Florida Institute of Technology
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Design Phase Identification of High Pile Rebound Soils Final Report

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Contract BDK81 Work Order 977-01

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Disclaimer

Conversions to SI Units

Symbol	When you know	Multiply by	To find	Symbol
Length				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
Area				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
Volume				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1 000 L shall be shown in m^3				
Mass				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or ``metric ton")	Mg (or ``t")
Temperature (exact degrees)				
F	Fahrenheit	5 (F-32) / 9	Celsius	C
Force and Pressure or Stress				
lbf	pound force	4.45	Newton's	N
lbf/in^2	pound force per square inch	6.89	kilopascals	kPa

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

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16. Abstract: An engineering problem has occurred when installing displacement piles in certain soils. During driving, piles are rebounding excessively during each hammer blow, causing delays and as a result may not achieve the required design capacities. Piles driven at numerous locations have recorded rebound values well over 1 inch per blow. The research objective was to determine geotechnical testing protocol to help engineers anticipate high rebound. There are high pile rebound sites throughout North America. This problem typically occurred when displacement piles driven with single acting hammers, encountered saturated silts and clays, in medium dense or stiff soils. Computer models indicated that soil quake and pile rebound were high. Within Florida, a geologic layer known as the Hawthorn Group was encountered when high pile rebound occurred.

Testing was conducted at three sites; two in the Orlando area and the third in the Florida Panhandle. Field tests included Standard Penetration Borings to produce N-values, Pocket Penetrometer tests to produce unconfined compressive strengths, Cone Penetrometer soundings to produce point bearing, sleeve friction and pore water pressures, Pencel Pressuremeter tests to produce *in situ* stress-strain data and Dilatometer testing to produce elastic moduli. Lab testing included natural moisture contents, grain size and hydrometer analyses, Atterberg limits, permeability and consolidated undrained triaxial testing. Orlando area test results combined with Pile Driving Analyzer (PDA) data, revealed one high pile rebound zone through which the piles were able to be driven over a lower zone which prevented pile penetration indicating a zone of influence effect on thesedisplacement piles. SPT N-values plotted versus elevation data indicated a large change in N-values when high pile rebound occurred. Within the Central Florida Sites, N-values increased from six to seven pile diameters into the rebound zone, while excessive rebound changed into pile bouncing when penetration was prevented from 7.5 to 9 pile diameters into the rebound zone. These changes also corresponded to the upper elevations reported for the Hawthorn Group. SPT N values increased to over 50 blows per foot at about the same elevations that the displacement piles were no longer able to achieve penetration, termed bouncing. The silt content increased to over 18 percent at the elevations the prevented pile penetration. The pocket penetrometer unconfined compression results increased to 1.9 tsf (182 kPa) at about this same elevation. CPT tip resistance values increased to over 65 tsf (6,234 kPa), while sleeve friction values increased to over 1.1 tsf (106 kPa). The CPT data produced negative pore water pressures in the soils overlying the rebound zone, which increased to positive values in excess of 100 psi (700 kPa) for all three sites, again at about the bouncing elevations. These variations in pore water pressures in combination with the increased stiffness and high silt contents in saturated soils, could be the geotechnical conditions that would produce high pile rebound. Large changes between the overlying no rebound and rebound zones data, determined from PDA data were, also identified based on all of the testing data. The results were reported as ratios between the rebound zone divided by the overlying no rebound zone. The grain size with hydrometer data showed that the silt content increased by a factor of 1.9, the pocket penetrometer unconfined compression data increased by a factor of about 2.6 in the rebound zone, the CPT point and friction data increased by a factor of about 3.8 and 4.4 respectively and the raw N-values increased by a factor of about3.7.

A detailed soil profile must be constructed, including geologic and construction history, plus engineering parameters plotted versus elevation. Changes in the soil strength and stiffness must be determined by evaluating variations of the parameters throughout the profiles. Ratios for each parameter of the higher strength/stiffness soil to the overlying lower strength/stiffness soil must be established and the ratios determined should be compared to those presented. A flow chart was developed to help guide engineers through the decision making process for anticipating high pile rebound. It includes the required input data, the output needed before the high pile rebound evaluation can be conducted. A logistic regression model was used to correctly predict high pile rebound over 70 percent of the time for two of the three sites The sites with the most promising results had clays present in the rebound zone, rather than silty fine sand. The site where poor statistical results occurred also had a larger variation in permeability than the other sites. Benefits from implementing this research include a geotechnical testing and evaluation flow chart that when followed may enable engineers to anticipate and avoid high pile rebound problems. Data from future pile driving projects can be used to validate this research and improve the database.

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Executive Summary

The Florida Department of Transportation has experienced problems when installing large diameter displacement piles in certain soils. During driving, piles rebound excessively during each hammer blow, causing delays and as a result they may not achieve the required design capacities as specified by current FDOT specification 455-5.10.2. Piles driven at numerous locations have recorded rebound values well over 1 to 2 inches per blow. The objective of this research was to determine geotechnical testing protocol that would help engineers anticipate high rebound.

The literature review revealed high pile rebound sites throughout North America. This problem typically occurred when displacement piles were driven into medium dense or stiff saturated silts and clays, using single acting hammers. Hammer blows between 2 and 50 blows per inch were recorded. Computer models indicated that both the soil quake and pile rebound were high. In Florida, a geologic layer known as the Hawthorn Group was encountered when high pile rebound occurred.

An extensive laboratory and field testing program was conducted at three existing FDOT project sites. Two were located in the Orlando area and the third in the Florida Panhandle. The field testing included Standard Penetration Borings with N-values; Pocket Penetrometer unconfined compressive tests; Cone Penetrometer soundings that produced point bearing; sleeve friction and pore water pressures; PENCEL Pressuremeter tests that produced in situ stress-strain data; and Dilatometer soundings to produce lift-off pressures and elastic moduli. The lab testing on disturbed samples produced natural moisture contents, grain size and hydrometer data, Atterberg limits; and tests on thin walled tube samples produced permeability and consolidated undrained triaxial testing parameters, including elastic moduli, friction, and cohesion.

To clarify the extent and amount of rebound, the test results were evaluated with Pile Driving Analyzer data obtained from the original installations at the three sites. The PDA data from the Central Florida sites revealed one high pile rebound zone through which the piles were able to be driven over a lower zone that prevented pile penetration. This lower zone, which prevented pile penetration, indicates that there was a zone of influence effect on these displacement piles. SPT N-values plotted versus elevation data indicated a large change in N-values when high pile rebound occurred. At the Central Florida Sites, N-values increased from 6 to 7 pile diameters into the rebound zone, while excessive rebound changed into pile bouncing when penetration was prevented from 7.5 to 9 pile diameters into the rebound zone. These changes also corresponded to the upper elevations reported for the Hawthorn Group.

SPT N values increased to over 50 blows per foot at about the same elevations that the PDA data indicated that the displacement piles were no longer able to achieve penetration (i.e., the piles were bouncing). The silt content increased to over 18 percent at the bouncing elevations. The pocket penetrometer unconfined compression results increased to 1.9 tsf (182 kPa) at about this same elevation. CPT point bearings values at the bouncing elevations increased to over 65 tsf (6,234 kPa), while sleeve friction values increased to over 1.1 tsf (106 kPa). The CPT data produced negative pore water pressures in the soils overlying the rebound zone, which increased to positive values in excess of 100 psi (700 kPa) for all three sites, again at about the bouncing elevations. These variations in pore water pressures in combination with the increased stiffness and high silt contents in saturated soils, could be the geotechnical conditions that would produce high pile rebound.

Large changes in soil properties occurred between the overlying no rebound zone and lower rebound zone, determined from PDA data. These changes were reported as property ratios calculated as the lower rebound zone divided by the overlying no rebound zone. The grain size ratios showed that the silt content increased by a factor of 1.9 in the rebound zone. The pocket penetrometer unconfined compression ratios increased by a factor of about 2.6 in the rebound zone. The CPT point and friction data increased by a factor of about 3.8 and 4.4 respectively in the rebound zone, and the raw N-values increased by a factor of about 3.7 in the rebound zone.

A statistical evaluation of thirteen geotechnical parameters indicated that a nonlinear logistic regression model with silt content and the pocket penetrometer unconfined compression strength inputs could be used to predict rebound. The model correctly predicted high rebound over 70 percent of the time for two of the three sites, corresponding to the sites where clays were present in the rebound zone, rather than the site with predominantly silty fine sand in the rebound zone. The site where the prediction was poor also had a large variation in permeability throughout the profile.

In order for engineers to evaluate and develop these ratios the following procedure has been proposed. First a detailed soil profile should be constructed, which includes geologic and construction history data, plus the various geotechnical engineering parameters plotted versus elevation. Then changes in the soil strength and stiffness should

be determined by evaluating the variations of the parameters throughout the profiles and the silt content and pocket penetrometer unconfined compressive strength values should be input into the recommended logistic regression equation. Once these variations and probabilities are established, then ratios for each parameter of the underlying higher strength/stiffness soil to the overlying lower strength/stiffness soil must be established. Outliers from these layers should be eliminated and the ratios determined should be compared to those presented. A flow chart outlining this process was developed to help guide engineers through the decision making process for anticipating high pile rebound. It was divided into sections for establishing the required input data (i.e., soil profile, testing results such as N-values, silt content etc.), the output needed, which would be the soil profiles, before the ratios can be used to conduct the high pile rebound evaluation.

High pile rebound was determined to be a concern when the following combination of effects occurred. It was a concern if the silt ratio increased by a factor of 2, and the N-value ratio increased by a factor of 3, and the Hawthorn Group was encountered. It was also a concern when the silt ratio increased by 2, and the point bearing and or sleeve friction ratios increased by about 4, and the Hawthorn Group was encountered. If pocket penetrometer ratios increase by about 2.5, and the silt content increases by 2, and the Hawthorn Group was encountered high pile rebound was also a concern.

Benefits from implementing this research include the development of a new geotechnical testing and evaluation approach, which includes a flow chart that will enable engineers to anticipate and possibly avoid high pile rebound during construction. Avoiding this problem will result in significant monetary savings for FDOT. Data from future pile driving projects can be used to validate this research and improve the data base and the research recommendations.

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1. Introduction

1.1 Background

Contractors and engineers have experienced serious pile installation problems while driving large diameter displacement piles with both diesel and air hammers in numerous Florida Department of Transportation (FDOT) construction projects. These problems have occurred throughout Central Florida, the Jacksonville area and the coastal region of the Florida Panhandle as depicted in Figure 1.1.

During portions of the driving, a large initial penetration per hammer blow is followed by a large elastic rebound (termed High Pile Rebound) resulting in a small or negligible permanent set per blow and very high blow counts. The combination of "high pile rebound" and high blow counts may not produce the required driving resistance and may prevent the pile driving process from being completed. Small amounts of rebound are common during driving; however, if high rebound is not identified or recorded, a pile without achieving the design resistance may be unknowingly accepted, placing the foundation performance at risk. These problems generally occurred in soils that did not display any unusual geotechnical properties during routine soil investigations.

Typically test borings with Standard Penetration Testing (SPT) and/or soundings with Cone Penetrometer testing (CPT) are used to develop soil profiles and engineering properties. No available process currently exists to help engineers anticipate high pile rebound.

Consultants who encounter high pile rebound problems have been asked to perform extensive re-evaluations on 1) site conditions, 2) foundation types, 3) field testing procedures and 4) pile driving equipment. In District 5, high pile rebound problems at the I-4 / SR 408 interchange project required a redesign of the deep foundation system, whereby the large displacement prestressed concrete piles (PCP's) were replaced with low-displacement steel H-piles.

Research is needed that would improve the engineer's knowledge of high pile rebound during driving with respect to potential soil types, subsurface stratigraphy, pile driving systems and pile types. Due to the complexity of this problem, the initial research focused on the soil and subsurface stratigraphy. If conditions leading to high rebound could be identified during initial subsurface soil investigation, engineers and contractors could implement designs and systems to avoid or reduce high pile rebound to workable levels.

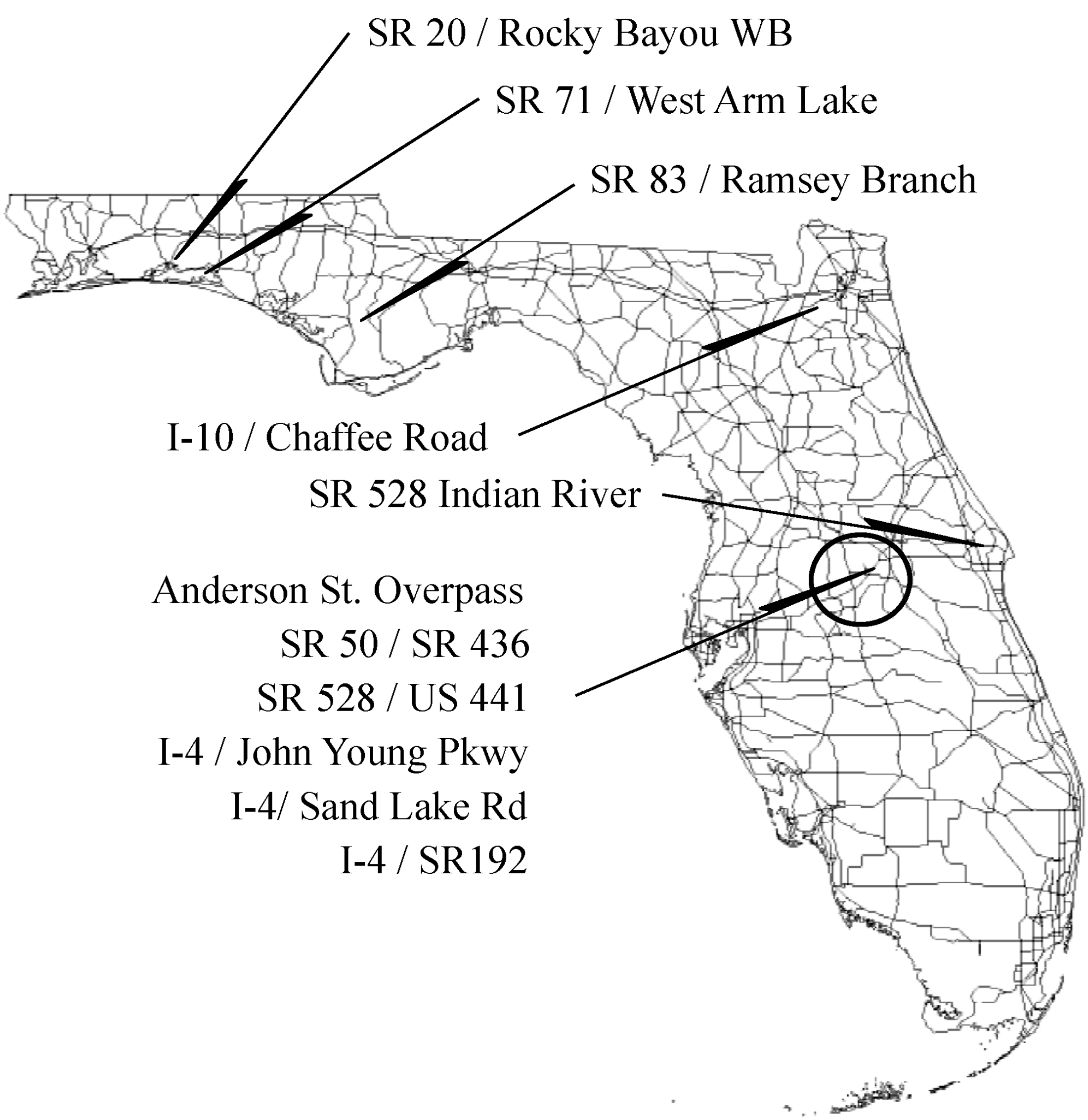


Figure 1.1 Existing High Pile Rebound Sites in Florida

1.2 Objective

The project objective is to develop design-phase geotechnical testing protocol that would help engineers identify geotechnical conditions when combined with certain pile types and pile-driving systems could produce high pile rebound.

1.3 Approach

In order to meet the objective an extensive literature search was conducted, followed by a comprehensive lab and field testing program at three sites within the high pile rebound areas of Florida. The data were evaluated and analyzed to develop a set of recommendations for FDOT. Details about the specific tasks are included below.

1.3.1 Task 1 - Review Literature

The literature review provided multiple case histories of high pile rebound at locations throughout Florida and North America. The publications and communications included a thorough review of critical geologic soil formations related to high pile rebound. For each case history, key engineering characteristics were analyzed and assimilated into a summary to determine any commonality between the cases. These characteristics include

- Generalized soil profiles based on elevations, including color, soil descriptions
- Grain size data
- Atterberg Limits
- *In situ* density and moisture content
- Engineering properties such as moduli, cohesion and friction
- Pile size and type
- Changes in pile driving techniques
- The pile driving equipment and hammer efficiency
- Pile Driving Analyzer® (PDA) and CAse Pile Wave Analysis Program (CAPWAP) data such as hammer blows per foot, stroke height, and maximum displacement per blow

Based on discussions with FDOT Districts 3 and 5, turnpike personnel, project consultants, GRL Engineers, Inc. (GRL) and Ardaman & Associates, Inc. (Ardaman), eight existing sites were identified in both District 5 near Orlando and District 3 in the Florida Panhandle. These sites were evaluated and characteristics from the each were summarized. The sites are as follows:

1. I-4 and SR 408 Interchange at the Anderson Street Overpass, Orlando, Florida, FDOT District 5
2. I-4 at the John Young Parkway, Orlando, Florida, FDOT District 5
3. I4 at US 192, Orlando, Florida, FDOT District 5
4. I4 at SR 482 Sand Lake Road, Orlando, Florida, FDOT District 5
5. SR 528 Bridge Over the Indian River, Brevard County, Florida FDOT District 5
6. SR 20 over Rocky Bayou WB FDOT District 3
7. SR 83 US 331 Bridge over Choctawhatchee Bay near Ramsey Branch Road, FDOT District 3
8. SR 71 West Arm Lake, FDOT District 3

Interviews were conducted with the appropriate engineers and project managers for the sites involved to gather this data.

1.3.2 Task 2 - Retesting Program Development

In order to improve the knowledge about the soil types that may produce high pile rebound, a specialized *in-situ* field and laboratory testing program was developed. Three accessible high pile rebound sites with extensive geotechnical data were chosen for the retesting program. The objective of the retesting program was to provide sufficient engineering data for the research team to determine if certain geotechnical properties would be associated with high pile rebound soils. Work was performed by the research team consisting of personnel at the Florida Institute of Technology (Florida Tech), GRL and Ardaman in Orlando and FDOT. The following FDOT bridge sites, where PDA test data were collected during high pile rebound driving, were chosen for retesting; 1) the Anderson Street Overpass at the I-4 SR 408 Interchange (Anderson Street Overpass), 2) the I-4 and John Young Parkway intersection (John Young) and 3) the SR 83-US 331 Bridge over Choctawhatchee Bay near Ramsey Branch Road (Ramsey Branch Bridge).

The location, number and types of tests conducted along with the depth, number and types of soil samples retrieved were specified by Florida Tech and performed in conjunction with FDOT, Ardaman and GRL. Conventional and specialized geotechnical field-tests, including SPT, CPT, Pocket Penetrometer, Dilatometer

(DMT) and PENCEL Pressuremeter (PPMT) were conducted and the data were evaluated. Throughout the SPT borings, the drill rod at the drive head was equipped with accelerometers and strain gages to allow PDA to collect data during each SPT hammer blow. These devices were supplied by GRL who also collected the original PDA data to produce SPT hammer efficiencies. In an attempt to visually access rebound, the Split Spoon Sampler tip was capped and driven with the 140 pound hammer and 30-inch drop to produce testing similar to driving large displacement piles. A flat plate slightly larger in diameter than the standard 2-inch diameter split spoon was welded to a drill rod. The enlarged plate diameter ensured that only end bearing resistance would be developed during testing. This equipment, along with the PDA instrumentation and a data acquisition system was used to perform "Closed End SPT (CESPT) testing" to provide additional information about the soil behavior and rebound during SPT impact driving. CESPT tests were videotaped using a JVC model GZ-MG155U digital video camera to allow the research team to view the driving.

The retesting program was developed to produce index properties, such as the Atterberg limits, soil stiffness and strength parameters, and pore pressure variations. It was anticipated that variations in soil stiffness and/or strength in combination with variations in excess pore pressures may be related to high pile rebound.

1.3.3 Task 3 - Retesting Program

The testing program developed in Task 2 was conducted at the three high pile rebound sites. Test borings were conducted by Ardaman & Associates at the Anderson Street Overpass site, and by FDOT at both the John Young and Ramsey Branch Bridge sites. All soundings, with cone DMT and PPMT equipment were conducted using FDOT rigs. Following completion of the SPT, CESPT, CPT, DMT and PPMT testing all three sites were visited for a second time and undisturbed thin walled tube samples (i.e., Shelby Tubes) were obtained for additional laboratory testing.

The Anderson Street Overpass site was selected as the first retesting site. It was the most accessible for the research team and allowed sufficient room to perform standard test borings along with CPT, DMT and PPMT soundings. It also enabled the research team to modify the testing protocol as needed to produce the most useful data. In September 2008, three SPT test borings were conducted with an automatic hammer and rods instrumented with the PDA accelerometers and strain gages. There were also five CPT soundings, three CPT with pore pressures (CPTu), one DMT sounding, and one PPMT sounding as well as one boring to obtain undisturbed thin walled tube samples in the high rebound soils. In early November 2008, CESPT testing was conducted in one boring. After a review of the test data, one final test boring with the PDA instrumented SPT tests with a safety hammer was performed in mid-December 2008. This final testing enabled comparisons between the automatic hammer and safety hammer used during the original investigation.

The Ramsey Branch Bridge site was retested immediately after the initial testing was completed for the Anderson Street Overpass in September 2008. Prior to the retesting, FDOT had conducted two sets of SPT borings at Ramsey Branch Bridge. The first set was for the original construction of the bridge and were completed in 1990. The second set was conducted for a proposed realignment of the roadway and bridge and were completed in 2006. The SPT borings from 2006 were conducted with an automatic SPT hammer; therefore, the retesting performed with the safety hammer at Anderson Street Overpass was excluded and only automatic hammer SPT tests were performed. Due to limited access at this site only one SPT test boring was performed, along with two CPTu soundings and one PPMT sounding.

The final site to be retested was the John Young site. The testing started in January, 2009. Three test borings were conducted with SPT tests equipped with an automatic hammer plus rods instrumented with the PDA accelerometers and strain gages. The remaining CPT, DMT, PPMT, Shelby Tube tests were completed over the next 5 months using a combination of Ardaman and FDOT drilling and sounding equipment. Two CPTu soundings, one DMT sounding and two PPMT soundings were completed, along with one test boring from which thin walled tube samples were retrieved.

After viewing the CESPT Anderson Street Overpass digital video, which was obtained using a JVC model GZ-MG155U digital video camera mounted on a tripod, it was determined that the camera sampling rate in frames per second, was too slow to record rebound in milliseconds. As a result, CESPT testing was not performed for either Ramsey Branch Bridge or John Young. High speed videos should be used during driving process high rebound piles. The equipment used must be able to record at a high enough rate to view the rebound process. Video cameras with sampling rates in excess of 400 frames per second are recommended, since cameras with rates at or below this value would produce less than 100 frames during the pile rebound process.

Kathy Gray, P.E., District 5 Geotechnical Engineer, coordinated SPT testing, Samuel Weede, P.E., District 3 Geotechnical Engineer conducted the SPT work for the sites in the Panhandle and Dr. David Horhota, P.E. with the State Materials Office (SMO) coordinated both the SPT and CPT testing using the SMO Cone Truck.

The disturbed samples from SPT split spoon samplers were tested on site using a pocket penetrometer to obtain unconfined compressive strengths. Then these samples were placed in airtight containers and transported to Ardaman, FDOT SMO or Florida Tech for grain size, Atterberg Limit testing and moisture content determinations. CPT data was used to determine cone point and friction values, while DMT and PPMT data were used to determine elastic moduli. Shelby tube samples were used for the following series of tests:

- Grain size distribution
- Moisture content
- Atterberg Limits
- Permeability using Flexible Wall Permeameters
- Triaxial Shear

1.3.4 Task 4 - Data Reduction

Test boring data were used to develop soil boring logs for each test boring location at each site. These borings, produced by Ardaman, were then used to develop generalized soil profiles, which included soil identification, classification, plus index and engineering properties. FDOT's State Materials Office (SMO) produced the CPT profiles from the cone point and friction resistance data collected from each sounding.

The laboratory testing conducted through a coordinated effort shared by Ardaman, Florida Tech and FDOT SMO produced:

- Grain sizes distribution curves, which in conjunction with the liquid and plastic limits, were used to determine the Unified Soils Classification System (USCS) symbol
- Permeability
- Elastic moduli, failure stresses and Mohr-Coulomb Failure envelopes

The field testing was used to produce the following pertinent data:

- SPT N values from either safety or automatic hammers
- Pocket Penetrometer unconfined compressive strengths
- CPT point and friction resistances
- DMT Elastic moduli
- PPMT Elastic moduli

Although a significant amount of additional data was available from this extensive testing program, the research team focused on data that most consulting engineers would produce and use for the design of foundation systems.

1.3.5 Task 5 - Data Analysis

The reduced data were analyzed to determine any special soil conditions that would indicate the possible occurrence of high pile rebound. The Florida Tech researchers worked with the professionals from FDOT, GRL and Ardaman to complete this work. This effort included several meetings and numerous conversations among all the professionals involved. Mr. Mohamad Hussein of GRL conducted a two day seminar with the research team to review the dynamic data obtained from both the instrumented SPT testing and the Pile PDA testing.

1.3.6 Task 6 - Develop Improved Geotechnical Testing Recommendations

Possible modifications to the design phase field-testing procedures are recommended based on the analysis of data from the retesting program.

1.3.7 Task 7 - Reporting and Technology Transfer

Throughout the project progress reports were submitted to FDOT and presentations were made to FDOT personnel. This final report summarizes the research findings.

2. Literature and Case Studies

According to Hussein et al. (2006) high pile rebound causes several pile driving problems, including premature driving refusal and high dynamic tensile stresses. The high stresses could result in damage to concrete piles. High pile rebound is predominately observed when driving displacement piles, such as solid concrete or closed-end steel or concrete piles, plugged pipes and H-piles, into saturated dense or hard soils. These soils are typically described as dense silty sand, hard silty clay, glacial till, etc. The pile rebound in these soils generally tends to increase as driving progresses due to increased pore water pressure. High pile rebound is often accompanied by high tensile reflection stresses in the pile when driving into a relatively weak toe soil. A thicker hammer cushion is often required to reduce these stresses in concrete piles, thereby reducing the available driving energy. High pile rebound is more likely when the toe penetrates cohesive soils, possibly with moderate to high shear strength and low stiffness.

2.1 Definition of High Pile Rebound

Pile rebound is defined as the upward elastic pile displacement that occurs during a hammer blow. Figure 2.1shows a typical pile-top displacement versus time record for a single hammer blow. The maximum initial downward motion is termed "Dmax," and is the sum of elastic and plastic deformations of the pile and soil system. The final value of the displacement is the permanent pile penetration for the blow, termed "set." Rebound is the difference between the pile maximum displacement and final set. High rebound describes the situation where the set (i.e., plastic soil deformation) represents a small portion of the maximum displacement and the rebound (i.e., recovered elastic deformation) constitutes the majority of the displacement. In high rebound cases, the hammer energy is absorbed primarily by elastic deformations, with very little remaining for useful pile penetration work. High rebound pile driving is typically characterized by very high pile driving blow counts (i.e., very small pile set per blow) and visibly large initial pile displacement under each hammer blow. This situation negatively affects pile drivability, induces high dynamic pile driving stresses, and complicates the assessment of static pile load bearing capacity based on observed driving blow counts. Moderate rebound does not typically present a problem during driving as long as the set is large enough to allow pile penetrations per blow that are reflected in reasonable driving blow counts.

FDOT specification 455-5.10.3 on Practical Refusal defines refusal as 20 blows per inch with the hammer operating at its highest fuel setting or at a setting determined by the engineer and less than 1/4 inch rebound per blow (FDOT, 2010). For this research high pile rebound was defined as any rebound exceeding the specified 1/4 inch limit.

2.2 Definition of Soil Quake

Quake is defined as the modeling parameter describing the soil's initial elastic response from the energy of a single hammer blow and is modeled by a straight line on a force displacement plot. Quake is used to compute the pile displacement when the soil goes from elastic to plastic behavior (Murrell et al., 2008). Figure 2.2 shows the simplified model of the soil response to a single hammer blow and the relationship of quake and rebound to the overall displacement. Quake is typically 0.05 to 0.1 inches or B/120 for granular soils, where B is the pile diameter, but may increase to 0.5 inches or more for cohesive soils (Smith, 1960). Hussein et al. (2006) concluded that high toe quake may result as displacement piles penetrate cohesive soils with low stiffness. High quake may be related to high pile rebound; therefore, case studies documenting high quake are included in the following sections.

2.3 High Pile Rebound PDA Output

PDA data recorded from a site with high rebound is shown in Figure 2.3. The plot, with two variables on the vertical axis, i.e., displacement in inches and energy in ft-kips, and one variable on the horizontal axis, time in milliseconds, shows a maximum displacement (D_{MAX}) of 0.75 inches, a set of 0.025 inches, thereby yielding a rebound of 0.725 inches. The software screen also displays a Time Scale (TS) of 204.8 milliseconds over which the data is recorded, a Time Beginning (TB) of 0.0, a Displacement (D) of 1.00 inch, which is located vertically and to scale, on the plot, and the Pile's Elastic Modulus (E) of 3,000 ksi.

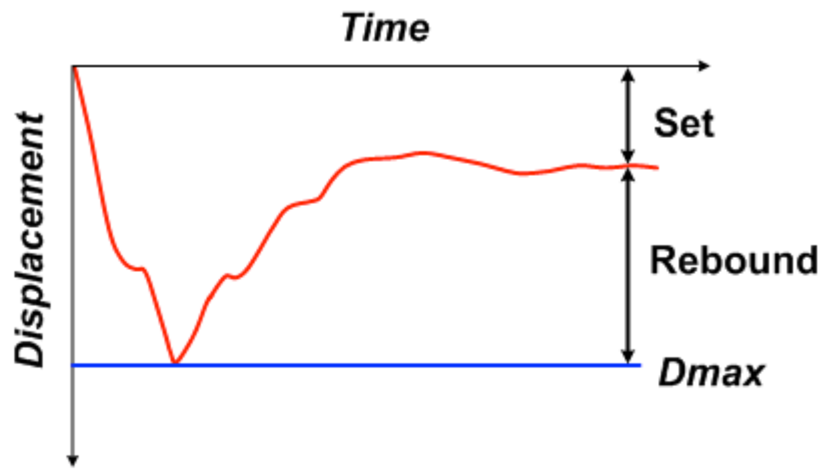


Figure 2.1 Displacement vs. Time

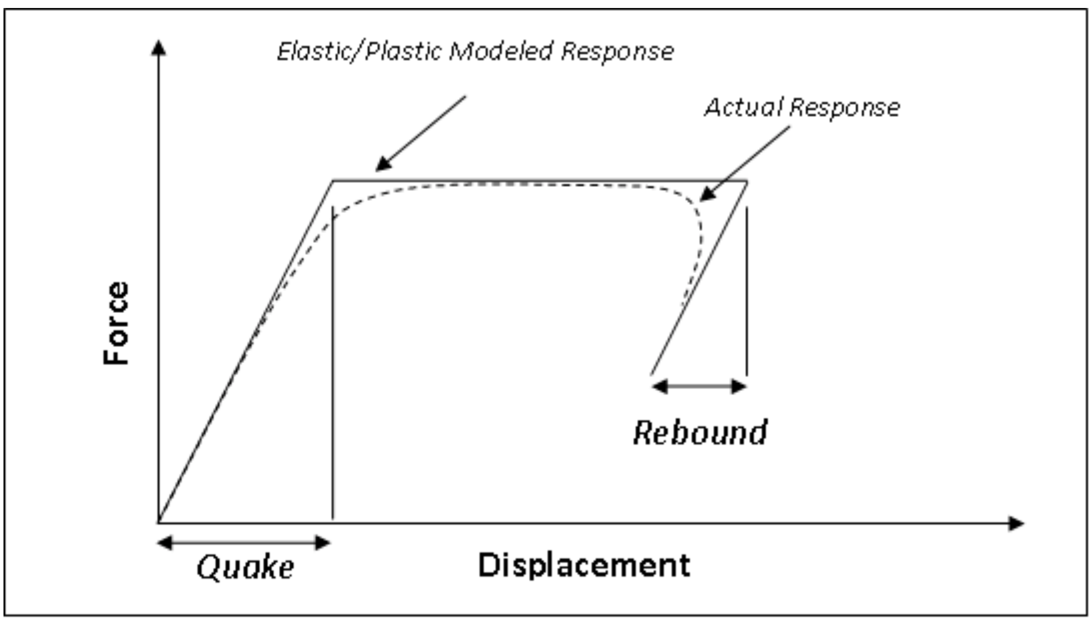


Figure 2.2 Simplified Force Displacement Model with Quake and Rebound

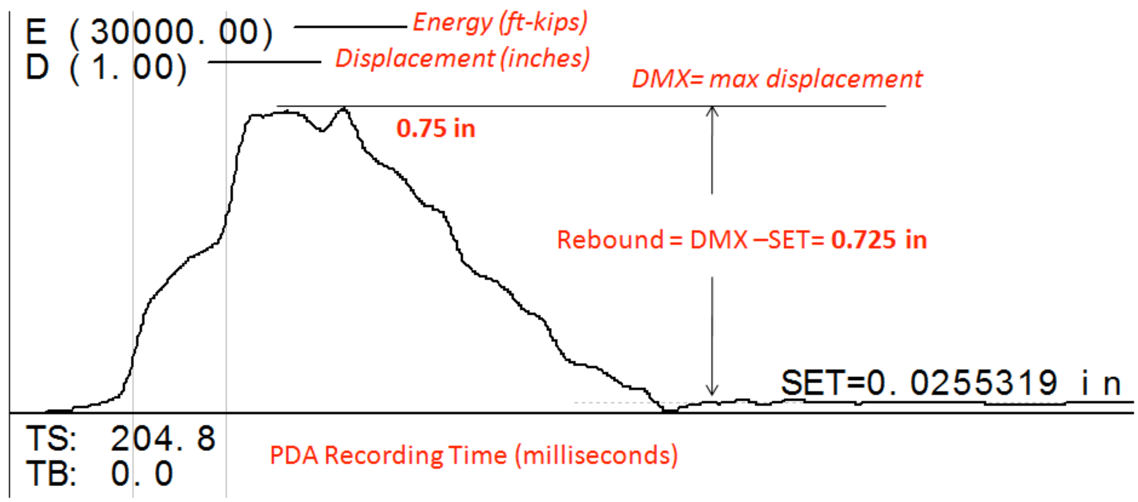


Figure 2.3 PDA Energy (ft-K) Displacement (in) versus Time (ms) Results Showing High Rebound

2.4 Overview of the Dynamic Pile Driving Analyses

E.A.L. Smith (1960) proposed using the wave equation to dynamically model pile driving. A second order differential equation of the wave speed with respect to time was the basis for the model of the pile's dynamic capacity. To account for instantaneous and time dependent movements, combinations of springs and dashpots were used to model the hammer, pile and soil responses. This analysis, although too complex for hand calculations, was incorporated into specialized software packages (i.e., GRLWEAP and CAPWAP) developed from research termed the Case Method (Goble and Raushe, 1976 and Goble et al., 2002). GRLWEAP simulation software can be used to predict a pile's response to driving and help engineers to select a suitable hammer for the given subsurface conditions. Data acquired from accelerometers and strain gages mounted near the top of pile, which is recorded using PDA, is integrated to produce force and velocity variations with time for each hammer blow. PDA force and velocity measurements from selected hammer blows can be input into signal matching software such as CAPWAP to estimate static resistances along the shaft and at the toe by inputting various trial combinations of damping factors and quakes for both the shaft and toe. The signal matching software produces force and velocity traces over time that match the recorded force and velocity traces during the 2L/c milliseconds after the hammer blow, where the pile length is L and c is the wave speed through the pile generated by the hammer.

2.5 USA High Pile Rebound Cases

2.5.1 Large Displacement Piles at State Road 528 over Indian River

Hussein et al. (2006) describes a case study of high pile rebound that occurred along State Road 528 over the Indian River in Brevard County, Florida, where, 3855 ft (1175 m) of the westbound lanes of the bridge were replaced. Six piles were driven at each end bent, and groups of 9 to 20 piles were placed at the piers. All of the piles installed were 30 inches (762 mm) square PCPs with a length of 114.8 ft (35 m). Each pile had an 18 inch (457 mm) circular hollow core throughout most of the pile length with a 3.9 ft (1.2 m) solid section on each end. Two test piles were installed using a Raymond 8/0 Single Acting Air Hammer: one on land and the other in the river. They were both tested dynamically and statically. This case history is based on the test pile installed in the river.

The river had a 6 foot water depth. The soil profile consisted generally of three layers. The upper stratum was saturated very loose to medium dense poorly graded silt with sand (SP-SM) to silty sand (SM) with shell and coquina fragments to elevation -89 ft. This stratum was underlaid by firm to hard clayey sand (SC) to sandy clay (CL) to elevation -144 ft, which was underlaid by limestone to the end of the boring at elevation -144.

Piles penetrated into the hard clayey sand (SC) to sandy clay (CL) between elevations -89 and - 144 ft. The Raymond 8/0 single-acting air hammer, ram weight was 24.9 Kips (111 kN) with a maximum energy of 81.1 Kip-ft (110 kJ). In this case, hammer energy was controlled using two different hammer strokes (1.5 ft and 3.25 ft; 460 mm and 990 mm). A 9 inch (230 mm) thick plywood pile cushion was used during the installation.

Pile driving at the beginning was relatively easy. Driving resistances were 3 to 17 blows per foot above elevation -60.7 ft (-18.5 m) with the hammer stroke set at 1.5 ft (460 mm). Between elevations -60.7 and – 63.7 ft (-18.5 to -19.4 m) the driving resistances increased to 43 blows per foot thus, the hammer stroke was increased to 3.25 ft (990 mm). Consequently, the blow counts decreased to 12 blows per foot within the next 3.9 ft (1.2 m). At elevation -67.2 ft (-20.5 m), tensile stresses in the pile exceeded the allowable stresses of 1200 psi (8.4 MPa). To reduce the tensile stresses the hammer stroke was decreased to 1.5 ft (460 mm) at elevation -67.3 ft (-20.5 m) and continued to elevation -97 ft (-29.5 m). Because blow counts increased to 61 blows per foot the hammer stroke was again increased to 3.25 ft (990 mm) and after 2 ft (600 mm) of driving tensile stresses to again exceeded the allowable values. At elevation -98.8 ft (-30.1 m) noticeable high elastic rebound occurred and driving was halted. To decrease the high tensile stresses in the pile, a 2.25 inch (57 mm) thick plywood cushion was added to the existing 9 inch (230 mm) cushion. An additional 5 ft (1.5 m) of pile length was driven with blow counts ranging from 103 to 170 blows per foot. At an elevation of -101.7 ft (-31 m), pile driving was stopped with 147 blows per foot. A fifteen minutes a set check was performed according to FDOT Specification 455. Rebound, recorded with PDA equipment during the set check, decreased significantly from 0.79 inch (20 mm) to 0.47 inch (12 mm), which was still excessive.

Nineteen days after installation, a static load test was conducted on the pile. Results indicated about 3/4 inch (20 mm) of pile top movement at 1125 Kips (5000 kN). Nineteen days after the static load test, a long term restrike was performed using the same hammer. The pile was subjected to blow counts of 12, 12 and 17 blows per inch to produce a penetration of 3 inches (75 mm). During the long term pile re-strike, the PDA instrumented pile rebounded 0.35 inches (9 mm) per blow at the start, increasing to 0.52 inch (13 mm) per blow at the conclusion of restrike. According to the author, excessive pore water pressures might have caused the pile to rebound. However, no analytical proof of this conclusion was presented.

2.5.2 Displacement Piles in Stiff Clays in North Carolina

Murrell et al. (2008) presented a case of high pile rebound, which he termed as "bouncing," during the construction of a new ferry terminal in coastal North Carolina. Twelve-inch (305 mm) square PCPs were used on land, and 20 inch (508 mm) square PCPs were used over water. The 12 inch (305 mm) piles were 55 feet (17 m) long and the 20 inch (508 mm) piles were 70 feet (21.3 m) long. At the land section, the subsoils from the ground surface to 38 feet (11.6 m) consisted of very loose to very dense sand, followed by 22 feet (6.7 m) of stiff clay, and finally 10 feet (3 m) of medium dense to very dense sand. At the water section, from the top of the mudline to 12 feet (3.7 m) was either soft clay or loose sand, followed by 14 feet (4.3 m) of medium dense to dense sand, underlaid by 20 feet (6.1 m) of firm to stiff clay and finally 26 feet (8 m) of dense to very dense sand. The rebound layer in this case was in the 22 foot thick (6.7 m) firm to stiff clay at elevation -52 ft (15.8 m).

The 12 inch (305 mm) square PCPs were installed using a single acting diesel hammer (Berminghammer

B3505) with a 4 kip (17.8 kN) ram. A 6 inch (152 mm) thick plywood pile cushion was used to install all of the piles. Even though high rebound was not observed during the installation of the 12 inch PCP piles, driving resistances were high for some layers. Blow counts reached 100 blows per foot in sand and 50 blows per foot in clay at a depth of 22 feet (6.7m). At 35 feet (10.7 m), the blow counts started at eight and increased to 33 blows per foot. After a four hour break, the blow counts decreased to 13 blows per foot. Calculations from PDA equipment indicated that high tensile stresses occurred during easy driving and high compressive stresses during hard driving. It was concluded that the pile cushion caused these high stresses since it was used several times.

The 20 inch (50.8 mm) square PCPs were installed using an open-end diesel hammer (Berminghammer B4505) with a 6.6 kip (29.4 kN) ram. Also a 6 inch (152 mm) thick plywood pile cushion was used for the installation of these piles. Bouncing was observed during the driving process at an elevation of -43 feet (-13.1 m). The pore water pressure at the rebound depth, determined from the CPT sounding, was 20 tsf (1915 kPa). When the blow counts reached 303 blows per foot, the pile displacement became zero. The pile driving process was stopped for two hours and then the pile was driven again; however, the driving resistances remained high (73, 112, 87 blows per foot). In order to achieve the pile capacity, after a period of four days the pile was driven again. According to the PDA data, there was no damage to the test pile and its capacity was satisfactory.

Immediately after installing the tests piles, eighteen production piles were installed. High pile rebound was observed in all the production piles. Circumferential cracks were reported in a few of the piles. To overcome pile rebound, a hammer with a larger ram and a short stroke was used (i.e., a Berminghammer B5505 open-end diesel hammer with a ram weight of 9.2 kips (41 kN)). This hammer impact duration is longer, reducing the tensile stresses and the amount of rebound. The pile cushion was also increased to an 8 ½ inches (216 mm). Although the hammer was changed, the piles were still bouncing. Nevertheless, the rebound recorded was less than that when using the previous hammer.

In summary, the amount of pile rebound was not reported. Use of the term bouncing needs further analysis because it may indicate the extreme rebounding that would occur near the depth of pile refusal. The piles encountered driving resistances of 175 and 159 blows per foot. Low-strain dynamic testing was performed on all piles. The testing identified five piles that were damaged at a depth between 14 feet to 32 feet (4.3 m to 9.8 m).

According to this case study, piles are most likely to rebound in saturated, firm to stiff, fine grained soils that originate from marine formations along the southeastern coast of the United States (Murrell et al., 2008).

2.5.3 Displacement Piles with Large Quake

Likins (1983) studied large displacement piles from three different sites driven into soils that produced large toe quakes. The site locations were not presented in the case study; however, email correspondence with the author revealed that one site was in Seattle, Washington and the locations of the other two sites are unknown. Based on the pile sizes the author inferred that these sites may have been in Florida (Likins, 2009). The soil at all of these sites is saturated. The author classified three variables of the pile driving process as the soil conditions, the driving forces and the pile movements.

At site 1, the performance of 24 inch (610 mm) octagonal prestressed concrete hollow piles is presented. These piles had an area of 300 in^2 (1935 cm^2) and were 70 ft (21.3 m) long. The soil is composed of glacial deposits overlaying hard silty clay at 27 ft (8 m) below the ground surface. The piles were installed using a Kobe K45 open-end diesel hammer with an estimated energy of 91 kip-ft (124 kJ). A 10 inch (250 mm) plywood cushion was used during pile driving. The piles were first predrilled to 12 ft (3.7 m), and then they were driven to 45 ft (13.7 m). After three days, restrikes were performed with PDA monitoring. Driving resistance increased to 21 blows per inch at 57 ft (17.4 m). Pile driving was stopped when blow counts increased to 50 blows per inch. In order to decrease the blow counts, the plywood cushion was reduced 4 inches (100 mm) and the blow counts were reduced to 22 blows per inch at a stroke of 7.7 ft (2.35 m).

High tensile stresses were recorded near the end of the restrike. CAPWAP analyses were conducted using the restrike data to determine a toe quake of 0.42 inches (11 mm). Several production piles, with PDA instrumentation, were dynamically tested during driving. One of these piles was a prestressed concrete pile with an area of 300 in^2 (1935 cm^2) and a length of 95 ft (29 m). The pile was designed to have an ultimate capacity of 550 kips (2475 kN). During driving, the tensile forces in the pile reached 500 kips (2250 kN) while compression forces reached 1250 kips (5625 kN). When the driving resistances reached 10 blows per inch, the pile broke due to the high tensile stresses. The failure occurred 56 ft (17 m) below the PDA transducers. From this data a CAPWAP analysis indicated the toe quake was 0.55 inch (14 mm). The maximum computed toe displacement using CAPWAP was 0.69 inch (17 mm).

After several weeks, the pile was tested statically. After the static test, restrikes were again performed using the same hammer and large quakes were not found. Between initial driving and the restrike driving, pore pressures dissipated and the pile capacity increased.

At site 2, seven piles were tested dynamically. These PCP piles were 122 ft (37.2 m) long and 24 inches (610 mm) square. They were prejetted through a gray clayey sand layer to 100 ft (30.5 m) and driven into dense light gray sand. The hammer used to drive the piles was a Raymond 80 hammer with energy of 80 kip-ft (109 kJ). The observed blow counts were slightly over 17 blows per inch and the pile skin friction was minimal. The associated toe quake from CAPWAP was approximately 0.7 inch (18 mm).

At site 3 the installation of twelve, 18 inch (457 mm) square PCPs with a length of 80 ft (24.4 m) was summarized. These piles were driven using a Delmag D-30 hammer. The soil consisted of saturated dense fine sand with some silt or clay content. The piles were prejetted through these soils. Blow counts ranged from 2 to 42 blows per inch. At the end of driving, the maximum computed tensile stress was 0.6 ksi (4.13 MPa). At the lower blow counts, the tensile stress was 1.3 ksi (9 MPa). Toe quakes were from 0.4 to 0.5 inch (10 to 13 mm). After the analysis of the forces and velocity of these piles, it was concluded that one third of the piles were damaged. Table 2.1 summarizes these case studies.

Site Number	Rebound Soil Description	Hammer Blows (blows/in)	Toe Quake (in)
1	Hard silty clay	21 to 50	0.42 to 0.69
2	Dense light gray sand	17	0.7
3	Dense fine sand with silt or clay	2 to 42	0.4 to 0.5

Table 2.1 Likins (1983) High Pile Rebound Literature Summary

2.5.4 Large Quake Displacement Piles Driven into Canadian Glacial Till

Authier and Fellenius (1980) presented a case study where dynamic pile testing was conducted at two sites containing silty glacial till in the Provinces of Ontario and Quebec, Canada. Site 1 was near Timmins, Ontario, and site 2 was on the south shore of Montreal, Quebec. These two sites produced large soil quake at the pile toe. At site 1, sixteen piles instrumented with PDA sensors were tested. These piles were 12.75 inch (324 mm) closed end steel pipe piles with wall thicknesses of 0.31 inch, 0.33 inch, and 0.375 inch (7.9 mm, 8.4 mm, and 9.5 mm), respectively. The soils at this site consisted of very dense sandy silty glacial till. Four hammers were used for the pile driving: one drop hammer and three open-end diesel hammers. The drop hammer had an energy of 20 ft-kips (27 kJ) and was not able to successfully drive the piles, and the diesel hammers had energies of 29, 34, 46 ft-kips (39 kJ, 46 kJ, and 62 kJ). The drop hammer was successfully used at a previous site in similar soil conditions.

A static load test was performed on one of the piles, which was driven by the lightest diesel hammer. The test showed that the pile did not achieve its design bearing capacity; however, the same hammer was used to install a similar pile nearby that did reach the design capacity. PDA data was analyzed using CAPWAP software indicating large toe quake had occurred during driving, with values ranging from 0.1 to 0.75-in (2.5 to 20-mm). The authors indicated that the CAPWAP force match comparison produced a good match between forces computed and those measured.

At the second site, twenty-four 12 inch (305 mm) square PCPs were driven into 36 ft (11 m) of thick clay underlaid by a dense clayey silty glacial till. A Berminghammer B-400 open-end diesel hammer with an energy of 46 ft-kips (62 kJ) was used for installation and a plywood cushion of unspecified thickness was used. The pile driving produced low blows through the clay layer. After 36 ft (11 m), the pile encountered the glacial till with blow counts of 1 blow per inch. Once into the glacial till, the driving resistance increased to 5 blows per inch. However, at 41 ft (12.5 m) the driving process became difficult and the blow counts increased to 50 blows per inch. The pile penetration at 37 ft (11.3 m) was 1.58 inch (40 mm); in contrast, the pile penetration at 41 ft (12.5 m) was 0.02 inch (0.5 mm) per blow. During the last 6 inches (150 mm) of pile driving, resistance increased from 25 blows per inch to 50 blows per inch. After one hour, a pile restrike was performed producing a resistance of 58 blows per inch. At the end of the final driving and at restrike, 0.6 inch (15 mm) of rebound was recorded for a maximum pile head displacement of 0.63 inch (16 mm). The maximum CAPWAP toe quake was 0.25 inch (6 mm). Table 2.2 summarizes the information from both sites.

Site	Site Location	Rebound Soil Description	Hammer Blows (blows/in)	Toe Quake (in.)	Rebound
1	Timmons, Ontario	Very Dense Sandy Silty Glacial Till	N/A	0.1 to 0.75	N/A
2	Montreal, Quebec	Dense Clayey Silty Glacial Till	25 to 50	0.25	0.6
N/A - Not Available					

Table 2.2 Canadian Silty Glacial Till High Pile Rebound Summary

2.5.5 Case Studies General Conclusions

Hussein et al. (2006) attributed high pile rebound at the Indian River Bridge over State Road 528 to excessive pore water pressure but this conclusion was supported analytically. In order to decrease rebound and tension stresses in the pile, the plywood cushion thickness was increased. In the North Carolina case study, piles developed circumferential cracks from excessive driving stresses. Murrell et al. (2008) explained that a hammer with a larger ram and a short stroke and therefore a longer contact time during each blow reduced rebound and tension stresses. Murrell et al. (2008) also mentioned that soil layers with pore pressures greater than 20 tsf (1915 kPa) recorded during the CPT are likely to cause high pile rebound. Likins (1983) case study pointed out that the piles had large quake and high tension stresses. The Canadian case study (Authier and Fellenius, 1980) presented piles with excessive toe quake.

Table 2.3 summarizes the case studies' information. The information shows the following:

- Piles were displacement piles;
- Soils in the rebound layers typically contained silts and clays;
- Soils in the rebound zone were considered to be dense to very dense or stiff;
- Both high quake and high rebound occurred when reported;
- Piles were longer than 40 feet;
- Pile driving hammers were single acting;

Author	Description and Location	Pile Type, Shape and Area (in^2)	Pile Length	Hammer Type and Model	Hammer Blows (Blows/in)	Rebound (in)	Toe Quake (in)	Rebound Soil Description
Hussein 2006	FDOT S.R. 528 Project Brevard County, Florida	2.5 ft Square PCP with 1.5 ft Circular Hollow Core 900-in^2	115	Raymond 8/0 single-acting air	8 to 12	0.5 - 0.8	Not Reported	Elev. -89-ft Hard Clayey Sand (SC) to Sandy Clay (CL) to Elev. -125-ft
Murrell 2008	Coastal North Carolina	1-ft Square PCP 144-in^2	55	Berminghammer B-3505 single acting diesel	3 to 8	None Observed	Not Reported	Firm to Stiff Clay
		1.67-ft Square PCP 400-in^2	70	Berminghammer B-4505 single acting diesel	25	Observed not Reported	Not Reported	
Authier & Fellenius 1980	Site 1 Timmons Ontario Canada	Closed-Toe Pipe piles Circular 12.75-in O.D. 128-in^2	41	Drop and three diesel Models N/A	Not Reported	Not Reported	0.1 - 0.78	Very Dense Sandy Silty Glacial Till
	Site 2 Montreal Canada	1-ft Square PCP 144-in^2	41	Berminghammer B-400 single acting diesel	25 to 50	0.6	0.8	Dense Clayey Silty Glacial Till
Likins 1983	Site 1 Seattle, Washington	2-ft Dia. Octagonal PCP (Hollow) 452-in^2	70	Kobe K45 single acting diesel	21 to 50	Not Reported	0.42	Hard Silty Clay
	Site 2	2-ft Square PCP 576-in^2	Up to 122	Raymond 80 (Raymond 8/0) single-acting air	17	Not Reported	0.7 [1]	Dense Light Gray Sand
	Site 3	1.5-ft Square PCP 324-in^2	80	Delmag D-30 single acting diesel	2 to 42	Not Reported	0.4 - 0.5 [1]	Dense Fine Sand with some Silt or Clay

[1] Calculated quake values from CAPWAP analysis.

Table 2.3 High Pile Rebound Literature and Case Histories Summary

2.6 Geology of Florida

High pile rebound may be related to the geology at the construction location. In Florida, the top of the Hawthorn Group matches the initial elevation associated with rebound. For that reason, the geology of Florida was studied to try to identify the rebound layer. Water Resource engineers reference the Hawthorn Group as an impermeable boundary. In regions where the Hawthorn Group is absent, the surficial aquifers extend to the Florida aquifer system.

The Florida peninsula has been inundated numerous times throughout its creation. These inundations were caused by saline water that deposited marine life throughout the peninsula. Florida geology consists of carbonate rocks and siliciclastic sediments. Sedimentary rocks that have been deposited through mechanical process form these sediments. Carbonate sediments in northern Florida could be approximated to be around 2000 feet thick, and in southern Florida these sediments have a thickness greater than 5000 feet (Scott, 1992).

Highland areas are located in the Florida Panhandle where karsts are part of the geology of this region; this area is characterized by high recharge. The presence of karsts in the northern part of the state helps water infiltrate in the aquifer system. The southern part of the state is swampy and poorly drained. The sediments in the south area have low permeability creating confining layers (Scott, 1992).

2.6.1 Hawthorn Group

The Hawthorn Group is one of many geological formations in the peninsula of Florida. Through research, geologists came to a conclusion that the Hawthorn Group was formed in the middle of the Miocene (in Greek "less recent") epoch. The sediments were placed after the erosion of the karstic limestone surface of Florida. The Hawthorn sediments deposition ended in the late Early Pliocene (Scott and MacGill, 1981). The group occurs eastward of the Apalachicola River, northward to Berkeley County in South Carolina, and southward through the Peninsula of Florida. In some parts of Florida this formation has been entirely eroded.

The sediments in the Hawthorn Group vary constantly with geography. As a result, geologists changed the status of formation to Group (Scott, 1990). However, some people still use the name Hawthorn Formation.

The Hawthorn Group is a combination of alluvial, terrestrial, marine, and deltaic beds. It is a great source of phosphate containing over 10 billion tons (Scott and MacGill, 1981). In the beginning, the Hawthorn Group was confused with the Alum Bluff Formation. The Hawthorn Group is contemporaneous with Tampa and Chattahoochee formations. In some cases, it is hard to differentiate between the Tampa and Hawthorn Group. Generally, the group contains a combination of sand, silt, clay, phosphate, limestone, and dolomite. Phosphate is present in the formation, as much as 60 percent in some cases, but the mineral is absent in other locations (Scott, 1990). The sediments found in the group come from the repetitive sea level events that inundated the peninsula of Florida.

A silty, sandy, phosphatic dolomite is the most common constituent in this group. The sizes of this mineral vary, and the most common size is 0.002 to 0.005 inch (0.0625 mm to 0.125 mm). It has a yellowish-gray (5Y 7/2) to white (9N) color (Scott and MacGill, 1981). These code numbers correspond to the Soil Color Chart.

The limestone in the formation contains varying amounts of sand, clay, and phosphate. The color of the limestone is mainly white (N9), yellowish-gray (5Y 7/2) to very pale orange (10YR 8/2). Limestone is not found everywhere in the formation; it is scattered throughout the region changing its thickness from two to thirty feet. In the group, clay beds can have different colors from yellowish-gray (5Y 7/2), to light green (5G 7/4), to moderately dark gray (N4). Clay soils represent a small part of the Hawthorn Group (less than 5 percent). The clay beds contain quartz silt and sand, dolomite, and phosphate in different percentages (Scott and MacGill, 1981). The abundant clays in the group are montmorillonite and palygorskite (Houstine, 1984).

Another constituent of the group is the sand beds which can vary in color from light gray (N7) to very pale orange (10YR 8/2), to dusky yellow-green (5GY 5/2). They also contains quartz particles with an angular to subangular shapes. The sand beds have some phosphate and silt present; usually these beds have a thickness of two to three feet.

The successive marine inundations that Florida peninsula suffered million years ago facilitated the presence of fossils in the Hawthorn Group. For example, chert is a sedimentary rock that contains small fossils and is present in the formation. This fine-grained sedimentary rock ranges from a few inches to two feet in thickness. The chert bed corresponds to a small percentage of the Hawthorn Group and is not present everywhere. The bed can be identified with the color medium (N5) to dark (N3) gray. Chert also contains quartz, sand, and phosphate.

Phosphate is one of the principal components in the Hawthorn Group. Some beds in Central Florida contain 25 percent phosphate, but generally the percentage of phosphate in a bed is less than 10. The color of phosphate ranges from black to tan and white in other sections. The grain size distribution of phosphate varies from coarse silt to gravel (Scott and MacGill, 1981).

The Hawthorn Group is not a homogeneous layer. Bioturbation (i.e., the stirring of mixing of sediments by organisms) may have caused the Hawthorn Groups' distinct types of beds(Scott and MacGill, 1981). When the formation was created, marine life was present in the soil. As a result, the sediments could have been mixed by marine organisms. The fossils of the marine organisms were not preserved in the sediments because they were soft-bodied (i.e., without skeleton) organisms.

2.6.2 Geologic Traits of the top of the Hawthorn Group

The top of the Hawthorn Group can be identified by the minerals that the soil contains, such as calcareous or dolomitic phosphatic sand. The sediments at the top lack shell material and are normally an olive-green color (Hoenstine, 1984). The Hawthorn Group has an irregular erosional upper surface. This top part can be identified since it is often comprised of a clayey phosphatic residuum. It is difficult to distinguish between the Hawthorn Group and the Bone Valley Formation due to the residuum (Scott and MacGill, 1981). The Tampa Stage limestone can also be confused with the Hawthorn Group; however, the Tampa limestone does not contain as much phosphate as the Hawthorn Group. Also the Tampa limestone includes fossils such as corals and mollusks. The Tampa limestone is a dense yellow limestone with alternating hard and soft layers (Scott and MacGill, 1981).

The top of the Hawthorn Group forms the base of the surficial aquifer system (Scott and MacGill, 1981). The sediments of this formation are impermeable in some parts of the region. Typically the upper part of the Hawthorn Group may act as an aquiclude (permeability $k \approx 0$) (Hoenstine, 1984). Table 2.4 summarizes the Hawthorn Group characteristics including the different beds found in this geologic formation. Figure 2.4 shows the elevation of the top of the Hawthorn Group throughout Florida with the approximate locations of the FDOT high rebound sites.

The elevation of the top of the Hawthorn Group varies in the state of Florida. In the northwest it is at a higher elevation approximately 200 to 100 ft above the sea level. In the south it is at an elevation of 100 ft to 150 ft below sea level. In Central Florida, the elevation is approximately 50 ft to 0 ft above sea level. In the northeast it is at an elevation from 0 ft to minus 100 ft below sea level. These variations are shown in Figure 2.4.

In the southernmost part of Florida, the Hawthorn Group is the thickest, ranging from 700 to 900 ft (213 to 274 m). Central Florida corresponds to a thickness of 100 to 300 ft. The northeast area (100-500 ft) (30.5 to 152 m) has thicker sediments than the northwest area (100-250 ft) (30.5 to 76.2 m) of Florida (Scott, 1990). In the Panhandle, due to marine erosion, many of the middle Miocene (Hawthorn Group) and Pliocene layers are largely missing (Scott and MacGill, 1981).

2.6.3 Engineering Properties of the Hawthorn Group near Okeechobee Florida

Brown et al. (2005) reported engineering properties of the Hawthorn Group for deep wells in the

Okeechobee region of South Florida. The properties are summarized in Table 2.5. In general the soils, which are at depths below 200 ft (61 m), are overconsolidated, have very low permeabilities and Atterberg limits that average 46 for the liquid limit and 22 for the PI.

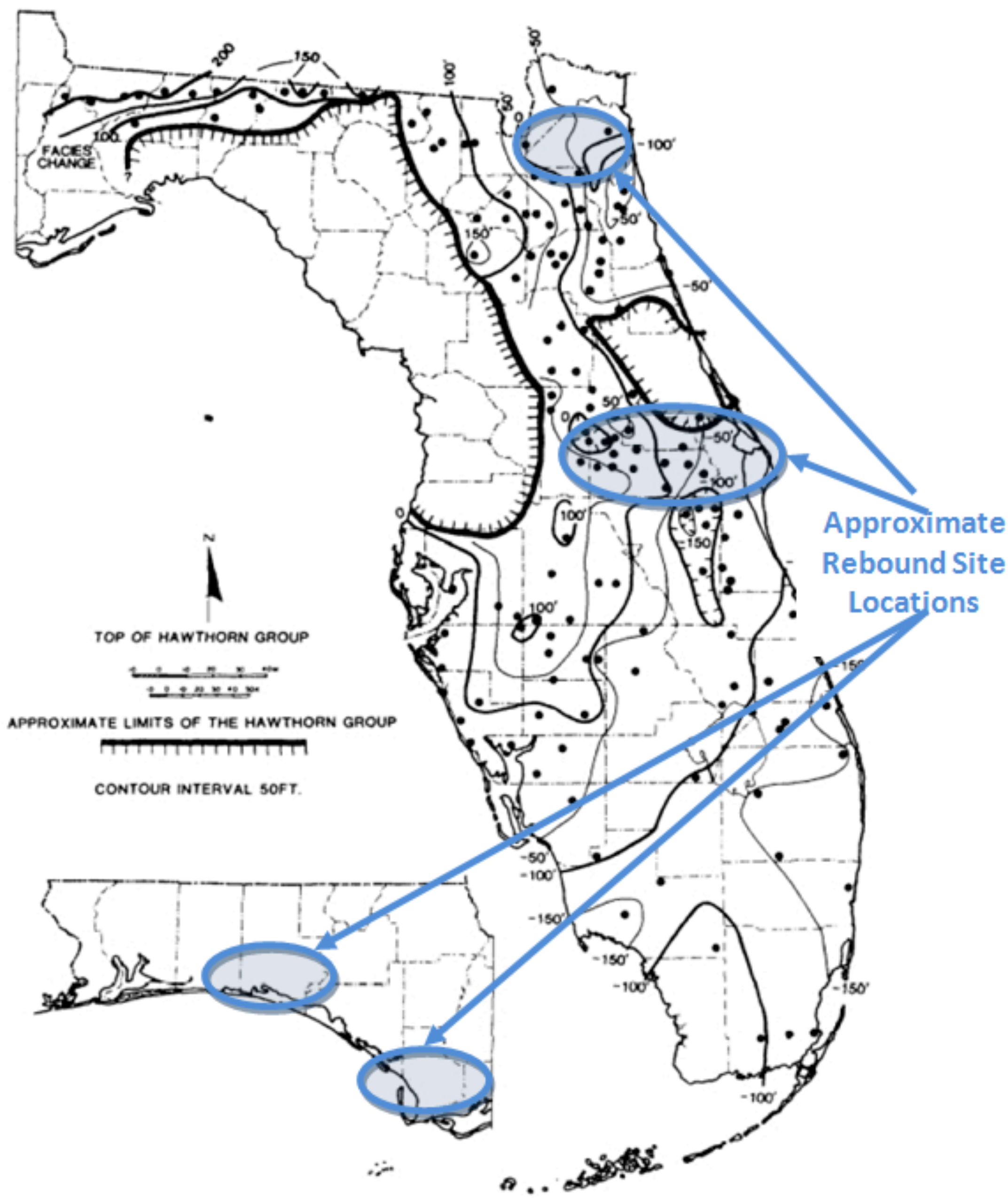


Figure 2.4 Top Elevation Contours of the Hawthorn Group with Approximate High Pile Rebound Site Locations [after (Scott, 1990)]

Top of Hawthorn Group in Florida	•Normally Olive Green Clayey Phosphatic Residuum •Calcareous or Dolomitic Sand •Impervious in some parts •Elevation 200-100 ft above sea level (North), 100-200 ft below sea level (South).
Clay Beds with quartz silt, sand, dolomite & phosphate	•Yellow-Gray or Light Green to Dark Gray •Less than 5 % of formation •Montmorillonite or palygorskite
Phosphate Beds 10 to 25 % of Central Florida formation	•Black to Tan & White •Over 10 Billion tons •Up to 60% in some places
Phosphatic Dolomite	•0.0625 to 0.125 mm grain sizes •Yellow Gray to White
Limestone	•White Yellow-Gray to Pale Orange •Between 2 ft and 30 ft •Scattered through region
Chert with Fossils	•2 inches to 2 feet thick •Medium to Dark Gray •Scattered throughout formation

Table 2.4 Summary of Hawthorn Layer Characteristics

Sample Depth (ft)	Natural Moisture (%)	Moist Unit Weight (PCF)	Liquid Limit (%)	Plastic Limit (%)	Specific Gravity	Effective Overburden Pressure (psf)	OCR	Compression Index	e_o	Kv (cm/sec)
220	23.7	111.49	40	22	2.647	10800	2.42	0.08	0.708	3.80E-09
220	18.8	115.73	77	55	2.625	11733	2.05	0.04	0.576	2.60E-09
292	20.9	112.94	26	4	2.692	14758	1.77	0.04	0.682	4.70E-07
292	21.6	114.98	29	8	2.653	15353	1.69	0.04	0.632	1.50E-07
380	16.0	116.98	52	22	2.757	20740	1.24	0.04	0.608	2.00E-09
380	20.2	113.40	55	21	2.657	19380	1.23	0.05	0.645	2.70E-09
218	20.2	108.30	65	41	2.681	10006	2.04	0.13	0.749	
285	22.8	117.13	28	5	2.692	15598	1.08	0.03	0.633	
382	21.6	112.18	40	18	2.723	19016	0.87	0.05	0.722	
Average	**20.6**	**113.68**	**46**	**22**	**2.681**	**15265**	**1.60**	**0.06**	**0.662**	**1.05E-07**

Table 2.5 Hawthorn Layer Engineering Properties from Brown et al. (2005)

2.7 Summary of Florida High Rebound Sites

During the investigation phase of the research, 11 problematic rebound sites in Florida were identified through verbal communications with various FDOT personnel and consultants including Ardaman and GRL. Three sites were located in the Florida Panhandle, six in the Orlando area of Central Florida, one in Jacksonville and one near the Cape Canaveral area of Central Florida. Table 2.6 lists the sites and their corresponding counties.

Sites	Name	County
1	Anderson St. Overpass at I-4/SR-408	Orange
2	I-4/SR-192	Osceola
3	I-4/SR-423 John Young Parkway	Orange
4	I-4/SR-482 Sand Lake Rd.	Orange
5	SR 50 and SR 436	Orange
6	SR 528 and US 441	Orange
7	SR-20 over Rocky Bayou WB	Okaloosa
8	SR-83 Ramsey Branch	Walton
9	SR-71 West Arm Lake	Gulf
10	SR-528 over Indian River	Brevard
11	I-10 at Chaffee Road	Jacksonville

Table 2.6 FDOT High Pile Rebound Sites

Seven of these sites were evaluated, with four selected for case study evaluations and three others selected for both case study evaluations plus an extensive retesting program. The four sites chosen for the case study evaluations were numbers 2, 5, 6, and 11 in the table above. These four sites covered a relatively large portion of Florida and information was readily supplied by FDOT personnel. The three sites chosen for the case study and retesting program were numbers 1, 3, and 8. These three sites were the most easily accessible of the known sites and information was also readily available.

Geotechnical information was gathered about the sites to help the researchers identify possible soil, pile and hammer types related to high rebound. This information included soil profiles, soil properties plus pile type and installation techniques.

2.7.1 Intersection of State Road 50 and State Road 436

Displacement piles were driven at the intersection of State Road 50 and State Road 436. Figure 2.5 is an aerial view of the site location. Twenty-four inch (610 mm) square PCPs were installed to support the bridge that measures 517 ft (158 m). These piles were 105 ft (32 m) long and the ground surface elevation was 98 ft. SPT testing was performed using a safety hammer. The soils at this site were mainly sand with varying percentages of silts and clays (SP, SP-SM, SC, SM). At elevation +15 ft (4.6 m), the soil description is light green clay (CH) with a trace of phosphate, cemented sand, and consolidated clay. This soil color and description matches the soil color and description encountered at the top of the Hawthorn group.

Figure 2.5 Aerial Map of SR 50 and SR 436 site, Orlando, Florida (from Google Maps)

The piles were driven using an APE model D62-42 diesel hammer with a ram weight of 13 kips (57.8 kN) and an energy of 154 ft-kips (209 kJ). A 9 inch (230 mm) thick plywood cushion was originally designed for the pile cushion. However, the pile cushion thickness during pile driving was changed to 12 or 14 inches (300 or 360 mm). The piles were installed in a hole predrilled to elevation +40 ft then were driven with a single acting diesel hammer. Several piles did not reach the specified minimum tip elevation of + 15.6 feet (4.75 m), corresponding to a depth of 82.4 ft (25.1 m), due to practical refusal of 20 blows per inch. Rebound was observed when installing the piles; however, no record of the amount or depths of rebound are shown in the pile logs. The piles were not instrumented during driving; therefore, no rebound data was available.

2.7.2 Intersection of State Road 528 and US 441

This case history describes a production pile installation in Central Florida where the overpass of State Road 528 (Beachline Expressway) intersects US 441 (Figure 2.6). These high displacement foundations are 18 inch (460 mm) square PCPs and measure 85 feet (25.9 m) in length. The tip elevation of the piles is between +10 feet to +14 feet (3.05 to 4.27 m), which matches the elevation of the top of the Hawthorn Group. The ground surface elevation in this site is approximately 96 ft (29.3 m). An APE model D30-32 open-ended diesel hammer was used for the pile driving. The production piles were predrilled to 25 feet (8 m). The pile cushion thickness was 14 inches (356 mm). The stroke height started at 6 feet (1.83 m) or less. At this site, the soil has layers of silty sand (SM), light and dark greenish clay (CH), and fine sand with silt (SP-SM). The clay and sand have traces of cemented sand, phosphate, and/or decomposed shell fragments. Rebound was observed, but not recorded by the inspector; therefore, PDA data from this site was used to determine rebound of 0.75 inch (20 mm). No quake values were generated.

Figure 2.6 Aerial Map of SR 528 and US 441 site, Orlando, Florida (from Google Maps)

2.7.3 Intersection of Interstate 4 and US 192

Pile rebound was observed in Central Florida where Interstate 4 and US 192 intersect (Figure 2.7). The approximate ground surface elevation along the site is 91 ft (28 m). The support piling for the bridge piers consisted of 24 inch (610 mm) square PCPs 95 ft (38 m) long. An ICE 120 S single acting diesel hammer with a rated energy of 120 ft-kips (163 kJ) was used at this location. The hammer stroke used was approximately 6.25 feet (1.91 m). The pile cushion consisted of 9 inch (230 mm) thick plywood. Pile installations began by first preforming to a depth of 25 feet (7.6 m) and followed driving. Many of the piles with tip elevations between +5 and +25 feet (1.52 to 7.62 m) experienced rebound between 0.50 and 1 inch (12 to 25 mm) per hammer blow. PDA equipment was used along with CAPWAP force matching software to produce toe quake values ranging from 0.2 to 0.9 inches (5 to 22.5 mm).

The soil profile includes brown fine sand (SP), fine sand with silt (SP-SM) and traces of sandstone, silty fine sand (SM) with abundant shell and traces of phosphate, gray sandy lean clay (CL). Rebound occurred when the piles encountered medium dense gray fine sand with silt (SM-SP).

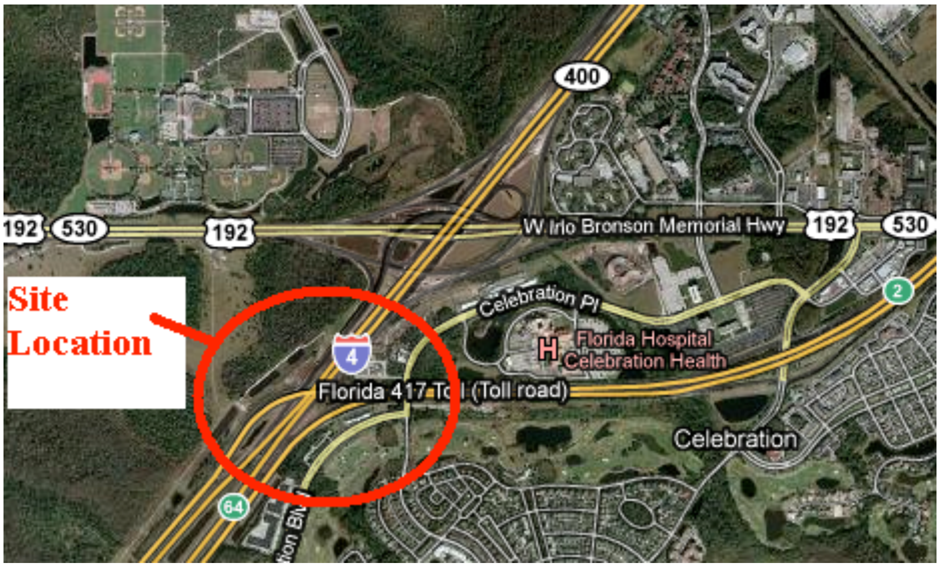


Figure 2.7 Aerial Map of I 4 and US 192, Orlando, Florida (from Google Maps)

2.7.4 Intersection of Interstate 10 and Chaffee Road

This construction took place near Jacksonville, Florida where Interstate 10 intersects Chaffee Road as shown in Figure 2.8. The ground surface elevation was approximately +64 ft (19.5 m). The piles used to support the bridge piers for Chaffee Road are 18 inch (460 mm) square PCPs with a length of 89 ft (27 m). A Pileco D36-32 diesel hammer was used to drive the piles. This hammer had an energy of 84 ft-kips (114 kJ) and a ram weight of 8 kips (35.6 kN). The pile cushion material was plywood with a thickness of 12 inches (305 mm). Pier 2, pile 9 had a very large rebound of 2 to 3 inches (50 to 80 mm). A CAPWAP analysis showed that the pile experienced 0.5 inch (12.5 mm) of toe quake at a penetration of 79 ft (24.1 m).

The soil at this site consists of dark brown fine sand (SP), gray-green fine sand with clay (SP-SC), clayey fine sand (SC), slightly sandy to sandy clay (CH), and gray-green clayey fine sand (SC) with trace of shell fragments. Records are unclear as to whether most of the rebound soil was medium dense green clayey fine sand (SC) to fine sand with Clay (SP-SC) or Gray Calcareous Limestone.

Figure 2.8 Aerial Map of I 10 at Chaffee Road, Jacksonville, Florida (from Google Maps)

2.8 Summary of FDOT Case Study High Pile Rebound Sites

Table 2.7 is a summary of the critical pile driving and soil data from the four case study. It shows that large displacement piles driven by single acting (i.e., open ended) hammers can rebound excessively when either fine sands or high plasticity clays are encountered. The elevations of the rebound soils in the Orlando and Jacksonville areas coincide with the top of the Hawthorn Group elevation reported by Scott (1982), Scott (1990), Scott (1992), Scott (2001).

Description and Location	Pile Shape and Area (in^2)	Pile Length (ft)	Hammer Manufacturer Model and Type	Rebound (in)	Toe Quake (in)	Rebound Layer Soil Description	Rebound Layer Elevation (ft)	Approximate Hawthorn Layer Elevation (ft)
SR 50 and SR 436 Orlando, Florida	2-ft Square PCP 576-in^2	105	APE D62-42 Single Acting Diesel	0.4 to 0.6	N/A	Light Green Clay Trace of Phosphate	15.6	0 to 50
SR 528 and US 441 Orlando, Florida	1.5-ft Square PCP 324-in^2	85	APE D30-32 Single Acting Diesel	0.75	N/A	Light Green Clay Trace of Phosphate	10 to 14	0 to 50
I-4 and US 192 Orlando, Florida	2-ft Square PCP 576-in^2	95	ICE 120 S Single Acting Diesel	0.5 to 1	0.2 to 0.9	Medium Dense Gray Fine Sand with Silt (SM SP)	5 to25	0 to 50
I-10 and Chaffee Road, Jacksonville, Florida	1.5-ft Square PCP 324-in^2	89	Pileco D36-32 Single Acting Diesel	2 to 3	0.5	Medium Dense Gray-Green Clay Fine Sand (SC) to Fine Sand with Clay (SP-SC) to Greenish Gray Calcareous Limestone	-15	0 to - 50
N/A = not avalaible								

Table 2.7 High Pile Rebound Summary from FDOT Case Study Sites

In summary, FDOT high pile rebound projects consistently show the following:

- Piles were large displacement piles;
- Soils in the rebound layers contained silts and clays;
- Soils in the rebound zone were considered at least medium dense or stiff;
- Both high quake and high rebound occurred when reported;
- Piles were longer than 85 feet;
- Pile driving hammers were single acting diesel;
- Piles rebounded near the top elevation of the Hawthorn Group;
- Rebound of at least 0.5 inches was observed.

2.9 Description of the Retesting Sites

Three sites in Florida were chosen for the case study and research phase retesting: two in FDOT District 5 in the Orlando area and one in FDOT District 3, of the Florida Panhandle. These sites were chosen because high pile rebound was recorded during installation. During construction, pile driving was monitored with PDA equipment, and the sites were accessible for mobilizing field testing equipment that would allow the research team to further evaluate the soils. The sites chosen for retesting in Orlando were: 1) Anderson Street Overpass 2) John Young and the site in the Florida Panhandle was 3) Ramsey Branch Bridge (Figure 1.1).

2.9.1 Anderson Street Overpass Description - FDOT District 5

The Anderson Street Overpass is located in downtown Orlando, Florida and is part of the I-4, SR 408 interchange. Figure 2.9shows an aerial photo of the general location of Anderson Street prior to construction of the overpass.

The overpass bridge consists of two end bents and six piers. The pile groups consist of between 12 and 16 piles for each pier. The majority of piles were 24 inch (610 mm) square PCPs. At the end of initial driving, the contractors and engineers encountered problems with pile rebound at Pier 2 on the west end of the bridge, but after allowing the piles to "set-up," the required capacities were achieved. Severe high pile rebound problems occurred during installation of the displacement piles at Pier 6 located on the east end of the overpass, causing the foundations to be redesigned using low displacement steel H-piles (HP 14 x 89). Rebound occurred only during installation of the concrete piles.

Figure 2.9 Preconstruction Aerial Map of Anderson Street in downtown Orlando, Florida (from Google Maps)

Prior to switching to H-piles, a 24 inch (610 mm) 124 ft (38 m) long square PCP test pile at Pier 6 was evaluated. The remaining 24 inch (610 mm) square PCP's were 110 ft (34 m) long. During PCP pile installation, preforming was conducted to 25 feet (7.6 m) and then the piles were driven to the desired elevation. A Delmag D62 diesel hammer with a rated energy of 90 ft-kips (122 kJ), was used for driving. When the test pile was driven, plywood cushions of either 12 inches (300 mm) or 16 inches (410 mm) were used. However, when installing the remaining production piles, a 12 inch (300 mm) plywood cushion was used. The HP 14 x 89 steel piles were 120 ft (37 m) long. A different diesel hammer, ICE I-30, with a rated energy of 71 ft-kips (96.3 kJ), was used for the installation of these piles.

During installation of the test pile (i.e., Pile 6, Pier 6), the inspector reported that high pile rebound between 0.5 inch and 1 inch (12.5 to 25.4 mm) occurred. The elevations, determined using PDA data from Appendix A, indicated the high pile rebound began at elevation 28 ft (8.5 m) and continued until the end of driving at elevation -7 feet (- 2.1 m). The ground surface elevation at this site was approximately 104 ft (31.7 m).

Figure 2.10, a generalized soil profile of Anderson Street Overpass, was developed from the design phase soil boring AS-103, the details of which are included in Appendix A. The raw N-values and the upper limit of the rebound zone were included in this figure. It shows that the soils within the high pile rebound zone generally were greenish gray silty clayey fine sand (SM-SC), greenish gray or green silty clay (CH), and dark greenish gray clayey fine sand (SC). The bottom soil layer in this high pile rebound zone was brown or greenish gray consolidated clay (CH) with a trace of phosphate and shell. This high pile rebound zone elevation of 28 ft (8.5 m) falls within the estimated elevation range from 0 to 50 ft (15.2 m) for the top of the Hawthorn Group (according to Figure 2.4) and its description matches the geologic soil description.

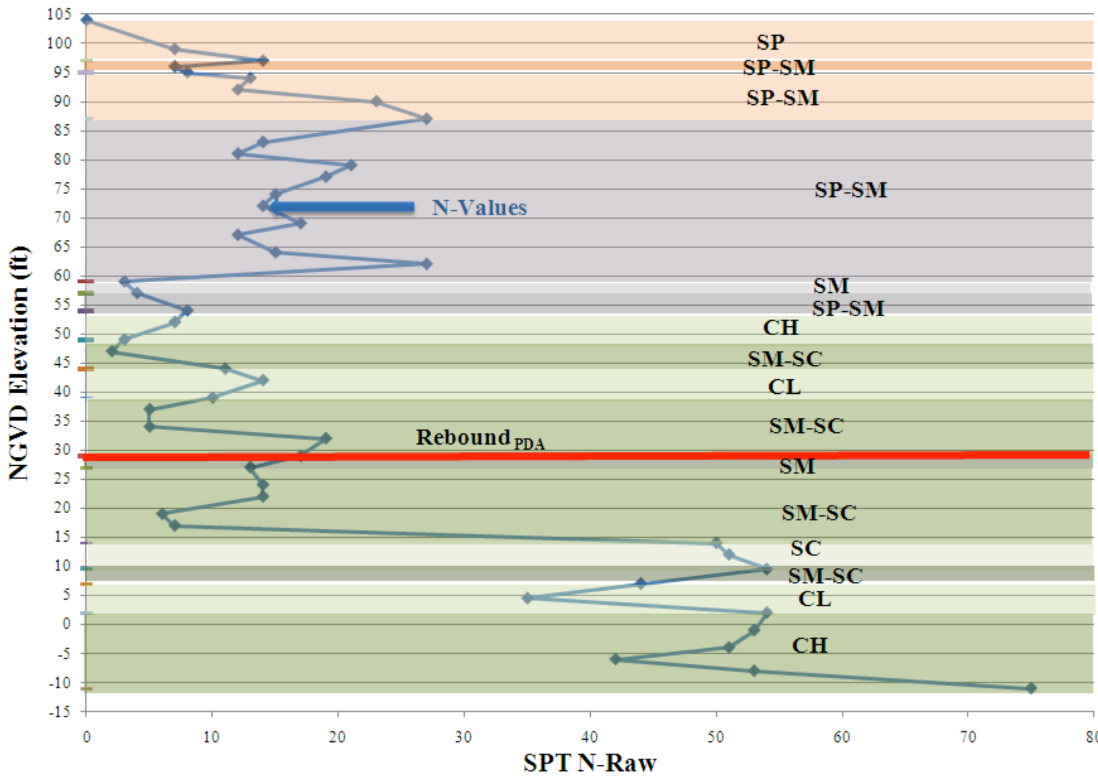


Figure 2.10 Anderson Street Overpass Soil Generalized Profile at Pier 6

2.9.2 John Young Description - FDOT District 5

The Interstate 4 overpass for John Young Parkway, located approximately three miles from downtown Orlando, was completed in 2006 (Figure 2.11). The ground surface elevation below the overpass is approximately 98 ft (30 m) and approximately 119 ft (36.3 m) along the peak of the I-4 overpass. The piles consist of 24 inch (610 mm) square PCPs, which varied in length depending upon where they were located along the access ramp and the bridge overpass elevation. The test piles measured 125 ft (38 m), and the production piles measured 110 ft, 91 ft, 73 ft, and 72 ft (35.5, 27.7, 22.3, and 22.0 m). The hammer used for the driving was an ICE 120S diesel single acting hammer with a rated energy of 120 ft-kips (163 kJ) and the plywood pile cushion was 11 inches (280 mm) thick.

Figure 2.11 Aerial Map of John Young at I-4, Orlando, Florida (from Google Maps)

The inspector recorded high pile rebound in the pile driving logs of several piles. When installing Pile 1 at end bent 2, the rebound was over 0.5 inch (12 mm) at a depth of 62 ft (elevation +36 ft, 11 m). The inspector recorded the maximum rebound of 1 inch (25 mm) in Pile 1, Pier 3. This pile had a length of 73 ft (28 m) and rebounded at a depth of 63 ft (elevation +35 ft, 11 m). According to the PDA data in Appendix B, the rebound layer at this site approximately at elevation 42 ft (12.8 m).

Figure 2.12, a generalized soil profile for this site, was developed from soil boring 04-S-40 near End Bent 2 of the westbound direction (See Appendix B). The figure includes the elevation of the rebound zone and the raw N-values. This profile was based on a ground surface of 105 feet (32.0 m), which was used since about 20 feet of fill was placed for the overpass areas so that their elevations began at 119 feet (36.3 m). The soils varied between silty fine sands (SM) and clayey fine sands (SC), and included thin layers of organic fine sand (PT) and sandy fat clay (CH). The soil colors changed from brown to light green near elevation 30 ft (9.1 m). The rebound zone, at elevation 42 ft (- 12.8 m), is within the estimated elevation range from 0 to 50 ft (15.2 m) shown in Figure 2.4 and its description matched the description of soils at the top of the Hawthorn Group.

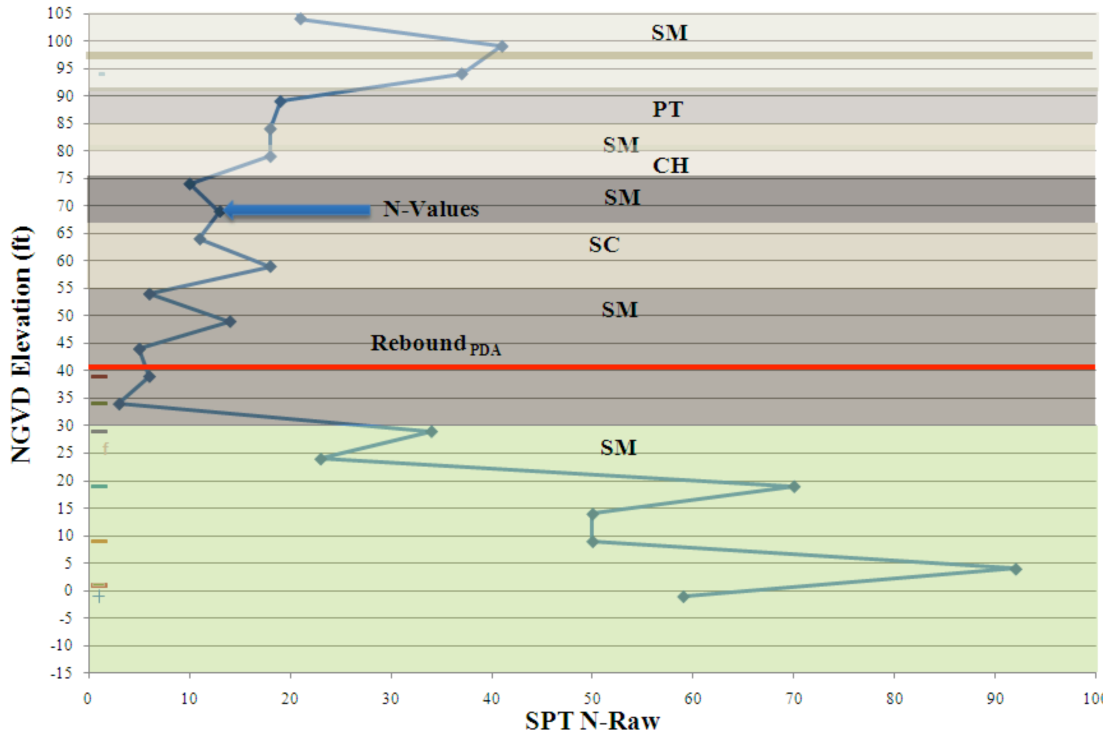


Figure 2.12 Generalized John Young Soil Profile from Borings near End Bent 2 Westbound

2.9.3 Ramsey Branch Bridge Description -FDOT District 3

The Ramsey Branch Bridge site is located in the Florida Panhandle in the town of Freeport. The bridge was constructed in 1996 on State Road 83 (US 331) to cross a waterway near Ramsey Branch as shown in Figure 2.13. Three test borings were conducted in 1990 to provide geotechnical information for the design (See Appendix C). In 2004, three additional test borings, also in Appendix C, were conducted in anticipation of a proposed roadway widening, which was not completed prior to completion of this report. These boring alignments were approximately 100 feet (30.5 m) west of the existing bridge.

Eighteen inch (460 mm) square PCPs were designed to support this bridge. The ground surface elevation was approximately 7 ft (2.1 m). Records indicate that a diesel hammer was used for pile installation and the length of the piles was 79 ft (24 m) with an area of 324 in^2 (0.209 m^2). High pile rebound, which was recorded using PDA equipment in nine piles, occurred between elevations -22 ft and -66 ft (-6.7 to -19.8 m). Rebound as high as 2 inches occurred in four of these piles. The PDA data from the nine piles, in the form of PDA rebound versus elevation plots on a single graph, is included in Appendix C.

A generalized soil profile was developed using borings near End Bent 1 (Figure 2.14). In addition to the raw N-values, the profile shows an upper elevation associated with high pile rebound of -22 ft (6.7 m). The soils in the high pile rebound zone include greenish gray clayey sand (SC), and greenish gray silty sand (SP-SM). A trace of shell was found throughout soils in the high pile rebound zone.

Figure 2.13 Aerial View of Ramsey Branch Bridge at Freeport, Florida (from Google Maps)

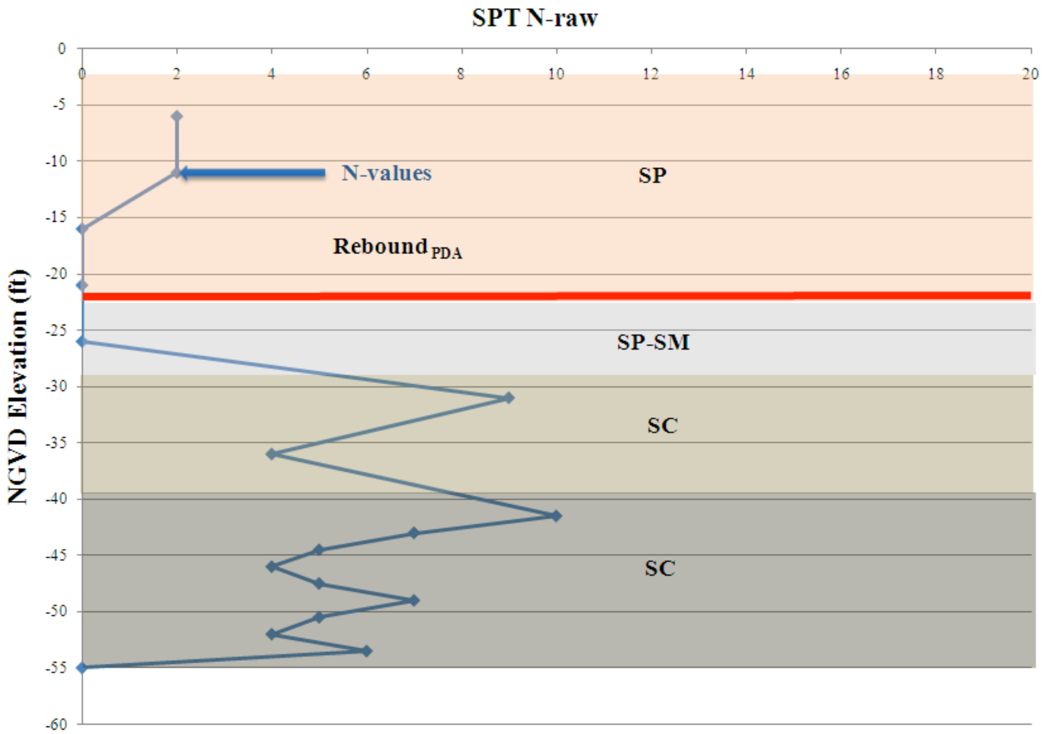


Figure 2.14 Generalized Ramsey Branch Bridge Soil Profile from Borings near End Bent 1

2.10 Research Phase Field Testing Locations

Retesting for all sites included SPT borings, CPT, PPMT, and DMT Soundings, plus borings for retrieving thin walled tube samples of minimum disturbance. The objective of the retesting program was to provide geotechnical data, which could be evaluated versus elevation for comparison to the reported pile rebound soils.

All retesting was conducted in the immediate vicinity of the pile that displayed high rebound for the site. For the Anderson Street Overpass, the retesting was centered on Pile 6, Pier 6 near Station 4013+00 as shown in Figure 2.15. For John Young, retesting centered around Pile 6, End Bent 2 on the westbound section near Station 140+00 as shown in Figure 2.16. For Ramsey Branch Bridge, retesting centered on Station 427+58 near End Bent 1 as shown in Figure 2.17.

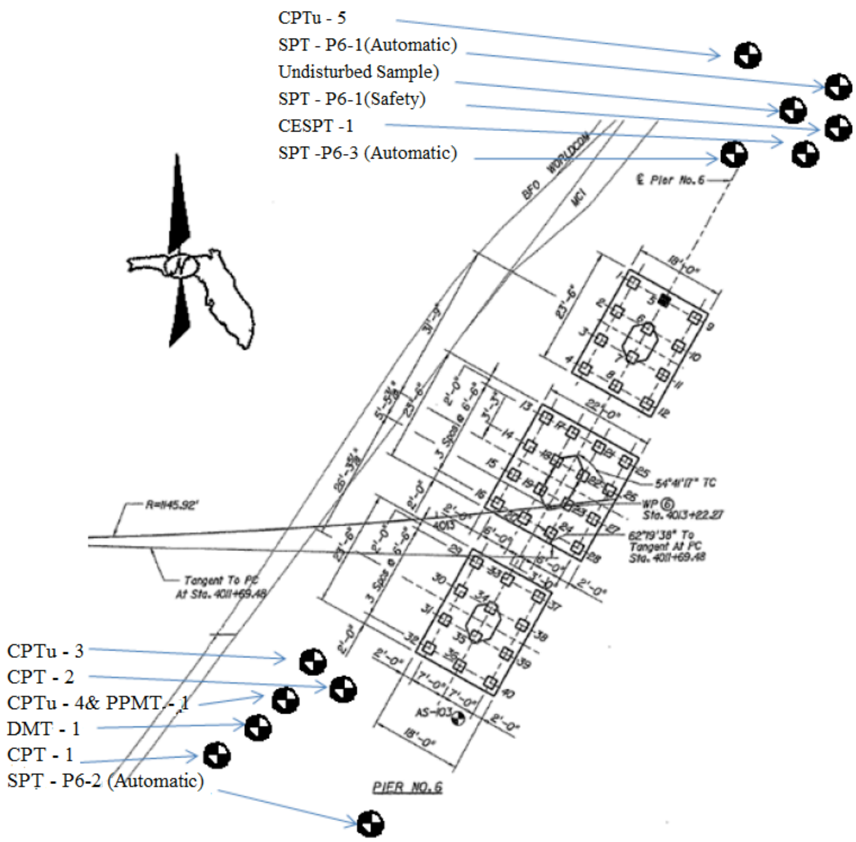


Figure 2.15 Field Retesting Boring Locations for Anderson Street Overpass near Pile #6 Pier 6

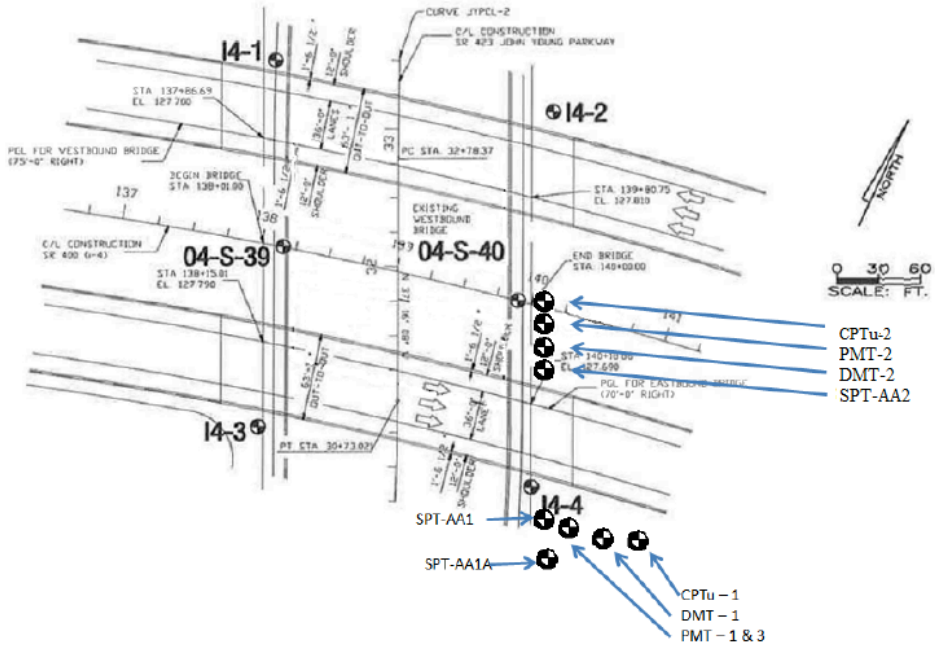


Figure 2.16 Field Retesting Boring Locations for John Young near Pile #6 in End Bent 2 Westbound

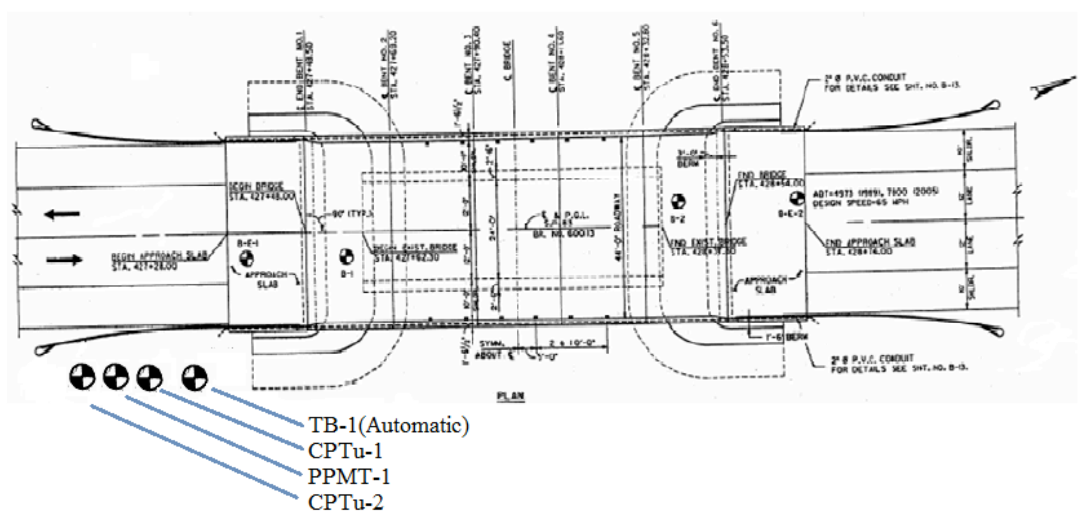


Figure 2.17 Field Retesting Boring Locations for Ramsey Branch Bridge near End Bent 1

3. Field and Laboratory Testing and Sampling Procedures

3.1 Research Phase Sampling Procedures

Standard testing procedures were followed, when available, for all tests conducted during the research phase.

3.1.1 Disturbed Split-Barrel Samples

Disturbed samples were collected from the material recovered from split-barrel samples, according to the procedures specified by AASHTO T 206 "Penetration Test and Split-Barrel Sampling of Soils." These samples were used to determine water contents, soil classifications and to estimate the unconfined compressive strength from pocket penetrometer testing.

The samples from the Anderson Street Overpass and John Young sites were divided and sent to three testing laboratories, Florida Tech, FDOT SMO, and Ardaman & Associates, for water content and classification testing. Samples from Ramsey Branch Bridge were collected and tested for water content and classification by the personnel from the FDOT District 3 Geotechnical Lab.

3.1.2 Thin Walled Tube Samples

Thin walled tube samples were collected according to the procedures specified in ASTM D 1587, from the retesting sites. The majority of the samples were collected at depths within the rebound zone. Two borings were conducted at both the Anderson Street Overpass and John Young sites while one boring was performed at the Ramsey Branch Bridge site. The samples were collected using the procedure outlined by AASHTO T 207 and then transferred to FDOT SMO for testing. These samples were taken to allow strength, stiffness and permeability testing to be performed.

3.1.3 Standard Penetration Testing (SPT)

3.1.3.1 SPT Procedure

The SPT test procedure followed was ASTM D 1586 "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils" (or AASHTO T 206). All samples were visually classified to include color from the Munsell Color Chart Munsell: 1971, moisture and general soil type descriptions. Instrumented SPT tests and split spoon sampling were performed continuously to a depth of 5 ft, followed by SPT tests at two foot centers, and then continuous instrumented SPT and split spoon sampling within the high rebound soils and until boring termination. The drill rods at the drive head, used for SPT borings, were equipped with accelerometers and strain gages to allow PDA software to measure actual hammer energy transferred to the rods during each hammer blow.

SPT blow counts were recorded over the entire 24-inch length of the spit spoon sampler to produce larger samples for laboratory testing. Only the blow counts associated with the middle 12-inches were used for determining the N-values. SPT tests and sampling were also conducted at any change of stratum. The N-values were correlated with the soil properties encountered at each site.

Soil samples were taken from the split spoon at each depth and stored in both jars and bags. These samples were tested by researchers from Florida Tech, Ardaman & Associates, and the FDOT SMO in their laboratories. The borings were continued to a depth approximately equal to the length of the piles associated with the test site.

Two different hammers, automatic and safety, were used at the central Florida sites. Two or three borings were performed using the automatic hammer and one using the safety hammer (**Error! Reference source not found.**). The purpose of using both safety and automatic hammers is to compare the SPT results. Both hammers were used because the original (i.e., design phase) borings were conducted using safety hammers. Testing at Ramsey Branch Bridge location was limited to one test for the following reasons: 1) design phase borings were conducted with an automatic hammer and 2) there was insufficient drilling space for multiple borings. **Error! Reference source not found.** is a summary of the SPT tests conducted at each site including the type of hammer used.

Site	Hammer Type	Number of Borings
Anderson Street Overpass	Safety	1
	Automatic	3
John Young	Safety	1
	Automatic	2
Ramsey Branch Bridge	Automatic	1

Table 3.1 Summary of Hammer Type and Number of SPT Borings

3.1.3.2 SPT Data Reduction

The raw SPT data was corrected using Equation 1 to account for variations in testing procedures following the recommendations presented by Skempton (1986). Corrections for hammer efficiency were based on use of a safety hammer with 60 percent efficiency. This efficiency was chosen such that the results could be applied by geotechnical engineers throughout Florida with the assumption that not all drill rigs are required to be equipped with the more efficient automatic hammers. Corrections for borehole diameter and sampler type were 1.00 because the hole diameter was between 2.5 and 4.5 inches and a standard split spoon sampler was used. The rod length correction varied between 0.75 and 1.00 as the total length of the rods increased from 10 to more than 30 feet.

$$N_{60} = \frac{E_m C_B C_s C_R N}{0.60} \tag{1}$$

$where:$

$N = \text{measured SPT N} - \text{value}$

$E_m = \text{hammer efficiency}$

$C_B = \text{borehole diameter correction}$

$C_s = \text{sampler correction}$

$C_R = \text{rod length correction}$

Liao and Whitman (1986) also recommend that N-value be corrected to account for overburden stresses. Since deep tests in dense sand would produce higher N-values than shallow tests in this same dense sand, Liao and Whitman (1986) recommend that all N-values be corrected to a standard overburden stress of 1 atm (i.e., 100 kPa or 2000 psf). The basic relationship between N_{60} and $N_{1(60)}$ in granular soils is shown in Equation 2.

$$(N_1)_{60} = N_{60}\sqrt{\frac{p_a}{\sigma'_{vo}}} \tag{2}$$

where:

N_{60} = Corrected SPT N – value

p_a = atmospheric pressure

σ'_{vo} = effective overburden pressure corresponding to the depth for the SPT data

3.1.4 Pocket Penetrometer Testing

Pocket Penetrometer tests were conducted on cohesive split spoon samples once they were brought to the surface and before they were placed in a sample jar. The spring loaded piston type device, with a spring constant of 12 pounds per inch, was used on saturated split spoon and thin walled tube samples to evaluate consistency and estimate the unconfined compression strength. Currently no standard method exists from ASTM or AASHTO for conducting this test; therefore, the procedure outlined by Soil Test was followed. A test was performed by first, sliding the red metal ring located on the piston to the lowest reading on the scale, then gripping the knurled handle and pushing the piston using steady pressure into the soil up to the calibration or machined groove, which is about 1/4 inch (6 mm) from the end, and finally reading the unconfined compressive strength in tons per square foot or kilograms per square meter on the instrument. The red indicator sleeve automatically remains in position for the reading. Several tests were performed on the least disturbed portion of the split spoon samples and an average was determined.

3.1.5 Electrical Cone Penetrometer Testing

Electrical friction cone tests were performed in soundings by hydraulically advancing the cone penetrometer while signals were recorded digitally using Hogentogler standard recording system. FDOT SMO personnel conducted all CPT soundings using their CPT rig following the procedures outlined in ASTM D 5778 "Electronic Friction Cone and Piezocone Penetration Testing." During testing, digital channels were used to record the tip, friction, inclination and pore water pressure every 2 inches or 5 cm. The rod insertion speed was 0.75 in/sec or 1.9 cm/sec. Soundings were terminated when high tip or high thrust readings occurred. CPT tests that included pore water readings were designated CPT_u tests, while tests without pore pressure data were designated simply as CPT tests. During CPT_u tests the cone point was paused at desired elevations to enable field personnel to record pore water pressure dissipation versus time data. Tests were conducted until refusal was met by the CPT rig. Table 3.3 shows a summary of the CPT tests. This data was used to delineate the soil layers, evaluate the changes in point and sleeve friction and to attempt to understand the effects of the pore water pressures both in the rebound and non-rebound soils.

Site	Type	Number of Soundings
Anderson Street Overpass	CPT	2
	CPT_u	3
John Young	CPT_u	2
Ramsey Branch Bridge	CPT_u	2

Table 3.3 Summary of the Number of CPT and CPTu Soundings

3.1.6 Flat Blade Dilatometer Testing

Dilatometer soundings were conducted at all three sites using the rig and personnel from FDOT SMO following the procedures outlined in ASTM D 6635 "Test Method for Performing Flat Plate Dilatometer"' The tests for the "A" and "B" parameters (i.e., membrane lift off pressure and pressure when membrane has expanded 1.1 mm) were performed every 20 cm (8 in.) to penetration refusal depth. The "C" readings, used to estimate pore water pressures, were obtained approximately every meter. Each test sequence required approximately 2 minutes.

Table 3.4 is a summary of the number of soundings per site and the total number of DMT tests. Due to space constraints, there were no DMT soundings conducted at Ramsey Branch Bridge. This test produced elastic moduli, lift off pressures and pore water pressures that could be compared to similar parameters from other tests to help identify high rebound soils.

Site	Number of Soundings	Number of Tests
Anderson Street Overpass	1	120
John Young	2	230

Table 3.4 Summary of the Number of DMT Soundings and Tests

3.1.7 PPMT Testing

PPMT tests were conducted at all three sites to provide complete in situ stress-strain information at various depths within the soil profiles. This information would enable estimates of stiffness (i.e., elastic moduli), ultimate strengths (known as limit pressures), and initial or lift off pressures. These geotechnical parameters could be compared to similar values from other testing to help identify high rebound soils.

PPMT testing, which is a smaller version of the standard 3-inch diameter (75 mm) 18-inch (450 mm) long unit, allows the 1.35 inch (33 mm) diameter probe with a cone point to be pushed hydraulically using the CPT rods and hydraulic equipment to the desired test depths. An instrumented PPMT control unit was used for all testing. This unit includes calibrated digital pressure and volume equipment that interfaces with a special software package known as APMT, for Automated Pressuremeter. Once at the desired depth, 10 minute tests were performed following the procedures proposed to FDOT by Cosentino et al. (2006) for the PENCEL Pressuremeter. This procedure calls for a strain controlled test to completely define the stress-strain behavior of the soil at the desired test depth. During tests, 5 cm^3 volume increments of water were injected into the probe and the corresponding pressures in kPa were recorded after system stabilization. Corrections for the volume or system expansion, the membrane resistance and the hydrostatic pressure were applied to the raw data to produce a stress-strain curve similar to that shown in Figure 3.1.

Each PPMT test included a complete loading curve with one unload-reload cycle plus several unloading points once the maximum volume was injected. By including an unload-reload loop and several unloading points in this manner, elastic moduli can be determined at numerous points along the curve. Elastic moduli corresponding to relatively large strains were determined from the initial slope and the reload slope of the unload-reload portion of the test, while elastic moduli associated with smaller strains were determined from each unloading point at the maximum volume. The result of this analysis was a series of elastic moduli and associated strains to enable evaluation of the variation of elastic moduli with strain.

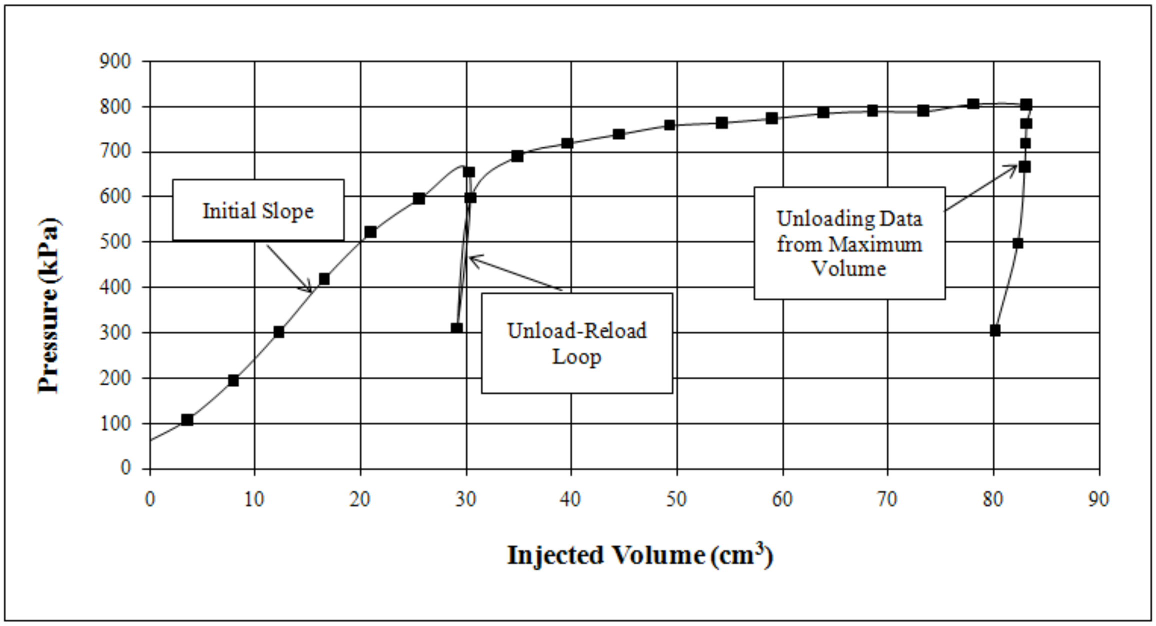


Figure 3.1 Typical PPMT Results with Unloading Data at Maximum Injected Volume

PPMT tests were conducted at all three retesting sites. At the Anderson Street Overpass site, the research team limited the PPMT testing to the rebound soils. The cone rig was not able to hydraulically push the probe to multiple test depths; therefore, only one PPMT test was conducted at a depth of 92.5 feet (28.2 m) or elevation 11.5 ft (3.5 m) in one sounding. Three soundings were conducted at the John Young site. Two were conducted by hydraulically pushing the PPMT probe to the desired test depths and the third was conducted by inserting the PPMT probe into a prebored hole. For the first two soundings at John Young, pushed PPMT tests were conducted every three meters until refusal to produce 18 tests or nine per sounding at between elevations 88.2 and 9.4 feet (26.9 and

2.8 m). The PPMT testing at the third sounding at John Young was conducted within the rebound soils by pre-drilling using the FDOT SMO drill rig and then inserting the PENCEL probe to the test depth. This process produced five tests between elevations 11 and - 4.5 feet (3.3 and -1.4 m). By first inserting the Cone rods to the required depth, next removing them, and then reinserting the PPMT down the hole left by the CPT equipment, seven PPMT tests were conducted at the Ramsey Branch Bridge site between elevations -11.4 and - 57.0 feet (-3.5 and -17.4 m). Table 3.5 is a summary of the number and elevation of PPMT tests conducted at the three retesting sites.

Anderson Street Overpass	John Young			Ramsey Branch Bridge
Sounding 1	Sounding 1	Sounding 2	Sounding 3	Sounding 1
Elevation (ft)	Elevation (ft)	Elevation (ft)	Elevation (ft)	Elevation (ft)
11.5	88.2	88.2	11.0	-11.4
	78.3	78.3	7.0	-40.6
	68.5	68.5	1.0	-43.9
	58.6	58.6	-1.0	-47.1
	48.8	48.8	-4.5	-50.4
	38.9	38.9		-53.7
	29.1	29.1		-57.0
	19.3	19.3		
	9.4	9.4		

Table 3.5 Summary of PPMT Testing Elevations

3.2 Laboratory Testing Procedures

The soil samples from these three sites were tested at Florida Tech laboratories in Melbourne, FL, at FDOT State Materials Office (SMO) laboratories in Gainesville, FL, and Ardaman & Associates laboratories in Orlando, FL. A combination of geotechnical engineering graduate students and laboratory technicians performed these tests. Florida Tech graduate research assistants performed the laboratory test for the high pile rebound soil layers at Anderson Street Overpass and John Young. FDOT laboratory personnel tested the remainder of the soils of these sites plus the Ramsey Branch Bridge soils. Ardaman & Associates technicians tested as set of samples from the Central Florida sites, to produce boring logs for the two sites. All laboratory procedures followed were from FDOT Florida Sampling and Testing Methods (FDOT, 1994).

3.2.1 Natural Moisture Content

To aid in the description and classification of the laboratory samples, moisture contents were determined. Representative samples obtained from either split spoon or thin wall tube samplers from each site were placed in containers, weighed and then oven dried to determine the natural moisture content according to the procedures specified in AASHTO T 265. The samples were oven dried for 24 hours, at 105 C. Moisture contents were determined by dividing the weight of water in each sample by the corresponding weight of dry soil and expressing the value in percent. The weight of water was found by subtracting the oven dried sample weight from the initial sample weight.

3.2.2 Atterberg Limits

The Atterberg limits were conducted on samples with sufficient amounts of plastic fines to enable plasticity behavior to be evaluated. The Florida Sampling and Testing Method designation FM 1-T 089 and 090 (AASHTO T89-86 and T90-87) was used for both the liquid and plastic limits. Samples were first air dried and then sieved through the #40 sieve. To avoid mineral interaction, de-mineralized water was added to the samples to perform these tests. The Casagrande liquid limit device and grooving tool were used in the determination of liquid limits (LL). After the liquid limit test, the same soil was used for the plastic limit (PL) test. After the liquid and plastic limits were determined, the plasticity index $(PI = W_L - W_P)$ was calculated.

3.2.3 Sieve and Hydrometer Testing

Because the soil samples contained a high percent of material passing the #200 sieve, a combined gradation analysis was performed using both sieves and the hydrometer. This information was combined with the Atterberg limits data such that the soil composition could be thoroughly described. Both portions of this analysis were conducted following the Florida Sampling and Testing Method FM 1-T 088 or AASHTO T 88 "Particle Size Analysis of Soils." The sieve analysis was used to determine the gain size distribution of material larger than the #200 sieve and the hydrometer for grain size distribution of the fines. The sieve analysis was performed using air dried soil samples with the following U.S. standard sieve sizes: 3/8 inch, #4, #10, #40, #60, #100, #200.

As per FM 1-T 088, approximately 50 g of soil passing the #40 sieve was used for the hydrometer tests and 10 g of this sample was used to determine the hygroscopic moisture content. De-mineralized water was used to avoid mineral interaction. A standard sodium hexametaphosphate solution was used to disperse the soil for at least 12 hours. Readings were taken according to the FM 1-T 088 standard with the final hydrometer reading taken after 24 hours.

3.2.4 Flexible Wall Permeability Testing

Samples selected for flexible wall permeability testing were tested according to the procedure specified by ASTM D 5084. After consolidation for a minimum of 24 hours, FDOT SMO conducted a series of flexible wall permeability tests that typically included four to six individual measurements at a pressure differential of two psi between the sample top and bottom.

3.2.5 Consolidation Testing

One 1-D oedometer consolidation tests were performed on one sample per project location as per ASTM D 2435. In addition to determining C_c and C_r, the overconsolidation ratio (OCR) was determined to aid in conducting the CU triaxial testing.

3.2.6 Consolidated Undrained (CU) Triaxial and Flexible Wall Permeameter Testing

A combination of consolidation, permeability and triaxial shear tests was performed on the thin walled tube samples. Once the samples were extruded, placed in the triaxial cell and saturated, they were consolidated to enable C_c, C_r, the OCR ratio to be found. Following consolidation, the coefficient of permeability was determined as the samples were subjected to flexible wall permeability tests. Finally, the triaxial shear characteristics were determined from CU triaxial shear testing. These shear tests provided elastic moduli, failure stresses, stress ratios, cohesion and friction angles.

The sequence of steps followed during the CU testing, which was performed on a minimum of one sample from each Thin Walled Tube (i.e., Shelby Tube), is presented below.

1. The bottom of each tube was cut to produce a section approximately 7" to 8" in length.

2. Using a horizontal sawing system (i.e., band saw) these shortened sections of the tubes were cut longitudinally so that the sample could be extruded.

3. The extruded sample was trimmed to the final dimensions of 2.8" diameter x 5.6" length once it was placed in a split mold.

4. The sample was placed in the triaxial chamber to conduct the Consolidated Undrained (CU) triaxial test with pore pressure measurements as per ASTM D 4767 (or US Army Corps EM 1110-2-1906).

5. The sample was saturated using a combination of vacuum saturation and back pressure saturation methods.

6. A confining pressure was applied to the sample, using an effective overburden pressure, calculated by assuming a unit weight of approximately 100 to 110 pcf and water table noted from field logs.

7. Normally consolidated conditions were assumed based on the 1-D oedometer consolidation test results; therefore, the confining pressures used were based on the existing overburden pressures. If the samples had been overconsolidated the maximum past pressures could have been used for confining pressures

8. The samples were consolidated using the confining pressure as determined in step 6 using single drainage from the top of the sample while monitoring the pore water pressures at the bottom of the sample.

9. Volume measurements were measured using burette readings.

10. After consolidation (for a minimum of 24 hours), a series of flexible wall permeability tests were conducted per ASTM D 5084. Typically 4 to 6 individual measurements were made with a pressure differential of 2 psi between top and bottom of the sample.

11. After the permeability testing (normally the following day), each sample was sheared at strain rate based on recommendations in ASTM D 4767. FDOT District 5 requested that samples be sheared until a decrease in deviator stress occurred which sometimes occurred at over 20% strain. This process is different from the

recommendation from ASTM D 4767, which states that samples should be loaded to 15% strain unless the deviator stress has dropped by 20% of the maximum value.

12. If a second CU triaxial test from the same tube was performed, a slightly different confining pressure ($\pm$ 5 psi) was used to produce two points that were used to define the failure envelope.

4. Results

The evaluation of the data that existed before this research (i.e., from the design phase geotechnical investigations) is presented separately from the research testing data obtained during this retesting program. The terminology used to distinguish between the two will be 1) design phase and 2) research phase.

The appendix data was divided as shown in Table 4.1. Each appendix contains the soil borings and CPT soundings from the existing data, the borings and CPT, DMT, PPMT soundings from the research phase, the PDA data obtained during the existing data from each site and the laboratory testing data obtained from the research phase.

Appendix	Title
A.1	Anderson Street Overpass Design Phase Soil Borings and CPT Soundings
A.2	Anderson Street Overpass Retesting Phase Soil Borings and DMT, PPMT Soundings
A.3	Anderson Street Overpass Design Phase PDA Data
A.4	Anderson Street Overpass Retesting Phase Laboratory Test Data
B.1	John Young Design Phase Soi Borings and CPT Soundings
B.2	John Young Retesting Phase Soil Borings and DMT, PPMT Soundings
B.3	John Young Design Phase PDA Data
B.4	John Young Retesting Phase Laboratory Test Data
C.1	Ramsey Branch Bridge Design Phase Soil Borings and CPT Soundings
C.2	Ramsey Branch Retesting Phase Soil Borings and DMT, PPMT Soundings
C.3	Ramsey Branch Design Phase PDA Plots
C.4	Ramsey Branch Bridge Retesting Phase Laboratory Test Data

Note: Retesting Phase CPT Soundings included in Analysis Chapter

Table 4.1 Appendix Data Sections

The existing PDA data are presented first in order to establish limits of rebound and trends that clarify the approach used with the analysis of the geotechnical information.

4.1 Evaluation of Design Phase PDA Data

The design phase PDA data for two of the three FDOT Projects, John Young and Anderson Street Overpass, were analyzed to determine trends that might indicate high pile rebound. The design phase PDA data from Ramsey Branch Bridge did not include enough original data to generate the plots developed in this section and were not used for the analysis in this section. Ramsey Branch PDA results did indicate high pile rebound between 0.5 and 2 inches (12 to 50 mm) from the nine piles driven. According to the PDA data (See Appendix Figure C.7), this rebound occurred at elevation - 22 ft (-6.7 m) for three of the nine piles and varied to elevation -66 ft (20.1 m). The inspector driving information, which was transferred to one of the borings from the 1990 investigation (i.e., Boring B-3), indicated possible rebound at elevation -42 ft (-12.8 m).

The PDA software output from Anderson Street Overpass and John Young included the following key information:

- The depth or elevation of the pile corresponding to each hammer blow;
- The maximum displacement of the pile at the end of each time scale (i.e., DMX in the PDA output);
- The permanent set of the pile (i.e., Set in the PDA output input from the inspectors log); and
- The final displacement of the pile at the end of each time scale (i.e., DFN in the PDA output).

Plots were developed relating elevations to the various pile displacements from the data. The elevation at the start of the PDA data corresponded to the depth at which pile driving commenced and was below the ground surface because piles at both sites were set into predrilled holes.

The PDA data includes both Set and DFN that may lead to confusion about the rebound. The "Set" is based on an average pile penetration per foot of driving and is input into the PDA software from the field inspector's data sheets or logs, while "DFN" is reduced digital data from the PDA accelerometer mounted on the pile. The pile rebound is the difference between DMX and the Set; however, the average set per blow is not necessarily equal to the difference between the pile's maximum displacement and final displacement, because the digital signal recording time is approximately 200 milliseconds and the pile may move for nearly a second as a result of a hammer blow. Therefore, both the rebound based on the inspectors "Set" and the rebound based on the PDA data (i.e., DMX-DFN) produce different results. They are distinguished from each other as follows:

- The rebound based on inspectors data are denoted with the symbol $Rebound_{insp}$.
- The rebound based on PDA data are denoted with the symbol $Rebound_{PDA}$.

4.1.1 Design Phase PDA Rebound and DMX versus Elevation

Figure 4.1 and Figure 4.2 were developed to illustrate the variation of DMX and $Rebound_{PDA}$ per hammer blow versus elevation at the Anderson Street Overpass and John Young sites. The PDA data from Ramsey Branch Bridge only includes $Rebound_{PDA}$ over limited pile penetration depths and was therefore excluded from this section. By matching the PDA data in Appendices A.3 and B.3 and the soil profile from the closest boring from the existing data investigations, conclusions were developed. Figure 4.1 shows data from Pier 6 Pile 6, which is located near the closest design phase boring AS-103. Figure 4.2 shows data from End Bent 2 Pile 6 for the westbound lanes, which is located near the closest design phase boring 04-S-40. The dashed vertical line indicates the 0.25-inch (6 mm) maximum allowable rebound according to FDOT Road and Bridge Construction Specification, Section 455. Rebound in excess of 0.25 inches (6 mm) is referred as high rebound. Both plots show $Rebound_{PDA}$ significantly increased near the end of driving, and was greater than DMX. In both cases, rebound exceeds the allowable 0.25 inches (6 mm).

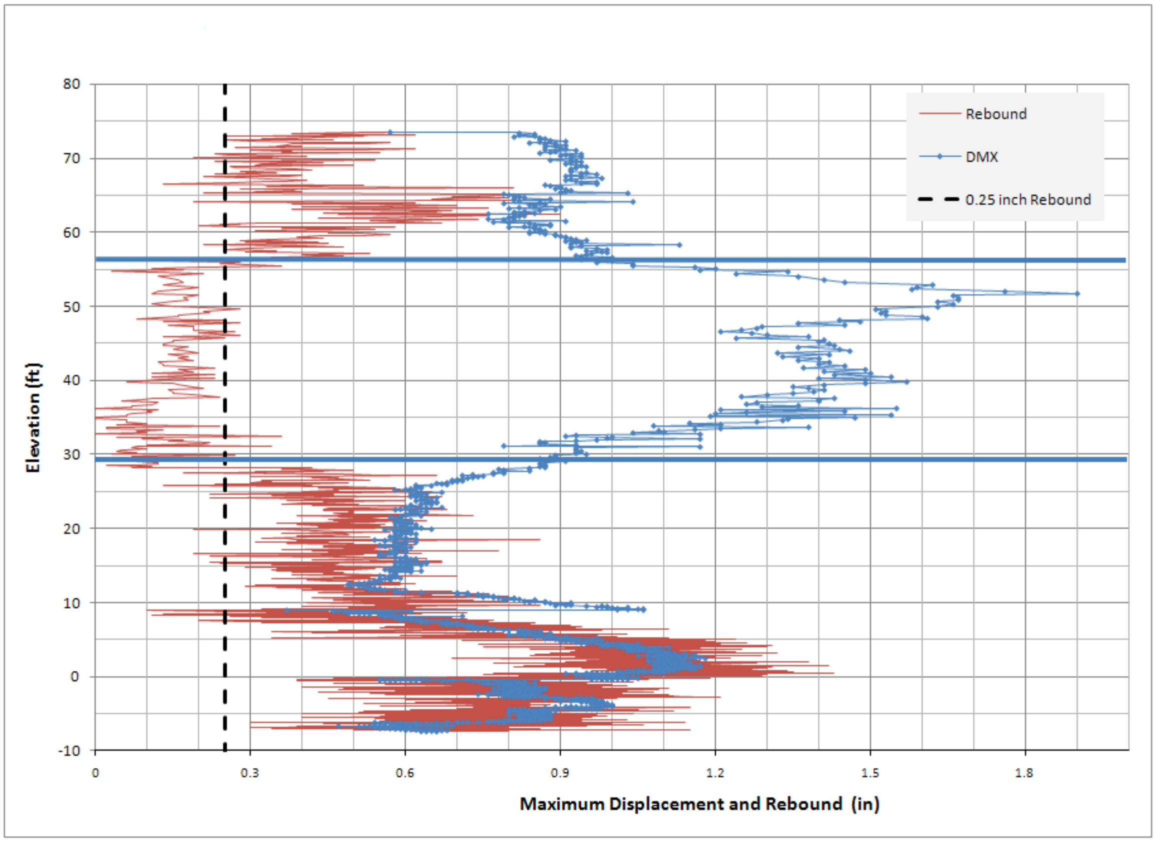


Figure 4.1 PDA Rebound and Maximum Displacement versus Elevation for Anderson Street Pile #6 in Pier 6

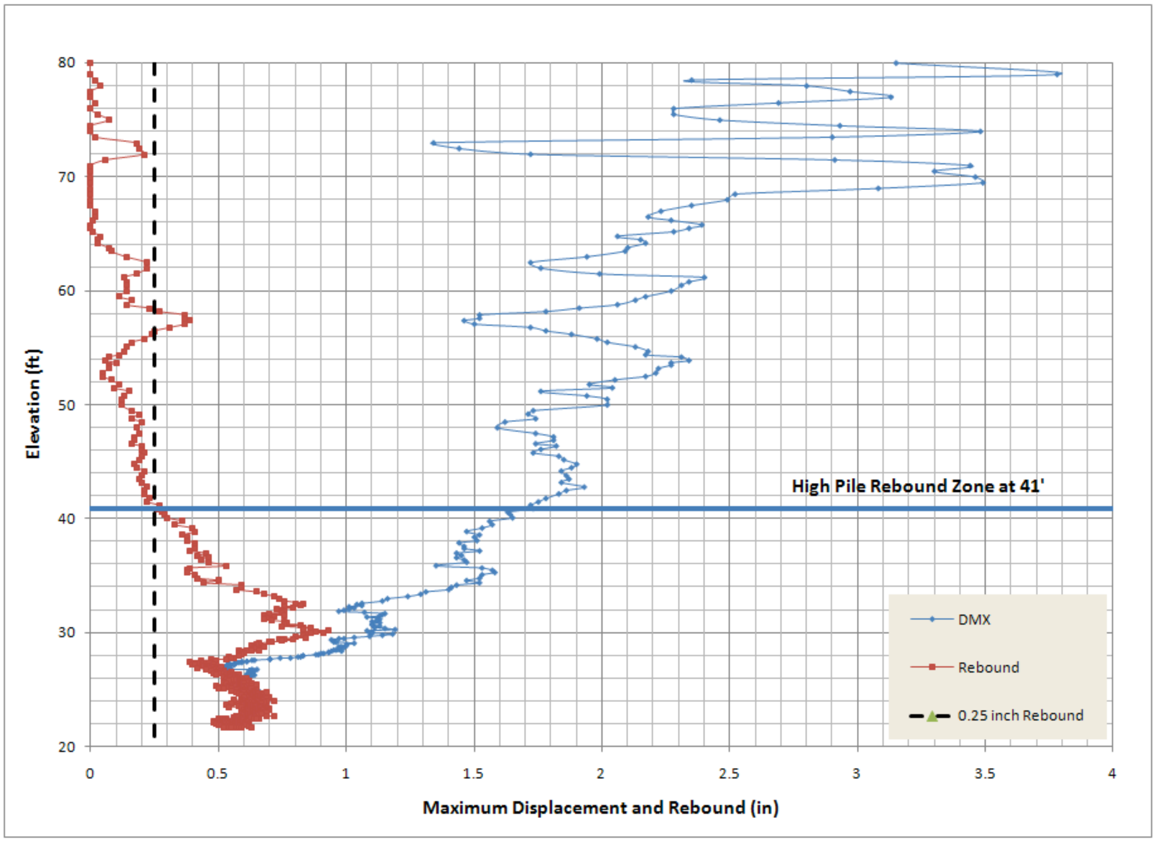


Figure 4.2 PDA Rebound and Maximum Displacement versus Elevation for John Young Pile #6 in End Bent 2 Westbound

The Anderson Street Overpass data shows high pile rebound in two zones. The upper high pile rebound zone extends from elevation 74 ft to 55 ft (22.5 and 16.7 m). In this upper zone $Rebound_{PDA}$ approaches DMX between elevations 60 and 65 ft (18.2 and 19.8 m). High pile rebound occurred between elevations 32 ft and 28 ft (9.7 and 8.5 m); however, DMX values were large enough such that continual pile penetration occurred. The lower high pile rebound zone, where $Rebound_{PDA}$ approached or exceeded DMX, begins at elevation 28 ft (8.5 m) and continues to the end of pile driving at elevation -7 ft (-2.1 m). The Anderson Street Overpass design phase boring AS-103, which is closest to the Pile 6 Pier 6 PDA data, contained medium dense dark green silty fine sand (SM) with trace phosphate at the lower rebound elevation.

Two high pile rebound zones also occurred at John Young. Data shows high pile rebound between elevations 58 and 56 ft (17.6 to 17.0 m); however, $Rebound_{PDA}$ values, did not approach DMX values, so this was not classified as a high rebound zone. The rebound increased between elevations 41 and 22 ft (12.5 to 6.7 m), which also corresponds to the end of the PDA data. DMX values again approach $Rebound_{PDA}$ values in this zone. The John Young design phase boring 04-S-40, contains medium dense brown silty fine sand (SM) with trace phosphate at the beginning of the rebound zone.

A comparison between the high rebound elevations, i.e., the lower rebound zones at elevation 28 feet (8.5 m) for Anderson Street Overpass and 41 feet (12.5 m) for John Young and the top of the Hawthorn Group whose elevations vary between 0 and 50 feet above sea level (See Figure 2.4), indicates a fairly close match. The two project sites are located approximately 3 miles apart.

4.1.2 Design Phase Final Displacement or Set and SPT N values versus Elevation

The final displacement (DFN) and SPT N-values were plotted against the elevation for both the Anderson Street Overpass and John Young sites. The PDA data from Ramsey Branch Bridge only includes $Rebound_{PDA}$ over limited pile penetration depths and was again excluded from this section. N-values from design phase boring AS-103 were used for the Anderson Street Overpass plot, and N-values from design phase boring 04-S-40 were used for the John Young plot. In general, the data for both sites are similar with slight variations at the rebound layer boundaries.

Figure 4.3 shows that DFN changes significantly at the high pile rebound boundaries. In the upper high pile rebound zone between elevation 74 and 55 feet (22.6 and 16.8 m), DFN decreases from about 0.5 inch (12.5 mm) at 65 ft (19.8 m) to about 0.25 (6 mm) between elevations 60 and 65 feet (18.3 and 19.8 m) and then increases to 0.5 inches (12.5 mm) below elevation 60 ft (18.3 m). At the boundary of upper elevation of high pile rebound near elevation 55 ft (16.8 m), DFN increases to 1.25 inches (37.5 mm) indicating that the pile driving is proceeding as desired. In the lower rebound zone below elevation 28 ft (8.5 m), DFN gradually decreases, approaching zero at an elevation of 9 feet (2.7 m). From this elevation until the end of driving, the PDA data indicated that the pile bounced above and below a DFN of zero. The distance between elevations 28 and 9 feet (8.5 and 2.7 m) was 19 feet (5.8 m), which corresponds to approximately 9.5 pile diameters into this rebound layer as shown in the figure.

The SPT N-values, which were recorded at 5 foot (1.5 m) intervals, change significantly between about six feet or elevation 61 ft (18.6 m) above the upper and about 14 feet (4.3 m) or elevation 14 ft (4.3 m) below the lower rebound zones. The figure also indicates that this lower change corresponds to 7 pile diameters [i.e., 7B for the 2 ft (600 mm) square PCP] into the lower rebound zone. The SPT N-values also increase to over 50 blows per foot near elevation 15 ft (4.6 m).

Figure 4.4 shows that DFN for the 2 ft (600 mm) square PCP, decreases significantly at the start of the high pile rebound elevation of 41 ft (12.5 m). Below this elevation, DFN gradually decreases from about 1.5 inches (37.5 mm) to zero at about elevation 26 feet (7.9 m). From this elevation until the end of driving, this pile also bounced above and below DFN of zero. The difference between elevations 41 and 26 feet (12.5 and 7.9 m) is 15 feet (4.5 m), which corresponds to approximately 7.5 pile diameters into this rebound layer as shown on the figure. There are less PDA data points associated with John Young than Anderson Street Overpass.

SPT N-values change significantly about 12 feet (3.6 m) below the start of lower rebound zone, corresponding to 6 pile diameters as shown on the figure. The SPT N-values also increase to over 50 blows per foot near elevation 18 ft (5.5 m). The sampling interval was approximately 5 feet (1.5 m) throughout the soil boring at John Young but it was approximately 3 feet (1 m) at Anderson Street Overpass.

In summary, the sampling interval may slightly influence the depth at which the N-values increase. N-values increased from 6 to 7 B below the pile tip in the high pile rebound zone, while pile driving problems occurred once the pile was driven from 7.5 to 9.5 B into the high rebound soils. These two findings need to be further evaluated using PDA and N values from more sites. The N-values should be taken at 3 foot intervals while the PDA data should be provided per hammer blow. Schmertmann and Nottingham, (1978) a common static end bearing approach predicts a zone of influence of 8B above the pile tip and 3.5B below the pile tip.

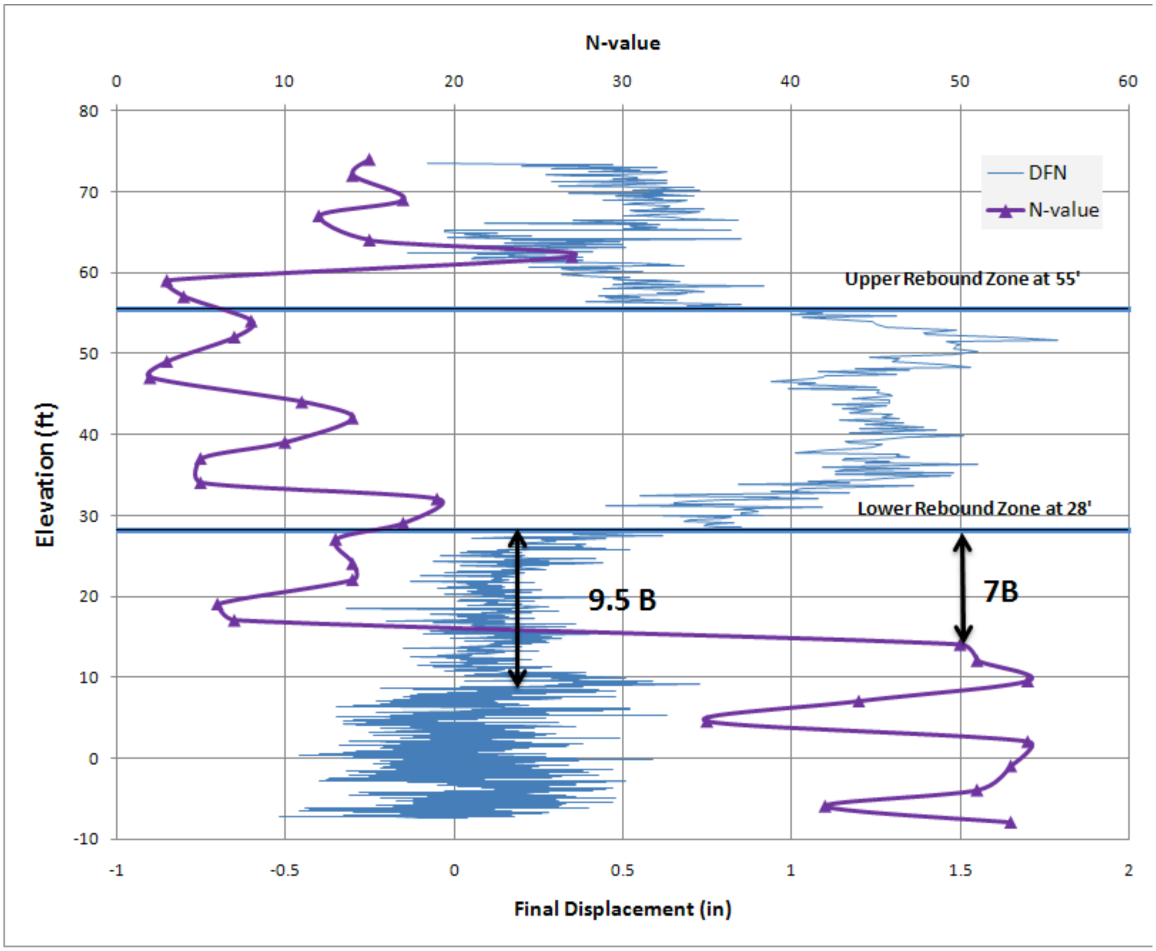


Figure 4.3 PDA Final Displacement and AS-103 N-values versus Elevation for Anderson Street Pile #6 in Pier 6

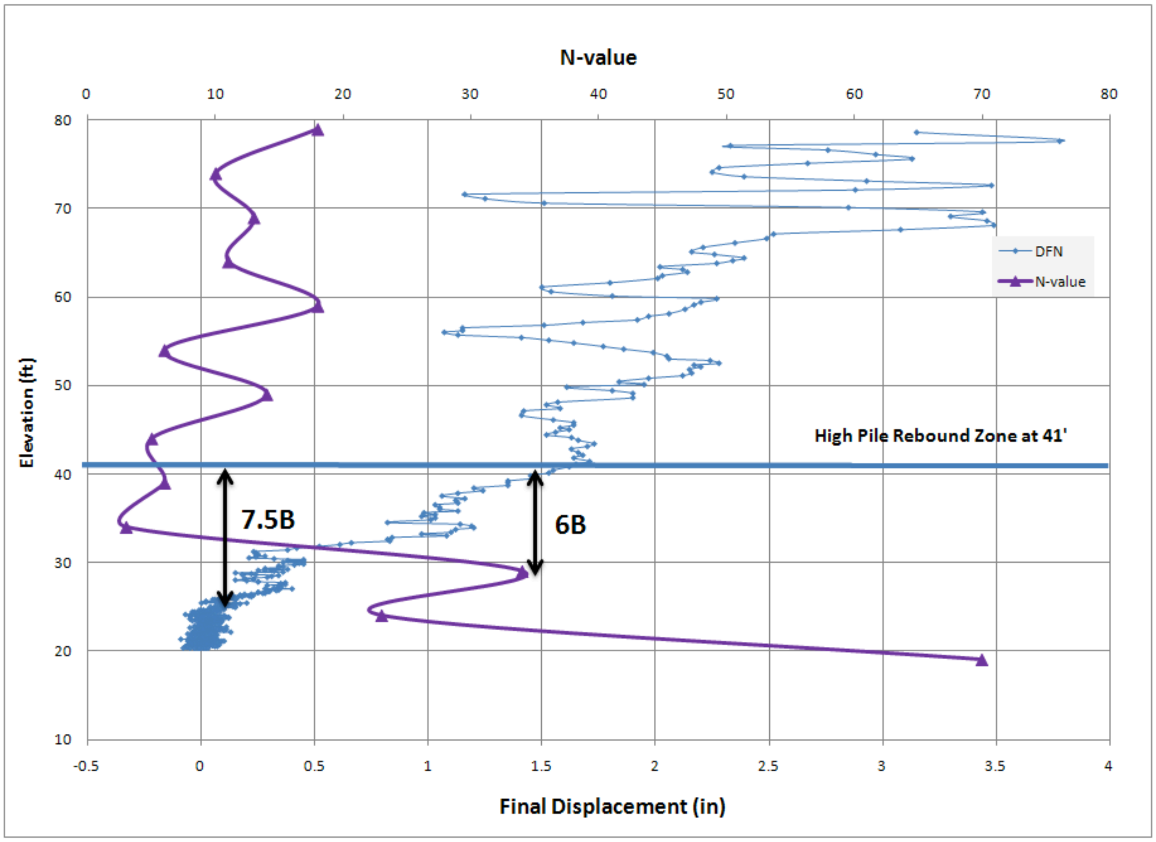


Figure 4.4 PDA Final Displacement and Boring 04-S-49 N-values versus Elevation for John Young Pile #6 in End Bent 2 Westbound

4.1.3 Design Phase Summary from PDA Data Analysis

The design phase PDA analysis of the Anderson Street Overpass and John Young projects produced the following findings:

• PDA data was used to show the extent of the high pile rebound zones.

• Both the Rebound/DMX and DFN N-values versus elevation plots show the piles "bouncing" somewhere between 8 and 10 pile diameters into the rebound layer.

• The DFN and N-value versus elevation plots are somewhat mirror images of each other indicating that SPT testing may be used to predict rebound.

• The pile "bouncing" elevation matches the elevation of the top of dense or very dense to hard soils.

• The high pile rebound zones for both sites starts near the top elevation of the Hawthorn Group as shown on Figure 2.4.

4.2 Analysis of Soil Profiles

Using the findings from the PDA analyses, the results obtained from both the design phase and the research phase were evaluated by comparing average data from the high pile rebound zones to average data from the overlying no rebound zones. The various soil parameters (i.e., N, q_c, etc.) were plotted versus elevation and the rebound and overlying no rebound zones based on the test pile PDA data were overlaid onto the plots. These plots were then evaluated to determine any trends or correlations.

The PDA data indicates that there are two high pile rebound zones at Anderson Street Overpass; however the pile was driven through the upper zone; therefore, analyses were based only on its lower high pile rebound zone. In this lower zone pile capacities were not achieved, which allows comparisons to be made for high pile rebound zones where piles did not meet the current FDOT Specification 455 of less than 0.25 inch (6 mm) of rebound and not more than 20 blows per inch at refusal. The high rebound zone for Ramsey Branch Bridge was chosen based on the PDA data shown for Pile RBB4P20, which displayed excessive rebound at an elevation of -22 ft (-6.7 m) as shown in Appendix C Figure C.7. Table 4.2 summarizes the elevations associated with the PDA data including elevations for the upper and lower rebound zones at Anderson Street Overpass along with the bouncing elevations where available.

Site Name	High Rebound Elevation (ft)	Bouncing Elevation (ft)	Relative Distance within High Pile Rebound Zone to Bouncing
Anderson Street Overpass *Upper Zone*	74 - 55	None	Not Applicable
Anderson Street Overpass *Lower Zone*	28	9	9.5
John Young	41	26	7.5
Ramsey Branch Bridge	-22	Not Available	Not Applicable

Table 4.2 Rebound Elevations Used For Analyses

The soil descriptions for each site were edited to reflect the appropriate modifiers and suffixes based on the percent passing the No. 200 sieve and are as follows: 0 to 15 percent, *trace*; 16 to 30 percent, *some*; 31 to 45 percent, -*y*; 45 to 50 percent *and* (Sowers, 1970). All raw N-values were corrected to N_{60} values according to Equation 1. The hammer efficiency factor, E_m, for the United States was 0.6 for a safety hammer and 0.9 for an automatic hammer; the borehole diameter correction factor, C_B, for a 2.5-in sampler was 1.0; the sampling method factor, C_S, for a standard sampler was 1.0; and the rod length factor, C_R, for rods greater than 30-ft was 1.0. The raw N and N_{60} values were equivalent when safety hammers were used with standard holes and rods as was the case for the design phase SPT borings.

4.2.1 Laboratory Results

Typical geotechnical engineering laboratory tests such as Atterberg limits, sieve analysis, and hydrometer tests were used to provide an understanding of high pile rebound. The engineering properties obtained were evaluated throughout the soil profiles for the three sites.

4.2.1.1 Soil Classification

Each appendix includes the grain size distributions from the disturbed split spoon samples obtained from each site. Using these results, the soils were classified according to the Unified Soil Classification System (USGS). The majority of the soils in the high pile rebound zones were classified as: SC at Anderson Street Overpass, SP-SM at John Young and SM at Ramsey Branch Bridge.

From the design and research phase borings, contained in Appendix A, the soils within the high pile rebound zone at Anderson Street Overpass were visually described as dark greenish gray silty clay and clayey fine sand. From the design and research phase borings for John Young (Appendix B) soils within the high pile rebound zone consisted of dark gray to green clayey fine sand, dark green fine sand. An olive green sandy clay was described in the John Young research phase borings at elevation 14 ft (4.3 m). At Ramsey Branch Bridge (Appendix C), the soil in the high pile rebound zone was greenish gray clayey sand with trace of shells.

According to its geologic description, an olive green color corresponds to the color of the soils at top of the Hawthorn Group. This color description may be the same as the geotechnical engineers' color presented in the soil borings for both Anderson Street Overpass and John Young; which was light green or greenish gray or dark green. Figure 2.4 showed the elevations associated with the top of the Hawthorn Group along with approximate locations for the research sites. This geologic group was mapped in the Panhandle, but not in the area of the Ramsey Branch Bridge. According to Figure 2.4 the elevation of the Hawthorn group in Central Florida ranges from approximately 50 to 0 ft (15.2 to 0 m) above sea level or about the same elevation associated with high pile rebound at both the Anderson Street Overpass and John Young sites.

4.2.1.2 Gradation Analysis

4.2.1.2.1 Sand and Silt Contents versus Elevation

For each site, both the average silt content and average sand content were plotted versus elevation as shown in Figure 4.5, Figure 4.6, and Figure 4.7. The rebound and overlying no rebound zones from the test pile PDA data were shown on the plots along with the average silt or sand content in percent for each zone.

Sand was the most prominent soil at the three sites, consistently making up over 50 percent of the subsoil's. At some depths, sand made up over 90 percent of the subsoil's. This was especially true at Anderson Street Overpass and John Young.

Soils in the high pile rebound zones contained a high percentage of silts, which on average were 19 percent for Anderson Street Overpass, 16 percent for John Young, and 20 percent for Ramsey Branch Bridge. In summary, silt constitutes over 15 percent of the rebound soils.

These plots showed an increase in silt content, and a corresponding decrease in sand content. The rebound soils at Anderson Street Overpass have a maximum of approximately 60 silt content and 40 percent sand near elevation 0 (Figure 4.5). For John Young (Figure 4.6), the silt content increases to over 70 percent while the sand content decreases to less than 30 near elevation 10 ft (3.3 m). The same magnitude of silt changes were not evident at Ramsey Branch Bridge (Figure 4.7); however, the silt content does increase from less than 10 percent to an average of over 20 percent at elevation -20 ft (6.7 m).

The clay content versus elevation was also determined for each site. The data analysis, which was originally included with the above discussion, did not provide any clear findings; therefore, it was excluded from this section. Details of this work are available in Chin Fong (2010).

The silt content increased below the top of the high pile rebound elevations for the Anderson Street Overpass and John Young sites. The elevations associated with the increase at these sites are 14 feet and 15 feet (4.3 and 4.5 m) for Anderson Street Overpass and John Young, respectively. These depths, within the rebound zone, are equivalent to 5B and 5.5B for Anderson Street Overpass and John Young, respectively. The silt content remained relatively constant throughout the Ramsey Branch Bridge rebound zone.

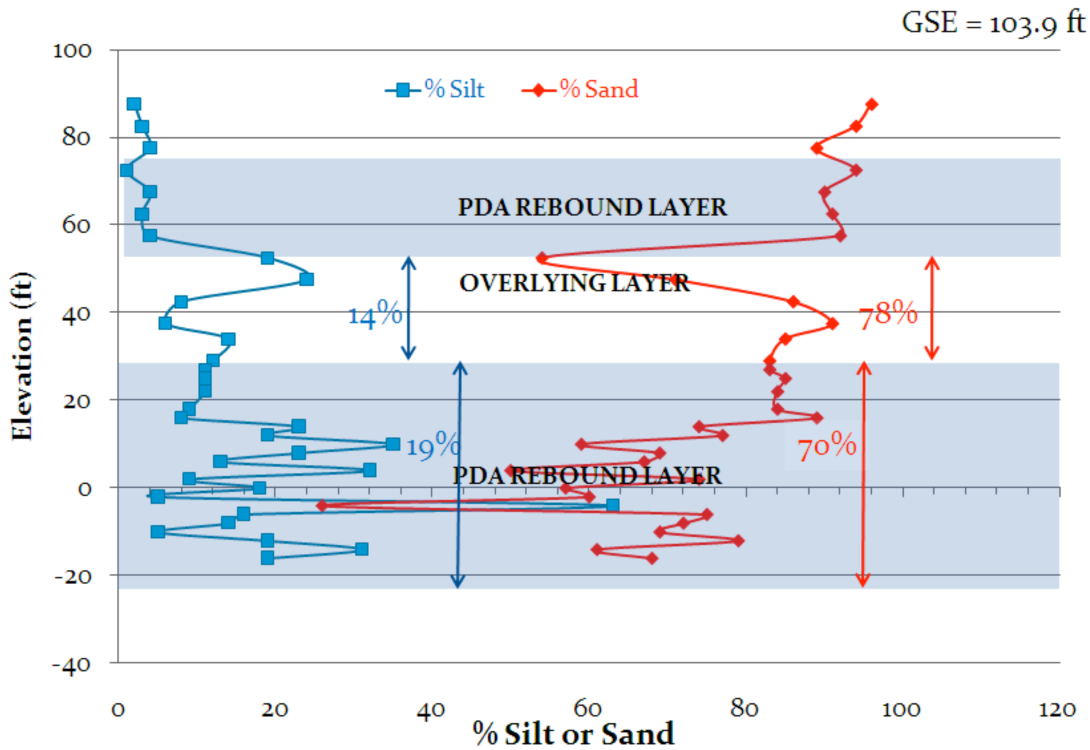


Figure 4.5 Anderson Street Overpass Elevation versus Sand Content and Silt Content

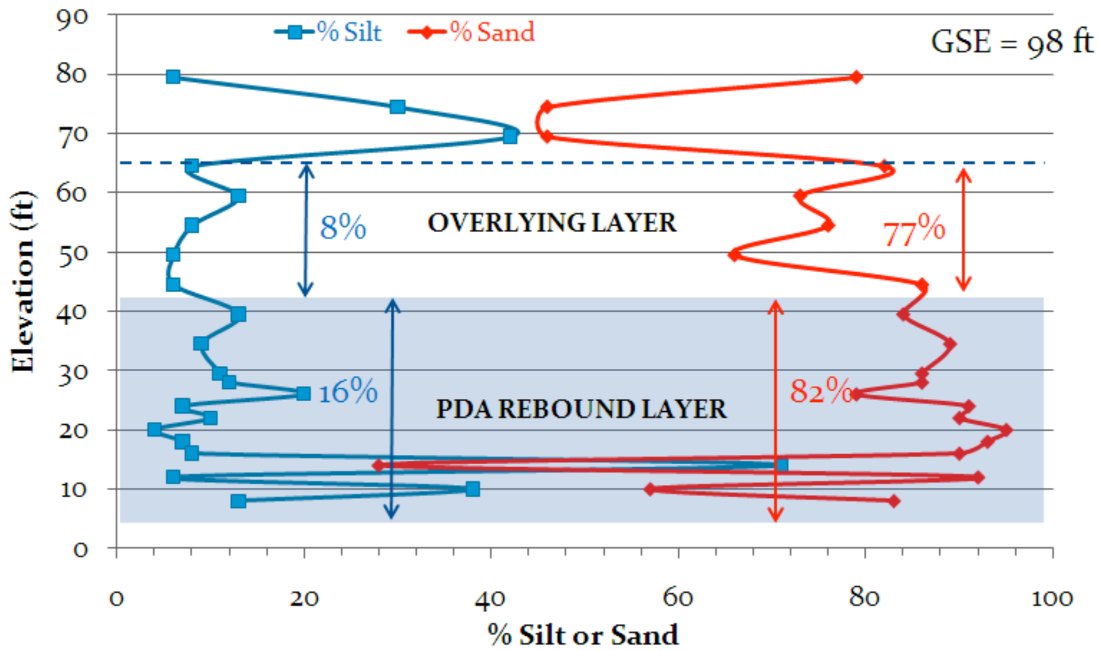


Figure 4.6 John Young elevation versus Sand Content and Silt Content

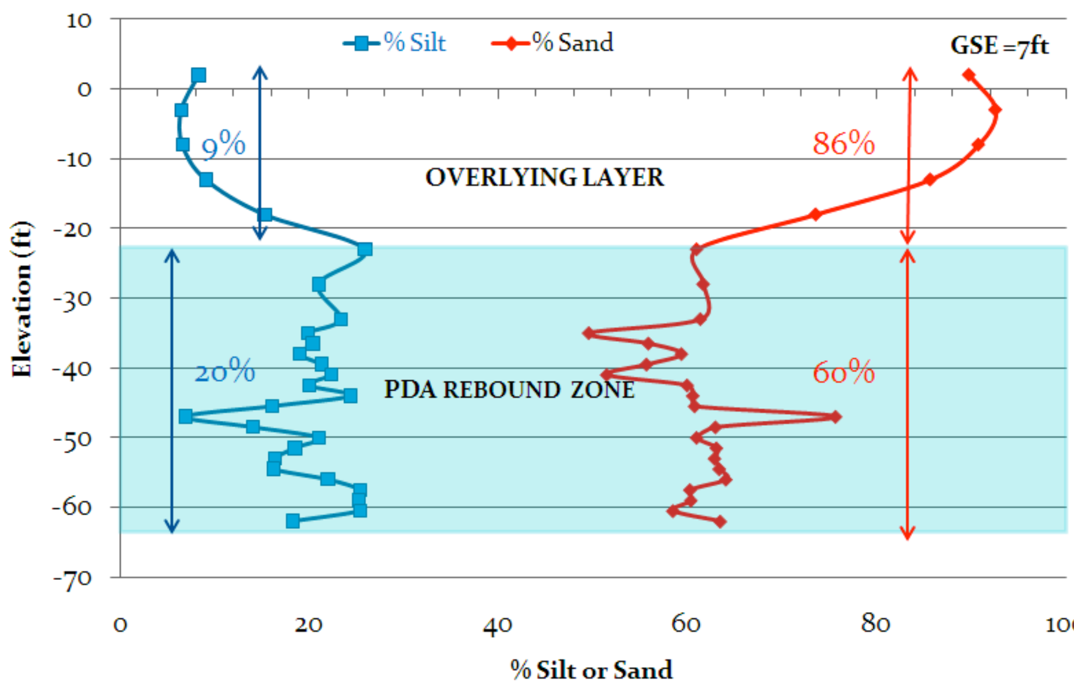


Figure 4.7 Ramsey Branch Bridge Elevation versus Sand Content and Silt Content

4.2.1.2.2 Sand Ratios and Silt Ratios of Rebound to No Rebound Soils

The average silt content and average sand content from the $Rebound_{PDA}$ soils was compared to the average silt content and average sand content from the overlying no $Rebound_{PDA}$ soils for each site. The thickness of the overlying layers was between 24 and 27 feet and the silt content associated in the overlying layer zone was between 8 and 14 percent. Table 4.3 contains these averages, plus ratios computed as the $Rebound_{PDA}$ soil content divided by the overlying no $Rebound_{PDA}$ soil content.

The Anderson Street Overpass data produced a silt ratio of 1.4. The John Young and Ramsey Branch Bridge data produced silt ratios of 2.0 and 2.2, respectively. The John Young ratio excluded data from two elevations. First, data from the fill above elevation 98 ft (29.8 m) used to construct the overpass, were excluded because they represented a different soil. Second, data associated with elevations 70 and 75 feet (21.3 and 22.9 m) were excluded because they were also considered part of a different layer. Other profiles may include layer variations and their thicknesses would affect these conclusions. These findings are based on data from one of the three sites and the excluded layers were relatively thick. If thinner layers are encountered these findings may not be accurate, therefore additional sites should be investigated until these layer variations can be validated.

The silt ratios are relatively consistent and were based on a significant number of grain size tests (i.e., 35, 22 and 27 for Anderson Street Overpass, John Young and Ramsey Branch Bridge). The silt contents, which were determined using the hydrometer, can help engineers anticipate high rebound soils. In general, about a 200 percent increase in the silt content occurred in the rebound.

The sand ratio between 0.8 and 1.1 occurred for the research sites (Table 4.3). For John Young, the data associated with the fill above 98 ft (29.8 m) and the data at 70 and 75 ft (21.3 and 22.9 m) were excluded as discussed above. The sand ratio remains near 1 and would not be easily used to predict or anticipate rebound soils.

Site Name	Overlying Layer Average Silt Content	Rebound Layer Average Silt Content	Silt Ratio (Rebound/ Overlying)	Overlying Layer Average Sand Content	Rebound Layer Average Sand Content	Sand Ratio (Rebound/ Overlying)
Anderson Street Overpass	14	19	1.4	78	70	0.9
John Young	8	16	2.0	77	82	1.1
Ramsey Branch Bridge	9	20	2.2	86	60	0.8

Table 4.3 Ratio of percentage of Silt and Sand from Rebound Soils over percentage of Silt and Sand from No Rebound Soils

4.2.1.3 Atterberg limits

4.2.1.3.1 Design Phase Atterberg Limits Results

Only the design phase soil properties from Anderson Street Overpass and John Young (Appendices A.1 and B.1) contained enough Atterberg limits data for analysis. There was only one sample with limits and natural moisture contents from Boring B-1 of the 2004 borings at Ramsey Branch Bridge.

From Anderson Street Overpass, data from borings AS-101, 102, 103, 107, 108, 109 were used and for John Young, data from the six borings near the overpass (i.e., I4-1, 2, 3, 4, 04-S-39, 40) were used. The liquid limit (LL) and the plasticity index (PI) values, shown on the plasticity chart in Figure 4.8(ASTM D2487), plotted between the U-line and the A-Line. Seven out of the 10 data points, or 70 percent, indicated that the soils were highly plastic.

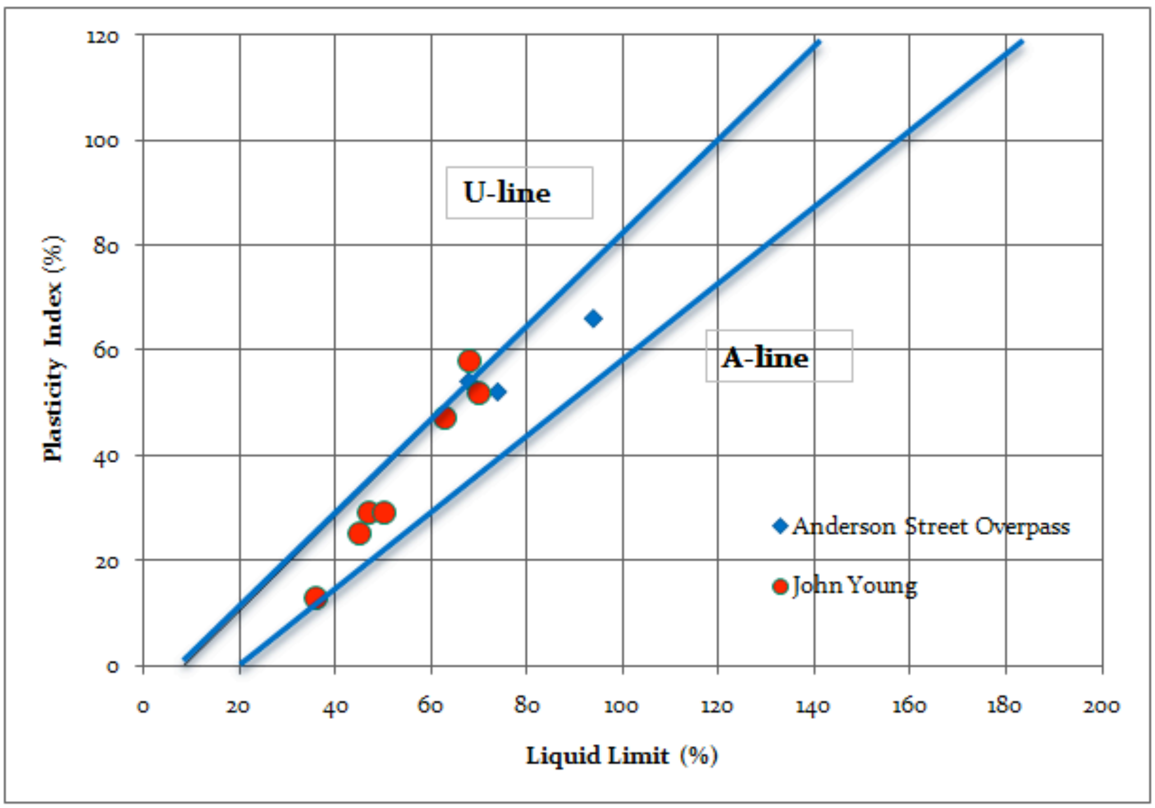


Figure 4.8 Design Phase Plasticity Chart for Anderson Street Overpass and John Young Data from No Rebound Zones

Several limits were recorded in the rebound zones at Anderson Street Overpass and John Young (i.e., the lower zones). For Anderson Street Overpass they were below elevation 30 ft (9.1 m) and for John Young they were below elevation 20 ft (6.1 m). They were plotted on the plasticity chart in Figure 4.9, which shows that the six of the eight soils, or 75 percent, were highly plastic. These highly plastic values plotted closer to the A-line than the values from the no rebound zones.

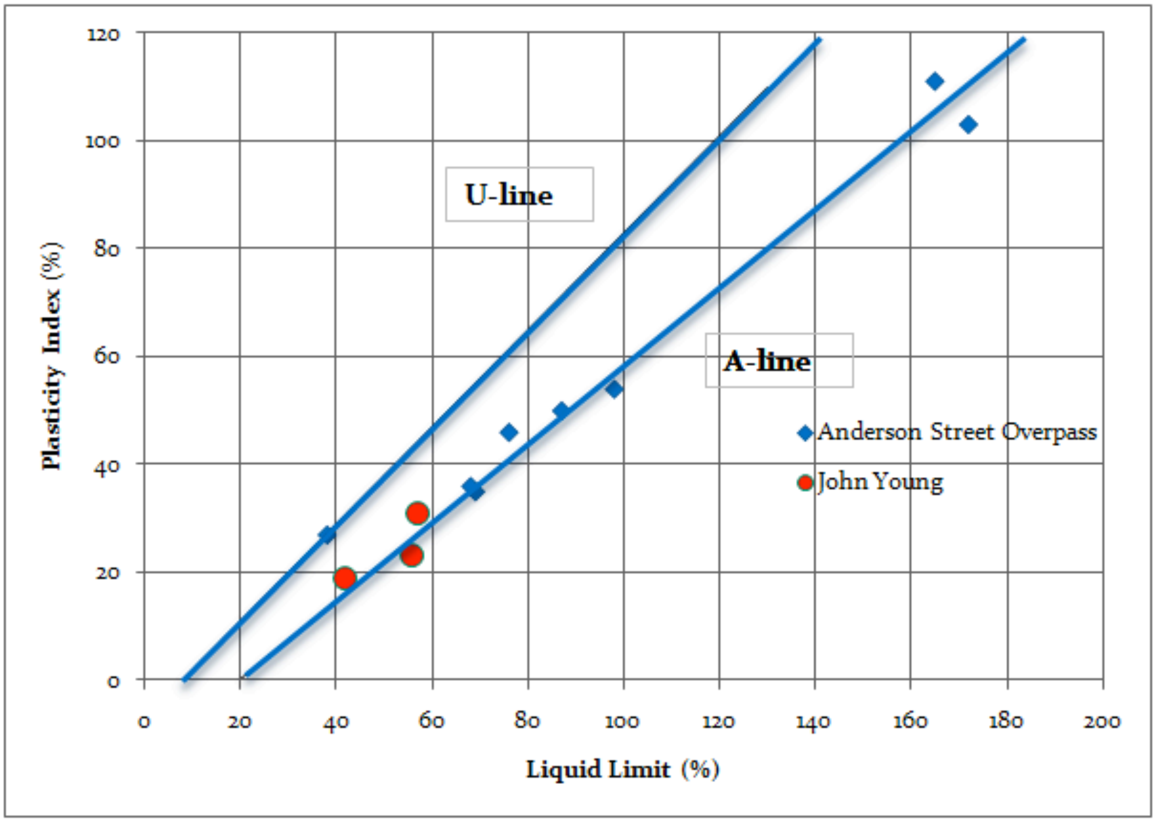


Figure 4.9 Design Phase Plasticity Chart for Anderson Street Overpass and John Young Data from Rebound Zones

Natural moisture content, Atterberg limits, Liquidity Index and USCS classification information from the design phase borings were compared and summarized in Table 4.4. The table includes a column indicating whether or not the soil sample was in the high pile rebound zones. The majority of the samples were highly plastic silts or clays. The Liquidity Index was generally less than one that would imply that the soil would produce a plastic shear failure. There were several low plasticity clays from the John Young site.

Site	Wn (%)	LL (%)	PL (%)	PI (%)	LI	Rebound	USCS Symbol
Anderson Street Overpass	51	87	37	50	0.28	Yes	CH
	28	38	11	27	0.63	Yes	MH
	35	68	32	36	0.08	Yes	CH
	49	98	44	54	0.09	Yes	CH
	63	165	54	111	0.08	Yes	CH
	78	172	69	103	0.09	Yes	MH
	46	76	30	46	0.35	Yes	CH
	55	68	14	54	0.76	No	CH
	47	94	28	66	0.29	No	CH
	50	74	22	52	0.54	No	CH
	28	63	16	47	0.26	No	CH
John Young Parkway	44	57	26	31	0.58	Yes	CH
	31	42	23	19	0.42	Yes	CL
	38	56	33	23	0.22	Yes	MH
	21	36	23	13	-0.15	No	CL
	28	50	21	29	0.24	No	CH
	32	70	18	52	0.27	No	CH
	32	68	10	58	0.38	No	CH* Above U
	28	47	18	29	0.34	No	CL
	29	45	20	25	0.36	No	CL

Table 4.4 Design Phase Summary of LL and Natural Moisture Content for Anderson Street Overpass and John Young in both the Rebound and No Rebound Soils

In summary, based on the existing design phase information, the Atterberg limits plotted between the U-line and the A-line for both sites. They were highly plastic at Anderson Street Overpass and varied between high and low plasticity at John Young. These trends were true for soils regardless of whether or not samples were located in the high pile rebound zones.

4.2.1.3.2 Research Phase Atterberg Limits Results

To determine if any correlations could be developed that were not evident from the design phase data evaluation, the research phase analysis focused on continuing the comparison of limit data to the natural water content. Direct correlations between either PL or LL and natural moisture content were developed using all Atterberg results from the three sites. Figure 4.10 shows the relationship between natural moisture content and LL while Figure 4.11 shows the relationship between natural moisture content and PL. The two regression coefficients were based either on all the limit data or only the limit data from soils within the high pile rebound zone. All combinations of LL and PL versus moisture content provided linear relationships however; the regression coefficients were relatively low lying between 0.34 and 0.55. Trends were examined between data from all soils and data based on the soils in the rebound zones and very little change occurred as shown by the regression coefficients in the figures. In summary, there was insufficient evidence to prove that direct correlations between moisture content and PL or LL data could be used to predict soils that may produce high pile rebound.

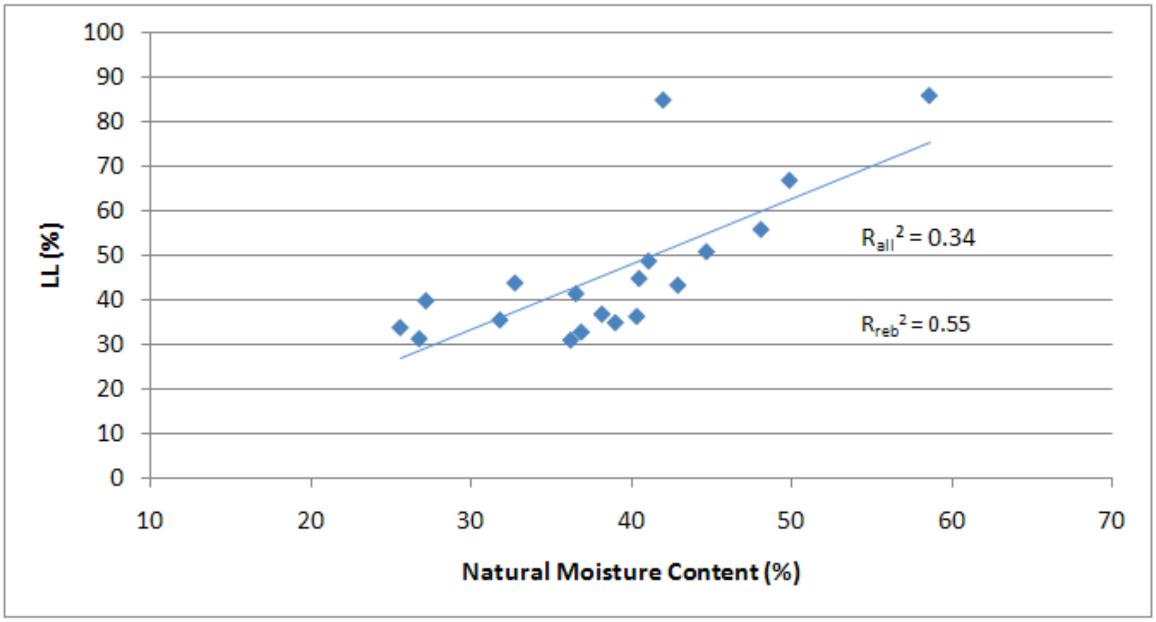


Figure 4.10 Research Phase Moisture Content versus Liquid Limit with Regression Coefficients for all Soils and Rebound Soils

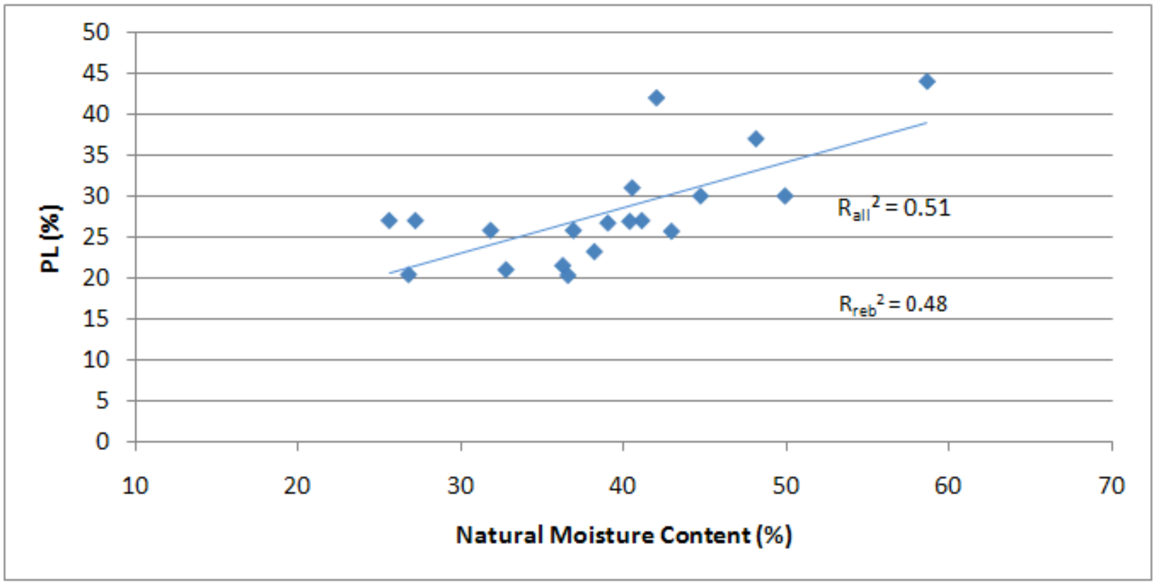


Figure 4.11 Research Phase Moisture Content versus Plastic Limit with Regression Coefficients for all Soils and Rebound Soils

Both the LL and moisture content were plotted versus elevation which was also overlaid with the rebound zone elevations for all three sites. The elevation versus LL plots shown in Figure 4.12, Figure 4.13 and Figure 4.14 indicate that the moisture content may approach the LL in the rebound layer. This observation was more predominant at both the Anderson Street Overpass and Ramsey Branch Bridge sites. John Young is mostly sand having less plastic soils. Anderson Street Overpass soils had the highest LL with a minimum value of 40 and the maximum value of 172. At elevation +50 ft (15.2 m) in John Young, the LL reaches a maximum value of 61.

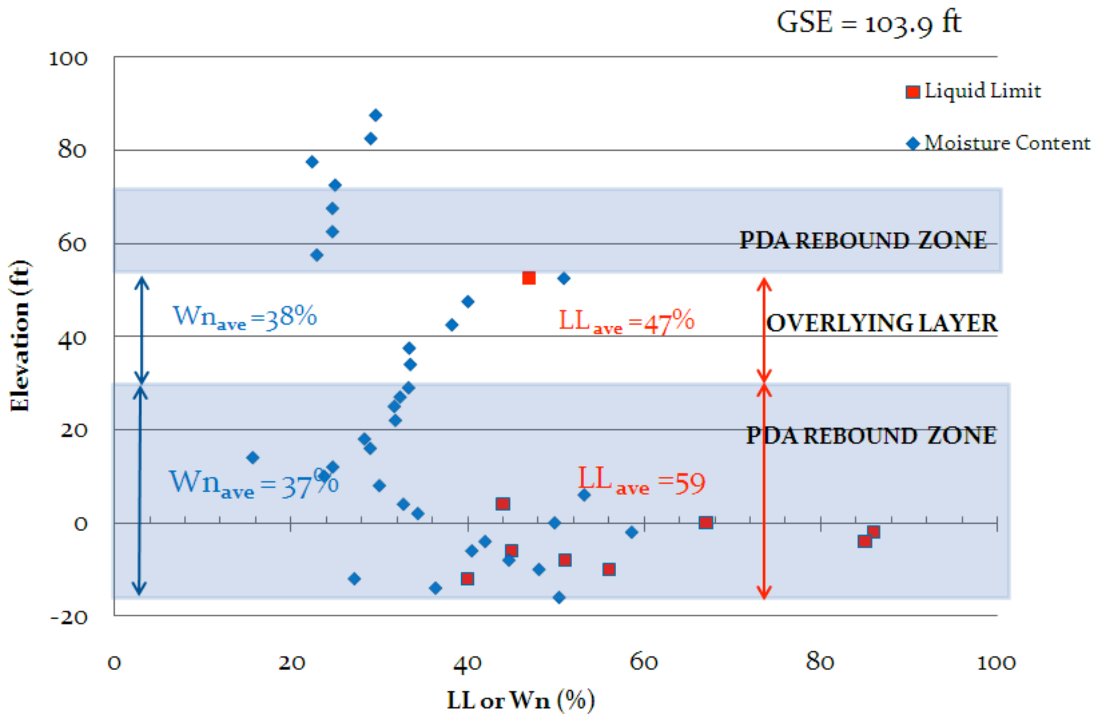


Figure 4.12 Research Phase Anderson Street Overpass Elevation versus Liquid Limit and Moisture Content

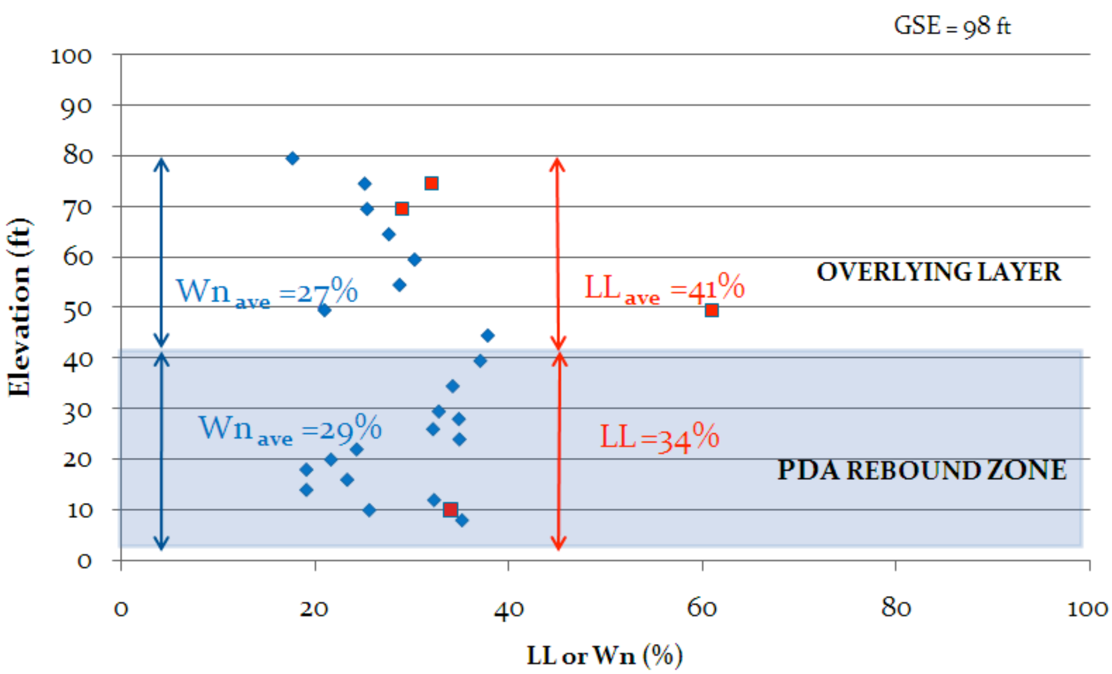


Figure 4.13 Research Phase John Young Elevation versus Liquid limit and Moisture Content

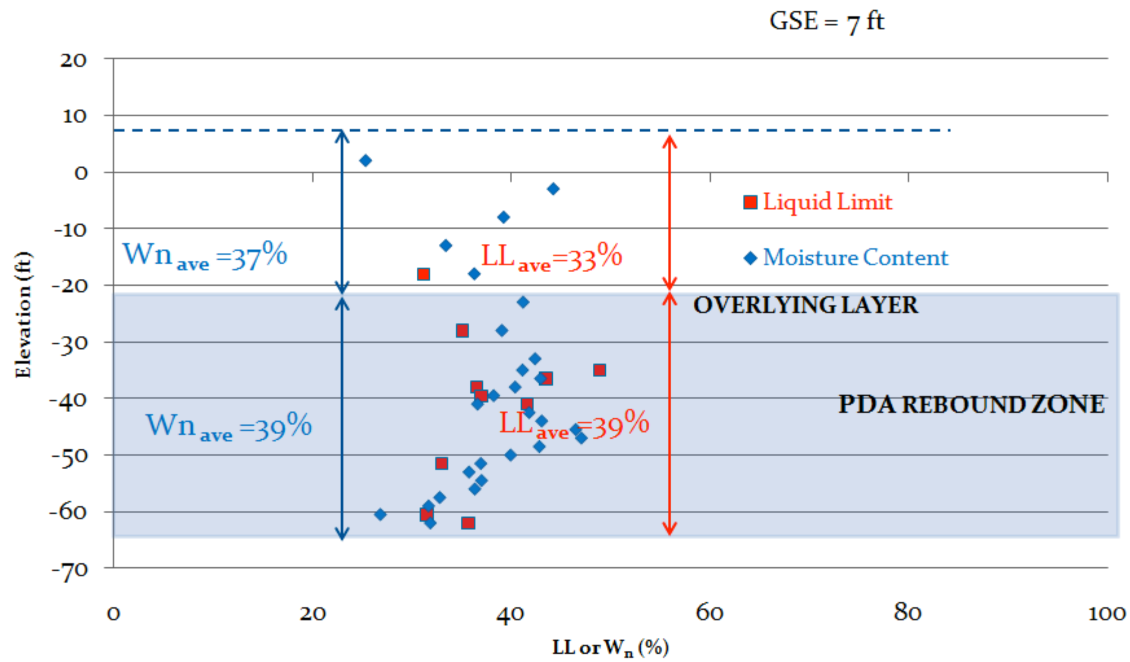


Figure 4.14 Research Phase Ramsey Branch Bridge Elevation versus Liquid Limit and Moisture Content

Table 4.5 shows the natural moisture content and the LL averages in the rebound layer. The LL and natural water content averages were calculated using the values in the PDA rebound zone. There was only one data point used for John Young data and the other two averages were based on 9 and 10 values, with Anderson Street Overpass having the lower number of points. The ratio of the natural water content and the liquid limit in the rebound layer of these soils ranges between 0.6 and 1.0. The range is not very consistent and is based on a limited number of data points. The Wn/LL ratio does not appear to be a useful indicator for the engineer who is trying to anticipate high rebound.

Site Name	Rebound Layer		Ratio
	W_{nave}	LL_{ave}	W_{nave} / LL_{ave}
Anderson Street Overpass	37	59	0.6
John Young	29	34	0.9
Ramsey Branch Bridge	39	39	1.0

Table 4.5 Research Phase Natural Moisture Content and Liquid Limit Summary and Ratios in Rebound Soils

4.2.2 Field Test Results

4.2.2.1 SPT Results

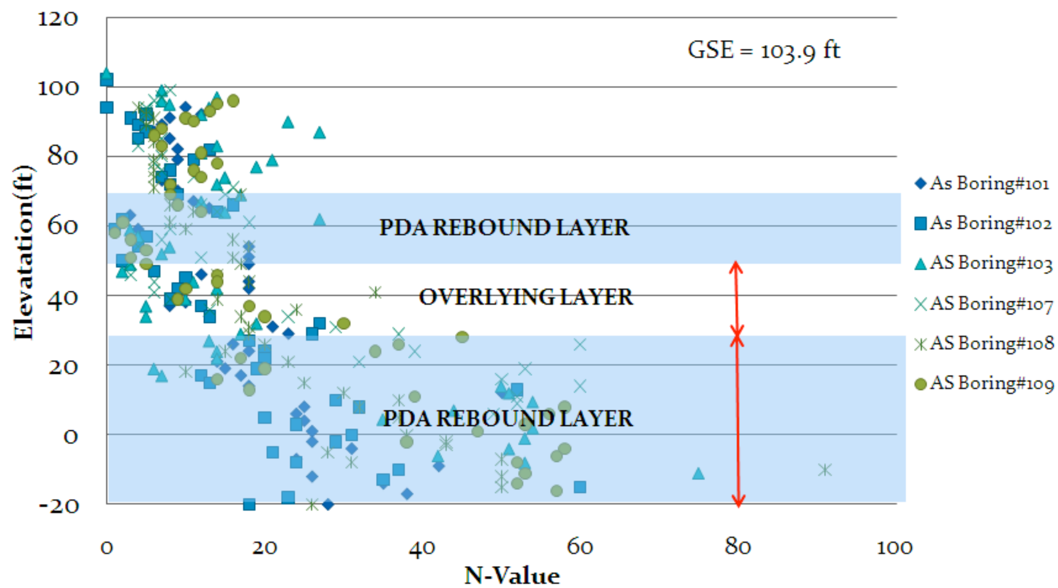
4.2.2.1.1 N and N_{60} versus Elevation

Both the raw N-values and corrected N_{60} values were plotted versus elevation at each site to determine possible trends. The raw N-values obtained from the safety hammer used during the design phase were equivalent to the N_{60} values because the efficiency was 60 percent. The raw N-values from the research phase were obtained using an automatic hammer with PDA sensors converted to N_{60} values based on a hammer efficiency of 90 percent.

The evaluation was performed by averaging N and N_{60} values above and within the $Rebound_{PDA}$ zones. Once the averages were determined, ratios were calculated by dividing the $Rebound_{PDA}$ averages by the average from the overlying no rebound zone. This approach produced the same ratios from both N and N_{60} values; therefore, the following results display N versus depth and not N_{60} versus depth data. The overburden correction from (2) also did not affect the results of this evaluation and therefore was not reported.

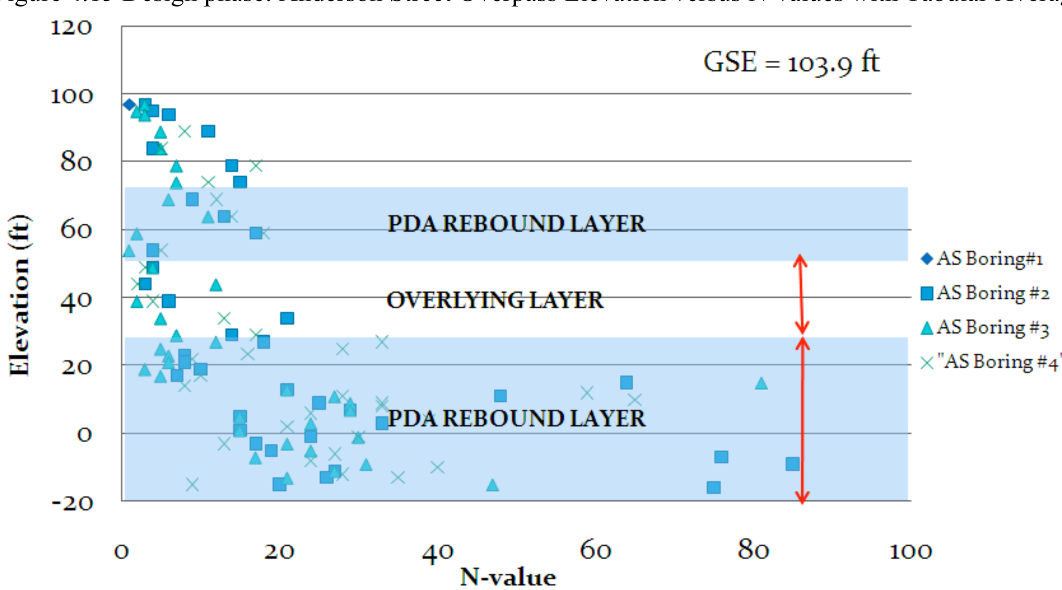
The design phase graphs for the raw N-value versus elevation are shown in Figure 4.15 for Anderson Street Overpass, Figure 4.17 for John Young and Figure 4.19 for Ramsey Branch Bridge. For the research phase, the elevations versus N-values graphs are shown in Figure 4.16, Figure 4.18 and Figure 4.20 for Anderson Street Overpass, John Young, and Ramsey Branch Bridge, respectively. Each plot includes a table with the average N-values for the $Rebound_{PDA}$ zone and the overlying no rebound zone for each test boring. For Anderson Street Overpass, the overlying no rebound zone was between the upper and lower high pile rebound zones as shown by the arrows in the figure. For John Young, the averages were taken for two scenarios. One was based on the overlying zone below elevation 98 ft (29.8 m), which was the beginning of the natural soil deposits or the approximate grade for John Young Parkway and the other was based on the ground surface associated with the top of the I-4 overpass or 119 ft (36.3 m). For Ramsey Branch Bridge, the averages were taken above and below - 22 ft (-6.7 m).

The average raw N values increased below the high rebound elevations for all three research sites. The elevation associated with the increase are 18 feet and 26 feet (5.5 and 7.9 m) for Anderson Street Overpass and John Young, respectively, and -30 feet (-9.1 m) for Ramsey Branch Bridge. These depths within the rebound zone are equivalent to 5B for both Anderson Street Overpass and Ramsey Branch Bridge and 5.5B for and John Young. Note that Ramsey Branch Bridge piles were 1.5 ft (450 mm) square PCP, while the other two sites had 2 ft (600 mm) square PCPs.



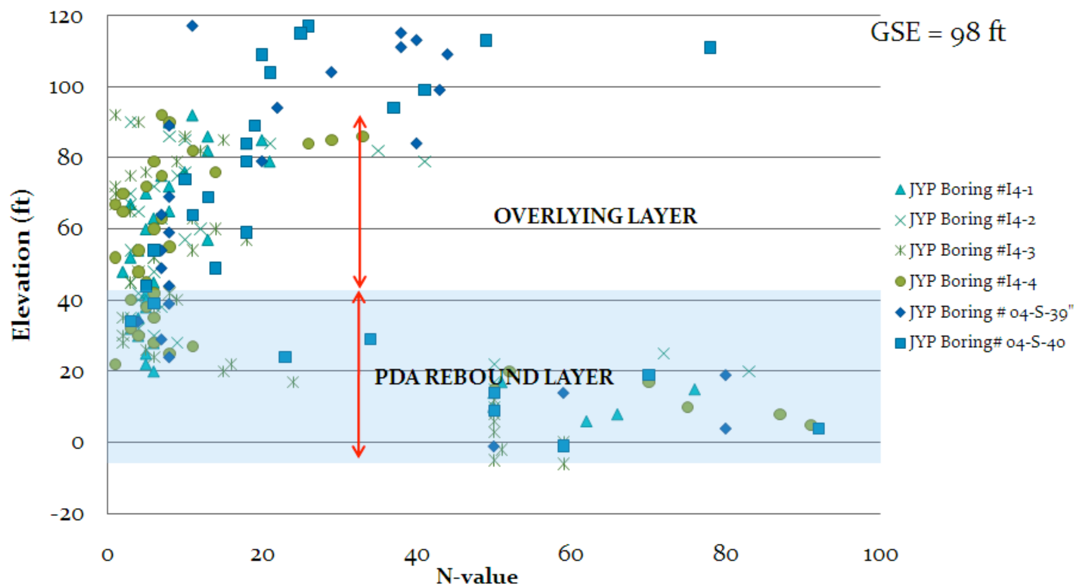
	Boring#101	Boring#102	Boring#103	Boring#107	Boring#108	Boring#109
Overlying Layer Average	15	12	9	13	19	13
Rebound Layer Average	28	30	39	50	36	42

Figure 4.15 Design phase: Anderson Street Overpass Elevation versus N-values with Tabular Averages



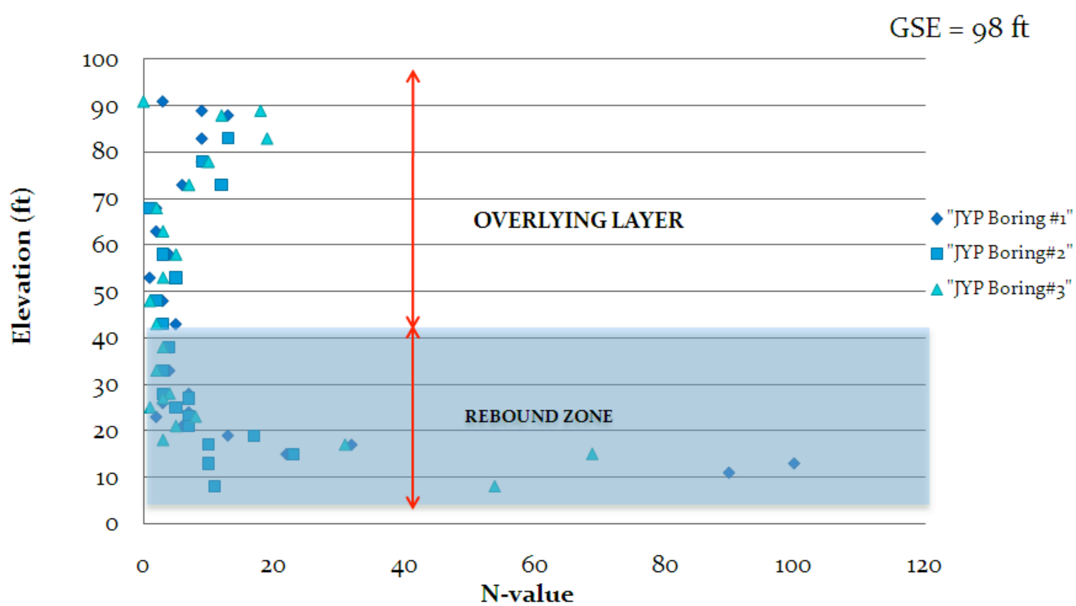
	Boring#1	Boring#2	Boring#3	Boring#4
Overlying Layer Average	7	9	5	8
Rebound Layer Average	24	31	23	28

Figure 4.16 Research phase: Anderson Street Overpass Elevation versus N-values with Tabular Averages



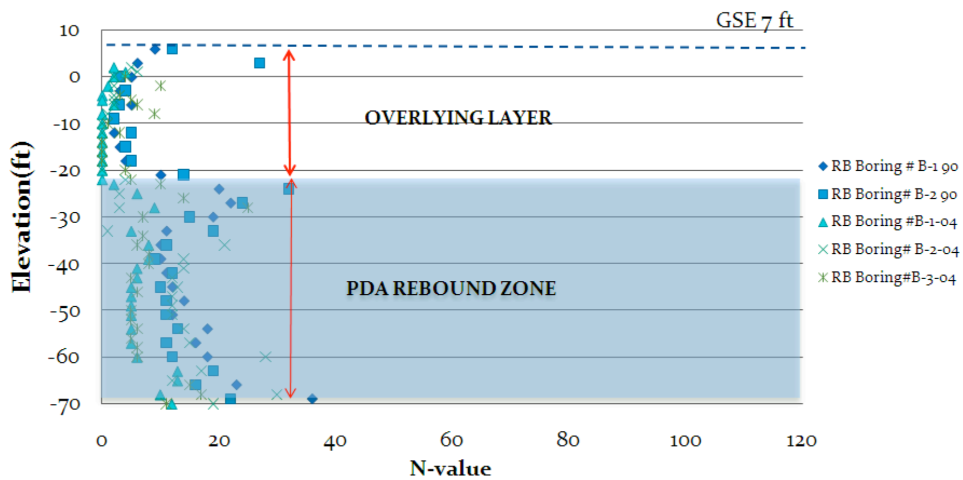
	Boring#I4-1	Boring#I4-2	Boring#I4-3	Boring#I4-4	Boring#04-S-39	Boring#04-S-40
Overlying Layer Average	9	10	8	9	13	15
Rebound Layer Average	27	36	28	33	38	43

Figure 4.17 Design Phase: John Young Elevation versus N-values with Tabular Averages



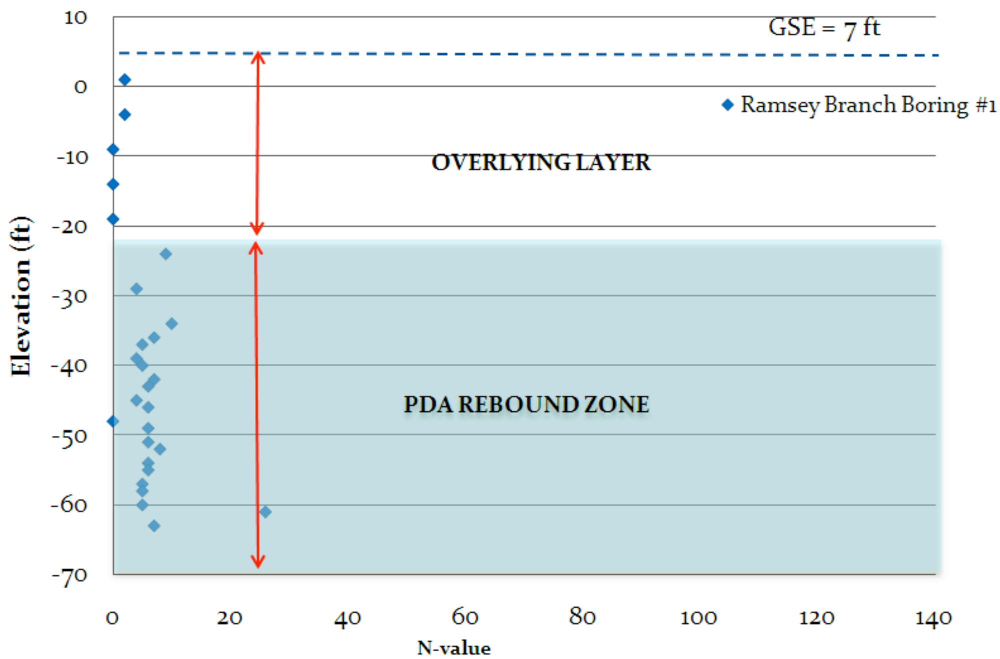
	Boring#1	Boring#2	Boring#3
Overlying Layer Average	6	6	8
Rebound Layer Average	23	9	17

Figure 4.18 Research Phase: John Young Elevation versus N-values with Tabular Averages



	Boring# B-1 90	Boring# B-2 90	Boring# B-1 04	Boring# B-2 04	Boring# B-3 04
Overlying Layer Average	5	8	1	2	4
Rebound Layer Average	17	15	7	14	9

Figure 4.19 Design Phase: Ramsey Branch Bridge Elevation versus N-values with Tabular Averages



	Boring# TB-1
Overlying Layer Average	1
Rebound Layer Average	7

Figure 4.20 Research Phase: Ramsey Branch Bridge Elevation versus N-values with Tabular Averages

4.2.2.1.2 N Ratios

Table 4.6 contains a summary of the N-values and corresponding ratios for the design phase, the research phase, and both phases combined for all three sites. It shows that the combined ratios varied from 2.8 to 5.1, the design phase ratios varied from 2.8 to 3.2, and the research phase ratios varied from 2.5 to 7.1. The averages from John Young, which were based on the elevation at the top of the overpass, were not included, because they produced results that were significantly different from the results based on the natural soil deposit. This finding indicates that engineers should only use zones deposited naturally. The standard deviation is larger for the research phase than the design phase, possibly due to the total number of design phase borings exceeding the number of research phase borings conducted for each site. The Anderson Street Overpass and John Young sites had six design phase borings to four and three research phase borings respectively, while the Ramsey Branch Bridge site had five design phase borings to one research phase boring. The ratios based on the combined data also have a larger standard deviation than the design phase ratios, because of the influence from the both data sets. The Ramsey Branch Bridge profile contains weaker soils than the Central Florida sites; however, the research phase ratio was 3.1 while the design phase ratio was somewhat higher than the others at 7.0.

The ratios indicate that the rebound soil N values increase by about a factor of 3. The design phase ratios are consistent and may be helpful to engineers trying to anticipate high rebound. The number of research phase borings should be increased to help clarify the inconsistencies with their ratios. These ratios could be determined from SPT borings, which are always completed during geotechnical investigations. The main concern that the geotechnical engineer would face would be how or where the averages should be taken to determine ratios.

Site Name	Design Phase Overlying Layer Average	Design Phase Rebound Layer Average	Design Phase N raw Ratio (Rebound/ Overlying)	Research Phase Overlying Layer Average	Research Phase Rebound Layer Average	Research Phase N_{raw} Ratio (Rebound/Overlying)	Combined N raw Ratio (Rebound/ Overlying)
Anderson Street Overpass	14	38	2.8	7	27	3.7	3.2
John Young	11	34	3.2	7	16	2	2.8
Ramsey Branch Bridge	4	12	3	1	7	7	5.1

Table 4.6 Ratios of N in Rebound Soils to N in Overlying No Rebound Soils for Design, Retesting and Combined Data

4.2.2.2 Pocket Penetrometer Results

4.2.2.2.1 Pocket Penetrometer q_u versus Elevation

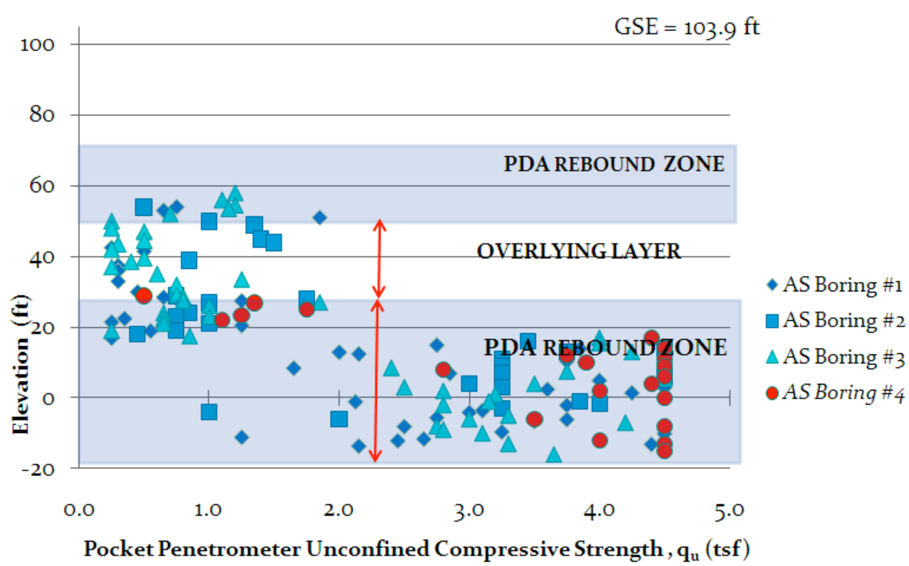
During SPT sampling, the unconfined compressive strength (i.e., q_u) was measured using the Pocket Penetrometer with tests conducted on any cohesive samples retrieved from the research phase borings. Several tests were performed while the samples were in the split spoon. The values were recorded in the field logs and an average q_u was determined for the entire sample. Values associated with the cuttings or backwash soils within the spoon were eliminated from the averages. The average q_u data were plotted versus elevation. The $Rebound_{PDA}$ and Overlying rebound zones, from the test pile PDA data, were overlaid onto the plots.

For Anderson Street Overpass, q_u values were analyzed from samples obtained from three of the four borings, because there were no q_u readings obtained in the overlying no rebound zone from AS Boring 4 (Figure 4.21). At this site, q_u increases from less than 2.0 tsf (190 kPa) to between 2.0 tsf and 4.5 tsf (190 to 430 kPa) about 8 feet into the rebound layer which started at elevation 28 ft (8.5 m). In the overlying no rebound soil, q_u ranged from 0.2 tsf to 1.7 tsf (19 to 163 kPa). The average q_u for the overlying and $Rebound_{PDA}$ zones are shown in tabular format for the research phase borings.

At John Young (Figure 4.22), there are fewer q_u values because the soils were not as cohesive; however, they follow the same pattern as found in Anderson Street Overpass. The results are based on Borings 1 and 3, because there was only one value recorded in the overlying no rebound layer for Boring 2. The q_u values were generally below 1.0 tsf (96 kPa) in the overlying no rebound zone and increased about 13 ft (4.0 m) into the rebound layer to approximately 1.5 tsf to 4.5 tsf (144 to 430 kPa).The average q_u for the overlying and $Rebound_{PDA}$ zones are shown in tabular format for the research phase borings.

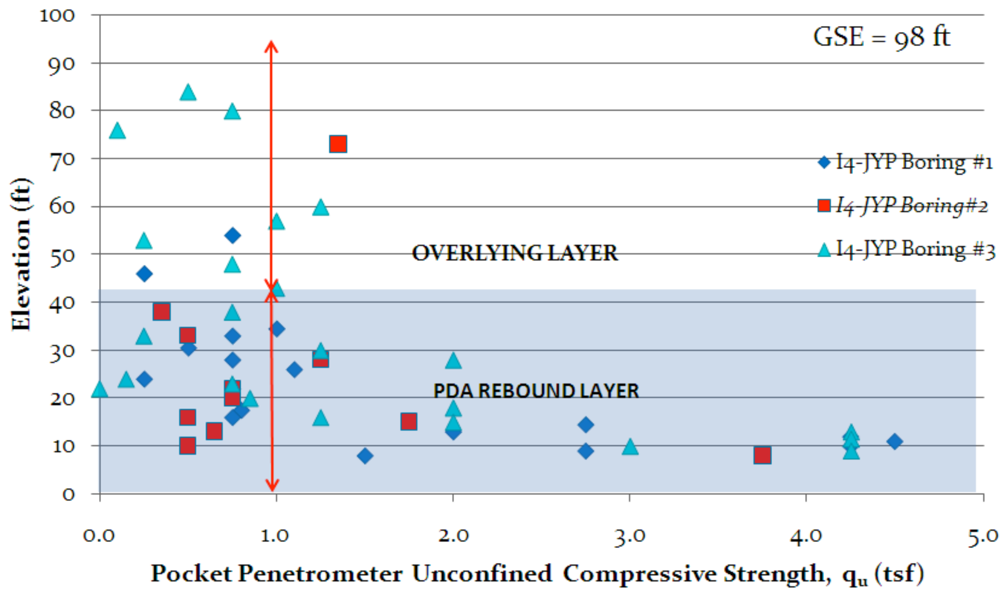
Figure 4.23 is the plot of elevation versus q_u for Ramsey Branch Bridge. Because only one research SPT was performed, fewer values are available. Within the rebound layer q_u ranged from 0.25 tsf to 2.7 tsf (24 to 259 kPa). Above the rebound layer the values range from 0.5 to 1.5 tsf (144 kPa). Overall, there was an increase in the average q_u values between the no rebound and rebound soils, but it was not as large as the increases observed in the Central Florida research sites. Florida Tech researchers performed all pocket penetrometer testing at the Central Florida sites using a similar protocol for all samples, but they were not involved in the Ramsey Branch Bridge testing. This inconsistency in testing procedures might have affected the q_u data from Ramsey Branch Bridge. The average q_u for the overlying and $Rebound_{PDA}$ zones are shown in tabular format for the research phase boring.

The average q_u values increased below the high rebound elevations for all three research sites. The elevation associated with the increase are 20 feet and 28 feet (6.1 and 8.5 m) for Anderson Street Overpass and John Young, respectively, and -25 feet (-7.6 m) for Ramsey Branch Bridge. These depths within the rebound zone are equivalent to 4B for Anderson Street Overpass 6.5B for and John Young and 2.5B for Ramsey Branch Bridge, based on the respective pile diameters.



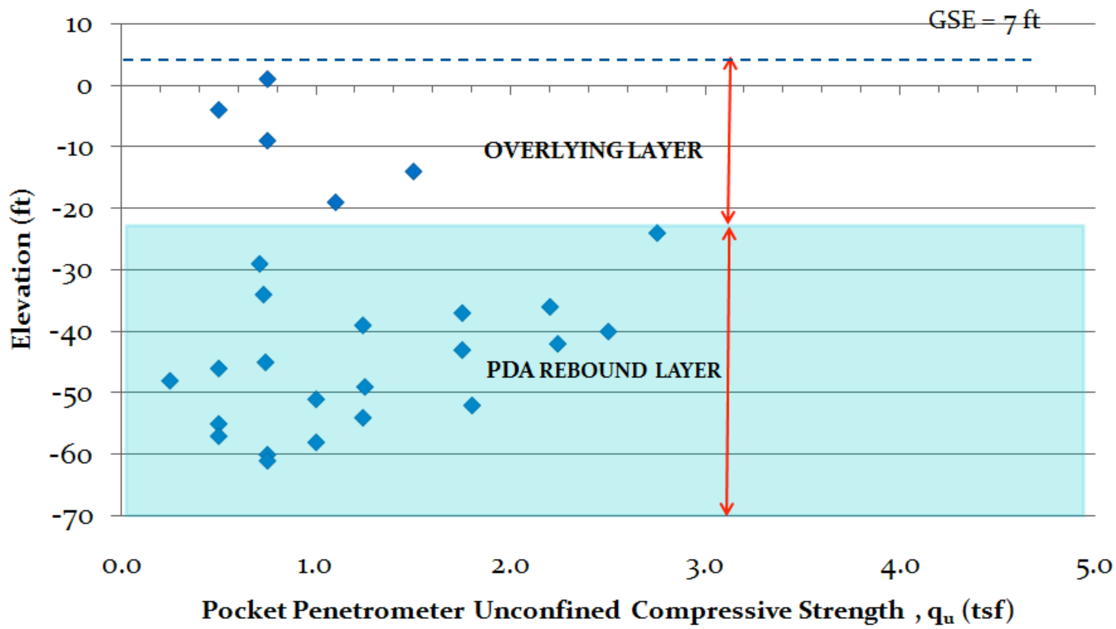
	Boring#1	Boring#2	Boring#3	Boring#4
Overlying Layer Average	0.6	0.6	1.1	NA
Rebound Layer Average	2.7	2.6	2.7	3.6

Figure 4.21 Pocket Penetrometer q_u versus Elevation for Anderson Street Overpass with Tabular Averages



	Boring#1	Boring#3
Overlying Layer Average	0.5	0.7
Rebound Layer Average	1.8	1.8

Figure 4.22 Pocket Penetrometer q_u versus Elevation for John Young with Tabular Averages



	Boring#1
Overlying Layer Average	0.9
Rebound Layer Average	1.3

Figure 4.23 Pocket Penetrometer q_u versus Elevation for Ramsey Branch Bridge with Tabular Averages

Site Name	Overlying Layer Average	Rebound Layer Average	q_u Ratio (rebound/ Overlying)
Anderson Street Overpass	0.8	2.7	3.6
John Young	0.6	1.8	3.0
Ramsey Branch Bridge	0.9	1.25	1.4

Table 4.7 Ratio of Pocket Penetrometer q_u from Rebound Soils over q_u from No Rebound Soils

4.2.2.2.2 Pocket Penetrometer q_u Ratios

The average q_u values determined from the research phase borings, overlying and $Rebound_{PDA}$ zones were evaluated using ratios of the $Rebound_{PDA}$ to the overlying no rebound zone. Table 4.7 presents a summary of these ratios. For Anderson Street Overpass and John Young, these ratios were 3.6 to 3.0, respectively. The 1.4 ratio for Ramsey Branch Bridge was lower, possibly because the soils were much weaker.

In summary, the pocket penetrometer, a quick and inexpensive testing device, does show differences between the $Rebound_{PDA}$ and overly no rebound soils. When the data are presented in terms of a ratio of rebound to no rebound soils, the ratios are similar to the ratios from SPT testing from two of the three sites. It may be possible to use average q_u values from pocket penetrometer data if the testing is performed consistently and sufficient soils can be tested.

4.2.2.3 CPT Results

CPT tests with and without pore pressure measurements were conducted at the three sites as described in Table 3.3. Both the tip resistance (q_c) and the sleeve friction (f_s) data were analyzed. The data were plotted versus elevation with the $Rebound_{PDA}$ zones overlaid onto the plots.

4.2.2.3.1 Tip resistance

4.2.2.3.1.1 CPT q_c versus Elevation

For Anderson Street Overpass, q_c was obtained from five different soundings; two were CPT tests conducted without pore pressure measurements (i.e., CPT 1 and CPT 2) and the remaining three were CPT tests with pore pressure measurements. Figure 4.24 shows the variation of q_c with elevation. The largest variations in q_c occur within both the upper and lower rebound zones. In the overlying layers with no rebound, average q_c values for each sounding ranged from 27 tsf to 87 tsf (2585 to 8331 kPa). In the $Rebound_{PDA}$ layer averages for each sounding ranged from 45 tsf to 138 tsf (43097 to 13215 kPa).

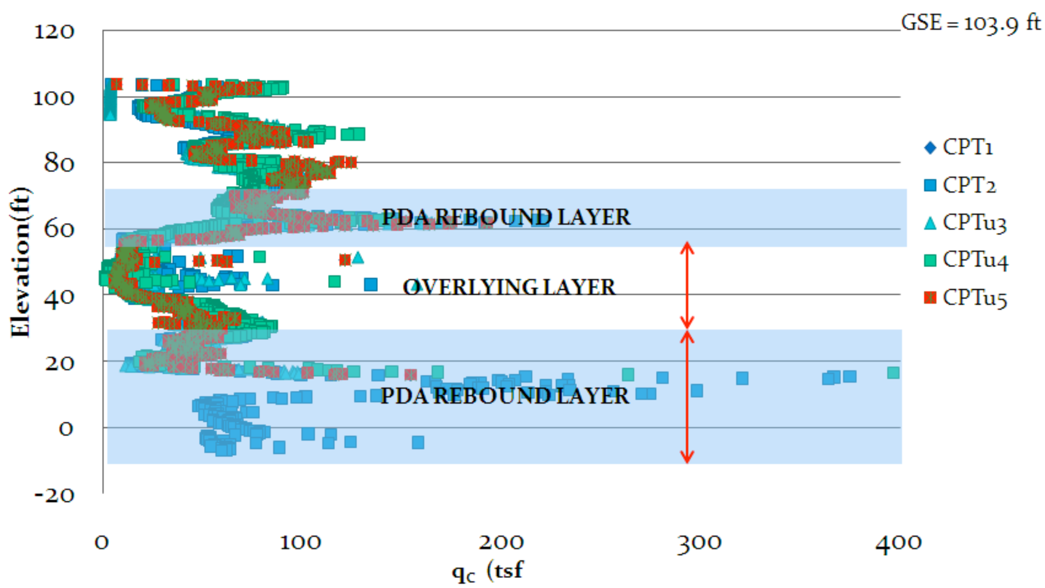
For John Young (Figure 4.25), q_c versus elevation values from the two CPTu tests follow a pattern similar to Anderson Street Overpass with q_c increasing in the upper elevations and the $Rebound_{PDA}$ zone. The q_c values near the surface for each sounding are significantly different. As shown in , CPT2u was conducted between the eastbound and westbound ramps, while CPTu1 was conducted south of both ramps. The additional construction near CPTu2 included about 20 ft (6.2 m) of fill plus pile driving near the soils between the two ramps and most likely increased the density and shear strength near the surface. As a result of these site variations the averages from the overlying no rebound and $Rebound_{PDA}$ zones did not produce expected results. The average q_c in the overlying zone varied from 53 to 65 tsf (4788 to 6224 kPa) while the averages in the $Rebound_{PDA}$ zone varied from 51 to 69 tsf (4884 to 6607 kPa). In order to clarify the data for this site either additional CPT testing should be performed or the data from CPTu2 should be discarded.

Figure 4.26 is the plot of elevation versus q_c from the two soundings at Ramsey Branch Bridge. In both soundings a large variation of q_c occurred in the top 10 feet (3.1 m) indicating the presence of compacted fill near the roadway. Below this depth there was very little tip resistance for nearly 20 feet (6.2 m), followed by an increase to about 60 tsf (5746 kPa) to a final sounding elevation of -65 feet (19.8 m). This upper 10 ft (3.1 m) was excluded from the analysis and average CPT values were determined above elevation - 5 ft (-1.5 m) to - 22 ft (-6.7 m) and below - 22 ft (-6.7 m). In the overlying no rebound zone, CPT q_c values averaged 4 or 8 tsf (383 to 766 kPa), while in the $Rebound_{PDA}$ zone the averages were 45 and 57 tsf (4309 to 5458 kPa).

The average q_c values increased below the high rebound elevations for all three research sites. The elevation associated with the increase were 16 feet and 23 feet (4.91 and 7.0 m) for Anderson Street Overpass and John Young, respectively, and -27 feet (-8.2 m) for Ramsey Branch Bridge. These depths within the rebound zone are equivalent to 5B for Anderson Street Overpass 9B for and John Young and 3.5B for Ramsey Branch Bridge.

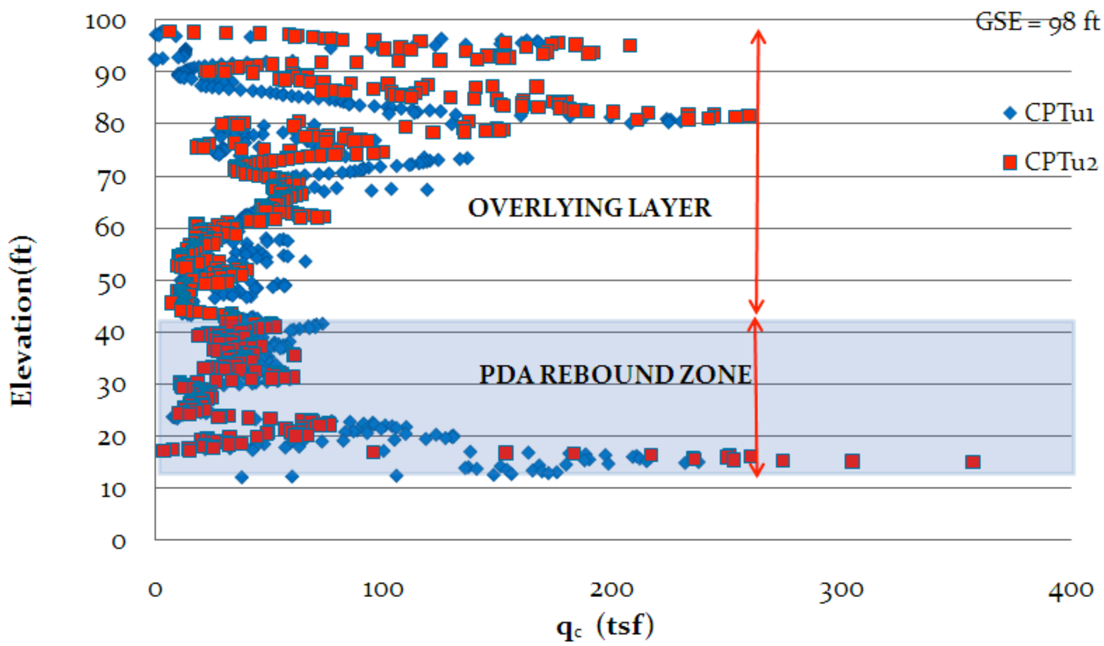
4.2.2.3.1.2 CPT q_c Ratios

Table 4.8 contains a summary of the q_c ratios determined using the $Rebound_{PDA}$ divided by the overlying no rebound zone. For Anderson Street Overpass the ratio was 2.0, while for John Young it was 1.0, and for Ramsey Branch Bridge it was 8.5. These variations may be the results of the zones used for the calculations. The procedure followed to determine the zones for the ratios was based on the PDA results. For Ramsey Branch Bridge the PDA data (See Appendix C) indicated that high pile rebound could have started from -22 ft to - 66 ft (-6.7 to -20.1 m) for the various piles driven. This variation caused the research team to choose the highest elevation for the high pile rebound zone. These ratios are not as consistent as those reported for the N and q_u values; however, they are relatively similar. They may be a useful tool for engineers trying to anticipate high pile rebound, as long as CPT data can be obtained. Additional evaluations should be performed at additional high pile rebound sites to refine this process.



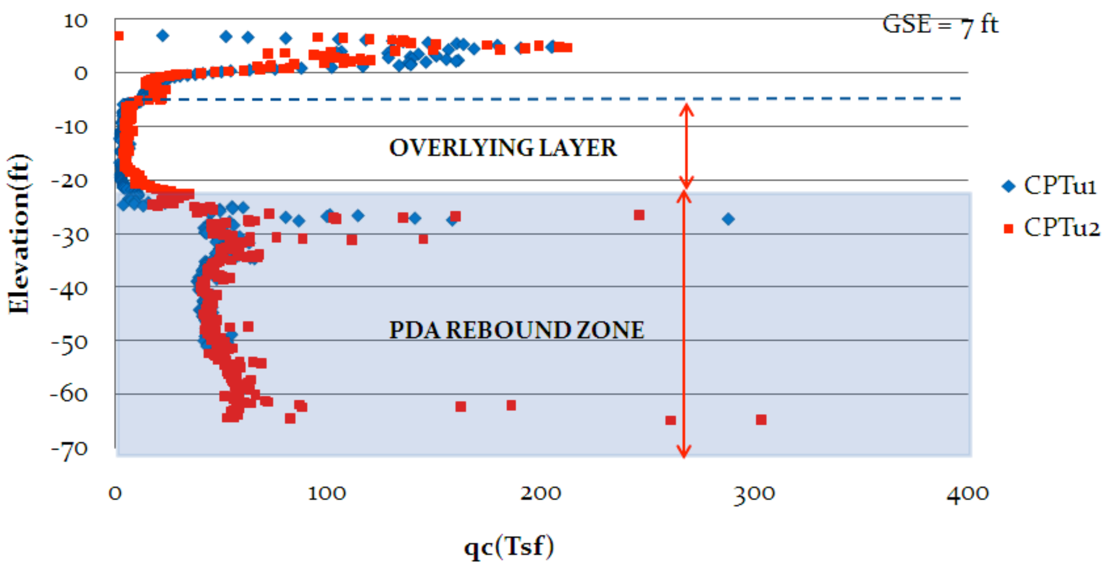
	CPT1	CPT2	CPTu3	CPTu4	CPTu5
Overlying Layer Average	27	38	33	33	87
Rebound Layer Average	48	86	45	111	138

Figure 4.24 Retesting Phase Anderson Street Overpass CPT q_c versus Elevation with Tabular Averages



	CPTu1	CPTu2
Overlying Layer Average	53	65
Rebound Layer Average	69	51

Figure 4.25 Retesting Phase John Young CPT q_C versus Elevation with Tabular Averages



	CPTu1	CPTu2
Overlying Layer Average	4	8
Rebound Layer Average	45	57

Figure 4.26 Retesting Phase Ramsey Branch Bridge CPT q_c versus Elevation with Tabular Averages

Site Name	Overlying Layer Average	Rebound Layer Average	q_c Ratio (Rebound/ Overlying)
Anderson Street	43.6	85.6	2.0
John Young Parkway	59	60	1.0
Ramsey Branch	6	51	8.5

Table 4.8 Retesting Phase Ratios of CPT q_c in Rebound Soils over CPT q_c in No Rebound Soils

4.2.2.3.2 CPT Sleeve Friction

4.2.2.3.2.1 CPT f_s versus Elevation

For Anderson Street Overpass, f_s versus elevation values were obtained from five soundings; two were CPT tests conducted without pore pressure measurements (i.e., CPT 1 and CPT 2) and the remaining three were CPT tests with pore pressure measurements. Figure 4.27 shows the variation in f_s with elevation. In the overlying layers with no rebound, average f_s values for each sounding ranged from 0.33 tsf to 0.85 tsf (32 to 81 kPa). In the $Rebound_{PDA}$ layer, averages for each sounding ranged from 0.84 tsf to 2.70 tsf (80 to 259 kPa).

The f_s versus elevation plot from the two CPTu soundings at John Young shown in Figure 4.28 indicates several increases in f_s throughout the profile. The data from both tests follow similar trends, unlike the q_c plots from this site, which showed higher values for the test between the two I-4 ramps. Friction values increased near elevations 80, 60 and 20 ft (24.4, 18.3, and 6.1 m). In the overlying zones with no rebound, f_s averages were fairly consistent at 0.65 and 0.67 tsf (62 to 64 kPa). Friction values in the $Rebound_{PDA}$ zone average 0.69 and 0.53 tsf (66 and 51 kPa) for CPTu1 and CPTu2, respectively.

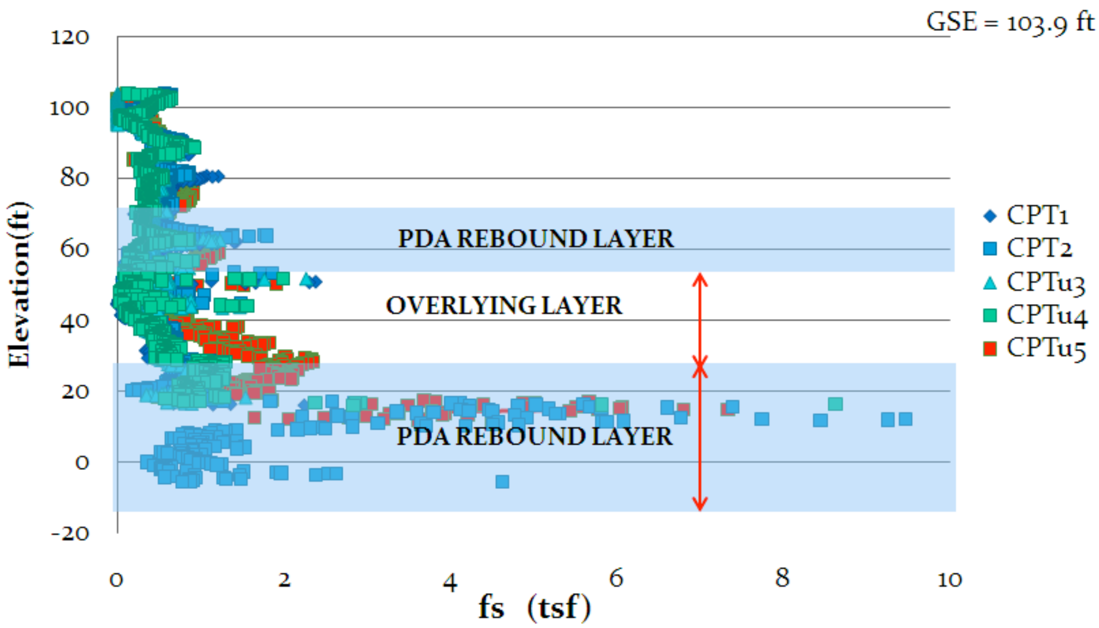
Figure 4.29 is a plot of elevation versus f_s at Ramsey Branch Bridge. As was the case with the q_c data, there are some higher f_s values near the surface, which were not used in the evaluation. From elevation - 5 ft (-1.5 m) to - 22 ft (-6.2 m), f_s averaged 0.1 tsf (9.6 kPa), while below this elevation it averaged 0.85 and 0.90 tsf (81.3 and 86.2 kPa).

The average f_s values increased below the high rebound elevations for all three research sites. The elevation associated with the increase are 16 feet and 20 feet (4.91 and 6.1 m) for Anderson Street Overpass and John Young, respectively, and -25 feet (-7.6 m) for Ramsey Branch Bridge. Converting these elevations into depths within the rebound zone and dividing by the respective pile diameters indicates that they are equivalent to 5B for

Anderson Street Overpass 10.5B for and John Young and 2.5B for Ramsey Branch Bridge.

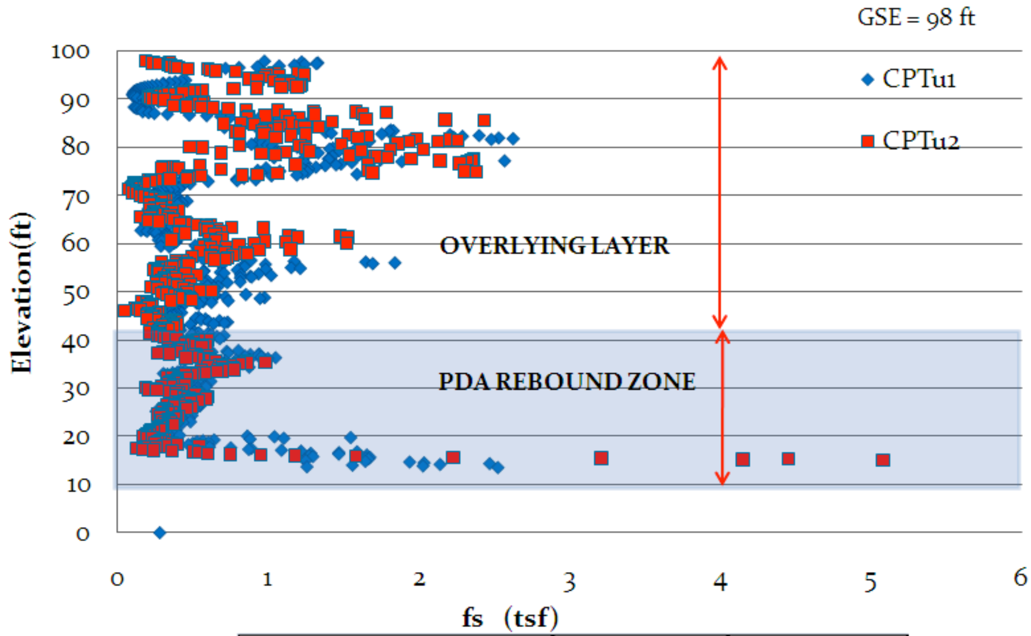
4.2.2.3.2.2 CPT f_s Ratios

Table 4.9 contains a summary of the friction ratios determined using the $Rebound_{PDA}$ divided by the overlying no rebound zone. The values for Anderson Street Overpass and Ramsey Branch Bridge (i.e., 3.5 and 8.8) are much higher than the 0.9 for John Young. These ratios are not very consistent and further studies are needed to produce values that would allow engineers to anticipate high pile rebound.



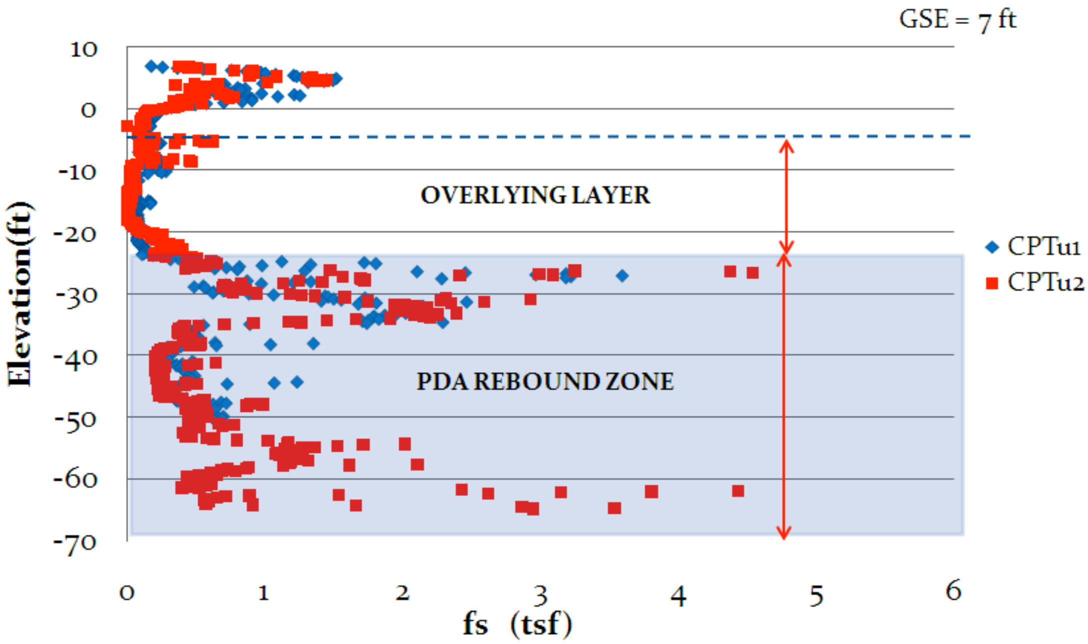
	CPT1	CPT2	CPTu3	CPTu4	CPTu5
Overlying Layer Average	0.33	0.50	0.46	0.40	0.85
Rebound Layer Average	0.84	1.80	0.90	2.70	2.60

Figure 4.27 Retesting Phase Anderson Street Overpass CPT Friction versus Elevation with Tabular Averages



	CPTu1	CPTu2
Overlying Layer Average	0.65	0.67
Rebound Layer Average	0.69	0.53

Figure 4.28 Retesting Phase John Young CPT Friction versus Elevation with Tabular Averages



	CPTu1	CPTu2
Overlying Layer Average	0.10	0.10
Rebound Layer Average	0.85	0.90

Figure 4.29 Retesting Phase Ramsey Branch Bridge CPT Friction versus Elevation with Tabular Averages

Site Name	Overlying Layer Average	Rebound Layer Average	f_s Ratio (Rebound/ Overlying)
Anderson Street Overpass	0.5	1.8	3.5
John Young	0.7	0.6	0.9
Ramsey Branch Bridge	0.1	0.9	8.8

Table 4.9 Retesting Phase Ratios of CPT Friction Values in Rebound Soils to CPT Friction Values in No Rebound Soils

4.2.2.3.3 CPTu Pore Pressure Evaluation

The CPTu soundings from each site were evaluated by plotting the pore water pressure versus elevation. The plots shown in Figure 4.30, Figure 4.31, Figure 4.32, indicate negative pore water pressures were encountered during CPTu testing at all three sites. The - 10 psi (-70 kPa) negative pore water pressures encountered at Anderson Street Overpass and John Young were significantly larger than the - 1.0 psi (-7 kPa) for Ramsey Branch Bridge. The zones associated with high pile rebound are also shown on the figures and for the Anderson Street Overpass and John Young sites the pore water pressures vary from negative to positive near these boundaries.

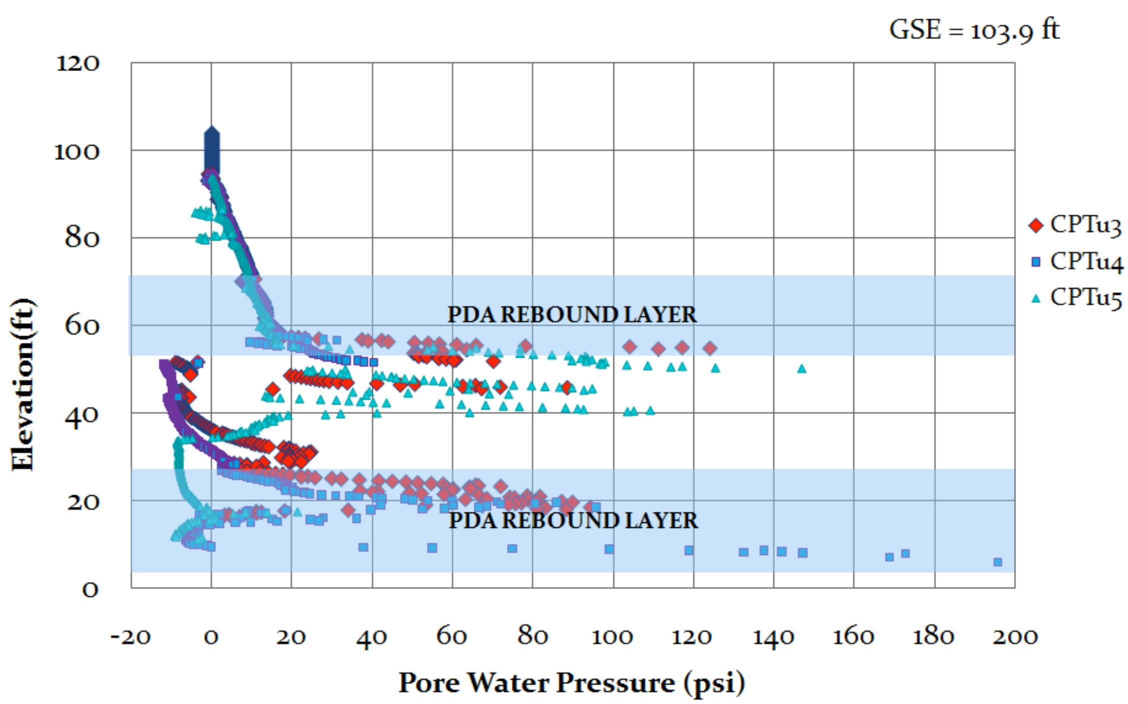


Figure 4.30 Research Phase: Anderson Street Overpass CPTu Pore Water Pressure versus Elevation

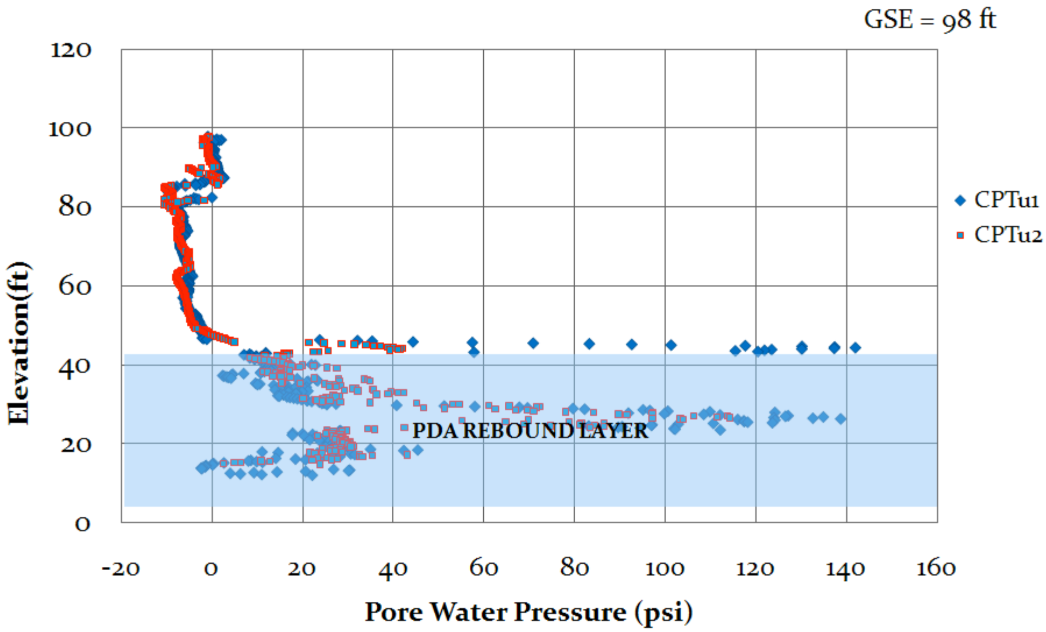


Figure 4.31 Research Phase: John Young CPTu Pore Water Pressure versus Elevation

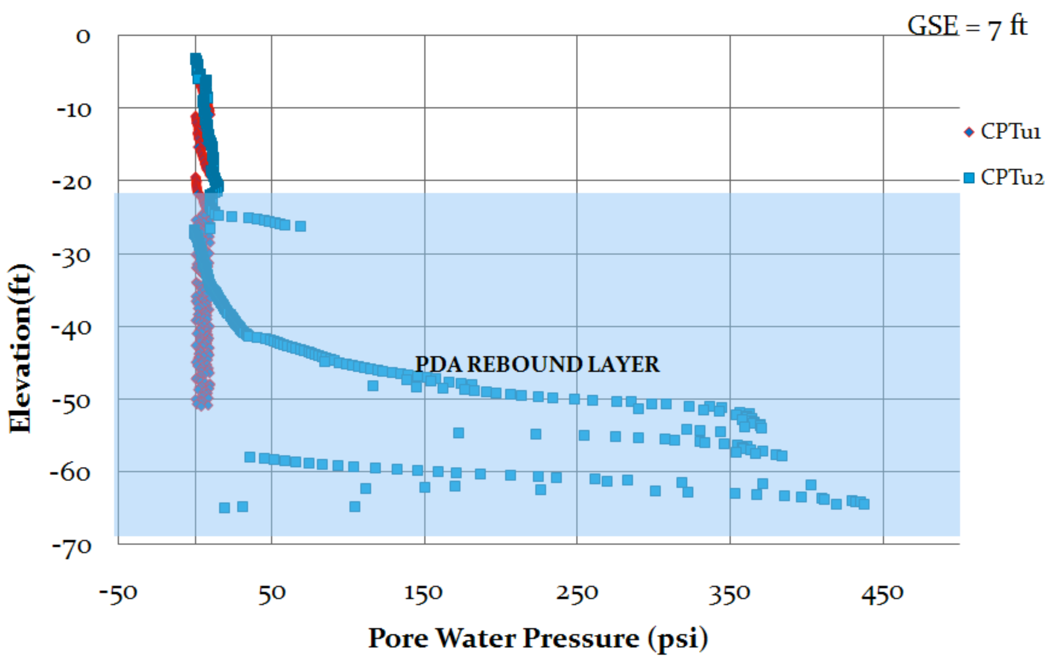


Figure 4.32 Research Phase: Ramsey Branch Bridge CPTu Pore Water Pressure versus Elevation

The pore pressures at the Anderson Street Overpass site increased from an average in the overlying zone of -10 psi (70 kPa) to an average in the $Rebound_{PDA}$ zone of about 40 psi (280 kPa), with the exception of the data from CPTu5. This sounding was conducted about 100 feet (30.5 m) from the other two CPTu soundings. Its data indicate that the negative pore pressures near -10 psi (-70 kPa) existed from elevation 30 ft (9.1 m) to the end of the sounding, with the exception of positive values reaching 20 psi (140 kPa), near elevation 15 ft (4.6 m). The pore pressures at the John Young site increased from an average in the overlying zone of -10 psi (70 kPa) to an average in the $Rebound_{PDA}$ zone about 30 psi (210 kPa). The pore pressures at the Ramsey Branch Bridge site increased from an average in the overlying zone of -1 psi (7 kPa) to an average in the $Rebound_{PDA}$ zone about 175 psi (1225 kPa).

In summary, there was a large increase from near zero or negative to positive pore water pressures in all the high pile rebound zones. This increase in conjunction with variations in the strength, stiffness and soil composition may be the combination of geotechnical properties that would indicate high pile rebound.

4.2.2.4 Elastic Modulus Evaluation

PPMT, DMT and CU triaxial tests results were analyzed to enable elastic moduli determinations. The elastic moduli (E) were plotted versus elevation for all three research sites. No one test produced a complete profile for E versus elevation; therefore, the evaluation was completed by combining moduli from PPMT and DMT field and CU triaxial lab results.

4.2.2.4.1 PPMT Results

The pressuremeter testing data, which is included in the appendices, was reduced and plotted as pressure versus injected volume for each test. A typical plot, shown in Figure 3.1 enables elastic moduli to be determined at various volume increments along the curve (Cosentino et al., 2006). The volumes are converted to strains by dividing them by the initial volume of the pressuremeter probe (i.e., approximately 200 cm^3). Each PPMT test included a complete loading curve with one unload-reload cycle plus several unloading points once the maximum volume was injected. Elastic moduli were determined from three portions of the curves. Elastic moduli related to relatively large strain were determined from the initial slope and the reload slope of the unload-reload portion of the test, while elastic moduli associated with smaller strains were determined from each unloading point after the maximum volume was injected (See Figure 3.1). This analysis produced a series of elastic moduli and associated strains.

Kondner (1963) evaluated the stress-strain curve of cohesive soils assuming they could be simulated mathematically by a hyperbolic shaped curve. He then used a secant modulus from the origin to a desired strain, to develop a linear relationship between 1/E and strain as shown in Figure 4.33, with an intercept labeled as a and a slope labeled as b. The resulting equation becomes $1/E=a+b\varepsilon$. PPMT elastic moduli and strains were plotted with 1/E on the y-axis and strain on the x-axis. These plots were evaluated to develop a and b parameters. The resulting equation was used to estimate Young's Modulus of the soil at three different strain levels (i.e., 1, 3, and 5 percent) that could be related to moduli from the CU triaxial tests.

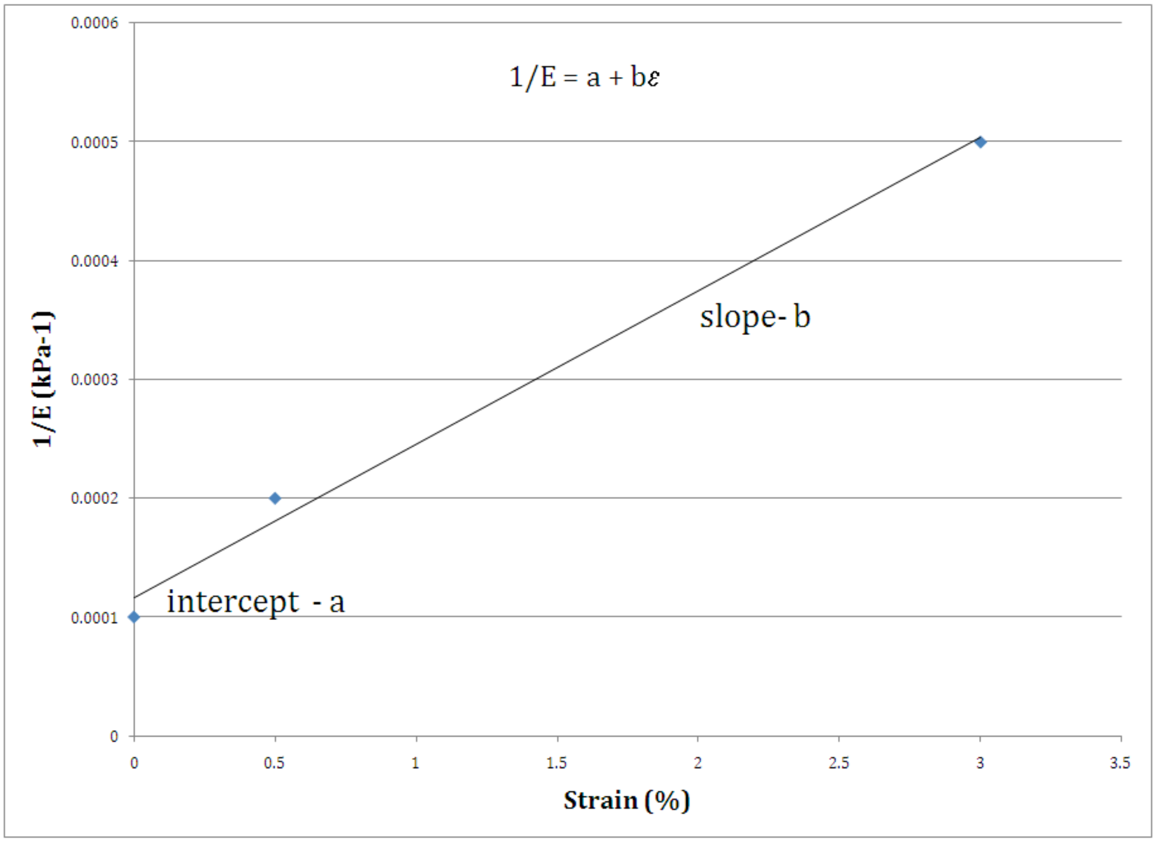


Figure 4.33 Typical 1/E versus strain plot based on the Kondner (1963) model

The estimates of E for each strain level were then used to develop moduli versus elevation plots using the elevations associated with each PPMT test. Only one PPMT test was performed at Anderson Street Overpass; therefore, no plot versus elevation was developed. The elastic moduli at 1, 3 and 5 percent strains for the one test at elevation 3.5 ft (1.1 m) are 1507, 1684, 1909 psi, (10526, 11765, and 13333 kPa) respectively.

Figure 4.34 and Figure 4.35 show the E versus elevation plots for John Young and Ramsey Branch Bridge. Only John Young produced data both above and below the $Rebound_{PDA}$ zone. This plot does show trends that are similar to those found in the N and CPT plots. Higher moduli were determined near elevation 80 ft (24.4 m) and within the $Rebound_{PDA}$ rebound zone near elevation 10 ft (3.1 m). The increase in stiffness at this lower elevation corresponds to 15.5 B [i.e., 31 ft (9.5 m)] into the high pile rebound zone. The plots from Ramsey Branch Bridge show a large increase in the elastic moduli at 5 percent strain near elevation - 57 ft (17.4 m).

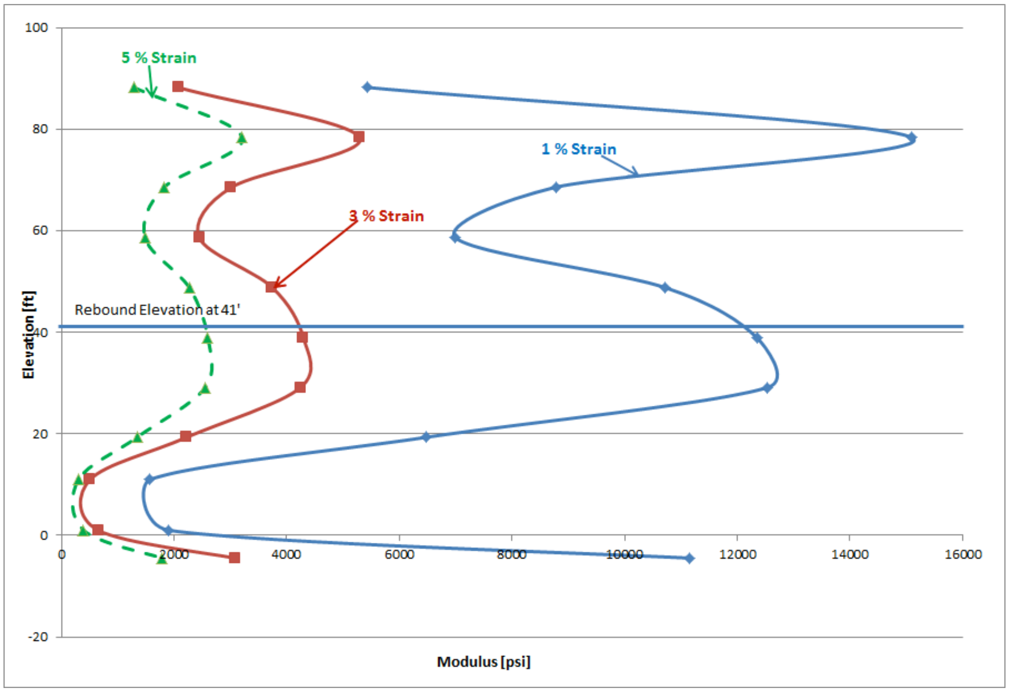


Figure 4.34 John Young Average PPMT Elastic Modulus versus Elevation

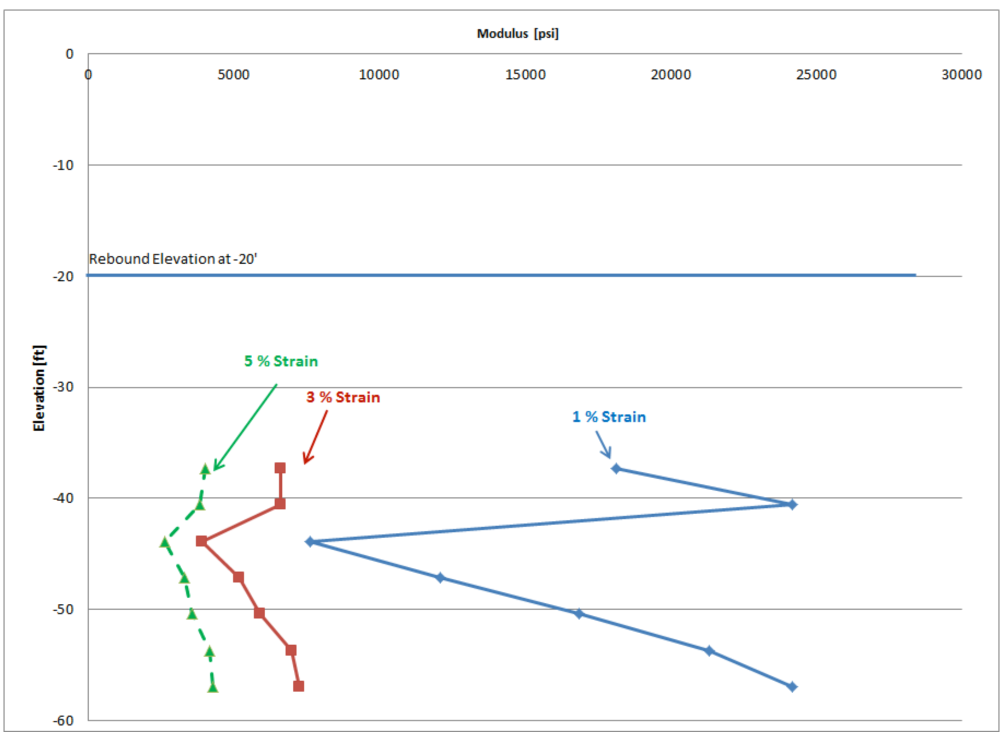


Figure 4.35 Ramsey Branch Bridge PPMT Elastic Modulus versus Elevation

The effect of strain level on E indicates that larger variations in moduli versus depth would be evident if the one (1) percent strains were considered instead of the higher percent strains. The plots for both John Young and Ramsey Branch Bridge show a large change in E, at one (1) percent strain within the rebound soils.

Based on the moduli versus elevation plots and the PDA rebound zones, ratios of average moduli from $Rebound_{PDA}$ zones divided by averages from the overlying no rebound zones were developed. Only one PPMT test was conducted at Anderson Street Overpass; therefore, no ratios were produced for this site. Also no tests were conducted in the overlying no rebound zone at Ramsey Branch Bridge; therefore, ratios were only determined for John Young. The ratios, presented in Table 4.10, show an average of 0.8, which is much lower than the ratios presented in the previous sections. If additional sites can be analyzed with the PPMT or the standard sized PMT, this data may be developed so that it could be used to help engineers anticipate high pile rebound.

Site*	Description	Elevation (ft)	$E_{1\%}$	$E_{3\%}$	$E_{5\%}$	E_{ave}
John Young	No Rebound	Above 41	9405.6	3315.4	2015.2	4912.1
	Rebound	Below 41	7666.3	2502.5	1498.1	3889.0
	Rebound/No Rebound		0.8	0.8	0.7	0.8

** - Data unavailable in no rebound zone at Anderson Street Overpass and Ramsey Branch Bridge*

Table 4.10 Retesting Phase PPMT Ratios in Rebound to No Rebound Soils

4.2.2.4.2 DMT Results

The reduced DMT data, provided by FDOT SMO, were used to develop elastic moduli versus elevation plots for Anderson Street Overpass and John Young. No DMT soundings were conducted at Ramsey Branch Bridge. Figure 4.36 and Figure 4.37 are the plots for Anderson Street Overpass and John Young, respectively. These plots do not clearly show increases at the rebound layers. Only one DMT test was taken in the lower rebound zone at Anderson Street Overpass because the equipment was not able to be pushed through this stronger soil. For John Young, there was an increase in the elastic moduli just above the rebound zone at elevation 43 feet (13.1 m) from about 5000 psi (35,000 kPa) to about 8000 psi (56,000 kPa).

The ratios of E in the rebound zone to the no rebound zone were calculated based on two DMT soundings at John Young. For Anderson Street Overpass, the upper rebound zone and the soil above were not included in order to consistently evaluate the zones where piles were unable to penetrate and achieve design capacities. These ratios are presented in Table 4.11. Results showed that E_{ave} in the rebound zone divided by E_{ave} in the overlying no rebound zone had a ratio of 1.2, which is closer to the ratios presented prior to the PPMT ratios. With more testing sites this may be developed into a possible high pile rebound soil indicator.

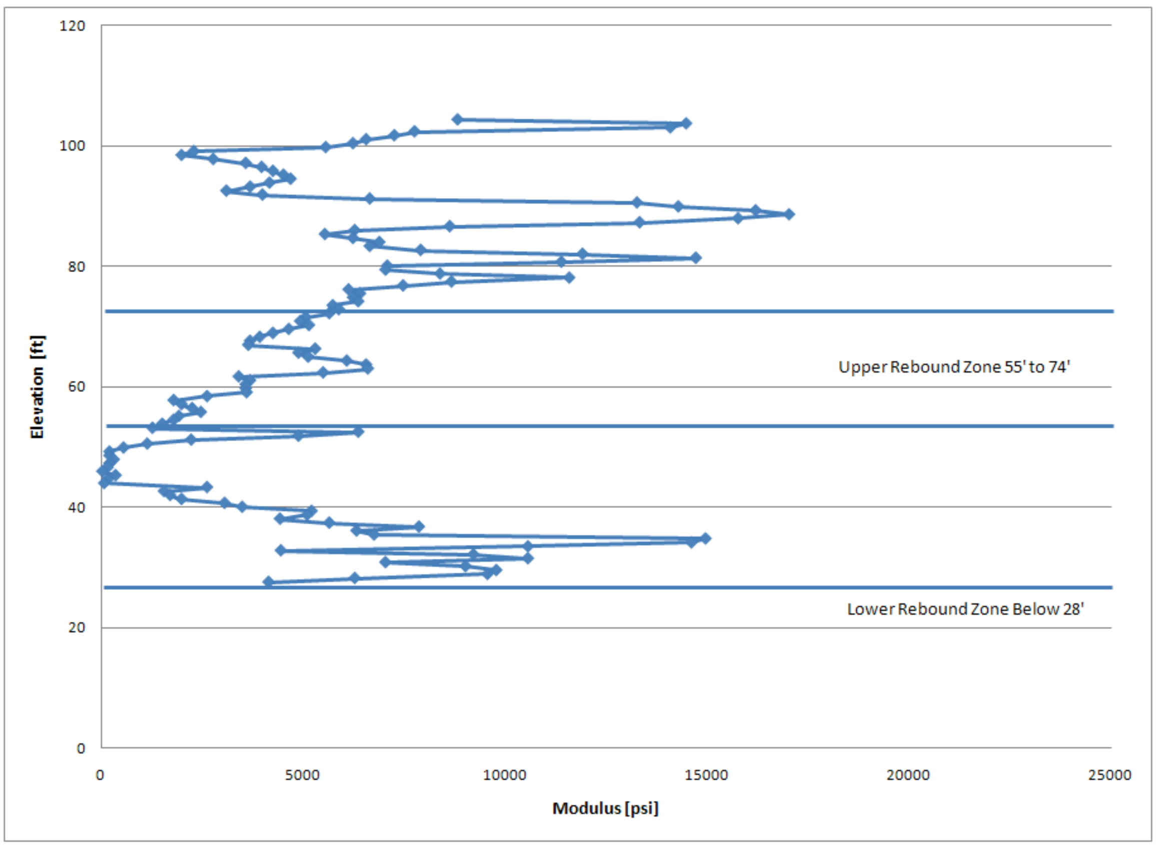


Figure 4.36 Anderson Street Overpass Average DMT Elastic Modulus versus Elevation

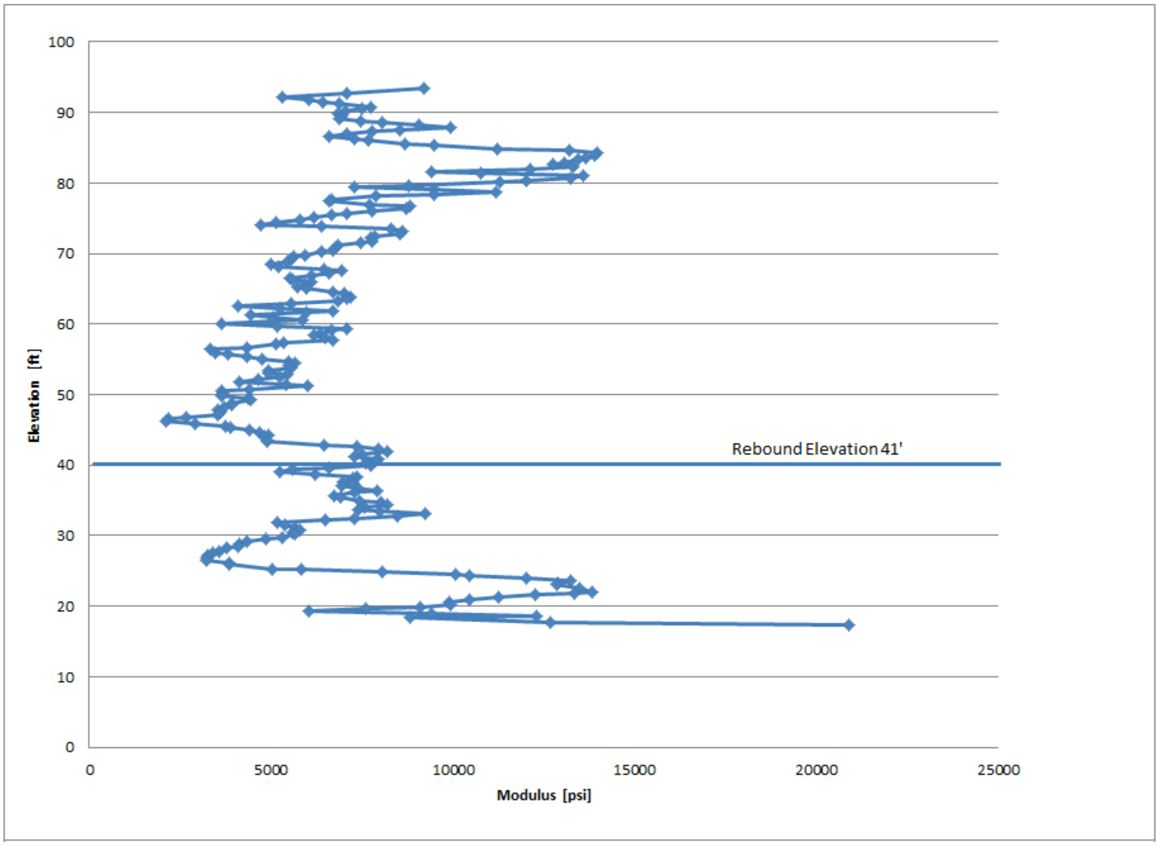


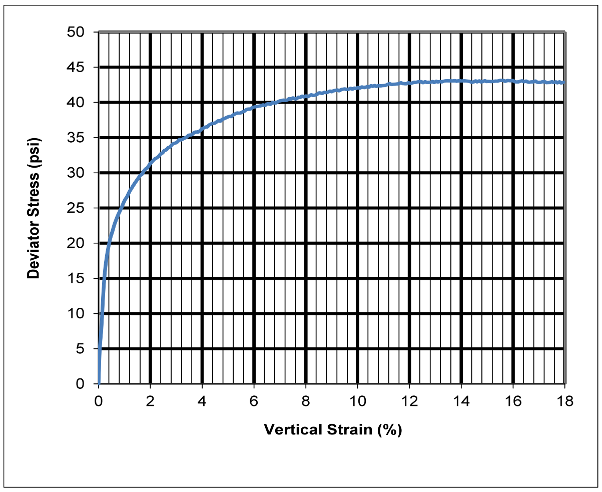
Figure 4.37 John Young Average DMT Elastic Modulus versus Elevation

Site*	Description	Elevation (ft)	E_{ave} (psi)
Anderson Street Overpass	No Rebound	Above 28	4415.6
	Rebound	Below 28	5201.8
	Rebound/No Rebound		1.2
John Young	No Rebound	Above 41	6753.1
	Rebound	Below 41	7787.0
	Rebound/No Rebound		1.2
** Ramsey Branch Bridge not tested*			

Table 4.11 Retesting Phase DMT Ratios in Rebound to No Rebound Soils

4.2.2.4.3 CU Triaxial Results

The CU triaxial data were reduced to stress versus strain and excess pore water pressure versus strain plots. A typical set of these plots is shown in Figure 4.38. In general, the pore pressures peaked between 1 and 3 percent strain then decreased. The pore water pressures ranged from +20 to -65 psi (+140 to - 455 kPa) when the tests were completed. The triaxial and excess pore pressure versus strain data from the testing at all three sites are contained in the appropriate appendix (i.e., Appendix A for Anderson Street Overpass, etc.) The ratios between the peak excess pore pressure and the pore pressure at 15 percent strain were determined and evaluated. Since there are positive and negative values the ratios cannot be easily used by engineers to predict or anticipate high pile rebound.



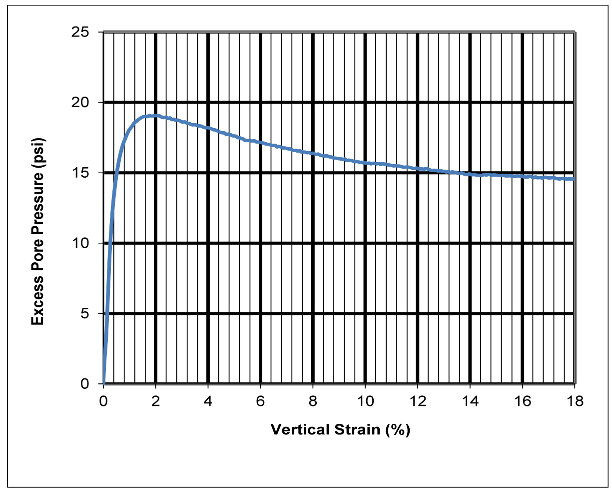


Figure 4.38 Typical CU Triaxial Results with Deviator Stress and Excess Pore Pressure versus Strain

The secant moduli were determined at 5, 10 and 15 percent strains. The data were evaluated by plotting the secant moduli versus elevation for each site with the rebound elevations given as shown in Figure 4.39, Figure 4.40and Figure 4.41. Values from the overlying no rebound zones and the $Rebound_{PDA}$ zones were averaged and ratios of the $Rebound_{PDA}$ over the no rebound values were calculated as shown in Table 4.12.

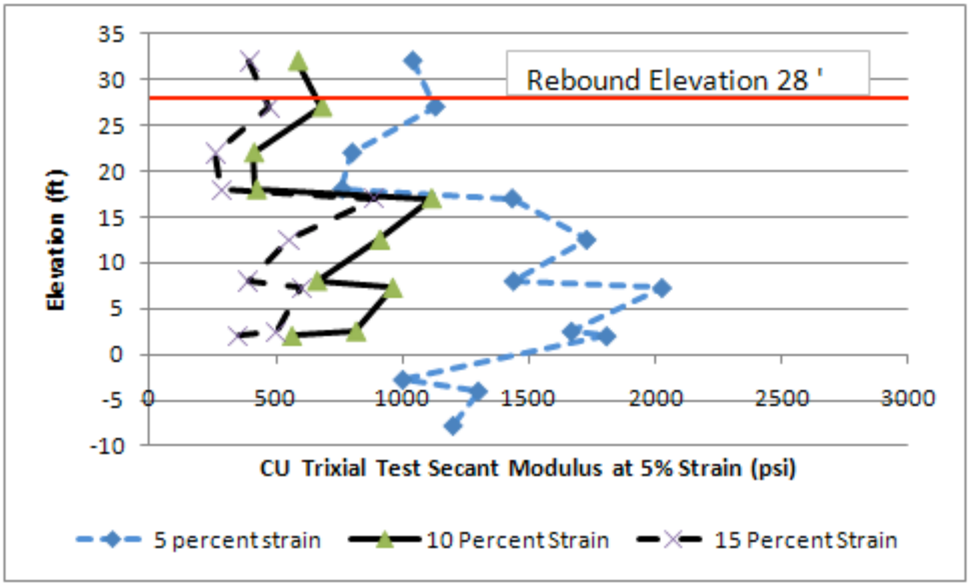


Figure 4.39 Anderson Street Overpass CU Triaxial 5, 10 and 15 percent Secant Modulus versus Elevation

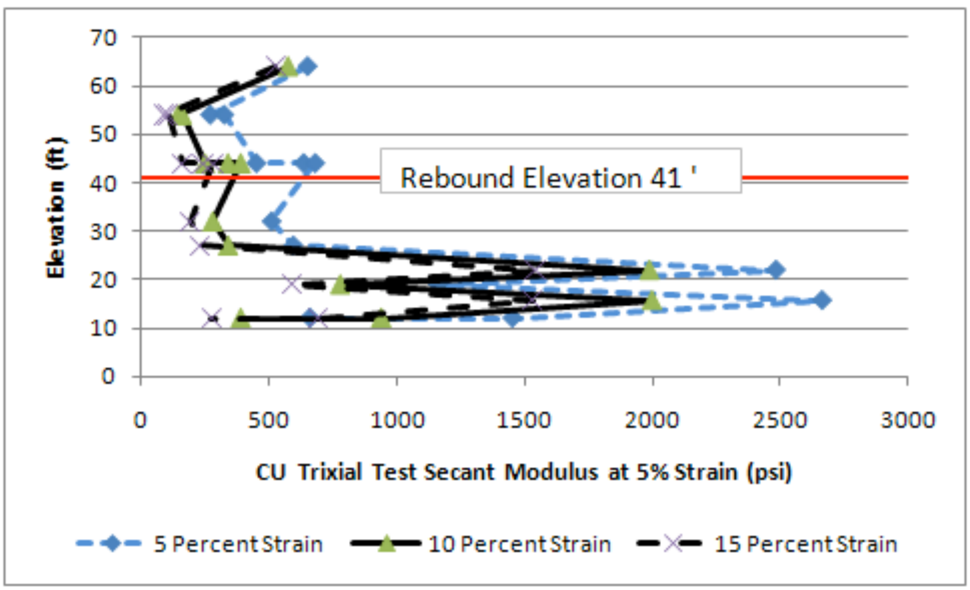


Figure 4.40 John Young CU Triaxial 5, 10 and 15 percent Secant Modulus versus Elevation

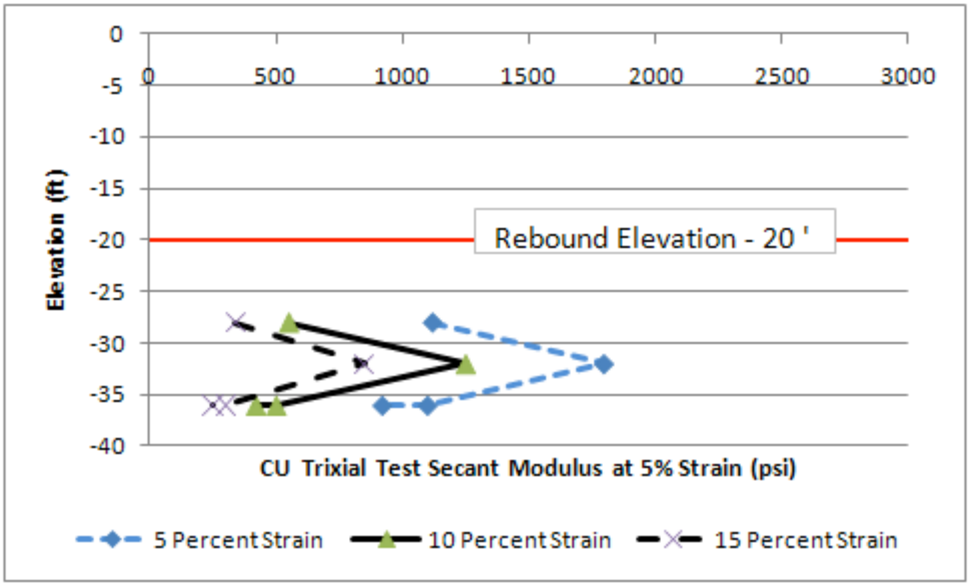


Figure 4.41 Ramsey Branch Bridge CU Triaxial 5, 10 and 15 percent Secant Modulus versus Elevation

Site*	Description	Elevation (ft)	Secant at 5% (psi)	Secant at 10% (psi)	Secant at 15% (psi)
Anderson Street Overpass	No Rebound	Above 28	1040.0	585.0	395.0
	Rebound	Below 28	1370.6	725.6	477.8
	Rebound/No Rebound		1.3**	1.2**	1.2**
John Young	No Rebound	Above 41	501.7	310.0	233.3
	Rebound	Below 41	1359.3	960.0	720.7
	Rebound/No Rebound		2.7	3.1	3.1
** - Data unavailable in no rebound zone at Ramsey Branch Bridge*					
***- Ratio based on 1 point in No Rebound Zone*					

Table 4.12 Retesting Phase CU Triaxial Moduli Ratios of Rebound to No Rebound Soils

The ratios shown in Table 4.12 are similar to those found from the majority of other results. The 1.2 ratio for the Anderson Street Overpass site, which is based only one point in the overlying no rebound zone, is lower than the ratio of nearly 3.0 for the John Young site. The Anderson Street Overpass ratio is the same as the ratio determined from DMT testing, but the John Young ratio from DMT testing is lower.

4.2.2.4.4 Elastic Moduli Ratios

The average elastic moduli from PPMT, DMT and CU Triaxial testing are shown in Table 4.13. No ratios were compiled for Ramsey Branch Bridge since data was unavailable in the no rebound soils. Also PPMT data was unavailable in the no rebound zone at the Anderson Street Overpass. In general, the ratios are similar to the ratios from the other testing, falling between 1.2 and 2.9. Anderson Street Overpass has a lower average ratio of 1.4 than John Young at 2.1. Although their ratios are similar, the elastic modulus is a stiffness parameter while the N, CPT, qu values are strength parameters. Determining elastic moduli of soils is typically more difficult and costly than other parameters; therefore, this may not be the most economical method available to engineers for predicting high pile rebound.

Site Name	Elastic Modulus Ratios (rebound/ no rebound)					
	PPMT 5% Strain	DMT	CU 5%	CU 10%	CU 15%	Average
Anderson Street	N/A	1.2	1.5	1.4	1.4	1.3
John Young Parkway	0.7	1.2	2.7	2.9	2.7	2.0
Ramsey Branch	N/A	N/A	N/A	N/A	N/A	N/A
N/A = data not available						

Table 4.13 Retesting Phase Elastic Moduli Summary Ratios of Rebound Soils to No Rebound Soils

4.2.2.5 Permeability Rebound Evaluation

The CU triaxial testing was used to determine permeabilities from the data at each site. This data was plotted versus elevation as shown in Figure 4.42 which shows a larger variation in permeability for the soils encountered at John Young than the other sites.

To enhance the differences between the permeability data from the three sites, the pile rebound in inches was multiplied by the hammer blow count in blows per foot and the resulting values were plotted versus the corresponding permeabilities as shown in Figure 4.43. This data shows that the John Young data generally falls below a rebound times blow count of 10. The two points from this site with y-axes values over 100 are from the deepest samples. The data from Anderson Street Overpass and Ramsey Branch Bridge generally show an increase in the y-axis values once permeabilities decreased below 10^{-5} cm/sec.

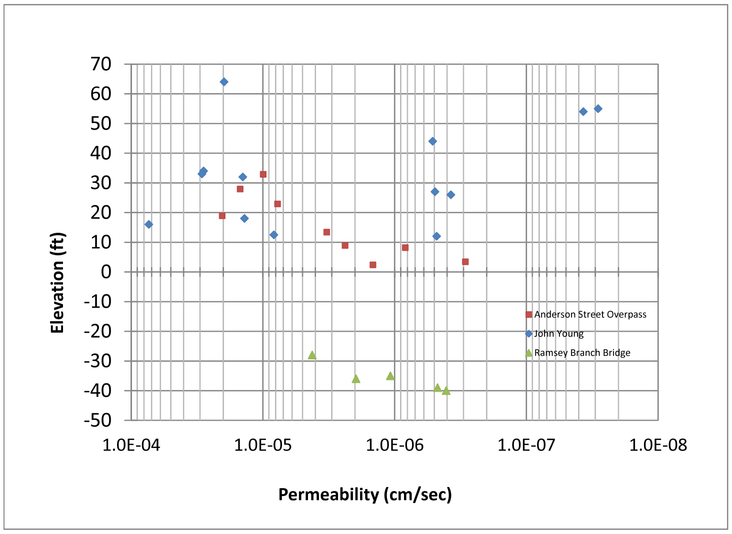


Figure 4.42 Permeability from CU Triaxial Testing versus Elevation

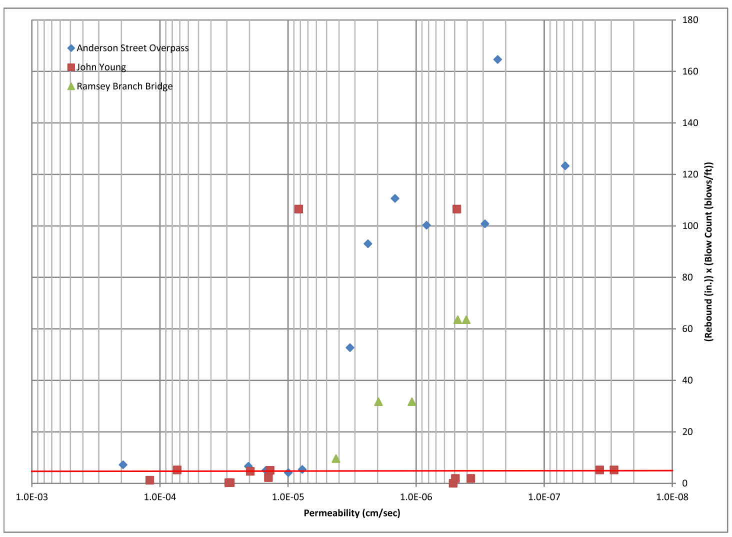


Figure 4.43 CU Triaxial Permeability versus Rebound times the Pile Driving Hammer Blow Count

In summary, the permeability variations at John Young are larger than the variations from the other sites. The enhanced plot shows these variations more clearly than the permeability versus elevation plot.

4.3 Summary of Rebound to No Rebound Ratios

Table 4.14 contains a summary of all the ratios compiled from the geotechnical information at the three research sites. It includes the average, standard deviation and the resulting range for each parameter. The ratios with the lowest standard deviations are based on laboratory testing. The ratios determined from CPT testing had the highest standard deviations.

	Lab Testing Data				SPT Data	CPT Correlations		Pocket Penetrometer
Site Name	Silt	Sand	Elastic Modulus	(Wn/LL)	N_{Raw} *	Tip Resistance (q_c)	Local friction (f_s)	Unconfined compressive strength (q_u)
Anderson Street Overpass	1.4	0.9	1.3	0.6	3.2	2.0	3.5	3.6
John Young	2.0	1.1	2.1	0.9	2.8	1.0	0.9	3.0
Ramsey Branch Bridge	2.2	0.8	N/A	1.0	5.1	8.5	8.8	1.4
Average	**1.9**	**0.9**	**1.7**	**0.8**	**3.7**	**3.8**	**4.4**	**2.6**
Std. Deviation	0.4	0.2	0.6	0.2	1.2	4.1	4.0	1.1
Range	**1.4-2.3**	**0.8-1.1**	**1.2-2.3**	**0.6-1.0**	**2.5-4.9**	**0.0-7.9**	**0.4-8.4**	**1.5-3.8**
* - N values from Design Phase and Research Phase Borings combined								

Table 4.14 Retesting Phases Summary Ratios of Rebound Soils to No Rebound Soils

Increases in the stiffness and strength based parameters were expected and found as shown by the elastic moduli, N, q_c, f_s and pocket penetrometer q_u ratios. There was also an increase in the silt content similar to the increases from the strength and stiffness parameters. Although the silt content increased, the corresponding sand content did not consistently decrease as would be expected, indicating that it may need further evaluation. The ratio between the natural moisture content and liquid limit was based on a limited number of data points and also needs further evaluation. The elastic moduli ratios, developed from combining the PPMT, DMT and CU triaxial test results, were the most difficult to determine.

Based on the relatively large standard deviations from CPT data and the inconsistencies from the sand, elastic modulus and natural moisture-liquid limit data, the ratios recommended for use during the design phase evaluation are the silt content, the SPT N-values and the Pocket Penetrometer q_u. Depending upon the soil encountered within the profile, the ratios from CPT testing may also be helpful, but CPT data may not be obtainable if the soils are firm cohesive soils or very dense granular soils. The ratios not recommended for use during the design phase evaluation are the sand content, elastic modulus, liquid limit and natural moisture content.

In general, the ratio of silt content in the rebound to no rebound soils remained near 2.0 and the combined ratios for N-raw were about 3. The ratios associated with the CPT tip and friction sleeve data were between 3.8 and 4.4. The ratio from the pocket penetrometer averaged 2.7 but varied between 1.6 and 3.8. The standard deviations for these parameters were relatively small, with the exception of the pocket penetrometer data.

The ratios did not account for the zone of influence associated with the pile tip during driving; they were based solely on the high rebound zones developed from the PDA data. This zone could be from eight diameters above the tip to four diameters below the tip.

4.3.1 Summary of Zone of Influence Effects

Table 4.15 is a summary of the zone of influence, in terms of pile diameters (B), between the elevations at the start of high pile rebound to the elevations where each test parameter (silt content, N, etc.) increased. The number of pile diameters into the high rebound deposit before the soil property increases are shown for each site. The piles at Anderson Street Overpass and John Young were both 24-inch (600 mm) diameter piles while those installed at Ramsey Branch were 18 inches (450 mm) in diameter. In general, the pile diameters before the increase for John Young are higher than for the other sites with a range from 5.5 to 13 B. The results from Anderson Street Overpass are consistently about 5B while the Ramsey Branch Bridge results varied from 3.3 to 6.7 B. The larger values from John Young may be associated with the multiple layers and the large variations in permeability within the soil profile.

In conclusion, there is a difference between the elevation where the pile begins to rebound and the elevation at which each test parameter increases, indicating that displacement piles begin to rebound above the soils causing high pile rebound. Overall there is a range from 3.3 to 13 pile diameters into the deposit with an average between 4.8 and 7.8 pile diameters. This difference is similar to the depths into the high pile rebound zone at which the piles bounced (i.e., 9.5 and 7.7 B) at Anderson Street Overpass and John Young (see Table 4.2).

Engineering Parameter	Test Required	Pile Diameters (B) into deposit before increase		
		Anderson Street Overpass	John Young	Ramsey Branch Bridge
Silt Content	Hydrometer Analysis	5	5.5	N/A
N	SPT	5	7.5	6.7
N_{60}	SPT	5	10.5	6.7
q_u	Pocket Penetrometer	4	6.5	3.3
q_c	CPT	5	9	4.7
f_s	CPT	5	10.5	3.3
E	PPMT*	N/A	13	N/A
E	DMT*	N/A	0	N/A
	Average	4.8	7.8	4.9
** Average excluding PPMT and DMT*			*8.3*	

Table 4.15 Pile Diameters into High Rebound Deposit before Increase in Soil Property

4.4 Baseline Values for Rebound to No Rebound Ratios

The ratios determined from the testing sites were calculated using the high rebound soils baseline values which are summarized in Table 4.16. These values could be used by engineers to determine if and when ratios should be determined. The results suggest that ratios from the recommended silt, N and pocket penetrometer parameters could be critical if the silt content ranges from 16 to 20 percent, the N_{raw} values range from 28 to 38 and the pocket penetrometer based q_u values range from 1.2 to 2.7 tsf (115 to 258 kPa).

Site Name	Lab Testing Data					SPT Data	CPT Correlations		Pocket Penetrometer
	Silt Content	Sand Content	Elastic Modulus (psi)	Wn (%)	LL (%)	N_{Raw} * (Blows/ft)	Tip Resistance (q_c) (tsf)	Local friction (fs) (tsf)	Unconfined compressive strength (q_u) (tsf)
Anderson Street Overpass	19	70	2586	37.0	59.0	32.0	85.0	1.8	2.7
John Young	16	80	4186	29.0	34.0	29.0	60.0	0.6	1.8
Ramsey Branch Bridge	20	60	4977	39.0	39.0	39.0	51.0	0.9	1.3
Average	**18.3**	**70**	**3916**	**35**	**44**	**33.3**	**65.3**	**1.1**	**1.9**
Std. deviation	2.1	10	1218	5.3	13.2	5.1	17.6	0.6	0.7
Range	**16-20**	**60-80**	**2700-5130**	**30-40**	**30-58**	**28-38**	**47-83**	**0.5-1.7**	**1.2-2.7**

* N-values from Design Phase and Research Phase Borings combined

Table 4.16 Baseline Values for determining Ratios of Rebound to no Rebound Soils

SPT N values increased to over 50 blows per foot near the bouncing elevations. The silt content increased to over 20 percent at about this same elevation. The pocket penetrometer unconfined compression results increased to 2.9 tsf (278 kPa) also at about this same elevation. CPT point bearings values increased to over 200 tsf (19,150 kPa), while sleeve friction values increased to over 3 tsf (287 kPa) at about this elevation. In the soils overlying the rebound zone, the CPT data produced negative pore water pressures, which increased to positive values in excess of 100 psi (700 kPa), again near the bouncing elevations. These variations in pore water pressures in combination with the increased stiffness and high silt contents in saturated soils could be the geotechnical conditions that would produce high pile rebound.

4.5 Statistical Analysis of Rebound at the Retesting Sites

Using SPSS (Statistical Package for the Social Sciences) software, statistical relationships were developed to predict the probability of high pile rebound using geotechnical data. The statistical analyses were performed using the laboratory and field data collected during the research phase testing.

4.5.1 Logistic Regression Description

Logistic regression is used to predict the probability of an event occurring. This statistical method requires a binary dependent variable with $0 \Rightarrow$ False and $1 \Rightarrow$ True and a set of independent variables. For example, in the medical field, logistic regression is used to identify if a patient suffers from an illness. The dependent variable in this case is assigned a value of 1 for the patient suffering the illness and 0 for a healthy patient. The independent variables can be continuous, discrete, categorical, or binary as long as they are related to the illness (Ioannou, 2006).

The binary dependent variable (Y equals 0 or 1, yes or no) and independent variables (X_1, X_2,..., X_n) were used in this statistical analysis. The basic equation for the logistic regression model was of the form:

$$Y_i = \beta_o + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_k X_k + \varepsilon \tag{3}$$

where:

Y = dependent variable

x_i = independent variables

β_i = regression coefficients

ε = error

Logistic regression modeling is divided in two phases: a training phase and an evaluation phase (Ioannou, 2006). During the training phase, the user develops a logistic regression model or equation for use during the evaluation phase. For this research the training phase model was developed to produce output that could be used to identify elevations where high pile rebound occurred. The model variables that were chosen were related to the elevations associated with high pile rebound determined from PDA information. Based on the variables identified during the training phase, regression coefficients for each variable were determined and then used in the probability equation to predict the chance of high pile rebound occurring. The probability that an event will occur can be predicted using the following equation described by (Brannick, 2007):

$$P = \frac{e^{\beta_o + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_k X_k}}{1 + e^{\beta_o + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_k X_k}} \tag{4}$$

where:

P = probability

e = the base of the natural logarithm (i.e., 2.718)

x_i = independent variables

$\beta_0, \ldots \beta_n$ = regression coefficients

4.5.2 Logistic Regression Results

Using the SPSS logistic regression, rebound was chosen as the dependent variable where one (1) implied rebound would occur and zero (0) no rebound would occur. The following thirteen independent variables were selected for the analysis:

1. Moisture content
2. Silt Content
3. Sand Content
4. Clay Content
5. Liquid limit
6. Plastic limit
7. Unconfined compressive strength from the pocket penetrometer
8. SPT N-values
9. Percent passing the number 10
10. Percent passing the number 40
11. Percent passing the number 60
12. Percent passing the number 100

13. Percent passing the number 200 sieves.

Several sieve sizes were used for the modeling that had not been considered for other analyses. They included the Nos. 10, 40, 60, and 100 sieves. The No. 40 sieve was indirectly considered since the Atterberg limits were conducted on the material passing this sieve. The No. 200 was also evaluated in the design phase work and found to provide inconclusive results. The other two sieve sizes were added to this analysis to check for any possible correlations between the percent material passing through them and high pile rebound.

The model was created using 83 cases from the three sites. Information was chosen randomly from each site. Only half of the records (i.e., 42 cases) were used for the training phase, while all 83 cases were used for the evaluation phase. The regression coefficients (β_o, β_1, … β_n) and their standard errors were produced from the model. The complete statistical process is detailed by Chin Fong (2010) and the results are shown in Appendix D.

The training phase modeling produced the probability that the independent variable was related to high pile rebound and the error associated with the independent variable. Table 4.17 summarizes these probabilities with data that shows two variables, the silt content and the unconfined compressive strength from the pocket penetrometer, produce a 100 percent likelihood of predicting rebound. The data also shows that the SPT N-values and the percent passing the # 10 sieve produce 68 and 80 percent likelihood of high pile rebound. After several computational iterations the N and percent passing the #10 sieve were eliminated, leaving only the silt content and q_u for use during the evaluation phase.

Variables		% Correct
1	Natural Moisture Content	13%
2	SPT N-value	68%
3	Liquid Limit	5%
4	Plastic Limit	27%
5	% Sand	45%
6	% Clay	44%
7	Sieve #10	80%
8	Sieve #40	54%
9	Sieve #60	1%
10	Sieve #100	22%
11	Sieve #200	46%
12	Pocket Penetrometer q_u	100%
13	% Silt	100%

Table 4.17 Summary of SPSS Model Probability of Correctness for 13 variables

From this training phase model the regression coefficients, β_o, β_1, β_2 were determined to be: -4.407, 0.090 and 0.215, where β_1 was the variable associated with q_u and β_2 was the variable associated with the silt content. Based on Equation 4 the evaluation phase equation obtained was:

$$P = \frac{e^{-4.407+0.090X_{q_u}+0.215X_{silt\%}}}{1+e^{-4.407+0.090X_{q_u}+0.215X_{silt\%}}} \tag{5}$$

The logistic regression model was used along with data from the 83 cases from the three sites to produce the results compiled in Table 4.18. These results show measured and predicted probabilities of rebound versus elevation. The table includes the site elevations, the corresponding measured high pile rebound displayed as 1 implying $Rebound_{PDA}$ or 0 implying no rebound, and a predicted rebound probability output based on Equation 5. The measured pile rebound values were based on the PDA rebound elevations. When the probability equation produces values greater than 50 percent, rebound would be expected; therefore, the highlighted probability cells imply rebound predicted by the model.

Anderson Street Overpass			John Young Parkway			Ramsey Branch Bridge		
Elev (ft)	Measured Rebound	Probability of Rebound	Elev (ft)	Measured Rebound	Probability of Rebound	Elev (ft)	Measured Rebound	Probability of Rebound
87.5	0	2%	79.5	0	4%	2	0	7%
82.5	0	2%	74.5	0	89%	-3	0	5%
77.5	0	3%	69.5	0	99%	-8	0	11%
72.5	0	1%	64.5	0	6%	-13	0	14%
67.5	0	3%	59.5	0	34%	-18	0	46%
62.5	0	2%	54.5	0	9%	-23	1	95%
57.5	0	3%	49.5	0	13%	-28	1	82%
52.5	0	42%	44.5	0	10%	-33	1	98%
47.5	0	68%	39.5	1	27%	-35	1	68%
42.5	0	6%	34.5	1	18%	-36.5	1	71%
37.5	0	4%	29.5	1	34%	-38	1	92%
34	0	20%	28	1	18%	-39.5	1	91%
29	1	14%	26	1	70%	-41	1	87%
27	1	20%	24	1	13%	-42.5	1	95%
25	1	41%	22	1	21%	-44	1	97%
22	1	54%	20	1	47%	-45.5	1	78%
18	1	29%	18	1	40%	-47	1	12%
16	1	21%	16	1	93%	-48.5	1	32%
14	1	100%	14	1	100%	-50	1	60%
12	1	100%	12	1	90%	-51.5	1	76%
10	1	100%	10	1	100%	-53	1	59%
8	1	100%	8	1	57%	-54.5	1	79%
6	1	96%				-56	1	87%
4	1	100%				-57.5	1	84%
2	1	74%				-59	1	84%
0	1	99%				-60.5	1	91%
-2	1	90%				-62	1	61%
-4	1	100%						
-6	1	99%						
-8	1	95%						
% Correct in Rebound Zone		72%			43%			91%

Table 4.18 Probability of Rebound from the SPSS Model based on Pocket Penetrometer and Silt Content versus Elevation at the Retesting Sites

4.5.3 Summary of Statistical Analyses

The PDA rebound zones from Anderson Street Overpass were above elevation 55 feet (16.7 m) and below elevation 28 feet (8.5 m). Table 4.18 data shows that no rebound would be predicted in this upper rebound zone; however, high pile rebound might occur below this at elevation 47.5 ft (14.6 m). It also shows high pile rebound would be predicted at elevation 22 ft (6.7 m) and below elevation 16 feet (4.9 m). This lower elevation is close the elevation where the pile was "bouncing", which actually begins at elevation 9 ft (2.7 m). There are 30 predictions for Anderson Street Overpass. High pile rebound was predicted correctly in the zone below elevation 29 ft (8.8 m) in 13 out of 18 possible depths or for about 72 percent of the elevations, as shown at the bottom of the table.

The high pile rebound zone for John Young was below elevation 41 feet (15.5 m). Table 4.18 data shows it would be predicted below elevation 18 feet (5.5 m), which was close to the bouncing elevation of 26 ft (7.9 m) and that it might occur at elevation 26 ft (7.9 m). The model prediction percentages increase to near 50 percent below elevation 26 ft (7.9 m). There are 22 predictions for John Young with 14 of those predictions in the high pile rebound elevations. Within these elevations rebound was predicted correctly for six out of 14 depths or for about 43 percent of the elevations, as shown at the bottom of the table.

The high pile rebound zone for Ramsey Branch Bridge was below elevation -22 feet (6.7 m). Table 4.18 shows that rebound would be predicted below elevation -18 feet. There are 27 predictions for Ramsey Branch Bridge and, within the high pile rebound elevations; it was predicted correctly in 20 out of 22 possible depths or for 91 percent of the elevations, as shown at the bottom of the table.

Overall, the statistical modeling showed better results for Anderson Street Overpass and Ramsey Branch Bridge than for John Young. The soils in the rebound zones at these two sites have more clay than the soils at John Young. The USCS descriptions for both sites include either SC or SM-SC descriptors while the symbol for John Young was SM. Because q_u controls the output from this model and John Young has less cohesion and therefore less q_u values, the model for this site may not be correct.

The statistical model results are based on a limited number of sites and tests. It shows promise in predicting rebound and needs to be evaluated with data from more sites.

5. Conclusions

The project objective was to develop a design-phase geotechnical testing protocol that would help engineers identify geotechnical conditions that may produce high pile rebound when combined with certain pile-driving combinations.

5.1 Overall Project Conclusions

Geotechnical engineers should anticipate high pile rebound problems when driving displacement piles into relatively fine grained saturated layered deposits where:

1. where negative pore water pressures occurred during CPTu testing in very soft sandy clays (SC) that overlay soft sandy clays (SC) and the silt content increases from the very soft to soft layer by a factor of about 2, while the SPT N-values increase by a factor of about 5 from the very soft to soft clays or
2. where negative pore water pressures occurred during CPTu testing in loose to medium dense silty to clayey sands (SM-SP) or (SM) or (SC) that overlay dense or very dense silty to clayey sands (SM-SP) or (SM) or (SC) and the silt content in the dense to very dense layer increases by a factor of about 2 and the N-values increase by a factor of about 3 from the weaker overlying to stronger underlying layers.

Geotechnical engineering parameters from SPT, Pocket Penetrometer Tests, CPT, DMT, PPMT and CU Triaxial tests all produced similar increases in their respective values (i.e., N, q_c, f_s, q_u, E) between the very soft to soft clay layers (SC) and the medium dense to dense or very dense silty sand layers (SM-SP) or (SM) or (SC). SPT N-values increased from about15 to over 50 blows/foot, q_u values increased from about 1.1 to 2.9 tsf (105 to 278 kPa), f_s values increased from 0.7 to over 3 tsf (67 to 287 kPa), q_c values increased from 50 to over 200 tsf (4790 to 19,150 kPa) and the silt content increased to 20 percent.

There were differences between the elevations where the pile began to rebound and the elevations at which each test parameter increased, indicating that displacement piles began to rebound above the soils causing high pile rebound. These differences averaged from 4.8 to 7.8 pile diameters into the deposit. It remained near 5B for Anderson Street Overpass regardless of which soil parameter was evaluated.

5.2 Typical High Rebound Piles

Both the literature and the research program produced similar conclusions. The problem piles:

- were all displacement piles
- rebounded at least 0.5 inches
- were driven with single acting hammers
- were installed into dense or stiff soils with high silt contents
- were most likely installed in the Hawthorn Group, an olive green clayey thick geologic deposit with phosphate that underlies most of Florida.

5.3 PDA Conclusions

The following conclusions were developed from the design phase PDA analysis of the Anderson Street Overpass and John Young projects.

- PDA displacement data when plotted versus elevation show the extent of the high pile rebound zones.
- Both the $Rebound_{PDA}$/DMX and $Rebound_{PDA}$/DFN plots show the piles may be "bouncing" somewhere between 7.5 and 9.5 pile diameters into the rebound layer.
- The "bouncing" elevation matches the elevation of the top of dense or very dense to hard silty sands.
- The high pile rebound elevation zones for both sites match the approximate elevation of the top of the Hawthorn Group shown on Figure 2.4.

5.4 Geotechnical Testing Conclusions

Ratios between rebound and no rebound soils were consistent and generally from 2 to 3 for the best indicators. The ratios that most clearly matched possible rebound layers were developed from:

1. Silt content

2. N or N_{60}

3. Pocket penetrometer q_u

4. CPT q_c and or f_s

The ratios which did not clearly match possible rebound layers were developed from:

1. Liquid limit

2. Natural moisture content

3. Sand content

6. Recommendations

6.1 Overall Recommendations

If geotechnical engineers in Florida expect to install high displacement piles into soils with varying silt contents using single acting hammers, they should conduct a thorough investigation of 1) the geologic and construction history of the site with emphasis on determining the elevations of the Hawthorn Group or amount of compacted fill, 2) the soil composition, including the grain size distribution with hydrometer testing, to delineate the silt and clays, 3) elastic modulus or stiffness from CU triaxial testing, DMT or PPMT tests and 4) strength parameters determined from either SPT N-values, pocket penetrometer testing, PPMT, DMT or CPT testing. Sufficient soil samples should be obtained during SPT testing to allow complete grain size analyses.

When evaluating the site conditions, compacted fill areas and depths should be outlined so that they can be evaluated separately during the high rebound evaluation process. Several Pocket Penetrometer q_u readings should be taken on each cohesive split spoon sample in areas within the spoon that show the least disturbances. If N-values increase by a factor of three or more, additional split spoon or thin walled tube samples should be obtained for further testing.

The statistical approach developed from the three sites shows promise and should be refined with future work. Sites should be chosen which have high quality PDA data such as the data from both the Anderson Street Overpass and John Young. The PDA data should include at least the blow number, elevation, DMX, DFN, set and rebound. The geotechnical subsurface investigation should include SPT, CPT, PPMT, DMT and thin walled tube sampling. This data should be used to develop soil profiles with classifications, moisture contents, permeability, stiffness and strength parameters from the ground surface to the boring termination depth. This data should be input into the SPSS logistic regression model described by Brannick (2007). This method may lead to a simpler evaluation process because other parameters may be found that are related to high pile rebound.

6.2 Recommended High Pile Rebound Evaluation Process

The complete grain size distribution should be obtained from samples throughout the soil profile, such that all significant soil layers can be classified using USCS symbols, and silt contents can be plotted versus elevation. Particular attention should be paid to the silt content and how this percentage varies throughout the profile. If the ratio of silt content changes drastically (i.e., by a factor of more than 2.0) from weaker soils to stronger soils there may be a potential for high rebound. After evaluating the silt content, the engineer should evaluate the SPT N-values versus depth over the same soil layers that displayed the silt content changes. This evaluation may be performed using either the raw or corrected N values. Table 4.14 contains the critical ratios that may indicate rebound. Figure 6.1 is a flow chart that may serve as a guide to help engineers identify high pile rebound soils. This flow chart provides engineers with the required input data and explains the required output needed before the high rebound evaluation can be completed.

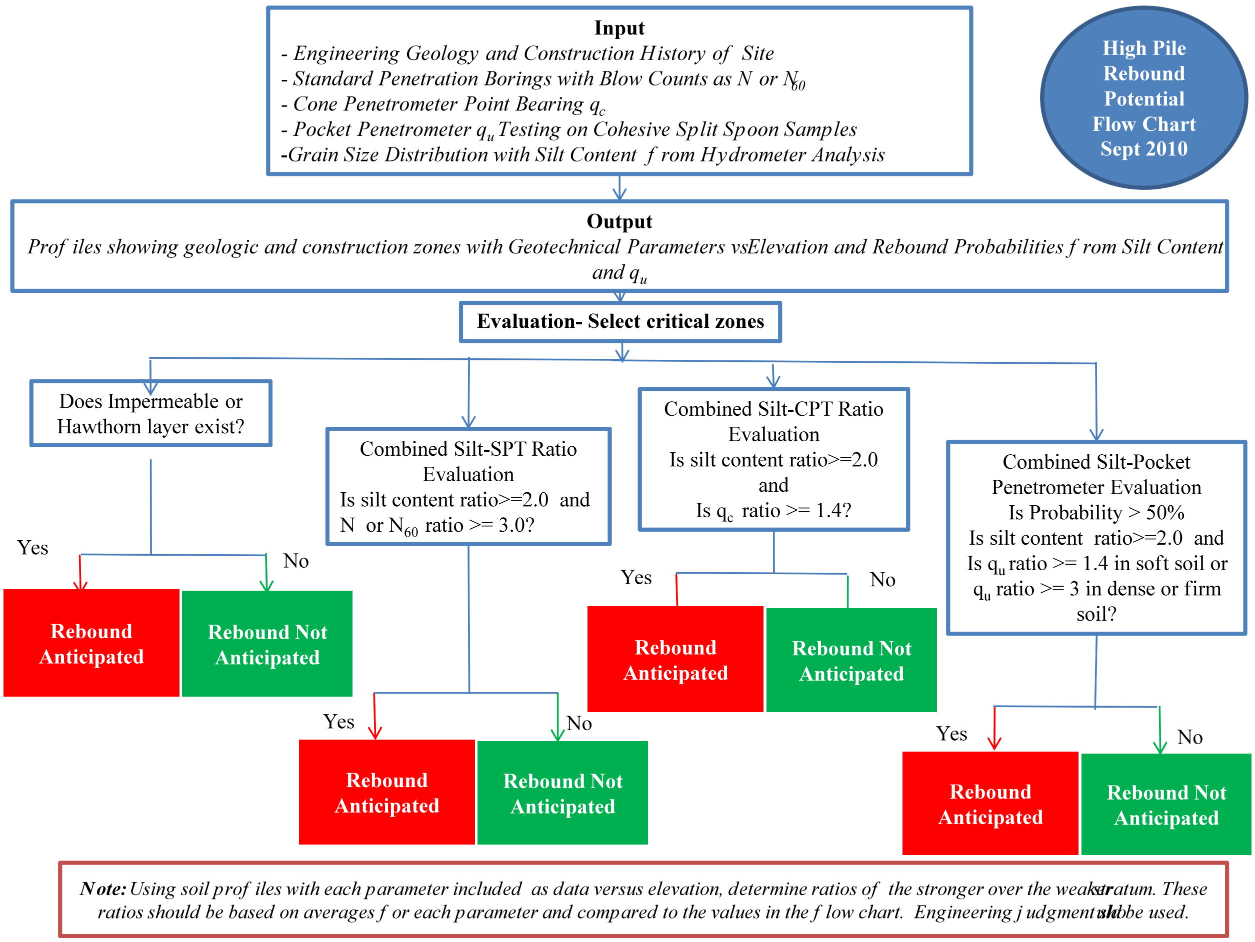


Figure 6.1 High Pile Rebound Prediction Flowchart

6.3 Recommended Future Studies

This work was based on data from a limited number of sites and should be refined by continually evaluating high displacement pile installations with PDA data and extensive geotechnical engineering testing and analysis. There was limited data available from Ramsey Branch Bridge and areas outside of the Orlando area. Additional sites outside of the Orlando area should be identified for further studies. Additional work should include more detailed analysis of the overlying strata in the potential high rebound sites. A complete profile of key geotechnical parameters including N, E, q_c, f_s, ϕ, c and γ versus elevation should be developed with soil sampling and testing conducted with continuous samples throughout the borings or soundings.

The method proposed for determining the zones for analyzing the soil properties and developing the ratios must be refined. The effects of the zone of influence affecting the piles during driving, based on the values in Table 4.15, need to be clarified.

The averages were based on PDA rebound elevations and should be refined because there are two pile behaviors occurring during the high pile rebound: one that allows pile penetration and a second that produces bouncing, which prohibits pile penetration. Future sites should be evaluated using both criteria.

The evaluation of the excess pore water pressures, which are believed to be one cause of high pile rebound, should be expanded. A series of CPTu tests should be conducted at new sites and plus laboratory CU triaxial tests with pore pressure measurements should be conducted. The lab testing should also include cyclic triaxial testing with pore pressure measurements. The critical data from the cyclic triaxial testing would be the volumetric changes and associated pore water pressures during each loading-unloading cycle. Based on the PDA data, the high pile rebound occurs in less than 200 milliseconds; therefore the pore pressure transducers used must have extremely fast response times. This data may help engineers explain how the high rebound soil reacts during the driving process and allow dynamic pile driving analyses to be improved.

High speed videos should be recorded during the driving process when high pile rebound is encountered. The equipment used must be able to record at a rate high enough to view the rebound process. Video cameras with sampling rates in excess of 400 frames per second are recommended.

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