Evaluation of Pile Load Testing State of Practice for ALDOT Projects

by

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A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
December 11, 2021

Keywords: Pile Driving Analyzer (PDA), Static Load Test, Ultimate Capacity, Driven Pile

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Abstract

A database of driven piles taken from archives of the Alabama Department of Transportation (ALDOT) was created for the purpose of evaluating the pile load testing state of practice for ALDOT. The dataset was derived from driven piles on ALDOT projects from 1995 to 2021. The database consisted of records from H-Piles, steel pipe piles, and prestressed square concrete piles. In this dataset, 81 piles possessed both static load test and high strain dynamic test (HSDT) data on the same pile. Of the HSDT, some piles were analyzed using CAPWAP (Case Pile Wave Analysis Program) and some were analyzed using iCAP. Analysis outputs consisted of end of drive (EOD), set checks (STCK - restrike within 24 hours of EOD), and beginning of restrike (BOR - restrike after 24 hours of EOD) data. There were 20 additional piles studied with static load test data only. All static load tests were further analyzed to observe the current state of static load testing practice. Many of the piles in this study were driven in the coastal plain region of Alabama. Currently, ALDOT commonly conducts static load tests and PDA-S and iCAP analysis on driven piles to predict ultimate capacity. A statistical analysis was conducted on the piles that had both static load test data and HSDT data to determine if ALDOT could rely less on static load tests and more on HSDT to determine ultimate capacity. If this were possible, it could potentially save ALDOT a tremendous amount of money and time, as static load tests are both expensive and time consuming. HSDT (both CAPWAP and iCAP) at EOD, STCK, and BOR were compared to the maximum load applied (MLA) during each static load test. It would have been preferred to compare a Davisson Failure criterion from each static load test to HSDT EOD, STCK, and BOR. Unfortunately, from the dataset gathered, only 18 out of 101 static load tests

conducted on ALDOT projects reached Davisson failure criterion. In the current state of practice, ALDOT conducts static load tests on test piles to either two and a half times or three times the design load and does not typically load test piles to Davisson failure criteria. Because of this, the MLA from the static load test was used to compare to HSDT results.

Because there was a mixture of piles having HSDT by both CAPWAP and iCAP, statistical tests were conducted to determine if there was a difference between CAPWAP and iCAP. In this dataset, there was no significant difference between CAPWAP and iCAP in their linear relationship with the MLA during the static load test. For the purposes of this analysis, CAPWAP and iCAP were considered equivalent.

Overall, there was a significant predictor effect of HSDT on MLA at the endpoints of EOD, STCK, and BOR as assessed by simple linear regression. However, it does not appear that HSDT is a surrogate or substitute for MLA across all subsets of the dataset examined. Further research will be required to better understand the complex relationship between dynamic and static load testing.

Dedication

This work is dedicated to my dog Magnolia, my three dogs in Heaven: Lovins, Chalk, and Andy Murray, and my horse Joe. It is also dedicated to every bass in Alabama that I was not able to catch, due to the time I spent working on this project. Those fish are extremely lucky.

Acknowledgements

I would like to thank Dr. Anderson for serving as an excellent mentor and friend while pursuing my graduate studies here at Auburn University. Dr. Anderson kept me on course with sound advice and wisdom throughout my graduate studies, offered personal, real-life advice, and helped me explore the job market. Many thanks to Dr. Montgomery for serving on my committee as well as being a mentor and a friend during my time at Auburn University. I also appreciate Dr. LaMondia for being willing to serve on my committee. I'd also like to thank everyone at the Alabama Department of Transportation for their help and guidance throughout the duration of this project. It has been a true pleasure working with Nate Maxwell and Alan Douglas.

I have been blessed to have a wonderful extended family that has supported me my entire life and especially while I have been at Auburn. Without faith and family, this journey would have been impossible!

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Defined Abbreviations and Terms

A – Cross Sectional Area of Pile

ALDOT – Alabama Department of Transportation

A-Pile – Pile Design software currently used by ALDOT

ASD – Allowable Stress Design

BOR – Beginning of Restrike - a restrike after 24 hours from EOD

BOR-days – EOD to BOR elapsed time (days)

CAPWAP – Case Pile Wave Analysis Program, PDI software used for signal matching on data acquired from PDA-S

DL – Design Load

E – Elastic Modulus of Pile Material

EOD – End of Drive

FS – Factor of Safety

iCAP – In field capacity prediction produced from data acquired from PDA-S

iCAP-BOR – iCAP at BOR

iCAP-EOD – iCAP at EOD

iCAP-STCK – iCAP at Set Check

MLA – Maximum Load Applied

L – Length of Pile

LRFD – Load Reduction Factor Design

N – sample size of a dataset or subset

PDA – Pile Driving Analyzer

PDA-S – PDA software that is used in the field for data acquisition

PDI – Pile Dynamics Incorporated

PSC – Prestressed square concrete pile

Q –Maximum Load Applied during static load test

SLT	– Static Load Test
STCK	– Set-Check – a restrike within 24 hours of EOD
STCK-hours	– EOD to STCK elapsed time (hours)
WBuzpile	– Pile Design software previously used by ALDOT

CHAPTER 1: INTRODUCTION

1.1 Overview of ALDOT State of Practice

Driven piles are commonly used in ALDOT projects to construct deep foundations for bridges and other roadway structures. ALDOT currently uses two methods for verifying the ultimate capacity of driven piles: static load tests and HSDT, known commercially as PDA (PDI 2021) tests combined with an iCAP analysis. These tests are commonly conducted throughout the state with more prevalence in the coastal plain regions of the state due to the soil type being more suitable for driven piles. PDA iCAP analysis is usually conducted at EOD, STCK, and BOR.

The capacity determination process of driven piles is a complex soil-structure interaction problem. The process from pile design, to installation, to testing, follows a common approach across ALDOT projects. Piles are initially designed by engineers using either a software called WBUZPILE (Ashour et al., 2012) or A-PILE (Ensoft, Inc. 2021). Designers utilize this software to determine the design load of the driven pile. The design load is based on soil properties that are estimated from prior field and laboratory tests and pile type, size, and length. After the design is complete, either a Load Reduction Factor Design (LRFD) or an Allowable Stress Design (ASD) approach is commonly used to compare the design with the results of the installed pile in the field. Static load tests and HSDT are commonly used to determine if a pile can support a predetermined factor of the design load. Commonly, static load tests are carried out to three times the design load, and HSDT are carried out to two times the design load. In the current state of practice, ALDOT commonly tests driven piles to three times the design load and does not necessarily test each pile to Davisson failure criterion. If the pile is being driven to refusal, a specific LRFD approach has been established, or a special circumstance arises on the job, either or both the static load test and HSDT test can be waived at the discretion of the ALDOT state geotechnical engineer.

Figure 1.1 ALDOT Job Site View from crane bucket – Semmes, AL

1.2 Problem Statement

Static load tests are commonly used in current ALDOT practice. Due to time and complexity, the cost of static load tests is consequential. An alternative to static pile load tests is the use of HSDT. Strain gauges and accelerometers are attached to piles during impact driving to measure stress and velocity. These measurements are used to monitor the function of the pile driving system and the integrity of the pile. The HSDT device records the waves of stress and velocity, and their deflections, that pass through the pile after the hammer strikes. Stress and velocity waves can then be used to predict pile capacity using either CAPWAP or iCAP. The current ALDOT (Davis, 2019) and AASHTO LRFD (AASHTO 2017) practice is to assign a nominal resistance factor to driven piles of 0.80 when a static load test is conducted and at least 2% of production piles are subject to dynamic testing (restrike). A resistance factor of 0.75 can be achieved if *all* piles are subject to HSDT (restrike) with no requirement for a static load test. According to AASHTO, the highest resistance factor achievable through standard penetration test (SPT) based static analysis is 0.45. The question of why not conduct only HSDT tests instead of static axial load tests has been posed to the Highway Research Center.

1.3 Research Objective

The objective of this research project was to:

- Evaluate the current practice of static load tests conducted on driven piles on ALDOT projects.
- Determine the characteristics of agreement between HSDT and static load tests. This may inform the decision to use HSDT as a replacement for static load tests in the future.

- Based on the results of modeling the agreement between HSDT and static load tests, make recommendations for the future practice of ALDOT pile testing.

1.4 Scope of Work

The scope of work involved in the production of this document are as follows:

- A literature review was conducted that evaluated recent and similar research that was performed involving static load tests and HSDT on driven piles.
- The author observed and participated in several ALDOT projects around the state of Alabama to develop a more complete understanding of the process at all steps from the field operations to data analysis and interpretation.
- An ALDOT statewide dataset that consisted of static load test and HSDT data was gathered and organized for use in evaluating the current ALDOT pile driving state of practice.
- A statistical analysis was performed on driven piles with both static load test data and HSDT data to determine how well the HSDT predicts MLA from the static load test.
- A survey was created to deliver to southeastern DOTs to determine the utilization of pile load tests within the DOT peer group (southern coastal states where piles are commonly used).
- Results of the study were compiled into a report that demonstrates the current state of pile load testing practice for ALDOT projects and provides recommendations for future research and practice.

CHAPTER 2: BACKGROUND

2.1 Introduction

This section details relevant components of the practice of pile driving. Elements of theory and field applications are included.

2.2. Basic Pile Design

Like all realms of geotechnical engineering, pile design begins with site characterization and a subsurface investigation. When bridges are at the design stage, ALDOT commonly conducts standard penetration tests (SPT) borings at each proposed abutment or bent location. Conducting SPTs enables the design engineer to estimate soil strength at the site by use of blow count (N) and determine the soil type at the same time. Soil strength must be considered for both the act of driving the pile, as well as the soil providing strength for the deep foundation around the pile. The pile designer would most likely prefer that the toe of the pile rest in a soil layer with a high blow count. If this is accomplished, it would help add capacity to the end bearing or the toe of the pile. The soil stiffness also affects the selection process of the pile driving hammer due to the driving stresses that must be considered. After the soil parameters have been adequately considered, the type, size, and length of pile is determined by a design engineer. The pile design process is performed to incorporate multiple unknowns. Unknowns can then be accounted for by either using an ASD or LRFD approach. After the design is complete and the driving hammer has been chosen, a GRLWEAP (PDI 2021) analysis is conducted to predict pile capacity with corresponding driving stresses and blow counts that can be expected in the field. Pile designs are then commonly verified in the field either using static load tests, HSDT, or both.

2.2.1 Allowable Stress Design (ASD)

The ASD method is simple and has been used for decades by engineers of all disciplines. This method features predicting an expected ultimate capacity and assigning a factor of safety to account for uncertainties in design, construction, and performance. This method is especially popular in geotechnical engineering due to the amount of uncertainty that exists in the industry. The ASD design equation is defined below in Equation 2.1 as:

$$\frac{Q_u}{FS} = \sum Q_a \tag{2.1}$$

where Q_u is the ultimate capacity of the pile, FS is the factor of safety, and $\sum Q_a$ is the sum of allowable capacity of the pile (Steward, 2020). The allowable capacity of the pile is the load that the engineer has determined to be the safe, working load of the pile. The factor of safety incorporates any uncertainties in the design, construction, testing, or performance of the pile, and is commonly based on experience. ASD is still used by ALDOT but is slowly being replaced due to the natural over conservatism that the method provides and the adoption of Load and Resistance Factor Design by the US Federal Highway Administration.

2.2.2 Load Resistance Factor Design (LRFD)

Load Resistance and Factor Design (LRFD) is the design method that has been replacing ASD nationwide and is slowly replacing ASD at ALDOT. LRFD uses resistance and load factors opposed to factors of safety. The load factors consider uncertainties in loads, and resistance factors consider uncertainties in pile testing methods. The general equation used for LRFD design as defined in Equation 2.2 as:

$$\phi Q_u = \sum \gamma_i Q_a \tag{2.2}$$

where ϕ represents the resistance factors, γ_i represents the load factors, Q_u represents ultimate pile capacity, and Q_a represents the allowable pile capacity. The LRFD takes a statistical design approach and can be calibrated based on past project results for certain regions of the state. The resistance factors in the equation depend on what type of field testing is being performed to determine pile capacity. The load factors depend on the type of load that will be applied to the structure.

This design method can enable engineers to avoid general over conservatism, a common flaw of the ASD design method. The current ALDOT (Davis, 2019) and AASHTO LRFD (AASHTO 2017) practice is to assign a nominal resistance factor to driven piles of 0.80 when a static load test is conducted and at least 2% of production piles are subject to HSDT (restrike). A resistance factor of 0.75 can be achieved if all piles are subject to HSDT (restrike) with no requirement for a static load test.

2.3 The Wave Equation Applied to Pile Driving

The wave equation was initially constructed modeling the transverse waves induced by the plucking of a guitar string (Powers, 2011). The wave equation discussed in this chapter pertains to the longitudinal strain/stress waves induced by a hammer striking a rigid rod. The general form of the wave equation is:

$$\frac{\partial^2 u}{\partial t^2} = c^2 \frac{\partial^2 u}{\partial x^2} \tag{2.3}$$

$$c^2 = E/\rho$$

where:
$u\,(x,t)$ is the strain value at position x and time t in the rod,
c = wave speed,
E=elastic modulus of the rod,
ρ=density of the rod,
L = length of the rod,
x = position along the length of the rod from 0 to L,
t = time, thus

One of the original solutions of the wave equation was based on assuming u is separable into a function F of x only and function G of t only:

$$u(x,t) = F(x) * G(t). \tag{2.4}$$

This leads to two second order ordinary differential equations. Each of these ordinary differential equations have solutions dependent on initial and boundary conditions. The reality in the pile driving world is that these equations are solved numerically because boundary conditions change with each hammer blow as the pile is driven. The numerical solutions to the wave equation are the theoretical foundation used for the estimation of pile capacity based on strain and velocity wave traces during driving.

2.4 GRLWEAP

The software, GRLWEAP, is based on the numerical solution to the wave equation. GRLWEAP is used to predict maximum compressive and tensile stresses, hammer blow count, hammer stroke, and hammer energy with depth based on anticipated pile capacity. The software allows the user to construct a soil-pile-hammer interaction model to predict these components. The user can input the manufacturer specifications of the hammer that is used to drive the pile. Then, the user can input the design details of the pile and a general soil profile of the site to complete the model. The user then inputs the desired pile capacities for use in the analysis. The software is also able to conduct valuable drivability analysis that can predict if a pile will be overstressed, damaged, or hit refusal prematurely. ALDOT commonly uses this software as a "field check" along with HSDT. The GRLWEAP-predicted compressive and tensile driving stresses in the pile are checked against AASHTO design standards (AASHTO 2002 Standard Specification Articles 4.5.7.3 and 4.5.11) to ensure that the pile will not be damaged during driving. The AASHTO equations used to verify acceptable driving stresses for steel H-Piles and PSC piles are detailed below. The AASHTO standards for compressive and tensile driving stresses in steel H-Piles are the same.

Steel H-Pile $$\text{Compressive or Tensile Driving Stresses} < 0.9f_y \tag{2.5}$$

Where: f_y is the steel yield stress

The following equation is the AASHTO standard for compressive driving stresses in prestressed concrete piles.

PSC Pile $$\text{Compressive Driving Stresses} < 0.85f'_c - f_{pe} \tag{2.6}$$

Where: f'_c is the concrete compressive strength

Where: f_{pe} is the effective prestress after losses

The following equation is the AASHTO standard for tensile driving stresses in prestressed concrete piles under normal environments.

PSC Pile
(US Units) $$\text{Tensile Driving Stresses} < 3(f'_c)^{1/2} + f_{pe} \tag{2.7}$$

The following equation is the AASHTO standard for tensile driving stresses in prestressed concrete piles under severe corrosive environments.

PSC Pile $\qquad$ Tensile Driving Stresses $< f_{pe}$ $\qquad$ (2.8)

The predicted hammer blow counts and stroke produced by the GRLWEAP analysis can be valuable information to the field engineer conducting the HSDT. The engineer can anticipate certain trends in the driving process and can alert the crane operator ahead of time to avoid driving the pile too far into the ground or damaging the pile. Example GRLWEAP inputs and outputs are provided below from an ALDOT project in Demopolis, Alabama (Marengo County). The outputs are for Bent 2 over French Creek, a tributary of the Black Warrior River. An input screen for hammer and pile details, soil-hammer-pile model inputs, and general output are provided below in Figure 2.1, Figure 2.2, and Figure 2.3, respectively. All provided windows were created by ALDOT engineers.

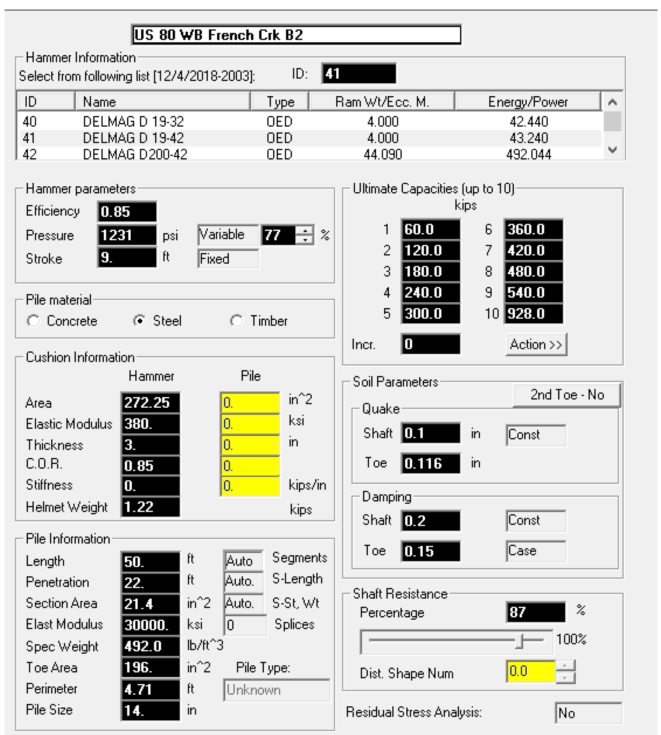


Figure 2.1 GRLWEAP Input Screen for Hammer and Pile Details

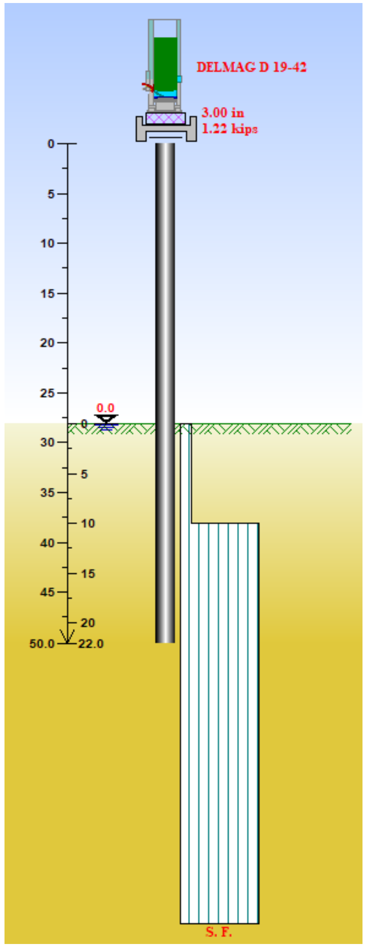
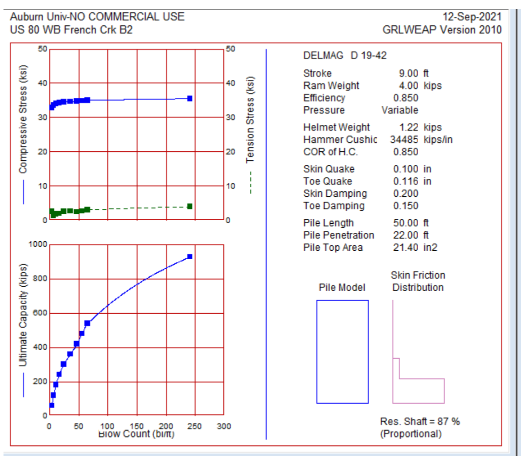


Figure 2.2 GRLWEAP Soil-Hammer-Pile Model Input Screen



Auburn Univ-NO COMMERCIAL USE
US 80 WB French Crk B2

12-Sep-2021
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke ft	Energy kips-ft
60.0	32.85	2.49	4.6	9.00	41.83
120.0	33.49	1.19	7.4	9.00	34.35
180.0	33.98	1.75	11.7	9.00	28.42
240.0	34.24	1.89	17.1	9.00	24.69
300.0	34.45	2.48	24.7	9.00	22.09
360.0	34.61	2.63	36.0	9.00	20.27
420.0	34.76	2.42	47.3	9.00	19.15
480.0	34.87	2.68	56.3	9.00	18.90
540.0	34.98	2.97	65.7	9.00	18.60
928.0	35.51	3.94	241.1	9.00	17.26

Figure 2.3 GRLWEAP General Output

2.5 Pile Driving Analyzer (PDA)

Two strain gauges and two accelerometers are attached to the head of the pile usually two and a half to three times the diameter of the pile from the top of the pile. In addition to the gauges and accelerometers being installed onto the pile, a small square must be torched above the gauges to run the wires between both sides of the pile. The edges of the torched square are then thoroughly duct taped to protect the wires from being cut on any sharp points left by the torch. In order to protect the PDA gage and accelerometer connections to the wires, an additional bolt is installed above the torched square to provide strain relief in the connections. Figure 2.4 features a strain gage and accelerometer bolted to a steel 18x135 H-Pile on an ALDOT project in Semmes, AL. The gauges and accelerometers were installed by ALDOT and the author while being hoisted by a crane in a bucket 85 ft in the air. Figure 2.5 displays the hammer mid stroke, striking the pile, while the PDA operator observes driving trends in the pile utilizing the PDA system.

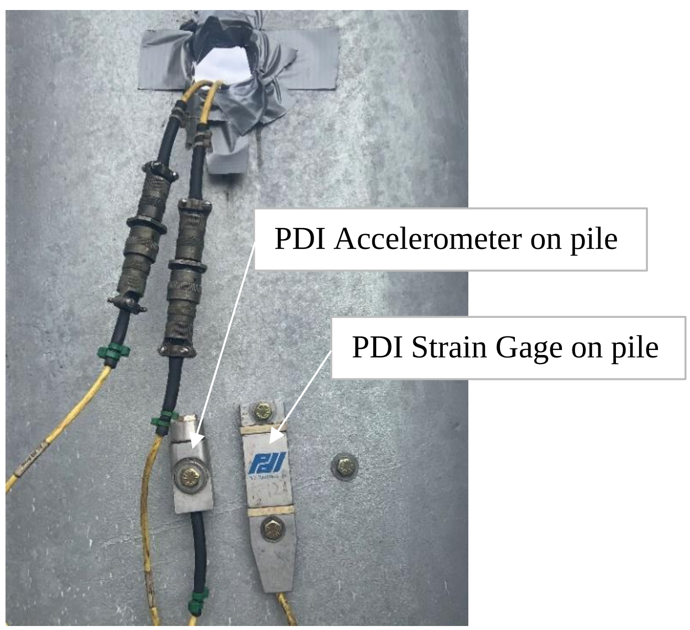


Figure 2.4 PDA gauges installed on 18x135 steel H-Pile – Semmes, AL

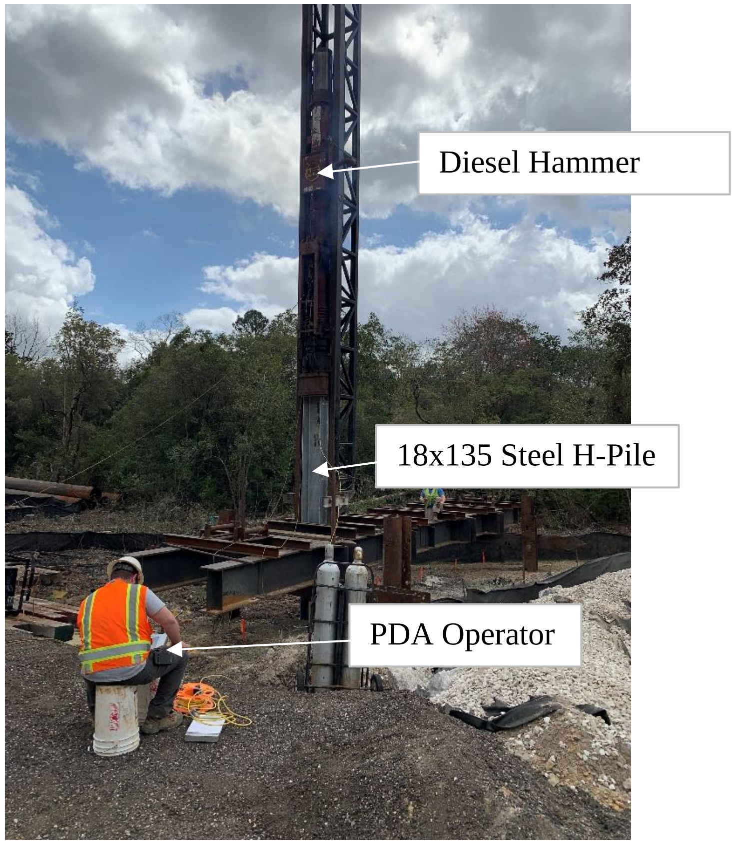


Figure 2.5 Overview of Pile to PDA Semmes, AL

As shown in Figure 2.5, gauges are then connected via wire to the PDA field unit that is being used by the pile driving engineer at the foot of the pile.

With each blow of the hammer, the accelerometer is measuring the acceleration of waves of theoretical particles from the pile head to the pile toe and back. The basic assumption of wave mechanics in driven piles is that the pile responds to the impact of the hammer in the form of a one-dimensional longitudinal wave (PDI, 2012). The PDA system can then use the measured acceleration to calculate velocity by integrating once and can calculate position by integrating twice. The strain gage is simultaneously measuring strain in the pile with each blow of the hammer. Strain can be calculated by dividing velocity (v) by the wave speed (c). Strain and velocity should be proportional to one another initially. This may be how the software estimates the integration constants that would be needed to calculate velocity from acceleration and position from velocity, though that appears to be a proprietary detail and not easy determined from the software documentation. The equation for strain is defined below in Equation 2.9.

$$\varepsilon = \frac{v}{c} \qquad (2.9)$$

Stress can then be calculated by multiplying both sides by the modulus of elasticity (E) of the pile material and then force can be calculated by multiplying both sides by the cross-sectional area (A) of the pile. This is detailed in Equations 2.10 and 2.11.

$$\sigma = v\frac{E}{c} \qquad (2.10)$$

$$F = \frac{EA}{c}v \qquad (2.11)$$

The quantity *z* is pile impedance; therefore, force is equal to the pile impedance multiplied by velocity as detailed below in Equation 2.12.

$$z = \frac{EA}{c} \qquad (2.12)$$

Impedance is a term from electric circuit analysis and is the sum of resistance (from circuit elements that consume power) and reactance (from circuit elements that store power by causing phase shifts of sinusoidal current and voltage). Pile impedance models the interaction of the pile with soil while it is being driven and fits well in the context of the wave models. Good proportionality is displayed when the force and zv wave traces match (PDI, 2012). However, proportionality is only expected before the wave traces reach 2L/c, which is defined as the time it takes the waves in the pile to travel from the top of the pile to the bottom and back to the top. The quantity 2L/c is an important benchmark to an experienced PDA operator. An experienced PDA operator can determine the difference between easy and hard driving, low and high end-bearing piles, and high and low shaft resistance piles based on the wave traces. There are two important wave trace conditions to understand to fully utilize the power of PDA: a free end condition and a fixed toe condition. An illustration of force and velocity wave traces for a free end condition is displayed in Figure 2.6 below.

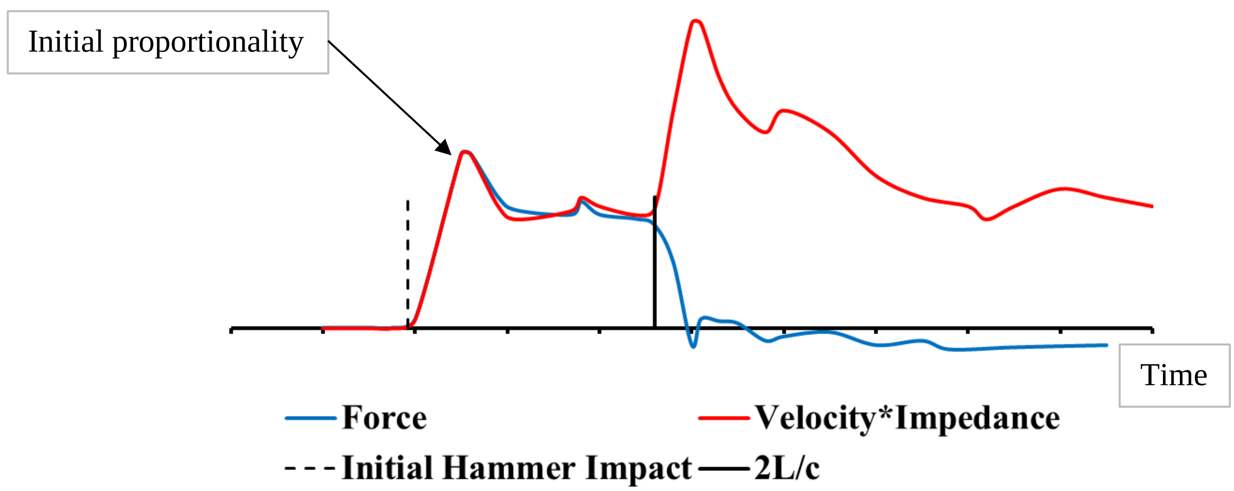


Figure 2.6 Wave Trace Illustration for Free End Pile

In a free end pile, the force cancels at the toe, and a tensile wave up curve is shown that dips below zero at 2L/c (which means low toe resistance). The force drops to zero due to the lack of driving resistance. The velocity at the toe then doubles at 2L/c which is a wave down curve. The reason the velocity wave returns, and the force wave does not, is because most of the resistance on the pile is on the shaft and not the toe. This would be considered easy driving, which generally puts the pile in tension. This is generally not a problem for steel piles but can be for concrete piles (concrete generally performs poorly in tension). The opposite scenario would be a fixed toe condition, which means the toe of the pile cannot move and is likely at refusal. An illustration of a fixed toe condition is displayed below in Figure 2.7.

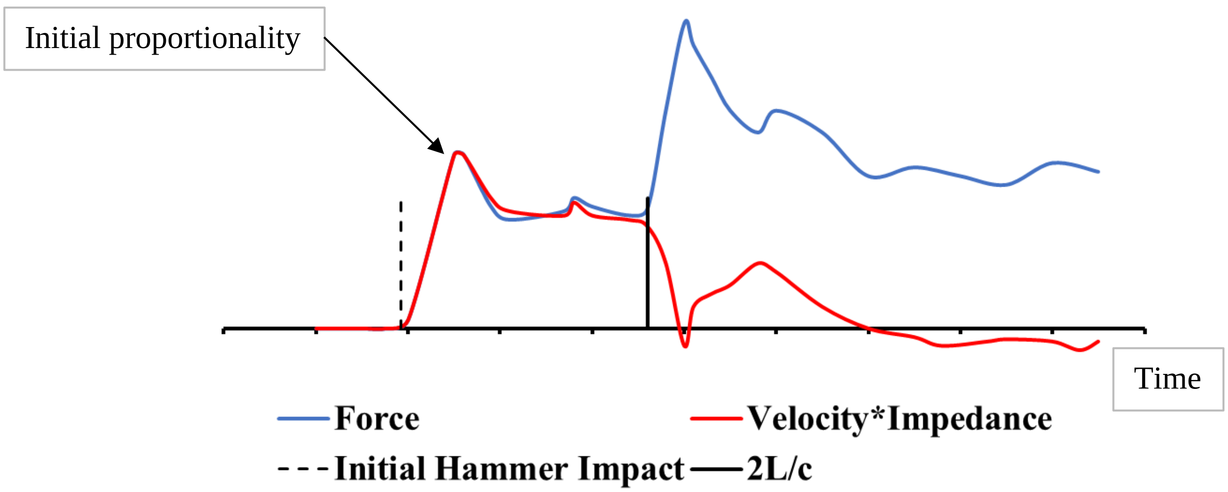


Figure 2.7 Wave Trace Illustration for Fixed End Pile

The critical stress for this condition exists at the toe of the pile (PDI, 2012). Velocity waves cancel at the toe and return to zero at 2L/c and display a compressive wave up (could be hard rock). The compressive force at the toe doubles at 2L/c. Because the toe of the pile cannot be driven any further, velocity waves return minimal resistance while the return of the force waves double. This trend can be expected during hard driving.

2.5.1 iCAP

At the end of driving, the PDA operator can quickly create an iCAP analysis of the recent data gathered by the HSDT system in the field. An iCAP is an automated pile capacity analysis that allows minimal user adjustments. An example iCAP input screen featuring the PDA-S system is displayed below in Figure 2.8.

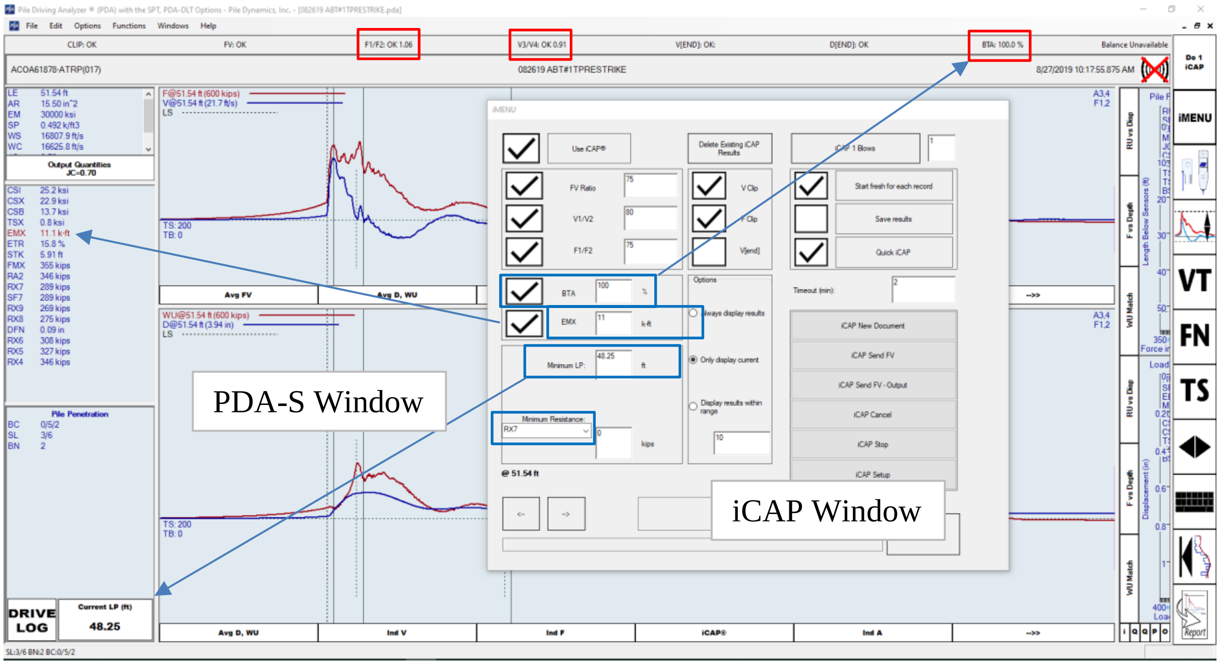


Figure 2.8 iCAP Input Screen with PDA-S system in background

The PDA operator must enter: BTA (pile integrity), EMX (maximum energy from hammer), minimum LP (minimum length of pile in ground), and the minimum resistance values into the iCAP window with corresponding values from the PDA-S window (boxed in blue). The minimum resistance input depends on the type of soil expected at the site (RX7 is for sand). The operator would prefer to analyze blows that contain: a F1/F2 ratio close to 1, a V3/V4 ratio close to 1, and a BTA value close to 100%. A BTA value of 100% predicts no pile damage, and a BTA value of 85% to 99% predicts slight damage. These values are all displayed in the top row of the PDA-S window and must be checked by the operator prior to conducting the analysis for reliable results. An iCAP analysis will output a settlement-load plot, force and velocity curves, and predicted ultimate capacity in terms of skin friction and end bearing. An EOD iCAP analysis usually utilizes one of the last ten blows of the initial drive, depending on which blow had the best force and velocity ratio and BTA value. Similarly, for iCAP STCK and BOR analysis, the first two to five blows are used, depending on data quality. Example iCAP analysis outputs from an ALDOT project in Tuscaloosa, AL for EOD, STCK, and BOR are shown below in Figures 2.9, 2.10, and 2.11, respectively.

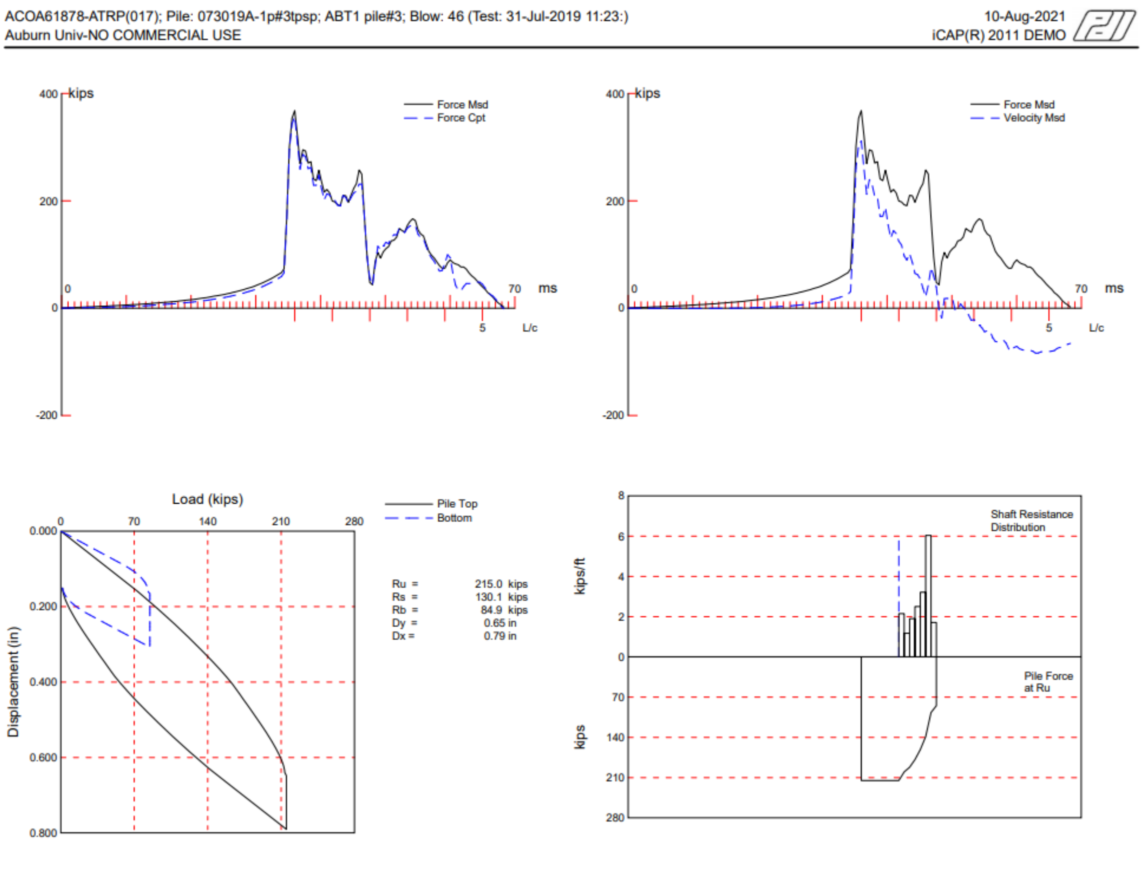


Figure 2.9 iCAP EOD Analysis – Tuscaloosa, AL

The EOD iCAP analysis was conducted on blow 46 (out of 50 blows). The BTA value was 87%, the F1/F2 ratio was 1.16, and the V3/V4 ratio was 0.93. The EOD analysis predicted an ultimate capacity of 215 kips.

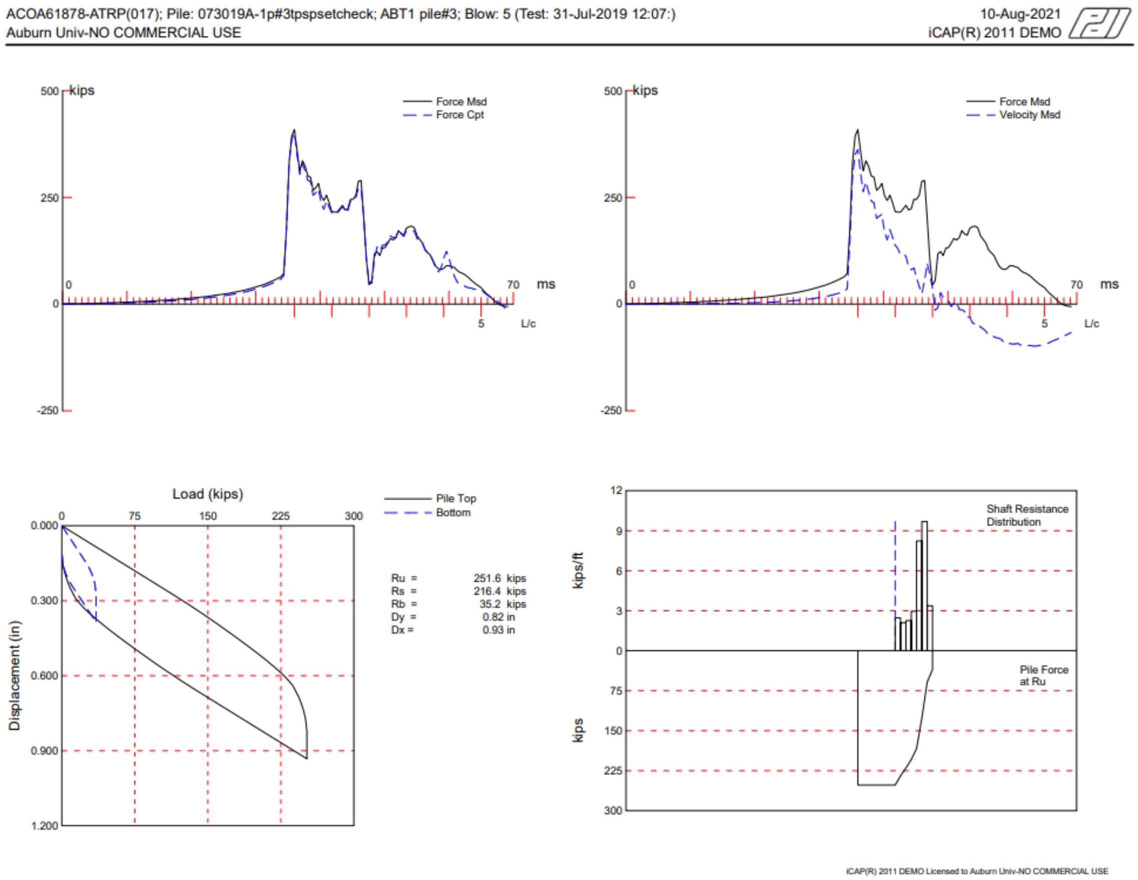


Figure 2.10 iCAP STCK Analysis – Tuscaloosa, AL

The iCAP STCK analysis was conducted on blow 5 (out of 7 blows). The BTA value was 88%, the F1/F2 ratio was 1.18, and the V3/V4 ratio was 0.92. The STCK analysis predicted an ultimate capacity of 251.6 kips. The pile gained 36.6 kips of capacity after a set-up time of roughly 45 minutes.

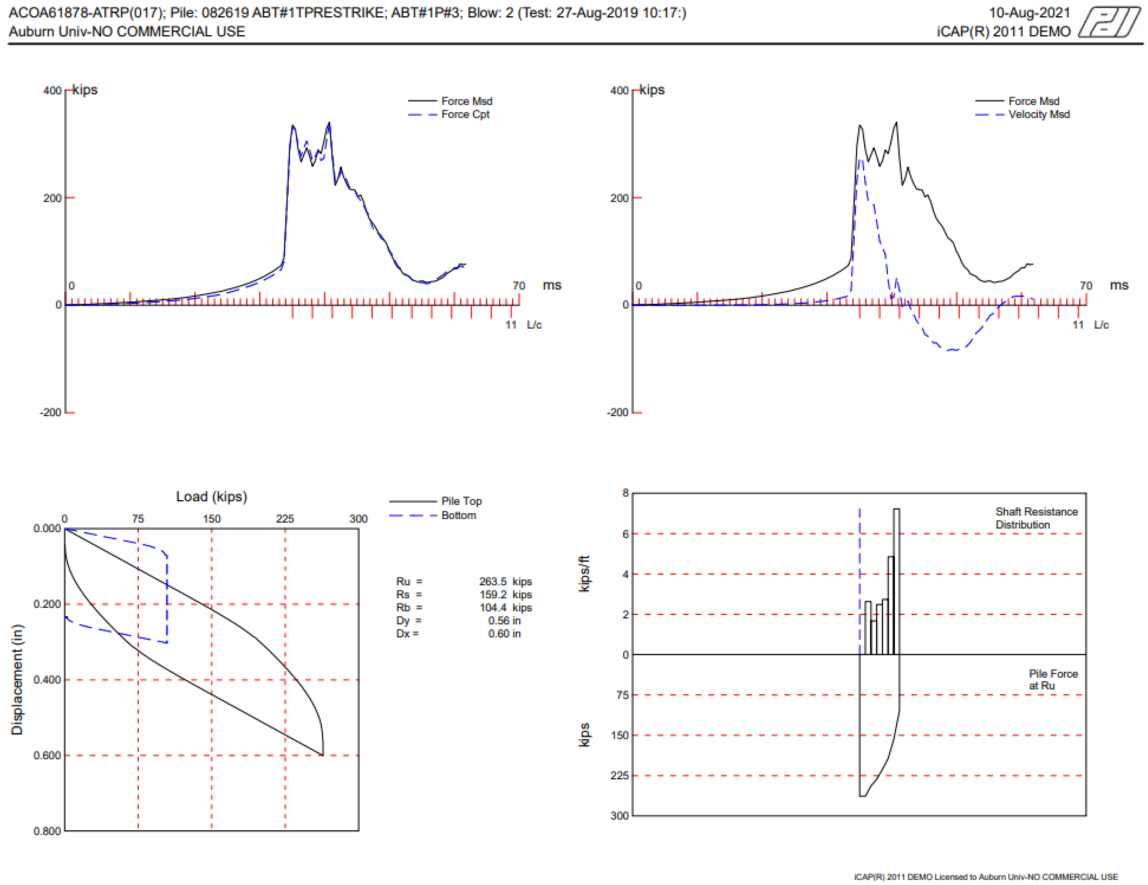


Figure 2.11 iCAP BOR results example

The iCAP BOR analysis was conducted on blow 2 (out of 6 blows). The BTA value was 100%, the F1/F2 ratio was 0.76, and the V3/V4 ratio was 0.95. The BOR analysis predicted an ultimate capacity of 263.5 kips. The pile gained roughly 12 kips of capacity after a set-up time of roughly 28 days.

2.5.2 iCAP versus CAPWAP

The Case Pile Wave Analysis Program, also known as CAPWAP, is an iterative, signal matching process of determining ultimate capacity. The process involves building a soil model in the program in concert with the field gathered data. Results vary based on the ability of the user and a substantial amount of time in the office is required to produce accurate results. An iCAP analysis can be created instantaneously for each desired blow with minimal human correction immediately in the field following the end of driving. iCAP produces one automated result per blow, is not user dependent, is faster, and easier to operate.

2.6 Static Load Test and Davisson Capacity

The static load test has been historically viewed as the gold standard of verifying pile integrity and predicting ultimate capacity. Historically, engineers tend to prefer verification techniques that can be objectively measured and observed. A static load test enables the engineer to carefully observe how a pile reacts to loads as they are applied. A static load test is an expensive and time-consuming process. A pile is statically loaded using a calibrated jack in increments of the maximum load while the settlement of the pile is recorded with time. Measurements of loading, elapsed time, and pile movement are recorded during the duration of the testing process (Hannigan et al., 2006).

Static load tests have been used in heavy construction by engineers for decades as a way of verifying the ultimate capacity and structural ability of a driven pile. A static load test consists of a test pile, typically two to four reaction piles for support, a load beam, a source of measurement, and a calibrated jack. Figure 2.12 shows a schematic of a typical set up for a static load test on an ALDOT project.

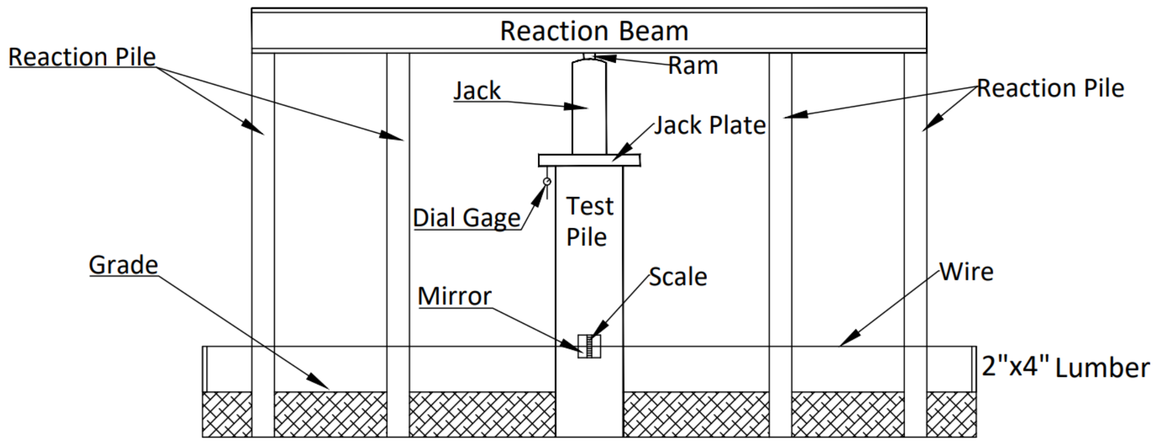


Figure 2.12 Typical ALDOT Quick Load Test Set up

In practice, a test pile is usually initially driven and allowed to set-up for three to seven days. The pile is allowed to sit for this time period to give the soil around the pile a chance to re-equilibrate. Upon test day, two to four reaction piles are typically driven and then stick-welded to a reaction beam that spans across the reaction and test piles. The reaction piles serve as anchors to stabilize the reaction beam to give the jack something to push against while applying a series of calibrated loads to the test pile. The loads that are applied to the test pile are usually applied in increments of 5% of the desired maximum applied load. Each additional load amount is applied for the same length of time (usually three to five minutes). Pile settlement is measured with a mirror and a scale and is recorded with corresponding time elapsed and applied load. After the pile has been fully loaded, the pile is usually unloaded in increments of 20% of the maximum load applied as settlement readings are taken. A load test was conducted on a 12x53 test pile at abutment 1 at an ALDOT project in Prattville, AL and is shown below in Figure 2.13.

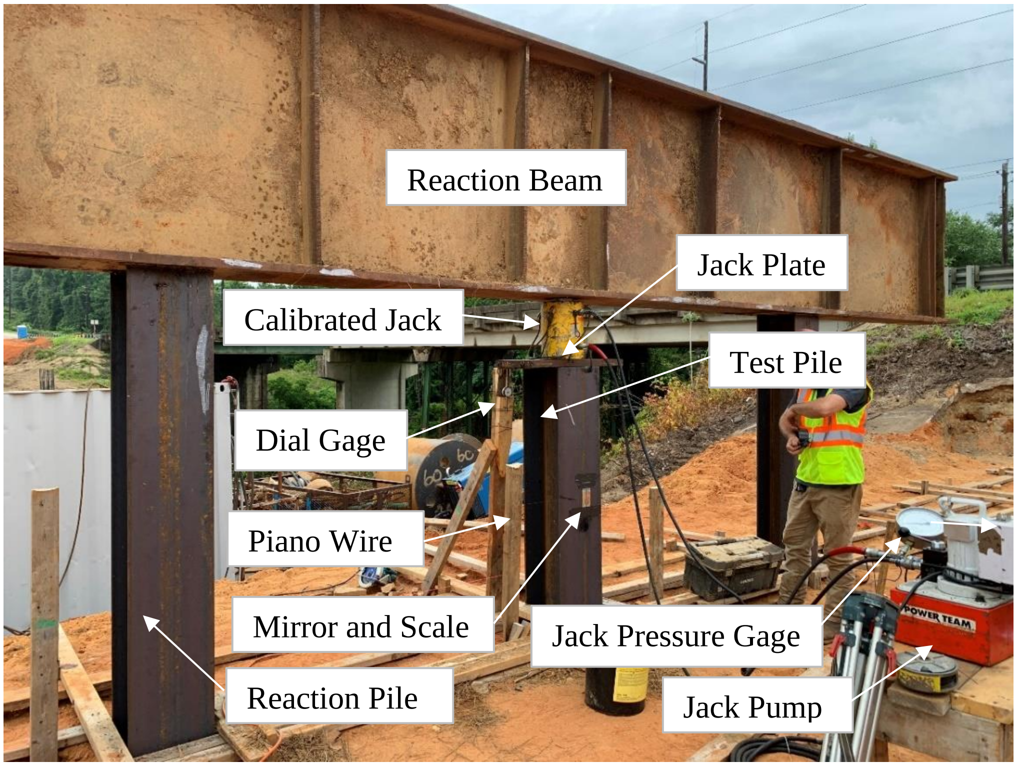


Figure 2.13 Static Load Test Setup from ALDOT Project in Prattville, AL

The test methods detailed in ASTM D1143/D1143M (ASTM 2013) measure the vertical, axial movement of a driven pile when loaded under static axial compression. This test method may also be conducted on other types of deep foundations. There are seven test procedures detailed in this ASTM. Because the "Quick Test" method is commonly used on ALDOT projects, the details of this procedure will be discussed in this section. As stated in Section 4.2 of the above referenced ASTM, "If feasible, without exceeding the safe structural load on the pile(s) or pile cap, the maximum load applied should reach a failure load from which the engineer may determine the ultimate axial static compressive load capacity of the pile(s)." (ASTM 2013). A failure load is not commonly achieved on ALDOT projects. In regards to the reference beams used for aiding measuring the settlement of the pile from section 7.1.1 of ASTM, "Reference beams and wirelines shall be supported independent of the loading system, with supports firmly embedded in the ground at a clear distance from the test pile of at least five times the diameter of the test pile(s) but not less than 8 ft, and at a clear distance from any anchor piles of at least five times the diameter of the anchor pile(s) but not less than 8 ft." (ASTM 2013). In Figure 2.14 above, it appears this stipulation was not adequately adhered to, which can be a safety hazard. Upon completion of the test, a load-settlement curve can be created from the recorded data. Settlement is plotted on the y-axis and applied load is plotted on the x-axis. Figure 2.14 displays an expected load-settlement plot.

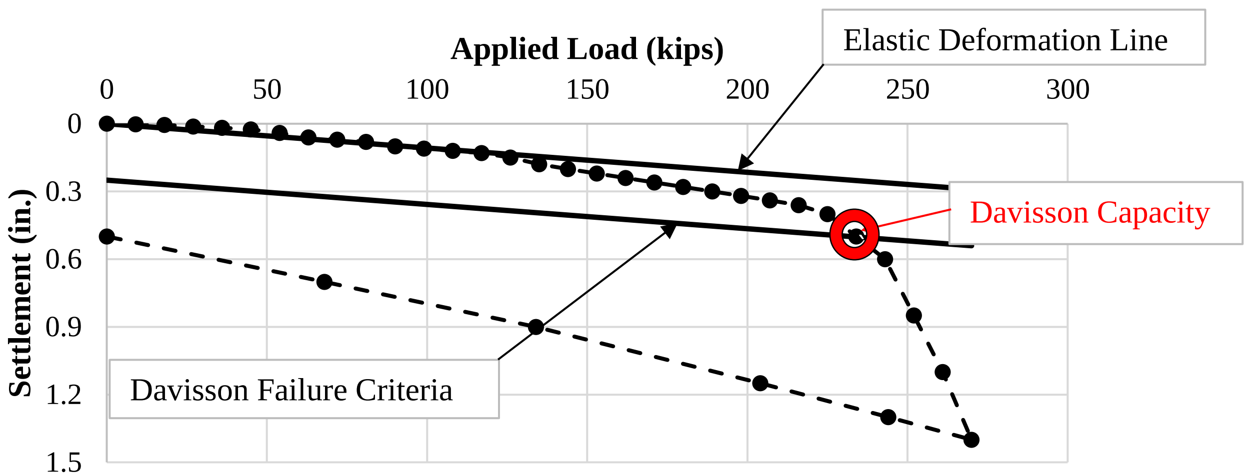


Figure 2.14 Expected Load-settlement Plot

After the load-settlement curve has been created, elastic deformation and failure criterion lines are created. The elastic deformation line is calculated based on the applied load, length of the pile, cross sectional area of the pile, and elastic modulus of the pile. The equation for the elastic deformation line is given in Equation 2.13.

$$\Delta = \frac{QL}{AE} \tag{2.13}$$

Where: Δ = Elastic deformation of pile (in.)
Q = Maximum Load Applied (kips)
L = Length of Pile (in.)
A = Cross Sectional Area of Pile (in^2)

E = Modulus of elasticity of pile material (ksi)

The failure criterion line is offset from the elastic deformation line by a factor of the size of the pile being driven. The failure criterion line is calculated (in U.S. units) as shown in Equation 2.14.

$$s_f = \Delta + \frac{B}{120} + 0.15 \tag{2.14}$$

where: Δ = Elastic deformation of pile (in.)
B = Pile diameter or width (in.)
s_f= Settlement at failure (in.)

The Davisson failure criterion is determined from where the load-settlement curve intersects the failure criterion line. The Davisson failure criterion is interpreted as the predicted ultimate capacity of the pile. The ultimate capacity is the maximum resistance mobilized by both the shaft (skin friction) and the toe (end bearing). If a static load test does not reach Davisson capacity, a predicted pile capacity cannot be determined, and the test only produces censored data. That is the pile has an ultimate capacity greater than the MLA during the static load test, a common trait of over designed piles. For example, results from a static load test were plotted on a 12x53 steel H pile at abutment #2 on an ALDOT project in Wilcox County, Alabama. The pile was designed for 72 kips and was loaded to 216 kips, three times the design load. The settlement of the pile does not achieve the elastic deformation line nor Davisson failure criterion. This load-settlement plot is shown in Figure 2.15.

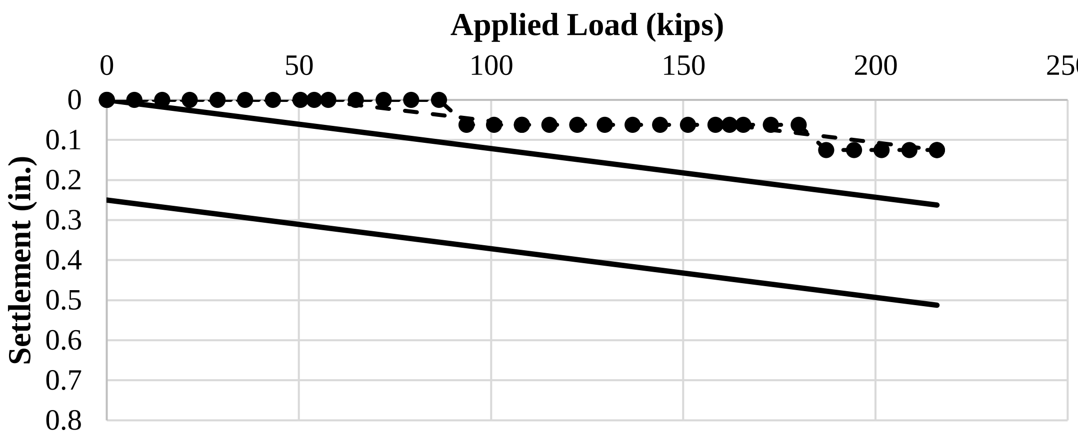


Figure 2.15 Wilcox County ALDOT Project Load-settlement curve

The elastic deformation line represents the elastic limit in the pile and when the limit is surpassed, the pile is permanently deformed. Because the load-settlement curve of the pile did not reach the elastic deformation line when the pile was unloaded, the settlement returned to zero. If the load-settlement curve had reached the elastic deformation line, the pile would have likely unloaded at a slope parallel to the slope that the pile was loaded at and not returned to zero. A load-settlement plot from a static load test conducted on a 12x53 steel H-Pile for an ALDOT project in Lee County, AL is displayed in Figure 2.16.

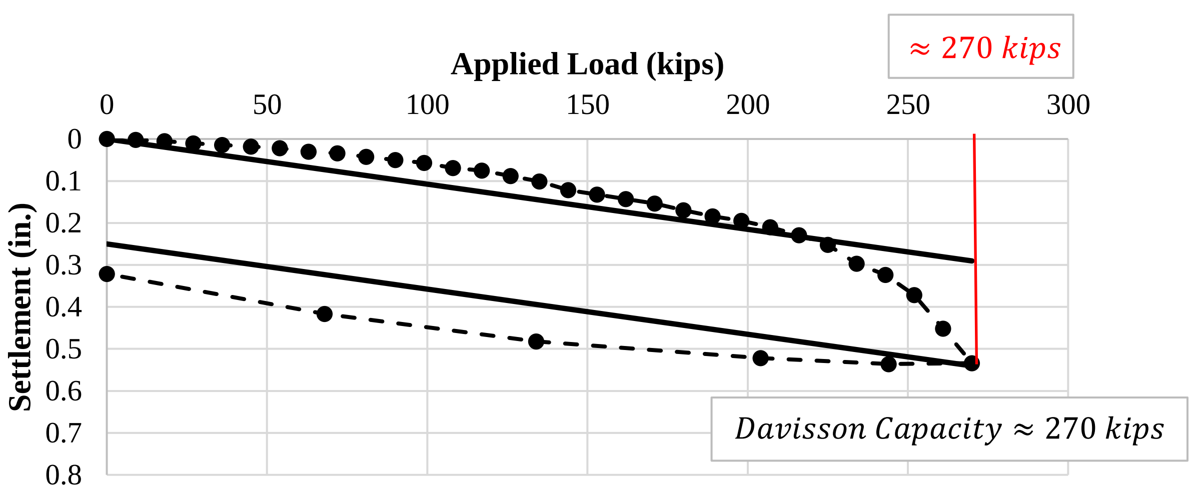


Figure 2.16 Lee County ALDOT Project Load-settlement curve

For the above static load test, the load-settlement plot achieved Davisson failure criterion and an ultimate capacity (Davisson capacity) of 270 kips was graphically predicted. After the pile was unloaded, the pile did not return to its initial position and thus was permanently deformed.

2.7 Soil Mechanics of Pile Driving

The purpose of conducting a static load test on a pile is to incrementally load the pile over a span of time until the toe of the pile is mobilized, and enough settlement occurs to determine Davisson failure criterion. From the Davisson failure criterion, an estimated ultimate capacity of the pile can be determined. If the pile toe is not mobilized and does not reach a Davisson failure criterion: it most likely means that the pile has been over designed. Most of the load was likely experienced by the shaft of the pile by means of skin friction, and not the toe of the pile. However, engineers could have purposely over designed a pile to avoid any chance of structural failure or to save any re-mobilization costs of preparing for a series of HSDT restrikes. Over designed piles could also be caused by using certain design methods. ALDOT uses software such as WBuzpile and A-Pile to design piles. The numerical modeling methods that this software utilizes could possibly lead to an overdesigned pile and could be analyzed for future research.

From a general database overview, sandy soils usually gain skin friction between the time of EOD and either a STCK or a BOR. Skin friction is usually the dominant source of capacity in sand and therefore requires longer piles to achieve additional capacity. This is because the longer the pile, the more pile surface area is available for contact with soil. In clayey soils, usually toe capacity is gained from EOD to either STCK or BOR. Clayey soils will often “cake around” the toe of piles and help piles gain end bearing capacity as the soil is allowed to set-up around the pile.

2.7.1 Practical Refusal

The purpose of a driven pile is to safely transfer loads into the ground from a structure. When the opportunity arises that a layer of rock lies beneath a site at a shallow depth, often the economical solution would be to drive the pile to refusal rather than subjecting the pile to a multitude of tests. This is usually the case when a shallow layer of rock exists below a site that a pile can safely rest on. Driving a pile to refusal consists of monitoring the hammer blow count and hammer stroke in the field that was estimated from the WEAP analysis. When a pile is driven to refusal, the HSDT tests and static load tests are often waived, saving project time and budget. The time period required for a pile to reach refusal is a function of the pile, hammer type and soil conditions (Hannigan et al., 2006). Practical refusal is often defined as a hammer blow count of roughly 20 blows per 25 mm for 50 mm of driving (Perez, 1998).

2.7.2 Set-Check (STCK) and Restrike (BOR)

The concept that set-checks and restrikes derive from is the phenomenon of soil set-up and relaxation. Allowing the soil to set-up is requiring the pile to sit idle for some time after EOD to regain soil equilibrium from the driven pile shearing and remolding the soil. This practice enables the engineer to determine if the pile is “setting up” (gaining capacity with time) or “relaxing” (losing capacity with time) (Steward et al., 2018). Set checks are commonly conducted the same day as EOD, while restrikes are often conducted days to weeks after EOD. For the sake of database organization in this investigation, set checks are any restrike within 24 hours of EOD, and a restrike is any restrike conducted after 24 hours of EOD. For both situations, iCAPs are created using PDA blows from the first 2-5 blows depending on force and velocity match quality and BTA.

Set checks are potentially more cost-effective than restrikes due to the cost of both time and mobilization of specialized equipment. Driving a pile requires a specialized crew and crane. Cranes are expensive to operate, own, rent, and use. A week or two restrike requires a crane to be remobilized back to the site of the pile, which can cause delays in a project timeline. Set checks can determine set up or relaxation trends without the cost of remobilization because they can be completed within 24 hours of initial driving. After the EOD, if capacity is still needed, the engineer can arrange a/multiple set check(s) to detect soil set-up trends. If piles are still not at capacity after multiple set checks, either an additional pile may be spliced on and driven further, or a restrike could be arranged in the coming days or weeks.

2.7.3 Soil Set-Up Mechanics

Set up is considered to be a time dependent increase in pile capacity, while relaxation would be a time dependent decrease in pile capacity (Komurka et al., 2003). As a pile is driven, the soil around the pile is displaced and remolded. Soil is predominantly displaced laterally along the shaft of the pile, and vertically beneath the toe of the pile. As the pile is driven and the soil is disturbed, the effective stress of the soil decreases due to the creation of excess pore water pressures in the soil. "Because the time to dissipate excess porewater pressure is proportional to the square of the horizontal pile dimension, larger diameter piles take longer to set up than smaller diameter piles." (Komurka et al., 2003).

Over time, pore water pressures dissipate, which causes the effective stress of the soil to increase. Simply put, this is the process that the soil must endure after being sheared and remolded by the initial pile driving process. Most set up occurs along the pile shaft, opposed to the toe, since the surface area along the shaft is larger than the surface area of the toe. Based on experience and from general dataset reduction, piles driven on ALDOT projects, generally, exhibit more set-up than relaxation. This is most likely due to most of the soil on the studied piles being driven in clays and sands. It is common to see soil set up in clays and sands in Alabama. Silts are more prone to display trends of relaxation than sands and clays.

2.8 ALDOT Pile Driving State of Practice

The current ALDOT pile driving state of practice follows this general order for the design and testing of bridge projects. All piles used in this project were piles driven for bridge abutments or bents.

1. A site characterization is performed that usually consists of SPT soil borings performed at each abutment and bent where piles are to be driven.
2. Pile design is conducted using APile. This software incorporates the soil strength determined from the borings, required ultimate capacity for the structure being constructed, strength of the pile based on material type, size, and length and any uncertainties involved.
3. The contractor then submits a C-14 form to ALDOT. A C-14 form is a hammer proposal requesting use of a specific hammer to drive a pile.
4. Hammer suitability is then verified by ALDOT using GRLWEAP. GRLWEAP is used to estimate both tensile and compressive driving stresses. These estimated driving stresses are then compared to AASHTO standards to determine if the pile can be adequately driven. GRLWEAP also estimates ultimate pile capacity with predicted field hammer blow count, estimates hammer stroke with capacity and conducts a drivability analysis.
5. Piles are then driven with a PDA iCAP analysis conducted on the pile at EOD. A STCK is then commonly conducted on the pile the same day to analyze pile-soil set-up trends. If an iCAP STCK displays a predicted ultimate capacity greater than or equal to two times

the design load of the pile, a PDA BOR will likely be waived and no further PDA testing on the pile will be performed.

6. Usually, three days later, a static load test is conducted on the test pile. After the load test is complete, if the previous iCAP STCK did not achieve two times the design load of the pile, a PDA BOR is commonly conducted the same day. A PDA iCAP analysis must predict ultimate capacity to be at least two times the specified design load to pass specification requirements. Static load tests are commonly loaded to three times the design load of the pile.

An example of this walkthrough from an ALDOT project that took place in 2019-2020 in Tuscaloosa County is detailed below. The example below is for a test pile at Abutment #1.

1. In situ SPT borings displayed sandy soils with N-values ranging from 8 to 23. The toe of the pile was designed to rest in a sandy soil layer with a N-value of 13.
2. ALDOT designers determined that the most adequate pile to be used for abutment #1 would be a steel 12X53 H-pile. The steel H pile was designed to be 53 ft long, and the design load was 102 kips. The pile tip elevation was designed to be at an elevation of 447 ft, and the elevation of the ground at the pile was designed to be 495 ft. This leaves 5 ft of the pile remaining above the ground which can then be used to construct the pile cap – a common ALDOT practice.
3. The pile driving contractor then submitted a C14 to ALDOT to request permission to use a Delmag D30-32 hammer to drive the piles at abutment #1.
4. ALDOT then conducted a GRLWEAP analysis of the proposed hammer-pile-soil model to ensure that the proposed hammer would safely and efficiently drive the pile. ALDOT estimated the maximum compressive and tensile stresses, hammer blow count, hammer

stroke and energy, and ultimate capacity using WEAP to ensure safe and efficient driving. ALDOT conducted this GRLWEAP analysis to two times the design load for the pile. The estimated hammer blow counts at corresponding pile capacities are useful for the engineers in the field to be able to anticipate the capacity of the pile as it is driven.

5. PDA was conducted on the test pile at abutment #1 and an iCAP EOD had an ultimate capacity of 215 kips. An iCAP STCK was then conducted after 30 minutes to allow the pile time to set-up. The 30-minute iCAP STCK conveyed an ultimate pile capacity of 251 kips.
6. A static load test was conducted on the test pile after a wait period of 24 days. The static load test was loaded to 285 kips (2.8 x the design load). The static load test did not reach Davisson failure criterion and was not loaded to the ALDOT standard of three times the design load. The reason for this was not stated in the field notes. Four days after the test pile underwent a static load test, the test pile was restruck and an iCAP BOR predicted an ultimate capacity of 263.5 kips. It is interesting to note that the iCAP values did not reach or exceed the MLA during the static load test, despite the static load test not reaching Davisson failure criteria.

2.9 PDA + SLT – A Site Characterization Approach

Historically, geotechnical engineering has relied on in situ testing to sample and determine strength parameters of soil. There are multiple methods of in situ testing, with the Standard Penetration Test (SPT) and Cone Penetration Test (CPT) being the most common. These test methods provide engineers with valuable information and a sense of familiarity of what resides below the ground surface. Geophysical tests such as seismic refraction and electrical resistivity have started to break into the industry of surveying soil layers due to high costs associated with the traditional SPT tests. However, geophysical tests are complicated to conduct and require experienced and well-paid operators to properly interpret results. Geophysical tests are most useful when used alongside in situ testing and can be used to “connect the dots” between in situ tests (Montgomery, 2021). Thus, saving money as geophysical tests are generally less expensive to conduct than in situ tests. A static load test is comparable to an in situ test. Static load tests are the historical gold standard of the industry, are expensive and time consuming, and offer the engineer a method of testing that can be objectively measured and observed in the field. When a static load test is performed, a pile is loaded to a series of applied loads and the piles’ reaction to the series of loads can be witnessed. The SPT test similarly offers hands-on results. HSDT could be viewed as being similar to geophysics because HSDT requires experienced, well-trained operators and is generally less expensive than its counterpart, static load testing. On large ALDOT jobs, a calibration approach could be used for testing pile integrity and ultimate capacity. If static load tests and HSDT are calibrated for a job site, HSDT could be used to fill in gaps between static load tests quicker, cheaper, and easier with the engineer feeling at ease that a static load test was involved.

CHAPTER 3: LITERATURE REVIEW

Chapter three features a condensed literature review that evaluated recent and similar research that was performed involving correlating static load tests and HSDT results on driven piles. Each sub-chapter in Chapter three discusses an individual paper.

3.1 Hill, J.W. (2007)

Hill (2007) assembled a database that consisted of dynamic and static load tests for 30 piles on ALDOT bridge projects. The dataset gathered for this research was from ALDOT projects between the years of 2001 and 2005, with one project coming from 1995. From the 30 piles gathered, only one pile was loaded to Davisson failure criterion during the static load test. According to the author, “Since only one of the static load tests was carried to failure, the estimated static resistance from the PDA cannot be directly compared to the measured resistance indicated from the static load tests.” Therefore, due to only one load test indicating failure, the author investigated if the PDA correctly indicated if a pile would hold 2.5 times the design load of the pile. Of the 30 piles in the dataset, 26 indicated sufficient capacity. Of the 30 piles studied, 20 piles had a CAPWAP analysis performed to predict capacity. At the time, the author states that “the other ten projects were no longer available on a data file, therefore CAPWAP could not be performed.”. The 20 piles that underwent CAPWAP analysis were performed by the author and not by ALDOT. The 10 piles that possessed only PDA data were performed by the ALDOT PDA operator and were not modified in any way. The author concludes his work with the following thoughts on future research, “In order to evaluate the reliability of dynamic load tests, ALDOT should perform static load tests to failure and perform restrikes on all piles subjected to a static load test. This would allow a statistical reliability study to be performed comparing the estimated resistance to the failure resistance from the static load tests as defined by criteria such as the Davisson failure criteria.” No statistical analysis was conducted on the database gathered in this project.

3.2 Steward et al. (2018)

Steward et al. (2018) constructed a dataset of 25 test piles that possessed dynamic testing data from ALDOT projects located throughout the state. From the 25 test piles, 74 unique blows were used to compare CAPWAP and iCAP analysis results. There were 16 precast concrete piles and 9 steel H-piles analyzed in the study. The authors state at the conclusion of the abstract of the paper, “The results show a reliable comparison between the iCAP and CAPWAP analysis for all piles in various soil types within the state of Alabama.” The author executed a basic linear regression analysis with the ‘CAPWAP Total Axial Resistance’ on the x-axis, and the iCAP Total Axial Resistance on the y-axis. From the analysis, an R-squared term of 0.894 was produced with the data generally following along the y=x line. Additionally, the author used a paired t-test to compare differences between the CAPWAP and iCAP measured values. The author claimed the differences were not significantly different from zero with p-value near 0.8 (Steward et al., 2018). The author then conducted a further analysis of the total axial resistance removing three outliers. After the author removed the outliers, an additional simple linear regression analysis was conducted that revealed an R-squared value of 0.9675 for the trend line of the data. The trend line for the data also lined up well with the y=x line. The author did further analysis dividing the dataset into H-Piles and concrete piles. The concrete piles possessed a more impressive R squared value (0.94), than that of the H-Piles (0.695). Both regression trend lines *generally* traced the y=x line. Based on the results of the simple linear regression analysis, the author states that “because the iCAP analysis method is fully automated, it has limitations” (Steward et al., 2018). The author then concludes the paper making the following statement and recommendation, “The iCAP analysis is not as thorough as the CAPWAP method and the CAPWAP analysis has been extensively correlated with static load test results. While utilizing the iCAP analysis software can be greatly beneficial, as seen with the results of this paper, it is suggested the iCAP results should be checked thoroughly and compared with the CAPWAP analysis at least on a spot check basis to determine reliable test results” (Steward et al., 2018).

3.3 Perez, A (1998)

Perez (1998) analyzed eight different methods that predict the static capacity of driven piles for the purpose of further developing the LRFD program for the state of Florida. Of the eight different methods analyzed, CAPWAP and PDA were analyzed. The other six methods analyzed included two other stress wave approaches (Paikowsky Energy, and Sakai Energy) as well as some older driving formulas (ENR, modified ENR, FDOT, and Gates). The author stated that, “It was demonstrated that the modern methods based on wave mechanics, such as CAPWAP, PDA, and Paikowsky’s energy method, are more accurate than the old driving formulas.” (Perez, 1998). The research conducted in the above referenced paper used a database titled “PILEUF” and consisted of 242 piles. Of the 242 piles analyzed, 198 were concrete, 21 were steel pipe piles, and only 9 were steel H-piles. As the author pursued LRFD calibration for each dynamic method, a simple linear regression analysis was conducted on piles with both determined Davisson capacity from a static load test and PDA BOR predicted capacity. While the slope from the trend line was close to 1, the R-squared term from the regression analysis was 0.4851. Regarding the statistical analysis, the author stated, “Based solely on the statistical analysis and the lognormal probability distribution, the assessment of the eight dynamic methods to determine pile capacity could not be successfully completed (i.e., no criterion).” (Perez, 1998). The author conducted a simple linear regression analysis on the following scenarios: Davisson

Capacity vs. CAPWAP EOD Capacity (R-squared = 0.6264, slope = 0.6745x), Davisson Capacity vs. CAPWAP BOR (R-squared = 0.2553, slope =0.7743x), and Davisson Capacity vs. PDA EOD Capacity (R-squared = 0.6042, slope = 0.7822x). None of the analyses conducted on the above stated simple linear regression analysis models exhibited excellent agreement. Data were consistently scattered and determining that capacity predictions from the two testing methods as being interchangeable would be erroneous. From the author's conclusions, "The CAPWAP procedure tends to underestimate the Davisson capacity by 28 percent, while the PDA underestimates the Davisson capacity by only 9 percent" (Perez, 1998). The above referenced paper did not discuss the direct results from the simple linear regression models from PDA/CAPWAP vs. Davisson capacity any further.

3.4 Likins and Rausche (2004)

Likins and Rausche (2004) compiled a dataset that consisted of 119 driven piles and 23 cast in-situ piles (drilled shafts and auger cast piles). Each pile has dynamic restrike test and static load test data. The authors also discussed previous studies of statistical evaluations of dynamic restrike and static load test data. Combining all datasets analyzed, the authors analyzed 303 cases total. A CAPWAP analysis was performed on the dynamic restrike data for each pile. Based on the author's results, CAPWAP restrikes show good correlation to the static load test results based on the simple linear regression models provided. Results appear slightly better for driven piles compared to the cast-in-situ piles. The author states, "differences between CAPWAP and SLT results are generally well within the range of SLT failure loads by different evaluation methods and are comparable to the statistics of different static tests on the same piles." (Likins and Rausche, 2004). It is worth noting that each of these static load tests were able to reach failure criteria. Of the 303 piles studied, the mean CAPWAP/SLT ratio was 0.98, and the COV was 0.169. Based on the ratio of 0.98, CAPWAP is slightly conservative, but accurate in determining ultimate capacity of piles.

3.5 Rausche (2018)

When static load tests are conducted in conjunction with dynamic tests and calibrated for a site, factors of safety can be vastly reduced. As stated by the author, “modern codes require reductions of the dynamic test capacity values if no static test is performed” (Rausche, 2018). The author then discusses the complicated methods of how to conduct such calibrations as well as how to set up effective testing programs (Rausche, 2018). The author states, “the calibration procedure is not a simple matter, because it has to take into account the reasons for the differences between the two test types” (Rausche, 2018). The ability to determine the reasons for differences between the two test types would rely on a substantially sized dataset and prior experience and knowledge one would think. The author then states, “while static and dynamic tests usually agree quite well with each other. Dynamic loading tests do not and cannot provide the exact same information that static testing provides. For example, data collected during pile driving may be affected by elevated pore water pressures or soil structural changes.” (Rausche, 2018). Driven piles are difficult to analyze as a group due to every site being different in layering, soil strength, etc.; few geotechnical projects are the same. Due to a multitude of unknowns, expecting fully reproducible tests would be ill-advised. The author summarized the following methods for reasoning through differences between the dynamic and static load tests:

- Multiple failure criteria may be used for comparing static and dynamic tests.
- Regarding soil set-up, the restrike time for the dynamic tests should be the same for the static load tests.

- If the load-settlement plot from the static load test does not reach failure criterion, the curve can be extrapolated.
- If low dynamic results are caused by low hammer energy levels, either a superposition method or a change of test hammer should be investigated.

3.6 Phetteplace et al. (2019)

The authors studied two test piles from two separate test locations at a project in Egypt for the construction of a two square kilometer oil jetty. Both studied piles were 1219mm diameter open ended steel pipe piles (Phetteplace et al., 2019). A PDA EOD, BOR, static compression test, and static tensile test was conducted on each pile. For test pile 1: the EOD predicted capacity was 2100 kN and the BOR predicted capacity was 10,000 kN. The Davisson failure criterion from the compressive static load test was 11,600 kN, a percent error of 13.8%. For test pile 2: the EOD predicted capacity was 1500 kN and the BOR predicted capacity was 8,800 kN. The Davisson failure criterion from the compressive static load test was also roughly 8,800 kN. The authors claimed that "with this agreement, no adjustments are necessary to be performed to the DLT results to correlate with the SLT results" (Phetteplace et al., 2019). While the results from the study on each test pile do exhibit good correlation, the sample size is small.

3.7 Hussein and Shlash (2009)

Hussein and Shlash (2009) analyzed the CAPWAP and static load test results from three concrete, bored piles that were installed at "…Basrah Water Towers & Ground Reservoirs- Site 3" (Hussein and Shlash) in Iraq. All tested piles were 700mm in diameter. The authors claim that "good agreements are found between the static and dynamic tests results regarding pile capacities" (Hussein and Shlash, 2009). The provided load-settlement plots do show relative agreement between the static load test and the predicted PDA/CAPWAP curves. However, the author only showed a predicted PDA/CAPWAP load-settlement curve up until the peak of where the static load test results plummeted. While the PDA/CAPWAP and static load test load-settlement curves show relative agrrement up until the point of the sharp slope downwards of ultimate settlement, the slope is not consistent throughout a load-settlement plot. Therefore, how can one assume the remaining portion of the load-settlement plot when the slopes are not consistent throughout the plot. In addition to this flaw, the sample size of the dataset is small (a dataset of only three piles).

3.8 Mhaiskar et al. (2010)

Mhaiskar et al. (2010) conducted a study on static load and high strain dynamic tests on rock socketed bored piles with diameters of 600mm, 800mm, and 1000mm. All piles studied in this article were driven during the construction of Mumbai International Airport. The author confirms that PDA was conducted throughout the duration of the testing process with accordance to ASTM D4945-00. Rock socketed bored piles were used due to the geotechnical investigation at the site revealing a relatively shallow layer of rock. The author had nine available vertical static load tests for analysis. Four out of the nine load tests possessed settlement from 12 to 50mm. The author then states, “In these piles the full socket friction may have been mobilized. The magnitude of elastic rebound is also found to be negligible which indicates that the characteristics of the rock along the socket and below the pile toe may have altered during static load test” (Mhaiskar, et. al, 2010). Based on this, the author claimed that the PDA prediction may be prone to underprediction on piles with settlement from 12 t0 50mm. Therefore, the author had two stipulations for which piles would have PDA conducted on them. The stipulations are as follows:

- If static vertical load test showed a total settlement of less than 12mm and if the elastic rebound was observed to be more than 75% of the total settlement then the same pile was subjected to HSDPT,
- If static vertical load test showed a total settlement of more than 12mm and if the elastic rebound was observed to be 75% of the total settlement or less, then HSDPT was carried out on adjoining pile after completing the lateral load test.

Mhaiskar et al. (2010) provided load-settlement plots for each pair of tests on the same pile. The load settlement plots for the 600mm diameter piles exhibit similar plot curves for each type of test. However, the PDA predicted load-settlement curve underpredicted the maximum settlement of the piles by roughly 300%. The load-settlement plots for the 800mm diameter piles displayed excellent prediction of ultimate settlement. Both tests roughly predicted within 1mm. The load-settlement plots for the 1000mm diameter piles displayed one PDA test overpredicting pile settlement and one PDA test underpredicting pile settlement.

3.9 Long et al. (2002)

Long et al. (2002) analyzed and compared the results of static load tests, dynamic formulae, WEAP analyses, PDA, and CAPWAP on two H-piles driven in Jacksonville, Illinois. The results from the two studied piles were then compared to a dataset that consisted of over 100 piles. From the static load test results, test pile one exhibited a Davisson capacity of 1202 kN. At BOR, a CAPWAP analysis displayed a predicted ultimate capacity of 1263 kN. Test pile two possessed a Davisson capacity of 2537 kN and the BOR CAPWAP analysis predicted an ultimate capacity of 2291 kN. In the case of both test piles, the CAPWAP prediction closely matches the Davisson capacity. The author then plotted all predictive methods on the y-axis, and the measured capacity on the x-axis (load tests) for all piles in the database. Based on the results from plotting the datasets, the authors stated, “The database shows that use of CAPWAP with BOR data to predict capacity results in the greatest precision of all predictive methods investigated” (Long et al., 2002). Although the authors did not conduct a simple linear regression analysis on the dataset, from general observation, the plotted results do show a relative linear relationship.

3.10 General Summary of Reviewed Literature

A previous study conducted at Auburn University by Hill (2007) attempted to investigate the reliability of CAPWAP predicted capacity with Davisson failure criterion on ALDOT jobs. However, due to only one out of thirty static load tests in the author’s dataset reaching Davisson capacity, a statistical analysis was not conducted. From most papers studied, CAPWAP does seem to generally correlate with results from Davisson failure criteria. This was especially shown in the study by Likins and Rausche (2004). Then, the study conducted by Steward et al., (2018) displayed good correlation between iCAP and CAPWAP results. Overall, the general consensus appears to be that iCAP and CAPWAP are interchangeable under many circumstances and that HSDT are positively correlated with static load tests but may not be interchangeable. There is still opportunity for research to clarify this relationship.

CHAPTER 4: RESEARCH METHODS AND RESULTS

4.1 Introduction

Information was reviewed and abstracted from ALDOT projects, selecting driven piles for which static load testing was done and HSDT data was obtained. The HSDT data could be any, or all load capacity estimates from end of drive, set check, or beginning of restrike. This dataset will be referred to as Dataset 1. Dataset 1 consisted of 88 piles at the beginning of the study. From this dataset, seven piles were discarded due to a variation of equipment failure during the static load test that caused the pile to fail prematurely. Equipment failures consisted of jacks failing due to not being able to hold pressure, welds failing, and reaction frames failing. Therefore, 81 piles were used for analysis. Of the total of 81 piles, 30 of these piles were previously reported by Hill (2007) in a thesis at Auburn University. All 30 piles were from ALDOT projects and were driven and tested between the years of 1995 and 2005 (the data is still available in ALDOT records). The other 58 piles in Dataset 1 were compiled from ALDOT projects and were driven and tested between the years of 2008 and 2021. An additional dataset was also compiled for this study, and it consists of static load test results on 21 piles from ALDOT projects but does not include dynamic testing data. This dataset will be referred to as Dataset 2, and all piles were driven and tested between the years of 2007 and 2020. The load tests from Dataset 1 and the load tests from Dataset 2 were combined to get an overview of how the current ALDOT load testing practice is being conducted. The combined load tests from Dataset 1 and Dataset 2 will be referred to as Dataset 3, and this dataset consisted of 101 load tests.

4.1.1 Research Methods and Experimental Design

The practice of pile driving is a complex and expensive field that is heavily reliant on experience. In order to fully understand the complexity of how the practice is performed on an individual project basis, the author observed and helped perform several HSDTs and a static load test on ALDOT projects. The author observed and helped perform HSDTs on ALDOT projects near the Alabama towns of Semmes, Prattville, and Pine Hill. The author also observed a static load test on an ALDOT project in Prattville. This enabled the author to fully grasp how the practice of pile driving is performed and the flow of work between the engineers and the contractors. A dataset that consisted of ALDOT HSDTs and static load tests was then created from the archives of ALDOT projects. A load-settlement plot was created for each static load test in the dataset to predict the Davisson capacity for each pile that experienced a static load test. If a Davisson capacity was not achieved, the MLA from the load-settlement plot was recorded. The load-settlement plot was created from the static load test field data that consisted of time, applied load, and settlement readings. HSDT results from EOD, STCK, and BOR were also gathered and organized into a spreadsheet. The spreadsheet included: county, description of pile (abutment or bent), year, ALDOT title, ALDOT job number, geographic region, latitude and longitude, pile type, pile size, pile length, design load, MLA, Davisson capacity (if applicable), HSDT-EOD, HSDT-STCK, HSDT-BOR, STCK set up time, BOR set up time, and the major soil type for each pile in the study. The major soil type for each pile was determined from the SPT bore logs that were provided in the foundation report for each project.

After a dataset was created and organized, a statistical analysis was performed comparing the HSDT-EOD, HSDT-STCK, and HSDT-BOR to the MLA for each pile. The intent behind the study was to determine if HSDT was able to predict MLA consistently and accurately. In the study of pile driving practices presented in this paper, there were no variables under the direct control of the investigator. The dataset was compiled from the archives of ALDOT and was purely an observational study. There was no randomization and no ability to control biases that could possibly be present. This type of experiment serves to suggest areas for future research. It is “hypothesis generating” and not “hypothesis confirming”. In a designed study, there is usually an independent variable that is hypothesized to influence an outcome. The sample size is calculated to assure proper statistical power and levels of the independent variable are randomly assigned to the experimental units. Pile driving is skilled-labor intensive and requires elaborate, expensive, and time-consuming preparation for each job. For this reason, it is unlikely that there will ever be a designed study of hundreds of piles that includes randomization to independent variables of interest. Field operations are not conducive to complex experiments involving randomization to groups of interest or precise replication to yield large sample sizes within groups. For this reason, observational studies of the type presented here can be important for understanding the state of the art and directions for future studies. This type of study helps to understand what “is” but cannot precisely determine cause and effect. For example, biases can be present because the design engineer is not randomly assigning pile types across all situations of soil type and terrain. The design engineer is using experience and expertise to carefully select the techniques used. In addition, the contractor is also using accumulated knowledge to attempt to bring the engineer's design to life. Biases can easily creep into this process. Some independent variables may become “confounded” within another. A larger H-pile may tend to be used in certain situations and not others. The ability of a study like this to discern between the effect of the pile type and the other factors surrounding the pile driving is not good in the absence of randomly assigned factors that can be individually assessed (using advanced statistical techniques such as multivariate regression or analysis of variance). In this study, the direct relationship of HSDT variables with capacity variables was performed with a simple linear regression analysis. The numerical analysis conducted behind the scenes in iCAP or CAPWAP took into consideration the soil type, hammer type, pile type, pile size, pile length, and numerous other factors. The claim is that these software solutions of the wave equation directly estimate the ultimate capacity of the pile. Arguably, multivariate statistical analysis would be superfluous. However, in this dataset, there were few complete rows (i.e., without missing data) so estimation of meaningful multivariate regression parameters would be quite limited and potentially misleading. Overfitting of small datasets can lead to published studies that cannot be replicated by future investigators.

In addition to the statistical analysis, a DOT peer review survey was created for southeastern DOTs with the intent of analyzing how regional DOTs conduct pile driving. The survey was created and delivered to a geotechnical engineering representative for each DOT in: Georgia, Florida, South Carolina, North Carolina, Kentucky, Tennessee, Mississippi, Louisiana, Arkansas, and Virginia. Southeastern DOT responses are summarized in Chapter 5.

4.2 Database Design and Organization

Information was acquired and organized based on ALDOT job number and job name for clarity. Each job was organized by county, geographic region, soil type, and GPS coordinates. Within each job, each pile was organized based on pile type, pile size, pile length, static load test data and results, and type of HSDT data and results. All data was acquired from the archives of ALDOT. H-Piles were further analyzed by soil type and were determined by using the SPT bore logs at each pile location. Each H-Pile was classified as either sand or clay based on which soil was in the majority on the SPT bore log. The following maps display the project locations for all piles studied, and the project locations for each type of pile, and are displayed in Figure 4.1 and Figure 4.2, respectively. The “Current jobs” indicate the locations where pile testing was directly observed by this author to develop a more complete understanding of the process at all steps from the field operations to data analysis and interpretation. Day to day field reports from these jobs are detailed in section 4.2.1 through 4.2.3.

Each map was created using the software, ArcMap 10.7.1 (Esri 2021). All shapefiles used to create the soil regions and waterways were downloaded from the Geological Survey of Alabama website (gsa.state.al.us 2021). The soils shapefile was found in the agriculture section and titled “Soils (SSURGO)”. The water shapefile was found in the water/hydrology section and titled “Hydrography.” The shapefile used to create the county outlines was downloaded from Data.gov and the shapefile was titled, “TIGER/Line Shapefile, 2016, state, Alabama, Current County Subdivision State-based.” The shapefile used to create the state outlines was also downloaded from Data.gov and the shapefile was titled, “TIGER/Line Shapefile, 2017, nation, U.S., Current State and Equivalent National.”

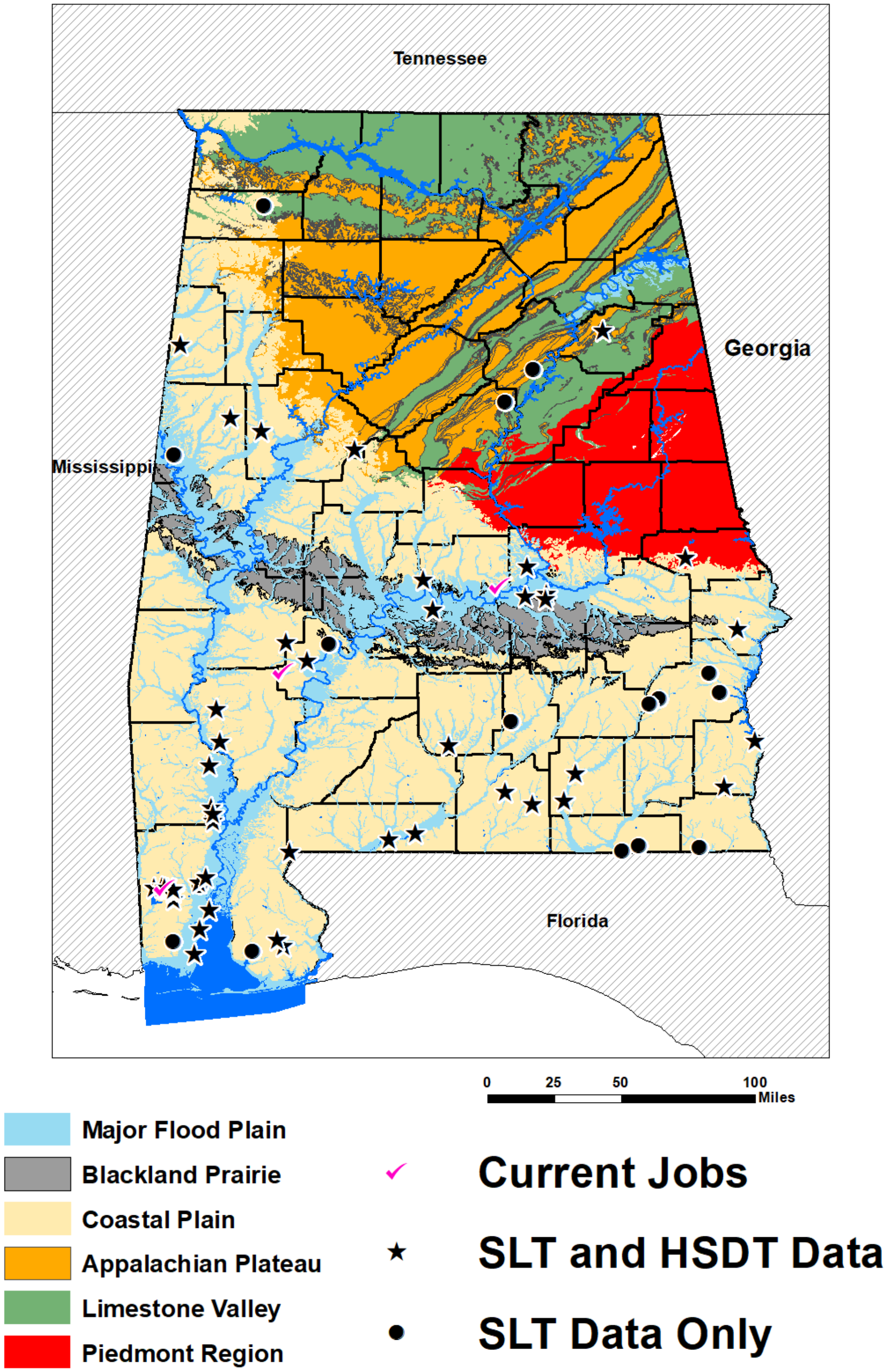


Figure 4.1 Map of ALDOT Data

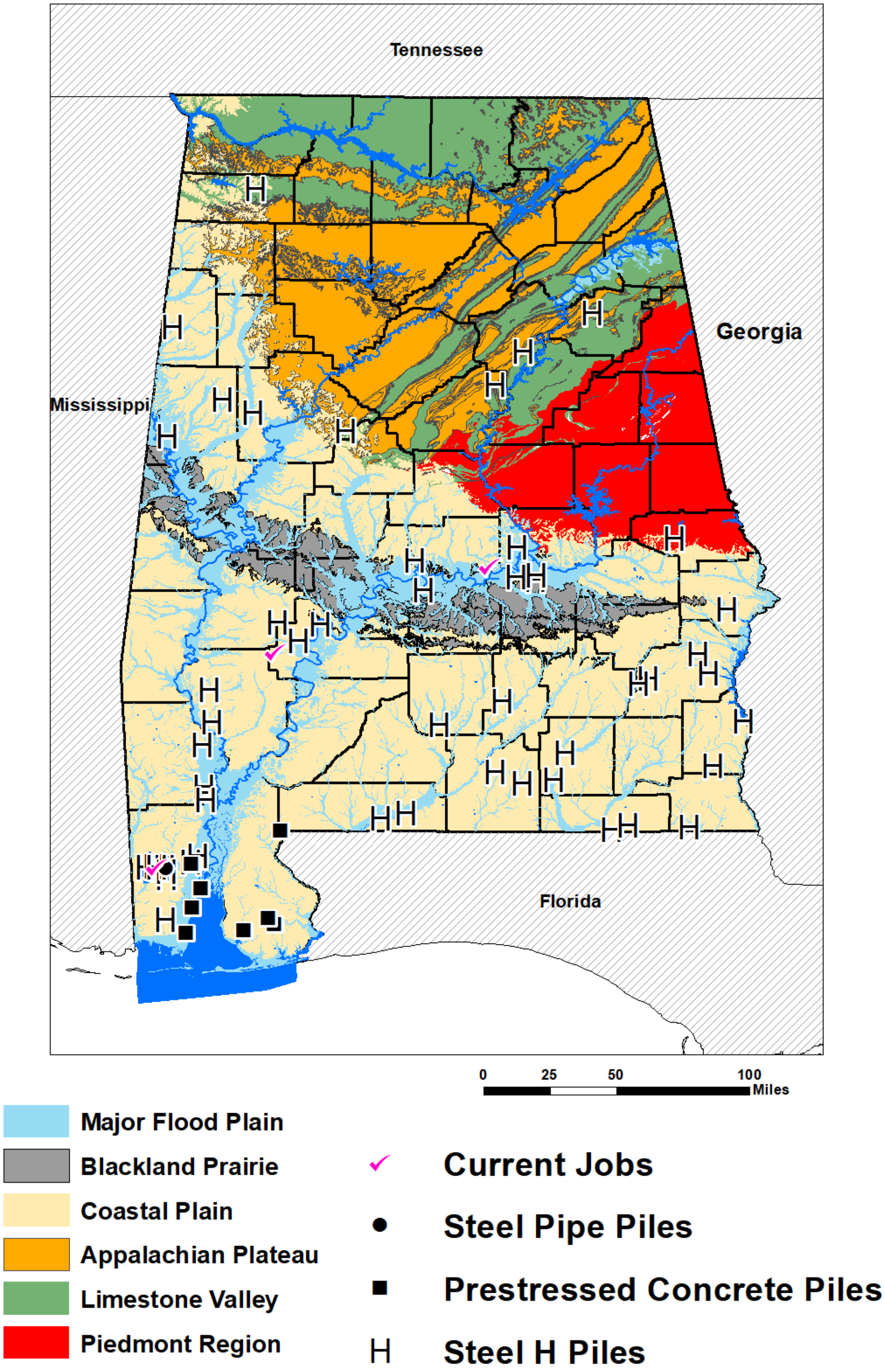


Figure 4.2 Map of ALDOT Data by Pile Type

4.2.1 Collins Creek Tributary Project, Semmes, AL – Steel H-Pile 18X135

A new bridge was being constructed over a series of creeks, streams, and swamp land as part of the US-98 extension. This job specifically was at Collins Creek Tributary. Bent #2 on this bridge consisted of galvanized steel 18x135 H piles. It is worth noting that this project was initially designed for 30" square concrete piles and was value engineered by an outside consultant and changed to the 18x135 H piles. The main thing to note is the change in cross sectional area. The concrete pile had a cross sectional area of 900 in^2, and the 18x135 H pile had a cross sectional area of 39.9 in^2. This was combined with the fact that the tip elevation remained unchanged, despite the large change in cross sectional area. The pile was initially driven to around 80 ft. Multiple set checks were conducted throughout the day, and the desired ultimate capacity was not achieved. The design load for the pile was 262 kips. An iCAP EOD and four-hour iCAP STCK predicted 350 kips and 208 kips of ultimate capacity, respectively. ALDOT looks to achieve two times the design load when testing PDA and three times the design load when conducting static load tests. An additional 30 ft of pile was spliced onto the existing pile, and the pile was continuously driven. The splice took a while to complete since the piles were galvanized steel. Galvanized steel is zinc coated and can be a serious health hazard when ground particulate matter is inhaled. The galvanized steel was ground off, and then the piles were spliced together. Then, a four-day iCAP BOR predicted a capacity of 285 kips. Eventually, the pile was allowed to set up for 12 days, and an ultimate capacity of 635 kips was predicted. It is worth noting that the load test for this location held 786 kips and the plotted static load test results were not close to Davisson failure criterion. Figure 5.3 shows the author in a lift bucket at the job site being hoisted by a crane to install the PDA gauges after the pile splice.

Figure 4.3 Author installing PDA gauges on steel H-Pile

4.2.2 Autauga Creek Project, Prattville, AL – Steel H-Pile 12X53

A new bridge was being constructed on U.S. 82 over Autauga Creek in Autauga County, Alabama. The bridge consisted of 12x53 steel H piles at both abutments and drilled shafts at the bents.

Day 1 – 6/25/2021

On Friday, June 25th, 2021, a 12x53 test pile was driven using a diesel hammer. PDA was conducted on the pile by ALDOT. A 60 ft, 12x53 steel H pile was driven 31 ft, and an EOD iCAP was performed. The EOD iCAP predicted 121 kips of ultimate capacity in the pile. A 30-minute set check was then conducted and the 30-minute STCK iCAP predicted 159 kips of ultimate capacity in the pile. A static load test and a restrike was then scheduled for the following Tuesday (6/29/2021) at 7 AM.

Day 2 – 6/29/2021

On Tuesday, June 29th, 2021, the previously driven test pile underwent a static load test. ALDOT's current practice consists of loading test piles to three times the design load. For this pile, the design load was 60 kips, and the static load test was to be loaded to 180 kips. The load test set up included a reaction beam stick-welded to two reaction piles, a hydraulic jack, piano wire, a mirror and a small tape measure for settlement readings, a jack pressure gage, and a jack pump. The contractor had initially calibrated the jack prior to the jack's arrival on site. The load test was conducted in increments of 5% of the maximum applied load. When the load test reached a load of close to 150 kips, the left side reaction pile began to move upwards, and the hydraulic jack lost pressure. Because the load test had already achieved two times the design load, the foreman, inspector, and ALDOT crew decided to terminate and pass the static load test.

The reaction frame was then removed from the test pile and reaction piles, and strain gauges and accelerometers were reattached to the test pile to conduct a HSDT restrike. Before the HSDT restrike began, the diesel hammer was warmed up on an adjacent reaction pile. The diesel hammer needed to be warmed up prior to the restrike due to the importance of the amount of energy transferred from the hammer to the pile in the first few blows of the restrike in the HSDT results. The iCAP from the restrike predicted an ultimate capacity of 177 kips. This value is close to three times the design load of the pile. The restrike took approximately 45 minutes to complete. That time included the crane operator picking up the leads and hammer and placing them back on the test pile, warming up the hammer, and the setup required with the strain gauges and accelerometers. The static load test took roughly four hours, and no useful information was determined from the test because of the equipment failure.

4.2.3 Cub Creek Project, Pine Hill, AL – Steel H-Pile 12X53

A 55 ft 12x53 steel H pile was driven in Pine Hill, AL on Monday July 12, 2021. Pine Hill is located between Camden, AL and Dixon Mills, AL in Wilcox County. The pile was driven at abutment 1 for a new bridge that is being constructed over state route five. The SPT bore logs drilled at abutment 1 displayed mainly clays, which would be expected in the blackland prairie region of Alabama. A diesel hammer was used to drive the pile. Waiting on several short rainstorms, initial driving began around 9:30 AM. The pile was driven to 29 ft, which was the minimum tip elevation detailed in the plans.

The initial plan was to wait 30 minutes to an hour for a set check, but rain delayed the project until the next day. The iCAP analysis at EOD predicted a capacity of 177 kips. The iCAP results after a set check wait period of 21 hours predicted a capacity of 460 kips. The iCAP results from a three day restrike predicted a capacity of 627.5 kips. A static load test was conducted the same day as the restrike, but data was not available.

4.3 General Dataset Information and Statistics Used for MLA versus HSDT

Of the 81 piles in Dataset 1, 66 were H-piles, 10 were prestressed square concrete piles, and 5 were 30-inch steel pipe piles. Out of the 81 piles in the study, only 14 reached Davisson failure criteria during the static load test. Therefore, the MLA during the static load test was used to compare to HSDT results produced from HSDT data. Generally, the MLA did not produce failure in the piles. For this reason, it is an underestimate of ultimate capacity or Davisson capacity. The customs and standards generally limited the MLA to no more than three times design load. Simple summary statistics were calculated for the original variables together with pertinent variables constructed as ratios of selected original variables.

Simple linear regression was used to model MLA on the *y*-axis and the three HSDT variables on the *x*-axis (HSDT-EOD, HSDT-STCK, and HSDT-BOR). The reference used for properties of linear regression was by Montgomery, Peck, and Vining (2012). Equation 5.1 is the linear model applied to data including the subscripts referencing the observations in the dataset (i = 1 to N) and the error term usually assumed to be normally distributed.

$$y_i = \beta_0 + \beta_1 x_i + \varepsilon_i \tag{5.1}$$

R-Software was used to implement linear regression and to calculate summary statistics of study variables (Wickham and Grolemund, 2017; R Core Team, 2021). The R package ggplot2 was used to generate scatter diagrams with fitted models and residual plots to assess the model fit. The null hypothesis in each case was that the slope and the intercept are equal to zero. The alternative hypothesis was that the slopes and the intercepts are not equal to zero (2-tailed tests).

Significant differences from zero were considered at $\alpha = 0.05$. Even though p-values less than or equal to 0.05 were considered significant, the 95% confidence intervals for the slope and intercept were considered more informative. R^2 values were used as a general assessment of the amount of variability in the data that could be explained by the simple straight-line regression model. Since the models were simple linear regression, unadjusted R^2 was used. The square root of R^2 together with the sign of the slope gives the correlation coefficient. All three of EOD, STCK, and BOR were not available except for a few piles. Therefore, meaningful statistical analysis was not possible if the dataset was restricted to piles with all three of HSDT EOD, STCK, and BOR available. Multiple subsets of the dataset were analyzed. No transformations of the original variables were used for the purpose of reducing variability. Apparent outliers were not removed. These statistical analysis decisions were made because the goal was to show the raw form of the data as might be seen in real engineering practice.

4.4 Russell Dataset versus Hill Dataset

Of the 81 piles studied, 51 piles were compiled by the author from ALDOT projects. All 51 of these projects utilized iCAP analysis. Of the 81 piles, 30 piles were previously studied (Hill, 2007). All 30 of Hill's piles were either analyzed using CAPWAP or just general PDA results. A statistical analysis was performed to determine if there was any significant difference between the two datasets because of this analysis difference. The form of the model used in this analysis is given in Equation 5.2. As before, MLA is the y-variable and HSDT-EOD, HSDT-STCK, and HSDT-BOR are the x-variables. The indicator variable I is 0 for membership in the Russell dataset and 1 for membership in the Hill dataset. Therefore, β_2 is a test of intercept difference between the Hill and Russell data and β_3 is a test of slope difference between the Hill and Russell data.

$$y = \beta_0 + \beta_1 x + \beta_2 I + \beta_3 I x \tag{5.2}$$

Raw linear regression data is presented in Table 4.1, and comparison statistics between the two datasets are presented in Table 4.2.

Table 4.1 Raw Linear Regression Data for Russell and Hill Data sets

	HSDT		Intercept				Slope				R-sq
Dataset	x-variable	N	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*	(unadj)
Russell	EOD	43	146.97	12.39	281.55	*	1.10	0.72	1.49	*	0.450
Russell	STCK	33	89.97	-25.52	205.45		0.98	0.67	1.29	*	0.568
Russell	BOR	30	81.33	-33.98	196.63		0.92	0.67	1.17	*	0.663
Hill	EOD	30	44.96	-16.33	106.24		0.87	0.65	1.09	*	0.703
Hill	STCK	3									
Hill	BOR	7	57.29	-40.38	154.96		0.64	0.43	0.84	*	0.927

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

Table 4.2 Comparison of Russell and Hill Data sets

	EOD (N=73)				BOR (N=37)			
Terms in Multiple Regression Model	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*
Overall Intercept (due to Russell data)	146.97	42.17	251.77	*	81.33	-25.08	187.73	
Overall Slope (due to Russell data)	1.10	0.80	1.40	*	0.92	0.69	1.15	*
Intercept Difference (between Hill and Russell data)	-102.02	-311.66	107.63		-24.03	-277.36	229.29	
Slope Difference (between Hill and Russell data)	-0.23	-0.94	0.49		-0.28	-0.82	0.26	

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

As may be seen, the linear regression analyses are not statistically different between the Russell and Hill datasets. At EOD, the intercept difference estimate was -102.02 (-11.66, 107.6) and the slope difference was -0.23 (-0.94, 0.49). Neither of which are significantly different from zero as noted by the 95% confidence intervals that include zero. At BOR, the slope difference was -24.03 (-277.36, 229.29) and the slope difference was -0.28 (-0.82, 0.26). Again, neither are significantly different from zero. The following Figures 4.4-4.5 show the Russell & Hill datasets separately in scatter diagrams with fitted lines and residual plots for each of EOD, STCK, and BOR. As shown in Table 4.1, each slope is significantly different from zero. The regression and residual plots are presented for completeness, without detailed commentary.

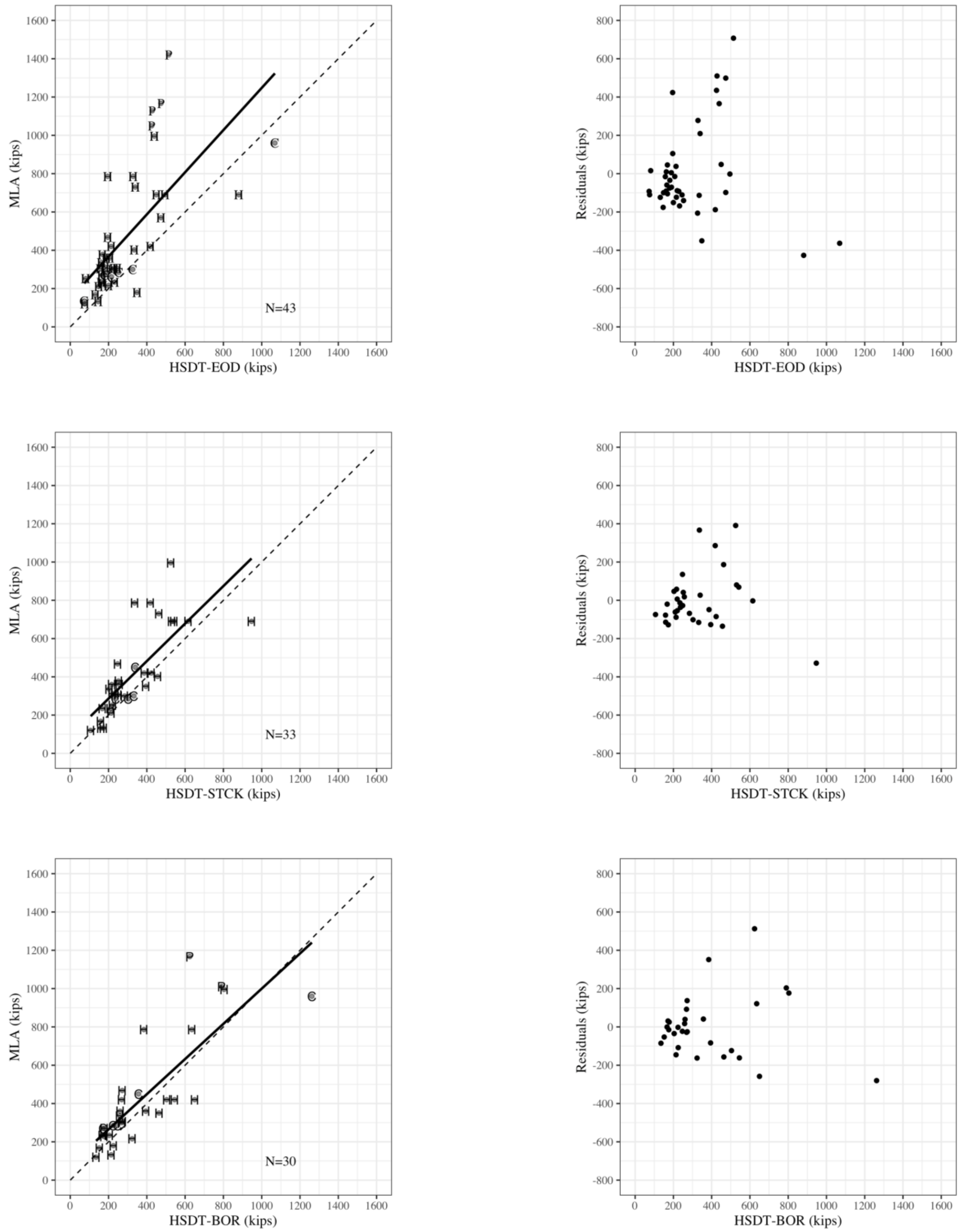


Figure 4.4 Russell Data - Regression models and residual scatter plots for all piles with y=x reference line: H-piles (H), pipe piles (P), concrete piles (C).

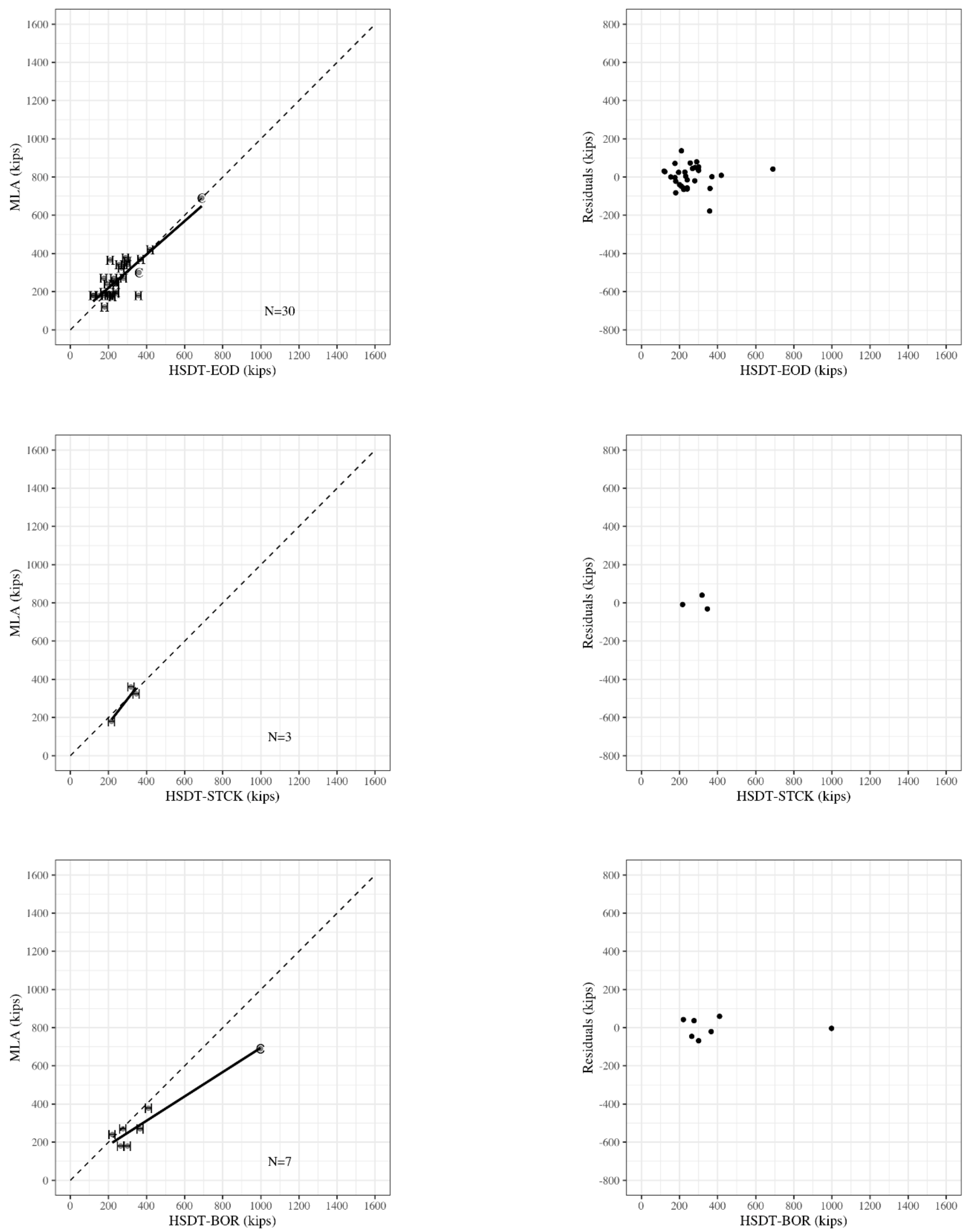


Figure 4.5 Hill Data - Regression models and residual scatter plots for all piles with y=x reference line: H-piles (H), pipe piles (P), concrete piles (C).

4.5 General outline and order for statistical analysis

The following subsets of Dataset 1 were considered based on potentially different pile or soil characteristics. For each subset, the following are displayed: a table for summary statistics for original and constructed ratio variables, a table containing linear regression details, regression model and scatter plots for EOD, STCK, and BOR, as well as residual plots for EOD, STCK, and BOR. Each subset is detailed and listed below and will appear in the order listed starting in chapter 4.5.1.

a) All piles, N=81
b) All H-piles, N=66
c) Common H-piles (12x53, 14x73, 14x89), N=53
d) 12x53 H-piles, N=31
e) All H-piles in sand, N=44
f) All H-piles in clay, N=22
g) All pipe piles, N=5
h) All concrete piles, N=10

The tables for pipe piles are included for completeness but no further analysis could be done for these subsets due to small sample sizes. All load measurements are presented in kips. Time elapsed until set check is measured in hours. Time elapsed until beginning of restrike is measured in days.

4.5.1 All Piles

Since the Russell and Hill datasets appear comparable in the relationship of MLA to HSDT variables, the datasets were combined for each further analysis (previously designated Dataset 1, N=81). Of the 81 piles, 66 were H-piles, 10 were prestressed square concrete piles, and 5 were 30-inch steel pipe piles. Out of the 81 piles in the study, only 14 reached Davisson failure criteria during the static load test. Therefore, for a large portion of the study, the MLA during the static load test was used to compare to HSDT variables. The panel of summary and regression analyses for this dataset of all piles consists of Tables 4.3-4.5 and Figure 4.6.

Table 4.3 Summary statistics for original variables for All Piles

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	81	135.3	93.1	40.0	104.0	474.0
Davisson Capacity (kips)	14	501.3	410.3	135.0	305.5	1420.0
MLA (kips)	81	393.4	274.9	120.0	305.0	1420.0
HSDT-EOD (kips)	73	277.4	166.3	73.1	228.0	1068.9
HSDT-STCK (kips)	36	325.0	164.0	106.5	270.4	946.9
STCK hours	34	2.8	6.0	0.3	0.5	24.0
HSDT-BOR (kips)	37	385.5	252.7	135.0	272.2	1262.0
BOR days	34	9.6	11.6	1.0	7.0	60.0

N=81, all piles included

Table 4.4 Summary statistics for constructed ratio variables for All Piles

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	81	2.91	0.32	1.20	3.00	3.10
HSDT-EOD (kips)	DL (kips)	73	2.37	0.99	0.75	2.35	5.97
HSDT-STCK (kips)	DL (kips)	36	2.54	0.74	1.22	2.56	4.12
HSDT-BOR (kips)	DL (kips)	37	3.01	0.99	1.47	2.69	5.00
HSDT-EOD (kips)	MLA (kips)	73	0.82	0.33	0.25	0.80	1.99
HSDT-STCK (kips)	MLA (kips)	36	0.87	0.23	0.43	0.88	1.37
HSDT-BOR (kips)	MLA (kips)	37	1.00	0.33	0.49	0.90	1.67
HSDT-STCK (kips)	HSDT-EOD (kips)	30	1.30	0.36	0.90	1.30	2.71
HSDT-BOR (kips)	HSDT-STCK (kips)	20	1.17	0.21	0.82	1.17	1.53
HSDT-BOR (kips)	HSDT-EOD (kips)	31	1.38	0.43	0.88	1.31	3.24

N=81, all piles included

In Table 4.3, the design load mean was 135.3 kips (93.1), and the maximum load applied mean was 393.4 kips (274.9). The sample size for each of these variables was 81. The Davisson capacity was available for 14 piles with a mean of 501.3 kips (410.3). The sample sizes available for HSDT measures were all less than 81. The sample size for EOD was 73, for STCK was 36, and for BOR was 37. The distributions of each of the load variables presented in Table 4.3 have long tails to the right as indicated by the means being greater than the median. This distributional characteristic is expected because the values are positive measurements with a tendency toward large values which have the effect of increasing the mean without affecting the median. In Table 4.4, MLA is compared to DL and found to be approximately 3 across the distribution. The HSDT capacity at EOD as a multiple of DL was 2.37 (0.99). Likewise, the HSDT capacity at STCK and BOR as a multiple of DL were 2.54 (0.74) and 3.01 (0.99), respectively. The HSDT capacity at EOD, STCK, and BOR as a multiple of MLA were 0.82 (0.33), 0.87 (0.23), and 1.00 (0.33), respectively. Thus, the values appear to be increasing with time. The HSDT capacity at STCK compared to EOD yields a ratio of 1.3, an increase of 30%, thus showing soil set up with time. The HSDT capacity at BOR compared to STCK exhibits a ratio of 1.17, an increase of 17%. Finally, the HSDT capacity at BOR compared to the HSDT capacity at EOD resulted in a ratio of 1.38, an increase of 38%. However, it is worth noting that EOD, STCK, and BOR are not all provided for each pile. Thus, the ratios are calculated for different subsets of the 81 piles, and this may be a source of error. The regression models are summarized in Table 4.5. The R-squared values are 0.451, 0.568, and 0.642 for EOD, STCK, and BOR, respectively. Each of the slopes are significantly different from zero, but none of the intercepts are significantly different from zero as shown. The sample sizes available were 73 at EOD, 36 at STCK, and 37 at BOR. The regression and residual plots corresponding to the values in Table 4.5 are given in Figure 4.6.

Table 4.5 Linear Regression Details for All Piles

	HSDT		Intercept				Slope				R-sq
Dataset	x-variable	N	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*	(unadj)
All piles	EOD	73	78.40	-15.29	172.09		1.11	0.82	1.40	*	0.451
	STCK	36	78.57	-30.99	188.15		0.99	0.69	1.30	*	0.568
	BOR	37	78.87	-22.08	179.82		0.86	0.64	1.08	*	0.642

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

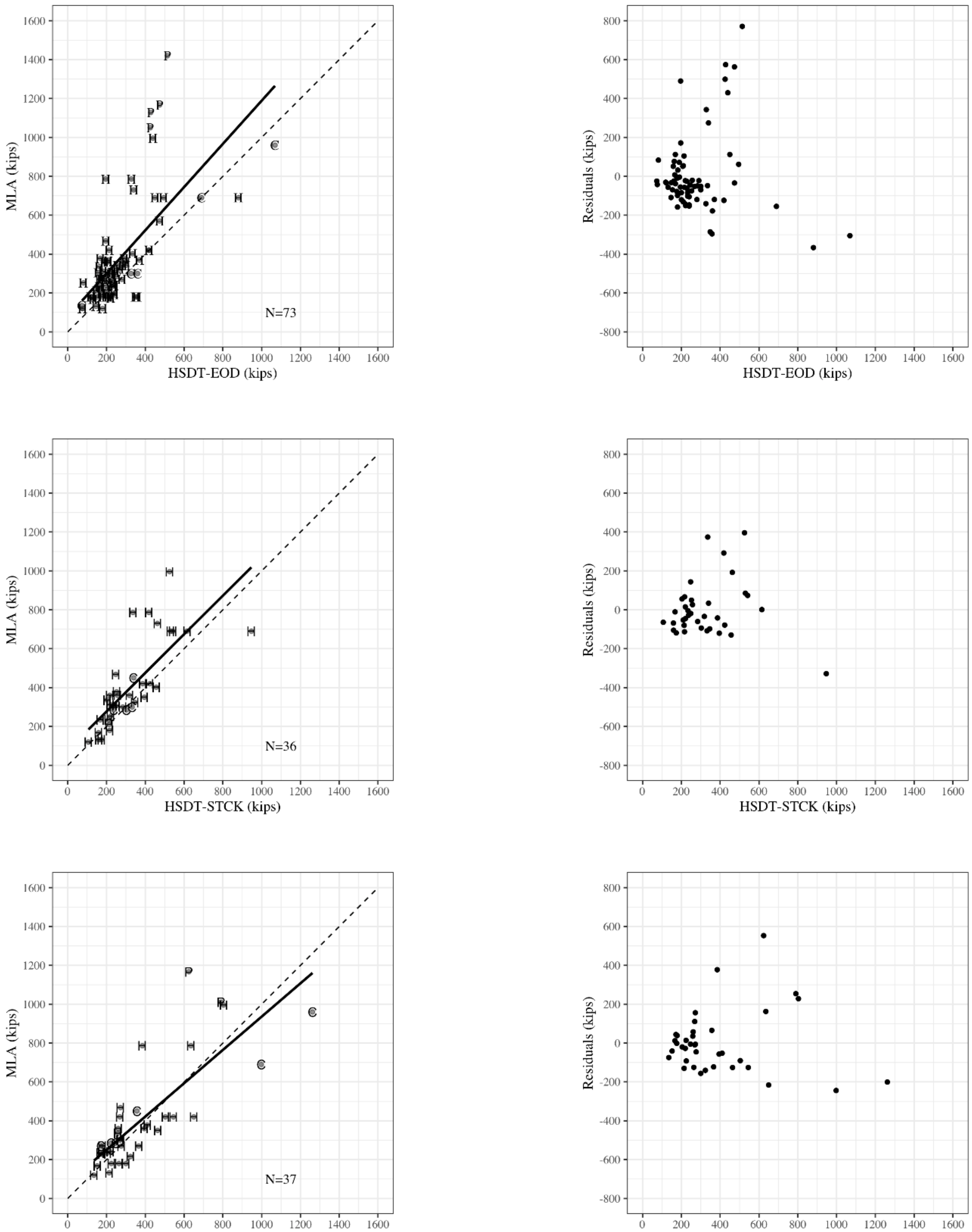


Figure 4.6 Regression models and residual scatter plots for all piles with y=x reference line: H-piles (H), pipe piles (P), concrete piles (C).

4.5.2 All H-Piles, N=66

The panel of summary and regression analyses for this dataset of all H-piles consists of Tables 4.6-4.8 and Figure 4.7. The summary statistics for all H-piles are presented in Tables 4.6 and 4.7. The DL mean was 115.2 kips (62.5), and the MLA mean was 335.7 kips (186.8). The sample size for each of these variables was 66. The sample sizes for HSDT capacity at EOD, HSDT capacity at STCK, and HSDT capacity at BOR were 60, 32, and 29, respectively. The HSDT capacity at EOD, HSDT capacity at STCK, and HSDT capacity at BOR were 250.9 kips (126.3), 327.8 kips (173.4), and 330.7 kips (164.4). Similarly, to the dataset for all piles, the H pile only dataset exhibited a trend of long tails to the right as indicated by the means being greater than the medians for each load variable. Maximum load applied for H piles compared to design load exhibits similar results to the overall dataset. The HSDT capacity at EOD as a multiple of design load was 2.43 (0.99). The HSDT capacity at STCK and BOR as a multiple of DL were 2.51 (0.76) and 3.10 (1.00), respectively. HSDT capacity estimates show trends of increasing with time but again it must be noted that these means represent different piles in the dataset and have potential for error.

Table 4.6 Summary statistics for original variables for all H-Piles

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	66	115.2	62.5	40.0	103.0	332.0
Davisson Capacity (kips)	9	336.0	163.0	196.0	305.0	730.0
MLA (kips)	66	335.7	186.8	120.0	302.5	996.0
HSDT-EOD (kips)	60	250.9	126.3	75.6	219.9	880.9
HSDT-STCK (kips)	32	327.8	173.4	106.5	254.4	946.9
STCK hours	31	3.1	6.2	0.3	0.5	24.0
HSDT-BOR (kips)	29	330.7	164.4	135.0	271.8	803.5
BOR days	27	7.8	8.3	1.0	6.0	37.0

N=66, all H-piles included

Table 4.7 Summary statistics for constructed ratio variables for all H-Piles

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	66	2.93	0.31	1.20	3.00	3.10
HSDT-EOD (kips)	DL (kips)	60	2.43	0.99	0.75	2.38	5.97
HSDT-STCK (kips)	DL (kips)	32	2.51	0.76	1.22	2.53	4.12
HSDT-BOR (kips)	DL (kips)	29	3.10	1.00	1.47	2.75	5.00
HSDT-EOD (kips)	MLA (kips)	60	0.84	0.34	0.25	0.81	1.99
HSDT-STCK (kips)	MLA (kips)	32	0.86	0.24	0.43	0.88	1.37
HSDT-BOR (kips)	MLA (kips)	29	1.03	0.33	0.49	0.92	1.67
HSDT-STCK (kips)	HSDT-EOD (kips)	27	1.32	0.38	0.90	1.24	2.71
HSDT-BOR (kips)	HSDT-STCK (kips)	17	1.21	0.19	0.95	1.19	1.53
HSDT-BOR (kips)	HSDT-EOD (kips)	25	1.43	0.46	0.88	1.38	3.24

N=66, all H-piles

Table 4.8 Linear Regression Statistics for all H-Piles

	HSDT		Intercept				Slope				R-sq
Dataset	x-variable	N	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*	(unadj)
All H-piles	EOD	60	101.03	15.81	186.25	*	0.92	0.62	1.23	*	0.389
	STCK	32	86.45	-31.42	204.31		0.99	0.67	1.31	*	0.571
	BOR	29	28.64	-83.61	140.89		0.96	0.65	1.26	*	0.607

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

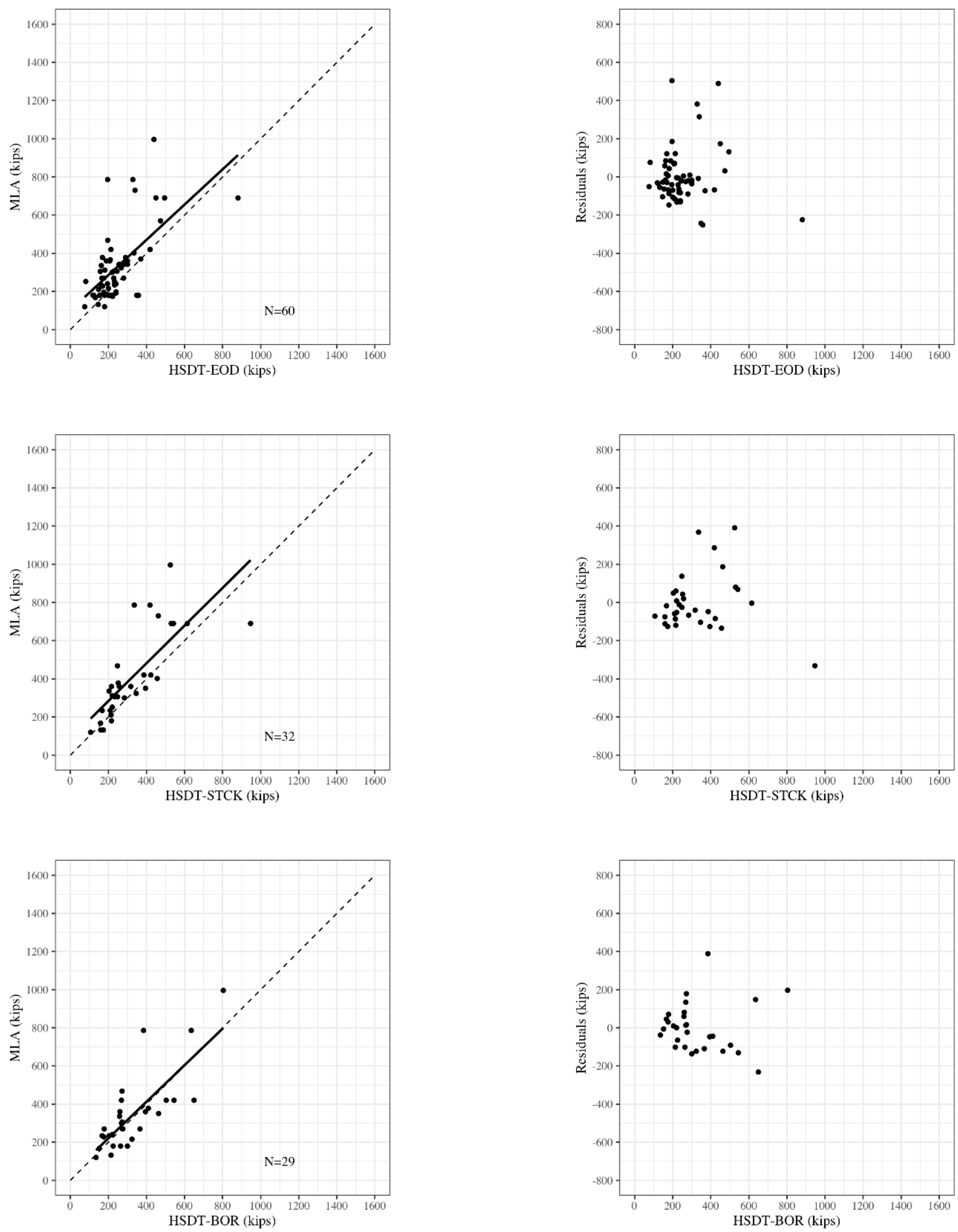


Figure 4.7 Regression models and residual scatter plots for all H-piles with y=x reference line.

4.5.3 Common H-Piles

As the author became more acquainted with ALDOT practice and the available data, it became clear that ALDOT most commonly drives piles of sizes: 12x53, 14x73, and 14x89. There were 31 piles of size 12x53, 13 of size 14x73, and 9 of size 14x89 for a total sample size of 53. The panel of summary and regression analyses for this dataset of common H-piles consists of Tables 4.9-4.11 and Figure 4.8. Tables 4.5 and 4.6 summarize the data from this subset. The design load mean was 107.75 kips (48.54), the maximum load of the static load test was 316.74 kips (143.03), and the mean of the ultimate capacity was 316.16 kips (142.99). The sample size for these variables was 53. The HSDT capacity at EOD, HSDT capacity at STCK, and HSDT capacity at BOR divided by the design load was 2.56 (1.01), 2.69 (0.72), and 3.30 (0.98), respectively. The results from this data set also displays trends of soil set up over time. The sample size for HSDT EOD, HSDT STCK, and HSDT BOR were 47, 25, and 23, respectively.

Table 4.9 Summary statistics for original variables for common H-Piles

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	53	107.8	48.5	44.0	104.0	230.0
Davisson Capacity (kips)	7	284.0	79.1	196.0	270.0	402.0
MLA (kips)	53	316.2	143.0	132.0	305.0	690.0
HSDT-EOD (kips)	47	256.2	134.2	81.3	219.8	880.9
HSDT-STCK (kips)	25	325.8	183.1	158.8	251.7	946.9
STCK hours	24	1.6	4.0	0.3	0.5	20.0
HSDT-BOR (kips)	23	310.5	130.4	151.9	268.9	649.9
BOR days	22	7.1	6.3	1.0	5.5	28.0
N=53, H-piles of sizes: 12x53, 14x73, 14x89						

Table 4.10 Summary statistics for constructed ratio variables for common H-Piles

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	53	2.96	0.25	1.20	3.00	3.10
HSDT-EOD (kips)	DL (kips)	47	2.56	1.01	0.84	2.48	5.97
HSDT-STCK (kips)	DL (kips)	25	2.69	0.72	1.22	2.67	4.12
HSDT-BOR (kips)	DL (kips)	23	3.30	0.98	1.92	3.25	5.00
HSDT-EOD (kips)	MLA (kips)	47	0.86	0.33	0.32	0.83	1.99
HSDT-STCK (kips)	MLA (kips)	25	0.93	0.22	0.60	0.90	1.37
HSDT-BOR (kips)	MLA (kips)	23	1.10	0.33	0.64	1.08	1.67
HSDT-STCK (kips)	HSDT-EOD (kips)	20	1.31	0.38	0.90	1.23	2.71
HSDT-BOR (kips)	HSDT-STCK (kips)	12	1.17	0.18	0.95	1.18	1.53
HSDT-BOR (kips)	HSDT-EOD (kips)	19	1.33	0.25	0.88	1.31	1.90

N=53, H-piles of sizes: 12x53, 14x73, 14x89

Table 4.11 Linear Regression statistics for common H-Piles

	HSDT		Intercept				Slope				R-sq
Dataset	x-variable	N	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*	(unadj)
Common H-piles	EOD	47	115.70	57.74	173.66	*	0.76	0.56	0.96	*	0.562
	STCK	25	92.55	20.34	164.77	*	0.81	0.62	1.01	*	0.765
	BOR	23	134.76	58.99	210.52	*	0.50	0.27	0.72	*	0.497

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

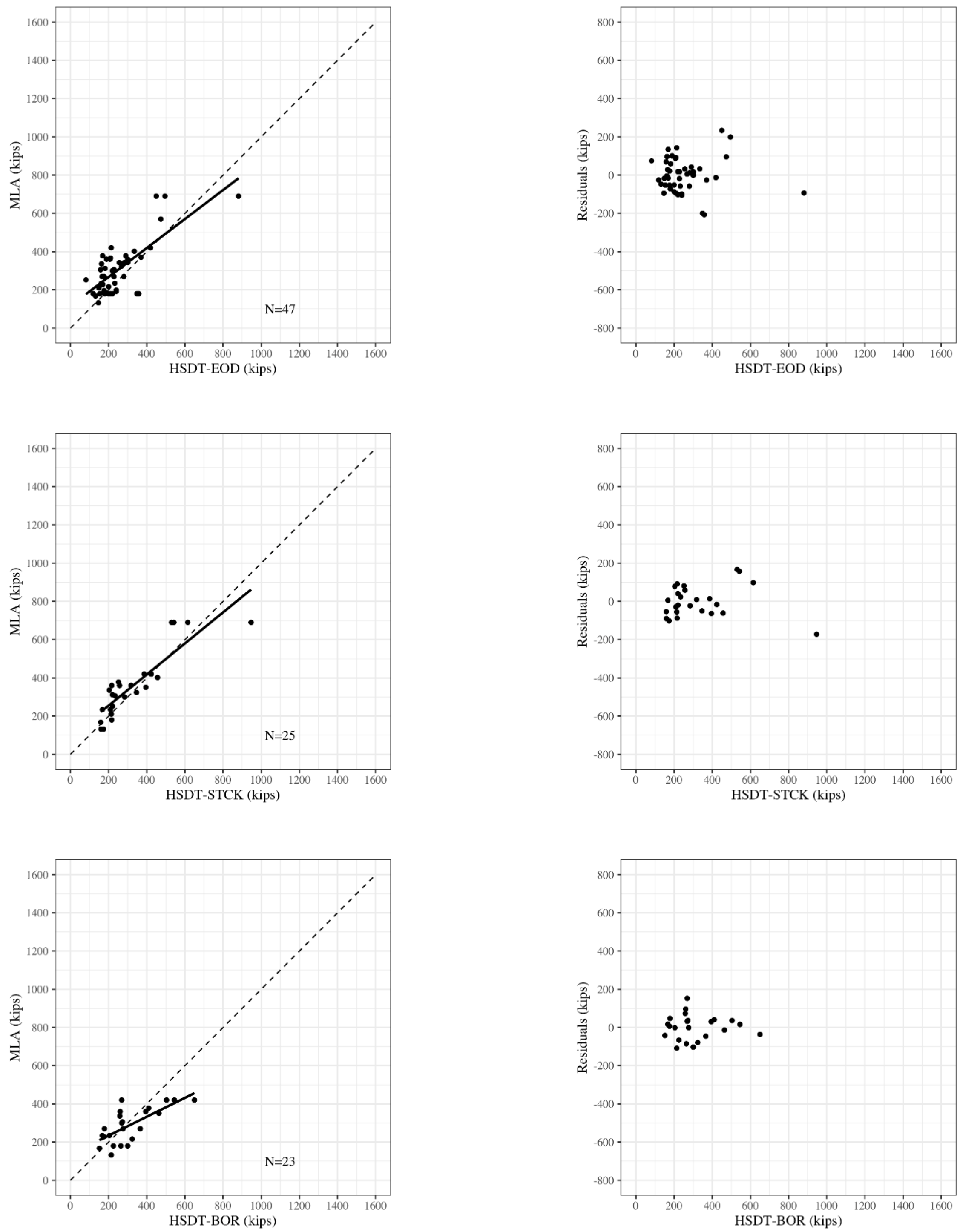


Figure 4.8 Regression models and residual scatter plots for common H-piles with y=x reference line.

4.5.4 12X53 H-Piles

The largest and theoretically most homogeneous subset of the entire dataset was the H-piles of size 12x53 with N=31. The panel of summary and regression analyses for this dataset of 12x53 H-piles consists of Tables 4.12-4.14 and Figure 4.9. The summary statistics for the original variables are presented in Table 4.15 and the ratio variables in Table 4.16.

Table 4.12 Summary statistics for original variables for 12x53 H-Piles

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	31	83.7	33.1	44.0	72.0	176.0
Davisson Capacity (kips)	4	227.8	32.2	196.0	222.5	270.0
MLA (kips)	31	239.8	82.5	132.0	211.0	420.0
HSDT-EOD (kips)	29	207.4	73.0	81.3	199.9	419.0
HSDT-STCK (kips)	12	211.6	45.2	158.8	215.0	318.0
STCK hours	12	0.9	0.9	0.3	0.5	3.5
HSDT-BOR (kips)	31	234.3	53.8	151.9	242.9	323.7
BOR days	31	6.2	7.1	1.0	5.0	28.0

N=31, H-piles of size 12x53

Table 4.13 Summary statistics for constructed variables for 12x53 H-Piles

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	31	2.94	0.33	1.20	3.00	3.10
HSDT-EOD (kips)	DL (kips)	29	2.68	1.20	0.84	2.53	5.97
HSDT-STCK (kips)	DL (kips)	12	2.60	0.82	1.22	2.64	3.94
HSDT-BOR (kips)	DL (kips)	14	3.15	1.12	1.92	2.69	5.00
HSDT-EOD (kips)	MLA (kips)	29	0.91	0.38	0.32	0.84	1.99
HSDT-STCK (kips)	MLA (kips)	12	0.92	0.23	0.60	0.89	1.31
HSDT-BOR (kips)	MLA (kips)	14	1.05	0.37	0.64	0.90	1.67
HSDT-STCK (kips)	HSDT-EOD (kips)	11	1.37	0.52	0.90	1.19	2.71
HSDT-BOR (kips)	HSDT-STCK (kips)	6	1.09	0.13	0.96	1.08	1.23
HSDT-BOR (kips)	HSDT-EOD (kips)	13	1.25	0.23	0.88	1.20	1.62

N=31, H-piles of size 12x53

Table 4.14 Linear Regression statistics for 12x53 H-Piles

	HSDT		Intercept				Slope				R-sq
Dataset	x-variable	N	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*	(unadj)
12x53 H-piles	EOD	29	180.78	87.34	274.22	*	0.31	-0.11	0.74		0.077
	STCK	12	-74.3	-260.63	112.03		1.51	0.65	2.37	*	0.604
	BOR	14	157.13	-58.26	372.52		0.36	-0.54	1.26		0.06

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

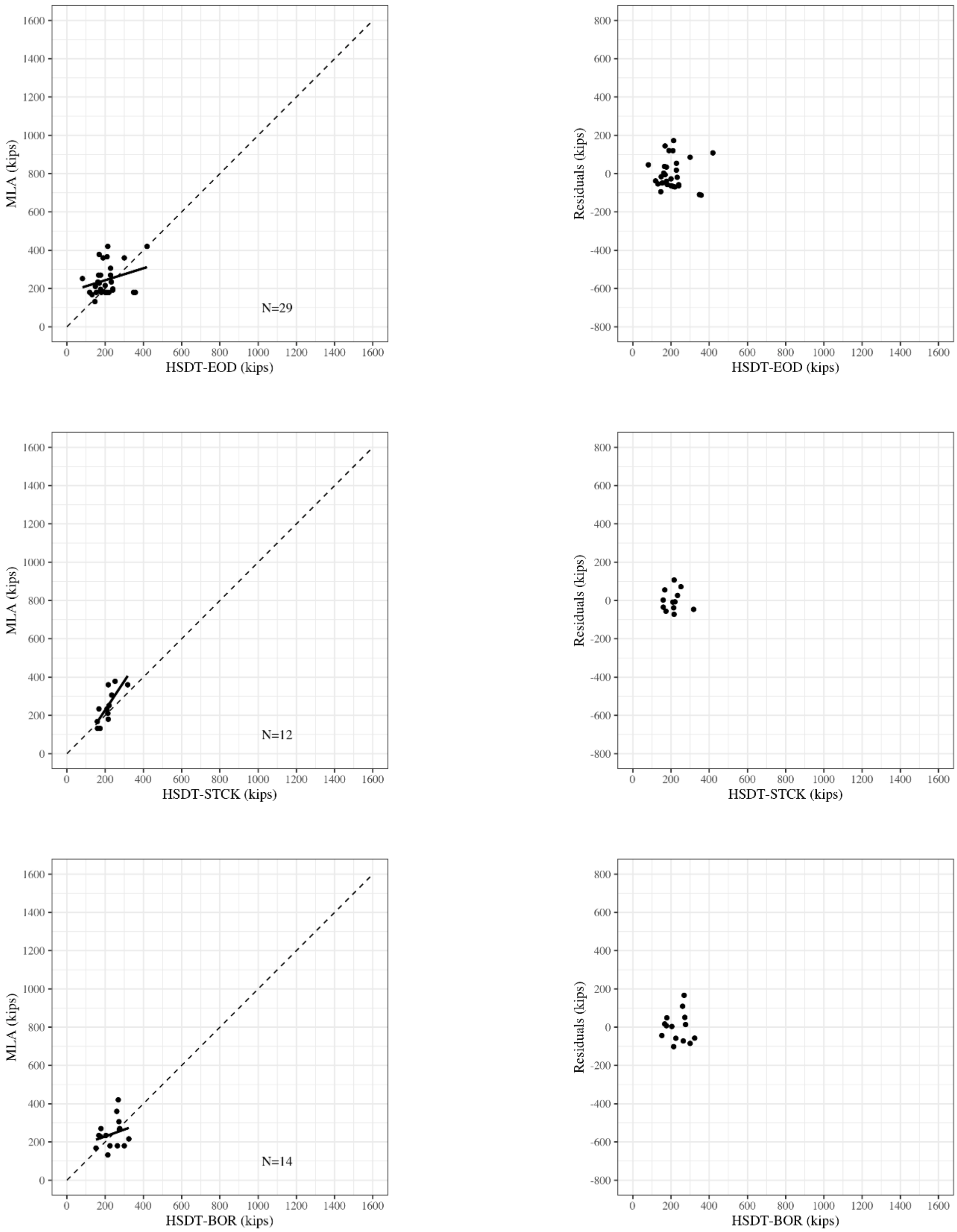


Figure 4.9 Regression models and residual scatter plots for 12x53 H-piles with y=x reference line.

4.5.5 H-Piles in Sand

Of the 66 H piles in the study, 44 were driven in sand and 22 were driven in clay. The panel of summary and regression analyses for this dataset of H-piles in sand consists of Tables 4.15-4.17 and Figure 4.10. The mean design load was 110.47 kips (60.13), the mean of the maximum load of the static load test was 323.40 kips (178.14), and the mean of the maximum load applied was 322.92 kips (178.20).

Table 4.15 Summary statistics for original variables for H-Piles in sand

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	44	110.5	60.1	40.0	101.0	332.0
Davisson Capacity (kips)	7	274.9	70.8	196.0	270.0	402.0
MLA (kips)	44	322.9	178.2	120.0	285.0	996.0
HSDT-EOD (kips)	41	225.5	93.2	75.6	207.2	473.6
HSDT-STCK (kips)	21	274.0	110.2	106.5	247.5	525.4
STCK hours	20	4.5	7.4	0.3	0.9	24.0
HSDT-BOR (kips)	24	322.6	172.7	135.0	268.7	803.5
BOR days	23	8.1	8.9	1.0	7.0	37.0

N=44, H-piles driven in sand

Table 4.16 Summary statistics for constr. ratio variables for H-Piles in sand

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	44	2.95	0.28	1.20	3.00	3.10
HSDT-EOD (kips)	DL (kips)	41	2.31	0.98	0.75	2.25	5.97
HSDT-STCK (kips)	DL (kips)	21	2.38	0.79	1.22	2.30	3.94
HSDT-BOR (kips)	DL (kips)	24	2.99	1.02	1.47	2.70	5.00
HSDT-EOD (kips)	MLA (kips)	41	0.78	0.31	0.25	0.75	1.99
HSDT-STCK (kips)	MLA (kips)	21	0.83	0.25	0.43	0.81	1.31
HSDT-BOR (kips)	MLA (kips)	24	1.00	0.34	0.49	0.90	1.67
HSDT-STCK (kips)	HSDT-EOD (kips)	19	1.38	0.43	1.01	1.25	2.71
HSDT-BOR (kips)	HSDT-STCK (kips)	15	1.22	0.19	0.95	1.19	1.53
HSDT-BOR (kips)	HSDT-EOD (kips)	21	1.46	0.48	1.03	1.38	3.24

N=44, H-piles driven in sand

Table 4.17 Linear Regression statistics for H-Piles in Sand

	HSDT		Intercept				Slope				R-sq
Dataset	x-variable	N	Estimate	Lower CI	Upper CI	Sig*	Estimate	Lower CI	Upper CI	Sig*	(unadj)
H-piles in sand	EOD	41	67.66	-58.36	193.67		1.13	0.62	1.65	*	0.335
	STCK	21	-64.28	-245.24	116.68		1.56	0.98	2.21	*	0.609
	BOR	24	26.87	-95.53	149.27		1.02	0.68	1.35	*	0.641

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

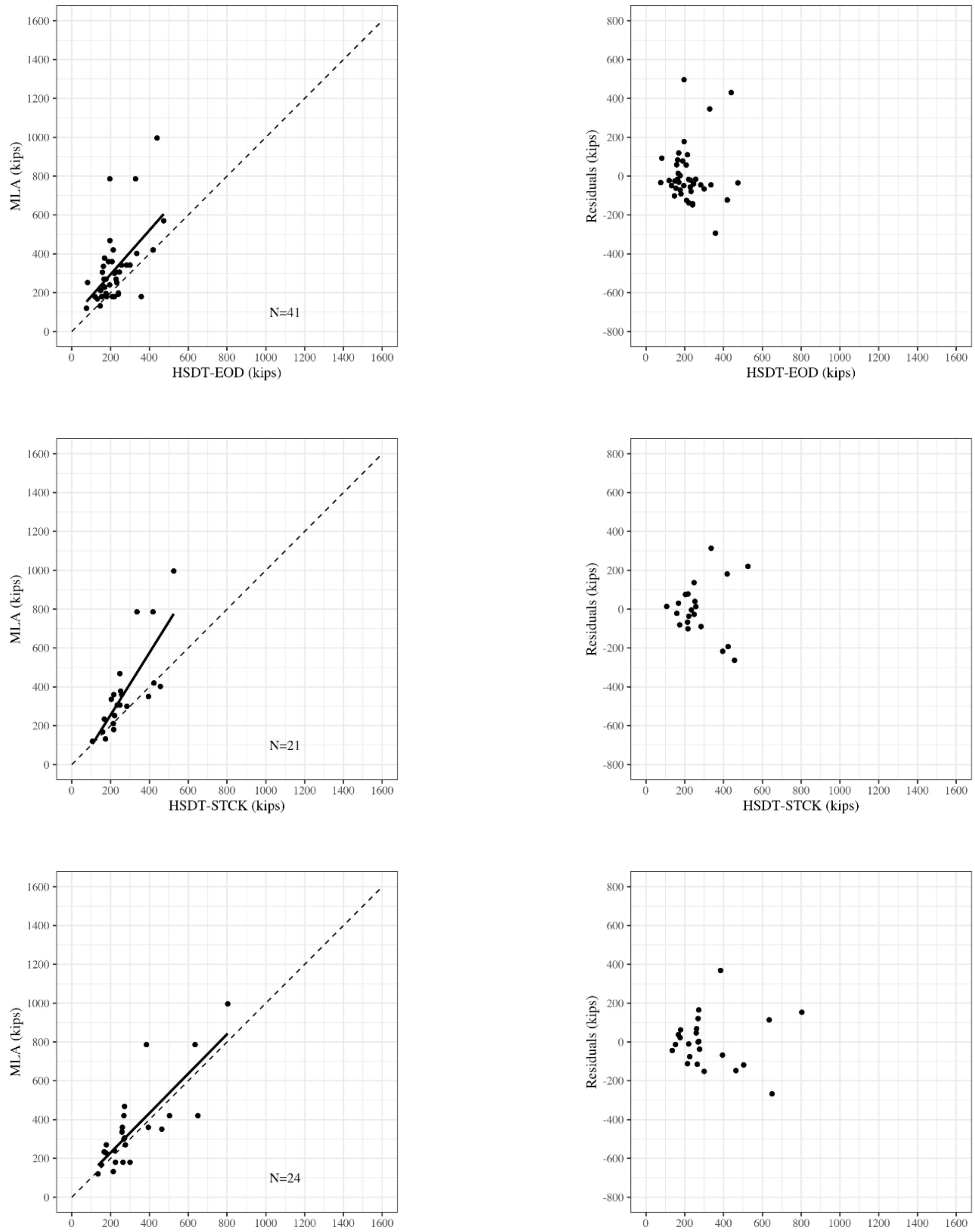


Figure 4.10 Regression models and residual scatter plots for H-piles in sand with y=x reference line.

4.5.6 H-Piles in Clay

The panel of summary and regression analyses for this dataset of H-piles in clay consists of Tables 4.18-4.20 and Figure 4.11.

Table 4.18 Summary statistics for original variables for H-Piles in clay

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	22	124.6	67.3	44.0	124.6	260.0
Davisson Capacity (kips)	2	550.0	254.6	370.0	550.0	730.0
MLA (kips)	22	361.2	204.8	120.0	318.0	730.0
HSDT-EOD (kips)	19	305.8	168.3	122.6	268.0	880.9
HSDT-STCK (kips)	11	430.5	226.3	159.6	386.3	946.9
STCK hours	11	0.5	0.1	0.3	0.5	0.7
HSDT-BOR (kips)	5	369.7	124.3	204.1	366.0	544.8
BOR days	4	6.3	2.5	5.0	5.0	10.0

N=22, H-piles driven in clay

Table 4.19 Summary statistics for constr. ratio variables for H-Piles in clay

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	22	2.88	0.37	1.53	3.00	3.05
HSDT-EOD (kips)	DL (kips)	19	2.68	0.99	1.31	2.50	5.81
HSDT-STCK (kips)	DL (kips)	11	2.75	0.68	1.78	2.67	4.12
HSDT-BOR (kips)	DL (kips)	5	3.66	0.74	2.62	3.89	4.50
HSDT-EOD (kips)	MLA (kips)	19	0.96	0.36	0.47	0.93	1.94
HSDT-STCK (kips)	MLA (kips)	11	0.92	0.22	0.63	0.89	1.37
HSDT-BOR (kips)	MLA (kips)	5	1.22	0.25	0.87	1.30	1.50
HSDT-STCK (kips)	HSDT-EOD (kips)	8	1.17	0.15	0.90	1.20	1.36
HSDT-BOR (kips)	HSDT-STCK (kips)	2	1.19	0.31	0.97	1.19	1.41
HSDT-BOR (kips)	HSDT-EOD (kips)	4	1.30	0.31	0.88	1.36	1.62

N=22, H-piles driven in clay

Table 4.20 Linear Regression statistics for H-Piles in clay

Dataset	HSDT x-variable	N	Intercept Estimate	Intercept Lower CI	Intercept Upper CI	Intercept Sig*	Slope Estimate	Slope Lower CI	Slope Upper CI	Slope Sig*	R-sq (unadj)
H-piles in clay	EOD	19	86.51	-55.92	228.93		0.87	0.46	1.28	*	0.541
	STCK	11	132.39	-69.58	334.35		0.81	0.39	1.23	*	0.677
	BOR	5	70.07	-186.03	326.16		0.63	-0.03	1.3		0.754

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

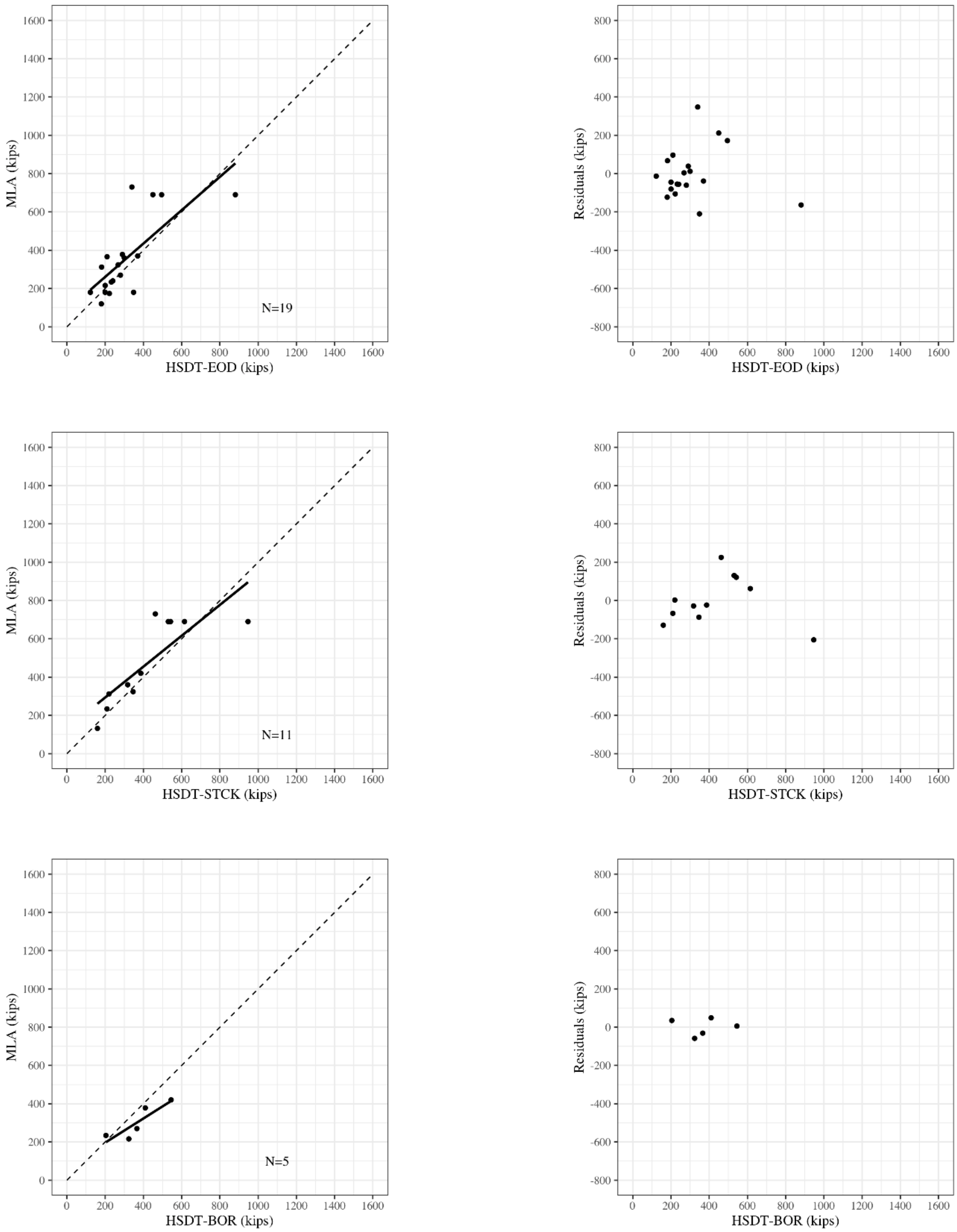


Figure 4.11 Regression models and residual scatter plots for H-piles in clay with y=x reference line.

4.5.7 Steel Pipe Piles

The panel of summary analyses for this dataset of steel pipe piles consists only of Tables 4.21 and 4.22. All 5 pipe piles were 30" in diameter and were all driven in sand. The information in these tables is shown to demonstrate completeness and thoroughness of data exploration, but the small sample size precludes regression analysis. The pipe piles were included in the regression analysis of all piles and were labeled for identification in the scatter plots for visual comparison.

Table 4.21 Summary statistics for original variables for steel pipe piles

Variable measured	N	Mean	Min	Median	Max
DL (kips)	5	399.1	336.4	389.0	474.0
Davisson Capacity (kips)	3	1199.3	1050.0	1128.0	1420.0
MLA (kips)	5	1155.1	1010.0	1128.0	1420.0
HSDT-EOD (kips)	4	460.3	425.4	450.8	514.0
HSDT-STCK (kips)	0				
STCK hours	0				
HSDT-BOR (kips)	2	707.2	624.2		790.1
BOR days	1	6.0			

N=5, 30-inch pipe piles driven in sand

Table 4.22 Summary statistics for constructed ratio variables for steel pipe piles

Numerator variable	Denominator variable	N	Mean	Min	Median	Max
MLA (kips)	DL (kips)	5	2.90	2.58	3.00	3.00
HSDT-EOD (kips)	DL (kips)	4	1.11	1.05	1.09	1.22
HSDT-STCK (kips)	DL (kips)	0				
HSDT-BOR (kips)	DL (kips)	2	1.98	1.61		2.35
HSDT-EOD (kips)	MLA (kips)	4	0.39	0.36	0.39	0.41
HSDT-STCK (kips)	MLA (kips)	0				
HSDT-BOR (kips)	MLA (kips)	2	0.66	0.53		0.78
HSDT-STCK (kips)	HSDT-EOD (kips)	0				
HSDT-BOR (kips)	HSDT-STCK (kips)	0				
HSDT-BOR (kips)	HSDT-EOD (kips)	1	1.32			

N=5, 30-inch pipe piles driven in sand

4.5.8 Concrete Piles

The panel of summary and regression analyses for this dataset of concrete piles consists of Tables 4.23-4.25 and Figure 4.12. Tables 4.13 and 4.14 show summary statistics for the 10 prestressed square concrete piles in the dataset. All concrete piles were driven in sand. The mean design load was 136.00 kips (78.31) and the mean of the maximum load applied was 393.60 kips (247.81). As with the pipe pile subset, the sparsity of this subset prevents in-depth considerations and is included for completeness.

Table 4.23 Summary statistics for original variables for concrete piles

Variable measured	N	Mean	SD	Min	Median	Max
DL (kips)	10	136.0	78.3	90.0	97.5	320.0
Davisson Capacity (kips)	2	198.0		135.0		261.0
MLA (kips)	10	393.6	247.8	135.0	292.5	960.0
HSDT-EOD (kips)	9	373.0	313.6	73.1	253.1	1068.9
HSDT-STCK (kips)	4	302.8	47.0	236.6	317.4	339.9
STCK hours	3	0.4	0.1	0.3	0.3	0.5
HSDT-BOR (kips)	6	543.3	466.0	171.8	301.9	1262.0
BOR days	6	18.3	20.7	7.0	11.0	60.0
N=10, all concrete piles, each driven in sand						

Table 4.24 Summary statistics for constructed ratio variables for concrete piles

Numerator variable	Denominator variable	N	Mean	SD	Min	Median	Max
MLA (kips)	DL (kips)	10	2.84	0.47	1.50	3.00	3.00
HSDT-EOD (kips)	DL (kips)	9	2.56	0.87	0.81	2.66	3.60
HSDT-STCK (kips)	DL (kips)	4	2.82	0.52	2.27	2.84	3.32
HSDT-BOR (kips)	DL (kips)	6	2.92	0.98	1.91	2.49	4.34
HSDT-EOD (kips)	MLA (kips)	9	0.89	0.23	0.54	0.89	1.20
HSDT-STCK (kips)	MLA (kips)	4	0.94	0.17	0.76	0.95	1.11
HSDT-BOR (kips)	MLA (kips)	6	0.97	0.33	0.64	0.83	1.45
HSDT-STCK (kips)	HSDT-EOD (kips)	3	1.15	0.12	1.02	1.20	1.25
HSDT-BOR (kips)	HSDT-STCK (kips)	3	0.94	0.12	0.82	0.95	1.05
HSDT-BOR (kips)	HSDT-EOD (kips)	5	1.15	0.20	0.95	1.18	1.45

N=10, all concrete piles, each driven in sand

Table 4.25 Linear Regression statistics for concrete piles

Dataset	HSDT x-variable	N	Intercept Estimate	Intercept Lower CI	Intercept Upper CI	Intercept Sig*	Slope Estimate	Slope Lower CI	Slope Upper CI	Slope Sig*	R-sq (unadj)
Concrete piles	EOD	9	79.77	22.77	136.76	*	0.83	0.71	0.94	*	0.974
	STCK	4	29.98	-1264.63	1324.59		0.99	-3.25	5.23		0.336
	BOR	6	167.71	63.48	271.94	*	0.59	0.44	0.74	*	0.967

The y-variable predicted is MLA. MLA is a censored variable usually tested to 2 to 3 times design load which is short of pile failure.

* Significance at alpha 0.05, the null hypotheses for slope and intercept are that they are equal to zero. Confidence intervals are 95%.

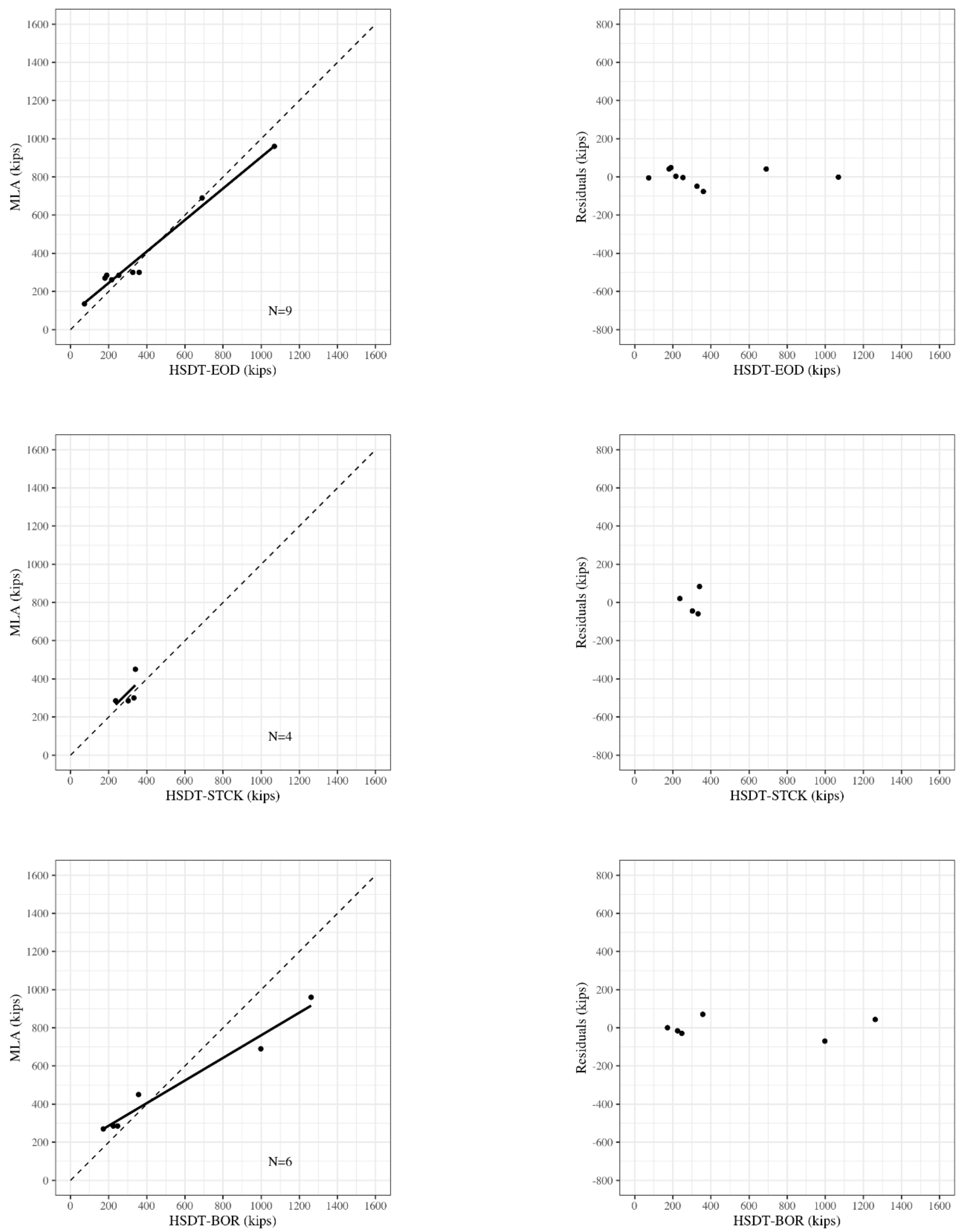


Figure 4.12 Regression models and residual scatter plots for concrete piles with y=x reference line.

4.5.9 Summary of Statistical and Graphical Analysis

1. In general, the methods of statistical and graphical analysis were selected for simplicity and for transparency. Transparency meaning that any unusual characteristics of the data points would be readily observed.

2. No variable transformations were used to reduce variance. This is part of the effort to allow observation of the native characteristics of the dataset.

3. No effort was made to reduce or eliminate outliers. This is another part of the effort to allow observation of the native characteristics of the dataset.

4. For the few piles with Davisson capacity reported, it was within a few percentage points of being the same as MLA. Since Davisson was rarely reported, MLA was used for analysis for all piles for which it was reported (N=81).

5. The mean values of the variables HSDT-EOD, HSDT-STCK, and HSDT-BOR tended to increase from EOD to STCK to BOR. This trend may be affected by the piles present but does appear fairly consistent across subgroups.

6. There appears to be a linear relationship between MLA and each of HSDT-EOD, HSDT-STCK, and HSDT-BOR across most subgroups. The linear relationship is not exactly the same across all subgroups because of the nature of retrospective studies (possible selection bias of piles) of this type and the errors that can arise from patterns of missing data.

7. If MLA is predicted from the HSDT variables, there appears to be quite a wide confidence interval around the predicted value. This calculation is easiest from the 12x53 H-piles subgroup, the mean (SD) of the MLA was 239.8 (82.5). Thus, a confidence interval that is

approximately 4 SD's wide would be 330 kips (±165 kips). The regression slope was not significantly different from zero in this subgroup at EOD so the mean±2SD is a good approximation to the prediction interval of a single observation.

8. MLA is presumed to be a significant underestimate of the failure capacity of each pile. It is difficult to estimate how much of an underestimate it is based on this dataset.

9. Given Points #6, 7, and 8, we cannot conclude that HSDT variables are surrogates for MLA or for ultimate capacity. Further research would be needed to fill this gap.

4.6 Static Load Test Results

Dataset 3 was used for this portion of the study. Dataset 3 consisted of all the static load tests from Dataset 1, and Dataset 2. Overall, there were 101 static load test results. Because ALDOT uses Davisson failure criterion to predict the ultimate capacity of piles, it was documented how many static load tests reached failure criterion. The number of static load tests that did and did not reach Davisson failure criterion are displayed below in Figure 4.13.

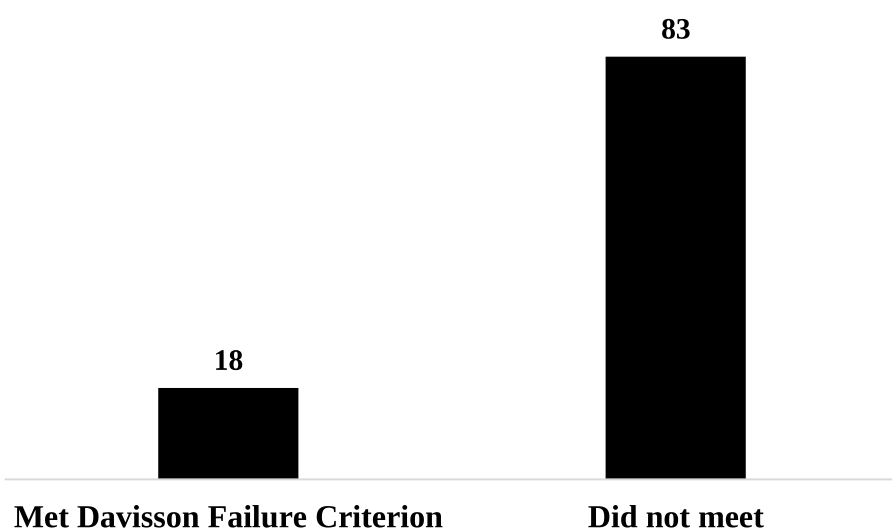


Figure 4.13 Static Load Test Davisson Failure Criterion Results

As seen above, the majority of the static load tests conducted in the studied dataset did not reach Davisson failure criterion. A further summary analysis was then conducted on the load-settlement plots from the static load tests to determine how many load tests were close to failure. The results for this are found below in Figure 4.14.

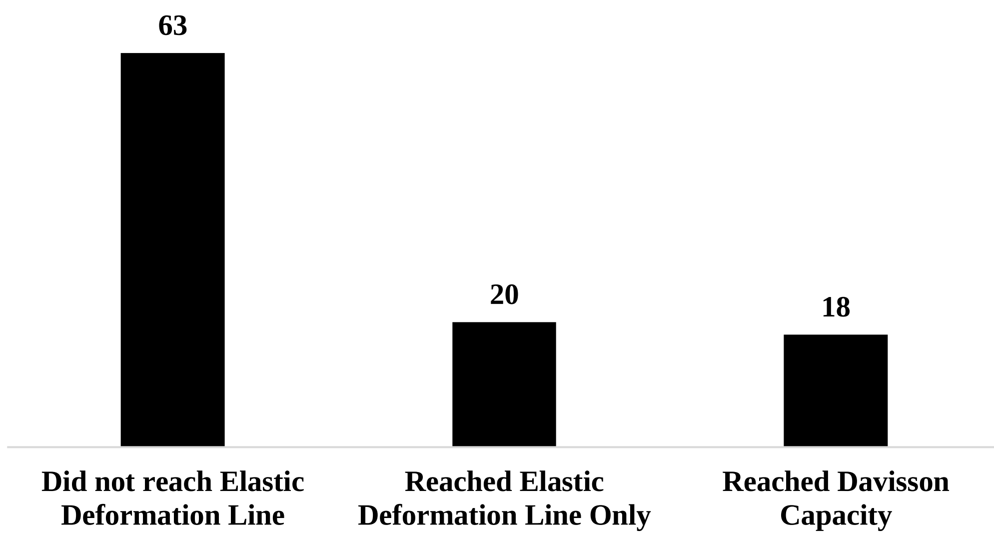


Figure 4.14 Static Load Test Davisson Failure Criterion Details

The results from Figure 4.14 show that not only did most static load tests not reach failure criterion, but the majority of static load tests were also not close to failure criterion. This is most likely due to extremely conservative pile design. A further summary analysis was then conducted on the static load tests in the dataset to display maximum settlement in the piles. The results for this analysis are provided below in Figure 4.15.

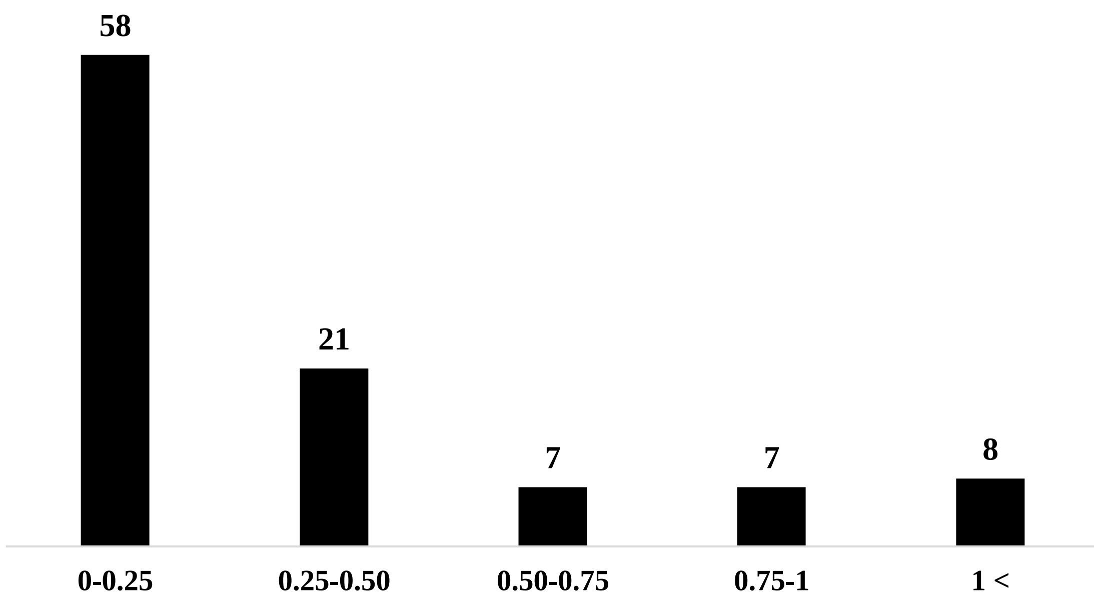


Figure 4.15 Maximum Settlement (in.) Reached during Static Load Test

Results from the plot above show that most of the static load tests in the dataset settled less than a quarter of an inch. This is a common trait of over designed piles.

CHAPTER 5: DOT PEER SURVEY

5.1 Introduction

A general survey with questions regarding the pile driving state of practice was created and delivered to the state geotechnical engineer of each DOT in the southeastern United States. The survey was sent to: Georgia, Florida, South Carolina, North Carolina, Kentucky, Tennessee, Mississippi, Louisiana, Arkansas, and Virginia. Responses from the southeastern states are summarized in this chapter.

5.2 Questions Included in Survey

The questions included in the survey are detailed below.

1. Are axial compression static load tests used as part of the acceptance criteria for driven piles on all bridge projects in your state?
2. Are static load tests loaded to a predetermined maximum test load?
3. What is the typical maximum test load achieved during the static load test?
4. Are static load test results used to predict pile capacity? Such as Davisson Capacity? If you do not use Davisson criterion, what do you use?
5. What type of instrumentation is used during the static load test to measure the settlement of test piles? Select all that apply.
6. Are static load tests routinely loaded to reach failure criterion (or other specified failure criterion)?
7. What project types [county, state, or federally funded] typically use static load tests as part of pile acceptance criteria?
8. Are static load tests conducted by independent contractors or state construction personnel?

9. Are static load tests performed according to ASTMD1143?
10. Are static load test results routinely archived for future use on state research projects?
11. Are high strain dynamic tests, known commercially as PDA or Pile Driving Analyzer, used on all state projects to estimate ultimate capacity?
12. Is PDA used to predict ultimate capacity at the end of initial drive (EOD or EOID)?
13. Is PDA used to predict ultimate capacity after some period of time (either a set-check or a restrike) to allow time for soil set-up/ relaxation?
14. Are consistent set-up times used throughout projects between EOD and BOR? Or does it vary greatly because of the variation of timeline of construction in each project and/or soil type??
15. Is there a specific multiple of design load that the PDA operator is looking to achieve when testing piles?
16. Are PDA restrikes and/or set checks performed on every pile that also undergoes a static load test?
17. Who performs the PDA tests?
18. Does your state archive PDA data for use on future state research projects?
19. Does your state perform either an iCAP or CAPWAP analysis to predict ultimate capacity?
20. What design/testing methods does your state follow for driven piles?
21. Has your state performed a research study attempting to correlate the archived static load and PDA tests from your state projects?

5.3 Survey Results and Conclusions

The Department of Civil Engineering at Auburn University reached out to ten southeastern DOTs to get a feel for how neighboring states were carrying out the practice of pile driving. Figure 5.1 displays a map that summarizes the results of the survey.

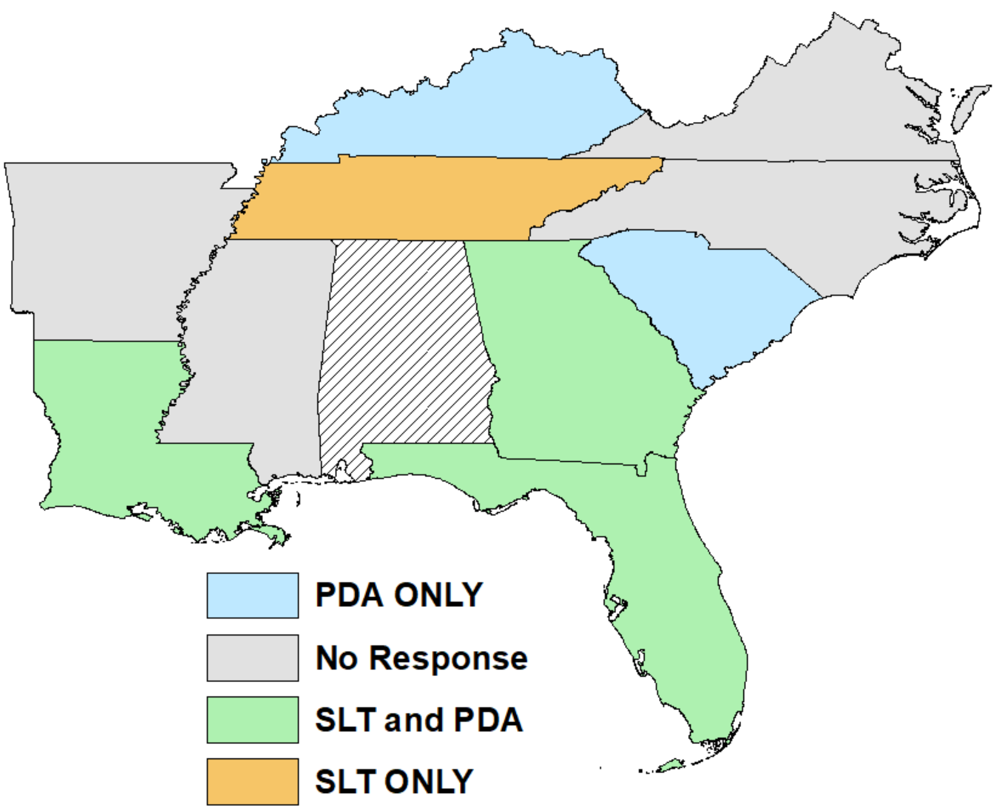


Figure 5.1 Southeastern DOT Peer Survey Results Map

The six states surveyed explicitly use LRFD when designing and testing driven piles throughout state projects. Of the six states surveyed, only Tennessee does not use HSDT across all state projects and solely relies on static load tests to verify pile capacity in the field. Two states, South Carolina and Kentucky, do not perform any static load tests and solely rely on CAPWAP analyses to predict pile capacity. Georgia, Florida, and Louisiana use a combination of both static load tests and HSDT, similar to Alabama. However, of the five states surveyed that use HSDT on state projects, CAPWAP is used exclusively for analysis. Only Georgia uses a combination of iCAP and CAPWAP for capacity analysis. All HSDT work on state projects in Georgia, Kentucky, and South Carolina is performed by a private consultant or contractor. In Florida, HSDT work is performed by either a state engineer or a private consultant, depending on the job. In Louisiana, HSDT work is performed by a state engineer.

Southeastern states appear to have made the full transition to LRFD for driven piles. When piles are verified in the field, a CAPWAP analysis appears to be the industry standard, and several states hire a private consultant or contractor to perform them. Static load tests are commonly used to verify pile capacity in conjunction with HSDT on southeastern DOT projects.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Project Summary

A dataset was constructed from ALDOT project archives consisting of 81 piles for which static load testing and HSDT were both performed. The database consisted of H-piles, prestressed square concrete piles, and steel pipe piles. After the pile information was acquired, the database was organized by project, county, geographic region, and soil type. Within each project, piles were organized by pile type, pile size, pile length, static load test data, and HSDT data. Some piles were analyzed using CAPWAP while others were analyzed using iCAP. Because of the analysis difference, a statistical test was performed to determine if there was a difference between CAPWAP and iCAP. All static load test results in the database were plotted in order to determine Davisson failure criterion for each test. Static load tests did not commonly achieve Davisson capacity and therefore the MLA was used from each static load test to compare to HSDT. HSDT was commonly performed at EOD, STCK, and BOR but rarely all three values were available for any single pile. Data analysis was intentionally restricted to summary tables of original and ratio variables and simple linear regression of MLA (y-axis) vs HSDT-EOD, HSDT-STCK, and HSDT-BOR (x-axis). Rather than attempt to identify outlier values based on statistical criteria, multiple subsets of the dataset were examined based on the similarity of piles and soil conditions. Subsets of the initial dataset included: All H-Piles, Common H-Piles (12x53, 14x73, 14x89), 12x53 H-Piles, H-Piles in sand, H-piles in clay, steel pipe piles, and concrete piles. An additional analysis was also performed on all the static load tests in the database. Each static load test was analyzed based on whether Davisson failure criterion was met and the maximum settlement the pile reached.

6.2 Project Conclusions

- The current state of driven piles practice for ALDOT projects consists of conducting static load tests to three times the design load and conducting HSDT field tests until an iCAP analysis predicts a pile capacity of at least two times the design load.
- Only 18 out of the 101 gathered static load tests reached Davisson failure criterion, and 63 out of the 101 static load tests did not reach the elastic deformation line. This is likely evidence that driven piles are commonly overdesigned across ALDOT projects.
- Because Davisson was rarely achieved, MLA was used to compare to the HSDT results.
- Since MLA is an underestimate of the pile capacity, the linear relationship between HSDT variables and pile capacity would be different. This is because MLA is a censored variable.
- Various subsets of the dataset were examined, and scatterplots of MLA vs HSDT variables were generated. Simple linear regression analysis gave a significant positive slope coefficient in almost every case. Thus, the HSDT variables are predictive of MLA but with varying levels of precision as the scatter plots show. The residual plots give a visual impression that the cloud of points around each line is haphazard (maybe not purely random) and not always tight or precise.
- A subset of the ALDOT piles selected for this study were previously considered by Hill (2007). In that study, CAPWAP analyses were used to measure HSDT variables. In the current study, iCAP was the software analysis method used to measure HSDT variables. A linear model was constructed to test for a difference in these methods of measuring HSDT variables. Within the limits of the dataset, no statistical difference could be

detected. This finding is consistent with reports in the literature (Steward et al., 2018) showing near equivalence between iCAP and CAPWAP.

- The summary statistics of the ratio values show that the HSDT-STCK and HSDT-BOR are most often larger than HSDT-EOD by as much as 30% (capacity increase due to soil set-up). However, there is wide variation.
- From these comparisons, there was a significant predictor effect of HSDT on MLA at the endpoints of EOD, STCK, and BOR as assessed by simple linear regression.
- However, in the current state of practice, it does not appear that HSDT is a precise predictor of MLA. The variability of the slopes and intercepts across the subgroups is indicative of the imprecise relationship between HSDT variables and MLA. An excellent example is found in the subgroup of 12x53 H-piles (Tables 4.12-4.14) in the relationship of HSDT-EOD and MLA. The slope is not significantly different from zero and thus the MLA mean plus and minus two times the standard deviation is a good measure of the prediction interval across the range of HSDT-EOD. In this case, the 95% confidence interval for a single observation is [75, 405] kips. This is a significant amount of uncertainty and is approximately four times the mean DL. Further research will be needed to better understand the complex relationship between HSDT and static load test results.

6.3 Recommendations for future research and practice

An “ideal” experimental design would have variables such as HSDT-EOD, HSDT-STCK, HSDT-BOR, and Davisson capacity, measured on every pile. Multiple HSDT-STCK and HSDT-BOR times might also be considered desirable in order to determine optimum set-up times for certain situations. Other information to include in this ideal experiment would be things such as: pile types, pile sizes, pile lengths, and soil types. Additionally, the ideal experiment would have multiple piles in each category. Given the time and expense of pile driving, an experiment of this type does not appear feasible with current technology and methods available.

1. Create an Auburn University – ALDOT database repository for the study of pile driving eventually developing a catalog of expected capacities of piles across the state and across various soil conditions and pile types.
2. Based on the likely evidence from the gathered static load tests that piles are commonly over designed, investigate the origin of the software and methods being used to design piles. Reducing the size and length of piles can save a significant amount of project budget and time.

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APPENDIX A: Russell Piles with HSDT and SLT Data Condensed

County	Type	Size	DL kips	MLA kips	Dav. Cap kips	iCAP EOD kips	iCAP STCK kips	STCK Set Up Time hrs	iCAP BOR kips	BOR Set Up Time days
Clarke	H	12x53	44	132	N/A	N/A	159.6	0.5	N/A	N/A
Clarke	H	14x73	140	420	N/A	N/A	386.3	0.5	544.8	5
Coffee	H	14x89	134	402	402	334.7	456.4	0.75	N/A	N/A
Elmore	H	14x73	117	350	N/A	N/A	395.1	0.5	463.7	6
Elmore	H	14x73	190	570	N/A	473.6	N/A	N/A	N/A	N/A
Escambia	H	12x53	60	180	N/A	N/A	N/A	N/A	225.2	1
escambia	H	14x73	140	420	N/A	419	N/A	N/A	649.9	20
Escambia	H	14x73	140	420	N/A	N/A	423.3	N/A	503.6	13
Lee	H	12x53	90	270	270	165.8	N/A	N/A	178	1
Lowndes	H	12x53	78	234	N/A	232.1	209.5	0.5	204.1	5
Mobile	H	12x53	120	360	N/A	189.1	216.5	0.25	260.7	1
Mobile	H	14x89	100	300	N/A	219.8	283.8	20	268.9	4
Mobile	H	12x63	102	306	306	245.2	248.4	0.75	N/A	N/A
Mobile	H	12x63	156	468	N/A	196.4	247.5	24	271.8	2
Mobile	H	12x53	56	168	N/A	131.5	158.8	0.33	151.9	8
Mobile	H	10x42	40	120	N/A	75.6	106.5	3	135	2
Mobile	H	14x73	120	360	N/A	207.2	257.1	0.5	394.3	7
Montgomery	H	12x53	60	180	N/A	348.6	N/A	N/A	N/A	N/A
Montgomery	H	14x89	230	690	N/A	880.9	946.9	0.5	N/A	N/A
Montgomery	H	14x89	230	690	N/A	449.6	530.1	0.33	N/A	N/A
Montgomery	H	14x89	230	690	N/A	495.2	614.9	0.5	N/A	N/A
Montgomery	H	14x73	104	312	N/A	181.4	220	0.67	N/A	N/A
Montgomery	H	14x102	260	780	730	339.6	462.5	0.33	N/A	N/A
Montgomery	H	14x89	230	690	N/A	N/A	541.7	0.33	N/A	N/A
Pickens	H	12x53	140	420	N/A	213.6	N/A	N/A	268.5	7
Tuscaloosa	H	12x53	102	286	N/A	228.3	234.1	0.5	272.2	28

Tuscaloosa	H	12x53	140	378	N/A	168.7	251.7	2	N/A	N/A
Washington	H	14x73	108	324	305	157.8	N/A	N/A	N/A	N/A
Wilcox	H	12x53	72	216	N/A	199.9	N/A	N/A	323.7	N/A
Mobile	H	18x135	262	786	N/A	328.4	335.8	17	384.6	7
Mobile	PSC	16	95	285	N/A	253.1	302.5	0.5	246.9	7
Mobile	PSC	16	95	285	N/A	189.8	236.6	0.33	224.2	14
Mobile	H	12x53	76	228	N/A	169.1	N/A	N/A	175.8	3
Mobile	H	18x135	262	786	N/A	195.9	418.5	12	635.3	7
Mobile	Pipe	30	389	1167	1128	428	N/A	N/A	N/A	N/A
Mobile	H	18x135	332	996	N/A	439.3	525.4	0.75	803.5	37
Clarke	H	12x53	84	252	N/A	81.3	220.5	3.5	N/A	N/A
Baldwin	PSC	16	100	300	N/A	326.4	332.2	0.33	N/A	N/A
Baldwin	PSC	30	320	960	N/A	1068.9	N/A	N/A	1262	8
Baldwin	PSC	16	90	270	N/A	180.2	N/A	N/A	171.8	14
Baldwin	PSC	16	90	135	135	73.1	N/A	N/A	N/A	N/A
Baldwin	PSC	20	150	450	N/A	N/A	339.9	N/A	356.8	7
Baldwin	PSC	16	90	261	261	215.5	361.7	N/A	N/A	N/A
Mobile	Pipe	30	389	1167	N/A	473.6	N/A	N/A	624.2	6
Mobile	Pipe	30	407	1058.2	1050	425.4	N/A	N/A	N/A	N/A
Mobile	H	12x53	78	234	234	161.8	167.4	1	166.8	7
Mobile	Pipe	30	336.4	1010	N/A	N/A	N/A	N/A	790.1	N/A
Mobile	Pipe	30	474	1422	1420	514	N/A	N/A	N/A	N/A
Henry	H	12x53	176	211	211	148.5	214	0.75	N/A	N/A
Mobile	H	14x73	112	336	N/A	163	203.2	1	258.6	5
Mobile	H	12x53	44	132	N/A	146.4	173.5	0.5	213.8	7

APPENDIX B: Hill Piles with HSDT and SLT Data Condensed

County	Type	Size	DL kips	MLA kips	Dav. Cap kips	HSDT EOD kips	HSDT STCK kips	STCK Set Up Time hrs	HSDT BOR kips	BOR Set Up Time days
Mobile	PSC	24	230	690	N/A	690	N/A	N/A	998	60
Washington	H	14x73	126	378	N/A	290	N/A	N/A	410	5
Covington	H	10x42	84	252	N/A	232	N/A	N/A	N/A	N/A
Covington	H	14x73	114	342	N/A	282	N/A	N/A	N/A	N/A
Washington	H	10x42	80	240	N/A	240	N/A	N/A	N/A	N/A
Washington	H	14x73	126	380	370	370	N/A	N/A	N/A	N/A
Coffee	H	12x53	60	180	N/A	154	N/A	N/A	N/A	N/A
Coffee	H	12x53	90	270	N/A	176	N/A	N/A	276	2
Coffee	H	12x53	90	270	N/A	228	N/A	N/A	N/A	N/A
Wilcox	H	12x53	120	360	N/A	300	318	0.5	N/A	N/A
Calhoun	H	12x84	60	180	N/A	300	N/A	N/A	N/A	N/A
Washington	H	10x42	80	240	N/A	195	N/A	N/A	220	N/A
Russell	H	12x53	58	180	N/A	180	N/A	N/A	N/A	N/A
Dallas	H	12x53	60	180	N/A	358	N/A	N/A	N/A	N/A
Butler	H	12x53	60	180	N/A	220	N/A	N/A	264	4
Tuscaloosa	H	12x53	60	180	N/A	200	N/A	N/A	N/A	N/A
Tuscaloosa	H	14x89	90	270	N/A	280	N/A	N/A	366	10
Baldwin	PSC	14	100	300	N/A	360	N/A	N/A	N/A	N/A
Henry	H	12x53	60	180	N/A	209	N/A	N/A	300	7
Montgomery	H	10x42	60	120	N/A	180	N/A	N/A	N/A	N/A
Covington	H	12x53	60	180	N/A	119	216	1	N/A	N/A
Covington	H	12x53	140	420	N/A	419	N/A	N/A	N/A	N/A
Lamar	H	14x73	108	324	N/A	268	346	0.5	N/A	N/A
Escambia	H	12x53	66	198	N/A	240	N/A	N/A	N/A	N/A
Escambia	H	12x53	64	192	N/A	240	N/A	N/A	N/A	N/A
Escambia	H	14x89	114	342	N/A	300	N/A	N/A	N/A	N/A
Escambia	H	14x89	114	342	N/A	256	N/A	N/A	N/A	N/A
Escambia	H	12x53	66	198	N/A	176	N/A	N/A	N/A	N/A
Montgomery	H	12x53	120	366	N/A	210	N/A	N/A	N/A	N/A

Montgomery	H	10x42	114	174	N/A	222	N/A	N/A	N/A	N/A

APPENDIX C: Russell Piles with SLT Data only Condensed

County	Type	Size	DL kips	MLA kips	Dav. Cap kips
Barbour	H	12x53	84	252	N/A
Barbour	H	14x102	118	354	N/A
Barbour	H	12x53	72	216	N/A
Barbour	H	12x53	72	216	N/A
Crenshaw	H	12x53	84	210	N/A
Franklin	H	12x53	80	240	218
Geneva	H	12x53	48	144	N/A
Geneva	H	12x53	84	252	N/A
Mobile	PSC	14	68	204	200
Shelby	H	12x53	40	120	N/A
St. Clair	H	12x53	100	300	N/A
St. Clair	H	12x53	100	300	N/A
Pickens	H	12x53	72	216	N/A
Pickens	H	12x53	98	294	N/A
Wilcox	H	12x53	90	270	N/A
Wilcox	H	14x89	100	300	N/A
Barbour	H	12x53	84	252	220
Barbour	H	12x53	72	216	N/A
Barbour	H	14x89	98	300	N/A
Mobile	H	12x53	140	420	340