Engineering Properties of Pile Rebound Soils

Based on Cone Penetration Testing

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We the undersigned committee hereby recommend
that the attached document be accepted as fulfilling in
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“Engineering Properties of Pile Rebound Soils
Based on Cone Penetration Testing”

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Abstract

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High pile rebound (HPR) has been identified by Florida Department of Transportation (FDOT) to occur during the installation of square prestressed concrete piles at many sites in Florida. Significant pile rebound values of up to 1.5 inch/blow were measured resulting in increased blow counts. Pile refusal is a common occurrence when blow count exceeds 240 blow/ft; leading to pile redesign and economic consequences.

The overall objective of this research is to identify the engineering properties of soil deposits which may cause HPR and develop improved correlations that may be used to predict HPR during the design process.

Seven sites were studied in this research. Pile driving analyzer (PDA) data was used to identify the rebound zones. Cone penetration tests (CPT) and Standard penetration tests (SPT) were conducted near the associated test piles. The SPT data was used to develop soil profile for each site. The CPT data was used to estimate
profiles of engineering soil properties. An existing correlation between the CPT pore pressure and pile rebound was evaluated and improved.

High CPT pore pressures measured at the rebound zones were found to correlate linearly with pile rebound. Using the CPT the rebound soils were classified as dense silty sands and highly overconsolidated or cemented silty clays. These soils are dilative under shear loading increasing the shear strength of the surrounding soil and the pile skin friction. As a result higher blow counts are required to reach pile penetration. The HPR soils have very low permeability; therefore, high compression-induced pore pressures may be generated near the pile tip during driving. These pore pressures at the pile tip may provide upward forces leading to rebound. The SPT data showed that cemented silty fine sand (SM) and clayey fine sand (SC) with trace phosphate and shell with fines content of 25 % to 40 % were found in the rebound zones. The CPT data superimposed on soil behavior type (SBT) charts provides an engineering method to predict pile rebound soils.
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List of Terms

HPR = High Pile Rebound

CAPWAP = Case Pile Wave Analysis Program

PDA = Pile Driving Analyzer

DMX = Maximum pile displacement

DFN = Final pile displacement

FDOT = Florida Department of Transportation

SPT = Standard Penetration Test

CPT = Cone Penetration Test

SMO = State Material Office

OCR = Overconsolidation Ratio

NC = Normally Consolidated

PCP = Prestressed Concrete Pile

PSCP = Prestressed Square Concrete Pile

USCS = Unified Soil Classification System

SR = State Road

ASTM = American Society for Testing and Material
FC = Fines Content

SBT = Soil Behavior Type

SBTn = Normalized Soil Behavior Type

iSet = inspector Set

G.S.E = Ground Surface Elevation

G.W.T = Ground Water Table

NAVD88 = North American Vertical Datum of 1988

SP = Sand Poorly Graded

SP-SM = Sand with Silt

SM = Silty Sand

SP-SC = Sand with Clay

SC = Clayey Sand

SM-SC = Silty Clayey Sand

CH = Clay High Plasticity

FD = Fine Dilative

FC = Fine Contractive

CD = Coarse Dilative

CC = Coarse Contractive
Notations

E = Elastic modulus

ε = Strain

σ = Stress

γ = Unit weight

k = Permeability

D_r = Relative density

ψ = State parameter

S_u = Undrained shear strength

q_c = Cone tip resistance

f_s = Cone sleeve friction

R_f = Cone friction ratio

F_r = Normalized cone friction ratio

Q_m = Normalized cone resistance

u_1 = CPT pore water pressure at cone point

u_2 = CPT pore water pressure at cone shoulder

u_3 = CPT pore water pressure behind the cone tip
\( q_t = \) Corrected cone tip resistance

\( q_E = \) effective cone resistance

\( B_q = \) Cone Pore pressure ratio

\( P_a = \) Atmospheric pressure

\( \gamma_w = \) Unit weight of water

\( I_c = \) Soil behavior type index

\( \sigma_{vo} = \) total overburden stress

\( u_o = \) Hydrostatic pore water pressure

\( \sigma'_{vo} = \) effective overburden stress

\( t_{50} = \) time for 50\% dissipation of pore water pressure

\( C_c = \) Coefficient of curvature

\( C_u = \) Uniformity coefficient

\( e = \) Void ratio

\( e_o = \) Initial void ratio

\( e_{cs} = \) Critical state void ratio

\( \sigma_{atm} = \) Atmospheric pressure

\( \sigma'_p = \) Preconsolidation stress

\( N = \) SPT number of blows per ft.

\( \Delta u_{com} = \) compression-induced pore water pressure
\( \Delta u_{\text{shear}} = \text{shear-induced pore water pressure} \)

\( \phi' = \text{Effective angle of friction} \)

\( G = \text{Undrained shear modulus} \)

\( c = \text{soil cohesion} \)

\( \sigma = \text{normal total stress on the failure plane} \)

\( \sigma' = \text{normal effective stress on the failure plane} \)

\( A_r = \text{Skempton’s pore water pressure coefficient} \)

\( R_p = \text{Radius of plastic zone} \)
Acknowledgment

I would like to take this opportunity to express my thanks and my sincere appreciation to my dissertation advisor, Dr. Edward H. Kalajian. I am indebted to my advisor for the valuable assistance, patient guidance, and encouragement he provided throughout this study. The constructive suggestions of my dissertation committee members, Dr. Paul J. Cosentino, Dr. Thomas Belanger, and Dr. Luis D. Otero, were particularly valuable. I would like to acknowledge the Civil Engineering Department at Florida Institute of Technology represented by the head department, Dr. Ashok Pandit, for providing encouragement and support.

I am indebted to Dr. Peter K. Robertson for his immeasurable help and support in CPT data analysis and for providing me with a free license of CPeT-IT software. I would also like to convey my sincere appreciation to Dr. Paul W. Mayne for his help in data analysis. Additionally, I am grateful to the Florida Department of Transportation for providing me valuable field testing data.

I would not miss this opportunity to convey my gratitude to my parents and husband for their love and encouragement, without which, this work would not be completed.
Dedication

To my parents, my husband, and my two young children, Jafar and Amir
Chapter 1
Introduction

1.1 General Introduction

Pile driving is a complex interaction between soil properties, hammer types and driving procedures. One of the more complex pile driving problems occurs when piles during the installation begin to rebound after each hammer blow rather than continue to advance to the design depth. Pile rebound is described as the upward movement of the pile after each hammer blow while set is the permanent pile displacement as shown in Figure 1.1. This problem becomes severe when the upward movement (rebound) exceeds approximately \( \frac{1}{4} \) inch and there is no or minimal pile penetration into the soil. The term most commonly used to describe this problem is high pile rebound (HPR) although some engineers term it as bounce (Cosentino et al., 2010, Murrell et al., 2008).

If the rebound is excessive, pile-driving contractors may need numerous hammer blows to drive or advance the pile to the design depth. In certain cases when the piles cannot be advanced, the excessive hammer blows result in high compressive and tension stresses that may damage the piles.
1.2 Pile Movement During Driving

A hybrid linear elastic-plastic model was used to depict pile load-displacement movement from a single hammer blow (Smith, 1960). Figure 1.2 shows the actual and modeled energy rebound during a single hammer cycle. The elastic compression of the soil or rock below the pile point results in an upward displacement (i.e. rebound) of the pile after the hammer blow. Rebound is typically associated with the reaction of the soil as opposed to the pile. Quake is measured as the pile displacement when the soil behavior changes from elastic to plastic. It is a modeling parameter describing the soil's initial elastic movement (i.e., similar to earthquake movements) from the dynamic energy resulting from a single hammer blow (Murrell et al., 2008).
Figure 1.2 Soil resistance versus penetration with quake for one hammer blow
(Smith, 1960)

1.3 Pile and Soil Modeling using the Wave Equation

Smith (1960) developed the discretized spring and dashpot model, shown in
Figure 1.3, to represent the pile and hammer system. It consists of a hammer, anvil,
hammer cushion material between the anvil and the pile cap, the pile cushion, the
pile, and the surrounding soil. Springs are used to represent elastic materials, while
spring and dashpot combinations are used to represent elasto-plastic materials such
as soil. Smith (1960) also included soil damping (J), along with quake (q), to help
describe the system. Smith's damping coefficient has units of 1/velocity. The soil
damping and the wave speed are proportional to the force or pile resistance ($R(t) = J \ast v(t)$).
1.4 Dynamic Pile Testing

Dynamic testing is performed during pile driving when real-time measurements are desired. The dynamic testing system as presented in Figure 1.4 consists of: a) field testing utilizing specialized equipment (strain transducers and accelerometers) and b) pile wave signal matching software (Case Pile Wave Analysis.
Program (CAPWAP®). The accelerometers and strain transducers are placed within 2 feet of the pile head. The signals are transmitted via cable to a data acquisition system provided with a software package named Pile Driving Analyzer (PDA).

Accelerations, measured by the accelerometers, are integrated once to produce velocity traces versus time and a second time to produce deflections versus time. The strains, measured by the strain transducer, are used along with the known pile properties (area and elastic modulus) to determine the force in the pile versus time at the transducer location. Based on Hooke’s Law (E=\( \sigma / \varepsilon \)), the strain (\( \varepsilon \)) and elastic modulus (E) are used to determine the stress (\( \sigma \)). The area of the pile is used to determine the calculated force as \( F = \sigma \times A \).

Figure 1.4 Dynamic testing system (a) Strain transducer and accelerometer attached to a test pile and (b) data acquisition system
The operator adjusts these traces to produce a match between the actual force trace and the computed force curve. After the measured force has been obtained for each hammer blow, engineers use the CAPWAP® signal matching process along with these forces to predict force versus time curves. This predicted curve is then compared to the actual force trace generated during pile driving. Figure 1.5 demonstrates five iterations of that matching process. Damping is added after the first iteration, then the capacity is increased, and finally the quakes are adjusted. This process typically produces a good match by five iterations. The CAPWAP software outputs included three sets of information: the depth or elevation of the pile tip, the maximum pile displacement (DMX), and the final pile displacement (DFN) per hammer blow.

Figure 1.5 CAPWAP® iterative process (Cosentino et al. 2010)
1.5 Problem Statement

Excessive pile rebound problems have occurred during the installation of several types of piles, typically square prestressed concrete, at many sites in Florida. The HPR problem in Florida was initially identified by Florida Department of Transportation (FDOT) at three sites where significant rebound values up to 1.5 inches were obtained and resulted in pile refusal. Two of these sites were located in Orlando, FL area while the third site was located in Panhandle, Florida. Pile refusal is identified when the number of blows required to drive a pile one inch exceeds 20 blows. As a result of pile refusal using the original design, piles at these three sites had to be redesigned to account for the rebound nature of soil. Pile redesign usually results in significant project delays and requires replacing the square prestressed concrete piles by more expensive hollow tube steel piles. Pile redesign increases the total cost of projects and claims made by the contractors.

In an attempt to reduce the consequences of HPR, Florida Department of Transportation (FDOT) funded a research project at Florida Institute of Technology in 2008 to study the HPR at the three locations previously identified (Cosentino et al., 2012). The main objective of that initial study was to identify a methodology to predict the possibility of the HPR occurrence at any site before the commencement of the design phase. At all three sites, single acting diesel hammers were used to drive piles to depths greater than 40 feet. Pile Driving Analyzer data was used to develop profiles of rebound versus elevation. Additional field testing was conducted at these sites and included Standard Penetration Testing (SPT), Piezocone
Penetration Testing (CPT), Pocket Penetrometer Testing (PPT), Dilatometer Testing (DMT), and PENCIL Pressuremeter (PPMT). Based on limited soil data from these sites, it was concluded that HPR occurred in very dense saturated silty fine sand or clayey fine sand soil layers. The limited data available to the researchers was insufficient to develop good correlations that could help to precisely identify soil types which may cause HPR.

In 2010, the research team at Florida Institute of Technology, coordinating with the FDOT State Material Office (SMO), identified eight additional HPR sites and conducted additional SPT and CPT soundings. Direct correlation between pile rebound and the pore water pressure measured from the CPT soundings was identified based on analysis of the field data at these eight sites.

High pile rebound is caused by the characteristics and engineering properties of soil deposits. Additional laboratory and field tests are needed to improve existing correlations and develop additional correlations of pile rebound with soil properties. Piezocone penetration testing can be used to obtain soil properties based on the existing CPT correlations such as soil unit weight (γ), permeability (k), overconsolidation ratio (OCR), relative density (Dr), fines content, and shear strength. This study will identify a methodology to enable engineers to identify sites with the potential for causing HPR during the design phase site investigation.
1.6 Study Objectives

The objective is to:

- Identify and evaluate the engineering properties of soil deposits which may cause high pile rebound.
- Evaluate the accuracy of existing correlations used to predict high pile rebound.
- Develop improved correlations with soil properties obtained from field testing with high pile rebound that may be used by engineers to predict high pile rebound during the design process.
Chapter 2
Review of Literature

2.1 Pile Rebound History

Likins (1983) discussed pile installation difficulties in sand and clay soils. Three sites, two in Florida and one in Seattle, Washington were characterized to have one common feature of being saturated soils. Large quakes (toe quakes between 0.4 and 1.0 inches) were observed during installation of large displacement type piles (18-inch, 24-inch square PSCP, and 24-inch octagonal PSCP). Piezometers were installed close to the piles to measure the pore pressure generation during pile driving. Excess pore pressure had built up during the cyclic pile driving and then dissipated. An improvement of soil friction was noticed to occur following the pore pressure dissipation. The three cases presented the adverse effects of large quake on pile drivability and capacity. The pile capacity was reduced by a factor of 3 due to the increase in pile tension stresses. Field observations often led to an interpretation that the original hammer energy was not large enough to properly drive the pile. In cases where the hammer size and energy was increased, the pile was damaged. It was believed that excess pore pressures, caused by displacement piles driven into poorly
drained soils was the primary cause of the large quake. The following advisements were proposed by Likins (1983) if large quakes are experienced:

- Predrilling with slightly undersize bits through weak layers and even through the problem soils.
- Non-displacement pile types could be considered.
- If concrete piles are long and tension stresses high, the ram weight may be increased and the ram stroke may be reduced.
- Pile cushion thickness may be increased resulting in a longer input pulse width and reduced compression and tension wave peaks.

Hussein et al. (2006) discussed the pile drivability and bearing capacity of piles in high-rebound soils encountered at westbound State Road 528 over Indian River Bridge on Central Florida's east coast. The piles were prestressed concrete, 762 mm square with a 457 mm circular hollow core extending throughout the piles 35 m lengths except the ends (i.e., 1.2 m at each pile end is solid). To assess pile drivability and bearing capacity at the site, a pile testing program evaluated 26 dynamically tested piles. Piles were installed in the river by using a Raymond 8/0 single-acting air hammer. This hammer model has a ram weight of 111 kN with a maximum rated energy of 110 kJ. The subsurface conditions of the river were described as saturated very loose to medium dense slightly silty sand (SP-SM) to silty sand (SM) to elevation -27 m, underlain by a layer of firm to hard clayey sand (SC) to sandy clay (CL) extending to elevation -38 m, with a water depth of 2 m. Initial pile driving was not difficult to elevation 18.5 m with blow counts 3 to 17 blows per 300 mm,
gradually increasing to 43 blows per 300 mm with a hammer stroke of 990 mm for the next meter and then decreasing to 12 blows per 300 mm over the next 1.2 m. The dynamic tension stress in the pile exceeded the maximum calculated tension stress but no damage to the pile was observed. The stroke height was reduced to 460 mm at elevation 20.5 m and driving continued to elevation 29.5 m with 18 to 61 blows per 300 mm. The pile tension stresses again exceeded the limit and a high elastic rebound was observed (20 mm). Pile driving was stopped at elevation -31 m with 147 blows per 300 mm and a 15 minutes wait set check was performed. The observed rebound during the set check was reduced to 12 mm due to dissipating excess pore water pressures. After nineteen days the pile was statically tested and then after another nineteen days it was dynamically tested. The resulting pile dynamic record was analyzed using CAPWAP and correlations of pile capacity to the actual static load test compared very favorably.

Regan and Higgins (2009) discussed some of the challenges faced with driving large displacement piles in the Potomac Formation in Washington DC. This formation, which is the oldest sedimentary deposit in that region, consists of very dense sands interbedded with layers of high plasticity over consolidated clays, is the bearing stratum for the deep foundations. Square 355 mm prestressed concrete piles, designed to support the National Harbor Hotel, were installed using a Delmag single-acting diesel hammer. The piles were dynamically load tested and monitored during driving using the Pile Driving Analyzer (PDA). Significant pile rebound (0.64 to 1 inches) was observed during the driving process due to the dynamic forces resisting
pile penetration within the bearing soils. These large quakes affected the efficiency of pile/hammer/soil system because larger energy was required to overcome the elastic rebound, thereby increasing the blow count to fully mobilize the pile resistance. High blow counts and corresponding high compressive stresses within the pile caused a cyclic loading condition and five of seventeen piles ruptured. The rupture was abrupt with little warning and developed within the lower 1/3 to 1/2 of the pile length. Based on one dimensional wave propagation theory, the impact loads imparted on piles with little tip resistance (i.e. early driving) cause tension reflectance from the pile toe and this reflectance decreases as the tip resistance increases. Routine changes to the driving system were made so that the ram weight and stroke were controlled to minimize early driving tensile stresses allowing the piles to be successfully installed. Regan and Higgins, (2009) related the observed large soil rebound to the degree of overconsolidation of this bearing stratum and to the increase of excess pore water pressure during driving.

### 2.2 Measurement of Pile Movement during Driving

#### 2.2.1 Manual Method

The manual method of measuring pile displacement and rebound consists of taping paper onto the pile near a horizontal reference board or beam. As the pile is driven, a pencil moved horizontally across the edge of the reference board records the pile’s movement as illustrated in Figure 2.1. The resulting plot shows the pile maximum displacement and rebound after each hammer blow. While the method is simple, it requires a high degree of dexterity and lacks the precision needed for
complex engineering investigations. In addition, there is a risk of injury to the operators.

Figure 2.1 Pile displacement and rebound recorded by the manual method
(Cosentino et al. 2010)

2.2.2 High Speed Visual Measurement System

Lee et al., (2002) have a patented approach for measuring pile movement during installation by using a high-speed camera. The measurement system, portrayed in Figure 2.2, consists of special marking paper, a high-speed line scan camera equipped with a zoom lens and a personal computer. Line scan cameras use a single line of pixels to scan images and, therefore, require less processing than conventional digital cameras. Fax machines are an example of line scan cameras. The method is based on two-dimensional motion achieved by stacking alternating white and black right-angled triangles on paper as shown in Figure 2.2. As the pile
is driven, the line-scan camera produces a line image by scanning from the top to the bottom of the attached marking paper. The base of each triangle (B) is 40 mm and the height (H) is 200 mm. The line-scanned image is used to determine a location along the pile.

Figure 2.2 Marking paper and line scan camera setup during pile driving (Lee et al. 2002)

Oliveira et al. (2013) developed an in situ, inexpensive, safe, and fast set-up tool to measure the elastic rebound and the final set for driven prestressed concrete piles with diameters varying from 60 to 80 cm and length varying from 20 to 50 m. The pile bearing capacity was evaluated based on the final set and rebound measurements. These measurements were performed by using digital image processing techniques. An A4 size seal containing a printed pattern was fixed on the pile and a standard video camera (30 Hz sampling rate) was used to capture the images. The optical rebound analyzer which consists of CCD camera, computer, and tripod was placed towards the pile at a distance between 5 and 10 m away from it to make sure that there is no significant effect of the vibrations due to driving process
on the results. This distance was chosen based on previous studies. Three locations in different parts of Brazil with known soil profile and N<sub>SPT</sub> were chosen to perform this test.

### 2.2.3 He-Cd Laser Beam Measuring System

Another method to physically measure pile displacements and rebound was proposed by Hattori (1974). A Helium-Cadmium (He-Cd) laser beam used with photosensitive oscillograph paper attached to the pile, produces traces of the pile movement. The laser beam has a high energy density and the proper convergence characteristics that allow it to transmit and focus the beam onto a point at a distance of 10 to 20 meters. The laser beam produced visual traces of pile movement including rebound.

### 2.3 Effect of Soil Properties on High Pile Rebound

Cosentino et al. (2012) evaluated many sites in Florida with high pile rebound (HPR) during driving of 24” square precast prestressed concrete piles into saturated fine silty to clayey sands and sandy clays. Rebound over 0.25 inch at depths typically greater than 50 ft was observed and followed by a small or zero set during each hammer blow. Pile rebound at these sites has occurred at depths ranging from 50 to 75 ft in silty sand soils with fines content more than 25% and coefficients of permeability less than 3.0E-05 cm/sec.

Jarushi et al. (2013) developed a correlation based on the analyses of field data at the eight sites listed Table 2.1 by Cosentino et al., (2012). Pile rebound was
correlated to measured pore pressure from CPT soundings conducted several months after pile driving. Data from the Pile Driving Analyzer (PDA) was used to develop correlations between piezocone penetration pore-water pressures and pile rebound. Fifteen piezocone penetration tests with pore water pressure measurements were evaluated and compared to HPR from nine piles driven at sites listed in Table 2.1. The following three conditions were experienced at these sites: (1) excessive HPR with no or minimal set with more than 20 tsf positive pore water pressure, (2) HPR with acceptable set with pore water pressures between 5 to 20 tsf, and (3) no HPR with pore water pressure less than 5 tsf.

Table 2.1 Summary of Soil Properties and Rebound Zones at Eight Florida Sites Experiencing HPR (Cosentino et al. 2012)

<table>
<thead>
<tr>
<th>Case</th>
<th>Site Name</th>
<th>Rebound &gt; 0.25 in</th>
<th>Depth (ft)</th>
<th>Soil Type (USCS)</th>
<th>FC (%)</th>
<th>K (cm/s)</th>
<th>fs (tsf)</th>
<th>qc (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>I-4 / SR 408 Anderson overpass</td>
<td>No</td>
<td>10 to 90</td>
<td>SP-SM, SM, SM-SC, &amp;CL</td>
<td>&lt; 20</td>
<td>5.0x10^6</td>
<td>0.8</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yes</td>
<td>90 to 110</td>
<td>SM, SC, CL &amp; CH</td>
<td>&gt; 40</td>
<td>6.8x10^4</td>
<td>3</td>
<td>200</td>
</tr>
<tr>
<td>2</td>
<td>SR 50 / SR 436</td>
<td>No</td>
<td>54 to 71</td>
<td>SP, SM, &amp; CH</td>
<td>30</td>
<td>2.7x10^5</td>
<td>0.5</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yes</td>
<td>71 to 82</td>
<td>CH</td>
<td>&gt; 40</td>
<td>2.7x10^5</td>
<td>1.5</td>
<td>50</td>
</tr>
<tr>
<td>3</td>
<td>I-4 / US 192 Ramp CA</td>
<td>No</td>
<td>30 to 65</td>
<td>SP &amp; SP-SM</td>
<td>&lt; 20</td>
<td>4.0x10^4</td>
<td>0.8</td>
<td>60</td>
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<tr>
<td></td>
<td></td>
<td>Yes</td>
<td>65 to 85</td>
<td>SM</td>
<td>25-50</td>
<td>5.0x10^4</td>
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<td>140</td>
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<td>No</td>
<td>8 to 29</td>
<td>SP-SM</td>
<td>&lt; 20</td>
<td>N/A</td>
<td>0.2</td>
<td>10</td>
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<td></td>
<td>Yes</td>
<td>29 to 77</td>
<td>SP-SC &amp; SC</td>
<td>&gt; 40</td>
<td>5.0x10^4</td>
<td>1.5</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>I-4 / John Young</td>
<td>No</td>
<td>10 to 60</td>
<td>SM, SC, &amp; CL</td>
<td>20</td>
<td>8.0x10^4</td>
<td>0.5</td>
<td>50</td>
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<tr>
<td></td>
<td></td>
<td>Yes</td>
<td>60 to 93</td>
<td>SM &amp; CL</td>
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<td>1.6x10^7</td>
<td>1.5</td>
<td>150</td>
</tr>
<tr>
<td>6</td>
<td>I-4 / SR 408 Ramp B</td>
<td>No</td>
<td>16 to 76</td>
<td>SP, SP-SM, SP-SC, &amp; CH</td>
<td>20</td>
<td>N/A</td>
<td>1</td>
<td>100</td>
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<tr>
<td></td>
<td></td>
<td>Yes</td>
<td>76 to 105</td>
<td>SC</td>
<td>20</td>
<td>N/A</td>
<td>2</td>
<td>150</td>
</tr>
<tr>
<td>7</td>
<td>I-4 / SR417</td>
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<td>50 to 75</td>
<td>SM</td>
<td>04</td>
<td>1.3x10^4</td>
<td>0.75</td>
<td>100</td>
</tr>
</tbody>
</table>
Robert et al. (2012) investigated the effect of grain shape on pile capacity in sand soil located at Escambia Bay, Pensacola, Florida. The soil condition at the site was very soft to soft clay to a depth of 40 ft, underlain by medium dense to dense fine to medium sand to a depth of 78 ft. No carbonate content, cementation, or high mica content in the sand soil was reported. Twenty inches square precast prestressed concrete piles were driven at blow counts higher than expected. The load tests indicated lower capacities than expected and large rebounds ranged from 0.53 to 0.62 inches. Microscopic analyses conducted on seven samples of dense to very dense silty fine sand showed that the sand grain shape was angular to subangular. It was concluded that the angular soil particles with high friction angle caused the observed large rebound due to the increased energy required to attain a failure state.

2.4 Pore Water Pressure Change during Pile Driving

Robertson et al. (1990) studied the distribution of excess pore water pressures and drainage conditions around 915 mm diameter, open-ended steel piles driven to a depth of approximately 90 m through normally consolidated marine clayey silt. These piles were part of the foundations for the Alex Fraser Bridge, British Columbia, Canada. A multipoint piezometer was installed close to the pile group in order to measure the pore water pressures before and after driving. A cone penetration test with pore pressure measurements was performed 5 m from one of the piles; 18 hours after driving. Six dissipation tests were performed to record the equilibrium pore pressures through the marine silt deposits. The dissipation data in terms of pore water pressures versus log time at different elevations was recorded.
Based on CPT dissipation, the average time required for 95% degree of dissipation of the excess pore pressure was 35-45 minutes. CPT cylindrical dissipation theory was used to estimate the excess pore pressures immediately after driving at 5 m from the pile. The time required for 95% degree of dissipation of pore water pressure 5 m from the pile was estimated to be between 16-21 days. The multipoint piezometer provided a series of pore pressure measurements at discrete depths over an extended period of time. The results were compared with the CPTu results and showed excellent agreement. It was observed that the excess pore pressures generated due to pile driving extends laterally for a distance of 25 to 35 pile radii.

Eigenbrod and Issigonis (1996) monitored pore water pressure responses during driving of steel piles through soft sensitive clay into very dense sand and gravel. Fifteen boreholes were dug in the site close to the driven piles. Two piezometers were installed in each borehole at depths of 9 m and 18 m to measure the generated pore water pressure during pile driving. The production piles were 22 m long to be driven as one single segment. For the test pile, two segments of pile were driven and welded to provide the designed length. The first segment was driven into a depth of approximately 14 m then the second segment of 8 m length was welded and driven. Very low pore water pressures were observed during the driving of the first segment while high pore water pressures were observed in clay during the driving of the second segment. Stresses and pore water pressures changes were analyzed by considering the driving load as a flexible load applied on the surface of an elastic half-space, which is in this case the soft clay layer. The authors concluded
that the clay layer was being loaded from below as the piles were driven into very
dense sand and gravel layer. In that case, pore water pressure increases were
associated with equivalent increases in total stresses. Therefore, minor changes will
occur in effective stresses and bearing capacity. The piezocone test can be considered
an efficient test to predict the pore water pressure increases in clays as well as dilation
of dense sands during pile driving.

Zhu (2011) studied the pore water pressure variation in saturated soft soil due
to driving a prestressed concrete pile. Two existing residential multistory buildings
were selected as a case study. The geological conditions in the site were characterized
as a high natural moisture content, high void ratio, high compressibility, low strength,
and low permeability silty clay to mucky clay with a water table 0.9-1.6 m below the
surface. The designed foundation was prefabricated concrete tubular piles, 500 mm
in diameter, 48-50 m long with a bearing capacity of 4000 kN. During driving in
saturated soft clay, soil squeezing and excess pore water pressure developed due to
the fast loading speed and low coefficient of permeability. In order to measure these
pressures, six piezometers were installed radically and symmetrically from both sides
of the pile (at radial distances of 900 mm, 1900 mm, and 2900 mm) at 14.5 m deep.
The variation of pore water pressure corresponding to the depth of the pile and the
radial distance during the driving was presented graphically. The author concluded
that the influenced pore pressure radius was 5 to 6 times the pile diameter. After the
completion of the pile driving, the observation of excess pore water pressure
dissipation was carried out within 40 days. Fifteen observations for each piezometer
were recorded to show the variation of excess pore water pressure with time at different radial locations. According to the test, the ultimate bearing capacity of the pile at different time period increased because the excess pore water pressure had turned into soil effective stress.

### 2.5 Estimation of Soil Properties Using CPT Data

“Soils are complex and diverse materials that exist within a natural geologic environment” (Mayne 2007). As a result, soil characterization is usually a challenge. Standard penetration testing (SPT) has been widely used for characterizing soils and producing vertical profiles by classifying extracted disturbed samples. However, the SPT technique does not give direct indications about soil density, permeability, and strength parameters. Therefore, adopting SPT only as a basis for soil characterization is usually erroneous for predicting true soil behavior. As an alternative, piezocone penetration test (CPT) can be used for in situ soil investigations due to its repeatability, economic efficiency and continuous data sampling with depth (Yi 2014). Data obtained from the CPT can used with the SPT results to give an improved understanding of engineering soil properties. The piezocone penetration test, if used with full scale testing, soil borings, and laboratory testing, provides excellent soil characteristics applicable to preliminary and final design (Mayne 2007).

The testing system consists of a series of hollow rods with a cone on the end. The cone consists of hardened steel conic tip facing down with an apex angle of 60° with projected areas. The most common and specified sizes are 10 cm² and 15 cm²
probes. Figure 2.3 shows a range of cones from a mini-cone at 2 cm$^2$ to a large cone at 40 cm$^2$. The mini cones are used for shallow investigations, whereas the large cones can be used in gravelly soils (Robertson and Cabal 2010).

Two load cells located on the cone tip and sleeve and pore pressure transducer are connected with cables passing through the hollow rods and connected to the data acquisition system. The load cells measure the total force acting on the cone ($Q_c$) and the total force acting on the friction sleeve ($F_s$). The tip force is divided by the projected area of the total cone ($A_c$) and produces the cone resistance ($q_c$) while the total friction force is divided by the surface area of the friction sleeve ($A_S$) and produces the sleeve friction ($f_s$). The pore pressure transducer on a piezocone can be placed at three positions as shown in Figure 2.4.
The electric cone is typically advanced in one meter increments at a relatively constant rate of 2 cm/sec using the hydraulic press of a specialized cone truck. A schematic of a complete CPT test system is shown in Figure 2.5. During the penetration, electrical signals from the point and sleeve load cells are transmitted to the surface through a cable housed within the cone rod. Specialized data acquisition hardware and software is used to record readings from the transducers at a frequency of approximately 5 readings per second. These electrical signals readings are then converted to engineering units of stress using device-specific calibration factors.
Figure 2.5 Overview of the cone penetration test per ASTM D5778-95

Three key parameters are measured continuously with depth during the CPT test: cone resistance \( q_c \), side friction \( f_s \), and excess pore water pressure \( u \). The pore water pressures are developed during the steady, slow penetration of the cone into the soil. The CPT test results can be used to determine soil types as well as engineering properties of the soils. The major applications of the CPT data have been for determination of soil stratigraphy and the identification of soil type as well as parameters assessment in geotechnical design (Schneider et al. 2008). This information typically has utilized charts that link cone parameters to soil type.

The cone resistance, \( q_c \), must be corrected for pore water pressures acting on unequal tip areas of the cone, especially in stiff clays and silts where significant pore water pressure is typically generated. In dense granular soils and clean sands, this correction is not paramount (Lunne et al. 1997). The corrected cone resistance or
total cone tip resistance is designated as \( q_t \), and is determined using Eq. (2.1) (Campanella et al. 1982).

\[
q_t = q_c + (1 - a_n) u_2
\]  

(2.1)

Where:

\( q_t \) = cone resistance corrected for pore water pressure at cone shoulder,
\( q_c \) = measured cone tip resistance, \( q_c = q_t \) in sandy soil,
\( u_2 \) = pore pressure measured at cone shoulder,
\( a_n \) = net area ratio for the cone, typical range between 0.70 and 0.85

Robertson et al. (1986) developed a soil classification chart based on the basic CPT parameters (i.e. cone resistance, \( q_c \) and sleeve friction, \( f_s \)). Since both the cone tip resistance (\( q_c \)) and sleeve friction (\( f_s \)) are affected by an increase in the effective overburden stress, the CPT data can be normalized to account for the influence of overburden stress (Robertson 1990). Wroth (1984) and Houlsby (1988) suggested that CPT data can be normalized using Eq. (2.2) and (2.3):

\[
Q_{tn} = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} 
\]  

(2.2)

\[
F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\% 
\]  

(2.3)

Where:

\( Q_{tn} \) = normalized cone tip resistance
\( F_r \) = normalized friction ratio
\( \sigma_{vo} \) = total overburden stress
\( \sigma'_{vo} \) = effective overburden stress
Using these normalized CPT parameters, a modified or normalized soil behavior type index ($I_c$) was proposed by Robertson (1990) as determined using Eq. (2.4)

\[ I_c = \left[ (3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2 \right]^{0.5} \] (2.4)

Using the normalized soil behavior type index ($I_c$), a normalized soil behavior type (SBTn) chart based on CPT parameters was developed, and is shown in Figure 2.6. The normalized chart has nine soil types with region of normally consolidated soils. Table 2.2 shows the nine SBTn zones corresponding to the soils type and $I_c$. Many studies have shown that the SBTn chart shown in Figure 2.6 typically has greater than 80% reliability when compared with the Unified Soil Classification System, USCS (Robertson and Cabal 2010).

![Figure 2.6 Normalized soil behavior type chart (SBTn) based on CPT normalized cone resistance ($Q_{tn}$) and normalized friction ratio ($F_r$) (Robertson 1990)](image-url)
Robertson and Cabal (2010) presented a new correlation between soil unit weight and CPT results. Approximate contours of soil unit weight in terms of dimensionless CPT corrected cone resistance \((q_t/p_a)\) and friction ratio \((R_f = (f_s/q_t) \times 100)\) were developed. The resulting correlation is shown in Figure 2.7. The numbers 1 to 12 in the background represent the soil behavior type. The chart was made dimensionless by using a dimensionless cone resistance \((q_t/p_a)\) and a dimensionless unit weight \((\gamma/\gamma_w)\), where \(\gamma_w\) is the unit weight of water in same units as \(\gamma\). The contours in Figure 2.7 show that the soil unit weight increases with increasing cone resistance and sleeve friction values. The contours were approximated using Eq. (2.5).
\[
\frac{y}{y_w} = 0.27 \log R_f + 0.36 \log \left( \frac{q_t}{p_a} \right) + 1.236
\]  \hspace{1cm} (2.5)

Where:

- \(R_f\) = measured cone friction ratio, \(R_f = \left( \frac{f_s}{q_c} \right) \times 100\)
- \(f_s\): measured cone sleeve friction
- \(q_c\): measured cone tip resistance
- \(q_t\): corrected cone tip resistance
- \(p_a\): the atmospheric pressure in the same units of \(q_t\).

Data were collected from sites where CPT results and measured soil unit weight were available to verify the proposed equation. The results showed a good agreement between the measured unit weight and the estimated unit weight using Eq. (2.5).

Figure 2.7 Relationship between CPT results and dimensionless soil unit weight 
(Robertson and Cabal 2010)
Yi (2014) conducted a study in order to verify the existing correlations used to estimate fines content based on CPT data. Three SPT borings and three CPT soundings were conducted at a selected site and each CPT was very close to an SPT to obtain very representative data. Disturbed soil samples were collected every 2.5 ft during SPT to measure the fines content by washing them on #200 sieve following ASTM specifications. Cone tip resistance ($q_c$) and sleeve friction ($f_s$) from each CPT sounding were used to estimate the fines content using existing correlations (Robertson and Wride (1998), Idriss and Boulanger (2008) and Cetin and Ozan (2009)). The measured and estimated fines content from the three locations were presented graphically versus depth. The results showed that the measured fines content were generally higher than estimated values. In order to produce a more representative correlation, 133 samples of measured fines content from a total of eleven sites including the site mentioned above were collected and utilized. A new correlation was proposed by Yi (2014) to estimate the fines content based on soil behavior type index ($I_c$) which is proposed by Robertson and Wride (1998) and presented in Eq. (2.6) to (2.8). The new proposed correlation for fines content is presented by Eq. (2.9a) to (2.9d) and verified in Figure 2.8. It was concluded that both Robertson and Wrides (1998) and Idriss and Boulanger’s (2008) methods seem to underestimate the fines content especially for measured fines content higher than approximately 25%, while Cetin and Ozan’s (2009) method may overestimate the fines content higher than approximately 15%.
\[ I_c = \left[ (3.47 - \log 10 \, Q_t)^2 + (\log 10 \, F_r + 1.22)^2 \right]^{0.5} \]  

(2.6)

\[ Q_t = \left[ \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \right] \]  

(2.7)

\[ F_r = \left[ \frac{f_s}{q_t - \sigma_{vo}} \right] \]  

(2.8)

Where:

- \( Q_t \) = normalized tip resistance
- \( F_r \) = normalized friction ratio
- \( q_t \) = corrected tip resistance
- \( \sigma_{vo} \) = total overburden stress
- \( \sigma'_{vo} \) = effective overburden stress

\[ I_c < 1.31, \quad FC \,(\%) = 0 \]  

(2.9a)

\[ 1.31 \leq I_c < 2.5, \quad FC \,(\%) = 42.0 \, I_c - 55.0 + 10 \sin \left( \frac{I_c - 2.5}{1.19} \right) \pi \]  

(2.9b)

\[ 2.5 \leq I_c < 3.1, \quad FC \,(\%) = 83.3 \, I_c - 158.3 \]  

(2.9c)

\[ I_c \geq 3.2, \quad FC \,(\%) = 100 \]  

(2.9d)
Moayed (2006) developed a new approach to evaluate the fines content (FC) of soil layers based on CPTu results. The fines content was correlated with time for 50% dissipation of pore water pressure ($t_{50}$). For this purpose eleven cone penetration tests were performed in silty sand samples with several different silt contents from 0% to 50% using 5% increments in the calibration chamber. The testing chamber consisted of a rigid thick wall steel cylinder of (0.76 m) internal diameter and (1.5 m) height with removable top and bottom plates. Clean fine sand with a specific gravity of 2.6 was used. This sand was rounded to sub-angular fine grained quartz sand with $D_{50} = 0.4$ mm and $C_u = 3.0$. Before filling the testing chamber with dry sample, a soil filter grading from coarse sand to fine gravel was formed at the bottom and another filter layer was formed at the top of the soil. In order to saturate the soil specimen, the top plate was fixed on the chamber and vacuum was applied inside the
chamber for 30 minutes. Then the bottom water supply was opened until a uniform slow upward flow was reached. The standard piezocone used in this investigation has 10 cm$^2$ tip area and 150 cm$^2$ friction sleeve area with filter element located immediately behind the cone tip to record pore water pressure. The piezocone was advanced through the soil by a hydraulic system at a constant rate of 20 mm/sec. Tip resistance, friction resistance, and pore water pressure were recorded continuously during sounding at each 1 cm of depth. The pore pressure dissipation tests were carried out at the midpoint of each sample. The $t_{50}$ parameter (time for 50% of pore water pressure dissipation) was determined for each sample containing different silt content. The relationship between $t_{50}$ and fines content values were presented in Figure 2.9. It was concluded that the $t_{50}$ parameter increases as the fines content increases especially for fines content greater than 30%. There was a good correlation between $t_{50}$ and fines content as presented in Eq. (2.10):

$$t_{50} = 10.235 e^{0.079(FC)}$$  \hspace{1cm} (2.10)

Where:

$t_{50} =$ time required for 50% dissipation for pore water pressure

e = void ratio

FC = fines content (%)
Jamiolkowski et al. (2001) correlated the penetration resistance of the cone penetration tests ($q_c$) to the relative density of sand. The CPT soundings had been performed on three types of silica sand in a calibration chamber with diameter and height of 1.2 m and 1.5 m respectively. The testing samples were reconstituted and subjected to the one-dimensional compression in order to apply the desired consolidation stress level and stress-history. The penetration test was performed using the cylindrical Fugro-type electrical cone tips having diameters ($d_o$) equal to 35.6, 25.4, 20, 11, and 10 mm to investigate the influence of the calibration chamber diameter ($D_o$) to ($d_o$) ratio ($R_d$) on the cone tip resistance ($q_c$). The measured penetration resistance appeared to be independent on the $R_d$ and as a result, the penetration resistance measured in the calibration chamber matches the field value. The experimental data was used to develop Eq. (2.11) to estimate the relative density based on CPT tip resistance and the effective stress.

$t_{50} = 10.235 e^{0.079 FC}$

$R^2 = 0.9815$

Figure 2.9 Experimental correlation between $t_{50}$ and fines content (Moayed 2006)
\[ D_{t, \%} = 100 \times \left[ 0.268 \ln \left( \frac{q_t / \sigma_{atm}}{\sigma'_{vo} / \sigma_{atm}} \right) - 0.675 \right] \] (2.11)

Where:

\( q_t = \) corrected tip resistance

\( \sigma_{atm} = \) atmospheric pressure in the same units of \( q_t \)

\( \sigma'_{vo} = \) effective overburden stress

Robertson et al. (1983) discussed the interpretations of the piezometer cone data that had been used to estimate soil properties and its engineering applications. In-situ piezometer cone testing had been carried out by the In-situ Testing Group at the University of British Columbia (UBC) at sites near Vancouver, B.C. Soil at these sites were deltaic sands, silts and clays or glaciomarine clays and silty clays. Different types of piezometer cones had been used and undrained shear strength and sensitivity measurements were obtained at these sites. Piezometer cone data from four of these sites were used to present soil classification, undrained shear strength \((S_u)\), sensitivity, and stress history \((OCR)\). For soil classification, it was recommended that all cone data \((q_c, f_s, \text{ and } u)\) plus pore pressure dissipation data be used to define soil behavior type more accurately. The cone data should be normalized to account for the effect of increasing overburden pressure in cone soundings deeper than 30 m. The dissipation data obtained during pauses in the cone penetration test can be used to improve soil classification and to provide an index on the soil permeability and consolidation characteristics. No unique relationship between CPT data and \(S_u\) was found for all soil types. The undrained shear strength
is strongly influenced by stress history, sensitivity, and stiffness, therefore; an iterative approach was recommended to estimate shear strength. Sensitivity was found to have a significant effect on the measured pore pressure. Increasing sensitivity caused pore water pressure to increase proportionally.

Chen and Mayne (1996) conducted a statistical analysis between piezocone measurements and “stress history of clays” to evaluate various piezocone parameters in interpreting the stress history of clay. Large field data of piezocone soundings from 205 clay sites around the world had been collected to develop statistical correlations. Most of the data was collected from the eastern and western United States, southern Canada, western Europe, and southeastern Asia, where piezocone penetration tests have been used more frequently than in other parts of the world. The collected piezocone data included tip resistance \((q_t)\) and the pore pressures measured at the tip apex \((u_1)\) and behind the tip \((u_2)\). In addition to the CPT field data, data of soil properties with an emphasis on the index properties including natural water content, liquid limit, plasticity index, and sensitivity was gathered. “Preconsolidation pressure” data from oedometer testing was included to make a comparison with the estimated preconsolidation pressure from CPT parameters. “Stress history” was measured in terms of preconsolidation pressure, \(\sigma'_p\), and the overconsolidation ratio. Correlations between the stress history and several frequently-used piezocone parameters were examined by the authors using simple linear, logarithm, and multiple regression methods. Direct correlations for \(\sigma'_p\) with stress difference \((q_t - \sigma_{vo})\), \(\Delta u_1\), and \((q_t - u_1)\) were developed. Plasticity index was
also incorporated to develop multiple regression analysis. This multiple regression improved statistical correlations by increasing the correlation coefficient.

Robertson (2010) presented an updated correlation to estimate the coefficient of permeability using CPT test results. The correlation evaluated the suggested soil permeability by Lunne et al (1997) with a range of k values for each Soil Behavior Type (SBT) as shown in Table 2.3. The suggested and modified k by Lunne et al (1997) and Robertson (2010) are shown in Figure 2.10 and simplified by Eq. (2.12a) and Eq. (2.12b).

### Table 2.3 Estimated Permeability Based on Normalized Soil Behavior Type, SBTn (Lunne et. al 1997)

<table>
<thead>
<tr>
<th>SBTn Zone</th>
<th>Soil Type</th>
<th>Range of k (cm/sec)</th>
<th>SBTn Index (Ic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive fine-grained</td>
<td>3×10^{-8} to 3×10^{-6}</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td>Organic soil-clay</td>
<td>1×10^{-8} to 1×10^{-6}</td>
<td>Ic &gt; 3.60</td>
</tr>
<tr>
<td>3</td>
<td>Clay</td>
<td>1×10^{-8} to 1×10^{-7}</td>
<td>2.95 &lt; Ic &lt; 3.60</td>
</tr>
<tr>
<td>4</td>
<td>Silt mixture</td>
<td>3×10^{-7} to 1×10^{-5}</td>
<td>2.60 &lt; Ic &lt; 2.95</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixture</td>
<td>1×10^{-5} to 1×10^{-3}</td>
<td>2.05 &lt; Ic &lt; 2.60</td>
</tr>
<tr>
<td>6</td>
<td>Sand</td>
<td>1×10^{-3} to 1×10^{-1}</td>
<td>1.31 &lt; Ic &lt; 2.05</td>
</tr>
<tr>
<td>7</td>
<td>Dense sand to gravelly sand</td>
<td>1×10^{-1} to 1×10^{2}</td>
<td>Ic &lt; 1.31</td>
</tr>
<tr>
<td>8</td>
<td>Very dense / Stiff soil</td>
<td>1×10^{-6} to 1×10^{-4}</td>
<td>NA</td>
</tr>
<tr>
<td>9</td>
<td>Very stiff fine grained soil</td>
<td>1×10^{-7} to 1×10^{-5}</td>
<td>NA</td>
</tr>
</tbody>
</table>

\[ k = 10^{(0.952 - 3.04 I_c)} \text{ m/sec} \] \hspace{1cm} (2.12a)

\[ k = 10^{(-4.52 - 1.37 I_c)} \text{ m/sec} \] \hspace{1cm} (2.12b)

Where:

\[ I_c = \text{soil behavior type index} \]
The state parameter ($\psi$) is defined as the difference between the initial void ratio, $e$ and the void ratio at critical state, $e_{cs}$. The critical state of a soil is the state at which the shear stress remains constant while the shear strain increases (Holtz and Kovacs 1981). A simplified method has been recently developed to estimate the state parameter ($\psi$) from the normalized cone tip resistance. Based on field and laboratory data, Robertson (2009) developed contours of state parameter ($\psi$) on the normalized soil behavior type (SBT) chart presented earlier in Figure 2.6. The SBT chart with the state parameter contours is shown in Figure 2.11. Robertson (2010) suggested an
approximate relationship between the state parameter and the clean sand equivalent normalized cone resistance \((Q_{m,cs})\) as presented in Eq. (2.13).

\[
\psi = 0.56 - 0.33 \log Q_{tn,cs}
\]  

(2.13)

Where:

\(Q_{m,cs} = \) clean sand equivalent normalized cone resistance which is a function of soil behavior type index \((I_c)\).

Figure 2.11 State parameter contours on the Robertson (1990) normalized SBT chart (Robertson 2009)
2.6 Effect of Silt Content and Void Ratio on Soil Hydraulic Conductivity

Bandini and Sathiskumar (2009) studied the effect of silt content and void ratio on the saturated hydraulic conductivity and compressibility of sand-silt mixtures. Two poorly graded quartz sands with rounded grains and a specific gravity of 2.65 were used and called host sands as they constitute the sand matrix hosting the fines. Ottawa sand, which has coefficients of uniformity \((C_u)\) and curvature \((C_c)\) of 1.87 and 1.04 respectively, was used as the host sand in the first set of tests. The second host sand was prepared by mixing equal parts, by dry weight, of Ottawa sand and ASTM 20-30 sand (C-778-93A) and was called 50:50 sand. This sand has \(C_u = 2.46\) and \(C_c = 1.09\). Sand-silt mixtures were prepared by adding Sil-Co-Sil #106 with specific gravity of 2.65 to the host sands with different percent in order to determine the effect of silt content on the hydraulic conductivity and compressibility. Flexible wall permeameter tests were performed on 60 specimens of the two types of host sands with 0, 5, 10, 15, 20, and 25% silt. The \(e_{\text{max}}\) and \(e_{\text{min}}\) values of the sand-silt mixtures were found for each type of sand. It was noticed that as the silt content increases, \(e_{\text{max}}\) and \(e_{\text{min}}\) of these mixtures decrease up to approximately 20 and 25% silt content, respectively. Hydraulic conductivity measurements were conducted at different effective confining stresses for all specimens. The experimental results show that the saturated hydraulic conductivity of sands with 25% silt can be, in average, two orders of magnitude smaller than those of clean sands with 0% silt.
content as shown in Figure 2.12. For given silt content, k varies mostly within one order of magnitude depending on the void ratio of the soil.

Figure 2.12 Hydraulic conductivity for various silt contents (Bandini and Sathiskumar 2009)
Chapter 3
Methodology, Description of Testing Sites and Data Collection

3.1 Dissertation Methodology

In order to meet the first objective of this study, which is identification and evaluation of the characteristics of soil deposits which may cause high pile rebound, existing PDA, SPT, and CPT data was collected and processed from seven sites identified by FDOT as pile rebound sites. Pile driving analyzer (PDA) results were used to identify the zones in the vertical soil profiles where high pile rebound occurred. Soil properties at these sites including, saturated unit weight ($\gamma$), soil behavior type (SBT), permeability ($k$), relative density ($Dr$), state parameter ($\psi$), overconsolidation ratio (OCR), undrained shear strength ($S_u$), and fines content (FC) were estimated from existing CPT data. Geotechnical software, CPeT-IT v.1.4 (2014), was used to process the CPT data. The software was developed by Gregg Drilling and Testing Inc. in collaboration with Professor Robertson.

Disturbed physical soil samples extracted during the SPT testing were used to measure the fines content. Existing data of fines content that was previously measured from samples directly collected from the field was available and was used
to verify the estimated fines content using CPT data. The results of all soil properties were presented versus depth as vertical profiles for each site. Using the PDA data, the rebound zones were identified on these profiles to investigate the difference between the rebound and non-rebound soil properties and to identify the soil properties that had a significant effect on soil rebound.

The second objective of this research is to evaluate the accuracy of existing correlations to predict high pile rebound. To fulfill this objective, the most recent pore pressure ($u_2$) – rebound correlation developed by Jarushi et al. (2013) was selected for evaluation. The mathematical model of the rebound correlation is based on CPT pore water pressure and rebound. Additional CPT pore water pressure data was used to calculate pile rebound using Jarushi et al. (2013) model. The calculated pile rebound was compared to the measured pile rebound data during driving to identify any discrepancies.

The main focus of the third objective of this research is to improve the existing correlation discussed above and to develop additional correlations to predict high pile rebound. A higher number of data points was used to redevelop an improved correlation between the same parameters involved in the previous model (i.e. pile rebound versus CPT pore water, $u_2$). Graphical correlations were produced by superimposing the CPT results on known soil behavior type charts (SBT) using CPeT-IT v.1.4 (2014) software. Details of each single step of the research methodology are explained below.
3.1.1 Identifying Testing Sites and Existing Data

Seven sites in Florida were identified for data analysis to support the objectives of this research. The selection was based on two reasons. The first reason was that these seven sites were clearly identified by FDOT as rebound-sites. The ability to access these sites easily and perform field testing was the other reason. Sites investigated including the following:

1. I-4 / US-192 Interchange / Osceola County / Florida.
2. State Road 417 International Parkway / Osceola County / Florida.
3. State Road 50 and State Road 436 / Orange County / Florida.
4. I-4 / State Road 408 Ramp B / Orange County / Florida.
5. Anderson Street Overpass at I-4/SR-408 / Orange County / Florida.
6. I-4 Widening Daytona / Volusia County / Florida.
7. State Road 83 over Ramsey Branch Bridge / Walton County / Florida.

Detailed geotechnical and construction data for each site was provided by FDOT and their contractors. The collected data included project contract drawings, test pile locations, PDA data; and locations and data of existing SPT borings and CPT soundings.
3.1.2 Analysis of PDA Data and Identify Rebound Zones

During the initial pile installation, test piles at the sites were instrumented with accelerometers and strain transducers to check pile drivability and identify rebound zones. Data recorders collect the maximum displacement (i.e. DMX) and digital final set (DFN) signals during pile driving and yielded data sets of maximum displacement-depth and final set-depth. At the same time, FDOT inspectors recorded the number of blows required to drive the pile one foot on pile driving logs. A parameter called the inspector set (iset) in the units of inch/blow is calculated by taking the reciprocal of the number of blows recorded during each foot. In case where the number of blows per foot exceeds 240 (i.e. 20 blow/inch), the test pile driving process is terminated. This situation is referred to as pile refusal. In this study, the inspector set was used to calculate pile rebound by subtracting it from the DMX as follow:

\[
\text{Rebound (in/blow)} = \text{DMX (in/blow)} - \text{inspector set (in/blow)} \quad (3.1)
\]

Pile rebound (in/blow) for each test pile at each site was presented versus depth in order to identify rebound zones. Pile rebound varied and ranged from 0 to 1.5 inch per blow. Excessive rebound was considered to be any rebound exceeding FDOT’s specification, Section 455 of 0.25 in/blow or blow counts exceeding 50 blow/ft.
3.1.3 Field Tests and Sampling

Piezocone penetration tests were conducted at FDOT project sites using electrical cone penetrometer with pore water transducer behind the cone tip. Piezocone penetration tests (CPT) were performed according to ASTM D-5778 near the associated test piles until refusal or desired depth was met. The measured CPT parameters are cone tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure at cone shoulder ($u_2$) were plotted versus depth and used to estimate soil properties.

Standard penetration tests (SPT) were performed by FDOT as near as possible to the test piles at each site. The SPT borings extended deeper than the associated test piles. Standard penetration test borings were performed according to the ASTM D-1586 standard. A 140-lb safety hammer was used to drive the SPT sampler with a 30 inch free dropping distance. Disturbed soil samples were retrieved every 5 ft. using 2-inch OD diameter and 24-inch long split spoon sampler. The number of blows required to drive the SPT sampler 1 ft. was recorded as the N values ($N_{SPT}$). The N-value was not adjusted for any corrections. When the number of blows exceeds 50 before the sampler is driven 1 ft., the number of blow is recorded as 50 blows per the distance driven, which is normally less than 1 ft. Representative disturbed soil samples were collected and packaged for further FDOT examination and laboratory testing.
3.1.4 Samples Testing and Data Processing:

A soil profile log for each SPT boring was developed based on field classification of retrieved samples during SPT sampling. The boring logs described the soil type at each layer using the Unified Soil Classification System (USCS) symbols (e.g., SP-SM) and American Society for Testing and Materials (ASTM) soil descriptions (e.g., sand with silt). The recorded number of blows per foot (N) was graphically represented versus depth for each boring at each site.

Piezocone penetration test data including cone tip resistance \((q_c)\), sleeve friction \((f_s)\), and pore water pressure \((u_2)\) were presented versus depth. Soil properties including saturated unit weight \((\gamma)\), soil behavior type (SBT), permeability \((k)\), relative density \((Dr)\), state parameter \((\psi)\), overconsolidation ratio \((OCR)\), undrained shear strength \((Su)\), and fines content \((FC)\) were estimated using the CPT data.

The CPT results were normalized for the effective stresses and used to develop soil behavior type charts (SBT) for the rebound and non-rebound zones in all seven sites. The normalized CPT data was overlaid on five existing SBT charts developed by Robertson (1990), Robertson (2012), Schneider (2008), and Eslami and Fellenius (1997).

3.2 Description of Sites and Field Testing Data

A general description included the geographic location and field testing locations of the studied sites is presented in this section. The main three sources of field data included pile driving analyzer (PDA) records, standard penetration tests
(SPT), and cone penetration tests (CPT). The SPT and CPT were conducted in the vicinity of the test piles of each site. These tests were performed by drilling crews from Ardaman & Associates, Inc. and the FDOT State Materials Office (SMO).

As previously mentioned, the main output data obtained from PDA is the maximum displacement, the final set, and the inspector set. In order to retrieve the rebound levels from the test and to further determine the rebound and non-rebound zones in the soil vertical profiles, it is necessary to calculate the rebound from the displacement difference before and after the rebound occurs. Usually, the difference between the maximum displacement and the final set gives the rebound level. However, it is important to mention that the data acquisition equipment used in the PDA test does not output a direct displacement measurement. The accelerometers attached to the test pile measure the pile acceleration levels. The displacement is indirectly calculated by double integrating the acceleration signal. Accelerometers often output their acceleration signals with significant noise levels. As a result, double integrating the acceleration signals causes the calculated displacement to drift far away from the actual level and significant error may be incorporated in the obtained displacement results. For this reason, it was decided that calculating the rebound from the inspector set records would produce more reliable results. Plots of vertical profile rebound histories were developed for all the study sites, and values of digital set retrieved from the data acquisition equipment were plotted with the rebound profiles. Vertical profiles of pile rebound, digital set, and driving blow count versus depth were developed and presented for each site in this chapter. These
profiles were used to clearly identify the rebound zones associated with pile rebound
greater than 0.25 inch/blow, low digital set, and high number of blows.

The second source of filed data for this study was the standard penetration
tests (SPT). Disturbed soil samples collected during the SPT testing were used for
soil classification both visually and based of USCS system. Results of soil
classification obtained were then used to develop boring logs or general soil profiles
for each tested site. The resulting profiles were presented in a geotechnical format.
The boring logs described the soil type at each layer using the Unified Soil
Classification System (USCS) symbols. The recorded number of blows per foot (N)
was graphically presented versus depth for each boring at each site. If the number of
blows (N) exceeds 50 blows with a distance (D) less than 12 inch, the number of
blows is multiplied by (12 / D) to calculate N per foot. General soil profiles showing
soil stratification and the actual number of blows per foot (N) were developed for all
the seven sites and presented in this section. These profiles were used to identify the
soil type exists in the rebound and non-rebound zones and discuss if there any change
in the number of blow (N).

The other field testing conducted in the study sites was the piezocone
penetration test (CPT). The CPT tests were performed after the pile construction was
finished by a period of time. The soil properties obtained from the CPT tests are the
cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u_2). Results of
the measured CPT parameters were plotted versus depth to create their vertical
profiles. The vertical profiles of the three CPT parameters measured at all soundings
for all study sites are presented in this section. Based on the rebound results drawn from the PDA testing, the identified rebound zones were highlighted in the profiles of the CPT parameters.

3.2.1 Location I-4 / US-192 Interchange

3.2.1.1 General Description and Field Testing Location

The I-4 / US-192 interchange is located in Central Florida, Osceola County. The site consists of two ramps (CA and BD) and two bridges (US-192 Westbound and US-192 Eastbound). The approximate ground surface elevation (GSE) ranges from 95 ft. to 109 ft. (NAVD88). The ground water table (G.W.T) is located at 10 to 15 ft. below the GSE. The bridge piers consist of 24-inch square prestressed concrete piles (PCP), 115 ft. long. Three piles (pier 6 / pile 16, pier 7 / pile 10 and pier 8 / pile 4) in ramp CA were used as test piles. These three test piles were driven with an ICE 120 S single-acting diesel hammer with a rated energy of 120 ft-kips (139 kJ) using a 9-inch thick plywood pile cushion. Pile installation began with predrilling the soil to 30 ft. from the GSE followed by pile driving approximately 70 ft. through the soil. Standard penetration test (SPT) and CPT tests were performed 15 ft. to 20 ft. from these test piles. Figure 3.1 shows the site with the approximate filed testing locations.
3.2.1.2 Pile Driving Analyzer (PDA) Data and Identification of HPR Zones

At location I-4 / US-192 Interchange, the PDA test started at a predrilling depth of 30 ft. At this site, Figure 3.2 shows a rebound of up to 0.25 in/blow at a depth of 40-70 ft for all three test piles and the corresponding pile set ranged from 0.25 to 0.5 in/blow. By inspecting Figure 3.2, the significant rebound zone can be identified at a depth 70-100 ft with a pile rebound ranges from 0.25 to 0.92 inch/blow and pile set being less than 0.25 inch/blow. This rebound zone also experienced a rapid increase in the number of blows from an average of 50 blow/ft to 300 blow/ft as can be seen in Figure 3.2.
Figure 3.2 Digital set, pile rebound, and blow count versus depth for (a) pier 6/pile 16, (b) pier 7/pile 10 and (c) pier 8/pile 4 for I-4 / US 192 interchange
3.2.1.3 Standard Penetration Test (SPT) Data and General Soil Profile

At location I-4 / US-192 Interchange, three SPT borings (i.e. B-39, B-40, and B-41) were performed approximately 15 to 20 ft away from the associated test piles at pier 6, 7, and 8, respectively. A general soil profile showing soil stratification and the actual number of blows per foot (N) was developed as illustrated in Figure 3.3. The measured G.S.E of borings B-39 and B-40 were 109.6 and 108.6, respectively. Boring B-41 had a GSE of 90.2 ft which was 18 ft lower than those of borings B-39 and B-40. For comparison purposes the depth difference (i.e., 18 ft) was adjusted to all depth data points at B-41. The SPT borings extended to a depth of 180 ft from the G.S.E. The rebound zone in location I-4 / US 192 interchange, which is located at depth 70 ft. to 100 ft., is characterized by cemented fine sand (SM) with trace phosphate and shell. The number of blows (N) ranges from 15 to 25 blow/ft.
Figure 3.3 General soil profile with USCS classification and the actual number of blows (N) from SPT borings B-39, B-40 and B-41 for I-4 / US 192 Interchange
3.2.1.4 Cone Penetration Test (CPT) Data

Three CPT soundings, CPT-4, CPT-3, and CPT-2 were conducted about 15 ft. to 20 ft. from the test piles in ramp CA, pier 6/ pile 16, pier 7/ pile 10, and pier 8/ pile 4 respectively. Three parameters: cone tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure at the cone shoulder ($u_2$) were measured every two inches in depth. The CPT soundings extended to a depth of 96 ft, 95 ft, and 105ft near pier 6, pier 7, and pier 8 respectively. The collected data was presented versus depth as illustrated in Figures 3.4, 3.5, and 3.6. The rebound zones at all the test piles were highlighted in the CPT profiles in order to identify if there is any difference in the measured CPT parameters (i.e. $q_c$, $f_s$, and $u_2$) at the rebound zones.

Figures 3.4, 3.5, and 3.6 show that the cone resistance has an average of 100 tsf in the rebound zones (70 ft. to 90ft.) while it ranges from 100 tsf to 600 tsf in the non-rebound zones. The sleeve friction ($f_s$) increased from 1 tsf to 4 tsf in the rebound zones, while it has an average of 1 tsf in the upper non-rebound zone (30 ft. to 70 ft.). However, a zone of high $q_c$ and $f_s$ is located at depth (10 ft. to 30 ft.). This zone was predrilled before pile driving; therefore, no pile rebound occurred. The CPT pore water pressure ($u_2$) profiles presented in Figures 3.4, 3.5, and 3.6, show a rapid increase in $u_2$ in the rebound zones for all the three CPT soundings. The pore water pressure increased from low values to 420 psi within a vertical distance about 10 ft. of soil.
Figure 3.4 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-4 near pier 6/pile 16 at I-4 / US 192

Figure 3.5 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-3 near pier 7/pile 10 at I-4 / US 192
Figure 3.6 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-2 near pier 8/pile 4 at I-4 / US 192
3.2.2 Location State Road 417 International Parkway

3.2.2.1 General Description and Field Testing Locations

The SR 417 International Parkway is located in Osceola County, 15 miles north of Orlando, Florida. It consists of one ramp bridge with two abutments (B1 and B2) and connects SR 417 westbound to I-4 westbound as shown in Figure 3.7. The GSE at both end bents was 72.3 ft. (NAVD88) and the G.W.T is located at 6 ft. below the GSE. Twenty four inch square PCP, 100 ft. in length were used to construct the two end piers to support the ramp bridge at SR 417. Two test piles were instrumented with PDA, pile 14 at abutment B1 and pile 5 at abutment B2, to check pile drivability. The piles at B1 and B2 were predrilled to 15 FT. and 27 ft. from GSE respectively and then driven with an APE D 46-42 single-acting diesel hammer with a rated energy of 120 ft-kips (139 kJ). A nine inch thick plywood pile cushion was used during pile driving. Two SPT borings and two CPT soundings were conducted at approximately 30 ft. away from the test piles.

3.2.2.2 Pile Driving Analyzer (PDA) Data and Identification of HPR Zones

A thin rebound zone at a depth of 70-80 ft. can be noticed at B1 / pile 14 in Figure 3.8a for location State Road 417 International Parkway with a maximum recorded rebound of 0.5 inch/blow and a maximum pile set higher than 0.25 inch/blow. Above this zone, low rebound can be noticed with an average value of about 0.1 inch except at a depth of 30-40 ft. where a rebound of about 0.35 inch/blow was recorded. The number of blows of test pile 14 at B1 also increased to 87 blow/ft
at the rebound zone as shown in Figure 3.8a. The test pile 5 at B2 only exceeded the rebound limit of 0.25 in/blow at a depth of 20-35 ft. and then an approximately constant low rebound zone (i.e., less than 0.25 in/blow) appeared at a depth of 35-80 ft. The corresponding number of blows presented in Figure 3.8b at this test pile came low (i.e., less than 50 blow/ft.) compared that recorded at test pile 14 at B1.

Figure 3.7 Test piles, SPT boring, and CPT sounding locations for SR 417 International Parkway
3.2.2.3 Standard Penetration Test (SPT) Data and General Soil Profile

Two SPT borings (i.e., SPT-B1 and SPT-B2) were performed at the State Road 417 International Parkway location. A general soil profile and the recorded number of blows per foot from SPT boings is presented graphically versus depth in Figure 3.9. Boring SPT-B1 was located 10 ft. away from test pile 14 at B1 while boring SPT-B2 was located 15 ft away from test pile 5 at B2. The G.S.E at these test
piles was 72.3 ft. The SPT borings extended to a depth of 93 ft from the G.S.E. It can be noticed that cemented silty fine sand (SM) with trace phosphate and shell exists in the rebound zone (i.e. 70 ft. to 80 ft.). The average number of blows, N is 50 blow/ft.

Figure 3.9 General soil profile with USCS classification and the actual number of blows (N) from SPT borings SPT-B1 and SPT-B2 for SR 417 International Parkway
3.2.2.4 Cone Penetration Test (CPT) Data

Two CPT soundings, CPT-1 and CPT-3 were conducted about 30 ft. from the test piles in B1 and B2 respectively. Three parameters measured during the test were: cone tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure at the cone shoulder ($u_2$). The data of these three parameters was collected every two inches. The CPT soundings extended to a depth of 74 ft and 70 ft near B1 and B2 respectively. The collected data is presented versus depth as illustrated in Figures 3.10 and 3.11. A thin rebound zone (70 ft. to 75 ft.) at B1 is highlighted in the CPT profiles in order to identify if there is any difference in the measured CPT parameters (i.e. $q_c$, $f_s$, and $u_2$) at the rebound zones. No rebound zones exist in B2 / pile 5.

Figures 3.10 and 3.11 show that the cone resistance ranges from 50 tsf to 250 tsf in all depths. The sleeve friction ($f_s$) ranges from 0.5 tsf to 2 tsf in all depths. Inspecting the CPT pore water pressure ($u_2$) profiles shown in Figures 3.10 and 3.11, the pore water pressure increased to a maximum value of 60 psi. State Road 417 International Parkway is considered non-rebound site.
Figure 3.10 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-1 near B1 / pile 14 at SR 417 International Parkway

Figure 3.11 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-3 near B2 / pile 5 at SR 417 International Parkway
3.2.3 Location State Road 50 over State Road 436

3.2.3.1 General Description and Field Testing Locations

This site consists of the intersection of two bridges, SR 50 and SR 436 and is located in Orlando, Orange County, Florida. State road 50 bridge extends over SR 436 from the west to the east of SR 436 as shown in Figure 3.12. The GSE of the site is 99 ft. (NAVD88) and the G.W.T is located at 4 ft. below the GSE. Twenty-four inch square prestressed concrete piles 101 ft. long were designed to support the SR50 / SR436 overpass. Pile No. 5 at the west bound of the overpass was instrumented with PDA to evaluate pile drivability. The test pile was placed in a predrilled hole to a depth of 32 ft. from GSE before driving. A pile cushion, 14 inch thick, was used to drive the test pile using an APE D62-42 single-acting diesel hammer with a ram weight of 13 kips and energy of 154 ft-kips (210 kJ). Field testing for the SPT boring and CPT sounding were conducted 35 ft. away from the test pile No.5.

Figure 3.12 Test Piles, SPT Boring, and CPT Sounding Locations for SR50/SR436
3.2.3.2 Pile Driving Analyzer (PDA) Data and Identification of HPR Zones

At location State Road 50 over State Road 436, and as shown in Figure 3.13, the rebound increases from zero to 0.65 inch/blow at a depth of 40 and is relatively constant to a depth of 55 ft. At a depth of 65 ft., it decreases to zero. A rebound zone with a maximum rebound of 1 inch/blow occurs at a depth of 70-80 ft.

In terms of the number of blows, Figure 3.13 shows that a minimum number of blows of up to 4 blow/ft. in the non-rebound zone and a maximum number of up to 300 blow/ft. in the rebound zone were recorded. In terms of the digital set; a maximum of 2 inches set was recorded at depth 32 ft. to 70 ft. and a minimum of zero to 0.35 inches at depth 70 ft. to 80 ft. as can be seen in Figure 3.13. Therefore, the rebound zone at location SR 50/ SR 436 is located at depth 70 ft. to 80 ft.
3.2.3.3 Standard Penetration Test (SPT) Data and General Soil Profile

Standard penetration test boring TH-4B at location SR 50/ SR 436 was driven 30 ft away from the test pile No.5 located at the west bound of the bridge. Figure 3.14 presents a general soil profile and actual number of blows per foot recorded during the SPT. The G.S.E of this boring was 99 ft. The SPT boring was predrilled to a depth of 10 ft and extended to a depth of 100 ft from the measured G.S.E. The rebound zones (70 ft. to 80 ft.) is characterized by silty fine sand (SM) with trace clay and high plasticity clay (CH) with trace phosphate. The average number of blows, N is 25 blow/ft for all depths except the rebound depth where N suddenly increased from 15 blow/ft. to 150 blow/ft.
Figure 3.14 General soil profile with USCS classification and the actual number of blows (N) from SPT boring TH-4B near pile No. 5 at the west bound of SR50/SR436
3.2.3.4 Cone Penetration Test (CPT) Data

A piezocone penetration test, CPT-1, was conducted 35 ft. away from test pile 5 in the west bound of location SR 50 / SR 436. Three parameters were measured during the test: cone tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure at the cone shoulder ($u_2$). The data of these three parameters was collected every two inches. The CPT sounding extended to a depth of 77 ft below G.S.E. The collected data was presented versus depth as illustrated in Figure 3.15. Using the PDA data presented in Figure 3.13, the rebound zone (i.e. 70 ft. to 80 ft.) was highlighted in the CPT profiles in order to identify if there is any difference in the measured CPT parameters (i.e. $q_c$, $f_s$, and $u_2$) at the rebound zone.

Figure 3.15 shows that the cone resistance ranges from 30 to 100 tsf in the rebound zone, while it ranges from 50 tsf to 300 tsf in the non-rebound zones. The sleeve friction ($f_s$) increased from 0.5 tsf to 2.5 tsf in the rebound zones, while it has an average of 1 tsf in the upper non-rebound zone (30 ft. to 70 ft.). However, a zone of high $q_c$ and $f_s$ is located at depth (10 ft. to 30 ft.). This zone was predrilled before piles driving; therefore, no information available about pile rebound. Inspecting the CPT pore water pressure ($u_2$) profile shown in Figure 3.15, a rapid increase in $u_2$ was found in the rebound zones. The pore water pressure increased from 50 psi to 300 psi within a thin vertical distance about 5 ft. of soil.
Figure 3.15 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-1 near pile 5 at the west bound of SR 50/SR 436

### 3.2.4 Location I-4 / State Road 408 Interchange, Ramp B

#### 3.2.4.1 General Description and Field Testing Locations

The I-4 / SR 408 interchange is located in downtown Orlando, Orange County, Florida. The GSE of the site is 106 ft. (NAVD88) and the G.W.T is located at 5 ft. to 10 ft. below the GSE. SR 408 where ramp B extend over I-4. Ramp B consists of 16 piers with bridge spans length varying from 139 ft. to 263 ft. Pile No. 5 at the second pier of ramp B was driven as an instrumented test pile. The test pile was 18-inch square PCP with a length of 97 ft. A Delmag D36-32 single-acting diesel hammer with a 9-inch plywood cushion was used to drive the pile into a predrilled
hole to a depth of 95 ft. from GSE. A SPT boring and a CPT sounding were conducted 75 ft. away from the test pile as shown in Figure 3.16.

Figure 3.16 Test Pile, SPT Boring, and CPT Sounding Locations for I-4 / SR 408 (Ramp B)

3.2.4.2 Pile Driving Analyzer (PDA) Data and Identification of HPR Zones

An average rebound of 0.35 inch/blow was recorded at a depth range of 10-95 ft during driving of pile 5 at the second pier of I-4 / State Road 408 Ramp B with an average pile set of 0.25 in/blows as can be seen in Figure 3.17. The average number of blows ranges from 25 to 50 blow/ft. This site can be considered a non-rebound site.
3.2.4.3 Standard Penetration Test (SPT) Data and General Soil Profile

The SPT boring B-101 with a GSE of 103.3 ft. was conducted a distance of 20 ft away from the test pile No.5 at the second pier of ramp B at the I-4/State Road 408 location. This boring was predrilled to a depth of 10 ft and extended approximately to a depth of 135 ft from the mentioned G.S.E. The number of blows per foot was recorded and presented versus depth as illustrated in Figure 3.18. The average number of blows, N is 10 blow/ft at depths 10 ft. to 70 ft. and then increased to an average of 20 blow/ft. at depth 70 ft. to 95 ft. The disturbed samples extracted during the test were classified according to the USCS and used to develop the geotechnical profile shown in Figure 3.18.

Figure 3.17 Digital set, pile rebound, and blow count versus depth for pier 2/pile 5 at Ramp B of I-4/SR 408
Figure 3.18 General soil profile with USCS classification and the actual number of blows (N) from SPT boring B-101 near pier 2/pile 5 at ramp B of the I-4/SR 408
3.2.4.4 Cone Penetration Test (CPT) Data

A piezocone penetration test, CPT B-109, was conducted 75 ft. away from the test pile 5 in the second pier of ramp B at the I-4 / SR 408 location. Three parameters were measured during the test: cone tip resistance \( q_c \), sleeve friction \( f_s \), and pore water pressure at the cone shoulder \( u_2 \). The data for these three parameters was collected every one inch. The CPT sounding extended to a depth of 92.5 ft below G.S.E. The collected data was presented versus depth as illustrated in Figure 3.19. No high pile rebound zones were highlighted because this site was considered a non-rebound site.

Figures 3.19 shows that the cone resistance ranges from 30 to 120 tsf in all depths. The sleeve friction \( f_s \) ranges from 0.25 tsf to 0.75 tsf through the whole profile. Inspecting the CPT pore water pressure \( u_2 \) profile shown in Figure 3.19, rapid increase in \( u_2 \) was found at depth 55 ft. to 99 ft. By matching the PDA in Figure 3.17, an increase in the number of blows from 25 to 50 blow/ft. occurred at that zone (55 ft. to 99 ft.). However, the pile rebound has an average of 0.25 inch/blow.
3.2.5 Location Anderson Street Overpass

3.2.5.1 General Description and Field Testing Locations

Anderson street overpass is located at the intersection of Interstate 4 (I-4) and State Road 408 (SR408), in downtown Orlando, Orange County, Florida. The GSE of this site is 104 ft. (NAVD88) and the G.W.T is located at 6 ft. to 8 ft. from the GSE. The bridge, was constructed using 24-inch square PCPs, 124 ft. in length, is supported by six piers and two end bents. Two piles at pier 6, pile No.5 and pile No.6, were selected as test piles. The piles were installed in predrilled holes (10 ft. and 30 ft. respectively) and driven with a Delmag D62 single-acting diesel hammer with a rated energy of 90 ft-kips. (122 kJ). Plywood cushions of 12 inch or 16 inch in thickness were used during driving process. A SPT boring and a CPT sounding were conducted 100 ft. away from the test piles as shown in Figure 3.20.
3.2.5.2 Pile Driving Analyzer (PDA) Data and Identification of HPR Zones

At the Anderson street overpass location, the records for the two test piles, pile 5 and pile 6, at pier 6 are presented in Figure 3.21a and 3.21b respectively. No significant rebound can be noticed for either pile to a depth with a recorded rebound average of up to 0.25 inch/blow. The corresponding pile set at the same depth interval ranges from 0.7 to 2 inch/blow. The significant rebound zone is located at a depth of 93-110 ft with a rebound up to 1.1 inch/blow and a corresponding average set of 0.12 inch/blow. Figure 3.21 shows the non-rebound zone located above 93 ft depth resulted in an average number of blows of about 20 blow/ft. The rebound zone located at a depth of 93-110 ft had an elevated number of blows of up to 365 blow/ft.
Figure 3.21 Digital set, pile rebound, and blow count versus depth for (a) pier 6 / pile 6 and (b) pier 6 / pile 5 at Anderson street overpass

3.2.5.3 Standard Penetration Test (SPT) Data and General Soil Profile

At a distance of 40 ft from the test piles 5 and 6 at pier 6 at location Anderson Street Overpass, the SPT P6-3 boring was driven with a 104 ft G.S.E. The predrilling depth of this boring was 7 ft and the test extended approximately to a depth of 120 ft from the G.S.E. Number of blows per foot was recorded and presented versus depth as illustrated in Figure 3.22. The number of blows, N at depth 7 ft. to 90 ft. has an average of 10 blows/ft. then increased to 40 blow/ft. at depth more than 93 ft. The general soil profile is shown in Figure 3.22. Three different soils, silty clayey fine sand (SM/SC), clayey fine sand (SC) and clay (CH), exist in the rebound zone (90 ft. to 110 ft.)
Figure 3.22 General soil profile with USCS classification and the actual number of blows (N) from SPT boring SPT P6-3 near pier 6 at Anderson street overpass.
3.2.5.4 Cone Penetration Test (CPT) Data

The CPT-5 sounding was conducted about 100 ft. from pier 6 at Anderson street overpass. Three parameters collected every two inches were measured during the test: cone tip resistance \((q_c)\), sleeve friction \((f_s)\), and pore water pressure at the cone shoulder \((u_2)\). The CPT sounding extended to a depth of 92 ft. below the G.S.E. The collected data was presented versus depth as illustrated in Figure 3.23. By matching with the PDA presented in Figure 3.21, the rebound zone was identified at a depth of 93 ft. to 110 ft. Therefore, no conclusion can be made in the rebound zone because the CPT sounding was terminated at the start point of the rebound zone. However, the CPT data were used and discussed as a non-rebound CPT data.

![Graph](image)

Figure 3.23 Tip resistance \((q_c)\), sleeve friction \((f_s)\), and pore water pressure \((u_2)\) versus depth from CPT-5 near pier 6 at Anderson street overpass
3.2.6 Location I-4 Widening Daytona

3.2.6.1 General Description and Field Testing Locations

I-4 / Deer Wildlife crossing is located in Daytona, Volusia County, Florida. The approximate GSE is 42 ft. (NAVD88) and the G.W.T is located at 7 ft. below the GSE. The site consists of three end bents (EB-1, EB-2, and EB-3). Prestressed concrete piles were used to support the bridges at this site. The test pile No.5, 24-inch square PCP, 115 ft. long, was located at EB-3 and was driven with an APE D46-42 S single-acting diesel hammer with a rated energy of 114.11 ft-kips. The pile cushion consisted of 3.5-inch Micarta 2×1 inch. The pile installation began with predrilling process of 33 ft. below the GSE. The test pile was driven 63 ft. through the soil yielding a total pile penetration below the ground of 96 ft. A SPT boring and a CPT sounding were performed at 56 ft from the test pile. The general site location and the approximate filed testing locations is shown in Figure 3.24.
3.2.6.2 Pile Driving Analyzer (PDA) Data and Identification of HPR Zones

At I-4 Widening Daytona location, PDA data collection started at a depth of 45 ft as can be seen in Figure 3.25. A low rebound zone is identified at a depth of 45-80 ft with a rebound being less than 0.25 in/blow. The significant rebound zone is found to be located at a depth of 80-90 ft where the rebound increases up to 0.5 inch/blow with a corresponding average set of up to 0.25 inch/bow. Below a depth of 90 ft, the rebound starts decreasing back again below 0.25 inch/blow. In Figure 3.25, a noticeable increase in the number of blows occurred at depth of 80 ft. from an average of 35 blow/ft in the non-rebound zone to about 150 blow/ft in the rebound zone.
Figure 3.25 Digital set, pile rebound, and blow count versus depth for EB3 / pile 5 at I-4 Widening Daytona

3.2.6.3 Standard Penetration Test (SPT) Data and General Soil Profile

Standard penetration test boring DC-1 at location I-4 Widening Daytona was driven 56 ft. away from the test pile No.5 located at the third end bent of the bridge. The measured G.S.E of this boring was 109.6 ft. The SPT boring was extended approximately to a depth of 110 ft. from the measured G.S.E. Figure 3.26 presents the recorded number of blows per foot recorded during the test. The average N is 12 blow/ft until a depth of 95 ft. and then a higher N was recorded. The soil in the rebound zone (i.e. 63 ft. to 76 ft.) was classified as silty fine sand with trace shell.
Figure 3.26 General soil profile with USCS classification and the actual number of blows (N) from SPT boring SPT DC-1 near EB3 / pile 5 at I-4 Widening Daytona
3.2.6.4 Cone Penetration Test (CPT) Data

A CPT sounding was conducted about 56 ft. from the test pile 5 EB3 of location I-4 Widening Daytona. Three parameters were measured during the test: cone tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure at the cone shoulder ($u_2$). The data of these three parameters was collected every two inches. The CPT sounding extended to a depth of 71.5 ft. below the G.S.E. The collected data was presented versus depth as illustrated in Figure 3.27. By matching with the PDA presented in Figure 3.25, the rebound zone was identified at a depth of 80 ft. Therefore, no conclusion can be made in the rebound zone because the CPT sounding was terminated before the start point of the rebound zone. However, the CPT data were used for discussion as a non-rebound site.

Figure 3.27 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth near EB3 / pile 5 at I-4 Widening Daytona
3.2.7 Location State Road 83 over Ramsey Branch Bridge

3.2.7.1 General Description and File Testing Locations

Ramsey Branch Bridge is located north of intersection of SR 83 (US 331) and Ramsey Branch Road in Walton County, Florida. The GSE of the site is 1 ft. (NAVD 88) and the G.W.T is located at 1 ft. below the GSE. The bridge consists of three middle bents and two end bents. The end bents EB1 and EB5 consist of four 24-inch square PCPs while the middle bents (EB2, EB3, and EB4) consist of six 24-inch square PCPs. Pile No. 2 at the end bent 5 was selected as a test pile for the analysis. Pile installation began with predrilling the soil to 33 ft. below the GSE followed by pile driving approximately 57 ft. through the ground. An APE D50-42 S single-acting diesel hammer was used for pile driving with a rated energy of 115.6 ft-kips. Two CPT tests, CPT-1 and CPT-2, and three SPT borings, B-1, B-2, and B-3, were conducted at the site. Figure 3.28 shows the overall view of the site and the field testing locations.
3.2.7.2 Pile Driving Analyzer Data (PDA) and Identification of HPR Zones

At the State Road 83 over Ramsey Branch Bridge location showing in Figure 3.29 all the tested depths experienced a rebound of higher than 0.25 inch/blow. The whole depth can be considered as a rebound zone. The maximum rebound of 0.5 inch/blow occurred at depths of 40 ft. and 63 ft. The set corresponding to the maximum rebound was 1 to 3 inch/blow. A maximum number of blows of 12 blow/ft. occurred at the depths 40 ft. to 63 ft. As can be seen in Figure 3.29, the zone from 63 ft. to 76 ft. has high pile rebound ranged from 0.5 to 1.5 inch/blow and an average low pile set of 0.15 inch/blow. At a depth of 63 ft., the number of blows was suddenly increased from 12 to 150 blow/ft. Therefore, the rebound zone occurred at depth 63 ft. to 76 ft.
3.2.7.3 Standard Penetration Test (SPT) Data and General Soil Profile

The SPT boring B-3 with a 1.0 ft GSE was driven on a distance of 60 ft away from the test pile No.2 at end bent 5 of the location State Road 83 over Ramsey Branch Bridge. This boring was extended approximately to a depth of 100 ft from the mentioned G.S.E. Number of blows per foot was recorded and presented versus depth as illustrated in Figure 3.30. The soil in the rebound zone (i.e. 63 ft. to 76 ft.) was classified as cemented clayey fine sand (SC)
Figure 3.30 General soil profile with USCS classification and the actual number of blows (N) from SPT boring B-3 near EB5 / pile 2 at SR 83/ Ramsey Branch Bridge
3.2.7.4 Cone Penetration Test (CPT)

Two CPT soundings, CPT-1 and CPT-2 were conducted about 28 ft. north EB5 / pile 2 at SR 83/ Ramsey Branch Bridge. Three parameters were measured during the test: cone tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure at the cone shoulder ($u_2$). The data for these three parameters was collected every two inches. The CPT soundings extended to an average depth of 77 ft. The collected data was presented versus depth as illustrated in Figures 3.31 and 3.32. The rebound zones at all the test piles were highlighted in the CPT profiles in order to identify if there is any difference in the measured CPT parameters (i.e. $q_c$, $f_s$, and $u_2$) at the rebound zones.

Figures 3.31 and 3.32 show that the cone resistance has an average of 50 tsf at the zone from the G.S.E to 62 ft. and then increased to an average of 250 tsf below 62 ft. The sleeve friction ($f_s$) fluctuated from 0.5 to 3.5 tsf through all depths. Inspecting the CPT pore water pressure ($u_2$) profiles shown in Figures 3.31 and 3.32, rapid increase in $u_2$ was started at 40 ft. 74 ft. The pore water pressure increased from 1 psi to 570 psi within a vertical distance about 5 ft. of soil.
Figure 3.31 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-1 near EB5 / pile 2 at SR 83/ Ramsey Branch Bridge

Figure 3.32 Tip resistance ($q_c$), sleeve friction ($f_s$), and pore water pressure ($u_2$) versus depth from CPT-2 near EB5 / pile 2 at SR 83/ Ramsey Branch Bridge
3.3 Summary of Field Testing Results

The SPT borings data shows that seven major soil types were found at all the investigated sites. These soil types are: clayey sand with and without trace shell (SC), silty sand soil with and without trace phosphate and shell (SM), clay (CH), poorly graded fine sand (SP), poorly graded fine sand with silt (SP-SM), fine sand with clay (SP-SC), and silty clayey fine sand (SM-SC). All soils in the rebound zones are characterized by cemented clayey fine sand (SC) or silty fine sand (SM) with trace phosphate and shell.

Measured CPT pore water pressure \((u_2)\) profiles show an excellent relationship between measured pore pressure during CPT soundings and pile rebound. Generally, CPT pore water pressure was found to increase rapidly from low values up to 570 psi within a vertical distance of 5 ft. to 10 ft. of soil. The rapid jump in pressure is coinciding with and started at the same depth of the measured increase in soil rebound. The identified zones with high pile rebound up to 1.5 inch/blow and blow count 100 to 300 blow/ft. are characterized by elevated pore pressure up to 570 psi which is noticeably higher than that in the zones of rebound less than 0.25 inch/blow and blow count less than 50 blow/ft. Therefore, a direct correlation between pile rebound and CPT pore water pressure was developed in order to help engineers to predict the rebound zones during the design phase by conducting a CPT sounding.
Inspecting the cone resistance \( (q_c) \) and sleeve friction \( (f_s) \) profiles, no noticeable difference in the soil behavior in terms of these two parameters between the rebound and non-rebound zones can be found. The cone tip resistance was slightly lower in the rebound zones while the sleeve friction was higher. The field testing data was summarized in Table 3.1.

The obtained CPT data of tip resistance and sleeve friction, while not correlated with HPR, were used with the CPT pore pressure to estimate soil properties that were evaluated for the possibility of having effects on the HPR. Existing correlations were used to estimate soil properties from the CPT data. The CPT data was superimposed on multiple soil behavior type (SBT) charts to aid in characterizing the soils having high rebound levels. The details are described in chapter 4.
Table 3.1 Summary of Field Testing Data for all Investigated Locations with Rebound Zones Shaded

<table>
<thead>
<tr>
<th>Location Name</th>
<th>Depth (ft.)</th>
<th>PDA Data</th>
<th>SPT Data</th>
<th>CPT Data</th>
<th>Cone resistance $q_c$ (tsf)</th>
<th>Sleeve friction $f_s$ (tsf)</th>
<th>Pore pressure $u_z$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rebound (inch/blow)</td>
<td>Inspector Set (inch/blow)</td>
<td>No. of Blows (blow/ft.)</td>
<td>N</td>
<td>USCS Soil Type</td>
<td>Cone resistance $q_c$ (tsf)</td>
<td>Sleeve friction $f_s$ (tsf)</td>
</tr>
<tr>
<td>I-4 / US-192 Interchange</td>
<td>40 – 65</td>
<td>≤ 0.25</td>
<td>0.25 to 0.5</td>
<td>Avg. 50</td>
<td>&gt; 25</td>
<td>SP, SP-SM</td>
<td>100-300</td>
</tr>
<tr>
<td></td>
<td>65 – 100</td>
<td>0.25 to 0.9</td>
<td>≤ 0.25</td>
<td>70 – 300</td>
<td>Avg. 20</td>
<td>SM*</td>
<td>50 – 100</td>
</tr>
<tr>
<td>SR 417 International Pkwy</td>
<td>40 – 70</td>
<td>0.15</td>
<td>0.35 to 0.75</td>
<td>18 – 40</td>
<td>Avg. 10</td>
<td>SP-SM, SM</td>
<td>50 – 100</td>
</tr>
<tr>
<td></td>
<td>30 – 40</td>
<td>0.25 to 0.4</td>
<td>≤ 0.32</td>
<td>50 – 87</td>
<td>Avg. 10</td>
<td>SP-SM, SM, SC</td>
<td>50 – 300</td>
</tr>
<tr>
<td></td>
<td>70 – 80</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SR50 / SR436</td>
<td>55 – 70</td>
<td>0.05</td>
<td>up to 3.5</td>
<td>Avg. 10</td>
<td>5 – 10</td>
<td>SP</td>
<td>50 – 200</td>
</tr>
<tr>
<td></td>
<td>40 – 55</td>
<td>0.5</td>
<td>≤ 0.27</td>
<td>25 – 45</td>
<td>Avg. 20</td>
<td>SP-SM, SM, SC</td>
<td>50 – 150</td>
</tr>
<tr>
<td></td>
<td>70 – 80</td>
<td>1.0</td>
<td>0.04-0.24</td>
<td>50 – 300</td>
<td>Avg. 15</td>
<td>SM, SM*, CH*</td>
<td>50 – 100</td>
</tr>
<tr>
<td>I-4 / SR 408 Ramp B</td>
<td>12 – 75</td>
<td>≤ 0.25</td>
<td>up to 1.5</td>
<td>15 – 45</td>
<td>5 – 10</td>
<td>SP, SP-SM</td>
<td>Avg. 100</td>
</tr>
<tr>
<td></td>
<td>75 – 80</td>
<td>0.3 to 0.4</td>
<td>0.2 to 0.35</td>
<td>50 – 88</td>
<td>10 – 20</td>
<td>SC</td>
<td>30 – 100</td>
</tr>
<tr>
<td>Anderson Street Overpass</td>
<td>10 – 90</td>
<td>≤ 0.25</td>
<td>up to 2.5</td>
<td>Avg. 15</td>
<td>3 – 10</td>
<td>SP, SP-SM, SP-SC</td>
<td>100 – 200</td>
</tr>
<tr>
<td></td>
<td>90 – 110</td>
<td>up to 1.0</td>
<td>0.05 to 0.15</td>
<td>50 – 365</td>
<td>10 – 20</td>
<td>SC*</td>
<td>T</td>
</tr>
<tr>
<td>I-4 Widening Daytona</td>
<td>45 – 55</td>
<td>≤ 0.25</td>
<td>0.25 to 0.35</td>
<td>Avg. 45</td>
<td>3 – 10</td>
<td>SP, SP-SM, SP-SC</td>
<td>100 – 200</td>
</tr>
<tr>
<td></td>
<td>55 – 73</td>
<td></td>
<td></td>
<td></td>
<td>10 – 25</td>
<td>SP-SM, SM</td>
<td>50 – 200</td>
</tr>
<tr>
<td></td>
<td>80 – 90</td>
<td>0.25 to 0.5</td>
<td>≤ 0.25</td>
<td>50 – 150</td>
<td>&gt; 50</td>
<td>SM*</td>
<td>T</td>
</tr>
<tr>
<td>SR 83 / Ramsey Branch Bridge</td>
<td>40 – 60</td>
<td>0.25 to 0.5</td>
<td>1.25</td>
<td>Avg. 15</td>
<td>Avg. 5</td>
<td>SC</td>
<td>Avg. 50</td>
</tr>
<tr>
<td></td>
<td>30 – 40</td>
<td>up to 0.5</td>
<td>0.05 to 0.25</td>
<td>50 – 150</td>
<td>Avg. 5</td>
<td>SC</td>
<td>50 – 150</td>
</tr>
<tr>
<td></td>
<td>60 – 90</td>
<td>up to 1.5</td>
<td>0.05 to 0.25</td>
<td>50 – 150</td>
<td>Avg. 5</td>
<td>SC*</td>
<td>50 – 150</td>
</tr>
</tbody>
</table>

*Cemented soil with trace phosphate and/or shell

T: Piezocone penetration test Terminated
Chapter 4

Analysis of Field Testing Data

4.1 Introduction

The analysis of High Pile Rebound (HPR) focused on developing relationships with soil properties at each of the previously discussed sites. This chapter presents the analysis of the piezocone penetration test data to characterize the soil behavior type (SBT) of soils at all seven sites. The CPT data was used to develop a soil stratigraphy for each site. Six geotechnical soil properties including soil unit weight, permeability, relative density, state parameter \((e_{\text{initial}} - e_{\text{critical}})\), over-consolidation ratio, and undrained shear strength were estimated using existing CPT correlations presented in Chapter 2. Each soil property was presented versus depth to develop a complete profile for each site and to identify the differences in soil properties in the rebound and non-rebound zones.

4.2 Soil Stratigraphy Using CPT Data

The normalized soil behavior type index (\(I_c\)) by Robertson (1990) presented in Chapter 2 as Table 2.2 was used to develop a vertical soil profile for each of the seven sites based on the calculated \(I_c\) using the normalized CPT data. Geotechnical software, CPeT-IT v.1.6 (2014), was used to process the CPT data. The software was
developed by Gregg Drilling and Testing Inc. in collaboration with Professor Robertson, co-author of a comprehensive text book on the CPT (Robertson and Cabal 2015). Using current published correlations with CPT data, CPeT-IT software uses the CPT results for each site to output basic normalized soil behavior type and geotechnical soil parameters.

In order to use the SBTn chart, the total and effective overburden stresses were calculated. The soil unit weight was first estimated for all the investigated sites based on the non-normalized CPT data using Eq. 2.5 (Robertson and Cabal 2010). The estimated distribution of soil unit weight with depth for each site is presented in Figures 4.1 – 4.7. Using the PDA data provided in Chapter 3, the rebound zones were shaded in the unit weight-depth distribution plots to better aid in the analysis. In general, a saturated soil unit weight range of 80 to 130 lb/ft$^3$ can be observed in all the presented unit weight profiles. A range of 115 to 130 lb/ft$^3$ with an average soil density of 125 lb/ft$^3$ can be identified at the rebound depths in all sites as presented in Figures 4.1 – 4.7. In order to verify the CPT-based soil unit weight at the rebound zones (i.e. 125 lb/ft$^3$), the results were compared to the estimated soil unit weight using SPT data. Based on the SPT data presented earlier in chapter 3, it was concluded that the number of blows, N at the rebound zones ranged from 15 to 35. For those N values, the saturated soil unit weight ranges from 110 lb/ft$^3$ to 130 lb/ft$^3$ (average = 120 lb/ft$^3$) (Bowles 1988). However, the saturated unit weight of soils at the rebound zones is located near the upper limit of the theoretical soil unit weight range of 90-130 lb/ft$^3$ (Coduto 2001).
Figure 4.1 Saturated unit weight versus depth for I-4 / US 192 at (a) Pier 6 / pile 16, (b) Pier 7 / pile 10, and (c) Pier 8 / pile 4 respectively with HPR zones shaded.
Figure 4.2 Saturated unit weight versus depth for SR 417 International Parkway at
(a) B1 / pile 14 (b) B2 / pile 5

Figure 4.3 Saturated unit weight versus depth for SR 50/SR436 at the
WB / pile 5 with HPR zones shaded

Figure 4.4 Saturated unit weight versus depth for I-4 / SR408 at pier 2 / pile 5
Figure 4.5 Saturated unit weight versus depth for Anderson Street at pier 6 / pile 5 and 6

Figure 4.6 Saturated unit weight versus depth for I-4 Widening Daytona at EB3 / pile 5 with HPR zones shaded

Figure 4.7 Saturated unit weight versus depth for SR 83 / Ramsey Branch Bridge at EB5 / pile 2 with HPR zones shaded
Using the CPT-based unit weight, profiles of the total and effective overburden stresses were calculated in order to normalize the CPT data. These profiles are presented in Appendix A. Based on the classification chart presented in chapter 2, profiles of normalized soil behavior type index ($I_c$), normalized soil behavior type (SBTn) zone, and typical geotechnical sections were developed for the seven sites as presented in Figures 4.8 to 4.17. The SBTn zones are color coded to aid in visual representation (CPeT-IT). The normalized soil behavior type index ($I_c$) profiles represent guides to the continuous variation of soil behavior type at all sites. The $I_c$ profiles illustrated in Figures 4.8 to 4.17 show that all rebound zones have an associated $I_c$ range from (2.4 to 3.0). Using Robertson (1990), this range of $I_c$ classifies the soils at the rebound zones as sand-mixture to silt/clay-mixture. The estimated typical geotechnical sections for all sites presented in Figures 4.8 to 4.17 show that the soils at the rebound zones classify as silty clay, clayey silt, sandy silt, and silty.

Using the USCS-based soil classification presented in Chapter 3, the soils at these zones were classified as silty sand (SM) or clayey sand (SC). This difference in soil classification is likely to occur in the mixed soils region (i.e. sand-mixture and silt-mixture) (Robertson and Cabal 2015). “However, the differences arose between USCS-based and CPT-based soil types can be related to soil classification criteria for each method. The CPT-based soil classification depends on the cone response to the in-situ mechanical behavior of the soil, while the USCS-based classification
depends on the grain size distribution and soil plasticity. Grain size distribution and soil plasticity relate reasonably well to the in-situ soil behavior” (Molle 2005).

“A soil with less than 50% fines may be classified as SM or SC using the USCS depending on its plasticity. Soil with low plasticity classifies as a silt whereas a soil with high plasticity classifies as a clay. When classifying with the CPT data, if the fines have high clay content with high plasticity, the soil behavior may be more controlled by the clay and the CPT-based classification will predict a more clay-like behavior, such as clayey silt or silty clay. If the fines were non-plastic, soil behavior will be controlled by the sand content and the CPT-based classification would predict a more sand-like soil type, such as silty sand or sandy silt” (Robertson and Cabal 2010). Based on the SBTn chart developed by Robertson (1990), it can be concluded that the mixed soils in the rebound zones contain a large percent of high plasticity fines and produce a clay-like behavior. These rebound soils are classified as clay to silty clay.
Figure 4.8 Normalized soil behavior type index and typical geotechnical section for I-4 / US 192 at Pier 6 / pile 16 with HPR zone shaded (G.S.E = 109.6 ft)
Figure 4.9 Normalized soil behavior type index and typical geotechnical section for I-4 / US 192 at Pier 7 / pile 10 with HPR zone shaded (G.S.E = 108.6 ft)
Figure 4.10 Normalized soil behavior type index and typical geotechnical section for I-4 / US 192 at Pier 8 / pile 4 with HPR zone shaded (G.S.E = 90.2 ft)
Figure 4.11 Normalized soil behavior type index and typical geotechnical section for SR 417 International Parkway at B1 / pile 14 (G.S.E = 72.3 ft)
Figure 4.12 Normalized soil behavior type index and typical geotechnical section for SR 417 International Parkway at B2 / pile 5 (G.S.E = 72.3 ft)
Figure 4.13 Normalized soil behavior type index and typical geotechnical section for SR 50 / SR436 at west bound / pile 5 with HPR zone shaded (G.S.E = 99.0 ft)
Figure 4.14 Normalized soil behavior type index and typical geotechnical section for I-4 / SR408 at pier 2 / pile 5
(G.S.E = 106 ft)
Figure 4.15 Normalized soil behavior type index and typical geotechnical section for Anderson street overpass at pier 6 / pile 5, 6 with HPR zone shaded (G.S.E = 104 ft)
Figure 4.16 Normalized soil behavior type index and typical geotechnical section for I-4 Widening Daytona at EB3 / pile 5
(G.S.E = 42.0 ft)
Figure 4.17 Normalized soil behavior type index and typical geotechnical section for SR 83 over Ramsey Branch Bridge at EB 5 / pile 2 with HPR zone shaded (G.S.E = 1.0 ft)
4.3 Estimation of Soil Properties using CPT Data

In order for pile construction sites to be characterized, soil properties along vertical profiles usually need to be identified. This identification requires expensive and time consuming laboratory testing conducted on samples extracted at short depth intervals. Laboratory testing has the drawback of not being representative of the whole volume of soil tested (Schneider et al. 2008).

It is helpful to estimate soil properties using simple, reliable, and inexpensive field tests (Robertson 2010). Soil characterization using in-situ testing has continued to increase, especially in materials that are difficult to sample and test using conventional methods (Mayne et al. 2009). The piezocone penetration test (CPT) has been widely accepted for in-situ soil characterization due to the ability for continuous data sampling and economic efficiency (Yi 2014). Several methods have been proposed to estimate soil properties using CPT data.

This section presents the results of the estimated soil properties at the seven sites used in this investigation. The soil properties identified are: permeability, relative density, state parameter, over-consolidation ratio, and undrained shear strength using the CPT data. A list of the equations used for the estimation of these soil properties using CPeT-IT software are tabulated in Appendix B. Complete profiles of the soil properties were developed by plotting each property versus depth. This data will be used for the comparison of the soil properties in the rebound and non-rebound zones. Using the PDA data provided in Chapter 3, the rebound zones were shaded in all soil property profiles to better aid in the analysis and discussion.
4.3.1 Soil Permeability

Soil permeability or hydraulic conductivity (k) is a key parameter when evaluating the generation and redistribution of excess pore pressures for sand-silt mixtures during undrained loading conditions (Belkhatir et al. 2013). Permeability varies by up to orders of magnitude and is difficult to either estimate or measure accurately. The majority of the methods used for measuring soil permeability are expensive, time consuming, and inaccurate due to sample disturbance and size effects. Therefore, estimating the permeability of a soil within one order of magnitude is often considered to be reasonable (Robertson 2010).

The coefficient of permeability in this investigation was estimated from CPT data using an approach developed by Robertson (2010). The proposed correlation relates soil permeability to soil behavior type index, \( I_c \) as given in Eq. (2.12a) and Eq. (2.12b) presented in chapter 2. Vertical profiles of the soil permeability in ft/s versus depth were developed for the seven sites using CPeT-IT as illustrated in Figures 4.18 to 4.24. The rebound zones are shaded where they exist. The permeability of soils in all rebound zones ranges from \( 1 \times 10^{-4} \) ft/s to \( 5 \times 10^{-9} \) ft/s whereas it ranges from \( 1 \times 10^{-3} \) ft/s to \( 1 \times 10^{-5} \) ft/s in the non-rebound zones. Details of the estimated permeability in the rebound and non-rebound zones at each site are presented in Table 4.1. In general the permeability in the rebound zones that are characterized by clay and silty clay soil is three orders of magnitude less than the permeability in the non-rebound zones characterized by silty sand to sandy silt. When
the rebound and the non-rebound zones are characterized by silty sand and sandy silt soils, the difference in the permeability is one order of magnitude.

For verification purposes, the estimated soil permeability within the rebound zones was compared to the typical permeability range for the soil types exist at these zones. Using the USCS, the soils at the rebound zones were classified as silty sand (SM) or clayey sand (SC). The estimated permeability for the CPT-based soil types at the same depths of both SM and SC soils agrees very well with the typical permeability of SM soil (1×10^{-8} ft/s to 5×10^{-5} ft/s) and SC soil (5×10^{-8} ft/s to 5×10^{-5} ft/s) (Geotechdata.info 2013).

The estimated permeability of the rebound zones is located within the semi permeable to impermeable soil range while the permeability of the non-rebound zones is located within the permeable range of soils (Holtz and Kovacs 1981). As a conclusion and according to the CPT classification, the soils at the rebound zones are described as impermeable fine to mixed grained soils (i.e. impermeable silty sand or silty clay). It is well documented that an undrained loading condition occurs in field situations when the loading rate is faster than the rate of water drainage. The undrained loading condition occurs in low permeability soils (i.e. water flow is restricted during loading). Pile driving is a quick loading; therefore, an undrained loading condition exists at the rebound zones for the investigated sites and the shearing represents undrained shear. As a result, soils in the rebound zones tend to respond to undrained shear and experience volume change that can be either dilative or contractive.
The soil parameters that identify if a soil will dilate or contract are: relative density ($D_r$) or the state parameter ($\psi$) for coarse grained soils (i.e. sand or sand mixtures) and the over consolation ratio (OCR) for fine grained soils (i.e. silt and clay) (Robertson and Cabal 2015). These parameters will be investigated and discussed within the rebound and non-rebound zones identified in this study.
4.18 Permeability versus depth for I-4 / US 192 at (a) pier 6 / pile 16, (b) pier 7 / pile 10, and (c) pier 8 / pile 4 with HPR zones shaded
4.19 Permeability versus depth for SR 417 International Parkway at (a) B1 / pile 14 and (b) B2 / pile 5
4.20 Permeability versus depth for SR 50 / SR 436 at west bound / pile 5 with HPR zone shaded

4.21 Permeability versus depth for I-4 / 408 at pier 2 / pile 5
4.22 Permeability versus depth for Anderson street overpass at pier 6 / pile 5 and 6

4.23 Permeability versus depth for I-4 Widening Daytona at EB3 / pile 5
4.24 Permeability versus depth for SR 83 over Ramsey Branch Bridge at EB5 / pile 2 with HPR zones shaded
4.3.2 Relative Density, State Parameter, and OCR

Relative density ($D_r$) is an important parameter in geotechnical engineering. This parameter provides an indication to whether sandy soils are loose or dense and provides a good indication of the mechanical properties of coarse grained soils (Juang 1996). Several empirical correlations between the relative density and cone penetration testing data are available in the literature. They usually relate the cone tip resistance ($q_c$) to the relative density with the consideration of effective overburden stress.

In this study, the relative density was estimated from the CPT data using formula developed by Kulhawy and Mayne (1990) as presented in the equations list in Appendix B. The suggested formula calculates the relative density by using the normalized cone tip resistance. Profiles of the relative density versus depth for all the seven sites were developed using CPeT-IT as illustrated in Figures 4.25 to 4.35. The CPeT-IT is programed to calculate the relative density where the soil is located in zones 5, 6, 7, and 8 in the soil behavior classification chart presented earlier in chapter 2 (i.e. coarse grained soil).

In general the coarse grained soils at the rebound zones are classified as medium dense to dense silty sand and sandy silt. In the non-rebound zones, they classified as loose to medium dense sand, silty sand, and sandy silt. However, the dense soils that exist in the non-rebound zones located at shallow depths were predrilled before the pile driving process; therefore no pile driving occurred above
these depths. Details of the estimated relative density in the rebound and non-rebound zones at each site are presented in Table 4.1

The response of sandy soil at a constant relative density will vary depending on the effective confining stresses (Bolton 1986). Therefore, another term, that is independent of the confining stress, was used to identify the sandy soil response. Research has shown that the state parameter ($\psi$) is a more meaningful parameter to represent the in-situ state of sandy soil than the relative density (Jefferies and Been 2006). The state parameter ($\psi$) is defined as the difference between the initial void ratio, $e$ and the void ratio at critical state, $e_{cs}$. The critical state of a soil is the state at which the shear stress remains constant while the shear strain increases (Holtz and Kovacs 1981). A simplified method has been developed by Robertson (2010) to estimate the state parameter ($\psi$) from the normalized cone tip resistance is presented in Chapter 2.

Profiles of state parameter ($\psi$) for all the seven sites were estimated using CPT results as shown in Figures 4.25 – 4.35. Since the state parameter ($\psi$) usually defines the response of sandy soil, CPeT-IT is programed to calculate this value wherever sandy soil exists. It can be noticed that there are some data missed at some depths because the soil at these depths is fine grained soil. Figures 4.25 – 4.35 show that the silty sand and sandy silt in the rebound zones have negative state parameter less than -0.05 (i.e. $e_{cs} > e_o$) while they have positive state parameter (i.e. $e_{cs} < e_o$) in the non-rebound zones. Table 4.1 presents the estimated state parameter for each site.
As previously mentioned, some rebound zones are characterized by clayey silt soil (i.e. fine soil) according to the CPT-based classification. In order to predict the response of the clayey silt soil during the undrained loading condition (i.e. pile driving), the overconsolidation ratio (OCR) was determined (Robertson, 2012). The overconsolidation ratio (OCR) is defined as the ratio of the maximum past effective consolidation stress ($\sigma'_p$) to the present effective overburden stress ($\sigma'_{vo}$) (Holtz and Kovacs 1981).

The OCR was estimated based on the CPT results using the normalized sleeve friction and cone tip resistance with an equation developed by Robertson (2009). Profiles of OCR for all the seven sites are shown in Figure 4.25 to 4.35 and are summarized in Table 4.1. The OCR data is missed at some depths because the soils at these depths are coarse grained soils. In general, the silty clay and clay soils in the rebound zones are overconsolidated with OCR ranges from 5 to 10. The fine grained soils in the non-rebound zones are characterized by silty clay and clay with OCR ranges from 0.5 to 3.

Soils with high OCR can be classified as cemented soils according to the SBT chart developed by Robertson (1990). Cemented soils have additional bonding between particles and exhibit higher resistance against deformation and behave like overconsolidated soils (Nagaraj et al. 1994). By comparing with the soil profiles developed based on the SPT presented in Chapter 3, it can be noticed that all soils in the rebound zones with high OCR may be classified as cemented soils.
4.25 Estimated relative density, state parameter, and overconsolidation ratio versus depth for I-4 / US 192 at pier 6 / pile 16 with HPR zones shaded
4.26 Estimated relative density, state parameter, and overconsolidation ratio versus depth for I-4 / US 192 at pier 7 / pile 10 with HPR zones shaded
4.27 Estimated relative density, state parameter, and overconsolidation ratio versus depth for I-4 / US 192 at pier 8 / pile 4 with HPR zones shaded
4.28 Estimated relative density, state parameter, and overconsolidation ratio versus depth for SR 417 International Parkway at B1 / pile 14
4.29 Estimated relative density, state parameter, and overconsolidation ratio versus depth for SR 417 International Parkway at B2 / pile 5
4.30 Estimated relative density, state parameter, and overconsolidation ratio versus depth for SR 50 / SR 436 at west bound / pile 5 with HPR zones shaded
4.31 Estimated relative density, state parameter, and overconsolidation ratio versus depth for I-4 / SR 408 at pier 2 / pile 5
4.32 Estimated relative density, state parameter, and overconsolidation ratio versus depth for Anderson street overpass at pier 6 / pile 5, 6 with HPR zones shaded
4.33 Estimated relative density, state parameter, and overconsolidation ratio versus depth for I-4 Widening Daytona at EB3 / pile 5 with HPR zones shaded
4.34 Estimated relative density, state parameter, and overconsolidation ratio versus depth for SR 83 / Ramsey Branch Bridge at EB 5 / pile 2 with HPR zones shaded (CPT1)
4.35 Estimated relative density, state parameter, and overconsolidation ratio versus depth for SR 83 / Ramsey Branch Bridge at EB 5 / pile 2 with HPR zones shaded (CPT2)
<table>
<thead>
<tr>
<th>Site Name</th>
<th>Depth (ft)</th>
<th>Rebound (inch/blow)</th>
<th>No. of blows (blow/ft)</th>
<th>Soil Type</th>
<th>k (ft/s)</th>
<th>Dr (%)</th>
<th>ψ</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-4 / US-192 Interchange</td>
<td>40 – 65</td>
<td>≤ 0.25</td>
<td>ave. 50</td>
<td>Sand, silty sand, sandy silt, and clay</td>
<td>1×10⁻³ - 1×10⁻⁴</td>
<td>Loose</td>
<td>-0.05 to 0.1</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>65 – 100</td>
<td>0.25 to 0.9</td>
<td>70 – 300</td>
<td>Clay and silty clay</td>
<td>1×10⁻³</td>
<td>N/A</td>
<td>N/A</td>
<td>6 – 10</td>
</tr>
<tr>
<td>SR 417 International</td>
<td>40 – 70</td>
<td>0.15</td>
<td>18 – 40</td>
<td>Sand, silty sand, and sandy silt</td>
<td>1×10⁻⁹ - 1×10⁻⁷</td>
<td>Loose</td>
<td>-0.05 to 0.1</td>
<td>N/A</td>
</tr>
<tr>
<td>Parkway</td>
<td>30 – 40</td>
<td>0.25 to 0.4</td>
<td>50 – 87</td>
<td>Silty sand</td>
<td>N/A</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>70 – 80</td>
<td>0.25 to 0.4</td>
<td>50 – 300</td>
<td>Clay and silty clay</td>
<td>1×10⁻⁴ - 1×10⁻⁶</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>SR50 / SR436</td>
<td>55 – 70</td>
<td>0.05</td>
<td>ave. 10</td>
<td>Silty sand and sandy silt</td>
<td>1×10⁻³</td>
<td>Loose</td>
<td>-0.03 to 0</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>40 – 55</td>
<td>0.5</td>
<td>25 – 45</td>
<td>Silty sand and sandy silt</td>
<td>1×10⁻⁴ - 1×10⁻⁶</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>70 – 80</td>
<td>1.0</td>
<td>50 – 300</td>
<td>Clay and silty clay</td>
<td>1×10⁻⁴ - 1×10⁻⁶</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>I-4 / SR 408 Ramp B</td>
<td>12 – 55</td>
<td>≤ 0.25</td>
<td>15 – 45</td>
<td>Sand, silty sand, and sandy silt</td>
<td>1×10⁻⁴ - 1×10⁻⁷</td>
<td>Loose</td>
<td>-0.04 to 0.1</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>55 – 75</td>
<td>≤ 0.25</td>
<td>15 – 45</td>
<td>Clay and silty clay</td>
<td>1×10⁻⁴ - 1×10⁻⁷</td>
<td>Loose</td>
<td>-0.04 to 0.1</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>75 – 80</td>
<td>0.3 to 0.4</td>
<td>50 – 88</td>
<td>Silty sand and sandy silt</td>
<td>1×10⁻⁴ - 1×10⁻⁵</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Anderson Street Overpass</td>
<td>10 – 50</td>
<td>≤ 0.25</td>
<td>ave. 15</td>
<td>Sand, silty sand, and sandy silt</td>
<td>1×10⁻⁴ - 1×10⁻⁷</td>
<td>Loose</td>
<td>-0.05 to 0.15</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>50 – 90</td>
<td>≤ 0.25</td>
<td>ave. 15</td>
<td>Clay and silty clay</td>
<td>N/A</td>
<td>Loose</td>
<td>N/A</td>
<td>2 – 4</td>
</tr>
<tr>
<td></td>
<td>90 – 110</td>
<td>up to 1.0</td>
<td>50 – 365</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
</tr>
<tr>
<td>I-4 Widening Daytona</td>
<td>45 – 55</td>
<td>≤ 0.25</td>
<td>ave. 45</td>
<td>Sand, silty sand, and sandy silt</td>
<td>1×10⁻⁴ - 1×10⁻⁷</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>55 – 73</td>
<td>≤ 0.25</td>
<td>50 – 150</td>
<td>Clay and silty clay</td>
<td>1×10⁻⁴ - 1×10⁻⁷</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>80 – 90</td>
<td>0.25 to 0.5</td>
<td>50 – 150</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
</tr>
<tr>
<td>SR 83 / Ramsey Branch</td>
<td>40 – 60</td>
<td>0.25 to 0.5</td>
<td>ave. 15</td>
<td>Silty sand and sandy silt</td>
<td>1×10⁻⁵</td>
<td>Loose</td>
<td>-0.05 to 0.02 N/A</td>
<td></td>
</tr>
<tr>
<td>Bridge</td>
<td>30 – 40</td>
<td>0.25 to 0.5</td>
<td>50 – 150</td>
<td>Silty sand and sandy silt</td>
<td>1×10⁻⁴ - 1×10⁻⁶</td>
<td>Medium to dense</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>60 – 90</td>
<td>up to 1.5</td>
<td>50 – 150</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
</tr>
</tbody>
</table>

T: Piezocone penetration test Terminated  
N/A: Not Applicable due to soil type
4.4 Analysis of the Estimated Soil Properties

According to the CPT-based soil classification presented in Table 4.1, it can be concluded that both the rebound and non-rebound zones are characterized by silty sand, sandy silt, clay, and silty clay soils. Therefore, a unique soil type is not an indicator to predict the rebound problem. The estimated relative density / state parameter for the sand or sandy mixed soils and overconsolidation ratio for the fine grained soils must be discussed to interpret the soil behavior more clearly.

The behavior of the dense sand / over consolidated clay and loose sand / normally consolidated clay is presented in Figure 4.36 (Das 2008). Figure 4.36 illustrate volume change behavior of sand and clay soil under shear loading condition. The dense sands and over consolidated clays expand or dilate (increase in volume) while the loose sands and normally consolidated clays contract (decrease in volume) under shear loading condition. Robertson (2009) suggested that soils with OCR more than 4 (i.e. OCR > 4) are dilative at large strains.

The state parameter \( (e_o - e_{cs}) \) describes the behavior of dense and loose sand more precisely in terms of initial and critical void ratio (Bolton 1986). Figure 4.37 illustrates the change of the void ratio with strain for loose and dense sands (Das, 2008). It can be observed that the void ratio increases in dense (i.e. soil expands) sand while it decreases in the loose sand (i.e. soil contracts). Therefore, dense soils are characterized by negative state parameter whereas loose soils are characterized by positive state parameter. Jefferies and Been (2006) suggested that soils with state
parameter less than -0.05 \((e_o - e_{cs} < -0.05)\) are dilative at large strains (i.e. the soil expands and increases the final void ratio) otherwise they are contractive.

Figure 4.36 Volume change behavior of sand and clay soil under shear loading condition (Das 2008)

Figure 4.37 Change of the void ratio with strain for loose and dense sands (Das 2008)
Table 4.1 shows that some of the rebound zones are characterized by medium dense to dense silty sand to sandy silty soils with negative state parameter ranges from -0.05 to -0.2. However, at location I-4 Widening Daytona, medium to dense silty sand soil exist at the depth 45 ft – 55 ft with state parameter ranges from -0.1 to -0.13 with no rebound occurring. This location was predrilled to 40 ft before pile driving and therefore no pile rebound was recorded. The other rebound zones are characterized by over consolidated clay to silty clay soils with OCR ranging from 5 to 10. The soils at the rebound zones tend to dilate (increase in volume) during the undrained loading condition.

Some of the non-rebound zones presented in Table 4.1 are characterized by loose to medium dense soils with state parameter ranges from -0.05 to 0.15. Other non-rebound zones are characterized by normally consolidated to over consolidated silty clay and clay soils with OCR ranges from 2 to 3. Therefore, the soils at the non-rebound zones tend to contract during pile driving. As a conclusion, the rebound zones are characterized by dilative soils whereas the non-rebound zones are characterized by contractive soils.

Soil dilation or contraction affects the generation of excess pore water pressures and shear strength of granular and cohesive soils during pile driving (Das 2008). Therefore, the behavior of dilative and contractive soils in terms of pore water pressures generation and shear strength was analyzed and is presented in the next section.
4.5 Analysis of Pore Water Pressures Induced by Piezocone Penetration

The penetration of a piezocone into a saturated soil results in generation of pore water pressures in the vicinity of the cone sleeve and tip. The generated pore water pressures occur because of the changes in the compression stress ($\Delta \sigma$), due to the displacement of the soil particles, and in the shear stress ($\Delta \tau$), due to the shear deformation of the soil adjacent to the cone. For penetration in saturated low permeability soils, these changes occur under undrained condition (Burns and Mayne 1998). Three components of pore water pressure exist when a cone penetrates any saturated soil deposit: hydrostatic, stress-induced and shear stress-induced pore water pressure as presented in Eq. (4.1).

$$u_m = u_0 + \Delta u_{com} + \Delta u_{shear}$$ (4.1)

Where: $u_m$ = measured pore pressure due to cone penetration  
$u_0$ = hydrostatic pore water pressure  
$\Delta u_{com}$ = excess compression-induced pore water pressure  
$\Delta u_{shear}$ = excess shear-induced pore water pressure

These three components cannot be measured or distinguished separately during the CPT. However, many approaches have been developed to estimate the compression-induced and the shear-induced pore water pressures due to cone penetration. The hydrostatic pore pressure is very well documented as the water density time the depth ($\gamma_w \times h$) and are typically a positive pressure. The
compression-induced pore water pressure models were developed using the cavity expansion theory. The principal of the expansion theory is that “the pressure required to produce a deep hole in an elastic-plastic medium is proportional to the required pressure of expanding a cavity of the same volume under the same conditions” (Gui and Jeng 2009). Torstensson (1977) assumed that a plasticized spherical zone is generated around the cone tip due to the changes in the normal stresses during cone penetration. The size of the plasticized zone is affected by the radius of the cone and the rigidity index of the surrounding soil.

The shear-induced pore water pressure represents the other component of the induced pore water pressure due to cone penetration. The zone of influence of the shear stress is limited to a thin layer along the cone sleeve (approximately 1 to 10 mm thick). Figure 4.38 shows the zones affected by cone penetration (Burns and Mayne 1999).

Figure 4.38 Zones affected by cone penetration (Burns and Mayne 1999)
Burns and Mayne (1999) developed two models to estimate both excess pore water pressure components induced due to penetration of a piezocone into clay soils. The models for compression and shear induced pore water pressures are presented in Eq. (4.2) and Eq. (4.3).

\[
\Delta u_{\text{compression}} = \frac{4}{3} \left[ \sigma'_v \frac{M}{2} \left( \frac{\text{OCR}}{2} \right)^{0.8} \right] \ln I_r \quad (4.2)
\]

\[
\Delta u_{\text{shear}} = \sigma'_v \left[ 1 - \left( \frac{\text{OCR}}{2} \right)^{0.8} \right] \quad (4.3)
\]

Where:

\[\sigma'_v = \text{effective overburden stress}\]
\[M = \frac{6 \sin \phi'}{3 - \sin \phi'}\]
\[\phi' = \text{effective angle of friction}\]
\[\text{OCR} = \text{over-consolidation ratio}\]
\[I_r = \text{rigidity index} = \frac{G}{S_u}\]
\[G = \text{undrained shear modulus}\]
\[S_u = \text{undrained shear strength}\]

From Eq. (4.2), it can be observed that the compression-induced excess pore water pressure is always a positive value and increases as the effective stresses and OCR increase. As mentioned previously, the behavior of OCR soils is very similar to dense soils; therefore, the compression-induced pore water pressure increases with increasing the OCR of fines grained soils or relative density of coarse grained soils.
It is obvious from Eq. (4.3) that the shear-induced component can be a negative or positive value depending on soil dilation or contraction under shear loading. Dilation of soil voids causes negative water pressures that draw the water into the pores. Contractive soils have a tendency to compress when the shear stress is increased. As a result, the water in the soil pores will increase in pressure (positive pressure) and attempts to flow out from these pores. Shear-induced negative pore water pressures will generate in the dilative soils (i.e. fine grained soils with OCR more than 4 or coarse grained soils with medium to dense relative density). Shear-induced positive pore water pressures generate in contractive soils (i.e. fine grained soils with OCR less than 4 or coarse grained soils with loose relative density) (Burns and Mayne 1999).

Since all rebound and non-rebound zones are fully saturated (i.e. below the water table), the presence of the water will result in generation of shear-induced and compression-induced pore water pressures. The generated excess-shear induced pore pressures in the rebound zones have negative values because of existing dilative soils. Contractive soils existing in the non-rebound zones will generate shear-induced positive pore water pressures.

Data gathered during the piezocone penetration tests (CPT) for all the seven sites indicated that high total positive pore water pressures (up to 570 psi) have developed at the rebound zones during cone penetration as presented in chapter 3. However, the measured total pore water pressures at these zones show high positive values because the compression-induced positive pressures are much larger than the
shear-induced negative pressures. The affected compression zone is much larger than the zone affected by shear.

As a conclusion, the rebound zones at the seven sites were characterized by high relative density or high OCR than the non-rebound zones. The soils at these rebound zones will generate higher compression-induced pore pressures and shear-induced negative pore pressures during cone penetration. The high total positive pore water pressures (up to 570 psi) at the rebound zones measured during the CPT will not indicate if the soil will contract or dilate unless the OCR (for fine soils) and the relative density (for coarse soils) are estimated. The total induced pore water pressures during the CPT has been used to predict the possibility of HPR (Jarushi et al. 2013).

4.6 Evaluation and Improvement of Existing Correlation between HPR and CPT Pore Water Pressure

A recent statistical correlation developed by Jarushi et al. (2013) is shown in Figure 4.39. This correlation was based on 26 data points obtained by the analysis of PDA and CPT data collected from eight sites described in Cosentino et al. (2012). Test piles at these eight sites were 24” precast concrete piles driven with single acting diesel hammer. As Figure 4.39 shows, pile rebound, measured in inch/blow, was found to correlate linearly with pore water pressure ($u_2$) obtained from CPT test measured in tsf. The coefficient of determination (i.e., $R^2$) obtained for the relationship was 0.76. According to this correlation, high pile rebound (i.e., > 0.25
in) occurs only when pore water pressure obtained from the CPT sounding is near or exceeds 5 tsf (70 psi).

One of the main objectives of this research is to improve the confidence to use this correlation. As mentioned previously, extensive PDA and CPT tests were conducted at the studied sites. Field measured pore water pressures from the CPT and rebound from PDA data were employed for this purpose. Logically, if the measured pore water pressures are to be substituted into a good correlation, very close predicted rebound values to the measured rebound would be obtained. The statistical equation given in Figure 4.39 was used to solve for predicted rebound in inch/blow by substituting the measured pore water pressure in tsf. The measured rebound values corresponding to the used measured pore pressures were then used to create the scatter plot shown in Figure 4.40. The scatter plot is created by plotting 200 data sets of the measured pile rebound ranging from 0 to 1.3 inch/blow versus predicted pile rebound obtained from the correlation after substituting the measured pore pressures ($u_2$).

The 45° reference line assists in comparing the predicted and measured values. The closer the points to the reference line, the closer the predicted and measured values are to each other. Inspecting Figure 4.40, the majority of data points beyond a measured rebound of about 0.5 inch/blow are below the reference line. It is evident that the statistical correlation presented by Jarushi et al. (2013) under predicts the actual rebound beyond 0.5 inch/blow. This discrepancy is noticed to increase with increased actual rebound. For example, an error of about 40% in the
predicted rebound is obtained corresponding to an actual rebound of 1.3 inch/blow. The relatively low number of data sets used to develop this correlation may be the main reason causing this discrepancy and under prediction issue.

![Graph](image_url)

Figure 4.39 Pile rebound for 24 inch precast concrete piles versus CPT pore water pressures ($u_2$) (Jarushi et al. 2013)

![Graph](image_url)

Figure 4.40 Scatter plot of measured versus predicted rebound using the statistical correlation developed by Jarushi et al. (2013)
To improve the correlation developed by Jarushi et al. (2013), the number of data sets was increased to 204 data points. These data points were obtained by analyzing the results of additional PDA and CPT testing conducted at the studied sites. An improved correlation was obtained as shown in Figure 4.41. The best fit equation shown on the scatter plot in Figure 4.41 was obtained by using linear regression. Comparing Figure 4.41 to Figure 4.39, it can be noticed that although the relationship between the two parameters is still linear, the data points are less scattered around the best fit line than in Figure 4.39. This is verified by the higher coefficient of determination (i.e. $R^2$) of 0.91 as compared with the previous 0.76.

Figure 4.41 Statistical correlation of pile rebound for 24 inch precast concrete and CPT pore pressure measured ($u_2$)
4.7 Analysis of Soil Properties Effect on the Induced Pore Water Pressures during Pile Driving

The pile performance during loading is directly influenced by the behavior of the surrounding soil during driving (Michael et al. 1982). Two categories of stresses changes resulting from pile driving: stresses occurring along the pile shaft (shear stresses) and stresses occurring at the pile tip (compressive stresses). Shear stresses develop during pile driving along the interface of the pile and soil due to the relative movement between the pile and the surrounding soil. Since pile driving movement is downward, the direction of the shear stresses is upward (Fellenius 1984). The upward shear stresses represent soil resistance to pile driving.

Soils respond to the stresses changes; therefore, soil deformations occur adjacent to the pile shaft due to shear forces and at the pile tip due to compressive forces as shown in Figure 4.42. Since soils are porous medium, compressive and shear stresses during pile driving will force water out of the voids. Due to the fast loading speed and low permeability coefficient of the soils at the rebound zones, water cannot flow out of the voids and cannot dissipate instantly. As a result, high pore water pressures will generate along the pile shaft (i.e. shear induced) and at the pile tip (i.e. compression induced). The effect of these two pore water pressure components on pile driving is discussed below.
4.7.1 Shear-Induced Pore Water Pressure

The generated pore water pressure along the pile shaft or shear induced pore pressure can be negative or positive depending on soil densification. As previously discussed, the soils at the rebound zones were classified as dilative soils (i.e. tend to increase in volume). Negative pore water pressures will generate along the pile shaft due to volume increase of the dilative soils existing in the rebound zones. The generated pore water pressure affects the soil shear strength along the pile shaft.

An estimate of the shear-induce pore water pressures at the soil-pile interface as a result of pile driving was developed by Raymond Lundgren (1979). Eq. (4.4) and Eq. (4.5) calculate the shear-induced pore water pressures developed along pile shaft in overconsolidated and normally/lightly overconsolidated clays respectively.
\[ \Delta u_{\text{shear}} = (6.5 \mp 1)S_u \quad \text{for OC clays} \quad (4.4) \]

\[ \Delta u_{\text{shear}} = (1.5 - 3)S_u \quad \text{for NC clays} \quad (4.5) \]

Where: \( S_u \) is the undrained shear strength

The undrained shear strength \( (S_u) \) is a commonly used soil parameter in case of cohesive soil under undrained loading condition. The undrained shear strength \( (S_u) \) was estimated based on the CPT data using CPeT-IT and was used to compare the results in the rebound and non-rebound zones. Profiles of the undrained shear strength versus depth for all the seven sites are presented in Figures 4.43 to 4.59.

CPeT-IT is programed to estimate \( S_u \) at the zones when cohesive soil exists; therefore; missing data can be observed at some depths. The undrained shear strength \( (S_u) \) in the rebound zones tends to be significantly higher than shear strength at the non-rebound zones. It can be noticed that the undrained shear strength of the cohesive soil at the rebound zones ranges from 4 tsf to 10 tsf while it ranges from 1 sf to 2 tsf at the non-rebound zones. The increase in the undrained shear strength at the rebound zones is related to the cementation and overconsolidation of soils at these zones (Mayne et al. 2009).
Figure 4.43 Undrained shear strength for I-4 / US 192 at (a) pier 6 / pile 16, (b) pier 7 / pile 10, and (c) pier 8 / pile 4 with HPR zones shaded
Figure 4.44 Undrained shear strength for SR 417 International Parkway at (a) B1 / pile 14 and (b) B2 / pile 5
Figure 4.45 Undrained shear strength for SR 50/SR 436 at west bound / pile 5 with HPR zone shaded

Figure 4.46 Undrained shear strength for I-4 / SR 408 at pier 2 / pile 5

Figure 4.47 Undrained shear strength for Anderson street at pier 6 with HPR zone shaded
Figure 4.48 Undrained shear strength for I-4 Widening Daytona at EB3 / pile 5

Figure 4.49 (a) Undrained shear strength for SR 83/Ramsey Branch Bridge at EB 5 / pile 2 with HPR zones shaded (CPT1)

Figure 4.49 (b) Undrained shear strength for SR 83/Ramsey Branch Bridge at EB 5 / pile 2 with HPR zones shaded (CPT2)
4.7.2 Compression-Induced Pore Water Pressure

High excess pore water pressures may be generated during the driving of prestressed concrete piles due to the low permeability of soil and the rapid soil loading by the hammer impact (Guihai et al. 2011). Bingjian (2011) found that driving piles in saturated soils creates a spherical cavity expansion at the pile tip due to pore water pressure and deformation. The influence of that sphere may expand up to 5 to 6 times the pile diameter. This region surrounding the pile is named a plastic zone. Maximum compression-induced excess pore water pressures are generated at the pile face and decrease with distance from the pile within the plastic zone and extend to the outer zone. The outer zone represents the elastic zone because reversible deformations occur beyond the plastic radius. The plastic and elastic zones around a driven pile are shown in Figure 4.50 (Wren 2007).
The radius of the plastic zone ($R$) can be determined using Eq. (4.6) and the excess pore water pressure within the plastic zone can be determined using Eq. (4.7) (Guihai et. al. 2011).

\[ R_p = r_0 \left( \frac{G}{S_u} \right)^{1/2} \]  \hspace{2cm} (4.6)

\[ \Delta u = S_u \left[ 2 \ln \left( \frac{R_p}{r} \right) + 1.73A_f - 0.58 \right] \]  \hspace{2cm} (4.7)

Figure 4.50 Plastic and elastic zones around single driven pile (Wren 2007)
Where:

\[ R_P = \text{Radius of plastic zone} \]
\[ r_o = \text{radius of pile} \]
\[ G = \text{Shear modulus} \]
\[ S_u = \text{Undrained shear strength} \]
\[ r = \text{Distance from pile center} \]
\[ A_f = \text{Skempton’s pore water pressure coefficient} \]

The shear modulus and undrained shear strength were estimated using the CPT data. Skempton’s pore water pressure coefficient \((A_f)\) has a range of \((0.5 \text{ to } 1)\), \((0 \text{ to } 0.5)\), and \((-0.5 \text{ to } 0)\) for normally, lightly, and heavily consolidated clays respectively (Holtz and Kovacs 1981). Since the maximum pore water pressures occur at the pile face within the plastic zone; therefore, Eq. (4.7) was used to calculate the pore water pressure with \(r = r_o\).

These equations were developed for clays or cohesive soils that have undrained shear strength \((S_u)\). Therefore, only three locations at site I-4 / US 192 (pier 6, 7, and 8) were selected because the soil in the rebound zones was classified as clayey silty to silty clay according to the CPT-based classification as presented earlier in Figures 4.8, 4.9, and 4.10. The maximum pore water pressures developed due to driving 24-inch prestressed concrete piles were estimated using Eq. (4.7). The calculated maximum pore water pressures due to pile driving versus depth for the three piers are shown in Figure 4.51. The calculated pore water pressures up to 870 psi developed at the rebound zones for piles driven at one site. The equivalent radius
of 24-inch square pile is 1.13 ft, therefore the radius of the plastic zone at the rebound zones is approximately 23 ft according to Eq. (4.6). The calculated maximum pore water pressure due to pile driving (780 psi) is approximately equal to 1.6 times the measured CPT pore water pressure at the same depth.

These pressures represent upward extra resistance forces to pile driving and resist the downward pile movement. As the compression-induced pore water pressures increase, the resistance forces to pile driving will increase. Therefore, the higher the compression-induced pore water pressure, the higher number of hammer blows are required to reach soil failure. Since the water is an incompressible fluid, pile rebound was observed or recorded. The generation and dissipation of the compression-induced pore water pressures are significantly affected by soil permeability. Since all the rebound zones have semi-impermeable to impermeable soils, the compression-induced pressures will take a longer time to dissipate.
Figure 4.51 Calculated maximum compression-induced pore water pressure due to driving 24-inch prestressed concrete piles using Eq. (4.10) with HPR zones shaded for I-4 / US 192 at (a) pier 6 / pile 16, (b) pier 7 / pile 10, and (c) pier 8 / pile 4.
4.8 Estimation and Analysis of Fines Content

The fines content is described as the soil passing the #200 sieve expressed in percentage. The soils at all investigated sites was determined using the soil behavior type index, $I_c$. Initially, $I_c$ was calculated from the collected CPT data using Eq. (4.4) (Robertson 1990). The soil fines content was estimated using Eq. (2.6a) to (2.6d) (Yi, 2014), from the calculated $I_c$.

Two steps were followed in this section. Firstly, estimating the soil fines content based on the suggested Eq. (2.6a) to (2.6d) was validated. The second step includes presenting complete profiles of the measured and estimated fines content results versus depth.

4.8.1 Validation of Yi (2014) Equation

Laboratory measured fines content of field data was available and used for validating the procedure of estimating fines content. Some measured fines content data was obtained from undisturbed soil samples extracted during SPT test at different depths for all studied sites. Laboratory sieve analysis was conducted on those samples and the resulting grain size distribution curves were then used to calculate the fines content. Based on 80 data points collected from all sites, Figure 4.52 shows a scatter plot of the predicted fines content based on $I_c$ versus the measured fines content. Each data point of predicted fines content used in this figure was obtained by taking the average of six CPT readings located within one foot
depth. This ensures that the data points are compared at the same depth. The 45° reference line helps indicating how the predicted and measured data match with each other. The closer the data points to the 45° reference line, the better they match with each other and consequently, the stronger validation is obtained. There is an excellent agreement between the measured and predicted fines content with very few data points as outliers.

Figure 4.52 Verification of fines content estimation procedure based on 80 data points from all sites

4.8.2 Profiles of Measured and Predicted Fines Content

After validating the procedure of using Eq. (2.6) to estimate the fines content, profiles of both predicted and measured fines content versus depth were developed for each site as shown in Figures 4.53 to 4.59. Using the PDA data provided in
chapter 3, the rebound zones were shaded in the fines content-depth distribution plots to better aid in the analysis. Rebound zones in Figures 4.57 and 4.58 for Anderson street and I-4 Widening Daytona, respectively were not shaded since CPT testing in these two locations was terminated at depths above the observed rebound depth.

Figures 4.53 to 4.59 showed that HPR zones had a fines content range of 28% - 38% in all sites. Fines content has been used to evaluate soil liquefaction or strength loss. Sand-like soils with fines content less than 20% and more than 35% are susceptible to strength loss, while clay-like soils with fines content more than 20% are not susceptible to strength loss (Robertson and Cabal 2009). The soils at the rebound zones were classified as clay-like soils based on CPT data. Therefore, there is no possibility for the soils at the rebound zones to liquefy or lose strength during pile driving.

Figures 4.54 to 4.56 showed non rebound zones corresponding to such a fines content range. This clearly indicates that a fines content of this range is not the only factor that may cause the HPR problem and there should be other soil parameters like soil type that interact and work all together in producing excessive HPR. Figures 4.53 to 4.59 represent the response of different soil types. Therefore, soil type and classification were investigated by separating the fines content for each soil type existing at all sites in this investigation. Analyzing of fines content versus pile rebound separately for each soil type would improve the ability to predict pile rebound.
Figure 4.53 Predicted and measured fines content versus depth for I-4 / US 192 at (a) Pier 6 / pile 16 (b) Pier 7 / pile 10 (c) Pier 8 / pile 4 with HPR zones shaded
Figure 4.54 Predicted and measured fines content versus for depth SR 417 International Parkway at (a) B1 / pile 14 (b) B2 / pile 5
Figure 4.55 Predicted and measured fines content versus depth for SR 50 / SR 436 at west bound / pile 5 with HPR shaded

Figure 4.56 Predicted and measured fines content versus depth for I-4 / SR 408 at pier 2 / pile 5
Figure 4.57 Predicted and measured fines content versus depth for Anderson street overpass at pier 6 / pile 5, 6 with HPR zone shaded

Figure 4.58 Predicted and measured fines content versus depth for I-4 Widening Daytona at EB 3 / pile 5 with HPR zone shaded
Figure 4.59 Predicted and measured fines content versus depth for SR 83 over Ramsey Branch Bridge at EB 5 / pile 2 with HPR zone shaded
4.8.3 Effect of Fines Content on Pile Rebound

In order to assess the effect of fines content on pile rebound, estimated fines content from CPT data was used. The fines content data for different soil types was separated to combine the effect of soil type and fines content on pile rebound. Pile rebound data obtained from the PDA test was plotted versus fines content for seven soil types according to USCS as shown in Figures 4.60. From Figure 4.60a, it can be noticed that the clayey sand soil (SC) with trace shell produces higher pile rebound up to 1.4 inch/blow when the fines content exceeds 20%. Many data points of SC soil have low pile rebound corresponding to the same percent of fines. These data points could be representing normally consolidated soil zones. The effect of cementation on the silty sand soil in terms of pile rebound behavior can also be noticed in Figures 4.60b and 4.60c. The piles driven in cemented silty sand soil, Figure 4.60b, experienced high rebound levels while the piles driven in the same soil type did not cause significant rebound when it was not cemented as shown in Figure 4.60c. Figures 4.60d and 4.60e show low pile rebound (less than 0.3 inch/blow) in all soil types (SP, SP-SM, SP-SC, and SM-SC) over a wide range of fines content. This implies that the fines content does not play any role in the soils that are not expected to cause high rebound. Table 4.2 summarizes pile rebound susceptibility as a function of fines content for seven soil types. According to Table 4.2, rebound susceptibility increases with the presence of cemented silty fine sand (SM) and clayey fine sand (SC) soils with fines content range of 20% to 40%.
Figure 4.60 Pile rebound versus fines content for different soil types
Table 4.2 Rebound Susceptibility as a Function of Fines Content for Different Soil Types

<table>
<thead>
<tr>
<th>Rebound group</th>
<th>Degree of rebound</th>
<th>Soil Type</th>
<th>Percentage Finer than 0.075 mm</th>
<th>Typical Soil Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>Negligible to Low</td>
<td>Fine Sand with Silt</td>
<td>More than 40</td>
<td>SP, SP-SM, SM, SM-SC</td>
</tr>
<tr>
<td>R2</td>
<td>Low to Medium</td>
<td>Fine Sand with Silt</td>
<td>5 to 20</td>
<td>SP, SP-SM, SP-SC, SM, SM-SC</td>
</tr>
<tr>
<td>R3</td>
<td>High to Very High</td>
<td>Silty Fine Sand</td>
<td>20 to 40</td>
<td>SC with trace shell, cemented SM with trace phosphate and shell</td>
</tr>
</tbody>
</table>

4.8.4 Effect of Fines Content on CPT Pore Water Pressure

The relationship of fines content with pore water pressure was also studied. Pore water pressure ($u_2$) recorded during CPT soundings was plotted versus fines content. In order to emphasize the soil type effect on this relationship, data points of each single soil type were plotted separately as shown in Figure 4.61a to 4.61e. From Figure 4.61a and Figure 4.61b, it is obvious that the highest pore water pressure of up to 550 to 570 psi was generated during the CPT soundings in SC with trace shell and cemented SM with trace shell and phosphate corresponding to fines content of 28% to 38%.

Figure 4.61c shows high pore pressure of up to 418 psi followed by the silty sand with pore water pressure of up to 370 psi at fines content rage of 30% to 38%. Low CPT pore water pressures, less than 100 psi, can be observed for other soil types (SP, SP-SM, SP-SC, and SM-SC) corresponding to all fines content. Slight increase
in $u_2$ occurs at fines content more than 30%. A summary of CPT pore water pressure ($u_2$) corresponding to fines content for different soil types is presented in Table 4.3. Inspecting Table 4.3, the CPT pore water pressure is noticed to increase with the presence of cemented silty fine sand (SM) and clayey fine sand (SC) soils characterized by a fines content ranging from 23% to 40%.
Figure 4.61 Pore water pressure ($u_2$) versus fines content for different soil types
Table 4.3 Measured CPT Pore Water Pressure ($u_2$) Ranges Corresponding to the Fines Content for Different Soil Types

<table>
<thead>
<tr>
<th>Pore Pressure, $u_2$ (psi)</th>
<th>Percentage Finer than 0.075 mm</th>
<th>Typical Soil Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 60</td>
<td>3 to 75</td>
<td>SP, SP-SC, SP-SM</td>
</tr>
<tr>
<td></td>
<td>10 to 20</td>
<td>SM, SM-SC</td>
</tr>
<tr>
<td>60 to 140</td>
<td>20 to 45</td>
<td>SM, SM-SC, SP-SM</td>
</tr>
<tr>
<td>0 to 350</td>
<td>30 to 40</td>
<td>SM</td>
</tr>
<tr>
<td>350 to 570</td>
<td>23 to 40</td>
<td>SC with trace shell, cemented SM with trace phosphate and shell</td>
</tr>
</tbody>
</table>
Chapter 5

Analysis of Soil Behavior Type

5.1 Basic Approach

The CPT data for all the seven sites was analyzed to study the effect of soil behavior on rebound of piles. This was accomplished by separating and normalizing the data from each site by soil type. These normalized parameters were used in conjunction with the soil behavior type (SBT) classification charts. CPeT-IT software charts developed by Robertson (1990), Robertson (2012), Schneider (2008), and Eslami and Fellenius (1997) were used to superimpose the CPT data for the rebound and non-rebound zones.

5.2 Soil Behavior Type Classification (Robertson, 1990)

5.2.1 Normalized Cone Resistance versus Normalized Friction Ratio

Two normalized soil behavior charts suggested by Robertson (1990) were used to classify soils in the rebound and non-rebound zones at all sites. In the first chart the normalized cone tip resistance ($Q_{nt}$) was plotted versus the normalized friction ratio ($F_I$) as illustrated in Figures 5.1 and 5.2 for the rebound and non-rebound zones respectively.
The majority of the normalized CPT data for rebound soils presented in Figure 5.1 are more closely clustered in SBTn Zones 3 and 4 with some located in Zone 5 on the normalized cone resistance ($Q_{tn}$) versus normalized friction ratio ($F_r$) SBT chart. Figure 5.1 shows that approximately 90% of the data points are located in the region of increasing OCR or cementation (i.e. the upper right region). As a result, Figure 5.1 indicates that the soils in the rebound zones are classified as over consolidated fine grained soil (clayey silt to silty clay) or cemented coarse grained soil (silty sand to sandy silt). The normalized friction ratio ($F_r$) for all points ranges from 0.4 to 10 (i.e. the right half of the chart) while the normalized cone resistance ($Q_{tn}$) ranges from 3 to 40.

The majority of the data points for the non-rebound zones are located in Zones 4, 5, and 6 as shown in Figure 5.2. Some of these points are located on Zones 1 and 3. Approximately 95% of the data in Zones 4 and 5 is located on and below the normally consolidated region (i.e. decreasing of OCR age or cementation). The soil in the non-rebound zones classifies as clean sand, uncemented silty sand, or under consolidated / normally consolidated clayey silt. The majority of the data points in Figure 5.2 are located in the left half with a normalized friction ratio ($F_r$) ranges from 0.1 to 4, while the normalized cone resistance ($Q_{tn}$) ranges from 2 up to 400.
Figure 5.1 Rebound data overlaid on Robertson (1990) normalized cone resistance versus normalized friction ratio SBT chart. Normally consolidated zone is also shown.

Figure 5.2 Non-rebound data overlaid on Robertson (1990) normalized cone resistance versus normalized friction ratio SBT chart. Normally consolidated zone is also shown.
5.2.2 Cone Resistance versus Pore Pressure Ratio

The rate of generation and dissipation of pore water pressure during a cone penetration can also be a guide to determine high pile rebound in a soil. Soil behavior type charts can be improved if pore pressure data is normalized and added to the chart (Robertson, 2009). Normalization of pore pressures first requires separation of pore pressures that are a function of soil response and those existing in the ground prior to the penetration. Measured penetration pore pressure during CPT testing ($u_2$) represents the sum of the in situ or hydrostatic pore pressure ($u_o$) and the excess pore pressure ($\Delta u_2$) (Schneider et al., 2008). A pore pressure normalization formula suggested by Wroth (1984) is presented in Eq. (5.1).

$$B_q = \frac{\Delta u}{q_t - \sigma_{vo}} = \frac{u_2 - u_o}{q_t - \sigma_{vo}}$$  \hspace{1cm} (5.1)

Where:

$B_q$ = Pore pressure ratio

$u_2$ = pore pressure measured at the cone shoulder

$u_o$ = in-situ pore pressure

$q_t$ = cone resistance corrected for pore water pressure at cone shoulder (Eq. 4.1)

$\sigma_{vo}$ = total overburden stress

A soil behavior classification chart was developed by Robertson (1990) to account for pore pressure in soil type. In this chart the pore pressure ratio ($B_q$) was plotted versus the corrected tip resistance ($q_t$). The data points from the rebound and non-rebound zones were superimposed as illustrated in Figures 5.3 and 5.4.
respectively. The data points in Figure 5.3 are mainly located in Zones 3, 4, and 5 in the direction of OCR or cementation increase. Therefore, the soil at the rebound zones can be classify as cemented silty clay to silty sand. This strongly supports the previous conclusion from Figure 5.1 with respect to soil type.

The majority of the data points in Figure 5.3 are located on the right side of the classification chart (i.e. positive CPT pore pressure ratio, \( B_q \)). The pore pressure ratio (\( B_q \)) for these point ranges from 0.2 to 0.9 corresponding to a corrected tip resistance range of 1 tsf to 5.5 tsf. Figure 5.4 presents \( B_q \) versus \( q_t \) chart for non-rebound zones. The data points are located longitudinally on the upper part of the chart with a range of cone resistance (\( q_t \)) from 1 tsf up to 70 tsf. The pore pressure ratio (\( B_q \)) is low and ranges from -0.05 to 0.22.

Based on Figures 5.1–5.4, the rebound and non-rebound soils may locate in Zones 3, 4 and 5 in the soil behavior type charts but the data at the rebound zones located above the normally consolidated region and high pressure ratio. The non-rebound zones data located on and below the normally consolidated region with low pore pressure ratio. These findings agree very well with the estimated soil properties discussed previously in chapter 4. A summary of Robertson (1990) charts in terms of rebound and non-rebound is presented in Table 5.1. The rebound soils data is located in zones 3, 4, and 5 with low cone tip resistance and high sleeve friction and pore water pressure while the non-rebound soils data is located in zones 1, 4, 5, 6, and 7 with higher tip resistance and lower sleeve friction and pore water pressure.
Figure 5.3 Rebound data overlaid on Robertson (1990) cone resistance versus pore pressure ratio SBT chart

Figure 5.4 Non-rebound data overlaid on Robertson (1990) cone resistance versus pore pressure ratio SBT chart
Table 5.1 Ranges from Robertson (1990) Soil Behavior Type Classification Charts for Rebound and Non-rebound Zones at all Sites

<table>
<thead>
<tr>
<th>Zone</th>
<th>SBTn Zone</th>
<th>Normalized cone resistance ($Q_{tn}$)</th>
<th>Normalized friction ratio ($F_r$ %)</th>
<th>Pore pressure ratio ($B_q$)</th>
<th>Corrected cone resistance, $q_t$ (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebound</td>
<td>3,4,5</td>
<td>3 – 40</td>
<td>0.4 – 10</td>
<td>0.2 – 0.9</td>
<td>1 – 5.5</td>
</tr>
<tr>
<td>Non-rebound</td>
<td>1,4,5,6,7</td>
<td>2 – 400</td>
<td>0.1 – 4</td>
<td>-0.05 – 2.2</td>
<td>1 – 70</td>
</tr>
</tbody>
</table>

### 5.3 Soil Behavior Type Classification (Robertson, 2012)

Robertson (2012) developed a new SBT chart which accounts for soil dilation and contraction during cone penetration. This chart allows engineers to classify the soil as either dilative or contractive, and is an updated version of the $Q_{tn} – F_r$ chart developed earlier by Robertson (1990). Two regions were identified on the Robertson (2012) SBT chart using the state parameter ($\psi$) and OCR. Coarse grained soils with a state parameter less than -0.05 and fine grained soils with an OCR greater than 4 are dilative at large strains (i.e. typical of pile driving and CPT testing). Robertson (2012) divided each region into three sub-regions based on drainage: undrained, transitional, and drained. The undrained region included fine grained soils while the drained region included coarse grained soils. The transitional region represents mixed soils (i.e. coarse and fine). Therefore, four major groups of soil behavior were identified on Robertson (2012) SBT chart: fine dilative (FD), coarse dilative (CD), fine contractive (FC), and coarse contractive (CC).
The normalized CPT data for the rebound and non-rebound zones at all the seven sites were superimposed into the updated SBT chart (Robertson, 2012) as shown in Figures 5.5 and 5.6. Figure 5.5 shows that the rebound soils are fine to mixed grained dilative soil. A large portion of the rebound CPT data is located within the FD zone (i.e. fine grained soils with high OCR). Some of the rebound CPT data plots in the transitional or mixed zone (i.e. medium to dense sandy mixed soil). The non-rebound soils are fine, transitional or mixed grained, and coarse contractive soils as shown in Figure 5.6. The majority of the non-rebound CPT data is located on the CC zone (i.e. loose sandy soils). The remaining portion of the non-rebound zones CPT data is located on the FC zone.

Based on the data presented in Figures 5.5 and 5.6, it can be concluded that the rebound soils plot in the FD (Fine Dilative) zone whereas the non-rebound soils plot in the FC (Fine Contractive) and CC (Coarse Contractive) zones. However, the transitional or mixed grained zone may include both the rebound and non-rebound soils depending on the state parameter or OCR of the mixed soil. In conclusion, the rebound soils are over consolidated fine grained or dense mixed grained while the non-rebound soils are normally consolidated fine grained or loose mixed/coarse grained.
Figure 5.5 Rebound data overlaid on the Robertson (2012) updated normalized cone resistance versus friction ratio SBT chart with soil behavior: Fine Dilative (FD), Fine Contractive (FC), Coarse Dilative (CD), and Coarse Contractive (CC)

Figure 5.6 Non-rebound data overlaid on the Robertson (2012) updated normalized cone resistance versus friction ratio SBT chart with soil behavior: Fine Dilative (FD), Fine Contractive (FC), Coarse Dilative (CD), and Coarse Contractive (CC)
5.4 Soil Classification using Schneider et al. (2008)

The soil classification chart developed by Schneider et al. (2008) was used to analyze the CPT data from all sites. This chart was proposed for classifying soil using normalized CPT data from the corrected tip resistance ($q_t$) and CPT pore pressure ($u_2$). The cone tip resistance was normalized by dividing by the vertical effective stress, $\sigma'_{vo}$, using Eq. (4.3) (Wroth 1984), while the pore pressure ($u_2$) was normalized using Eq. (5.2).

\[
\text{Normalized pore water pressure} = \frac{\Delta u_2}{\sigma'_{vo}} = \frac{u_2 - u_o}{\sigma'_{vo}} \tag{5.2}
\]

Where:

- $u_2$ = pore pressure measured at cone shoulder,
- $u_o$ = in-situ pore pressure,
- $\sigma'_{vo}$ = effective overburden stress.

The normalized CPT data for the rebound and non-rebound zones were plotted on a semi-log $Q$ versus $\Delta u_2/\sigma'_{vo}$ chart as illustrate in Figures 5.7 and 5.8 respectively. It is evident from Figure 5.7 that most rebound data lies within the region of silts and low rigidity index (Ir) clays (i.e. Zone 1a) and some lies in the clays zone (i.e. Zone 1b). The normalized pore water pressure ranges from 2 up to 10 for the data in Zone 1a and 1 to 5 for the data in Zone 1b.

The soil classification chart in Figure 5.8 indicates that most the non-rebound data falls in Zone 2, essentially drained sands, and a small number in Zone 3,
transitional and silt soils. The data in Figure 5.8 has very low normalized pore water pressures and very high cone tip resistance.

Figure 5.7 Rebound data overlaid on Schneider et al. (2008) SBT chart

Figure 5.8 Non-rebound data overlaid on Schneider et al. (2008) SBT chart
5.5 Soil Classification using Eslami and Fellenius (1997)

A soil profiling method was developed by Eslami and Fellenius (1997) to classify the soil using CPT data. This classification method depends on the CPT parameters measured directly during the test (i.e. $q_t$ or $q_c$, $f_s$, and $u_2$). Therefore, it can be developed directly during the CPT sounding because the normalization by division with effective overburden stress is not required.

The classification procedure is accomplished using a log-log plot of the cone sleeve friction ($f_s$) versus the effective cone resistance ($q_E$), determined by Eq. (5.3).

$$q_E = q_t - u_2$$  \hspace{1cm} (5.3)

Where:

- $q_E$ = effective cone resistance,
- $q_t$ = cone resistance corrected for pore water pressure on cone shoulder (Eq. 4.1),
- $u_2$ = pore water pressure measured at cone shoulder

Rebound and non-rebound CPT data from all sites was superimposed on Eslami and Fellenius (1997) SBT chart as shown in Figures 5.9 and 5.10. Figure 5.9 indicates that the soil at the rebound zones can be classified as silty sand to silty clay. The data points have cone sleeve friction ranges ($f_s$) from 50 kPa to 700 kPa and cone effective resistance ($q_E$) ranges from 1 MPa to 10 MPa.
Figure 5.10 represents the classification of the soils in the non-rebound zones. There is significantly more scatter in this data. The soil can be classified as sand to sandy silt with a low fraction in the sensitive – collapsible clay and/or silt zone. The majority of the data has a sleeve friction ranging from 2.0 kPa to 80 kPa and effective cone resistance from 0.4 MPa to 40 MPa. It can be concluded that the rebound data tends to have high sleeve friction and low cone tip resistance while the data at the non-rebound zones have low sleeve friction and from low to high cone tip resistance. Some CPT data points for the rebound and non-rebound soils are overlapped in the silty sand zone. The behavior of these points as rebound or non-rebound soils can be related to the fines content discussed earlier.
Figure 5.9 Rebound data overlaid on Eslami and Fellenius (1997) SBT chart

Figure 5.10 Non-rebound data overlaid on Eslami and Fellenius (1997) SBT chart
5.6 Summary of Soil Behavior Type Classification

The soil behavior type (SBT) charts identify the rebound and non-rebound soils. Based on all SBT charts discussed earlier, the rebound soils are silty clay to clayey silt or silty sand to sandy silt. These soil types also exist in the non-rebound zones in addition to the sand soil. Although same soils exist in both rebound and non-rebound zones, there are other soil properties that identify the rebound and non-rebound soils in addition to the soil type. The rebound soils have higher CPT pore water pressure ($u_2$), higher sleeve friction, lower tip resistance, and located in the zone of increasing OCR / cementation. In contrast, the non-rebound soils have low CPT pore water pressure ($u_2$), low sleeve friction, high tip resistance, and located in and/or below the normally consolidated zone. The rebound soils are fine undrained dilative (FD) soils while the non-rebound soils are coarse and fine drained contractive (FC and CC) soils.
Chapter 6

Conclusions and Recommendations

6.1 Conclusions

- Rebound soils are characterized by cemented silty fine sand (SM) with trace phosphate and shell or cemented clayey fine sand (SC) with fines content ranging from 25 % to 40 %.
- The cemented soils in the rebound zones were identified by high overconsolidation ratio (OCR) and behave similar to overconsolidated soils.
- The estimated permeability of the rebound zones is within the semi permeable to impermeable soil range while the permeability of the non-rebound zones is within the permeable to semi permeable range.
- Rebound soils are dilative while non-rebound soils are contractive. Dilative soils have higher maximum shear strength compared to contractive soils.
- The measured CPT pore water pressures ($u_2$) are linearly correlated to the pile rebound with an excellent $R^2$.
- The soil behavior type (SBT) charts with CPT pore water pressure (Robertson (1990) and Schneider et al. (2008)) were found to give very clear indication of type and behavior of rebound and non-rebound soils.
6.2 Recommendations

In order to predict high pile rebound during the design phase and avoid pile redesign, it is recommended that the geotechnical engineer use the following methodology:

1- Conduct sufficient SPT and CPT depending on the project size.

2- High pile rebound can be initially predicted from the SPT test. Disturbed soil samples can be extracted to identify the grain size distribution and fines content. A general soil profile can be developed according to the USCS. The expected rebound zone is typically: the zone with cemented silty fine sand (SM) soil or clayey fine sand (SC) soil with fines content range from 25% to 40%.

3- The HPR zones can be directly identified from the pore water pressure ($u_2$) measured from the CPT. Pile rebound more than 0.25 inch/blow occurs when the CPT pore water pressure exceeds 150 psi.

4- General soil profile and soil properties can be estimated from the CPT data ($q_c, f_s,$ and $u_2$) using CPeT-IT software. The rebound zones are classified as dense silty sand to sandy silt with a state parameter less than -0.1 or silty clay to clay with an overconsolidation ratio (OCR) greater than 4.

5- The CPT data can be superimposed on existing soil behavior type (SBT) charts to identify the pile rebound zones. Robertson (1990) and Schneider et al. (2008) charts with CPT pore water pressure are recommended.
References


FDOT. Standard Specification for Road and Bridges Section 455. (2010).


A.1 Total stress, hydrostatic pore water pressure, and effective stress versus depth for I-4 / US 192 at (a) pier 6 / pile 16 and (b) pier 7 / pile 10
A.2 Total stress, hydrostatic pore water pressure, and effective stress versus depth for SR 417 International Parkway at (a) B1 / pile 14 and (b) B2 / pile 5
A.3 Total stress, hydrostatic pore water pressure, and effective stress versus depth for SR 50 / SR 436 at west bound / pile 5
A.4 Total stress, hydrostatic pore water pressure, and effective stress versus depth for I-4 / SR 408 at pier 2 / pile 5
A.5 Total stress, hydrostatic pore water pressure, and effective stress versus depth for Anderson Street at pier 6 / pile 5, 6
A.6 Total stress, hydrostatic pore water pressure, and effective stress versus depth for I-4 Widening Daytona at EB3 / pile 5
A.7 Total stress, hydrostatic pore water pressure, and effective stress versus depth for SR 83/Ramsey Branch Bridge at EB 5 / pile 2
Appendix B

- **Unit Weight, \( \gamma \) (kN/m\(^2\))**
  \[
  \gamma = \gamma_w \left( 0.27 \log(BF) + 0.38 \log \left( \frac{1}{P_2} \right) + 1.28 \right)
  \]
  where \( \gamma_w \) = water unit weight

- **Permeability, \( k \) (m/s)**
  \[
  k = \begin{cases} 
  10^{-3.27} & \text{for } I_1 < 3.27 \\
  10^{-4.00} & \text{for } I_1 = 3.27 \\
  10^{-4.54} & \text{for } I_1 > 4.00
  \end{cases}
  \]

- **N\(_{60}\) (blows per 30 cm)**
  \[
  N_{60} = \frac{Q_{60}}{P_{60}} \left( 1 + \frac{Q_{60}}{P_{60}} \right)
  \]
  \[
  N_{60,\max} = \frac{Q_{60}}{P_{60}} \left( 1 + \frac{Q_{60}}{P_{60}} \right)
  \]

- **Young's Modulus, \( E \) (MPa)**
  \[
  E = 10^{-0.15} \cdot 10^{6.81 \cdot \log(Q_{60})}
  \]
  (applicable only to \( I_1 < I_{1,\max} \))

- **Relative Density, \( Dr \) (%)**
  \[
  Dr = \frac{Q_{60}}{100} \left( \frac{Q_{60}}{P_{60}} \right)^{2.28}
  \]
  (applicable only to SBT: 5, 6, 7 and 8 or \( I_1 < I_{1,\max} \))

- **State Parameter, \( \psi \)**
  \[
  \psi = 0.56 - 0.35 \log(Q_{60,\max})
  \]

- **Peak defined friction angle, \( \varphi^p \) (°)**
  \[
  \varphi^p = 17.60 + 11 \log(Q_{60})
  \]
  (applicable only to SBT: 5, 6, 7 and 8)

- **1-D constrained modulus, \( M \) (MPa)**
  \[
  \begin{align*}
  &\text{if } I_1 > 2.20 \\
  &\sigma = 14 \text{ for } Q_{60} > 14 \\
  &\sigma = Q_{60} \text{ for } Q_{60} \leq 14 \\
  &M = \sigma \left( (\gamma - \sigma) \right) \\
  \end{align*}
  \]
  \[
  \begin{align*}
  &\text{if } I_1 \leq 2.20 \\
  &M_{c,\max} = \sigma_1 \left( (\gamma - \sigma_1) \right) \cdot 0.0188 \cdot 10^{0.3151 \cdot \psi^{0.55}}
  \end{align*}
  \]

- **Small strain shear Modulus, \( G_0 \) (MPa)**
  \[
  G_0 = (Q_{60} - \sigma) \cdot 0.0188 \cdot 10^{0.3151 \cdot \psi^{0.55}}
  \]

- **Shear Wave Velocity, \( V_s \) (m/s)**
  \[
  V_s = \frac{Q_{60}}{L}
  \]

- **Undrained peak shear strength, \( S_u \) (kPa)**
  \[
  S_u = 10^{-0.50} \cdot 7 \log(F_c) \text{ or user defined}
  \]
  \[
  S_u = \frac{Q_{60} - \sigma}{N_{60}}
  \]
  (applicable only to SBT: 1, 2, 3, 4 and 9 or 11 < \( I_{1,\max} \))

- **Reinforced undrained shear strength, \( S_{u,r} \) (kPa)**
  \[
  S_{u,r} = f_s
  \]
  (applicable only to SBT: 1, 2, 3, 4 and 9 or 11 < \( I_{1,\max} \))

- **Overconsolidation Ratio, OCR**
  \[
  OCR = \frac{Q_{c,\max}^{20}}{0.1 - \log(Q_{c,\max})^{1.72}} \text{ or user defined}
  \]
  \[
  OCR = \kappa_{OCR} \cdot Q_{c,\max}
  \]
  (applicable only to SBT: 1, 2, 3, 4 and 9 or 11 < \( I_{1,\max} \))

- **In situ Stress Ratio, \( K_o \)**
  \[
  K_o = (1 - \sin \varphi^p) \cdot OCR^{1/3}
  \]
  (applicable only to SBT: 1, 2, 3, 4 and 9 or 11 < \( I_{1,\max} \))

- **Soil Sensitivity, \( S_s \)**
  \[
  S_s = \frac{N_{60}}{P_{60}}
  \]
  (applicable only to SBT: 1, 2, 3, 4 and 9 or 11 < \( I_{1,\max} \))

- **Effective Stress Friction Angle, \( \psi^e \) (°)**
  \[
  \psi^e = 29.5 \cdot B \cdot 10^{1.61} \cdot 0.236 - 0.336 \cdot B_2 \cdot Q_{60}
  \]
  (applicable for \( 0.10 < I_x \leq 1.00 \))

References