# CPT Pore Water Pressure Correlations with PDA to Identify Pile Drivability Problem

Fauzi Jarushi, Paul Cosentino, Edward Kalajian, Hadeel Dekhn

*Abstract*—At certain depths during large diameter displacement pile driving, rebound well over 0.25 inches was experienced, followed by a small permanent-set during each hammer blow. High pile rebound (HPR) soils may stop the pile driving and results in a limited pile capacity. In some cases, rebound leads to pile damage, delaying the construction project, and the requiring foundations redesign. HPR was evaluated at seven Florida sites, during driving of square precast, prestressed concrete piles driven into saturated, fine silty to clayey sands and sandy clays. Pile Driving Analyzer (PDA) deflection versus time data recorded during installation, was used to develop correlations between cone penetrometer (CPT) pore-water pressures, pile displacements and rebound. At five sites where piles experienced excessive HPR with minimal set, the pore pressure yielded very high positive values of greater than 20 tsf. However, at the site where the pile rebounded, followed by an acceptable permanent-set, the measured pore pressure ranged between 5 and 20 tsf. The pore pressure exhibited values of less than 5 tsf at the site where no rebound was noticed. In summary, direct correlations between CPTu pore pressure and rebound were produced, allowing identification of soils that produce HPR.

*Keywords*—CPTu, pore water pressure, pile rebound.

## I. INTRODUCTION

AT numerous sites throughout the state, the Florida Department of Transportation (FDOT) contractors and engineers have experienced serious pile installation problems while driving large diameter displacement piles (i.e., 18 in, 24 in, 30 in) with diesel and air hammers. During these installations, high pile rebound (HPR) occurred; followed by a small or no permanent-set. Soils, referred to herein as "high rebound soil," stop the pile driving and result in limiting pile capacity [7], [3].

Pile rebound is defined as the upward elastic pile displacement that occurs during a hammer blow. The maximum initial downward motion is termed "DMX," and is the sum of elastic and plastic deformations of the pile and soil system. The final value of the displacement is the permanent pile penetration for the blow, termed "set." Rebound is the difference between the pile maximum displacement and final set. High rebound describes the situation where the set (i.e.,

plastic soil deformation) represents a small portion of the maximum displacement and the rebound (i.e., recovered elastic deformation) constitutes the majority of the displacement. In some cases, rebound leads to pile damage, delaying the construction project, and requires foundation redesign [3]. Schedule delays ranged from 15 minutes to several weeks with cost overruns more than $20,000 reported [2].

HPR has occurred during pile driving of high displacement concrete and steel piles with different dimensions (e.g., solid concrete, closed-ended steel or concrete pipes, plugged pipes and H-piles) [7]. FDOT considers that excessive rebound takes place when there it is greater than 0.25 inch per hammer blow [5].

## II. HISTORY OF PORE WATER PRESSURE ON HPR

Murrell [9] presented a case history of HPR, which occurred during the construction of a new ferry terminal in coastal North Carolina. The 20-inch, square 70 ft long, Prestressed Concrete Pile (PCP) was designed to support an over water structure. The authors describe the high rebound using the term "Bounce". Pile bounce was observed at overburden depth of 53 ft (elevation -43ft) when the piles penetrated into saturated, firm to stiff, fine-grained soils that originate from marine formations along the southeastern coast of the United States.

Excess pore water pressures $u_2$ obtained during CPTu at the bouncing depth were greater than 20 tsf. When the blow counts during pile driving were at 303 blows per foot (bpf), the pile displacement became zero. The driving process was then stopped for two hours and restrike was then carried out; however, an additional 2.5 ft of pile length was driven with blow counts of 73 bpf, 112 bpf, and 87 blows/6 in. The driving was again halted when large rebound resulted in zero set. After four days the pile was driven using a hammer with a larger ram and a short stroke in order to achieve pile capacity and overcome pile rebound.

Hussein [7] discussed a case study related to HPR during driving of PCP for the State Road 528 Bridge over the Indian River, Florida. A group of 30-inch square PCP with a length of 115 ft and 18-inch circular hollow core were used to support the bridge. The piles rebounded when they penetrated into hard soils consisting of saturated medium dense sand with silt (SP-SM) to fine silty sand (SM) to clayey sands and sandy clays (SC). The authors believe that that the incompressible water in the soil near and below the pile tip produced excessive pore pressure during the driving process which caused the tip to exert an upward force on the pile causing it to

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rebound. However, no analytical proof of this conclusion is available.

Likins[8] analyzed three sites with large toe quakes between 0.4 and 1 inch. He determined that the only common geotechnical parameter observed at each site was the fully saturated soils. Therefore, research focused on analyzing the dynamics of pile driving. Preliminary analysis using the basic wave equation was conducted for each site. The author then modified the results to match field data acquired by CAPWAP (Case Pile Wave Analysis Program). It was proposed that the only reasonable cause of the HPR was the buildup of excess pore pressure beneath the pile tip. It was also clear through testing, that pile capacities decreased when high quake/ rebound occurred. Findings from the work indicate that high quake lowers the pile capacity by a factor of 3. Field observations often lead to a false interpretation that the hammer is not large enough for the pile, and in cases where the hammer size is increased, the pile can be damaged. They conclude that alternative pile designs, such as hollow piles, should be considered as an effective way to avoid high soil quake.

### III. Pore Water Pressure during Pile Driving in Saturated Soils

During a hammer blow, the pile is loaded for 200 milliseconds, which is longer than a dynamic pile load test. For this cycle loading, excess pore water pressure is developed near tip of the pile even in sandy soils. The excessive pore water pressure may affect the stiffness or the ultimate bearing capacity of the pile 7 [6].

Pile driving causes the mechanism of soil stress around and under the pile. During the loading phase, compressive deformation forces water out of the voids. Due to fast loading (40 blows/minute) and low permeability of saturated silty clay soils, positive pore pressure is developed along and under the tip of the pile [1]. Bingjian [1] discussed the effect of pore water pressure during driving of 20-inch diameter prefabricated concrete piles. He reported that excess pore pressure generated under the tip of the pile was equivalent to more than the effective stress, and the soil disturbance was obvious. The developed pore pressure caused effective stress of the soil adjacent to the pile to a radius of 5 to 6 pile diameters. Therefore, the shaft resistance along the pile and tip resistance below the pile was reduced.

Eigenbord [4] studied the generation of pore pressure during driving of 16-inch closed-end pipe piles, into silty clay overlying very dense sand and gravel. Due to driving difficulties, the piles were driven in two stages: During the first stage into silty clay, very low pore pressure was recorded using installed piezometers at depths of 30 ft and 60 ft below ground surface. As the pile was driving into the very dense sand and gravel, very high pore pressures were generated in both shallow and deep piezometer readings to a horizontal distance of 39 ft from the piezometer location. The authors concluded that as the piles penetrated into the dense sand, the upper silty clay soil was loaded from the bottom, and led to high positive pore pressure. The ratio between the pore water

pressure of the deep piezometer reading of 60 ft and the shallow piezometer reading of 30 ft was around 2.7, which had an effect on the shaft resistances along the pile. Fig. 1 shows the mechanism of the loading and pore pressure changes due to pile driving.
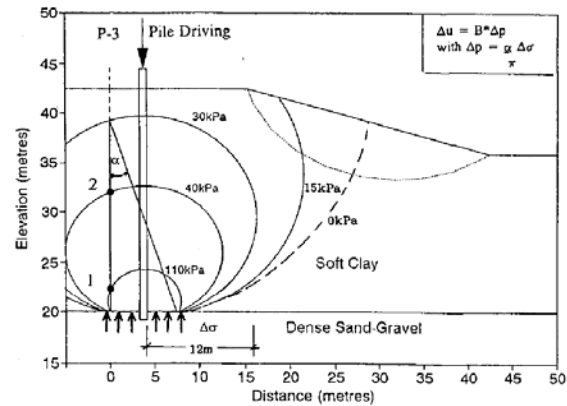


Fig. 1 Pore water pressure changes due to pile driving [4]

The activity of pore water pressure during driving of large-diameter steel piles was studied using CPTu [10] . Multipoint piezometers were installed to record pore pressure before and after the driving process. A 36-inch diameter pile was driven into sand underlain by marine clayey silt, to a depth of 295 ft. CPTu field test was conducted immediately after the driving and excessive pore pressure generated close to the pile was recorded from both the CPTu and piezometer, extending laterally to 30-35 pile diameters. The CPTu dissipation test showed that pore pressure can dissipate very quickly, but dissipation takes longer for larger-diameters piles.

Jackson [11] presented a case history of pore pressure measurements during the jacking of a 50 ft closed pile in sand and silt. During the installation, the shaft resistance and tip resistance were recorded using installed sensors. Data from the transducer showed that excessive pore pressures were recorded on the instrumented pile. The authors mentioned that the shaft and the base resistances were less than predicted from the CPT. However, excessive pore pressure developed during the jacking process, reducing the shaft and the tip resistances. During this investigation, the piles were jacked at different velocities; however, it was noticed that faster installation led to higher pore pressure, which resulted in lower resistances.

### IV. Methods

#### A. Dynamic (PDA) Testing

During the driving of test piles, electronic measurements such as velocity, forces, stroke, blow count, and penetration were determined using PDA sensors. This data was used to clarify the pile movement per blow. Fig. 2 shows a typical HPR PDA data. The plot, with displacement in inches on the vertical axis, and time in milliseconds on the horizontal axis, shows a maximum displacement (DMX) of 1 inch, a set of 0.11 inches, thereby yielding a rebound of 0.89 inches.

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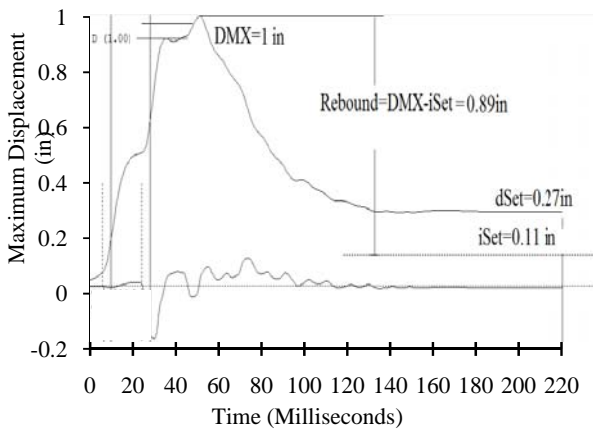Vol:9, No:2, 2015

Fig. 2 Typical PDA pile top displacement versus time diagram from one hammer blow for FDOT HPR site

In addition to the digital set in the PDA output (DFN or dSet) which is not used for this study because it is recorded in short time of 200 milliseconds, the number of hammer blows per foot is used to produce an average inspector set (iSet) per blow. The maximum displacements and inspector set were subtracted to determine the rebound per hammer blow (i.e., DMX-iSet =Rebound). Plots for each of the case studies were developed relating elevations to DMX, iSet and rebound from the PDA data. The elevation associated with the start of the PDA data corresponded to the depth at which pile driving commenced, which was below the ground surface because piles at these sites were set into predrilled holes.

### B. Sites Description

Seven sites in Florida listed below were evaluated by comparing PDA output to CPTu pore pressure $u_2$. Each of these sites contained a number of instrumented piles, allowing eleven piles to be analyzed. Site 1 through 5 displayed HPR with minimal set, site 6 displayed HPR with an acceptable set and site 7 displayed rebound of less than FDOT limit (i.e., 0.25 inches).

1) Intersection of I-4 and SR408 (Anderson Street Overpass),
2) Intersection of SR50 and SR436 Overpass (SR50/SR436 Overpass),
3) Intersection of I-4 and US.Highway192 (I-4/US.192 Ramp CA),
4) Intersection of I-4 and Osceola Parkway (I-4/Osceola Parkway),
5) SR.83-US.Highway 331 Ramsey Branch Road (Ramsey Branch Bridge),
6) Intersection of I-4 and SR408 (Ramp B),
7) Intersection of SR 417 and International Parkway.

### C. Soil Properties

Split-barrel and Shelby tube samples were obtained in order to establish the soil profile. The soil samples were classified in accordance with Unified Soil Classification System (USCS). At each site, a generalized soil profile was developed from the SPT boring closest to the rebounded pile.

Sand was the predominate soil at the HPR sites consistently representing over 50 percent of the soil. The soil strata where HPR occurred can be classified as one of the following groups: SC, SM-SC, SM, CL, SP-SM, SP-SC and CH. Most HPR layers had high fines content with a natural moisture content less than the liquid limit. The soils plotted near the A-line on the Cassagrande plasticity chart. These soils displayed an olive green to light green color with visual descriptions ranging from clayey and silty fine sands, to highly plastic clays with low permeability:

### D. Piles and Driving Equipment

A summary of pile driving information obtained from the case histories is presented in Table I. It includes information such as site description, pile description, pile spacing, hammer characteristics, driving blow counts, rebound and elevations. As the information suggests, there are several common characteristics among the HPR sites:

1) Piles were displacement piles ranging between 18 and 24 inches;
2) Tested and production piles were longer than 70 ft;
3) Pile groups spaced at 6 to 11 ft (2.5 to 5.5B);
4) Pile were set into predrilled hole with varying depth;
5) Pile driving hammers were single acting;
6) Rebound occurred in Central Florida sites (Site 1 to 4) between elevations 35 to -10 ft;
7) Average pile driving blow counts in the rebound layers was greater than 105 bpf while in the no rebound sites were less than 50 bpf.

## V. RESULTS AND DISCUSSIONS

### A. Site 1 Anderson Street Overpass

The approximate ground surface elevation (GSE) at the site was 104 ft. The piles were designed as 24-inch square PCP, 124 ft long. Severe HPR problems occurred between elevation 15 and -10 ft during installation of the displacement piles at Pier 6 located on the east end of the overpass. As a result, the foundations were redesigned using low displacement steel H-piles. Rebound occurred only during installation of the concrete piles.

Three CPTu tests were conducted near the rebounded piles. Fig. 3 shows the variation of pore pressure and PDA pile displacement with elevation. An increase in pore pressures ranged between 1 and 10 tsf in the no HPR soils above elevation 15 ft, and more than 20 tsf in the rebound soils from elevation 15 to near 0 ft. By comparing the PDA rebound, set and CPTu pore pressure in Figs. 3 (b) and (c), it is evident that when the pore pressure is approximately 30 tsf; the pile set became zero and pile driving difficulties arisen as blow counts reached 365 bpf.

Driving average blow counts at no rebound soils above elevation 15 ft were less than 30 bpf.

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TABLE I
PILE DRIVING INFORMATION SUMMARY

| | Site Description | Pile size and type | Pile Length (ft) | Pile Spacing (ft) | [a]Hammer Model Type | Ram Wight (kips) | [b]Average BL (blows/ft) | [c]Total BL | Rebound Elevation (ft) | MAX Rebound (in) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Anderson Street Overpass | 24-in SPCP | 124 | 7 | Delmag D62-22 | 13.67 | 135 | 3674 | +15 to -10 | 1.4 |
| | | HP (14 x 89) | 120 | NA | ICE I-30 | 6.6 | NA | NA | No Rebound | |
| 2 | SR 50/SR 436 overpass | 24-in SPCP | 105 | 8 | APE D62-42 | 13.7 | 143 | 526 2599 | +26 to +17 | 1.1 |
| 3 | I-4 / US.192 Pier 6 | 24-in SPCP | 106 | 7 | ICE 120 S | 12 | 220 | 3108 | +35 to 25 | 0.6 |
| | Pier 7 | 24-in SPCP | 112 | 9 | ICE 120 S | 12 | 140 | 5183 | +35 to 20 | 0.6 |
| | Pier 8 | 24-in SPCP | 100 | 6 | ICE 120 S | 12 | 111 | 4431 | 30 to 15 | 1.25 |
| 4 | I-4/ Osceola Parkway | 24-in SPCP | 95 | 6 | ICE 120 S | 12 | 105 | 2687 | +15 to +8 | 0.9 |
| 5 | Ramsey Branch Bridge | 18-in SPCP | 100 | 10 & 20 | Vulcan 512 Air | 12 | 110 | 3054 | -28 to -70 | 1.2 |
| 6 | I-4/ SR 408 (Ramp B) | 18-in SPCP | 100 | NA | Delmag D36-32 | 7.94 | 50 | 3101 | +30 to 0 | 0.5 |
| 7 | SR 417 /International Parkway | 24-in SPCP | 100 | 6 | APE D46-42 | 13.7 | 42 | 1797 | +5 to 0 | 0.25 |

SPCP=square prestressed concrete pile; [a]single acting; [b]average driving blow counts at HPR layer; [c]total pile driving; BL= blow counts; NA= not available.
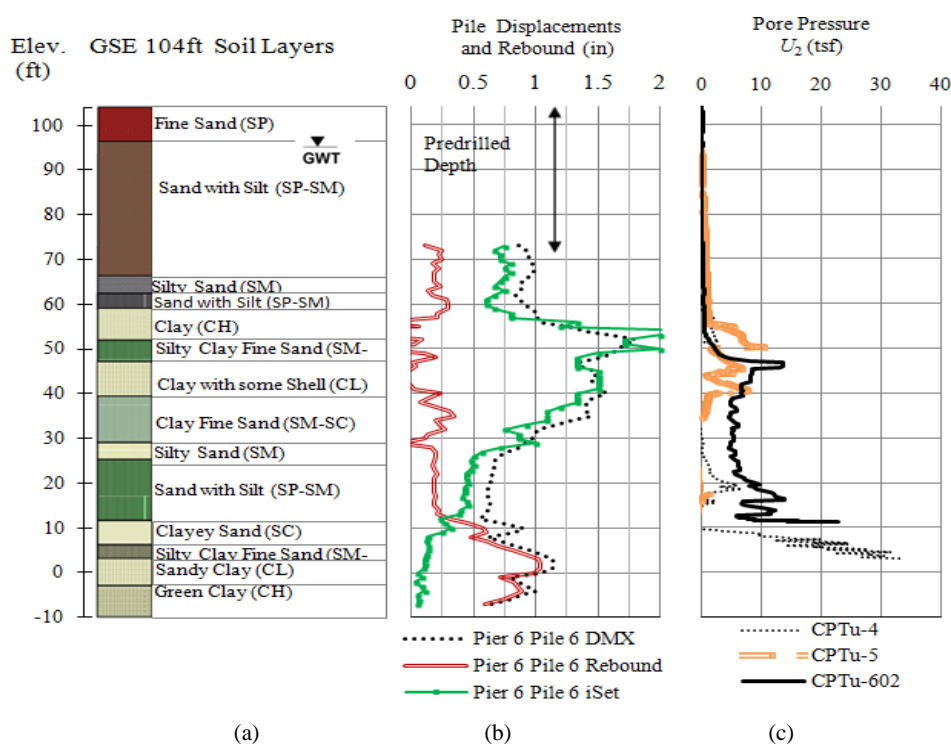


Fig. 3 (a) Soil profile, (b) PDA diagrams, and (c) CPTu $u_2$ for site 1 Anderson Street Overpass

### B. Site 2 SR50/SR436 Overpass

Twenty-four inch square, PCP piles were installed to support the overpass at the intersection. These piles were 105 ft long with a GSE of 98 ft. Due to practical refusal of 20 blows per inch; several piles did not reach the specified minimum tip elevation of 15.6 ft, corresponding to a depth of 82.4 ft. Fig. 4 (b) shows the observed PDA rebound varied from 0.25 to 1 inch, and was first encountered at an elevation of 26 ft and continued to increase until driving terminated at 18 ft. Two CPTu tests were performed at this site to a depth of 80 ft. By matching the PDA data and the pore pressure in Fig. 4 elevation 26 to 18 ft, it was observed that the pile displacement decreased as the pore pressure reached a very high value of more than 20 tsf. A peak pore pressure was recorded at elevation 23 ft, corresponding to a significant decrease in displacement at the same elevation. Due to excessive HPR, pile driving blow counts increased from less than 20 bpf to over 140 bpf in the rebound soils.

### C. Site 6: Ramp B

Two piles were driven as instrumented test piles; pile 5 pier 2 and pile 2 end bent 1. These piles were 100 ft long PCP 18-inch square piles. Three CPTu sounding were conducted near the two test piles. The increase of pore pressure (Fig. 5 (c)) encountered during CPTu testing, ranged between 10 and 17 tsf. This correlates to a small amount of rebound followed by an acceptable pile displacement. Due to low $u_2$ at this site, there were no driving difficulties and therefore piles were driven at low blow counts of 40 bpf.

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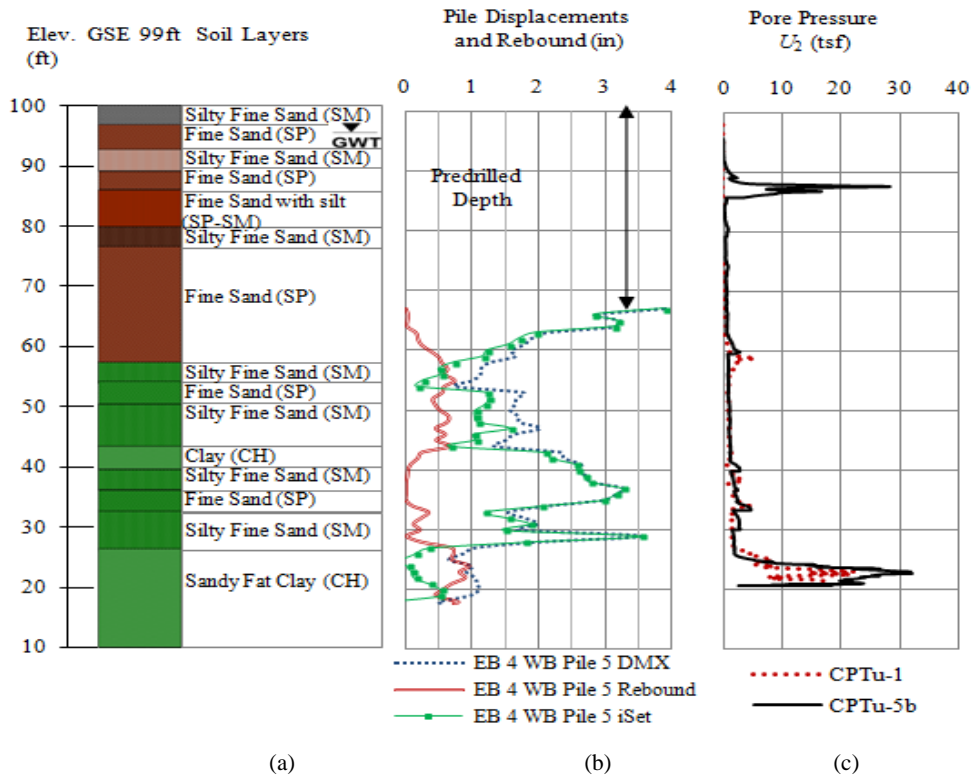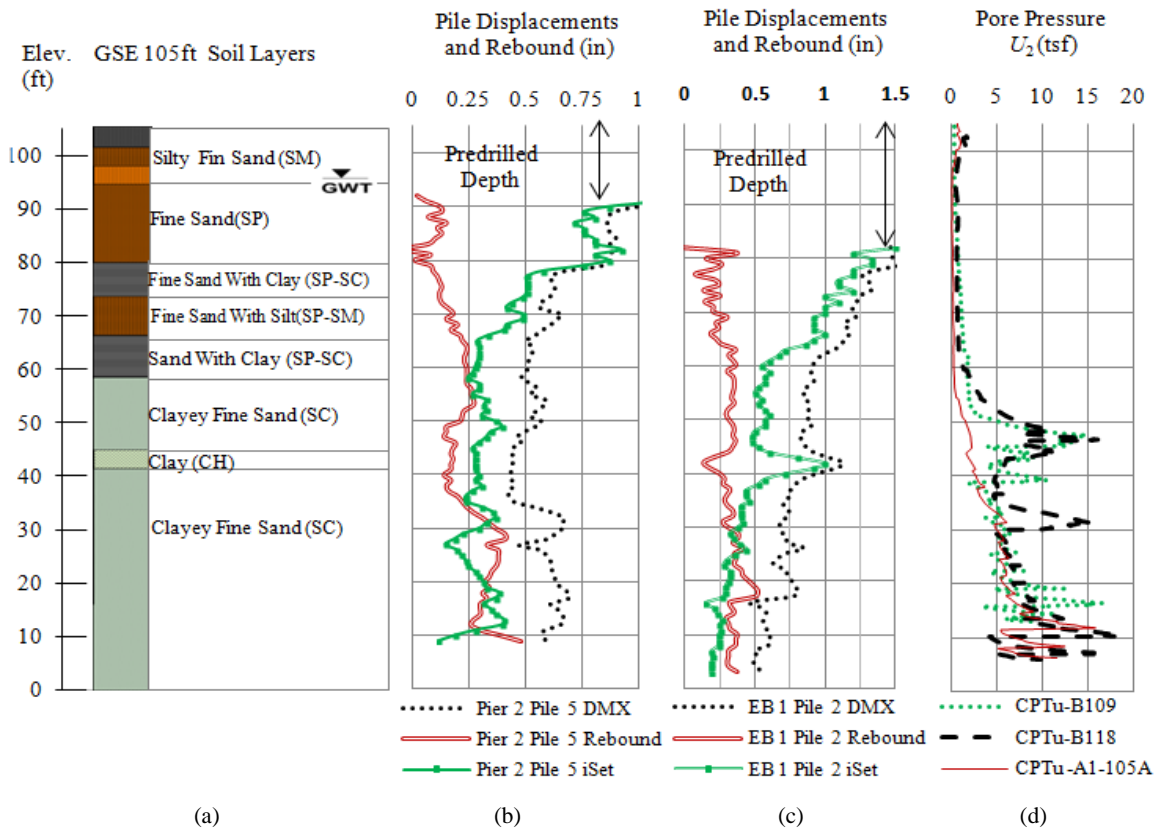Vol:9, No:2, 2015

Fig. 4  (a) Soil profile, (b) PDA diagram and (c) CPTu $u_2$ for site 2 SR50/SR436 Overpass



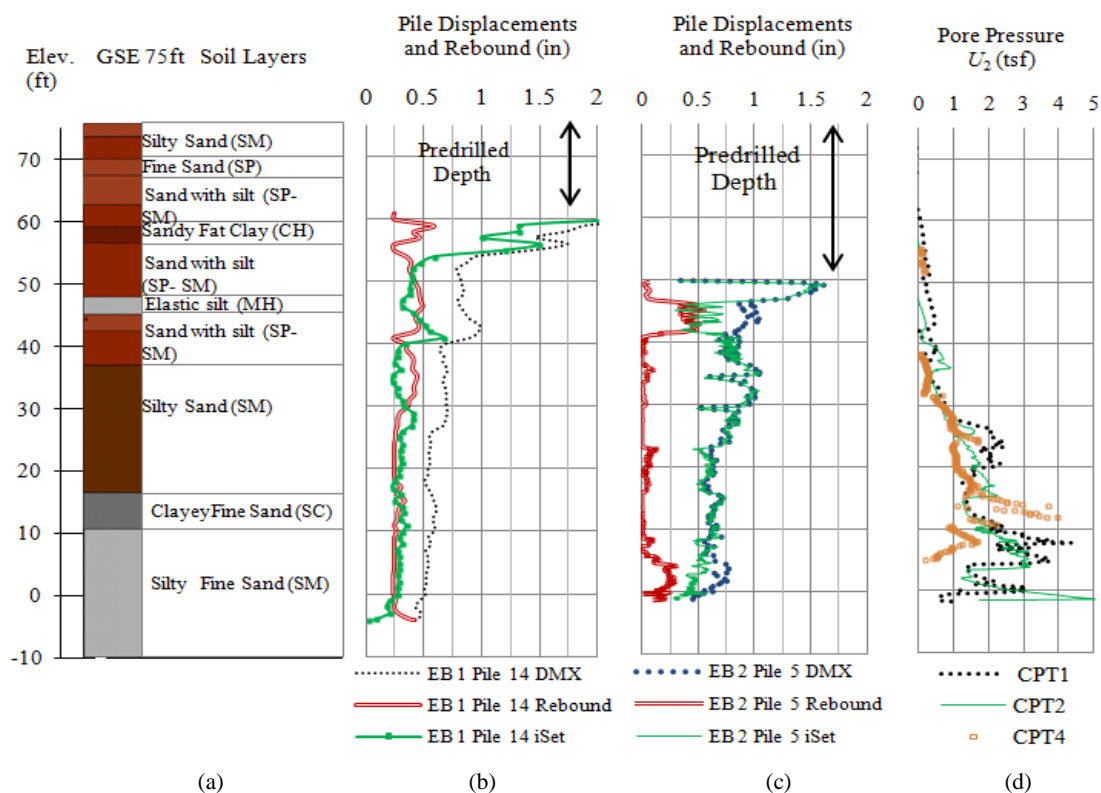Fig. 5 (a) Soil profile, (b) and (c) PDA diagrams and (d) CPTu $u_2$ for site 6 Ramp B

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Fig. 6 (a) Soil profile, (b) and (c) PDA diagrams, and (d) CPTu $u_2$ for site 7 SR417/International Parkway

## D. Site 7: SR417 /International Parkway

Two piles instrumented with PDA sensors were tested at this site. These piles were 24-inch square, PCP and 100 ft in length. A small amount of rebound was observed (Figs. 6 (b) and (c)) followed by a large undergoing set; however, the piles met driving specifications set forth by the FDOT (i.e., less than 0.25 inch rebound per blow). Contractor and engineers did not experience hard driving or difficulties. Piles were driven at low blow counts.

Three CPTu soundings were conducted near the two test piles. Pore pressure $u_2$ increased from an average in the overlying soils of-0.3 tsf to less than 5 tsf where the piles experienced a small amount of rebound. Fig. 6 (d) shows this increase in the pore pressure and rebound (Fig. 6 (c)) between elevation 10 and 0 ft. This data is also consistent with the finding previously described, that rebound increases with pore pressure, soil layers with pore pressures $u_2$ less than 5 tsf determined during the CPT are not likely to cause any HPR.

## VI. CORRELATIONS BETWEEN REBOUND AND CPTu PORE PRESSURE

Linear correlations between the pile rebound, permanent-set and the maximum pore pressure are presented in Fig. 7. These correlations were developed within the HPR zone using CPTu pore pressure $u_2$ and both rebound or inspector permanent-set at the same elevation (e.g., Maximum pore pressure at site 1 is 33 tsf and corresponding rebound at the same depth is 1 in and the set is 0.13 in). Fig. 7 (a) has plots of rebound and set versus pore pressure while (b) presents the same variables

versus the ratio of pore pressure $u_2$ divided by the calculated (hydrostatic) pressure ($u_o$). The data from this study plus the data presented by [9] was combined. It consistently produced strong linear correlations with regression coefficients $R^2$ of 0.6 or higher. The permanent-set decreased and rebound increased as pore pressure increased. Rebound versus either pore pressure or $u_2/u_o$ nearly plots through the origin, indicating rebound would equal approximately 2.5% of the CPTu $u_2$ or 5.5% of the $u_2/u_o$ ratio. Slightly higher correlation coefficients in Fig. 7 (b) indicate increased agreement between HPR and the $u_2/u_o$ ratio. The data from Murrell [9]agrees with the results of this study.

## VII. CONCLUSIONS AND RECOMMENDATIONS

The overburden depth at which HPR occurred was typically greater than 50 ft. Large displacement piles predrilled to 25 ft and driven at numerous Florida locations have recorded rebound values over 1 to 1.5 inches per hammer blow. These problems generally occurred in soils that did not display any unusual properties during routine soil site investigations. In general, HPR soils displayed an olive green to light green color with visual descriptions ranging from dense clayey and silty fine sands, to hard highly plastic clays with very low permeability. Most HPR layers had high fines content, with a natural moisture content less than the liquid limit.

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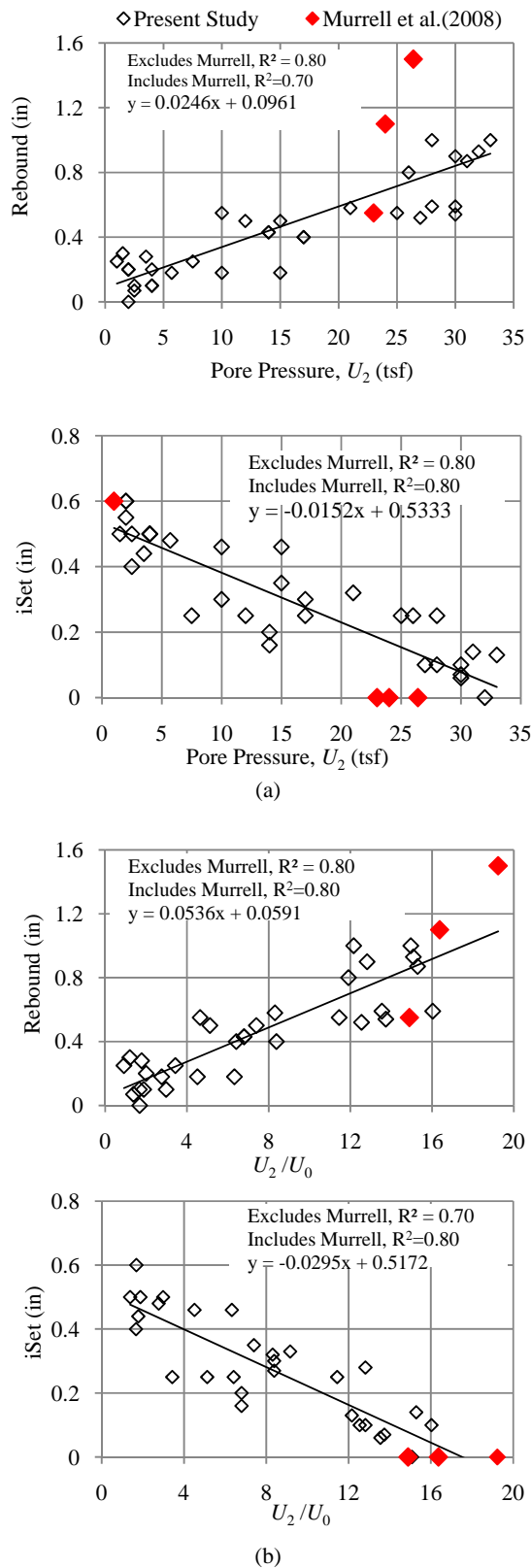Vol:9, No:2, 2015

Fig. 7 Correlation includes [9] between Rebound, Permanent-set and (a) CPTu pore water pressure, and (b) ratio of CPTu pore water pressure and hydrostatic pressure ($u_0$)

There was a large increase in the CPTu pore pressure $u_2$ values from near zero or negative pressure to high positive pore pressures in all the HPR zones identified by the PDA data. This increase in pore pressure, in conjunction with variations in the strength, stiffness and soil composition may be the combination of geotechnical properties that could identify the potential for high pile rebound. Good correlations between rebound, permanent-set and pore pressure indicated that permanent-set decreases and rebound increases linearly with either pore pressure $u_2$ or $u_2/u_o$.

Geotechnical engineers can expect to encounter HPR problems when driving displacement piles if the CPTu pore pressure $u_2$ is greater 20 tsf. It is possible to drive piles through saturated fine silty sand to sandy silt or clayey sand if pore pressure $u_2$ is less than 5 tsf; while pile driving difficulty may increase if $u_2$ ranged between 5 and 20 tsf.

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