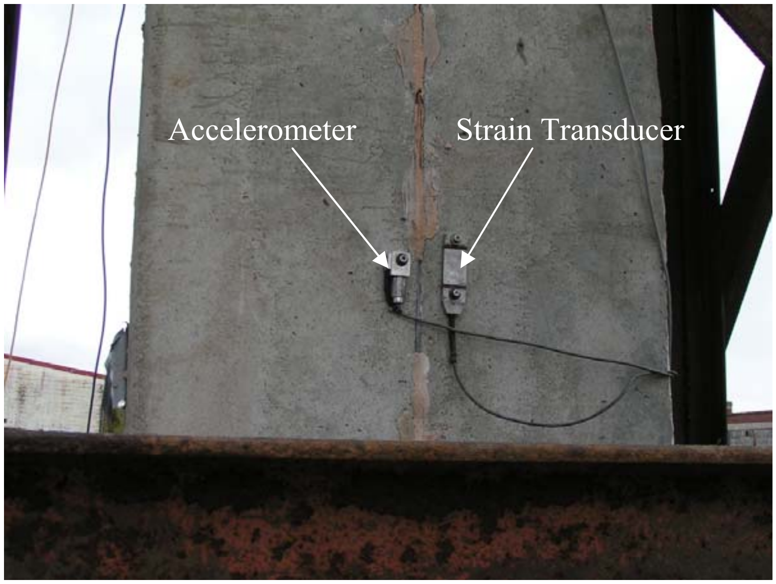
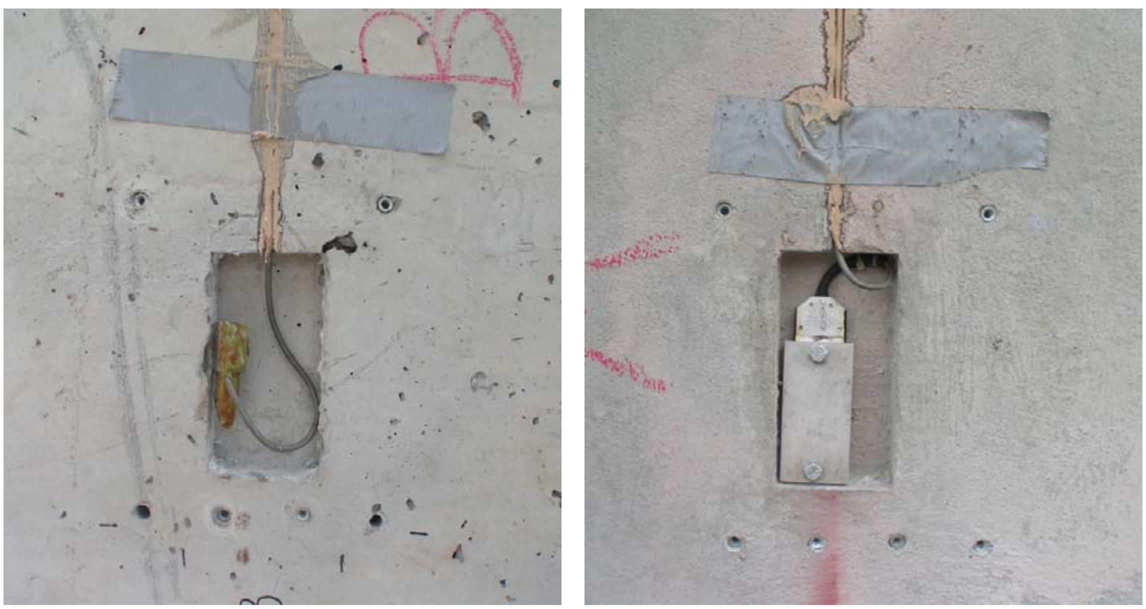



Figure B-2 Middle set of instruments, accelerometer on left side and strain transducer on right side.



A B

Figure B-3 Bottom set of instruments with concrete anchor sleeves installed, A) accelerometer ready, B) strain transducer with casing ready.


Figure B-4 Bottom set of instruments, with steel cover plates attached on Pile #2; Pile #1 driven to cutoff elevation with tip at -14 feet.

APPENDIX C
PDA OUTPUT FROM PILE DRIVING

This appendix contains the PDA output for each pile in Tabular Form. The software PDIPLOT was used to create the tables.

The PDA results presented in the tables below inlcude:

- FMX Max COMPRESSIVE FORCE at sensors (MEX - Max STRAIN)
- CTN Max TENSION FORCE at or below sensors (1ST 2L/C only)
- CTX Max TENSION FORCE (UP 1ST 2L/C, or DOWN TENSION later)
- TSX* Max TENSION STRESS below sensors (CTX/AREA); TSN=CTN/AR
- CSX* Max average axial COMPRESSION STRESS at gage (FMX/AREA)
- CSI* Max INDIVIDUAL COMPRESSION STRESS for either transducer
- EMX* ENERGY TRANSFERRED to pile - (most important measure)
- ETR ENERGY TRANSFER RATIO (EMX/ER) (must input "ER" RATING)
- VMX Max VELOCITY at sensors

Applied Foundation Testing, Inc. — Page 1 of 3
Case Method Results — PDIPLOT Ver. 2005.1 - Printed: 28-Apr-2005

FDOT SPLICE RESEARCH - TP-1ARS

Test date: 17-Sep-2004

AR: 645.53 in^2 — SP: 0.151 k/ft3
LE: 34.00 ft — EM: 5,672 ksi
WS: 13,200.0 f/s — JC: 0.50

FMX: Maximum Force
RMX: Max Case Method Capacity
CSI: Max F1 or F2 Compr. Stress
CSX: Max Measured Compr. Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio
CTN: Max Computed Tension
CTX: Max Computed Tension
TSX: Tension Stress Maximum

| BL# | depth | BLC | FMX | RMX | CSI | CSX | EMX | ETR | CTN | CTX | TSX |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | bl/ft | kips | kips | ksi | ksi | k-ft | (%) | kips | kips | ksi |
| 1 | 12.25 | 4 | 635 | 198 | 1.2 | 1.0 | 15.1 | 2,044.7 | 0 | -65 | 0.1 |
| 6 | 13.11 | 18 | 884 | 209 | 1.6 | 1.4 | 16.1 | 2,176.2 | 0 | -65 | 0.1 |
| 11 | 13.39 | 18 | 763 | 180 | 1.4 | 1.2 | 12.2 | 1,651.4 | 0 | -65 | 0.1 |
| 16 | 13.67 | 18 | 935 | 199 | 1.7 | 1.4 | 16.5 | 2,233.4 | 0 | -65 | 0.1 |
| 21 | 13.94 | 18 | 877 | 185 | 1.5 | 1.4 | 14.2 | 1,927.3 | 0 | -22 | 0.0 |
| 26 | 14.15 | 26 | 908 | 199 | 1.7 | 1.4 | 15.9 | 2,162.1 | 0 | -20 | 0.0 |
| 31 | 14.35 | 26 | 882 | 192 | 1.6 | 1.4 | 14.6 | 1,977.1 | 0 | -20 | 0.0 |
| 36 | 14.54 | 26 | 929 | 202 | 1.7 | 1.4 | 16.0 | 2,172.4 | 0 | -11 | 0.0 |
| 41 | 14.73 | 26 | 848 | 176 | 1.5 | 1.3 | 13.4 | 1,810.3 | 0 | -17 | 0.0 |
| 46 | 14.92 | 26 | 1,020 | 212 | 1.8 | 1.6 | 17.9 | 2,431.5 | 0 | -36 | 0.1 |
| 51 | 15.05 | 56 | 1,055 | 186 | 1.7 | 1.6 | 18.0 | 2,445.9 | 0 | -24 | 0.0 |
| 56 | 15.14 | 56 | 1,028 | 216 | 1.9 | 1.6 | 18.3 | 2,482.7 | 0 | -24 | 0.0 |
| 61 | 15.23 | 56 | 1,057 | 203 | 1.7 | 1.6 | 18.0 | 2,437.6 | 0 | -27 | 0.0 |
| 66 | 15.32 | 56 | 1,022 | 190 | 1.7 | 1.6 | 17.3 | 2,350.2 | 0 | -17 | 0.0 |
| 71 | 15.41 | 56 | 1,137 | 194 | 1.9 | 1.8 | 19.9 | 2,694.5 | 0 | -37 | 0.1 |
| 76 | 15.50 | 56 | 1,143 | 188 | 1.9 | 1.8 | 19.9 | 2,694.7 | 0 | -38 | 0.1 |
| 81 | 15.59 | 56 | 1,116 | 202 | 1.9 | 1.7 | 19.1 | 2,595.6 | 0 | -31 | 0.0 |
| 86 | 15.68 | 56 | 1,014 | 191 | 1.7 | 1.6 | 16.7 | 2,266.6 | 0 | -25 | 0.0 |
| 91 | 15.77 | 56 | 1,137 | 182 | 1.9 | 1.8 | 19.6 | 2,653.1 | 0 | -39 | 0.1 |
| 96 | 15.86 | 56 | 1,061 | 187 | 1.8 | 1.6 | 18.1 | 2,457.7 | 0 | -37 | 0.1 |
| 101 | 15.95 | 56 | 1,119 | 187 | 1.9 | 1.7 | 19.7 | 2,669.0 | 0 | -41 | 0.1 |
| 106 | 16.33 | 6 | 1,030 | 189 | 1.8 | 1.6 | 17.4 | 2,365.2 | 0 | -36 | 0.1 |
| 111 | 17.08 | 13 | 1,142 | 190 | 2.0 | 1.8 | 20.4 | 2,767.2 | 0 | -50 | 0.1 |
| 116 | 17.46 | 13 | 1,040 | 196 | 1.8 | 1.6 | 17.9 | 2,429.6 | 0 | -40 | 0.1 |
| 121 | 17.85 | 13 | 1,020 | 201 | 1.8 | 1.6 | 18.0 | 2,435.7 | 0 | -43 | 0.1 |
| 126 | 18.14 | 22 | 1,014 | 216 | 1.9 | 1.6 | 18.0 | 2,437.4 | 0 | -37 | 0.1 |
| 131 | 18.36 | 22 | 1,097 | 196 | 1.9 | 1.7 | 18.6 | 2,523.6 | 0 | -49 | 0.1 |
| 136 | 18.59 | 22 | 527 | 116 | 0.9 | 0.8 | 7.4 | 996.8 | 0 | -39 | 0.1 |
| 141 | 18.82 | 22 | 1,297 | 225 | 2.3 | 2.0 | 24.0 | 3,248.7 | 0 | -47 | 0.1 |
| 146 | 19.09 | 11 | 1,213 | 154 | 2.1 | 1.9 | 20.4 | 2,759.7 | 0 | 5 | 0.0 |
| 151 | 19.55 | 11 | 1,171 | 221 | 2.0 | 1.8 | 22.1 | 3,002.0 | 0 | -29 | 0.0 |
| 156 | 20.00 | 11 | 1,306 | 220 | 2.1 | 2.0 | 24.1 | 3,273.8 | 0 | -46 | 0.1 |
| 161 | 20.20 | 25 | 1,112 | 235 | 1.7 | 1.7 | 21.6 | 2,923.5 | 0 | -10 | 0.0 |
| 166 | 20.40 | 25 | 1,267 | 231 | 2.0 | 2.0 | 22.7 | 3,079.7 | 0 | -34 | 0.1 |
| 171 | 20.60 | 25 | 1,278 | 227 | 2.1 | 2.0 | 22.8 | 3,096.9 | 0 | -41 | 0.1 |
| 176 | 20.80 | 25 | 1,231 | 203 | 2.0 | 1.9 | 21.6 | 2,923.8 | 0 | -36 | 0.1 |
| 181 | 21.00 | 25 | 1,318 | 215 | 2.2 | 2.0 | 23.5 | 3,182.6 | 0 | -38 | 0.1 |
| 186 | 21.22 | 23 | 1,287 | 210 | 2.2 | 2.0 | 23.0 | 3,115.2 | 0 | -39 | 0.1 |

FDOT SPLICE RESEARCH - TP-1ARS

Test date: 17-Sep-2004

| BL# | depth | BLC | FMX | RMX | CSI | CSX | EMX | ETR | CTN | CTX | TSX |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | bl/ft | kips | kips | ksi | ksi | k-ft | (%) | kips | kips | ksi |
| 191 | 21.43 | 23 | 1,232 | 206 | 2.1 | 1.9 | 21.7 | 2,947.2 | 0 | -39 | 0.1 |
| 196 | 21.65 | 23 | 1,343 | 205 | 2.3 | 2.1 | 23.9 | 3,247.0 | 0 | -37 | 0.1 |
| 201 | 21.87 | 23 | 1,292 | 200 | 2.2 | 2.0 | 23.4 | 3,173.8 | 0 | -37 | 0.1 |
| 206 | 22.09 | 23 | 1,296 | 208 | 2.3 | 2.0 | 24.1 | 3,264.3 | 0 | -41 | 0.1 |
| 211 | 22.30 | 23 | 1,305 | 204 | 2.3 | 2.0 | 24.1 | 3,269.4 | 0 | -38 | 0.1 |
| 216 | 22.52 | 23 | 1,227 | 203 | 2.2 | 1.9 | 22.3 | 3,025.1 | 0 | -36 | 0.1 |
| 221 | 22.74 | 23 | 1,241 | 207 | 2.2 | 1.9 | 22.5 | 3,055.7 | 0 | -41 | 0.1 |
| 226 | 22.96 | 23 | 1,236 | 192 | 2.2 | 1.9 | 22.2 | 3,014.5 | 0 | -39 | 0.1 |
| 231 | 23.17 | 24 | 1,296 | 196 | 2.3 | 2.0 | 23.6 | 3,193.7 | 0 | -45 | 0.1 |
| 236 | 23.38 | 24 | 1,273 | 211 | 2.3 | 2.0 | 23.2 | 3,139.9 | 0 | -40 | 0.1 |
| 241 | 23.58 | 24 | 1,331 | 205 | 2.3 | 2.1 | 24.4 | 3,307.8 | 0 | -42 | 0.1 |
| 246 | 23.79 | 24 | 1,296 | 219 | 2.3 | 2.0 | 23.8 | 3,220.8 | 0 | -43 | 0.1 |
| 251 | 24.00 | 24 | 1,403 | 223 | 2.3 | 2.2 | 26.2 | 3,552.9 | 0 | -50 | 0.1 |
| 256 | 24.38 | 13 | 1,321 | 226 | 2.3 | 2.0 | 23.8 | 3,232.2 | 0 | -39 | 0.1 |
| 261 | 24.77 | 13 | 1,263 | 203 | 2.2 | 2.0 | 22.3 | 3,027.8 | 0 | -44 | 0.1 |
| 266 | 25.18 | 11 | 1,214 | 200 | 2.1 | 1.9 | 21.2 | 2,875.2 | 0 | -40 | 0.1 |
| 271 | 25.64 | 11 | 1,318 | 220 | 2.3 | 2.0 | 23.1 | 3,127.2 | 0 | -36 | 0.1 |
| 276 | 26.05 | 21 | 1,467 | 230 | 2.5 | 2.3 | 27.0 | 3,654.2 | -45 | -46 | 0.1 |
| 281 | 26.29 | 21 | 1,386 | 217 | 2.4 | 2.1 | 24.9 | 3,375.4 | -8 | -38 | 0.1 |
| 286 | 26.52 | 21 | 1,346 | 214 | 2.3 | 2.1 | 23.7 | 3,215.9 | 0 | -35 | 0.1 |
| 291 | 26.76 | 21 | 1,318 | 203 | 2.2 | 2.0 | 23.0 | 3,123.0 | 0 | -33 | 0.1 |
| 296 | 27.00 | 21 | 1,296 | 204 | 2.2 | 2.0 | 22.5 | 3,051.8 | 0 | -31 | 0.0 |
| 301 | 27.22 | 23 | 1,354 | 204 | 2.3 | 2.1 | 23.8 | 3,223.0 | -2 | -34 | 0.1 |
| 306 | 27.43 | 23 | 1,354 | 205 | 2.3 | 2.1 | 23.9 | 3,234.7 | 0 | -34 | 0.1 |
| 311 | 27.65 | 23 | 1,378 | 204 | 2.3 | 2.1 | 24.5 | 3,322.0 | -2 | -40 | 0.1 |
| 316 | 27.87 | 23 | 1,309 | 196 | 2.2 | 2.0 | 22.8 | 3,092.9 | 0 | -36 | 0.1 |
| 321 | 28.09 | 23 | 1,347 | 197 | 2.3 | 2.1 | 23.4 | 3,175.4 | 0 | -36 | 0.1 |
| 326 | 28.30 | 23 | 1,325 | 197 | 2.3 | 2.1 | 23.1 | 3,133.1 | 0 | -41 | 0.1 |
| 331 | 28.52 | 23 | 1,388 | 195 | 2.3 | 2.2 | 24.5 | 3,315.6 | -10 | -44 | 0.1 |
| 336 | 28.74 | 23 | 1,346 | 177 | 2.2 | 2.1 | 23.0 | 3,116.6 | -1 | -44 | 0.1 |
| 341 | 28.96 | 23 | 1,319 | 189 | 2.2 | 2.0 | 22.7 | 3,079.4 | 0 | -37 | 0.1 |
| 346 | 29.22 | 18 | 1,374 | 192 | 2.4 | 2.1 | 23.9 | 3,242.0 | -4 | -44 | 0.1 |
| 351 | 29.50 | 18 | 1,382 | 198 | 2.3 | 2.1 | 24.0 | 3,247.3 | -12 | -42 | 0.1 |
| 356 | 29.78 | 18 | 1,420 | 207 | 2.2 | 2.2 | 24.0 | 3,254.3 | -8 | -56 | 0.1 |
| 361 | 30.04 | 25 | 1,436 | 214 | 2.4 | 2.2 | 24.2 | 3,277.0 | -9 | -53 | 0.1 |
| 366 | 30.24 | 25 | 1,385 | 214 | 2.3 | 2.1 | 22.8 | 3,094.6 | 0 | -49 | 0.1 |
| 371 | 30.44 | 25 | 1,365 | 222 | 2.3 | 2.1 | 22.2 | 3,012.6 | 0 | -48 | 0.1 |
| 376 | 30.64 | 25 | 1,390 | 214 | 2.3 | 2.2 | 22.8 | 3,093.1 | -1 | -42 | 0.1 |
| 381 | 30.84 | 25 | 1,367 | 210 | 2.2 | 2.1 | 22.2 | 3,016.6 | 0 | -43 | 0.1 |
| 386 | 31.06 | 18 | 1,368 | 211 | 2.3 | 2.1 | 22.3 | 3,025.4 | -9 | -51 | 0.1 |
| 391 | 31.33 | 18 | 1,339 | 205 | 2.2 | 2.1 | 21.8 | 2,950.5 | -4 | -56 | 0.1 |
| 396 | 31.61 | 18 | 1,361 | 207 | 2.2 | 2.1 | 22.1 | 3,001.0 | -9 | -54 | 0.1 |
| 401 | 31.89 | 18 | 1,352 | 209 | 2.2 | 2.1 | 22.2 | 3,006.2 | -8 | -49 | 0.1 |

Applied Foundation Testing, Inc. Page 3 of 3
Case Method Results PDIPLOT Ver. 2005.1 - Printed: 28-Apr-2005

FDOT SPLICE RESEARCH - TP-1ARS

Test date: 17-Sep-2004

| | FMX | RMX | CSI | CSX | EMX | ETR | CTN | CTX | TSX |
|---|---|---|---|---|---|---|---|---|---|
| | kips | kips | ksi | ksi | k-ft | (%) | kips | kips | ksi |
| Average | 1,209 | 202 | 2.1 | 1.9 | 21.2 | 2,872.8 | -2 | -39 | 0.1 |
| Std. Dev. | 181 | 22 | 0.3 | 0.3 | 3.7 | 501.7 | 7 | 11 | 0.0 |
| Maximum | 1,532 | 256 | 2.6 | 2.4 | 31.6 | 4,290.6 | 0 | 5 | 0.1 |
| @ Blow# | 57 | 47 | 252 | 57 | 134 | 134 | 1 | 146 | 1 |

Total number of blows analyzed: 403

Time Summary

| | | |
|---|---|---|
| Drive | 2 minutes 47 seconds | 4:03:28 PM - 4:06:15 PM (9/17/2004) |
| Stop | 29 minutes 8 seconds | 4:06:15 PM - 4:35:23 PM |
| Drive | 11 seconds | 4:35:23 PM - 4:35:34 PM |
| Stop | 37 minutes 47 seconds | 4:35:34 PM - 5:13:21 PM |
| Drive | 39 minutes 18 seconds | 5:13:21 PM - 5:52:39 PM |

Total time [1:49:11] = (Driving [0:42:16] + Stop [1:06:55])

Applied Foundation Testing, Inc. — Page 1 of 3
Case Method Results — PDIPLOT Ver. 2005.1 - Printed: 28-Apr-2005

FDOT SPLICE RESEARCH - TP-1RS2

Test date: 21-Sep-2004

AR: 645.53 in^2 — SP: 0.151 k/ft3
LE: 34.00 ft — EM: 5,506 ksi
WS: 13,000.0 f/s — JC: 0.50

FMX: Maximum Force — ETR: Energy Transfer Ratio
RMX: Max Case Method Capacity — CTN: Max Computed Tension
CSI: Max F1 or F2 Compr. Stress — TSN: Max Tension Stress - 1st 2L/c only
CSX: Max Measured Compr. Stress — TRP: Time from rise to peak
EMX: Max Transferred Energy

| BL# | depth | BLC | FMX | RMX | CSI | CSX | EMX | ETR | CTN | TSN | TRP |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | bl/ft | kips | kips | ksi | ksi | k-ft | (%) | kips | ksi | ms |
| 1 | 1.31 | 1 | 897 | 174 | 1.4 | 1.4 | 11.6 | 1,577.3 | 0 | 0.0 | 5.20 |
| 3 | 3.92 | 1 | 1,214 | 236 | 1.9 | 1.9 | 19.8 | 2,678.8 | 0 | 0.0 | 5.40 |
| 5 | 6.54 | 1 | 1,217 | 241 | 2.2 | 1.9 | 19.3 | 2,618.6 | 0 | 0.0 | 5.00 |
| 7 | 9.15 | 1 | 1,315 | 232 | 2.2 | 2.0 | 22.2 | 3,009.9 | 0 | 0.0 | 4.60 |
| 9 | 11.77 | 1 | 1,276 | 243 | 2.2 | 2.0 | 20.7 | 2,801.2 | 0 | 0.0 | 5.00 |
| 11 | 14.38 | 1 | 1,264 | 237 | 2.1 | 2.0 | 20.6 | 2,786.7 | 0 | 0.0 | 4.80 |
| 13 | 17.00 | 1 | 1,256 | 238 | 2.1 | 1.9 | 20.5 | 2,777.0 | 0 | 0.0 | 5.00 |
| 15 | 19.62 | 1 | 1,416 | 236 | 2.2 | 2.2 | 24.2 | 3,275.8 | -86 | 0.1 | 4.60 |
| 17 | 22.23 | 1 | 1,447 | 237 | 2.3 | 2.2 | 25.1 | 3,404.6 | -100 | 0.2 | 4.60 |
| 19 | 24.85 | 1 | 1,410 | 236 | 2.2 | 2.2 | 23.8 | 3,227.1 | -78 | 0.1 | 4.60 |
| 21 | 27.46 | 1 | 1,459 | 246 | 2.3 | 2.3 | 25.2 | 3,419.0 | -138 | 0.2 | 4.60 |
| 23 | 30.08 | 1 | 1,416 | 239 | 2.2 | 2.2 | 23.5 | 3,190.1 | -83 | 0.1 | 4.60 |
| 25 | 32.69 | 1 | 1,401 | 233 | 2.2 | 2.2 | 23.3 | 3,160.9 | -83 | 0.1 | 4.60 |
| 27 | 34.03 | 34 | 1,407 | 238 | 2.2 | 2.2 | 23.8 | 3,225.7 | -87 | 0.1 | 4.80 |
| 29 | 34.09 | 34 | 1,451 | 234 | 2.3 | 2.2 | 24.8 | 3,363.8 | -112 | 0.2 | 4.60 |
| 31 | 34.15 | 34 | 1,388 | 235 | 2.2 | 2.2 | 23.1 | 3,135.5 | -67 | 0.1 | 4.80 |
| 33 | 34.21 | 34 | 1,462 | 248 | 2.3 | 2.3 | 25.2 | 3,414.8 | -114 | 0.2 | 4.60 |
| 35 | 34.26 | 34 | 1,510 | 256 | 2.4 | 2.3 | 26.6 | 3,613.0 | -177 | 0.3 | 4.40 |
| 37 | 34.32 | 34 | 1,441 | 247 | 2.3 | 2.2 | 24.6 | 3,334.4 | -108 | 0.2 | 4.40 |
| 39 | 34.38 | 34 | 1,517 | 268 | 2.4 | 2.4 | 26.7 | 3,625.5 | -162 | 0.3 | 4.40 |
| 41 | 34.44 | 34 | 1,470 | 273 | 2.3 | 2.3 | 25.4 | 3,442.2 | -108 | 0.2 | 4.40 |
| 43 | 34.50 | 34 | 1,430 | 277 | 2.3 | 2.2 | 24.1 | 3,271.0 | -66 | 0.1 | 4.60 |
| 45 | 34.56 | 34 | 1,415 | 277 | 2.2 | 2.2 | 24.0 | 3,249.3 | -55 | 0.1 | 4.80 |
| 47 | 34.62 | 34 | 1,494 | 273 | 2.4 | 2.3 | 26.4 | 3,574.9 | -130 | 0.2 | 4.60 |
| 49 | 34.68 | 34 | 1,522 | 277 | 2.4 | 2.4 | 27.4 | 3,709.3 | -138 | 0.2 | 4.40 |
| 51 | 34.74 | 34 | 1,473 | 286 | 2.3 | 2.3 | 25.8 | 3,504.2 | -107 | 0.2 | 4.60 |
| 53 | 34.79 | 34 | 1,512 | 295 | 2.4 | 2.3 | 26.8 | 3,638.7 | -131 | 0.2 | 4.60 |
| 55 | 34.85 | 34 | 1,419 | 298 | 2.3 | 2.2 | 24.2 | 3,277.7 | -52 | 0.1 | 4.80 |
| 57 | 34.91 | 34 | 1,473 | 300 | 2.3 | 2.3 | 25.7 | 3,490.0 | -104 | 0.2 | 4.60 |
| 59 | 34.97 | 34 | 1,533 | 299 | 2.4 | 2.4 | 27.5 | 3,725.2 | -148 | 0.2 | 4.40 |
| 61 | 35.03 | 29 | 1,421 | 303 | 2.3 | 2.2 | 24.0 | 3,251.4 | -78 | 0.1 | 4.80 |
| 63 | 35.10 | 29 | 1,394 | 308 | 2.2 | 2.2 | 23.4 | 3,171.3 | -61 | 0.1 | 4.80 |
| 65 | 35.17 | 29 | 1,429 | 297 | 2.2 | 2.2 | 24.3 | 3,293.5 | -100 | 0.2 | 4.60 |
| 67 | 35.24 | 29 | 1,477 | 296 | 2.3 | 2.3 | 25.8 | 3,492.5 | -133 | 0.2 | 4.60 |
| 69 | 35.31 | 29 | 1,481 | 292 | 2.3 | 2.3 | 25.8 | 3,493.4 | -135 | 0.2 | 4.40 |
| 71 | 35.38 | 29 | 1,470 | 287 | 2.3 | 2.3 | 25.4 | 3,449.7 | -114 | 0.2 | 4.60 |
| 73 | 35.45 | 29 | 1,410 | 290 | 2.2 | 2.2 | 23.6 | 3,194.9 | -81 | 0.1 | 4.60 |
| 75 | 35.52 | 29 | 1,458 | 291 | 2.3 | 2.3 | 25.1 | 3,407.1 | -124 | 0.2 | 4.80 |

FDOT SPLICE RESEARCH - TP-1RS2

Test date: 21-Sep-2004

| BL# | depth | BLC | FMX | RMX | CSI | CSX | EMX | ETR | CTN | TSN | TRP |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | bl/ft | kips | kips | ksi | ksi | k-ft | (%) | kips | ksi | ms |
| 77 | 35.59 | 29 | 1,475 | 286 | 2.3 | 2.3 | 25.6 | 3,470.7 | -144 | 0.2 | 4.40 |
| 79 | 35.66 | 29 | 1,418 | 305 | 2.2 | 2.2 | 23.7 | 3,217.4 | -190 | 0.3 | 4.60 |
| 81 | 35.72 | 29 | 1,042 | 321 | 1.8 | 1.6 | 14.0 | 1,898.2 | 0 | 0.0 | 6.40 |
| 83 | 35.79 | 29 | 1,494 | 302 | 2.5 | 2.3 | 26.1 | 3,535.8 | -110 | 0.2 | 4.40 |
| 85 | 35.86 | 29 | 1,353 | 302 | 2.1 | 2.1 | 21.8 | 2,954.7 | -53 | 0.1 | 4.80 |
| 87 | 35.93 | 29 | 1,456 | 299 | 2.4 | 2.3 | 24.3 | 3,294.3 | -101 | 0.2 | 4.60 |
| 89 | 36.00 | 29 | 1,523 | 272 | 2.5 | 2.4 | 28.4 | 3,847.7 | -151 | 0.2 | 4.00 |
| 91 | 36.07 | 29 | 1,473 | 303 | 2.3 | 2.3 | 25.5 | 3,451.3 | -121 | 0.2 | 4.40 |
| 93 | 36.14 | 29 | 1,399 | 303 | 2.2 | 2.2 | 23.6 | 3,199.4 | -49 | 0.1 | 4.80 |
| 95 | 36.21 | 29 | 1,502 | 294 | 2.4 | 2.3 | 27.1 | 3,675.9 | -105 | 0.2 | 4.60 |
| 97 | 36.28 | 29 | 1,392 | 318 | 2.3 | 2.2 | 23.2 | 3,144.9 | -33 | 0.1 | 4.60 |
| 99 | 36.34 | 29 | 1,419 | 319 | 2.2 | 2.2 | 24.0 | 3,249.6 | -64 | 0.1 | 4.80 |
| 101 | 36.41 | 29 | 1,448 | 330 | 2.4 | 2.2 | 24.7 | 3,344.5 | -63 | 0.1 | 4.60 |
| 103 | 36.48 | 29 | 1,456 | 339 | 2.4 | 2.3 | 24.7 | 3,348.4 | -80 | 0.1 | 4.40 |
| 105 | 36.55 | 29 | 1,445 | 363 | 2.3 | 2.2 | 24.6 | 3,334.8 | -30 | 0.0 | 4.60 |
| 107 | 36.62 | 29 | 1,458 | 394 | 2.4 | 2.3 | 25.5 | 3,452.6 | 0 | 0.0 | 4.60 |
| 109 | 36.69 | 29 | 1,493 | 453 | 2.5 | 2.3 | 26.8 | 3,638.3 | 0 | 0.0 | 4.40 |
| 111 | 36.76 | 29 | 1,541 | 477 | 2.6 | 2.4 | 30.0 | 4,061.1 | 0 | 0.0 | 4.60 |
| 113 | 36.83 | 29 | 1,572 | 604 | 2.7 | 2.4 | 32.5 | 4,409.6 | 0 | 0.0 | 4.60 |
| 115 | 36.90 | 29 | 1,717 | 847 | 2.8 | 2.7 | 42.5 | 5,763.7 | 0 | 0.0 | 4.60 |
| 117 | 36.97 | 29 | 1,685 | 717 | 2.7 | 2.6 | 36.0 | 4,882.5 | 0 | 0.0 | 4.60 |
| 119 | 37.03 | 30 | 1,609 | 712 | 2.5 | 2.5 | 34.5 | 4,682.4 | 0 | 0.0 | 4.40 |
| 121 | 37.10 | 30 | 1,473 | 654 | 2.3 | 2.3 | 30.0 | 4,062.5 | 0 | 0.0 | 4.60 |
| 123 | 37.17 | 30 | 1,381 | 596 | 2.2 | 2.1 | 27.2 | 3,681.0 | 0 | 0.0 | 4.60 |
| 125 | 37.23 | 30 | 1,229 | 485 | 2.0 | 1.9 | 21.4 | 2,898.6 | -150 | 0.2 | 4.00 |
| 127 | 37.30 | 30 | 869 | 211 | 1.4 | 1.3 | 9.5 | 1,289.7 | -153 | 0.2 | 5.00 |
| 129 | 37.37 | 30 | 1,068 | 392 | 1.7 | 1.7 | 15.7 | 2,135.0 | -182 | 0.3 | 4.60 |
| 131 | 37.43 | 30 | 1,173 | 489 | 2.2 | 1.8 | 20.3 | 2,750.8 | -93 | 0.1 | 4.60 |
| 133 | 37.50 | 30 | 1,340 | 539 | 2.2 | 2.1 | 24.3 | 3,289.1 | -120 | 0.2 | 4.40 |
| 135 | 37.57 | 30 | 1,329 | 563 | 2.2 | 2.1 | 23.7 | 3,207.7 | -45 | 0.1 | 4.60 |
| 137 | 37.63 | 30 | 1,319 | 563 | 2.2 | 2.0 | 22.9 | 3,109.3 | -54 | 0.1 | 4.60 |
| 139 | 37.70 | 30 | 1,334 | 554 | 2.2 | 2.1 | 22.6 | 3,061.5 | -88 | 0.1 | 4.60 |
| 141 | 37.77 | 30 | 1,305 | 530 | 2.1 | 2.0 | 21.2 | 2,874.6 | -134 | 0.2 | 4.40 |
| 143 | 37.83 | 30 | 1,329 | 564 | 2.3 | 2.1 | 22.0 | 2,986.7 | -180 | 0.3 | 4.60 |
| 145 | 37.90 | 30 | 1,291 | 567 | 2.3 | 2.0 | 21.2 | 2,875.5 | -101 | 0.2 | 4.60 |
| 147 | 37.97 | 30 | 1,349 | 628 | 2.4 | 2.1 | 24.2 | 3,284.9 | -89 | 0.1 | 4.60 |
| 149 | 38.03 | 35 | 1,358 | 643 | 2.5 | 2.1 | 24.9 | 3,380.0 | -76 | 0.1 | 4.40 |
| 151 | 38.09 | 35 | 1,351 | 669 | 2.5 | 2.1 | 25.1 | 3,405.4 | -29 | 0.0 | 4.60 |
| 153 | 38.14 | 35 | 1,386 | 789 | 2.7 | 2.1 | 28.2 | 3,816.9 | 0 | 0.0 | 4.40 |
| 155 | 38.20 | 35 | 1,539 | 1,077 | 3.2 | 2.4 | 35.6 | 4,829.2 | 0 | 0.0 | 4.40 |
| 157 | 38.26 | 35 | 1,463 | 817 | 2.9 | 2.3 | 30.3 | 4,101.4 | 0 | 0.0 | 4.40 |
| 159 | 38.31 | 35 | 1,425 | 930 | 2.8 | 2.2 | 30.5 | 4,134.8 | 0 | 0.0 | 4.40 |
| 161 | 38.37 | 35 | 1,429 | 749 | 2.9 | 2.2 | 29.6 | 4,006.8 | 0 | 0.0 | 4.40 |
| 163 | 38.43 | 35 | 1,260 | 556 | 2.4 | 2.0 | 23.1 | 3,133.5 | 0 | 0.0 | 4.40 |
| 165 | 38.49 | 35 | 1,230 | 578 | 2.2 | 1.9 | 22.8 | 3,086.9 | -73 | 0.1 | 4.40 |
| 167 | 38.54 | 35 | 1,144 | 635 | 2.1 | 1.8 | 21.8 | 2,961.9 | 0 | 0.0 | 4.80 |

Applied Foundation Testing, Inc. Page 3 of 3
Case Method Results PDIPLOT Ver. 2005.1 - Printed: 28-Apr-2005

FDOT SPLICE RESEARCH - TP-1RS2

Test date: 21-Sep-2004

| BL# | depth | BLC | FMX | RMX | CSI | CSX | EMX | ETR | CTN | TSN | TRP |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | bl/ft | kips | kips | ksi | ksi | k-ft | (%) | kips | ksi | ms |
| 169 | 38.60 | 35 | 1,215 | 664 | 2.3 | 1.9 | 24.7 | 3,342.6 | 0 | 0.0 | 4.60 |
| 171 | 38.66 | 35 | 1,169 | 628 | 2.2 | 1.8 | 23.4 | 3,173.3 | 0 | 0.0 | 4.60 |
| 173 | 38.71 | 35 | 1,081 | 503 | 2.0 | 1.7 | 19.8 | 2,683.4 | -88 | 0.1 | 5.00 |
| 175 | 38.77 | 35 | 972 | 448 | 2.0 | 1.5 | 17.3 | 2,346.0 | -34 | 0.1 | 5.40 |
| 177 | 38.83 | 35 | 1,045 | 481 | 2.2 | 1.6 | 19.9 | 2,702.6 | -124 | 0.2 | 4.80 |
| 179 | 38.89 | 35 | 1,044 | 524 | 2.3 | 1.6 | 21.2 | 2,876.9 | -86 | 0.1 | 4.80 |
| 181 | 38.94 | 35 | 985 | 548 | 2.2 | 1.5 | 20.4 | 2,766.2 | -13 | 0.0 | 5.40 |
| 183 | 39.00 | 35 | 477 | 301 | 1.3 | 0.7 | 5.6 | 755.5 | -22 | 0.0 | 5.20 |
| | | Average | 1,365 | 410 | 2.3 | 2.1 | 24.4 | 3,302.9 | -71 | 0.1 | 4.62 |
| | | Std. Dev. | 181 | 192 | 0.3 | 0.3 | 4.9 | 668.1 | 58 | 0.1 | 0.30 |
| | | Maximum | 1,782 | 1,077 | 3.3 | 2.8 | 42.5 | 5,763.7 | 0 | 0.5 | 6.40 |
| | | @ Blow# | 116 | 155 | 156 | 116 | 115 | 115 | 1 | 128 | 81 |

Total number of blows analyzed: 183

Time Summary

Drive 13 minutes 35 seconds 1:32:51 PM - 1:46:26 PM (9/21/2004)

Applied Foundation Testing, Inc. — Page 1 of 3
Case Method Results — PDIPLOT Ver. 2005.1 - Printed: 28-Apr-2005

FDOT SPLICE RESEARCH - TP-2RS

Test date: 21-Sep-2004

AR: 645.53 in^2 — SP: 0.151 k/ft3
LE: 34.00 ft — EM: 5,506 ksi
WS: 13,000.0 f/s — JC: 0.50

FMX: Maximum Force — ETR: Energy Transfer Ratio
RMX: Max Case Method Capacity — CTN: Max Computed Tension
CSI: Max F1 or F2 Compr. Stress — CTX: Max Computed Tension
CSX: Max Measured Compr. Stress — TSX: Tension Stress Maximum
EMX: Max Transferred Energy

| BL# | depth ft | BLC bl/ft | FMX kips | RMX kips | CSI ksi | CSX ksi | EMX k-ft | ETR (%) | CTN kips | CTX kips | TSX ksi |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 14.06 | 17 | 814 | 163 | 1.3 | 1.3 | 15.1 | 2,047.5 | 0 | -56 | 0.1 |
| 5 | 14.29 | 17 | 1,342 | 215 | 2.2 | 2.1 | 26.2 | 3,547.1 | -182 | -182 | 0.3 |
| 9 | 14.53 | 17 | 1,239 | 203 | 2.1 | 1.9 | 22.9 | 3,108.7 | -108 | -108 | 0.2 |
| 13 | 14.76 | 17 | 1,204 | 200 | 2.1 | 1.9 | 21.9 | 2,963.3 | -46 | -53 | 0.1 |
| 17 | 15.00 | 17 | 1,364 | 183 | 2.3 | 2.1 | 26.8 | 3,628.9 | -252 | -252 | 0.4 |
| 21 | 15.22 | 18 | 1,254 | 175 | 2.2 | 1.9 | 23.1 | 3,128.5 | -128 | -128 | 0.2 |
| 25 | 15.44 | 18 | 1,296 | 174 | 2.3 | 2.0 | 24.3 | 3,295.4 | -218 | -218 | 0.3 |
| 29 | 15.67 | 18 | 844 | 158 | 1.5 | 1.3 | 12.6 | 1,714.1 | 0 | -34 | 0.1 |
| 33 | 15.89 | 18 | 783 | 153 | 1.4 | 1.2 | 12.0 | 1,620.7 | 0 | -38 | 0.1 |
| 37 | 16.05 | 40 | 802 | 157 | 1.5 | 1.2 | 12.1 | 1,637.3 | 0 | -36 | 0.1 |
| 41 | 16.15 | 40 | 965 | 167 | 1.8 | 1.5 | 15.2 | 2,058.8 | -48 | -48 | 0.1 |
| 45 | 16.25 | 40 | 882 | 159 | 1.6 | 1.4 | 13.5 | 1,835.0 | -11 | -39 | 0.1 |
| 49 | 16.35 | 40 | 974 | 159 | 1.8 | 1.5 | 15.7 | 2,122.8 | -60 | -60 | 0.1 |
| 53 | 16.45 | 40 | 1,057 | 161 | 1.9 | 1.6 | 17.7 | 2,399.6 | -104 | -104 | 0.2 |
| 57 | 16.55 | 40 | 1,166 | 160 | 2.1 | 1.8 | 20.5 | 2,775.2 | -171 | -171 | 0.3 |
| 61 | 16.65 | 40 | 982 | 148 | 1.7 | 1.5 | 16.1 | 2,177.1 | -64 | -64 | 0.1 |
| 65 | 16.75 | 40 | 1,009 | 153 | 1.8 | 1.6 | 16.6 | 2,248.7 | -77 | -77 | 0.1 |
| 69 | 16.85 | 40 | 854 | 142 | 1.5 | 1.3 | 13.2 | 1,785.6 | 0 | -28 | 0.0 |
| 73 | 16.95 | 40 | 884 | 142 | 1.5 | 1.4 | 14.0 | 1,903.1 | -7 | -29 | 0.0 |
| 77 | 17.13 | 15 | 894 | 127 | 1.6 | 1.4 | 14.2 | 1,921.2 | -12 | -37 | 0.1 |
| 81 | 17.40 | 15 | 919 | 135 | 1.6 | 1.4 | 14.7 | 1,989.1 | -28 | -40 | 0.1 |
| 85 | 17.67 | 15 | 975 | 133 | 1.8 | 1.5 | 16.2 | 2,195.3 | -49 | -49 | 0.1 |
| 89 | 17.93 | 15 | 1,022 | 132 | 1.8 | 1.6 | 16.8 | 2,277.8 | -85 | -85 | 0.1 |
| 93 | 18.27 | 11 | 1,059 | 142 | 1.9 | 1.6 | 18.2 | 2,474.3 | -114 | -114 | 0.2 |
| 97 | 18.64 | 11 | 1,059 | 146 | 1.9 | 1.6 | 18.2 | 2,468.8 | -110 | -110 | 0.2 |
| 101 | 19.00 | 11 | 957 | 151 | 1.7 | 1.5 | 16.4 | 2,229.1 | -54 | -54 | 0.1 |
| 105 | 19.25 | 16 | 983 | 150 | 1.7 | 1.5 | 16.8 | 2,271.8 | -69 | -69 | 0.1 |
| 109 | 19.50 | 16 | 729 | 134 | 1.3 | 1.1 | 11.4 | 1,543.6 | 0 | -22 | 0.0 |
| 113 | 19.75 | 16 | 770 | 139 | 1.4 | 1.2 | 12.2 | 1,649.6 | 0 | -22 | 0.0 |
| 117 | 20.00 | 16 | 966 | 168 | 1.8 | 1.5 | 16.5 | 2,240.4 | 0 | -32 | 0.1 |
| 121 | 20.14 | 28 | 1,278 | 183 | 2.2 | 2.0 | 24.9 | 3,379.1 | -205 | -205 | 0.3 |
| 125 | 20.29 | 28 | 1,186 | 173 | 2.1 | 1.8 | 21.7 | 2,936.7 | -64 | -64 | 0.1 |
| 129 | 20.43 | 28 | 1,007 | 180 | 1.8 | 1.6 | 17.8 | 2,409.2 | -107 | -107 | 0.2 |
| 133 | 20.57 | 28 | 1,031 | 118 | 1.9 | 1.6 | 20.0 | 2,707.5 | -107 | -107 | 0.2 |
| 137 | 20.71 | 28 | 1,064 | 169 | 1.9 | 1.6 | 19.3 | 2,620.5 | -148 | -148 | 0.2 |
| 141 | 20.86 | 28 | 1,028 | 145 | 2.0 | 1.6 | 18.5 | 2,502.5 | -112 | -112 | 0.2 |
| 145 | 21.00 | 28 | 1,053 | 146 | 2.0 | 1.6 | 19.1 | 2,585.9 | -117 | -117 | 0.2 |
| 149 | 21.29 | 14 | 865 | 107 | 1.6 | 1.3 | 12.6 | 1,712.1 | -66 | -66 | 0.1 |

FDOT SPLICE RESEARCH - TP-2RS

Test date: 21-Sep-2004

| BL# | depth | BLC | FMX | RMX | CSI | CSX | EMX | ETR | CTN | CTX | TSX |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | bl/ft | kips | kips | ksi | ksi | k-ft | (%) | kips | kips | ksi |
| 153 | 21.57 | 14 | 1,121 | 171 | 2.2 | 1.7 | 21.3 | 2,884.4 | -97 | -97 | 0.2 |
| 157 | 21.86 | 14 | 1,105 | 169 | 2.1 | 1.7 | 20.7 | 2,810.6 | -108 | -108 | 0.2 |
| 161 | 22.29 | 7 | 1,096 | 175 | 2.0 | 1.7 | 20.7 | 2,809.5 | -113 | -113 | 0.2 |
| 165 | 22.86 | 7 | 1,191 | 205 | 2.2 | 1.8 | 22.2 | 3,015.0 | -171 | -171 | 0.3 |
| 169 | 23.38 | 8 | 1,062 | 203 | 1.9 | 1.6 | 18.4 | 2,499.8 | -86 | -86 | 0.1 |
| 173 | 23.88 | 8 | 1,156 | 192 | 2.1 | 1.8 | 20.5 | 2,781.5 | -159 | -159 | 0.2 |
| 177 | 24.09 | 32 | 1,116 | 207 | 2.0 | 1.7 | 19.6 | 2,653.8 | -107 | -107 | 0.2 |
| 181 | 24.22 | 32 | 1,140 | 196 | 2.0 | 1.8 | 19.8 | 2,686.8 | -118 | -118 | 0.2 |
| 185 | 24.34 | 32 | 1,271 | 202 | 2.2 | 2.0 | 23.6 | 3,197.2 | -173 | -173 | 0.3 |
| 189 | 24.47 | 32 | 1,180 | 199 | 2.1 | 1.8 | 21.3 | 2,891.2 | -47 | -47 | 0.1 |
| 193 | 24.59 | 32 | 1,131 | 189 | 2.0 | 1.8 | 20.0 | 2,715.3 | -23 | -31 | 0.0 |
| 197 | 24.72 | 32 | 1,138 | 179 | 2.0 | 1.8 | 20.2 | 2,743.1 | -40 | -40 | 0.1 |
| 201 | 24.84 | 32 | 1,172 | 182 | 2.1 | 1.8 | 20.6 | 2,794.1 | -77 | -77 | 0.1 |
| 205 | 24.97 | 32 | 965 | 174 | 1.6 | 1.5 | 15.4 | 2,088.9 | 0 | -18 | 0.0 |
| 209 | 25.38 | 8 | 1,136 | 206 | 2.0 | 1.8 | 19.5 | 2,649.4 | -67 | -67 | 0.1 |
| 213 | 25.88 | 8 | 1,199 | 211 | 2.1 | 1.9 | 21.4 | 2,903.2 | -53 | -53 | 0.1 |
| 217 | 26.17 | 18 | 1,253 | 212 | 2.3 | 1.9 | 22.9 | 3,111.2 | -123 | -123 | 0.2 |
| 221 | 26.39 | 18 | 1,283 | 195 | 2.3 | 2.0 | 24.0 | 3,257.6 | -145 | -145 | 0.2 |
| 225 | 26.61 | 18 | 1,096 | 224 | 1.9 | 1.7 | 20.6 | 2,787.0 | 0 | -28 | 0.0 |
| 229 | 26.83 | 18 | 1,149 | 188 | 2.0 | 1.8 | 20.0 | 2,718.1 | -55 | -55 | 0.1 |
| 233 | 27.05 | 22 | 1,140 | 182 | 2.1 | 1.8 | 19.8 | 2,680.4 | -61 | -61 | 0.1 |
| 237 | 27.23 | 22 | 1,320 | 199 | 2.4 | 2.0 | 25.3 | 3,431.8 | -171 | -171 | 0.3 |
| 241 | 27.41 | 22 | 1,226 | 198 | 2.2 | 1.9 | 22.0 | 2,989.4 | -109 | -109 | 0.2 |
| 245 | 27.59 | 22 | 1,247 | 191 | 2.3 | 1.9 | 22.7 | 3,077.3 | -122 | -122 | 0.2 |
| 249 | 27.77 | 22 | 1,258 | 188 | 2.3 | 1.9 | 23.0 | 3,120.7 | -118 | -118 | 0.2 |
| 253 | 27.95 | 22 | 1,228 | 185 | 2.2 | 1.9 | 22.1 | 3,001.7 | -107 | -107 | 0.2 |
| 257 | 28.13 | 23 | 1,255 | 183 | 2.3 | 1.9 | 23.2 | 3,142.7 | -121 | -121 | 0.2 |
| 261 | 28.30 | 23 | 1,256 | 179 | 2.3 | 1.9 | 23.2 | 3,149.6 | -118 | -118 | 0.2 |
| 265 | 28.48 | 23 | 1,245 | 179 | 2.2 | 1.9 | 22.9 | 3,099.7 | -117 | -117 | 0.2 |
| 269 | 28.65 | 23 | 1,321 | 183 | 2.4 | 2.0 | 25.7 | 3,483.2 | -165 | -165 | 0.3 |
| 273 | 28.83 | 23 | 1,238 | 182 | 2.3 | 1.9 | 22.8 | 3,093.7 | -106 | -106 | 0.2 |
| 277 | 29.00 | 23 | 1,270 | 183 | 2.3 | 2.0 | 23.7 | 3,215.5 | -120 | -120 | 0.2 |
| 281 | 29.19 | 21 | 1,276 | 179 | 2.4 | 2.0 | 23.9 | 3,243.9 | -124 | -124 | 0.2 |
| 285 | 29.38 | 21 | 1,256 | 179 | 2.3 | 1.9 | 23.4 | 3,169.8 | -112 | -112 | 0.2 |
| 289 | 29.57 | 21 | 1,191 | 170 | 2.2 | 1.8 | 21.5 | 2,911.6 | -23 | -24 | 0.0 |
| 293 | 29.76 | 21 | 1,247 | 167 | 2.3 | 1.9 | 22.9 | 3,106.4 | -89 | -89 | 0.1 |
| 297 | 29.95 | 21 | 1,227 | 162 | 2.2 | 1.9 | 22.2 | 3,012.0 | -67 | -67 | 0.1 |
| 301 | 30.14 | 22 | 1,181 | 148 | 2.1 | 1.8 | 21.3 | 2,883.5 | -14 | -28 | 0.0 |
| 305 | 30.32 | 22 | 1,252 | 149 | 2.2 | 1.9 | 23.0 | 3,122.7 | -95 | -95 | 0.1 |
| 309 | 30.50 | 22 | 1,267 | 151 | 2.3 | 2.0 | 23.1 | 3,131.7 | -90 | -90 | 0.1 |
| 313 | 30.68 | 22 | 1,213 | 137 | 2.1 | 1.9 | 21.8 | 2,961.4 | -42 | -42 | 0.1 |
| 317 | 30.86 | 22 | 1,287 | 149 | 2.4 | 2.0 | 23.7 | 3,210.5 | -105 | -105 | 0.2 |
| 321 | 31.04 | 23 | 1,280 | 147 | 2.3 | 2.0 | 23.8 | 3,223.8 | -102 | -102 | 0.2 |
| 325 | 31.22 | 23 | 1,235 | 137 | 2.3 | 1.9 | 22.0 | 2,983.9 | -63 | -63 | 0.1 |
| 329 | 31.39 | 23 | 1,260 | 143 | 2.3 | 2.0 | 23.1 | 3,129.6 | -91 | -91 | 0.1 |
| 333 | 31.57 | 23 | 1,269 | 142 | 2.3 | 2.0 | 23.1 | 3,127.6 | -88 | -88 | 0.1 |

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Case Method Results | PDIPLOT Ver. 2005.1 - Printed: 28-Apr-2005

FDOT SPLICE RESEARCH - TP-2RS

Test date: 21-Sep-2004

| BL# | depth ft | BLC bl/ft | FMX kips | RMX kips | CSI ksi | CSX ksi | EMX k-ft | ETR (%) | CTN kips | CTX kips | TSX ksi |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 337 | 31.74 | 23 | 1,231 | 133 | 2.2 | 1.9 | 21.9 | 2,966.4 | -49 | -49 | 0.1 |
| 341 | 31.91 | 23 | 1,226 | 131 | 2.2 | 1.9 | 21.6 | 2,926.2 | -51 | -51 | 0.1 |
| 345 | 32.10 | 21 | 1,210 | 137 | 2.2 | 1.9 | 21.2 | 2,875.5 | -28 | -30 | 0.0 |
| 349 | 32.29 | 21 | 1,187 | 125 | 2.1 | 1.8 | 20.5 | 2,785.3 | -19 | -29 | 0.0 |
| 353 | 32.48 | 21 | 1,222 | 129 | 2.2 | 1.9 | 21.5 | 2,918.3 | -53 | -53 | 0.1 |
| 357 | 32.67 | 21 | 1,213 | 132 | 2.1 | 1.9 | 21.3 | 2,885.9 | -40 | -40 | 0.1 |
| 361 | 32.86 | 21 | 1,215 | 138 | 2.1 | 1.9 | 21.1 | 2,866.6 | -68 | -68 | 0.1 |
| 365 | 33.05 | 19 | 1,258 | 134 | 2.2 | 1.9 | 22.7 | 3,070.9 | -106 | -106 | 0.2 |
| 369 | 33.26 | 19 | 986 | 139 | 1.6 | 1.5 | 15.4 | 2,081.9 | 0 | -43 | 0.1 |
| 373 | 33.47 | 19 | 1,223 | 164 | 2.1 | 1.9 | 21.9 | 2,965.5 | -81 | -81 | 0.1 |
| 377 | 33.68 | 19 | 1,243 | 165 | 2.0 | 1.9 | 22.5 | 3,053.2 | -98 | -98 | 0.2 |
| 381 | 33.89 | 19 | 1,238 | 163 | 2.1 | 1.9 | 22.2 | 3,009.4 | -105 | -105 | 0.2 |
| | | Average | 1,133 | 165 | 2.0 | 1.8 | 20.2 | 2,734.2 | -83 | -88 | 0.1 |
| | | Std. Dev. | 153 | 27 | 0.3 | 0.2 | 3.8 | 509.5 | 54 | 48 | 0.1 |
| | | Maximum | 1,558 | 242 | 2.7 | 2.4 | 35.4 | 4,795.1 | 0 | -18 | 0.4 |
| | | @ Blow# | 226 | 226 | 226 | 226 | 226 | 226 | 1 | 205 | 18 |

Total number of blows analyzed: 383

Time Summary

Drive 20 minutes 33 seconds 10:36:47 AM - 10:57:20 AM (9/21/2004)
Stop 10 minutes 32 seconds 10:57:20 AM - 11:07:52 AM
Drive 6 minutes 56 seconds 11:07:52 AM - 11:14:48 AM

Total time [0:38:01] = (Driving [0:27:29] + Stop [0:10:32])

# APPENDIX D
# MATHCAD WORKSHEET CALCULATIONS

This appendix contains a copy of a MATHCAD worksheet used to calculate the transformed section properties in the splice. Also, with a perfect bond between the pile and the HSS steel pipe, the strains in both materials is equal. The stress and equivalent force carried by each component is also computed for the maximum compressive and tensile forces in the splice.

The maximum compressive force at the joint of the splice was 1700 kips during pile driving. The maximum tensile force at the joint of the splice was 335 kips during pile driving. The steel pipe was designed to transfer the entire tensile load across the splice.

$\text{ORIGIN} \equiv 1$

Units $\text{kip} := 1000\text{lbf}$

$\text{ksi} := \frac{1000\text{lbf}}{\text{in}^2}$

Input Material Properties

Modulus of Elasticity

$\text{Est} := 29000\text{ksi}$ Steel strands and HSS pipe

$\text{Econc} := 5300\text{ksi}$ Modulus of pile used in PDA unit

$\text{Egrout} := 2820\text{ksi}$ Masterbuilders Master Flow Product 928

$\text{Eset45} := 4500\text{ksi}$ Masterbuilders Product Set 45

Prestressing Steel

$\text{Strand} := 0.217\text{in}^2$ strand x-sectional area

$n := 20$ number of strands used

$\text{Ast} := n \cdot \text{Strand}$ $\quad \text{Ast} = 4.34\,\text{in}^2$ Area of prestressing steel reinforcement

Pile Dimensions

$w := 30\text{in}$ width of pile

$D_v := 18\text{in}$ diamter of void $\quad D_1 := 18\text{in}$

$\text{Dpipe} := 14.0\text{in}$ Outside diamter of HSS pipe

$\text{tpipe} := 0.5\text{in}$ thickness of HSS pipe

$\text{Dvent} := 3\text{in}$ Diameter of vent in HSS to allow gases to escape

$\text{Apipe} := 19.8\text{in}^2$ HSS14.000 x 0.500

Specific Weight of Materials

$\gamma\text{conc} := 150\frac{\text{lbf}}{\text{ft}^3}$ $\quad \rho\text{conc} := \frac{\gamma\text{conc}}{g}$ unit weight and density of concrete

$\gamma\text{ste} := 490\frac{\text{lbf}}{\text{ft}^3}$ $\quad \rho\text{ste} := \frac{\gamma\text{ste}}{g}$ unit weight and density of steel

## Cross Section #1 and #5: Above/Below the Splice in the Voided Section of Pile

Area of concrete in Voided Cross Section

$$Aconc := w^2 - \pi \cdot \frac{\left(D_v\right)^2}{4} - Ast \qquad Aconc = 641.19\,in^2$$

Young's Modulus for Voided Cross Section

$$E_1 := \frac{Econc \cdot Aconc + Est \cdot Ast}{Aconc + Ast} \qquad E_1 = 5459.34\,ksi \qquad E_5 := E_1$$

Density of Voided Cross Section

$$\rho_1 := \frac{\rho conc \cdot Aconc + \rho ste \cdot Ast}{Aconc + Ast} \qquad \rho_1 = 0.01\,lb\,ft\,in^{-4} \qquad \rho_5 := \rho_1$$

$$\gamma_1 := \rho_1 \cdot g \qquad \gamma_1 = 0.15\,\frac{kip}{ft^3} \qquad \gamma_5 := \gamma_1$$

Wave Speed in Voided Cross Section

$$c_1 := \sqrt{\frac{E_1}{\rho_1}} \qquad c_1 = 12887.67\,\frac{ft}{sec} \qquad c_5 := c_1$$

Impedence of Voided Cross Section

$$Z_1 := \frac{E_1(Aconc + Ast)}{c_1} \qquad Z_1 = 273.45\,kip \cdot \frac{sec}{ft} \qquad Z_5 := Z_1$$

## Cross Section #2 and #4: In the Steel Pipe Spliced Cross Section

Total Area of Steel

$Asteel := Apipe + Ast$ $Asteel = 24.14\,in^2$

Cross Sectional Area of Concrete and Grout

$$Aannulus := \frac{\pi}{4}\left[\left(D_v\right)^2 - (Dpipe)^2\right]$$

Area of grout in Annulus of pile void masterflow 928

$Aannulus = 100.53\,in^2$

$$Ainner := \frac{\pi}{4}\left[(Dpipe - 2\cdot tpipe)^2 - (Dvent)^2\right]$$

Area of concrete inside HSS Pipe

$Ainner = 125.66\,in^2$

$Agroucon := Aconc + Aannulus + Ainner$

Total area of concrete and grout

$Agroucon = 867.39\,in^2$

Composite Young's Modulus for Spliced Cross Section: including concrete, grout and ste

$$E_2 := \frac{Econc\cdot(Ainner + Aconc) + Egrout\cdot Aannulus + Est\cdot Asteel}{Agroucon + Asteel} \qquad E_2 = 5662.08\,ksi$$

$E_4 := E_2$

Density of Composite in Spliced Cross Section

$$\rho_2 := \frac{\rho conc\cdot Agroucon + \rho ste\cdot Asteel}{Agroucon + Asteel} \qquad \rho_4 := \rho_2 \qquad \rho_2 = 0.01\,lb\,ft\,in^{-4}$$

$$\gamma_2 := \rho_2\cdot g \qquad \gamma_4 := \rho_2\cdot g \qquad \gamma_2 = 5122.31\,\frac{lb}{ft^2\,sec^2}$$

Wave Speed in Spliced Cross Section

$$c_2 := \sqrt{\frac{E_2}{\rho_2}} \qquad c_4 := c_2 \qquad c_2 = 12836.36\,\frac{ft}{sec}$$

Impedence of Spliced Cross Section

$$Z_2 := \frac{E_2(Agroucon + Asteel)}{c_2} \qquad Z_4 := Z_2 \qquad Z_2 = 393.249\,\frac{sec}{ft}\,kip$$

## Cross Section #3: At the mating surface (joint) between piles.

<u>Cross Sectional Area of Concrete and Grout</u>

$$\text{Aouter} := w^2 - \pi\cdot\frac{\left(D_v\right)^2}{4}$$

$\text{Aouter} = 645.53\,\text{in}^2$ Bonded or Not Bonded

$\text{Aannulus} = 100.53\,\text{in}^2$ Annulus grout

$\text{Ainner} = 125.66\,\text{in}^2$ Fills Pipe

$$\text{Agrout3} := \text{Aouter} + \text{Ainner} + \text{Aannulus}$$

$\text{Agrout3} = 871.73\,\text{in}^2$ $\quad$ $\text{Agroucon} = 867.39\,\text{in}^2$

<u>Composite Young's Modulus for X-section #3 Bonded</u>

$$E_3 := \frac{\text{Econc}\cdot\text{Ainner} + \text{Eset45}\cdot\text{Aouter} + \text{Est}\cdot\text{Apipe} + \text{Egrout}\cdot\text{Aannulus}}{\text{Agrout3} + \text{Apipe}}$$

$E_3 = 4967.44\,\text{ksi}$

<u>Density of Composite at Cross Section #3 Bonded</u>

$$\rho_3 := \frac{\rho\text{conc}\cdot\text{Agrout3} + \rho\text{ste}\cdot\text{Apipe}}{\text{Agrout3} + \text{Apipe}}$$

$\rho_3 = 0.0076\,\text{lb}\,\text{ft}\,\text{in}^{-4}$

$$\gamma_3 := \rho_2\cdot g$$

$\gamma_3 = 5122.31\,\frac{\text{lb}}{\text{ft}^2\,\text{sec}^2}$

<u>Wave Speed in Cross Section #3 Bonded</u>

$$c_3 := \sqrt{\frac{E_3}{\rho_3}}$$

$c_3 = 12086.2\,\frac{\text{ft}}{\text{sec}}$

<u>Impedence in Cross Section #3 Bonded</u>

$$Z_3 := \frac{E_3(\text{Agrout3} + \text{Apipe})}{c_3}$$

$Z_3 = 366.418\,\frac{\text{sec}}{\text{ft}}\,\text{kip}$

# Summary of X-sections

<u>Cross Section #1 and #5 in the void</u>

$A_1 := Aconc + Ast \qquad A_5 := A_1$

$A_1 = 645.53\,in^2$

$E_1 = 5459.34\,ksi$

$\rho_1 = 152.29\,\frac{lb}{ft^3}$

$c_1 = 12887.67\,\frac{ft}{sec}$

$Z_1 = 273.45\,\frac{sec}{ft}\,kip$

<u>Cross Section #2 and #4 in the splice</u>

$A_2 := Agroucon + Asteel$

$A_2 = 891.53\,in^2 \qquad A_4 := A_2$

$E_2 = 5662.08\,ksi$

$\rho_2 = 159.21\,\frac{lb}{ft^3}$

$c_2 = 12836.36\,\frac{ft}{sec}$

$Z_2 = 393.25\,\frac{sec}{ft}\,kip$

<u>Cross Section #3 at the Joint</u>

$A_3 := Aouter + Ainner + Aannulus + Apipe$

$A_3 = 891.53\,in^2$

$E_3 = 4967.44\,ksi$

$\rho_3 = 157.55\,\frac{lb}{ft^3}$

$c_3 = 12086.2\,\frac{ft}{sec}$

$Z_3 = 366.42\,\frac{sec}{ft}\,kip$

<u>Maximum Compressive Force of 1700 kips at the joint of the splice, cross section #3</u>

$Fcomp := 1700kip$

| | | |
|---|---|---|
| $\sigma := \dfrac{Fcomp}{A_3}$ | $\sigma = 1.91\,ksi$ | Avg stress in X-section #3 |
| $\varepsilon := \dfrac{\sigma}{E_3}$ | $\varepsilon = 0.000384$ | Avg Strain in X-section #3 |
| $\sigma conc := \varepsilon \cdot Econc$ | $\sigma conc = 2.03\,ksi$ | stress in concrete |
| $\sigma annu := \varepsilon \cdot Egrout$ | $\sigma annu = 1.08\,ksi$ | stress in annulus grout |
| $\sigma set45 := \varepsilon \cdot Eset45$ | $\sigma set45 = 1.73\,ksi$ | stress in mating surface grout |
| $\sigma st := \varepsilon \cdot Est$ | $\sigma st = 11.1\,ksi$ | stress in steel pipe |
| $Fset45 := \sigma set45 \cdot Aouter$ | $Fset45 = 1115\,kip$ | Force in set 45 grout |
| $Finner := \sigma conc \cdot Ainner$ | $Finner = 255.7\,kip$ | Force in concrete inside HSS pipe |
| $Fannu := \sigma annu \cdot Aannulus$ | $Fannu = 108.8\,kip$ | Force in annulus 928 grout |
| $Fst := \sigma st \cdot Apipe$ | $Fst = 220.4\,kip$ | Force in steel pipe |
| $Fstrand := \sigma st \cdot 0in^2$ | $Fstrand = 0\,kip$ | No strand at joint |

$Ftotal := Fset45 + Finner + Fannu + Fst$

$Ftotal = 1700\,kip$ $\quad Fcomp = 1700\,kip$

<u>Maximum Tensile Force of -335 kips at the joint of the splice</u>

$Ftens := -335kip$

| | | |
|---|---|---|
| $\sigma := \frac{Ftens}{A_3}$ | $\sigma = -0.38\,ksi$ | Avg stress in X-section #3 |
| $\varepsilon := \frac{\sigma}{E_3}$ | $\varepsilon = -0.000076$ | Avg Strain in X-section #3 |
| $\sigma conc := \varepsilon \cdot Econc$ | $\sigma conc = -0.4\,ksi$ | stress in concrete |
| $\sigma annu := \varepsilon \cdot Egrout$ | $\sigma annu = -0.21\,ksi$ | stress in annulus grout |
| $\sigma set45 := \varepsilon \cdot Eset45$ | $\sigma set45 = -0.34\,ksi$ | stress in mating surface grout |
| $\sigma st := \varepsilon \cdot Est$ | $\sigma st = -2.19\,ksi$ | stress in steel pipe |
| $Fset45 := \sigma set45 \cdot Aouter$ | $Fset45 = -219.7\,kip$ | Force in concrete |
| $Finner := \sigma conc \cdot Ainner$ | $Finner = -50.4\,kip$ | Force in concrete inside HSS pipe |
| $Fannu := \sigma annu \cdot Aannulus$ | $Fannu = -21.4\,kip$ | Force in annulus 928 grout |
| $Fst := \sigma st \cdot Apipe$ | $Fst = -43.44\,kip$ | Force in steel pipe |
| $Fstrand := \sigma st \cdot 0in^2$ | $Fstrand = 0\,kip$ | No strand at joint |

$Ftotal := Fset45 + Finner + Fannu + Fst$

$Ftotal = -335\,kip \qquad Ftens = -335\,kip$

<u>If assume steel pipe carries entire tensile force:</u>

| | | |
|---|---|---|
| $\sigma := \frac{Ftens}{Apipe}$ | $\sigma = -16.92\,ksi$ | Avg stress in steel pipe |
| $\varepsilon := \frac{\sigma}{Est}$ | $\varepsilon = -0.000583$ | Avg Strain in steel pipe |
| $Fst := \sigma \cdot Apipe$ | $Fst = -335\,kip$ | Force in steel pipe |

$Fst = -335\,kip \qquad Ftens = -335\,kip$

APPENDIX E
CAPWAP OUTPUT FOR TENSILE FORCES

Appendix E contains figures showing a comparison between the PDA output and the CAPWAP output for Pile #2 blow numbers 17, 18, 119, and 227, which were the hammer impacts that caused high tensile stresses. The figures included for each blow number are:

- CAPWAP computed force at top, middle, and segment 27 of pile versus time.
- PDA measured force at top of pile and CAPWAP computed force at top of pile versus time.
- PDA measured wave up at top of pile and CAPWAP computed wave up at top of pile versus time.
- PDA measured force at lower gage and CAPWAP computed force at segment 27 versus time.

The maximum value table output from CAPWAP was also included because it shows the maximum force in each pile segment defined in Figure E-1 below.

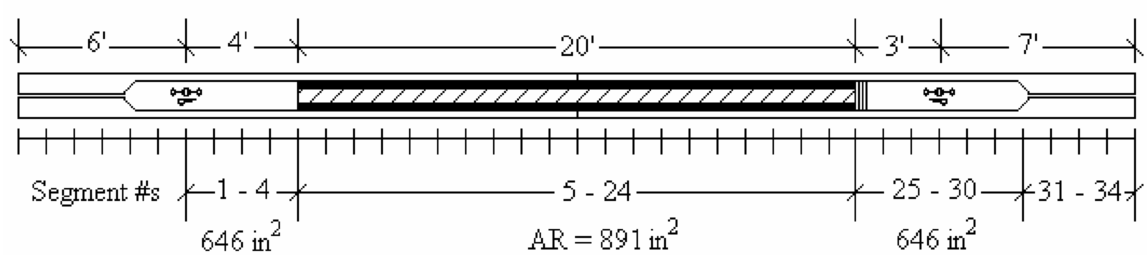


Figure E-1 Pile divided into 1 foot long segments for CAPWAP software.

Table E-1 CAPWAP output of final results for BN 17 of 383.

**Total CAPWAP Capacity: 280.0; along Shaft 154.6; at Toe 125.4 kips**

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 280.0 | | | | | |
| 1 | 22.2 | 3.2 | 0.0 | 280.0 | 0.0 | 0.00 | 0.00 | 0.000 | 0.880 |
| 2 | 25.2 | 6.2 | 0.0 | 280.0 | 0.0 | 0.00 | 0.00 | 0.000 | 0.880 |
| 3 | 28.1 | 9.1 | 0.0 | 280.0 | 0.0 | 0.00 | 0.00 | 0.000 | 0.880 |
| 4 | 31.1 | 12.1 | 74.3 | 205.7 | 74.3 | 25.27 | 2.53 | 0.129 | 1.000 |
| 5 | 34.0 | 15.0 | 80.3 | 125.4 | 154.6 | 27.28 | 2.73 | 0.129 | 1.000 |
| Avg. Skin | | | 30.9 | | | 10.31 | 1.04 | 0.129 | 1.000 |
| Toe | | | 125.4 | | | | 20.07 | 0.200 | 1.110 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.073 | 0.092 | Smith Type |
| Unloading Quake | (% of loading quake) | 30 | 6 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Unloading Level | (% of Ru) | 97 | | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.260 | |
| Soil Plug Weight | (kips) | | 2.41 | |

CAPWAP match quality: 5.85(Wave Up Match)
Observed: final set = 0.706 in; blow count = 17 b/ft
Computed: final set = 0.870 in; blow count = 14 b/ft

Table E-2 CAPWAP output of extreme values for BN 17 of 383.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1345.0 | -148.6 | 2.083 | -0.230 | 26.61 | 7.1 | 1.150 |
| 2 | 2.0 | 1339.9 | -151.9 | 2.075 | -0.235 | 26.58 | 7.0 | 1.150 |
| 4 | 4.0 | 1323.5 | -155.1 | 1.509 | -0.177 | 26.51 | 7.0 | 1.149 |
| 6 | 6.0 | 1299.2 | -158.5 | 1.458 | -0.178 | 26.42 | 6.9 | 1.149 |
| 8 | 8.0 | 1271.4 | -173.4 | 1.427 | -0.195 | 26.32 | 6.9 | 1.148 |
| 10 | 10.0 | 1236.6 | -221.4 | 1.388 | -0.248 | 26.23 | 6.8 | 1.147 |
| 12 | 12.1 | 1191.0 | -257.6 | 1.336 | -0.289 | 26.14 | 6.7 | 1.146 |
| 14 | 14.1 | 1138.4 | -290.5 | 1.277 | -0.326 | 26.05 | 6.6 | 1.145 |
| 16 | 16.1 | 1074.3 | -314.6 | 1.205 | -0.353 | 25.96 | 6.5 | 1.143 |
| 18 | 18.1 | 1001.1 | -325.9 | 1.123 | -0.366 | 25.95 | 6.5 | 1.142 |
| 20 | 20.1 | 933.0 | -335.3 | 1.047 | -0.376 | 25.93 | 6.8 | 1.141 |
| 22 | 22.2 | 857.5 | -329.0 | 0.962 | -0.369 | 25.92 | 7.1 | 1.139 |
| 24 | 24.2 | 770.9 | -302.4 | 0.909 | -0.357 | 25.90 | 7.3 | 1.138 |
| 26 | 26.2 | 699.8 | -267.6 | 1.084 | -0.414 | 25.88 | 7.6 | 1.136 |
| 27 | 27.1 | 662.9 | -245.8 | 1.027 | -0.381 | 25.87 | 7.7 | 1.135 |
| 28 | 28.1 | 625.3 | -221.9 | 0.968 | -0.344 | 25.86 | 7.9 | 1.134 |
| 29 | 29.1 | 585.6 | -195.8 | 0.907 | -0.303 | 25.85 | 8.0 | 1.133 |
| 30 | 30.1 | 543.0 | -166.8 | 0.813 | -0.250 | 25.84 | 8.1 | 1.132 |
| 31 | 31.1 | 485.4 | -124.7 | 0.545 | -0.140 | 25.84 | 8.1 | 1.131 |
| 32 | 32.0 | 378.8 | -165.3 | 0.425 | -0.185 | 17.59 | 8.2 | 1.131 |
| 33 | 33.0 | 322.3 | -113.2 | 0.362 | -0.127 | 17.55 | 8.2 | 1.131 |
| 34 | 34.0 | 313.4 | -55.2 | 0.352 | -0.062 | 8.88 | 8.2 | 1.130 |
| Absolute | 1.0 | | | 2.083 | | | (T = | 20.8 ms) |
| | 25.2 | | | | -0.444 | | (T = | 25.0 ms) |

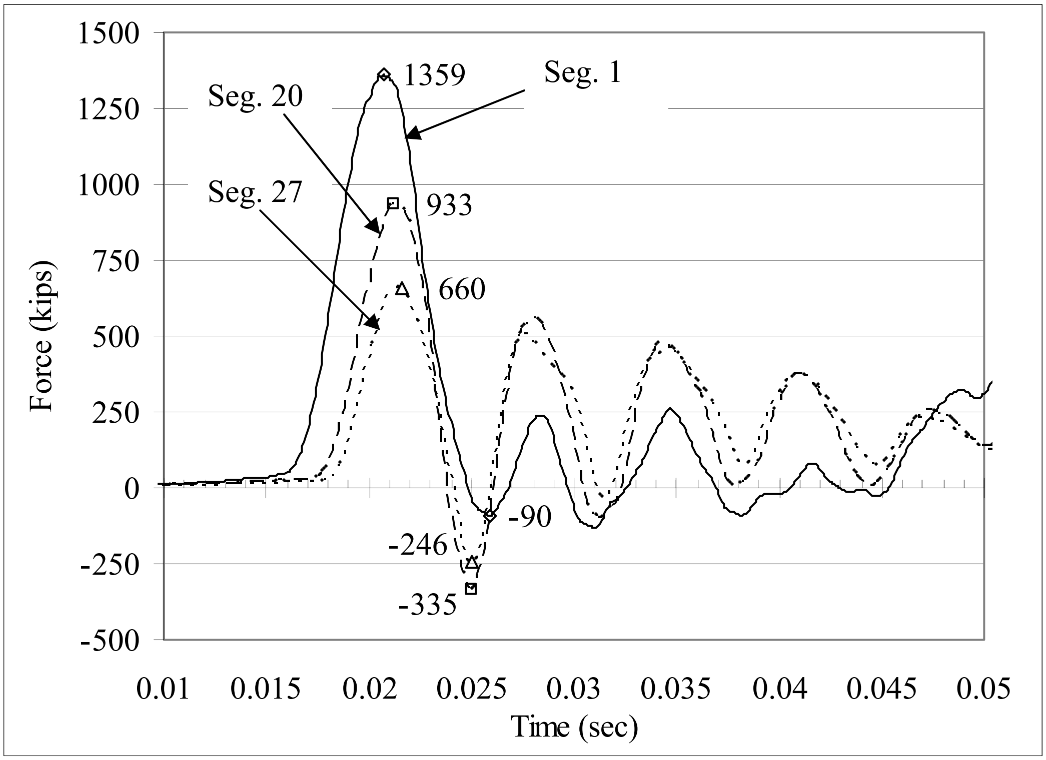


Figure E-2 CAPWAP output of force at three pile segments for BN 17 of 383.

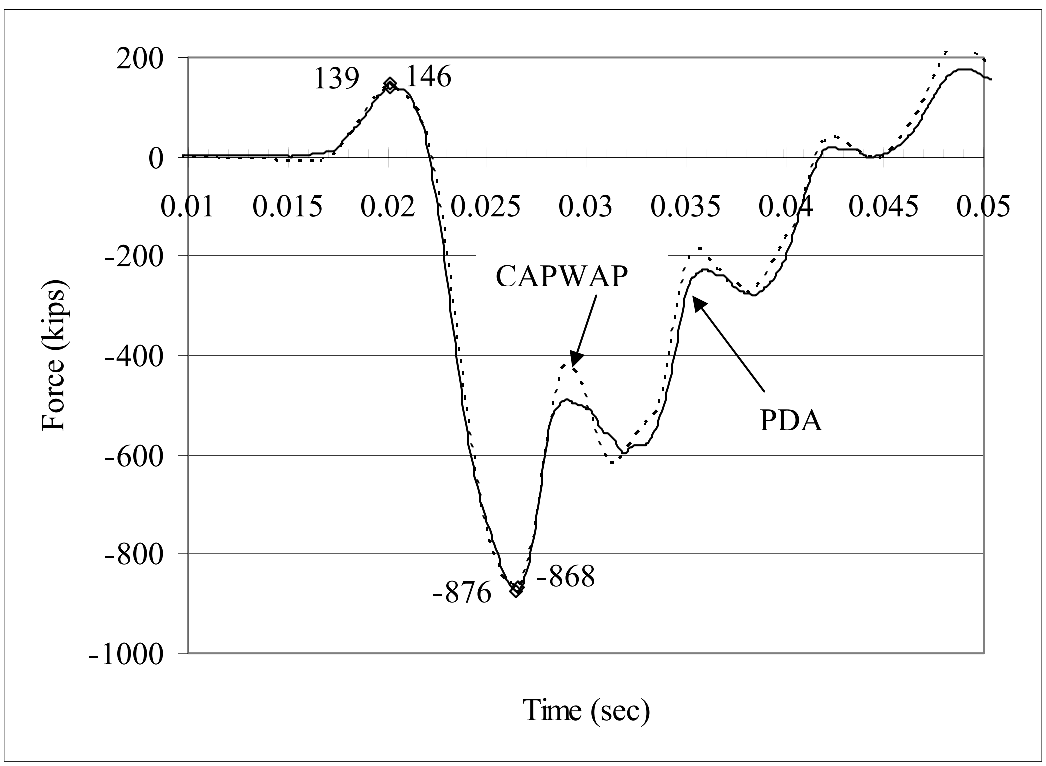


Figure E-3 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #2 for BN 17 of 383.

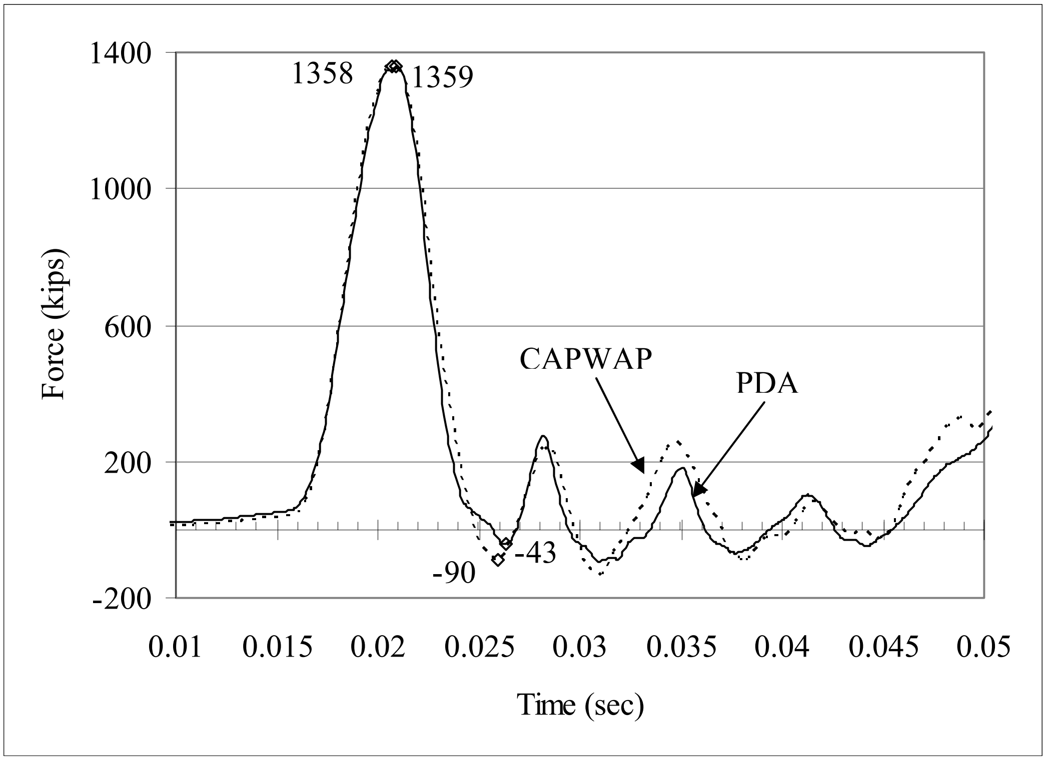


Figure E-4 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #2 for BN 17 of 383.

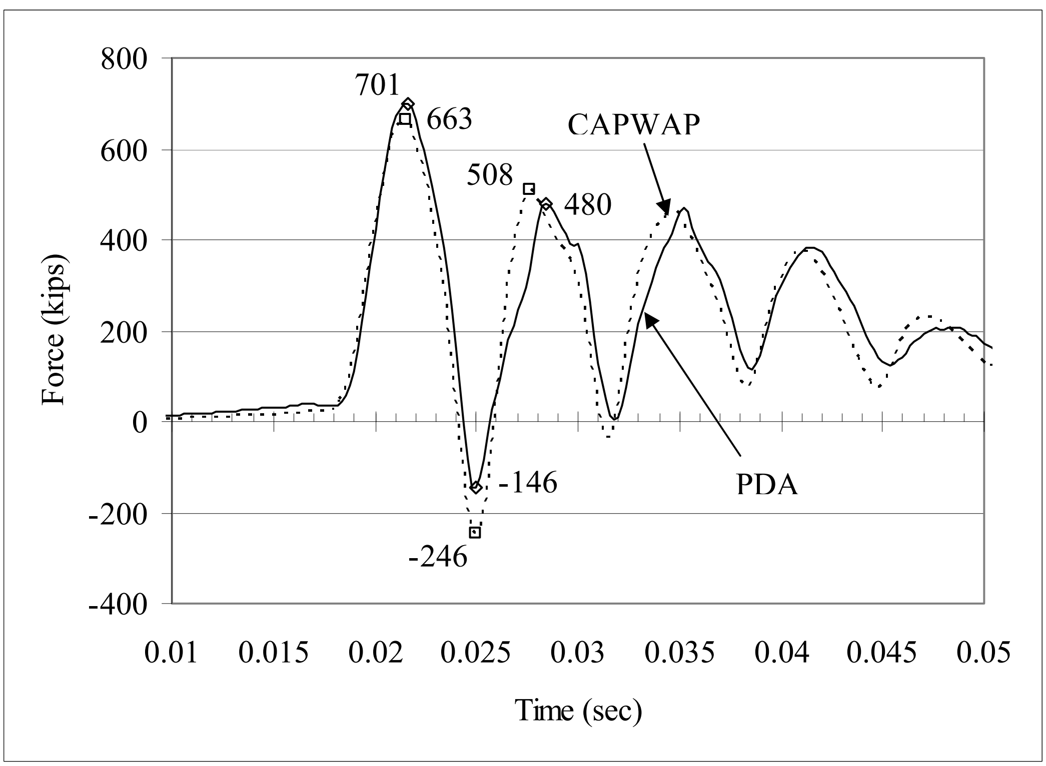


Figure E-5 BN 17 of Pile #2 comparison of PDA output and CAPWAP output at the lower gage location.

Table E-3 CAPWAP output of final results for BN 18 of 383.

Total CAPWAP Capacity: 273.2; along Shaft 151.3; at Toe 121.9 kips

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 273.2 | | | | | |
| 1 | 22.2 | 3.2 | 0.0 | 273.2 | 0.0 | 0.00 | 0.00 | 0.000 | 0.740 |
| 2 | 25.2 | 6.2 | 0.0 | 273.2 | 0.0 | 0.00 | 0.00 | 0.000 | 0.740 |
| 3 | 28.1 | 9.2 | 0.4 | 272.8 | 0.4 | 0.14 | 0.01 | 0.047 | 0.772 |
| 4 | 31.1 | 12.1 | 88.9 | 183.9 | 89.3 | 30.21 | 3.02 | 0.057 | 0.770 |
| 5 | 34.0 | 15.1 | 62.0 | 121.9 | 151.3 | 21.07 | 2.11 | 0.057 | 0.774 |
| Avg. Skin | | | 30.3 | | | 10.05 | 1.02 | 0.057 | 0.771 |
| Toe | | | 121.9 | | | | 19.50 | 0.279 | 0.970 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.032 | 0.124 | Smith Type |
| Unloading Quake | (% of loading quake) | 60 | 4 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Unloading Level | (% of Ru) | 22 | | |
| Soil Plug Weight | (kips) | | 2.48 | |

CAPWAP match quality: 5.80(Wave Up Match)
Observed: final set = 0.667 in; blow count = 18 b/ft
Computed: final set = 0.819 in; blow count = 15 b/ft

Table E-4 CAPWAP output of extreme values for BN 18 of 383.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1347.0 | -108.9 | 2.086 | -0.169 | 27.68 | 6.9 | 1.136 |
| 2 | 2.0 | 1341.5 | -111.9 | 2.078 | -0.173 | 27.65 | 6.9 | 1.136 |
| 4 | 4.0 | 1326.1 | -115.8 | 1.508 | -0.132 | 27.56 | 6.8 | 1.135 |
| 6 | 6.0 | 1305.5 | -118.0 | 1.465 | -0.132 | 27.46 | 6.8 | 1.134 |
| 8 | 8.0 | 1280.2 | -153.9 | 1.436 | -0.173 | 27.35 | 6.7 | 1.134 |
| 10 | 10.0 | 1247.9 | -200.0 | 1.400 | -0.224 | 27.25 | 6.7 | 1.133 |
| 12 | 12.1 | 1205.0 | -235.4 | 1.352 | -0.264 | 27.17 | 6.6 | 1.132 |
| 14 | 14.1 | 1148.0 | -270.1 | 1.288 | -0.303 | 27.16 | 6.5 | 1.131 |
| 16 | 16.1 | 1081.9 | -295.8 | 1.214 | -0.332 | 27.15 | 6.3 | 1.130 |
| 18 | 18.1 | 1007.9 | -302.7 | 1.131 | -0.340 | 27.13 | 6.4 | 1.128 |
| 20 | 20.1 | 935.8 | -310.5 | 1.050 | -0.348 | 27.12 | 6.7 | 1.127 |
| 22 | 22.2 | 859.9 | -308.2 | 0.965 | -0.346 | 27.11 | 6.9 | 1.126 |
| 24 | 24.2 | 768.3 | -278.8 | 0.906 | -0.329 | 27.10 | 7.2 | 1.125 |
| 26 | 26.2 | 691.7 | -243.0 | 1.071 | -0.376 | 27.08 | 7.4 | 1.123 |
| 27 | 27.1 | 650.6 | -220.6 | 1.008 | -0.342 | 27.07 | 7.5 | 1.122 |
| 28 | 28.1 | 609.7 | -195.1 | 0.944 | -0.302 | 27.07 | 7.6 | 1.121 |
| 29 | 29.1 | 567.8 | -167.5 | 0.879 | -0.259 | 27.03 | 7.7 | 1.121 |
| 30 | 30.1 | 521.8 | -136.1 | 0.782 | -0.204 | 27.02 | 7.8 | 1.120 |
| 31 | 31.1 | 466.8 | -92.4 | 0.524 | -0.104 | 27.01 | 7.9 | 1.119 |
| 32 | 32.0 | 372.6 | -120.2 | 0.418 | -0.135 | 19.88 | 7.9 | 1.119 |
| 33 | 33.0 | 334.4 | -67.5 | 0.375 | -0.076 | 19.83 | 7.9 | 1.118 |
| 34 | 34.0 | 325.8 | -50.3 | 0.366 | -0.056 | 15.19 | 8.0 | 1.118 |
| Absolute | 1.0 | | | 2.086 | | | (T = | 20.9 ms) |
| | 25.2 | | | | -0.408 | | (T = | 25.0 ms) |

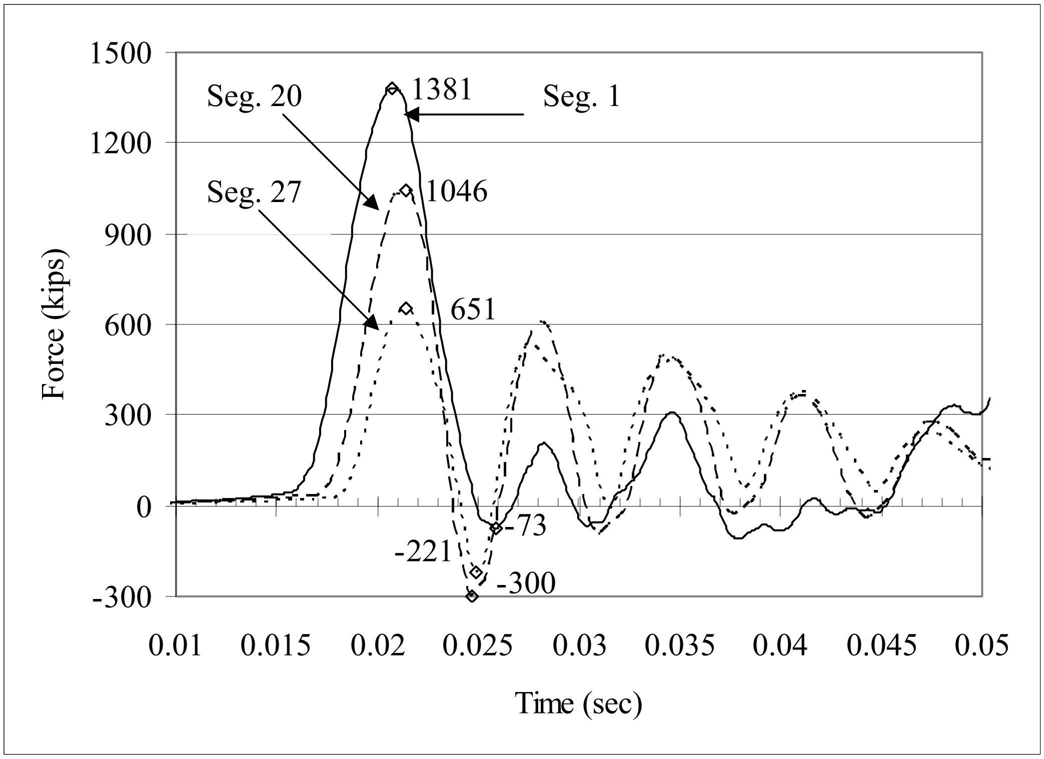


Figure E-6 CAPWAP output of force at three pile segments for BN 18 of 383.

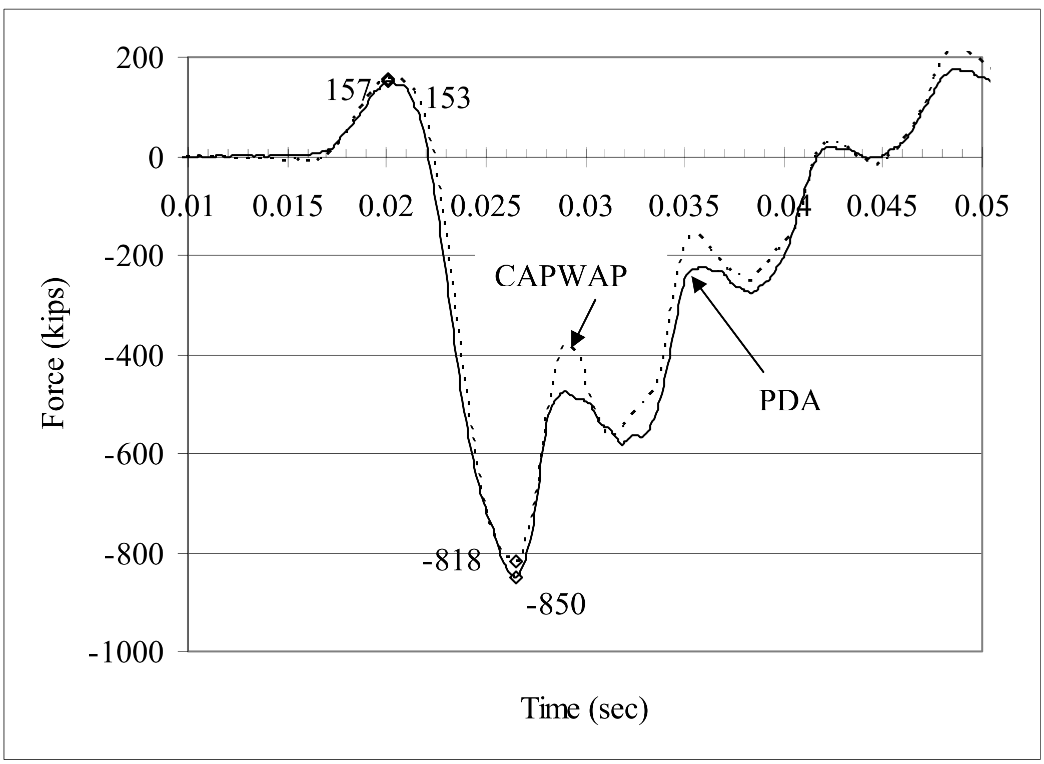


Figure E-7 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #2 for BN 18 of 383.

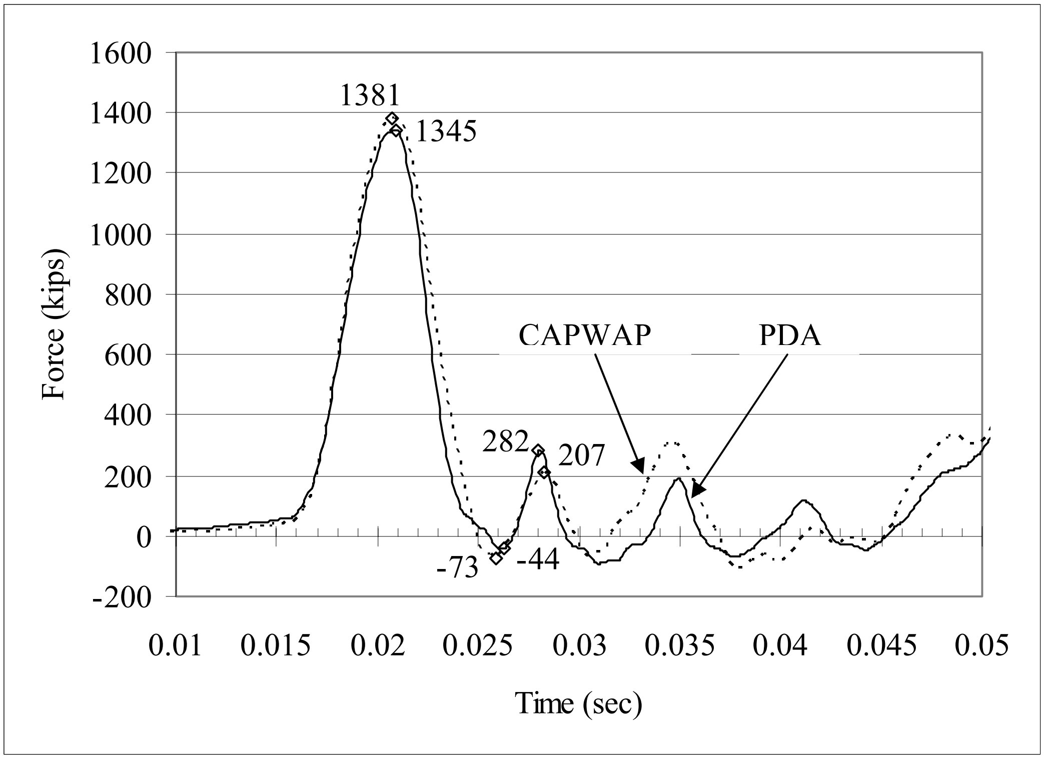


Figure E-8 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #2 for BN 18 of 383.

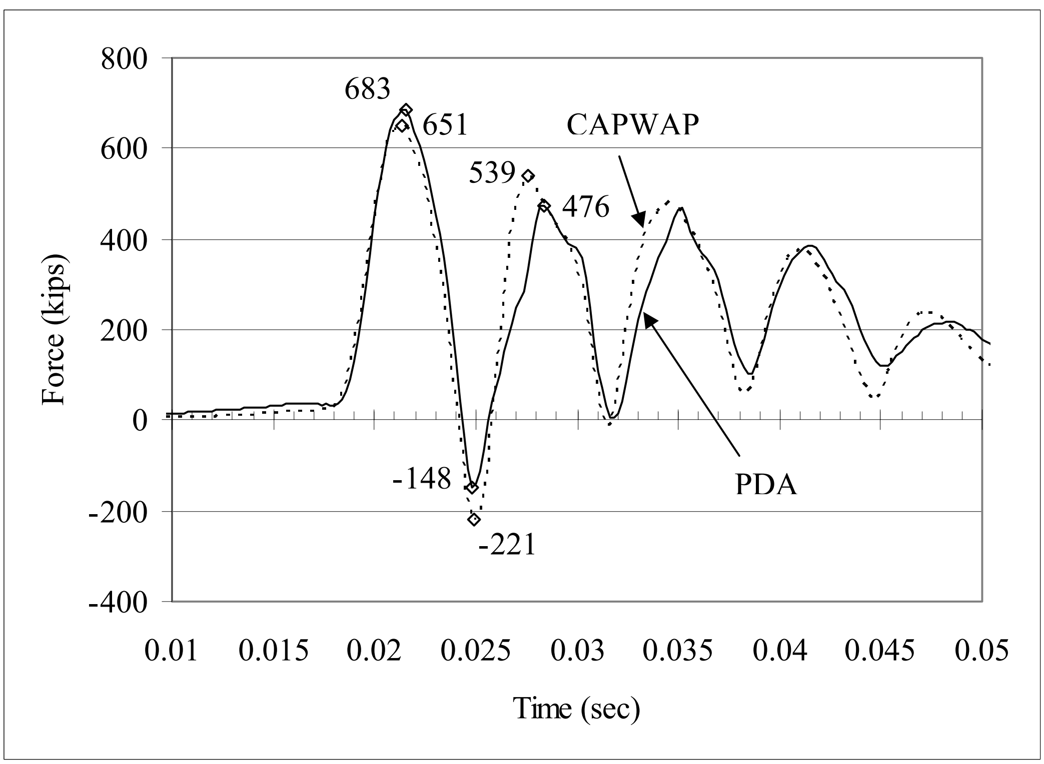


Figure E-9 Pile #2 BN 18 comparison of PDA output and CAPWAP output at the lower gage location.

Table E-5 CAPWAP software output of final results for BN 119 of 383.

**Total CAPWAP Capacity: 216.6; along Shaft 38.9; at Toe 177.7 kips**

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 216.6 | | | | | |
| 1 | 18.1 | 4.2 | 0.0 | 216.6 | 0.0 | 0.00 | 0.00 | 0.000 | 0.070 |
| 2 | 22.2 | 8.2 | 0.0 | 216.6 | 0.0 | 0.00 | 0.00 | 0.000 | 0.070 |
| 3 | 26.2 | 12.2 | 8.0 | 208.6 | 8.0 | 2.01 | 0.20 | 0.286 | 0.070 |
| 4 | 30.1 | 16.1 | 14.7 | 193.9 | 22.7 | 3.75 | 0.37 | 0.286 | 0.070 |
| 5 | 34.0 | 20.1 | 16.2 | 177.7 | 38.9 | 4.13 | 0.41 | 0.286 | 0.070 |
| Avg. Skin | | | 7.8 | | | 1.94 | 0.20 | 0.286 | 0.070 |
| Toe | | | 177.7 | | | | 28.43 | 0.256 | 1.120 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.041 | 0.166 | Smith Type |
| Unloading Quake | (% of loading quake) | 49 | 100 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Unloading Level | (% of Ru) | 0 | | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.060 | |
| Soil Plug Weight | (kips) | | 1.98 | |

CAPWAP match quality: 2.92(Wave Up Match)
Observed: final set = 0.429 in; blow count = 28 b/ft
Computed: final set = 0.230 in; blow count = 52 b/ft

Table E-6 CAPWAP software output of extreme values for BN 119 of 383.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1343.2 | -142.9 | 2.080 | -0.221 | 26.07 | 7.0 | 1.173 |
| 2 | 2.0 | 1340.0 | -146.3 | 2.075 | -0.227 | 26.05 | 6.9 | 1.173 |
| 4 | 4.0 | 1328.3 | -152.6 | 1.511 | -0.174 | 25.99 | 6.9 | 1.172 |
| 6 | 6.0 | 1311.4 | -160.0 | 1.471 | -0.180 | 25.96 | 6.9 | 1.172 |
| 8 | 8.0 | 1289.5 | -181.1 | 1.447 | -0.203 | 25.96 | 6.8 | 1.171 |
| 10 | 10.0 | 1258.4 | -226.5 | 1.412 | -0.254 | 25.96 | 6.7 | 1.171 |
| 12 | 12.1 | 1218.8 | -261.5 | 1.368 | -0.293 | 25.95 | 6.6 | 1.170 |
| 14 | 14.1 | 1168.2 | -294.5 | 1.311 | -0.330 | 25.95 | 6.5 | 1.170 |
| 16 | 16.1 | 1104.9 | -313.8 | 1.240 | -0.352 | 25.95 | 6.4 | 1.169 |
| 18 | 18.1 | 1036.5 | -317.2 | 1.163 | -0.356 | 25.94 | 6.5 | 1.168 |
| 20 | 20.1 | 970.0 | -318.8 | 1.088 | -0.358 | 25.94 | 6.8 | 1.168 |
| 22 | 22.2 | 891.2 | -306.4 | 1.000 | -0.344 | 25.93 | 7.1 | 1.167 |
| 24 | 24.2 | 797.7 | -275.1 | 0.941 | -0.324 | 25.93 | 7.3 | 1.166 |
| 26 | 26.2 | 718.9 | -238.0 | 1.113 | -0.369 | 25.92 | 7.6 | 1.165 |
| 27 | 27.1 | 656.7 | -236.3 | 1.017 | -0.366 | 24.14 | 7.7 | 1.164 |
| 28 | 28.1 | 613.6 | -210.5 | 0.950 | -0.326 | 24.13 | 7.8 | 1.164 |
| 29 | 29.1 | 569.3 | -182.3 | 0.882 | -0.282 | 24.13 | 7.9 | 1.163 |
| 30 | 30.1 | 521.4 | -150.5 | 0.781 | -0.225 | 24.12 | 8.0 | 1.163 |
| 31 | 31.1 | 425.7 | -144.5 | 0.478 | -0.162 | 20.81 | 8.0 | 1.162 |
| 32 | 32.0 | 359.9 | -92.2 | 0.404 | -0.103 | 20.81 | 8.1 | 1.162 |
| 33 | 33.0 | 332.3 | -33.8 | 0.373 | -0.038 | 20.81 | 8.1 | 1.161 |
| 34 | 34.0 | 325.1 | -0.2 | 0.365 | -0.000 | 17.12 | 8.1 | 1.161 |
| Absolute | 1.0 | | | 2.080 | | | (T = | 20.8 ms) |
| | 25.2 | | | | -0.401 | | (T = | 25.0 ms) |

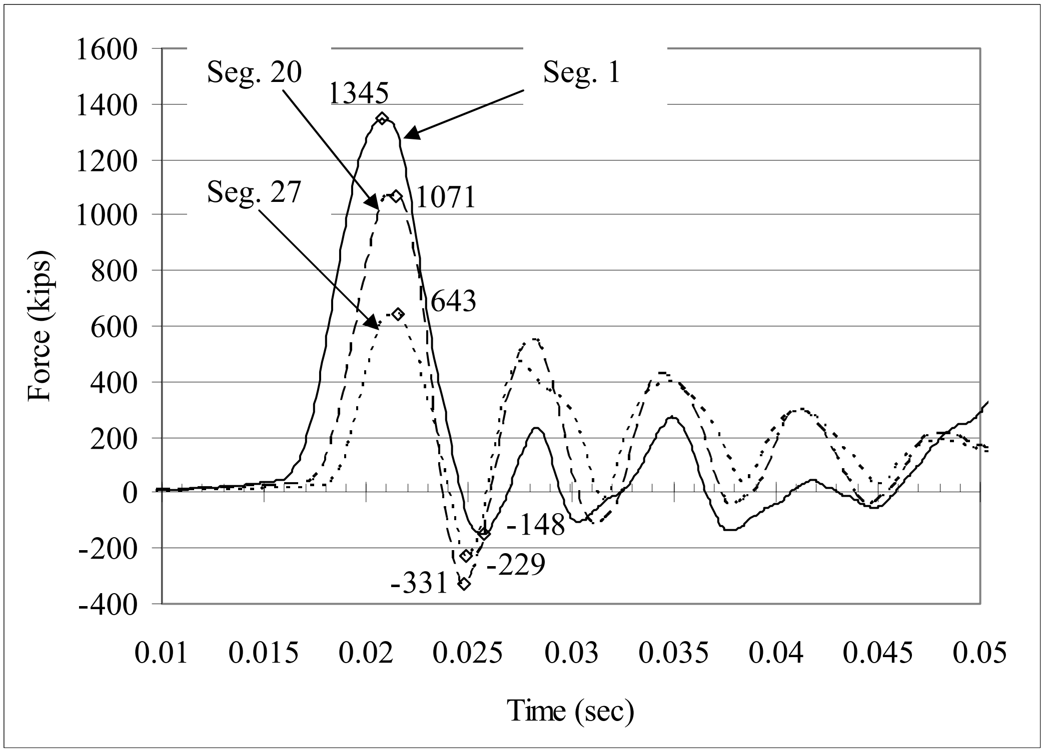


Figure E-10 CAPWAP output of force at three pile segments for BN 119 of 383 of spliced Pile #2.

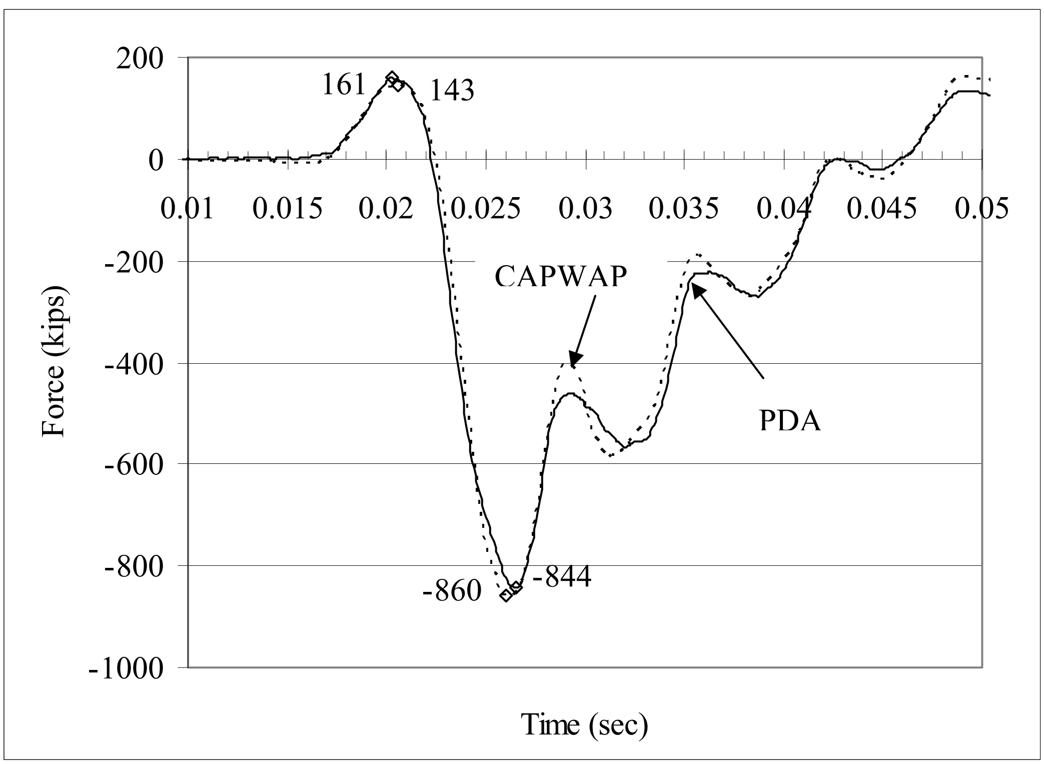


Figure E-11 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #2 for BN 119 of 383.

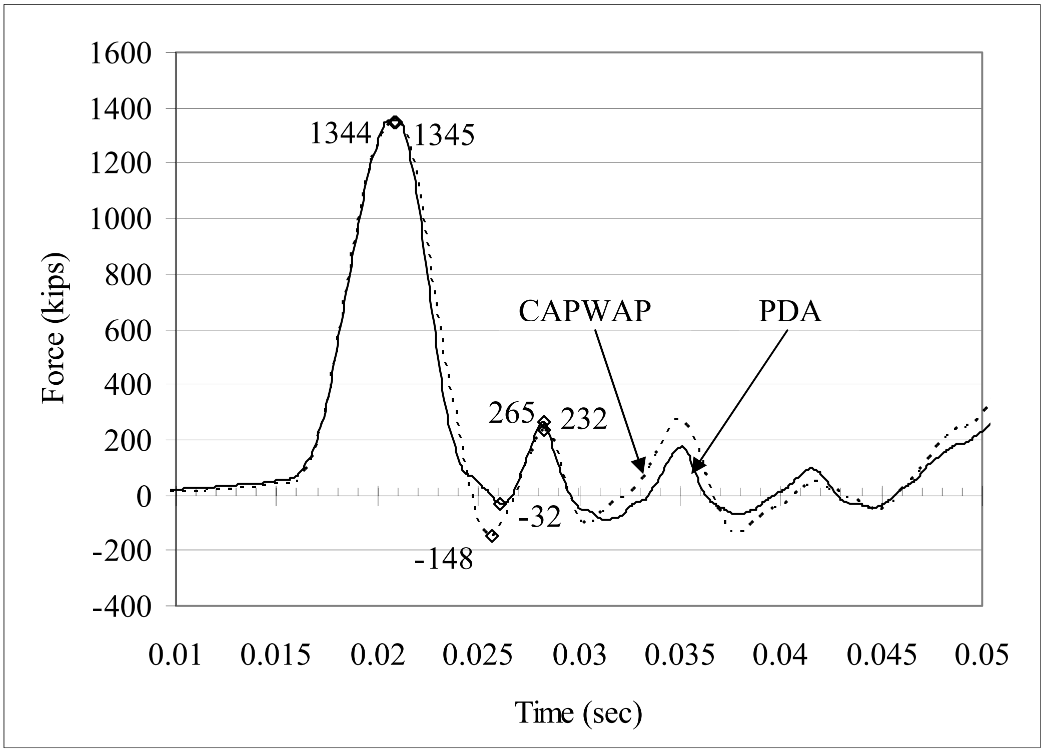


Figure E-12 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #2 for BN 119 of 383.

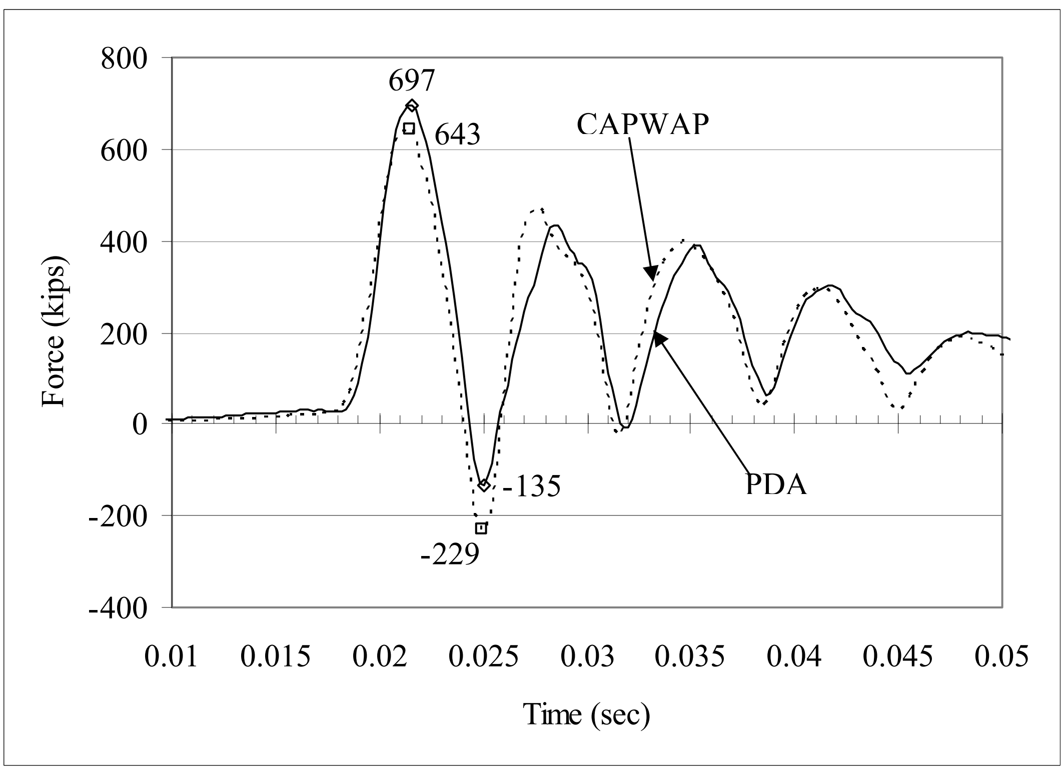


Figure E-13 Pile #2 BN 119 comparison of PDA output and CAPWAP output at the lower gage location.

Table E-7 CAPWAP output of final results for BN 227 of 383.

**Total CAPWAP Capacity: 265.9; along Shaft 56.4; at Toe 209.5 kips**

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 265.9 | | | | | |
| 1 | 31.1 | 23.8 | 0.0 | 265.9 | 0.0 | 0.00 | 0.00 | 0.000 | 0.130 |
| 2 | 32.0 | 24.8 | 0.5 | 265.4 | 0.5 | 0.51 | 0.05 | 0.141 | 0.130 |
| 3 | 33.0 | 25.7 | 15.4 | 250.0 | 15.9 | 15.69 | 1.57 | 0.141 | 0.130 |
| 4 | 34.0 | 26.7 | 40.5 | 209.5 | 56.4 | 41.26 | 4.13 | 0.141 | 0.150 |
| Avg. Skin | | | 14.1 | | | 2.11 | 1.44 | 0.141 | 0.144 |
| Toe | | | 209.5 | | | | 33.52 | 0.368 | 0.890 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.029 | 0.282 | Smith Type |
| Unloading Quake | (% of loading quake) | 25 | 20 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Unloading Level | (% of Ru) | 93 | | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.030 | |
| Soil Plug Weight | (kips) | | 2.34 | |
| Soil Support Dashpot | | 0.090 | 0.000 | |
| Soil Support Weight | (kips) | 2.10 | 0.00 | |

CAPWAP match quality: 2.24(Wave Up Match)
Observed: final set = 0.667 in; blow count = 18 b/ft
Computed: final set = 0.703 in; blow count = 17 b/ft

Table E-8 CAPWAP output of extreme values for BN 227 of 383.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1462.9 | -172.3 | 2.266 | -0.267 | 28.56 | 6.9 | 0.951 |
| 2 | 2.0 | 1458.1 | -182.1 | 2.258 | -0.282 | 28.54 | 6.8 | 0.951 |
| 4 | 3.9 | 1446.0 | -186.3 | 2.239 | -0.288 | 28.52 | 6.7 | 0.951 |
| 6 | 6.0 | 1424.4 | -199.4 | 1.598 | -0.224 | 28.48 | 6.7 | 0.950 |
| 8 | 8.0 | 1395.9 | -237.7 | 1.566 | -0.267 | 28.44 | 6.6 | 0.950 |
| 10 | 10.0 | 1359.9 | -278.7 | 1.526 | -0.313 | 28.39 | 6.5 | 0.949 |
| 12 | 12.0 | 1314.8 | -299.8 | 1.475 | -0.336 | 28.35 | 6.4 | 0.949 |
| 14 | 14.1 | 1253.9 | -315.9 | 1.407 | -0.354 | 28.31 | 6.3 | 0.948 |
| 16 | 16.1 | 1186.6 | -319.7 | 1.331 | -0.359 | 28.27 | 6.6 | 0.947 |
| 18 | 18.1 | 1124.0 | -298.2 | 1.261 | -0.335 | 28.22 | 6.9 | 0.947 |
| 20 | 20.1 | 1051.6 | -286.6 | 1.180 | -0.322 | 28.18 | 7.1 | 0.946 |
| 22 | 22.2 | 969.4 | -261.7 | 1.088 | -0.294 | 28.14 | 7.4 | 0.945 |
| 24 | 24.2 | 871.1 | -215.6 | 1.024 | -0.254 | 28.10 | 7.6 | 0.945 |
| 26 | 26.1 | 788.3 | -165.8 | 1.221 | -0.257 | 28.07 | 7.8 | 0.943 |
| 27 | 27.1 | 748.6 | -138.8 | 1.159 | -0.215 | 28.05 | 7.8 | 0.943 |
| 28 | 28.1 | 707.6 | -112.9 | 1.096 | -0.175 | 28.04 | 7.9 | 0.942 |
| 29 | 29.1 | 662.2 | -82.5 | 1.026 | -0.128 | 28.02 | 8.0 | 0.941 |
| 30 | 30.1 | 613.4 | -52.2 | 0.919 | -0.078 | 28.00 | 8.0 | 0.941 |
| 31 | 31.1 | 586.9 | -51.4 | 0.652 | -0.057 | 28.00 | 8.0 | 0.940 |
| 32 | 32.0 | 570.9 | -51.1 | 0.634 | -0.057 | 27.99 | 8.1 | 0.939 |
| 33 | 33.0 | 553.3 | -50.3 | 0.615 | -0.056 | 27.92 | 8.1 | 0.939 |
| 34 | 34.0 | 510.1 | -33.9 | 0.567 | -0.038 | 21.69 | 8.1 | 0.938 |
| Absolute | 1.0 | | | 2.266 | | | (T = | 20.8 ms) |
| | 15.1 | | | | -0.362 | | (T = | 24.4 ms) |

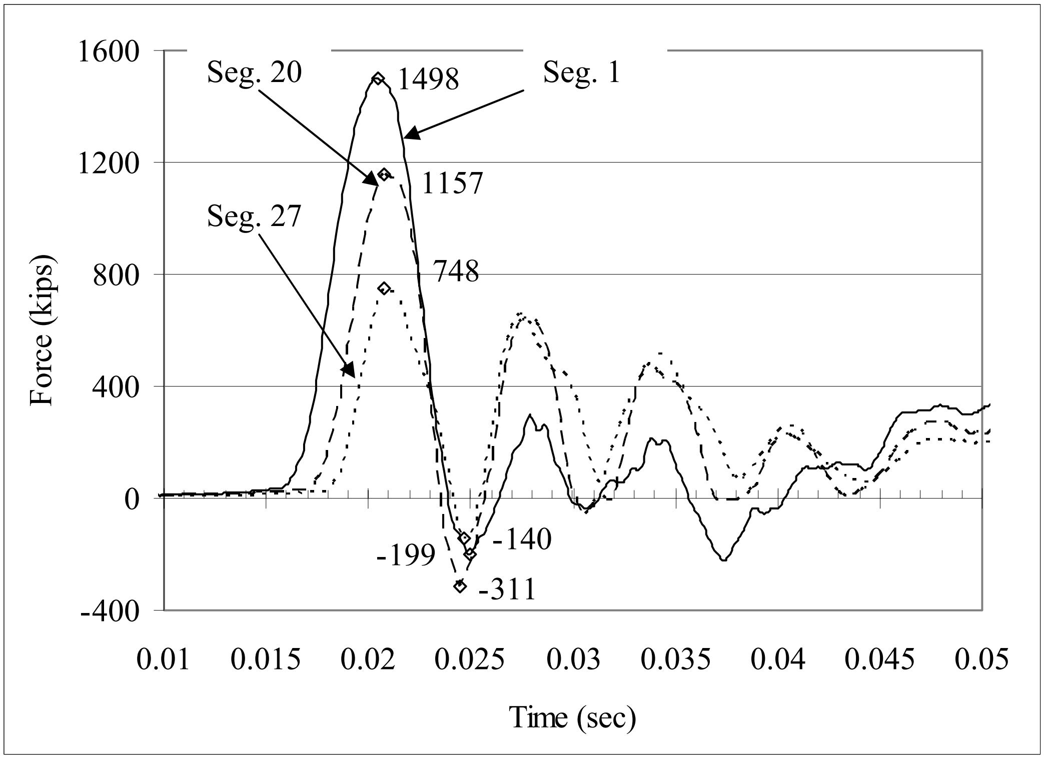


Figure E-14 CAPWAP output of force at three pile segments for BN 227 of 383 with maximum tensile force for spliced Pile #2.

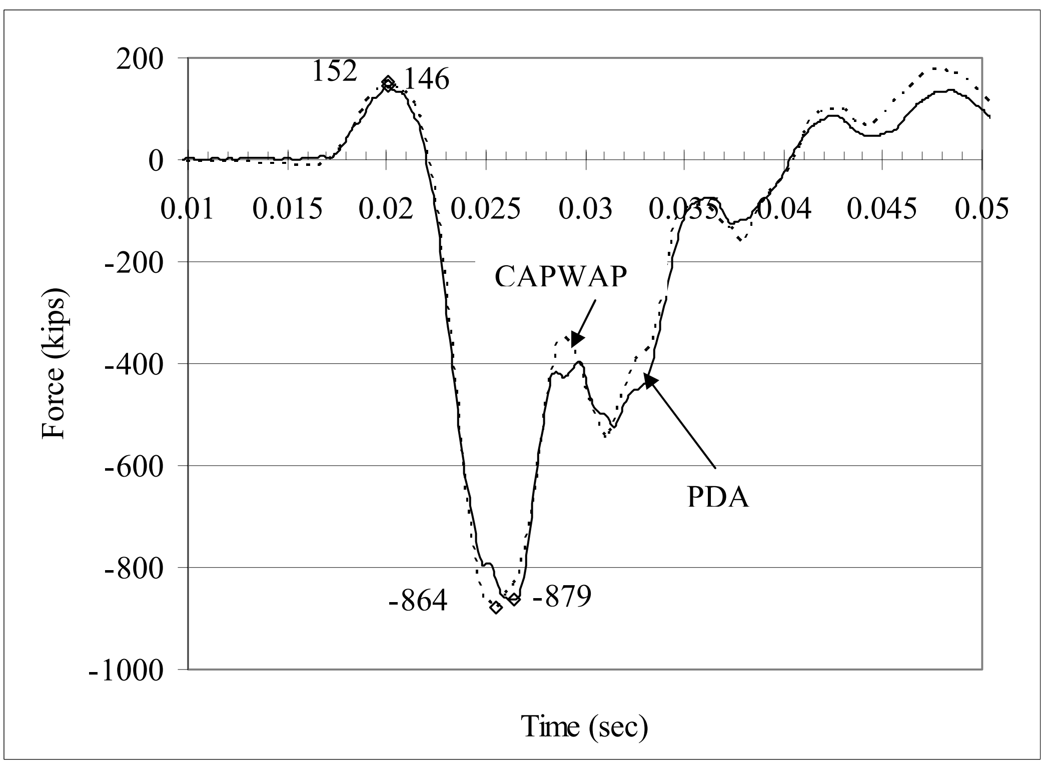


Figure E-15 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #2 for BN 227 of 383.

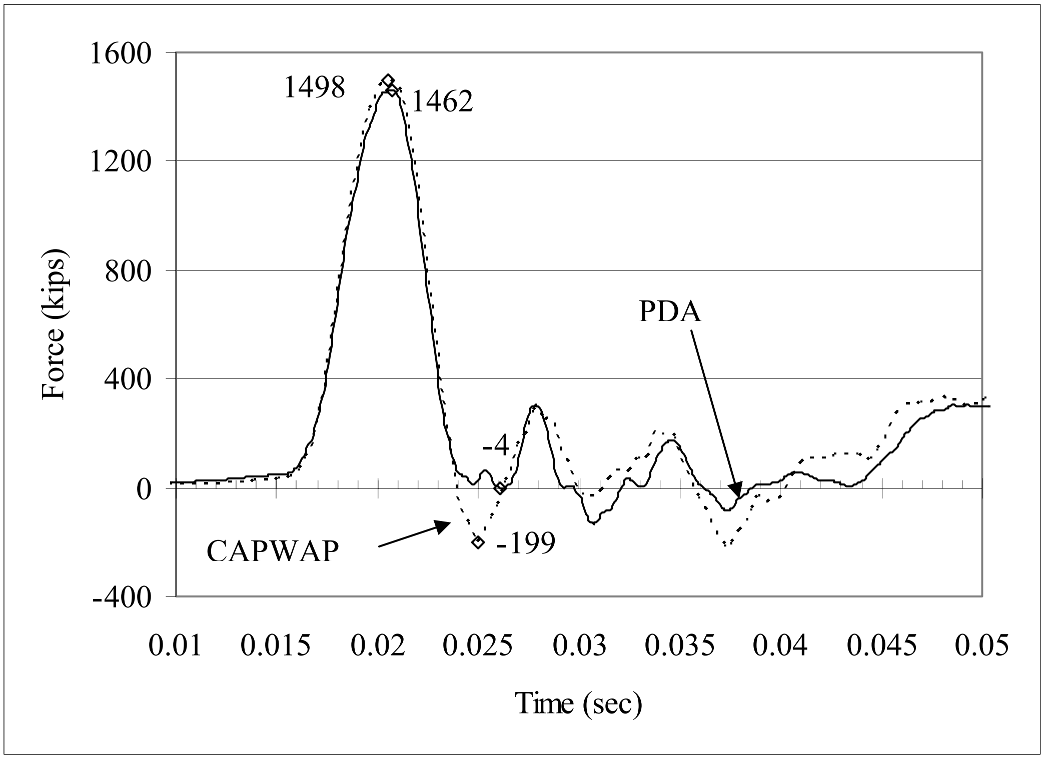


Figure E-16 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #2 for BN 227 of 383.

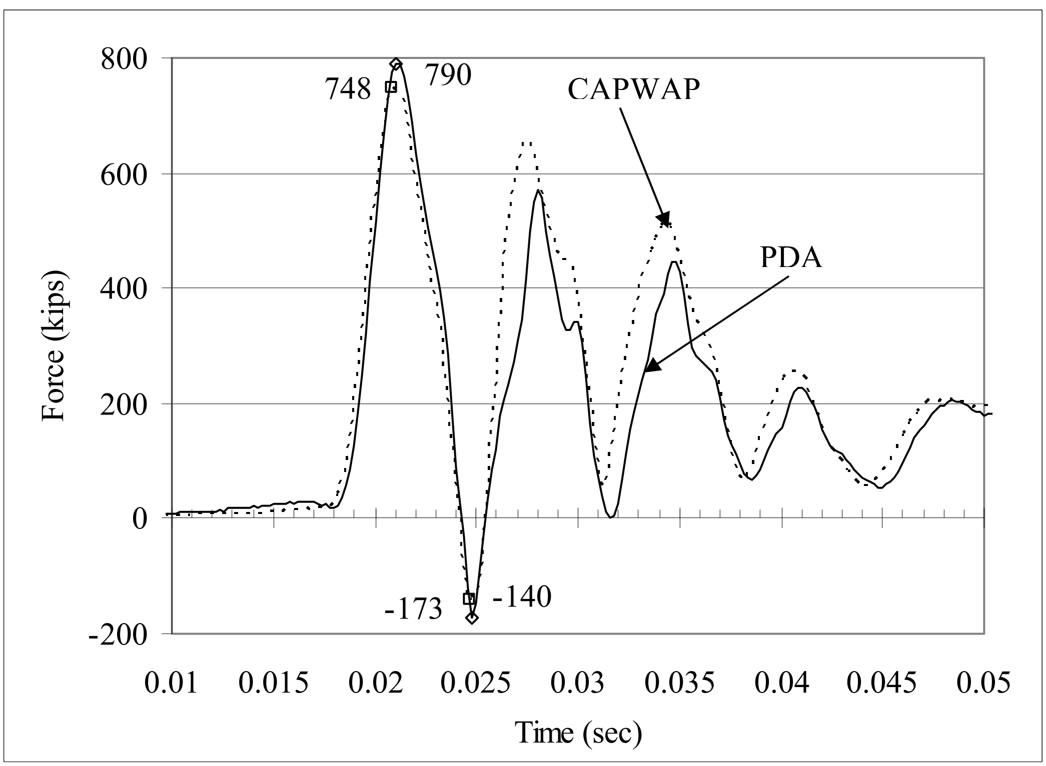


Figure E-17 Pile #2 BN 227 comparison of PDA output and CAPWAP output at the lower gage location.

# APPENDIX F
# CAPWAP OUTPUT FOR COMPRESSIVE FORCES

Appendix F contains figures showing a comparison between the PDA output and the CAPWAP output for Pile #1 blow numbers 116, 117, 154, and 155, which were the hammer impacts that caused high compressive stresses. The figures included for each blow number are:

- CAPWAP computed force at top, middle, and segment 27 of pile versus time.
- PDA measured force at top of pile and CAPWAP computed force at top of pile versus time.
- PDA measured wave up at top of pile and CAPWAP computed wave up at top of pile versus time.
- PDA measured force at lower gage and CAPWAP computed force at segment 27 versus time.

The maximum value table output from CAPWAP was also included because it shows the maximum force in each pile segment defined in Figure F-1 below.

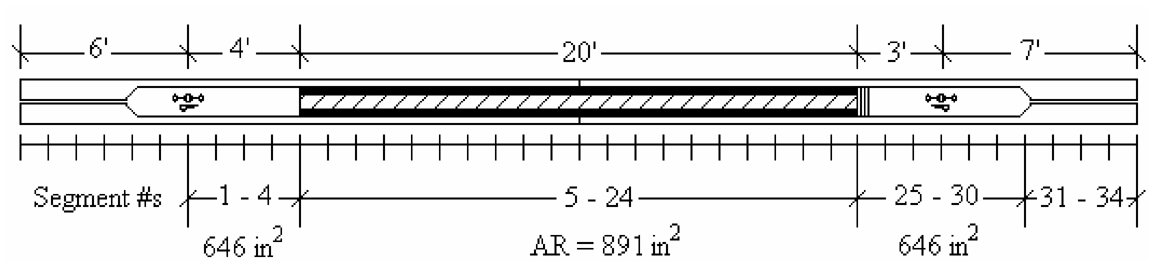


Figure F-1 Pile divided into 1 foot long segments for CAPWAP software.

Table F-1 CAPWAP output of final results for BN 116 of 183.

**Total CAPWAP Capacity: 782.0; along Shaft 466.9; at Toe 315.1 kips**

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 782.0 | | | | | |
| 1 | 18.1 | 18.1 | 164.6 | 617.4 | 164.6 | 40.65 | 4.07 | 0.193 | 0.040 |
| 2 | 22.2 | 22.2 | 131.9 | 485.5 | 296.5 | 32.58 | 3.26 | 0.193 | 0.040 |
| 3 | 26.2 | 26.2 | 78.2 | 407.3 | 374.7 | 19.65 | 1.96 | 0.193 | 0.090 |
| 4 | 30.1 | 30.1 | 50.9 | 356.4 | 425.6 | 12.97 | 1.30 | 0.193 | 0.050 |
| 5 | 34.0 | 34.0 | 41.3 | 315.1 | 466.9 | 10.53 | 1.05 | 0.193 | 0.445 |
| Avg. Skin | | | 93.4 | | | 13.73 | 2.34 | 0.193 | 0.085 |
| Toe | | | 315.1 | | | | 50.42 | 0.400 | 0.420 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.330 | 0.461 | Smith Type |
| Unloading Quake | (% of loading quake) | 19 | 66 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Unloading Level | (% of Ru) | 33 | | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.040 | |
| Soil Plug Weight | (kips) | | 2.69 | |

CAPWAP match quality: 2.29(Wave Up Match)

Observed: final set = 0.414 in; blow count = 29 b/ft

Computed: final set = 0.364 in; blow count = 33 b/ft

Table F-2 CAPWAP output of extreme values for BN 116 of 183.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1771.4 | -0.6 | 2.743 | -0.001 | 42.88 | 4.4 | 0.498 |
| 2 | 2.0 | 1776.4 | -0.5 | 2.751 | -0.001 | 42.85 | 4.3 | 0.496 |
| 4 | 4.0 | 1778.8 | -3.3 | 2.023 | -0.004 | 42.80 | 4.1 | 0.493 |
| 6 | 6.0 | 1771.5 | -3.9 | 1.988 | -0.004 | 42.75 | 4.1 | 0.491 |
| 8 | 8.0 | 1761.8 | -1.8 | 1.977 | -0.002 | 42.70 | 4.2 | 0.488 |
| 10 | 10.0 | 1748.8 | -13.2 | 1.962 | -0.015 | 42.65 | 4.3 | 0.486 |
| 12 | 12.1 | 1722.6 | -17.9 | 1.933 | -0.020 | 42.59 | 4.4 | 0.484 |
| 14 | 14.1 | 1683.2 | -15.3 | 1.889 | -0.017 | 42.53 | 4.6 | 0.481 |
| 16 | 16.1 | 1642.4 | -13.8 | 1.843 | -0.015 | 42.50 | 4.8 | 0.479 |
| 18 | 18.1 | 1589.8 | -7.6 | 1.784 | -0.009 | 42.46 | 5.0 | 0.477 |
| 20 | 20.1 | 1223.5 | 0.0 | 1.373 | 0.000 | 32.13 | 5.2 | 0.476 |
| 22 | 22.2 | 1151.4 | 0.0 | 1.292 | 0.000 | 32.10 | 5.4 | 0.475 |
| 24 | 24.2 | 943.3 | 0.0 | 1.112 | 0.000 | 23.80 | 5.5 | 0.473 |
| 26 | 26.2 | 948.5 | 0.0 | 1.469 | 0.000 | 23.77 | 5.6 | 0.471 |
| 27 | 27.1 | 830.3 | 0.0 | 1.286 | 0.000 | 19.02 | 5.6 | 0.470 |
| 28 | 28.1 | 828.0 | 0.0 | 1.282 | 0.000 | 19.01 | 5.7 | 0.469 |
| 29 | 29.1 | 823.5 | 0.0 | 1.275 | 0.000 | 19.00 | 5.7 | 0.468 |
| 30 | 30.1 | 815.9 | 0.0 | 1.222 | 0.000 | 18.99 | 5.8 | 0.467 |
| 31 | 31.1 | 724.9 | -0.2 | 0.813 | -0.000 | 15.76 | 5.8 | 0.467 |
| 32 | 32.0 | 715.2 | -0.3 | 0.803 | -0.000 | 15.76 | 5.8 | 0.466 |
| 33 | 33.0 | 707.6 | -0.4 | 0.794 | -0.000 | 15.75 | 5.8 | 0.465 |
| 34 | 34.0 | 696.0 | -0.3 | 0.781 | -0.000 | 13.84 | 5.8 | 0.465 |
| Absolute | 2.9 | | | 2.756 | | | (T = | 21.1 ms) |
| | 12.1 | | | | -0.020 | | (T = | 37.4 ms) |

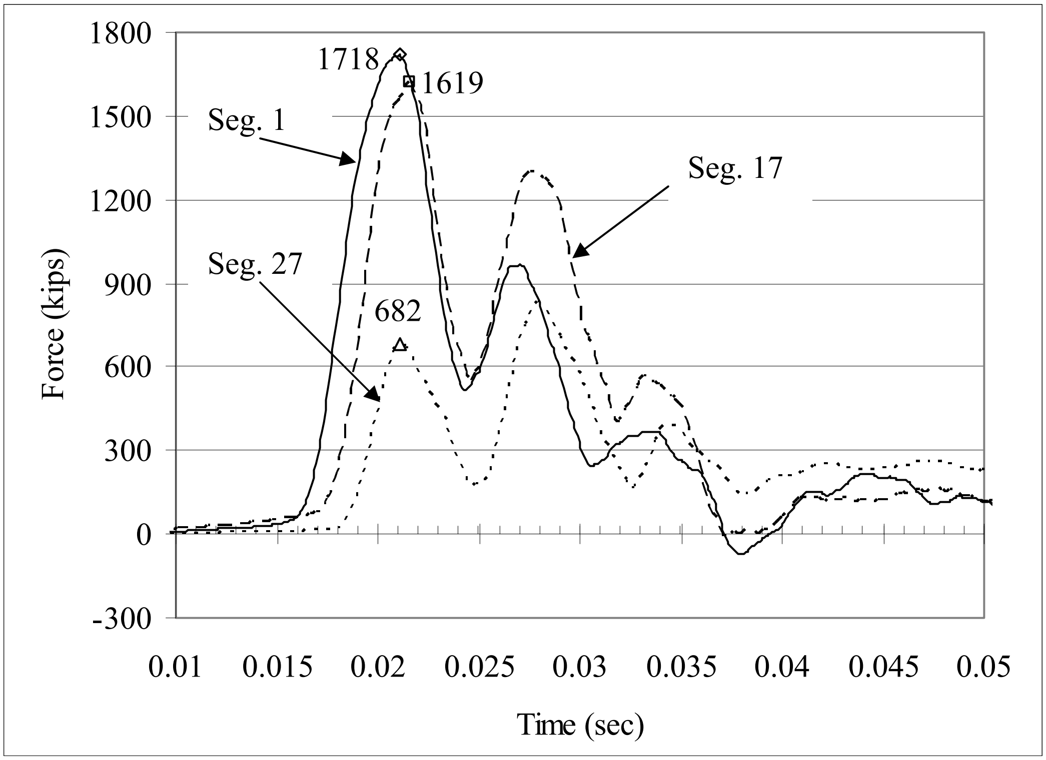


Figure F-2 CAPWAP output of force at three pile segments for BN 116 of 183.

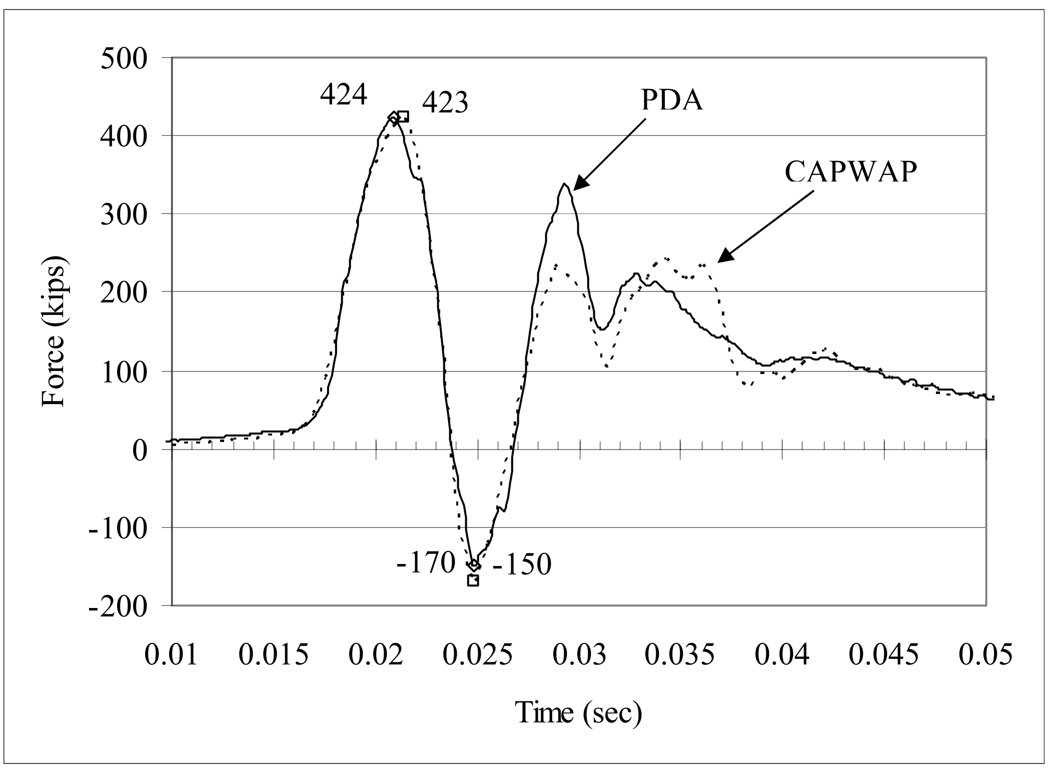


Figure F-3 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #1 for BN 116 of 183.

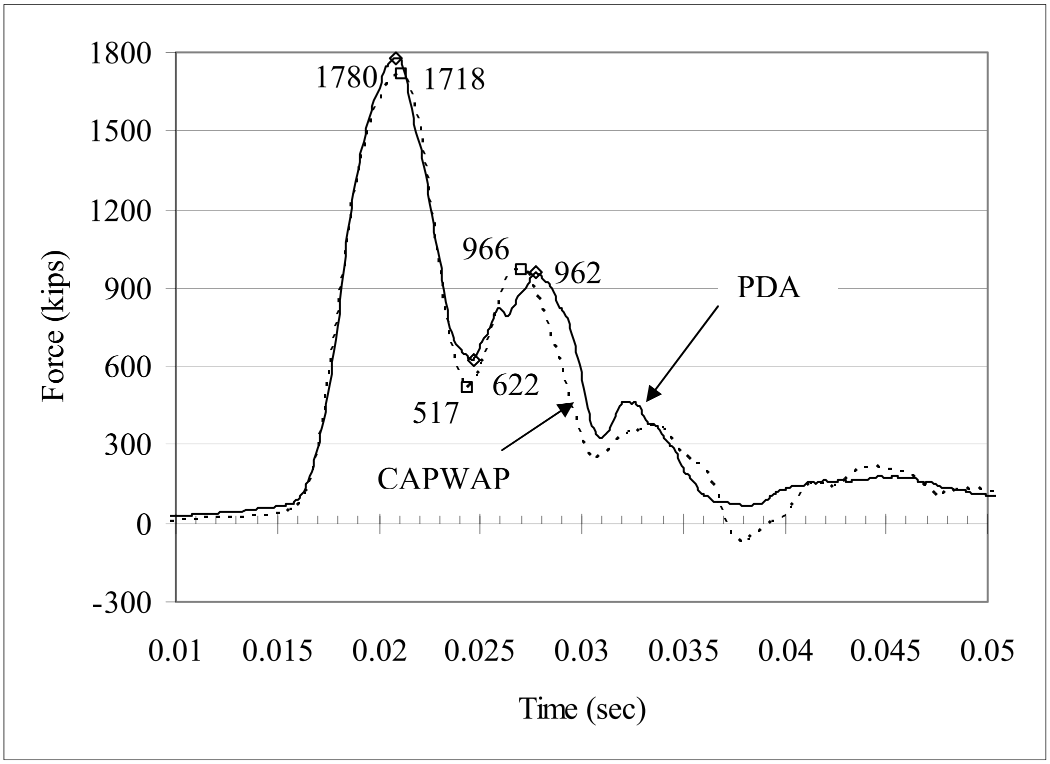


Figure F-4 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #1 for BN 116 of 183.

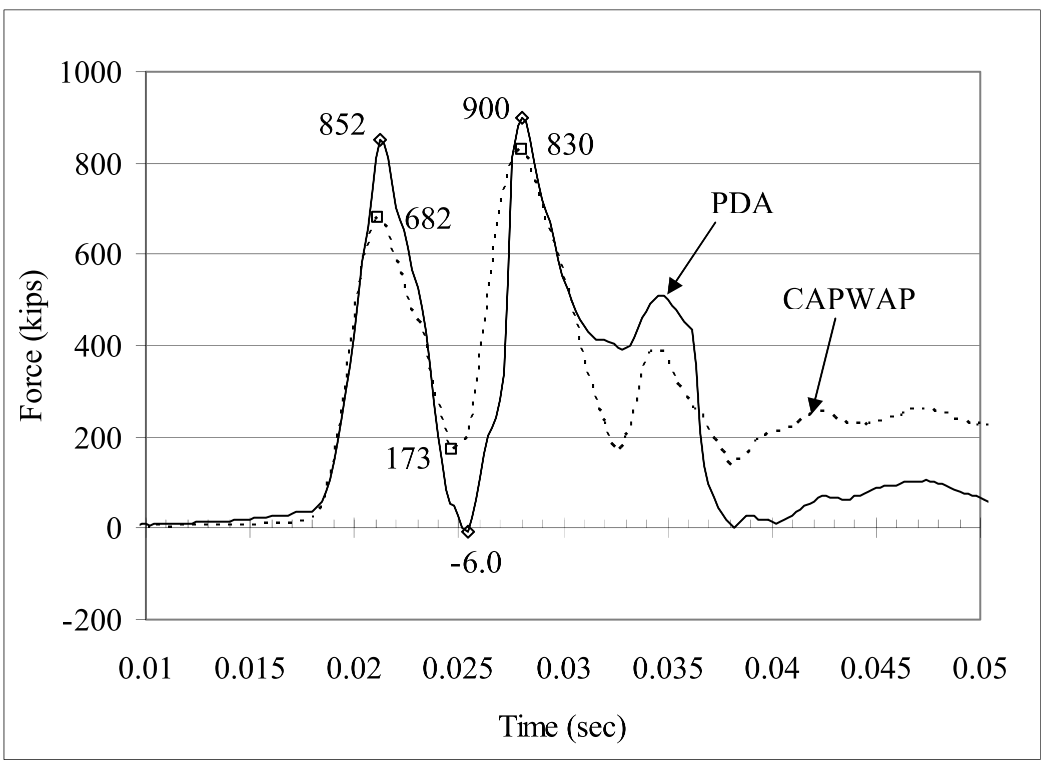


Figure F-5 BN 116 of Pile #1 Comparison of PDA output and CAPWAP output at the lower gage location.

Table F-3 CAPWAP output of final results for BN 117 of 183.

**Total CAPWAP Capacity: 698.6; along Shaft 356.2; at Toe 342.4 kips**

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 698.6 | | | | | |
| 1 | 18.1 | 18.1 | 189.3 | 509.3 | 189.3 | 46.71 | 4.67 | 0.209 | 0.120 |
| 2 | 22.2 | 22.2 | 106.2 | 403.1 | 295.5 | 26.20 | 2.62 | 0.209 | 0.120 |
| 3 | 26.1 | 26.1 | 22.3 | 380.8 | 317.8 | 5.60 | 0.56 | 0.209 | 0.120 |
| 4 | 30.1 | 30.1 | 19.2 | 361.6 | 337.0 | 4.89 | 0.49 | 0.209 | 0.120 |
| 5 | 34.0 | 34.0 | 19.2 | 342.4 | 356.2 | 4.89 | 0.49 | 0.209 | 0.120 |
| Avg. Skin | | | 71.2 | | | 10.48 | 1.79 | 0.209 | 0.120 |
| Toe | | | 342.4 | | | | 54.79 | 0.363 | 0.470 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.272 | 0.454 | Smith Type |
| Unloading Quake | (% of loading quake) | 10 | 33 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Unloading Level | (% of Ru) | 34 | | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.010 | |
| Soil Plug Weight | (kips) | | 3.13 | |

CAPWAP match quality: 2.73(Wave Up Match)

Observed: final set = 0.414 in; blow count = 29 b/ft

Computed: final set = 0.365 in; blow count = 33 b/ft

Table F-4 CAPWAP output of extreme values for BN 117 of 183.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1682.9 | -15.5 | 2.606 | -0.024 | 36.20 | 4.2 | 0.490 |
| 2 | 2.0 | 1685.3 | -14.9 | 2.610 | -0.023 | 36.18 | 4.1 | 0.489 |
| 4 | 3.9 | 1685.3 | -17.7 | 2.610 | -0.027 | 36.16 | 4.1 | 0.487 |
| 6 | 6.0 | 1680.1 | -24.4 | 1.885 | -0.027 | 36.14 | 4.1 | 0.485 |
| 8 | 8.0 | 1670.4 | -32.4 | 1.874 | -0.036 | 36.11 | 4.1 | 0.483 |
| 10 | 10.0 | 1657.2 | -42.7 | 1.859 | -0.048 | 36.08 | 4.2 | 0.482 |
| 12 | 12.0 | 1634.3 | -47.9 | 1.834 | -0.054 | 36.04 | 4.3 | 0.480 |
| 14 | 14.1 | 1598.8 | -49.0 | 1.794 | -0.055 | 36.01 | 4.5 | 0.478 |
| 16 | 16.1 | 1562.5 | -48.9 | 1.753 | -0.055 | 35.99 | 4.8 | 0.477 |
| 18 | 18.1 | 1514.6 | -38.9 | 1.699 | -0.044 | 35.98 | 5.1 | 0.475 |
| 20 | 20.1 | 1122.7 | 0.0 | 1.260 | 0.000 | 24.55 | 5.3 | 0.475 |
| 22 | 22.2 | 1067.5 | 0.0 | 1.198 | 0.000 | 24.55 | 5.4 | 0.474 |
| 24 | 24.2 | 853.0 | 0.0 | 1.003 | 0.000 | 18.01 | 5.5 | 0.474 |
| 26 | 26.1 | 798.7 | 0.0 | 1.237 | 0.000 | 18.01 | 5.6 | 0.473 |
| 27 | 27.1 | 737.7 | 0.0 | 1.142 | 0.000 | 16.61 | 5.6 | 0.472 |
| 28 | 28.1 | 711.3 | 0.0 | 1.102 | 0.000 | 16.60 | 5.7 | 0.472 |
| 29 | 29.1 | 704.7 | 0.0 | 1.091 | 0.000 | 16.60 | 5.7 | 0.471 |
| 30 | 30.1 | 697.1 | 0.0 | 1.045 | 0.000 | 16.59 | 5.7 | 0.470 |
| 31 | 31.1 | 655.9 | 0.0 | 0.729 | 0.000 | 15.37 | 5.7 | 0.470 |
| 32 | 32.0 | 646.6 | -0.2 | 0.718 | -0.000 | 15.37 | 5.7 | 0.469 |
| 33 | 33.0 | 638.7 | -0.5 | 0.709 | -0.001 | 15.36 | 5.7 | 0.469 |
| 34 | 34.0 | 628.0 | -0.6 | 0.698 | -0.001 | 14.17 | 5.7 | 0.468 |
| Absolute | 2.9 | | | 2.611 | | | (T = | 21.1 ms) |
| | 15.1 | | | | -0.056 | | (T = | 37.1 ms) |

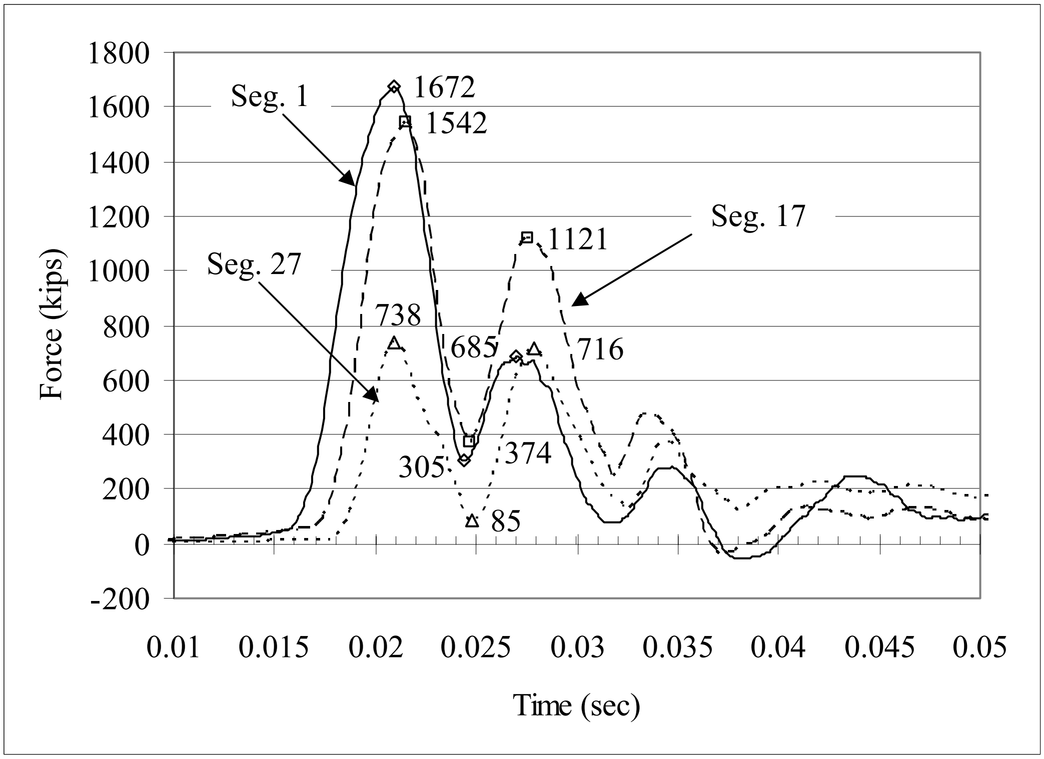


Figure F-6 CAPWAP output of force at three pile segments for BN 117 of 183

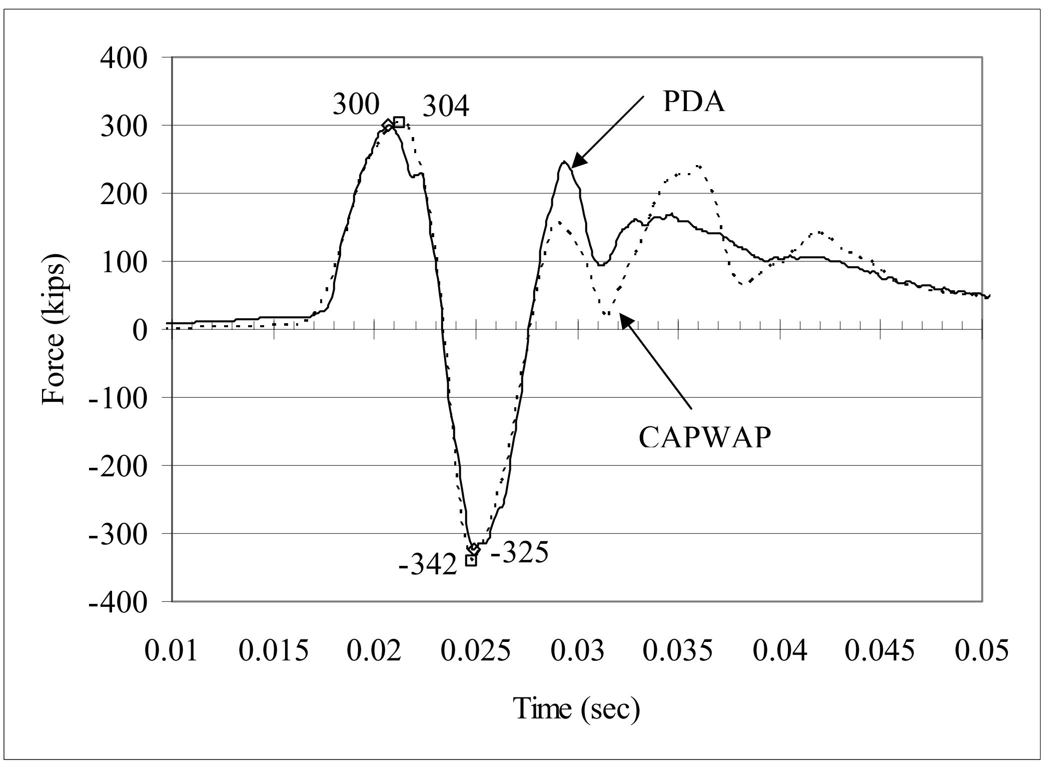


Figure F-7 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #1 for BN 117 of 183.

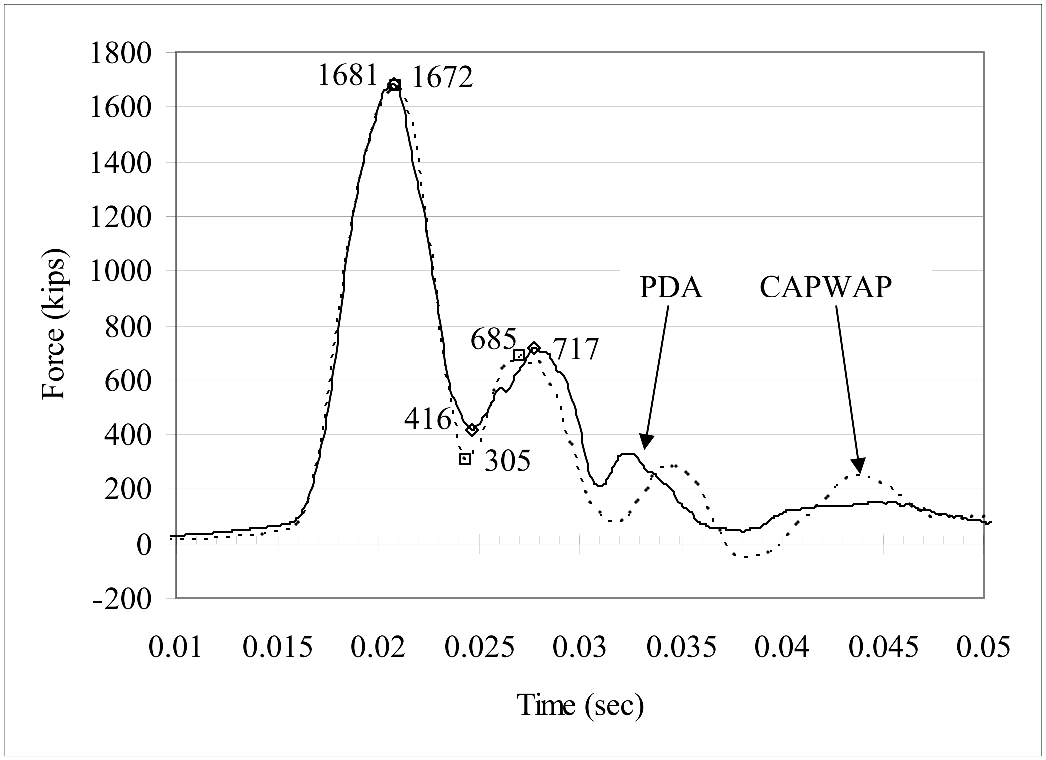


Figure F-8 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #1 for BN 117 of 183.

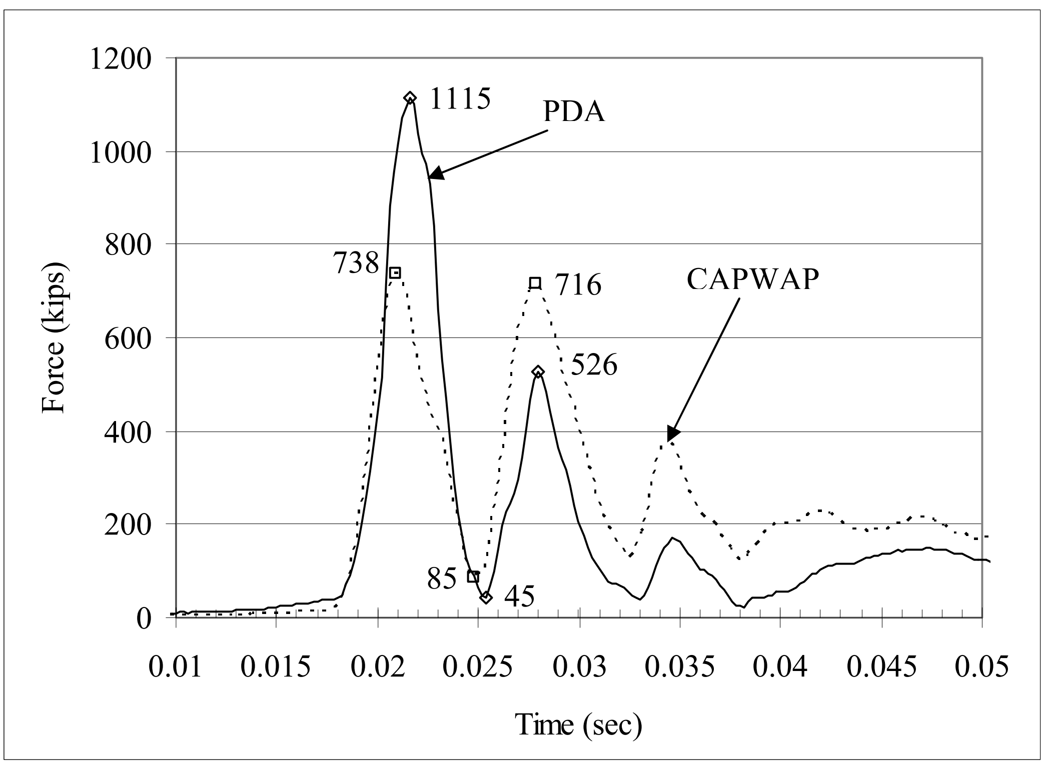


Figure F-9 Pile #1 BN 117 Comparison of PDA output and CAPWAP output at the lower gage location.

Table F-5 CAPWAP software output of final results for BN 154 of 183.

**Total CAPWAP Capacity: 950.6; along Shaft 251.7; at Toe 698.9 kips**

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 950.6 | | | | | |
| 1 | 10.0 | 10.0 | 97.7 | 853.0 | 97.7 | 16.07 | 1.61 | 0.154 | 0.120 |
| 2 | 16.1 | 16.1 | 48.3 | 804.7 | 146.0 | 7.95 | 0.79 | 0.154 | 0.120 |
| 3 | 22.2 | 22.2 | 53.0 | 751.6 | 199.0 | 8.72 | 0.87 | 0.154 | 0.120 |
| 4 | 28.1 | 28.1 | 6.3 | 745.3 | 205.3 | 1.06 | 0.11 | 0.154 | 0.120 |
| 5 | 34.0 | 34.0 | 46.4 | 698.9 | 251.7 | 7.88 | 0.79 | 0.154 | 0.100 |
| Avg. Skin | | | 50.3 | | | 7.40 | 0.84 | 0.154 | 0.116 |
| Toe | | | 698.9 | | | | 111.82 | 0.188 | 0.370 |

| Soil Model Parameters/Extensions | | Skin | Toe | |
|---|---|---|---|---|
| Case Damping Factor | | 0.142 | 0.481 | Smith Type |
| Unloading Quake | (% of loading quake) | 6 | 70 | |
| Reloading Level | (% of Ru) | 100 | 100 | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.030 | |
| Soil Plug Weight | (kips) | | 2.60 | |

CAPWAP match quality: 2.23(Wave Up Match)
Observed: final set = 0.343 in; blow count = 35 b/ft
Computed: final set = 0.357 in; blow count = 34 b/ft

Table F-6 CAPWAP software output of extreme values for BN 154 of 183.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1478.1 | 0.0 | 2.289 | 0.000 | 33.77 | 4.3 | 0.473 |
| 2 | 2.0 | 1481.6 | 0.0 | 2.295 | 0.000 | 33.70 | 4.1 | 0.470 |
| 4 | 3.9 | 1483.8 | 0.0 | 2.298 | 0.000 | 33.57 | 4.1 | 0.465 |
| 6 | 6.0 | 1478.6 | 0.0 | 1.659 | 0.000 | 33.51 | 4.1 | 0.462 |
| 8 | 8.0 | 1467.8 | 0.0 | 1.647 | 0.000 | 33.46 | 4.2 | 0.460 |
| 10 | 10.0 | 1452.4 | 0.0 | 1.630 | 0.000 | 33.40 | 4.4 | 0.457 |
| 12 | 12.0 | 1266.9 | 0.0 | 1.421 | 0.000 | 28.38 | 4.5 | 0.455 |
| 14 | 14.1 | 1226.2 | 0.0 | 1.376 | 0.000 | 28.33 | 4.7 | 0.452 |
| 16 | 16.1 | 1177.5 | 0.0 | 1.321 | 0.000 | 28.27 | 5.0 | 0.450 |
| 18 | 18.1 | 1110.8 | 0.0 | 1.246 | 0.000 | 25.77 | 5.3 | 0.447 |
| 20 | 20.1 | 1130.8 | 0.0 | 1.269 | 0.000 | 25.72 | 5.5 | 0.445 |
| 22 | 22.2 | 1140.2 | 0.0 | 1.279 | 0.000 | 25.67 | 5.6 | 0.442 |
| 24 | 24.2 | 1077.7 | 0.0 | 1.267 | 0.000 | 22.93 | 5.6 | 0.440 |
| 26 | 26.1 | 1086.6 | 0.0 | 1.683 | 0.000 | 22.84 | 5.7 | 0.436 |
| 27 | 27.1 | 1086.5 | 0.0 | 1.683 | 0.000 | 22.80 | 5.7 | 0.433 |
| 28 | 28.1 | 1083.2 | 0.0 | 1.678 | 0.000 | 22.75 | 5.8 | 0.431 |
| 29 | 29.1 | 1070.0 | 0.0 | 1.657 | 0.000 | 22.38 | 5.8 | 0.429 |
| 30 | 30.1 | 1065.2 | 0.0 | 1.596 | 0.000 | 22.34 | 5.8 | 0.427 |
| 31 | 31.1 | 1063.1 | 0.0 | 1.181 | 0.000 | 22.30 | 5.8 | 0.426 |
| 32 | 32.0 | 1056.5 | 0.0 | 1.174 | 0.000 | 22.26 | 5.7 | 0.424 |
| 33 | 33.0 | 1046.9 | 0.0 | 1.163 | 0.000 | 22.22 | 5.7 | 0.422 |
| 34 | 34.0 | 1030.1 | 0.0 | 1.144 | 0.000 | 20.03 | 5.7 | 0.421 |
| Absolute | 3.9 | | | 2.298 | | | (T = | 20.8 ms) |
| | 1.0 | | | | 0.000 | | (T = | 0.0 ms) |

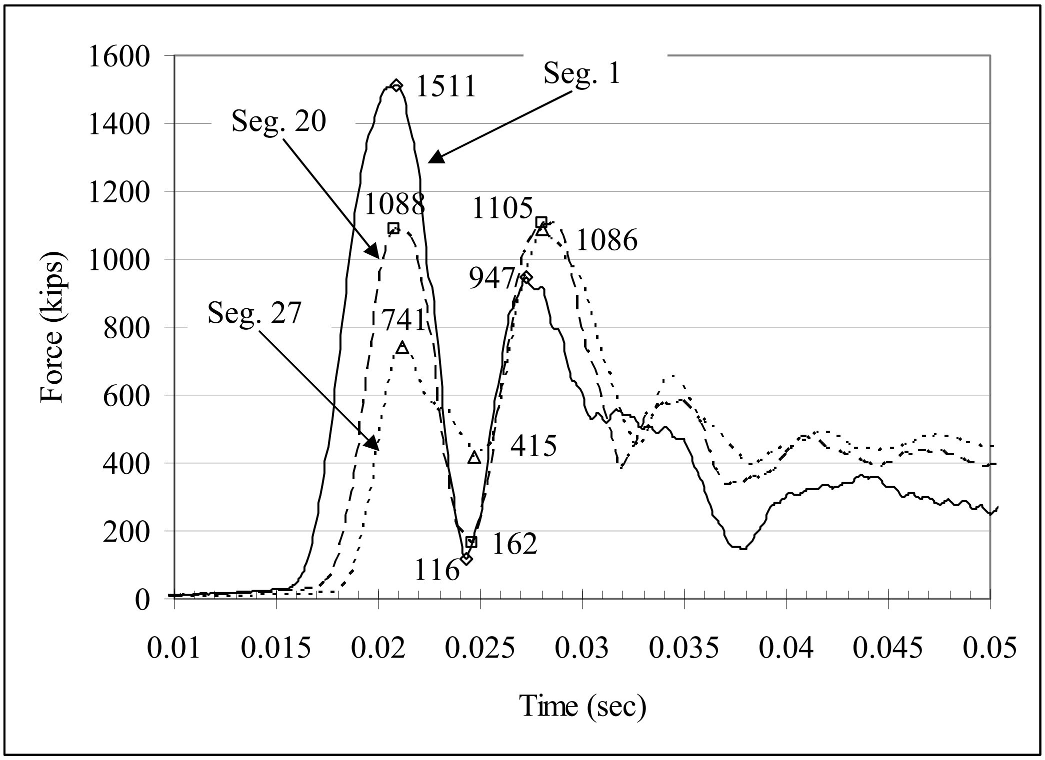


Figure F-10 CAPWAP output of force at three pile segments for BN 154 of 183.

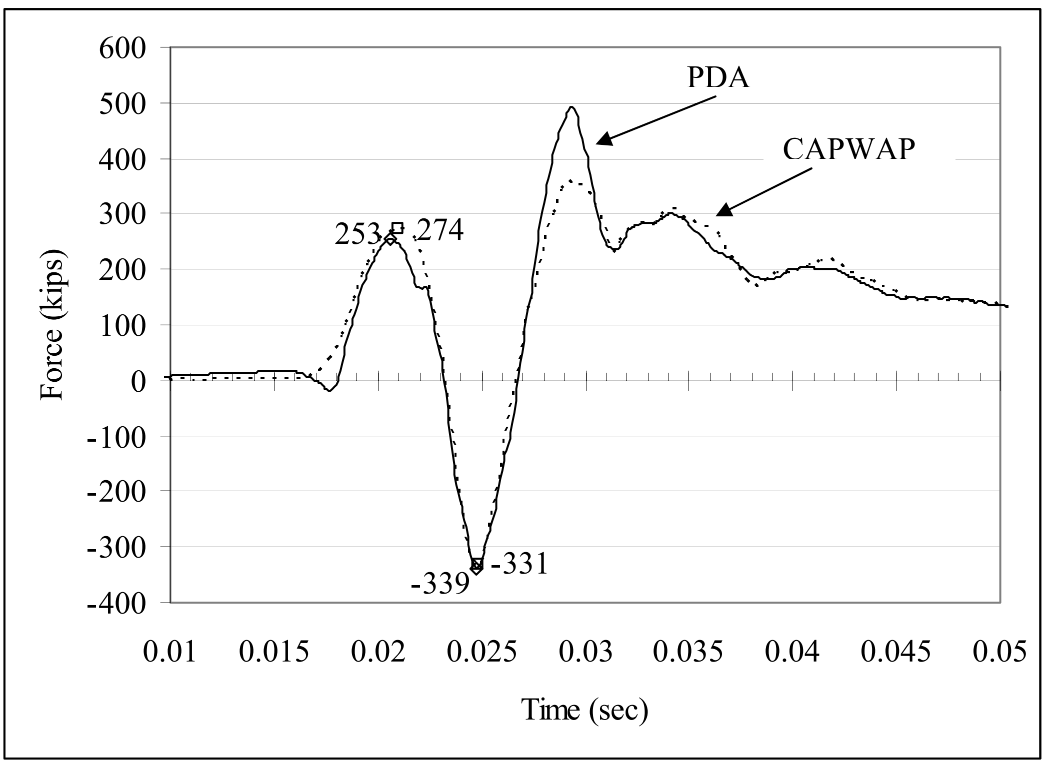


Figure F-11 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #1 for BN 154 of 183.

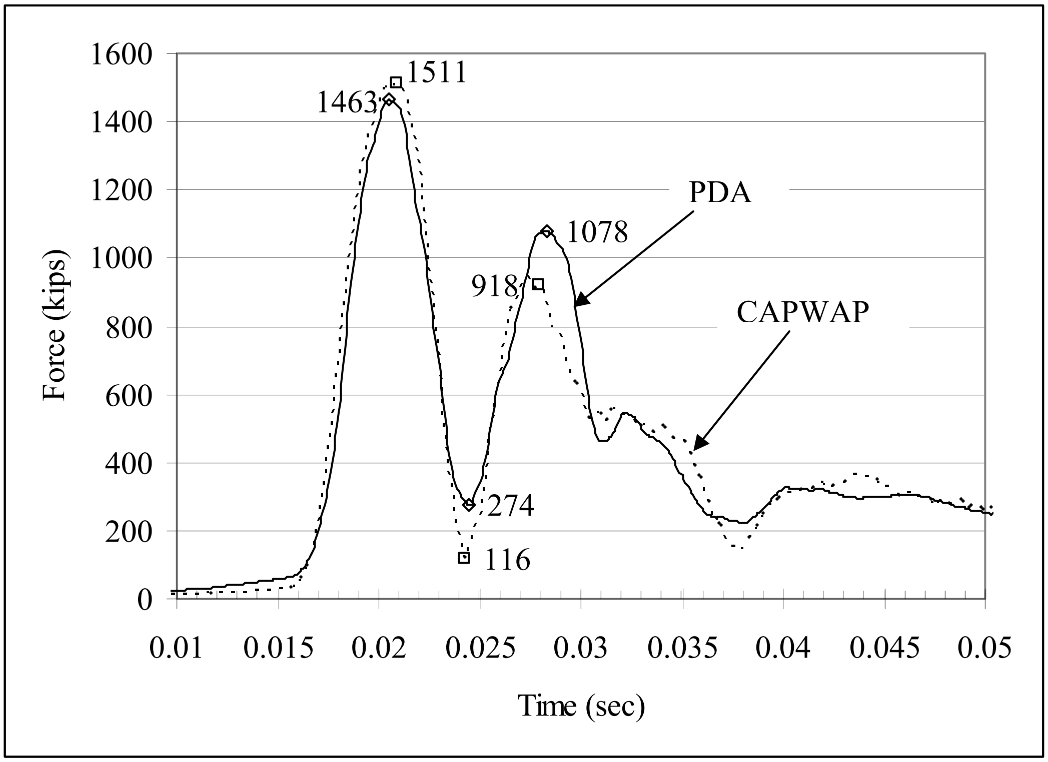


Figure F-12 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #1 for BN 154 of 183.

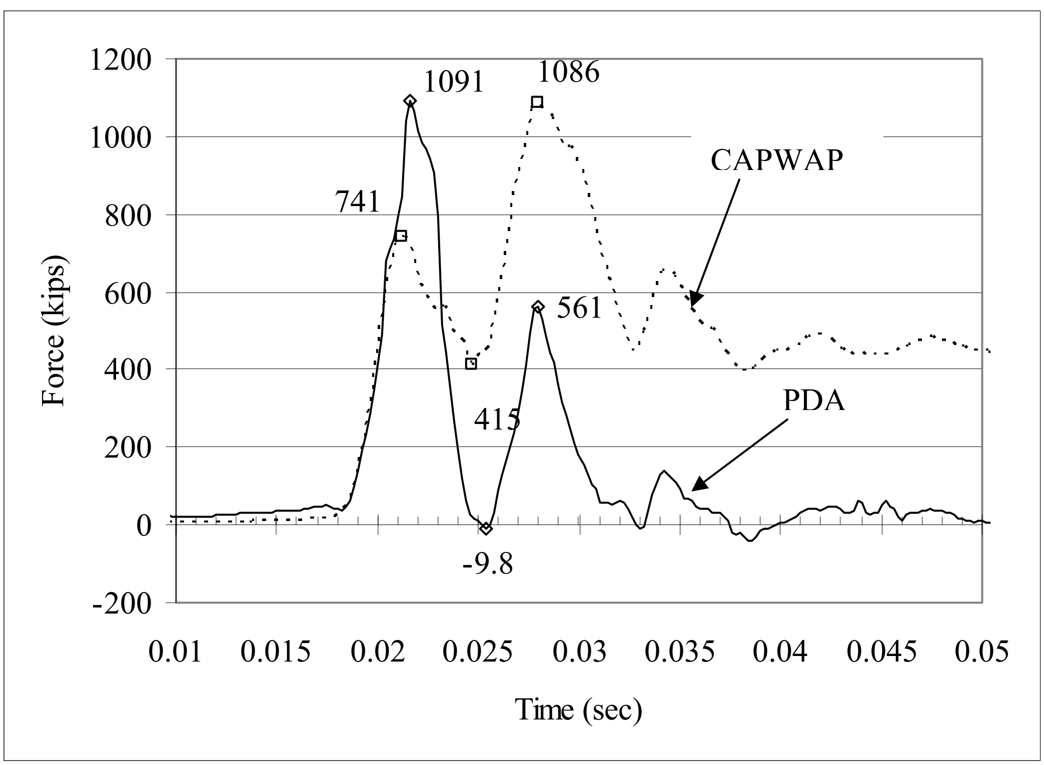


Figure F-13 Pile #1 BN 154 comparison of PDA output and CAPWAP output at the lower gage location.

Table F-7 CAPWAP output of final results for BN 155 of 183.

Total CAPWAP Capacity: 1184.3; along Shaft 575.9; at Toe 608.4 kips

| Soil Sgmnt No. | Dist. Below Gages ft | Depth Below Grade ft | Ru kips | Force in Pile kips | Sum of Ru kips | Unit Resist. (Depth) kips/ft | Unit Resist. (Area) ksf | Smith Damping Factor s/ft | Quake in |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 1184.3 | | | | | |
| 1 | 26.1 | 26.1 | 236.5 | 947.8 | 236.5 | 120.46 | 12.05 | 0.045 | 0.100 |
| 2 | 28.1 | 28.1 | 166.7 | 781.1 | 403.2 | 84.91 | 8.49 | 0.045 | 0.100 |
| 3 | 30.1 | 30.1 | 94.9 | 686.2 | 498.1 | 48.34 | 4.83 | 0.045 | 0.100 |
| 4 | 32.0 | 32.0 | 51.9 | 634.3 | 550.0 | 26.43 | 2.64 | 0.045 | 0.100 |
| 5 | 34.0 | 34.0 | 25.9 | 608.4 | 575.9 | 13.19 | 1.32 | 0.045 | 0.100 |
| Avg. Skin | | | 115.2 | | | 16.94 | 5.87 | 0.045 | 0.100 |
| Toe | | | 608.4 | | | | 97.34 | 0.114 | 0.270 |

| Soil Model Parameters/Extensions | | Skin | Toe |
|---|---|---|---|
| Case Damping Factor | | 0.094 | 0.254 |
| Unloading Quake | (% of loading quake) | 12 | 70 |
| Reloading Level | (% of Ru) | 100 | 100 |
| Unloading Level | (% of Ru) | 55 | |
| Resistance Gap (included in Toe Quake) (in) | | | 0.120 |
| Soil Plug Weight | (kips) | | 2.85 |
| Soil Support Dashpot | | 0.600 | 0.000 |
| Soil Support Weight | (kips) | 2.10 | 0.00 |

CAPWAP match quality: 2.23(Wave Up Match)
Observed: final set = 0.343 in; blow count = 35 b/ft
Computed: final set = 0.304 in; blow count = 39 b/ft

Table F-8 CAPWAP output of extreme values for BN 155 of 183.

| Pile Sgmnt No. | Dist. Below Gages ft | max. Force kips | min. Force kips | max. Comp. Stress ksi | max. Tens. Stress ksi | max. Trnsfd. Energy kip-ft | max. Veloc. ft/s | max. Displ. in |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.0 | 1519.4 | 0.0 | 2.353 | 0.000 | 35.51 | 4.0 | 0.438 |
| 2 | 2.0 | 1518.9 | 0.0 | 2.352 | 0.000 | 35.41 | 4.0 | 0.435 |
| 4 | 3.9 | 1514.2 | 0.0 | 2.345 | 0.000 | 35.24 | 4.0 | 0.429 |
| 6 | 6.0 | 1505.1 | 0.0 | 1.689 | 0.000 | 35.17 | 4.1 | 0.427 |
| 8 | 8.0 | 1494.7 | 0.0 | 1.677 | 0.000 | 35.12 | 4.1 | 0.424 |
| 10 | 10.0 | 1480.6 | 0.0 | 1.661 | 0.000 | 35.07 | 4.2 | 0.422 |
| 12 | 12.0 | 1458.8 | 0.0 | 1.637 | 0.000 | 35.02 | 4.2 | 0.420 |
| 14 | 14.1 | 1436.6 | 0.0 | 1.612 | 0.000 | 34.97 | 4.3 | 0.417 |
| 16 | 16.1 | 1475.0 | 0.0 | 1.655 | 0.000 | 34.91 | 4.4 | 0.415 |
| 18 | 18.1 | 1505.2 | 0.0 | 1.689 | 0.000 | 34.85 | 4.6 | 0.412 |
| 20 | 20.1 | 1536.6 | 0.0 | 1.724 | 0.000 | 34.78 | 4.7 | 0.409 |
| 22 | 22.2 | 1550.0 | 0.0 | 1.739 | 0.000 | 34.70 | 4.8 | 0.407 |
| 24 | 24.2 | 1551.8 | 0.0 | 1.825 | 0.000 | 34.61 | 5.0 | 0.403 |
| 26 | 26.1 | 1544.5 | 0.0 | 2.392 | 0.000 | 34.47 | 5.1 | 0.399 |
| 27 | 27.1 | 1297.9 | 0.0 | 2.010 | 0.000 | 27.40 | 5.1 | 0.397 |
| 28 | 28.1 | 1285.1 | 0.0 | 1.990 | 0.000 | 27.34 | 5.1 | 0.394 |
| 29 | 29.1 | 1098.7 | 0.0 | 1.702 | 0.000 | 22.21 | 5.1 | 0.392 |
| 30 | 30.1 | 1083.3 | 0.0 | 1.623 | 0.000 | 22.17 | 5.1 | 0.390 |
| 31 | 31.1 | 964.3 | 0.0 | 1.071 | 0.000 | 19.17 | 5.1 | 0.389 |
| 32 | 32.0 | 947.1 | 0.0 | 1.052 | 0.000 | 19.14 | 5.1 | 0.388 |
| 33 | 33.0 | 873.4 | 0.0 | 0.970 | 0.000 | 17.46 | 5.1 | 0.387 |
| 34 | 34.0 | 854.2 | 0.0 | 0.949 | 0.000 | 16.75 | 5.1 | 0.385 |
| Absolute | 25.2 | | | 2.401 | | | (T = | 27.7 ms) |
| | 1.0 | | | | 0.000 | | (T = | 0.0 ms) |

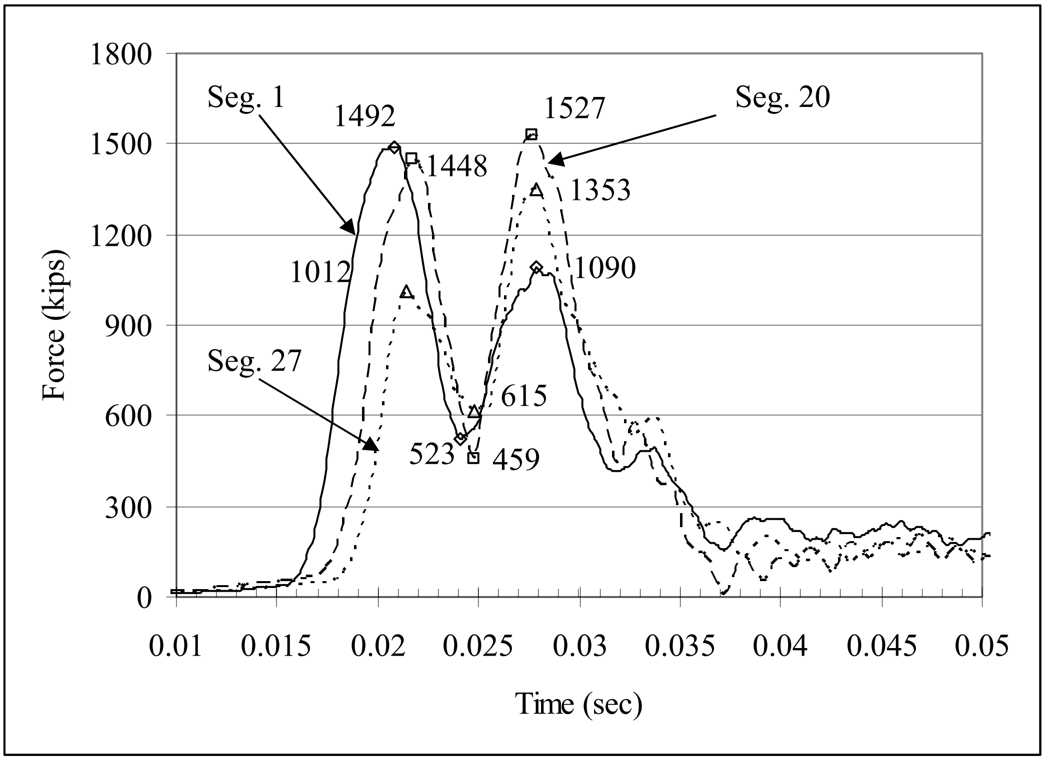


Figure F-14 CAPWAP output of force at three pile segments for BN 155 of 183.

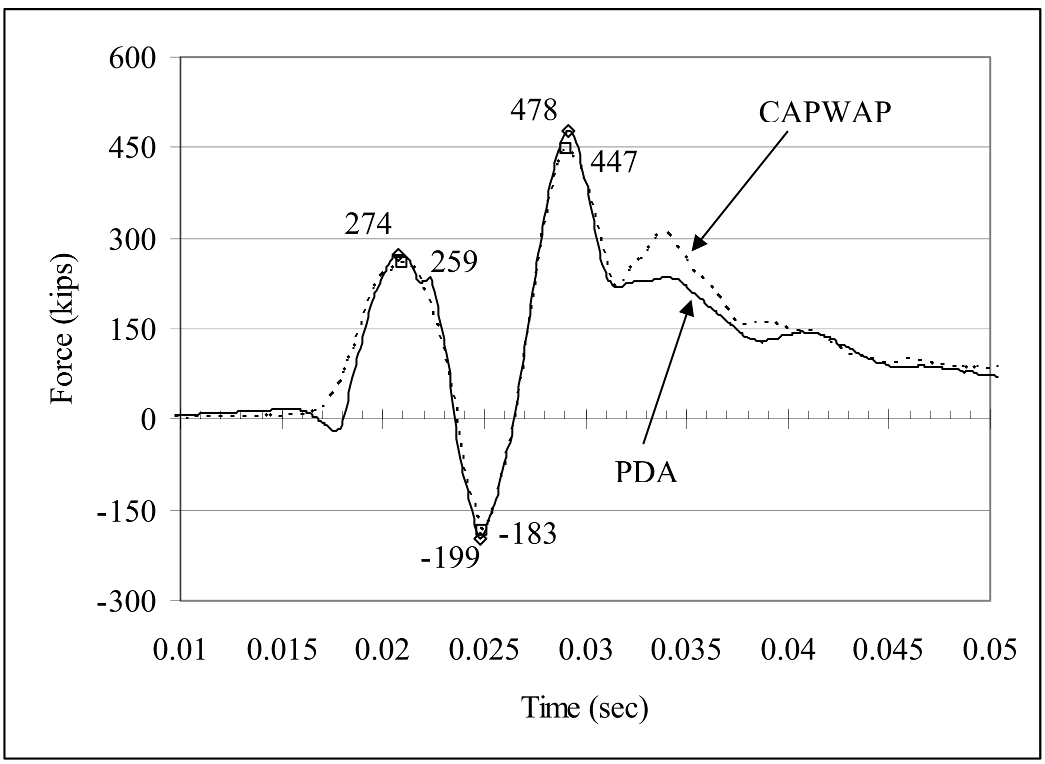


Figure F-15 Match quality of CAPWAP computed wave up and PDA measured wave up at the top of Pile #1 for BN 155 of 183.

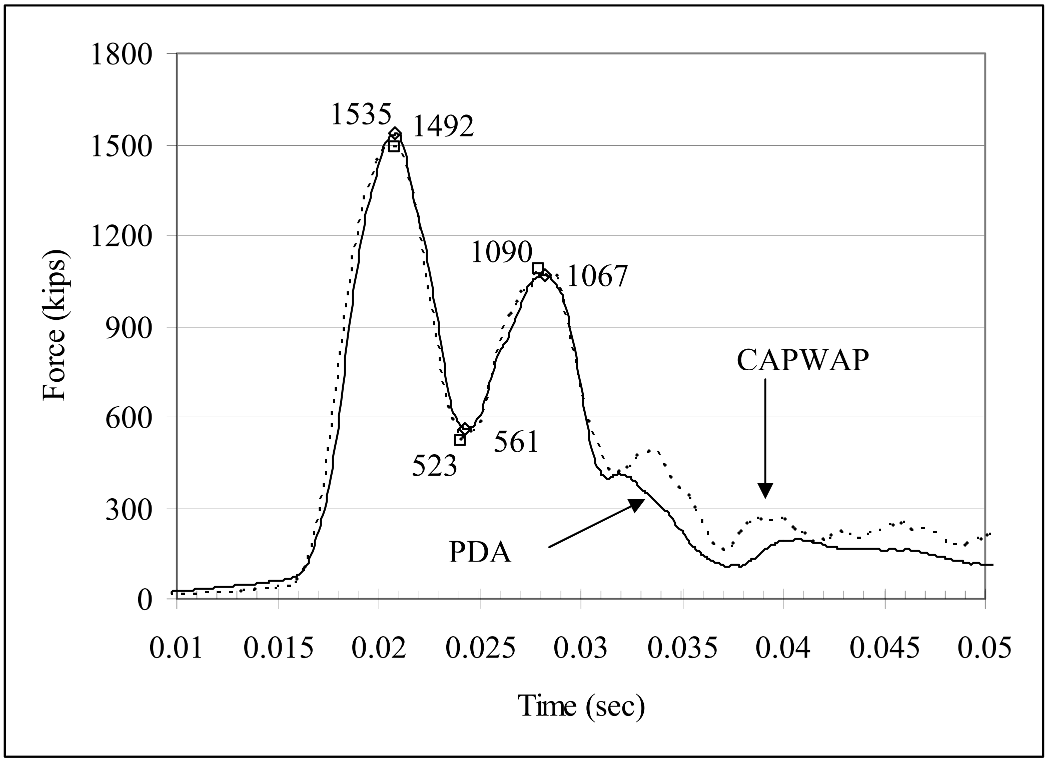


Figure F-16 Match quality of CAPWAP computed force and PDA measured force at the top of Pile #1 for BN 155 of 183.

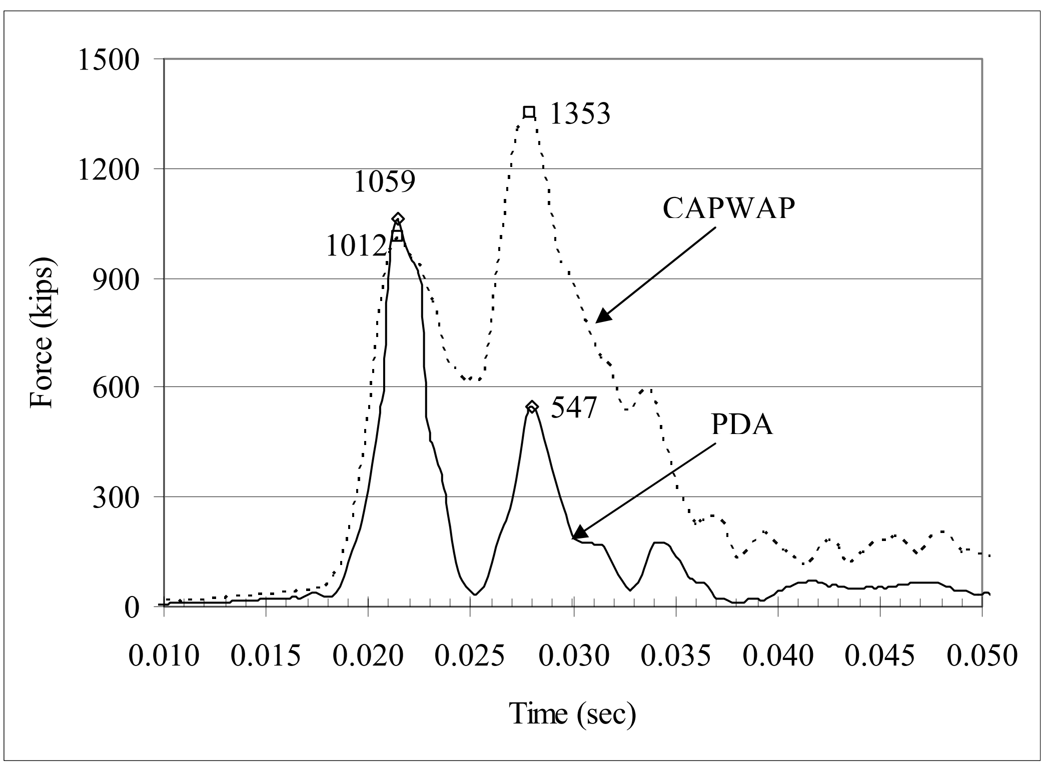


Figure F-17 Pile #1 BN 155 Comparison of PDA output and CAPWAP output at the lower gage location.

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