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Assessment of matrix soil improvement using displacement aggregate piers

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Assessment of matrix soil improvement using displacement aggregate piers

by

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LIST OF SYMBOLS

Symbol	Description	Units
A_r	Area replacement ratio	–
Dr	Relative density	%
e_{max}	Maximum void ratio	–
e_{min}	Minimum void ratio	–
f_g	Group effective factor	–
F_r	Normalized friction ratio	–
f_s	Sleeve friction	%
I_c	Soil behavior type index	–
I_D	Density index	%
I_D	Improvement index	–
K_o	Coefficient of earth pressure at rest	–
K_s	Lateral earth coefficient during penetration	–
$N_{((1),60)}$	The SPT blow count at a vertical effective overburden stress of 100 kpa and an energy level equal to 60% of the theoretical free-fall hammer energy to the drill stem	–
N_{60}	SPT energy ratio	–
N_k	Cones factors (11 to 19)	–
P_o	Cavity pressure at infinite distance	tsf/MPa
Q_t	Normalized cone resistance	–
q_c	Measured cone resistance	tsf/MPa

q_t	Corrected cone resistance $[q_c + (1\text{-}a)\, u_2]$	tsf/MPa
s_i	Tip movement	inch/cm
S_i	Total settlement	inch/cm
S_u	Undrained shear strength	tsf/MPa
u_o	In situ pore water pressure	tsf/MPa
ε_r	Radial strain	–
ε_θ	Tangential strain	–
σ_h	Horizontal stress	tsf/MPa
σ_{vo}, σ_{vo}'	Overburden stress (total, effective)	tsf/MPa
σ_θ	Tangential stress	tsf/MPa
ϕ'	Effective friction angle	degree
$\Delta q_{(c,ef)}$	Average tip resistance values within the top 3 diameters of displacement aggregate pier	tsf/MPa
$\Delta q_{(c,ef)}$	Effective amount improvement (11 tsf or 1 mpa)	tsf/MPa
a	Radial distance	ft/m
D	Diameter	ft/m
DAP	Displacement aggregate pier	–
DCP	Cone penetration test	–
E	Young's modulus	tsf/MPa
FC	Fines content	%
G	Shear modulus	tsf/MPa
IRAP	Impact rammed aggregate pier	–
N	Number of blows in an SPT	Blow/ft; blow/0.3m

OCR	Over consolidation ratio	–
P	Cavity pressure at the r distance	tsf/MPa
SPT	Standard penetration test	–
e	Void ratio	–
r	Radius	ft/m
u	Pore water pressure	tsf/MPa
ϕ	Total friction angle	degree

Unit Convert

1 ft = 0.31 m

1 ksf = 48 kPa

1 tsf = 96 kPa

ABSTRACT

Many ground improvement techniques are subjected to the limitations of cost, safety and construction time. Innovations in industry are resulting in new technologies and construction methods to overcome these limitations. Displacement Aggregate Pier (DAP) technology developed by *Geopier^{TM} Foundation Company* is one such technology and is the focus of this research. Specifically, the influence of pier installation on matrix soil densification is addressed based on evaluation of several full-scale field studies. Cases histories are presented describing the use of cone penetration test (CPT) and standard penetration test (SPT) to investigate matrix soil densification for a range of ground conditions. Additionally, full scale load tests were studies for single piers and pier groups to confirm the current design approach. Data from sixteen of sites were analyzed. Although site specific analysis reveals the unique behavior of IRAP elements, an effort was made to combine data from multiple sites to investigate general relationships between matrix soil densification and soil type, depth, initial relative density, pier spacing, radial distance, groundwater table locations and soil strata. Key findings from this study show that ground densification is highest for matrix soils with less than 20% fines and that the relative density increases for groups of piers and in particular for sandy matrix soils. Evaluations of group effective factor, improvement index, ground modification and settlement are also presented in this research. Simplied design tables presenting change the CPT tip resistance for individual DAPs and group of DAPs are present as one outcome from this study.

CHAPTER 1: INTRODUCTION

This chapter is arranged in five sections, background about displacement aggregate pier technology, the research goals, the research objectives, benefits and significance of this research, and the arrangement of the thesis.

BACKGROUND

Because many projects are constructed on weak soils, such as soft clay and loose sand, and because all structural loads finally transfer to the matrix soil beneath the foundation, that soil often requires improvement to adequately support the structures. Traditional ground improvement techniques, such as deep foundations, preloading, and overexcavation and replacement, can be costly and time consuming. In recent years, displacement aggregate piers (DAP) have been developed and increasingly are used to improve such soils and reduce foundation settlement.

The Geopier Foundation Company ™ has developed DAP technologies including Geopier®, Impact® Rammed Aggregate Piers (IRAP or IP), Pyramid Piers™, and Taper Mandrel Rammed Aggregate Piers (TMRAP). These technologies have been used to support economical construction of commercial buildings, oil tanks, warehouses, and highway embankments.

Many instruments are used to investigate ground improvement, such as Pressuremeters and large plate load tests, but Cone Penetration Tests (CPT), and Standard Penetration Tests (SPT) are the most commonly used. CPTs provide continuous, detailed profiles of tip resistance, sleeve friction, and pore water pressure, and SPTs provide a measure of penetration resistance or blow count incrementally with depth. These tests can be empirical related to engineering parameter values that estimate matrix soil characteristics before and after the installation of displacement aggregate piers. Full-scale modulus load tests are used to investigate the performance of pier elements and confirm pier performance when it is subjected to structural loads. The combination of modulus load test results and results from CPTs and SPTs may be used to describe the interactions between piers and matrix soils, and more importantly, the overall effectiveness of DAPs as a ground improvement system.

CPTs, SPTs, and modulus load tests conducted on three kinds of displacement aggregate piers—IRAPs, pyramid piers™, and TMRAPs—from 16 sites provided the data used in this study.

RESEARCH GOAL

The main goal of this research was to produce two simple design tables indicating matrix soil improvement for DAPs. Three areas of investigation were planned to meet this goal: an investigation of matrix soil improvement around displacement aggregate piers; the identification and examination the matrix soil factors that influence the load-settlement response of these piers during vertical loading; and a study of the interactions between matrix soil and piers.

RESEARCH OBJECTIVES

In order to create the design tables that indicate matrix soil improvements, the first objective was to investigate matrix soil improvement around displacement aggregate piers based on soil parameters determined by CPTs and SPTs that were conducted before and after installation of the piers at the case sites. The magnitudes of improvement factors were studied with respect to soil types, fines content, initial relative density, and pier spacing.

The second objective was to evaluate the performance of these piers based on full-scale modulus load tests and soil information from CPT profiles and SPT boring logs. The third objective was to study the interactions between matrix soil and piers.

RESEARCH BENEFITS AND SIGNIFICANCE

The most important result of this research will be two design tables that indicate matrix soil improvement for displacement aggregate piers which will allow design engineers to predict the matrix soil improvement for known ground conditions. Using these tables can reduce the number of tests that have been required, which will result in savings of both time and money as well as more efficient engineering design and construction processes.

ARRANGMENT OF THE THESIS

The next chapter is a review of literature about aspects of constructing displacement aggregate piers identified from the case studies, some theoretical background related to soil improvement, and information about in situ testing methodology. The third chapter presents sixteen case histories that the Geopier™ Foundation Company provided.[1]

The fourth chapter presents the results, and the fifth chapter provides a discussion and analysis of the results. The final chapter outlines the conclusions of the research and suggests applications for industry and future research.

[1] I am grateful to the Geopier™ Foundation Company for supporting this research.

CHAPTER 2: BACKGROUND AND LITERATURE REVIEW

This chapter first presents background information on equipment, construction procedures, and materials involved in displacement aggregate piers. Second, information about the theoretical background of lateral stress, cavity expansion, and settlement will be presented. The chapter concludes with a review of the literature about in situ testing methods, including CPTs, SPTs, and full-scale modulus load tests.

EQUIPMENT, CONSTRUCTION PROCEDURES, AND MATERIALS

This section discusses the equipment, construction procedures, and materials that are used in constructing displacement aggregate piers.

Equipment

Specific equipment is used in the process of constructing displacement aggregate piers and the selection of particular equipment depends on ground conditions and design purposes at a given site. In general, displacement aggregate piers are constructed with installation machines and mandrels; aggregate is usually delivered by loaders.

- Installation machines
  - Excavator
  - ABI or Liebherr Mobileram
  - Piling hammer
- Mandrels
  - Cylinder mandrels
  - Pyramid mandrels
  - Taper mandrels

Figure 1 shows examples of these kinds of equipment, a telltale used in testing bottom settlement of a pier, and a profile of a displacement aggregate pier.

Figure 1: Selected equipment for DAP installation and an illustration of a DAP

Construction Procedures

The major construction procedures for DAPs are:

1. Drive the mandrel through the soil to the design depth using heavy crowd force and vertical hammer rams.
2. Load hopper and mandrel with 2–4 yd^3 crush rock.
3. Raise the mandrel to charge hole with rock.
4. Ram mandrel into rock to expand the rammed aggregate pier diameter, densify loose sand, and stiffen weak soil.
5. Repeat steps 3 and 4 until the DAP is installed to the design elevation.

Figure 2 shows the typical construction procedures of impact DAPs that are discussed in this report. The uses of alternative methods, such as different mandrel shapes and raise/rammed thicknesses, normally depend on design requirements. In most cases, three construction methods are used: raised 3 ft (1 m) and rammed 2 ft (0.65 m) (3'/2'); raised 4 ft (1.33 m) and rammed 3 ft (1 m) (4'/3'); and raise 4 ft rammed 4 ft then raises 4 ft rammed 3 ft (4'/4' and 4'/3').

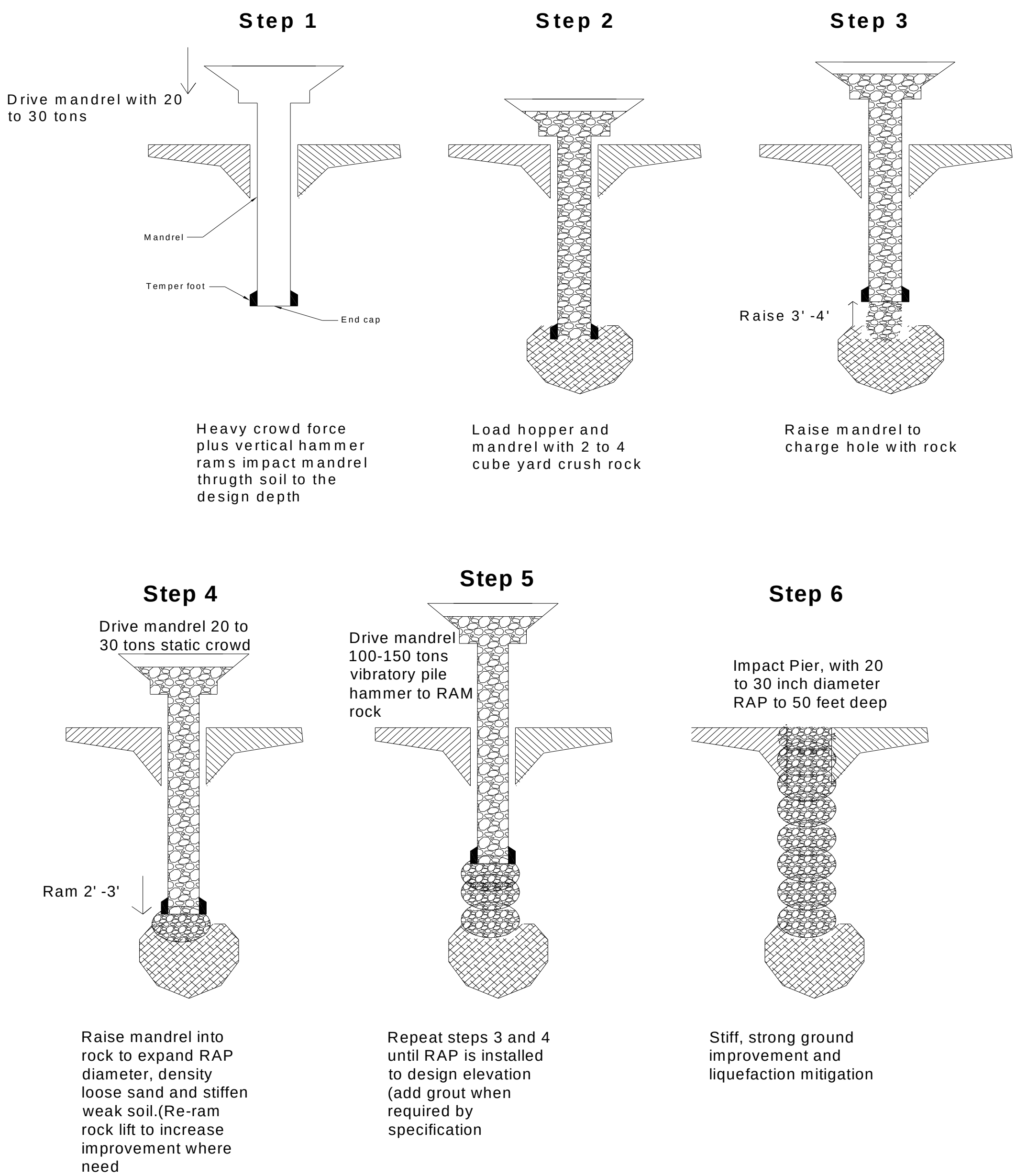


Figure 2. Impact rammed aggregate pier installation process (after Farrell Inc.)

Materials

DAPs typically use both open graded and well graded aggregate with approximately 1.0 in. (2.5 cm) maximum particle size. Flow restriction is observed during construction for aggregate with 2– 2.5 in. (5 to 6.3 cm) particle size. The internal friction angles of open graded aggregate and well graded aggregate are about 48 degrees and 52 degrees, respectively (Fox and Cowell 1998). The high friction angle of the aggregate after ramming is related with the stiffness ratio, which is defined as the relative stiffness between the aggregate pier elements and the matrix soil (Pitt and White 2003). Open graded aggregates are typically used below the water table to provide vertical drainage, and well graded aggregates, which have larger internal friction angles, are typically used above the water table. The permeability of the well gradated aggregate is similar to that of fine grained soils (Pitt and White 2003). In some cases, clean sand can be used as an alternative material.

THEORETICAL BACKGROUND

This section discusses prior research and theory about lateral stress, cavity expansion, and DAP settlement during loading.

Lateral Stress

DAP installation involves the lateral displacement of the soil surrounding the pier during mandrel penetration and ramming action. The lateral stresses induced during DAP installation are radial stress, tangential stress, and vertical stress; these stresses are shown in Figure 3. Pier element lateral outward displacement tends to increase the lateral pressure in soil around the piers. Handy (2001) explained the lateral stress change using the stress path that is illustrated in Figure 4. Due to the cavity expansion, horizontal stress increasing in matrix soil allows the bearing capacity to increase as well. White et al. (2000) pointed out that the high lateral stress induced by the high-energy impact rammed action might generate the soil passive stress conditions in matrix soil.

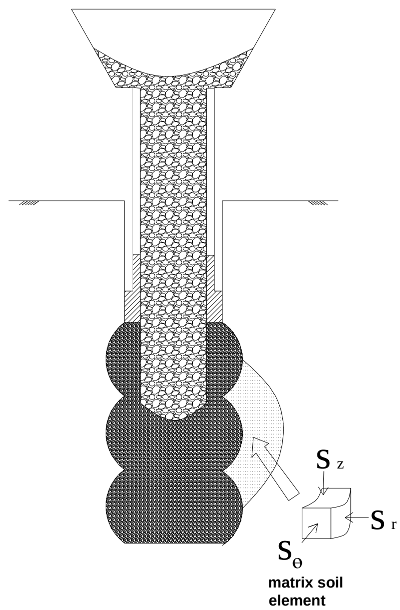


Figure 3. The stresses induced by ramming of IRAP

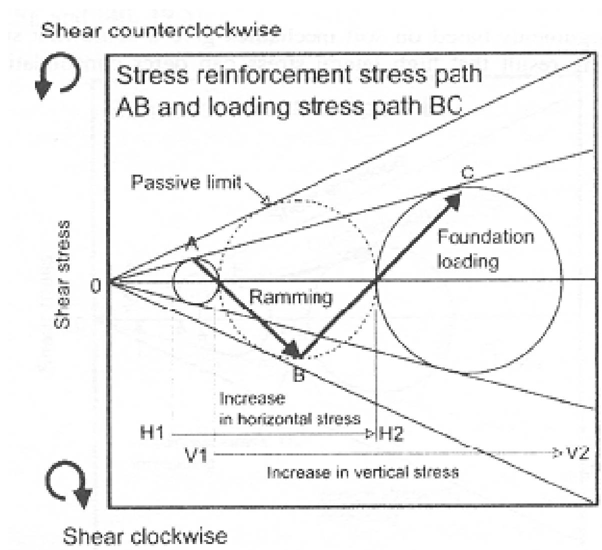


Figure 4. Lateral stresses change due to the cavity expansion (from Handy 2001)

According to White et al., (2000), the passive zone seems to be related to the overburden pressure and the soil conditions (Figure 5). Further, Handy and White (2006) reported that radial cracks that occur during construction would affect the lateral stress distributions in the fine grain ground.

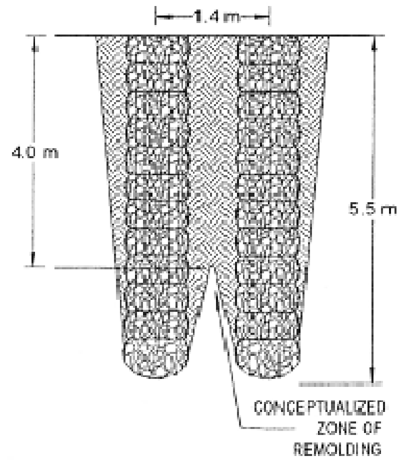


Figure 5. Passive zone due to the rammed aggregate pier (from White et al. 2000)

Figure 6 illustrates the bulging behavior of the rammed aggregate pier during loading, Wissmann et al. (2001) found that lateral stress increases as the result of pier bulging during loading. The similar results were confirmed by the in situ tests results and numerical analysis (Ham and White 2006).

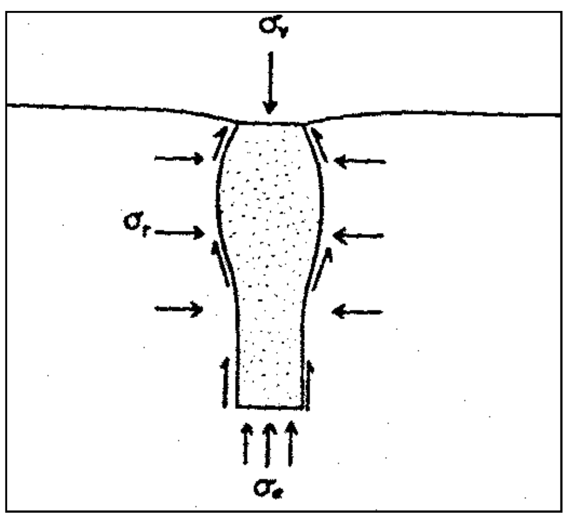


Figure 6. Schematic of bulging behavior and stress distribution in one dimension (from Hughes and Withers 1974)

Cavity Expansion Theory

Construction processes of the mandrel penetration and impact ramming action involve with the cavity expansion phenomena. Based on different assumptions, cavity expansion can be classified as cylinder cavity expansion and spherical cavity expansion. This report will mainly discuss the situation of cylinder cavity expansion. The equilibrium of the cylinder cavity expansion problem for the infinite boundary conditions in 2-dementions can be expressed as follow (Figure 7):

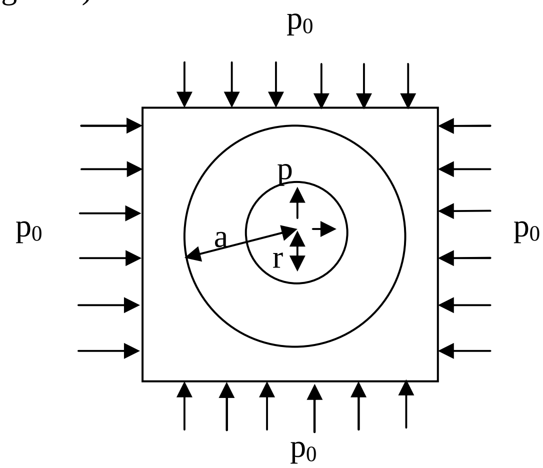


Figure 7. Cavity expansion modes

$$r\frac{d\sigma_r}{dr} + (\sigma_r - \sigma_\theta) = 0 \quad (1)$$

Where σ_r= radial stress, σ_θ= tangential stress. The equation is subject to two boundary conditions:

$$\sigma_r|_{r=a} = -P \quad (2)$$

$$\sigma_r|_{r=\infty} = -P_0 \quad (3)$$

Yu (2000) derived the following equations under elastic-perfectly plastic condition in for cylinder cavity expansion in the forms:

Stresses in the plastic region:

$$\sigma_r = \frac{Y}{a-1} + Ar^{-\frac{(a-1)}{a}} \quad (4)$$

$$\sigma_\theta = \frac{Y}{a-1} + \frac{A}{\alpha} r^{-\frac{(a-1)}{a}} \quad (5)$$

Where,

$Y = \frac{2C\cos\phi}{1-\sin\phi}$; C = cohesion = 0 (clean sand)

$a = \frac{1+\sin\phi}{1-\sin\phi}$

A = constant of integration

Stress in the elastic region:

$$\sigma_r = -p_0 - Br^{-2} \quad (6)$$

$$\sigma_\theta = -p_0 + Br^{-2} \quad (7)$$

B = second integration constant

The strain in plastic zone:

$$\varepsilon_r = \ln(\frac{dr}{dr_0}) \quad (8)$$

$$\varepsilon_\theta = \ln(\frac{r}{r_0}) \quad (9)$$

The displacement in elastic zone:

$$u = \delta(\frac{c}{r}) \quad (10)$$

Where:

$\delta = \frac{Y+(a-1)p_0}{2(1+a)G}$

Both Randolph et al (1979) and Yu (2000) modeled pile driving that involved with the cylindrical cavity expansion as the undrained expansions. The excess pore water pressures generated in driving were assumed to dissipate by means of outward radial flow of the pore water. Radial displacement can be estimated by the known soil parameters.

DAP Settlement During Loading

Rammed aggregate piers were well documented to design to reduce the settlement and increase the footing bearing capacity (Lawton et al. 1994; Wissmann et al. 2001). The settlements of the DAP were normally overestimated than the predicted values (Handy et al. 1999; Wissmann et al. 2001). The mechanisms of the pier-soil interactions are not fully understood. Hughes and Withers (1974) indicated that the stress transfer from the pier to the soil through the skin friction that vertical stress would rapidly diminish and the aggregate pier would likely bulge near the top of the pier. Fox and Cowell (1998) proposed that vertical stress was transferred to the matrix soil more than 90% downward beyond about four times diameters depth.

Aboshi (1991) introduced the settlement reduction factor to estimate the settlement. The settlement reduction factor was determined from the area replacement ratio. The after-treatment settlement was obtained from the original ground without any improvement settlement multiplied by the settlement reduction factor. Fox and Cowell (1998) proposed a method to estimate the settlement. They separated the stress affected zones to upper zone and lower zone. The upper zone consists of the DAP zone while the lower zone is found below the upper zone soil layers (Figure 8).

Wissmann et al (2001) proposed that the load response and aggregate pier deformation was indicated in the top settlement and bottom settlement curves. The bulging behavior indicated that the inflection point for the top plate but not for the tell-tale near the bottom (Figure 9).

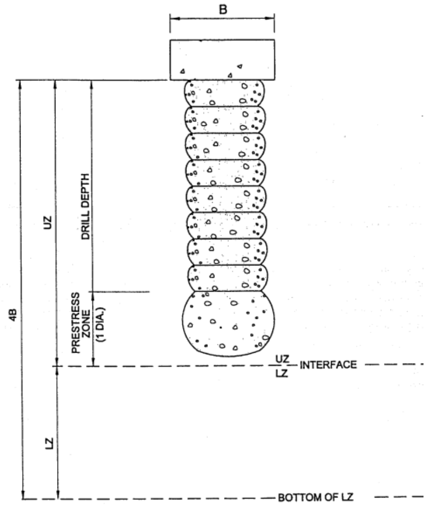


Figure 8. Schematic of upper zone and lower zone (from Fox and Cowell 1998)

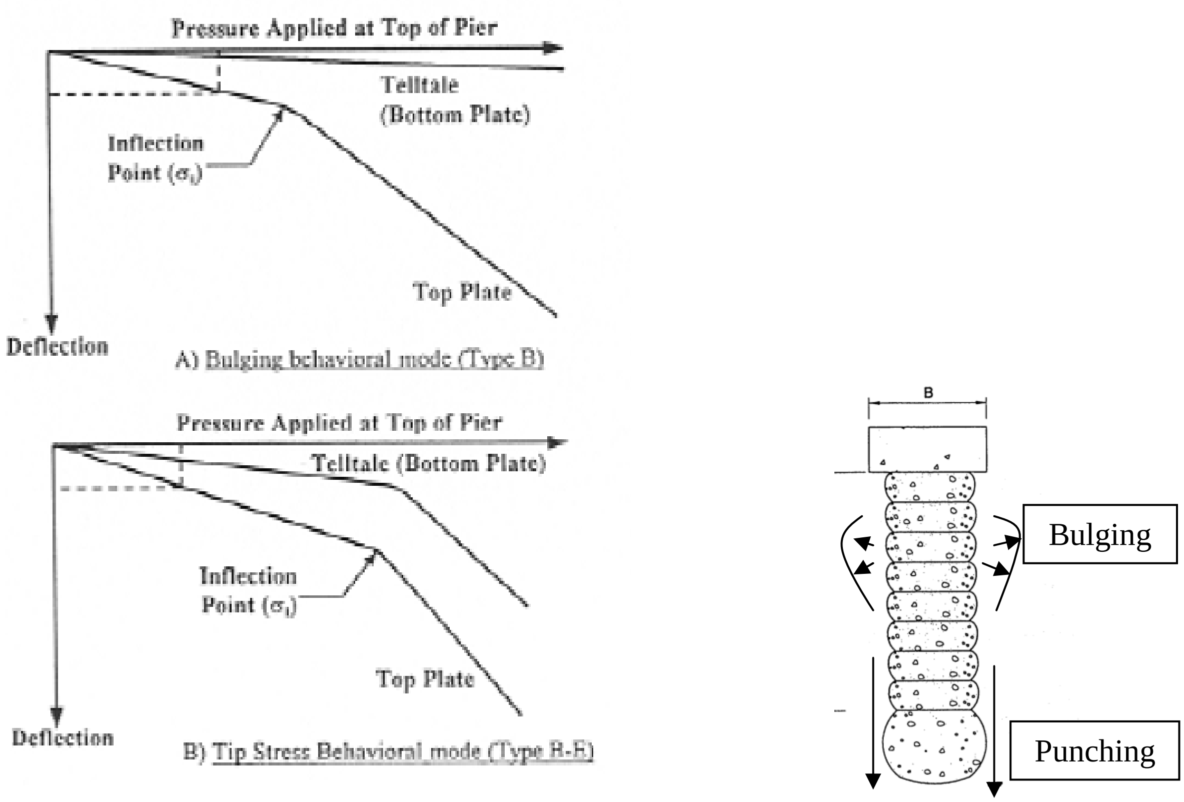


Figure 9. Typical modulus load test results (from Wissmann et al 2001)

Punching deformation was defined that the inflection of the top of pier response corresponds to an inflection point in the tell-tale response.

Wissmann and Fox (2000) reported that the stiffness of the pier related with the effective friction angle of the matrix soil. Figure 10 shows the total stress states induced the aggregate pier installation. They concluded the three failure models for pier groups: upper zone shear failure, individual punching failure and composite punching failure.

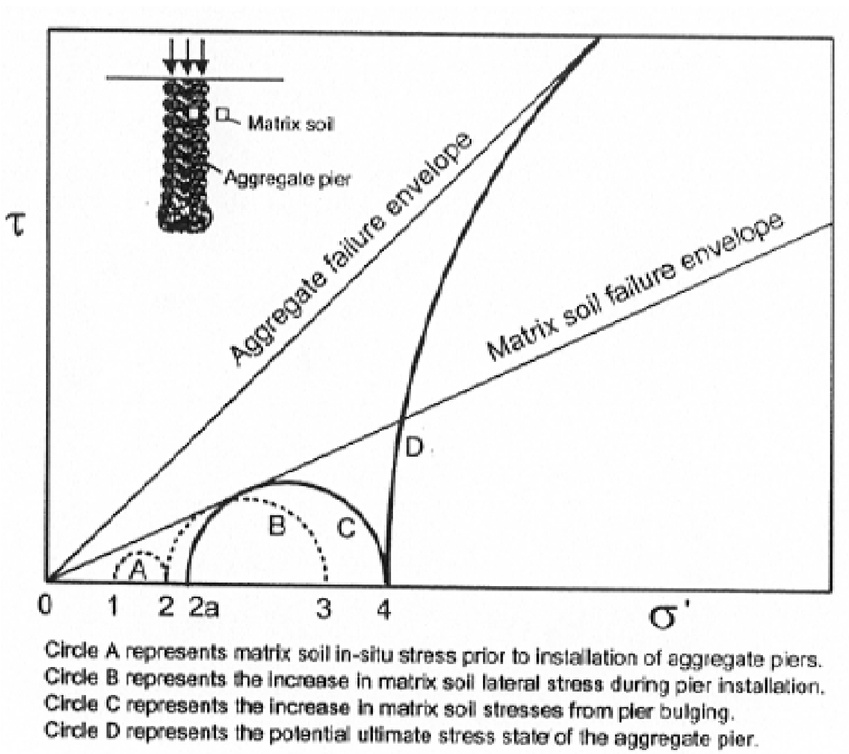


Figure 10. Stress path of the matrix soil and pier element (from Wissmann 1999)

IN SITU TESTING

Many field testing methods have been used to evaluate the efficiency of the soil improvement. The performance of cone penetration test (CPT) and standard penetration test (SPT) are most commonly used to field verification in current research objectives. Additionally, modulus load tests were performed in most IRAP projects to confirm the design approach.

In testing procedures, it is important to recognize the differences between the improved ground and elsewhere. The testing of soils reinforced by lateral reinforced pier must recognize the different response of the ground when testing granular in comparison to predominantly cohesive soils (Slocombe and Moseley 1991). Table 1 provides their suggestions on how useful certain commonly performed methods are for testing treated soils. Recently, new equipment is used to investigate the soil improvement, such as PMTs and K_o stepped blade tests (Pitt and White 2003). Full-scale load tests on isolated pier and pier groups were conducted to verify the DAP bearing capacity and settlement.

Cone Penetration Test (CPT)

The cone penetration test (CPT) has been widely used to evaluate the ground soil profile because it is simple, quick and economical test. The data obtained from CPT sounding can be used to determine the soil parameters in certain level reliable. Figure 9 illustrates a typical piezocone penetrometer apparatus. Three main measurements are cone tip resistance (q_c), sleeve friction (f_s) and pore water pressure (u). The tip resistance (q_c) is calculated as the force (Q_c) and the project area of the cone (A_c). The sleeve friction was calculated by the net force acting on the friction sleeve divided by the surface of the sleeve (A_s). The friction ratio (R_f) is the ratio of the sleeve friction dived by the tip resistance.

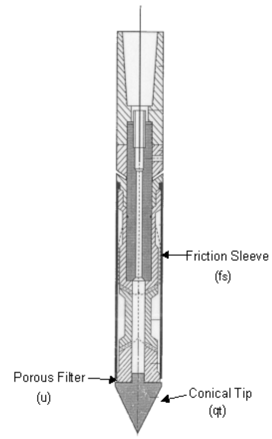


Figure 11. Schematic showing of typical cone penetrometer and location of components

Table 1: Suitability of testing lateral displacement pier (from Slocombe and Moseley 1991)

Test	Granular	Cohesive	Comments
Boreholes + SPT	***	**	Efficiency of test and recovers samples
CPT	****	**	Can be affected by lateral earth pressures generated by treatment. Best test for seismic liquefaction evaluation.
Full-Scale	****	****	Could be installed some other instrument such as inclinometer, stress cell etc., to study the mechanism.
Pressuremeter	***	*	No often used
Small Plate	*	*	Does not adequately confine pier and affected by pore water pressures
Large Plate	**	**	Better confining action

Note: * least suitable, **** most suitable

$$R_f = \frac{f_s}{q_c} \times 100\% \tag{11}$$

The empirical and theoretical correlations between the soil parameters and CPT results were used to estimate the soil parameters in this report. Table 2 illustrates the applicability of using the CPT to estimate the soil parameters.

Table 2: Suitability of testing matrix soil improvement (from Lunne et al. 1997)

Soil type	profile	u	$\emptyset'$	S_u	I_D	M_{vu}	C_v	k	G_o	σ_h	OCR	σ-ε
A	A	A	B	B	A/B	B	A/B	B	B	B/C	B	C

Note: Applicability: A = high; B = moderate; C = low.

Soil parameter define: u = in situ static pore water pressure, $\emptyset'$ = effective internal friction angle, S_u = undrained shear strength, mv = constrain modulus, C_v= coefficient of consolidation, k = coefficient of permeability, G_0= shear modulus at small strains, σ_h = horizontal stress, OCR = over consolidation ratio, σ-ε = stress-strain relationship, I_D = density index

Soil Classification from CPT

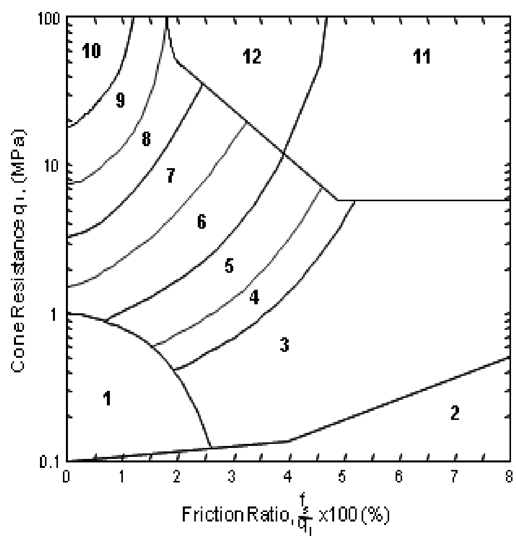
It is observed that different types of soils exhibit distinctive responses during the cone penetration, which make it possible to classify the soils based on their responses. Many researchers characterized that sandy soils were high cone resistance and low friction ratio, soft clays are low cone resistance and high friction ratio (Douglas et al 1985 and Robertson 1990). Roberson and Campanella (1988) summarized the soil behavior types based on the tip resistance (q_c) values and friction ratio (R_f) (Figure 12). Recently, Lunne (1992) noted that the soil classification correlated with pore water pressure.

To conduct a better method to estimate the soil behavior types, Jefferies and Davies (1991) suggested the following conversions to provide a conventional method to classify the soil behavior types from CPT. They introduced the soil behavior type index, I_c, which was defined as follows:

$$I_c = ((3.47 - logQ_t)^2 + \ (logF_r + 1.22)^2)^{0.5} \quad (12)$$

Where, $Q_t = \frac{q_{t-\sigma_{vo}}}{\sigma'_{vo}}$, normalized cone resistance, q_t = corrected cone resistance, F_r= normalized friction ratio.

They tabulated the boundary of the soil behavior types and the range of the corresponding I_c values (Table 3).



1 - Sensitive, Fine Grained, 2 - Organic Material, 3 - Clay, 4 - Silty Clay to Clay, 5 - Clay Silt to Silt Clay
6 - Sandy silt to Clayed Silt, 7 - Silty Sand to Sandy Silt, 8 - Sand to Silty Sand, 9 - Sand, 10 - Gravelly Sand
11 - Very Stiff Fine Grained *, 12 - Sand to Clayey Sand *. (note: * Overconsolidated cemented)

Figure 12. Soil classification chart present by Robertson and Champanella (1997)

Table 3: I_c boundaries of soil behavior types (from Jefferies and Davies 1991)

Soil Behavior Types Index, Ic	Zone	Soil Behavior Types
$I_c < 1.31$	7	Gravel sand
$1.31 < I_c < 2.05$	6	Sands- clean sand to silty sand
$2.05 < I_c < 2.60$	5	Sand mixtures – silty sand to sandy silt
$2.60 < I_c < 2.95$	4	Silt mixtures – clayey silt to silty clay
$2.95 < I_c < 3.60$	3	Clays
$I_c > 3.60$	2	Organic soils - peats

Fines Content(FC) from CPT

Recently, CPT is commonly applied to evaluate soil fines content and particle mean grain size, D_{50}. Robertson et al (1983) studied the correlations between CPT-SPT with mean grain size, and found that increase gain size would increase the $(q_c/p_a)/N_{60}$ values. Kulhawy and Mayne (1990) found that increasing FC (particle size < 0.075 mm) will decrease the $(q_c/p_a)/N_{60}$ values. Further, Robertson and Fear (1995) reported a method to approach the fines content from CPT soundings.

$$FC(\%) = 1.75 {I_c}^{3.25} - 3.7 \tag{13}$$

Where, I_c = soil behavior types index.

The equation provided above is only a method to estimate the percentage of the fines, but it does not provide the information to classify the silt or clay types of the fines.

Relative Density from CPT

The relative density (D_r) of sand is an important engineering index property for cohesionless soil that gives the level of compaction. It is defined in the term of:

$$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})} \tag{14}$$

Where, e is the void ratio, e_{max} is the maximum void ratio and e_{min} is the minimum void ratio. Table 4 shows the different methods have been approached by researchers.

Table 4: The methods to approach the relative density by different researchers

Equations	researchers	Comments
$D_r = -98 + 66log_{10}\frac{q_c}{(\sigma_{vo}^{'})^{0.5}}$	Jamiolkowski et al (1985)	Predicted by five silica sands used under control laboratory conditions; Use to calculate in this report; Figure 13(a)
$D_r = C_2\, ln\frac{q_c}{C_0(\sigma_{vo}^{'})^{C_1}}$	Baldi et al (1985)	Based on extensive calibration testing on Ticino sand;
${D_r}^2 = \frac{q_{c1}}{305Q_c \cdot Q_{OCR} \cdot Q_A}$	Kulhawy and Mayne (1990)	Consider the interbedded deposits where the cone resistance may not have reached the full value within thin layer;
${D_r}^2 = \frac{q_{c1}}{305Q_c \cdot OCR^{0.15} \cdot Q_A} \cdot \frac{q_c}{p_a} \cdot (\frac{p_a}{\sigma_v{'}})^{0.5}$	Kulhawy and Mayne (1990)	Experiments performed on clean fine to medium silica sands;
$D_r = 100 \cdot \sqrt{\frac{(N_{1(60)}}{60}}$	Skempton (1986)	Results from SPT and quartz sands; Figure 13 (b)
$D_r = 100 \cdot \sqrt{\frac{q_{t1}}{300}}$	Marcuson and Bieganowsky (1991)	Normal consolidation for unaged uncemented sands; Figure 13 (c)
$D_r = 100 \cdot \sqrt{\frac{q_{t1}}{300 \cdot OCR^{0.2}}}$		over consolidation for unaged uncemented sands

Where, σ'_{vo}= effective vertical stress; C_0, C_1, C_2 = soil constants (Table 3.7); $q_{c1} = \frac{(\frac{q_c}{p_a})}{(\frac{\sigma'_v}{p_a})^{0.5}}$ = dimensionless normalized cone resistance; p_a= atmosphere pressure in same units as qc; Q_c= compressibility factor, 0.91 < Q_c<1.09; Q_{OCR}= over consolidation factor = 0.18 ; Q_A= aging factor; $q_{t1} = \frac{q_c}{(\sigma'_{vo})^{0.5}}$ = normalized tip resistance.

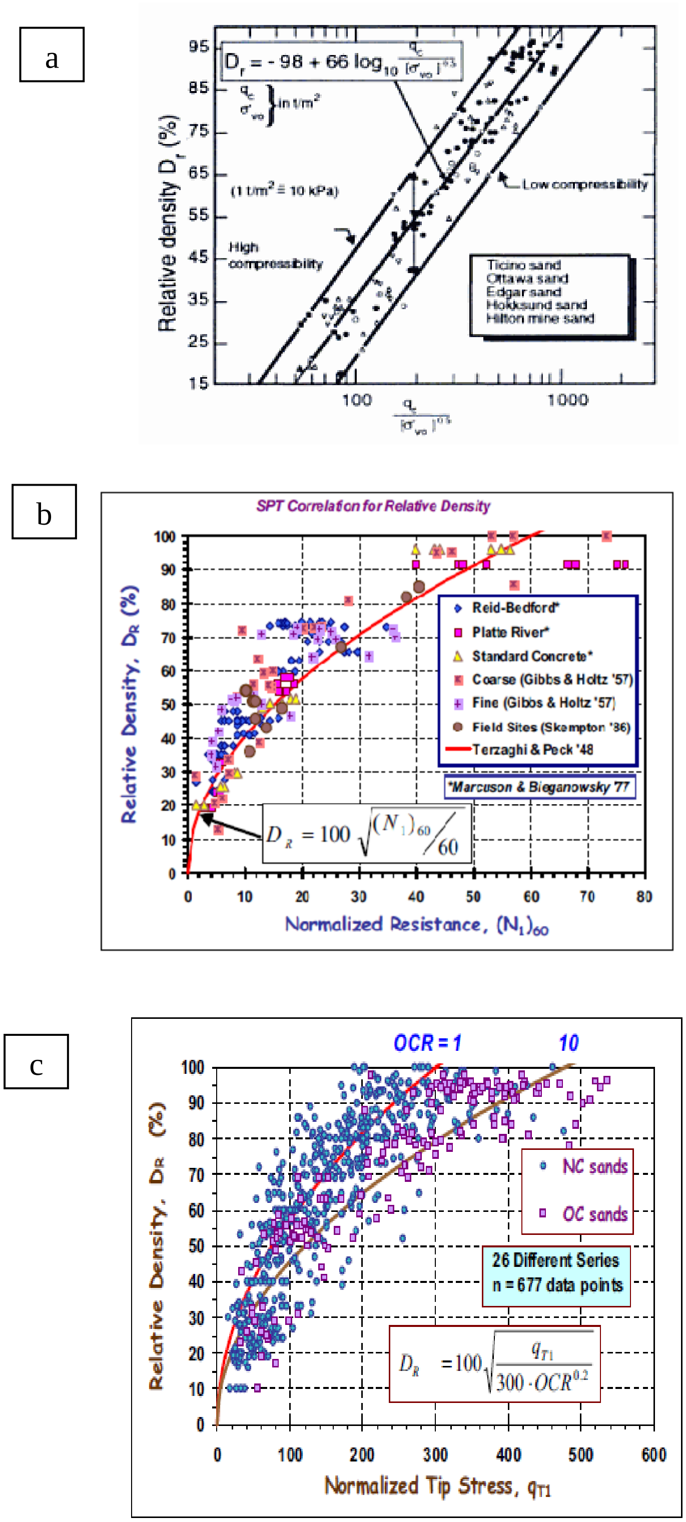


Figure 13. Several methods to approach the relative density: (a) from Jamiolkowski et al. 1985; (b) from Skempton et al. 1986 and (c) from Marcuson et al.1991

Lateral Earth Pressure from CPT

Many theories exist to evaluate the in situ lateral stress, σ'_{ho}, or coefficient of earth pressure at rest. K_o:

$$K_o = \frac{\sigma'_{ho}}{\sigma'_{vo}} \tag{15}$$

Hughes and Robertson (1985) reported that the estimated value of K_o was extremely sensitive to the measurement accuracy of the horizontal stress (σ'_{ho}), which might be result of the extreme stresss relief and disturbance that occurs as elements of soil pass the tip of the penetrometer.

Masood and Mitchell (1991) studied the relationships between Rankine passive coefficient and the sleeve friction (f_s) from CPT during penetration.

$$f_s = c_a + K_s \sigma'_v \tan\delta \tag{16}$$

Since the sleeve frictions (f_s) are different between pre- and post-installation, it is possible to evaluate the post-installation soil Rankine passive coefficient factor K_s.

$$f_{s0} = c_{a0} + K_{s0} \sigma'_{v0} \tan\delta_0 \tag{17}$$

$$f_{s1} = c_{a1} + K_{s1} \sigma'_{v1} \tan\delta_1 \tag{18}$$

Let $c_{a0}= c_{a1}$, $\sigma'_{v0}= \sigma'_{v1}$, and $\delta_0= \delta_1=\phi'/3$, (18) - (17) rewrite:

$$K_{s1} = \frac{f_{s1} - f_{s0}}{\sigma'_{v0} \tan\left(\frac{\emptyset'}{3}\right)} + K_{s0} \tag{19}$$

Where, denote 0 = pre-installation, 1 = post-installation, f_s = sleeve friction, c_a= adhesion between soil and the sleeve, , δ = angle of friction between soil and sleeve, ϕ' = effective stress friction angle of the displaced soil.

The authors noted that use of K_s for loose sand may overestimate the horizontal stress during penetration, since the K_s lead to overestimate the post-installationK_o.

Calculating soil parameters from CPT data

Design engineers calculate soil parameters by using data from CPT soundings in several equations. Table 5 summarizes the kinds of correlations, the equations used to obtain the correlations, and the researchers who proposed the equations.

Table 5: The equations for soil parameters from CPT and their references

Terms	Equations	References
CPT-SPT correlation	$N_{60} = \frac{({q_t}/{p_a})}{8.5(1 - I_c)}$	Lunne et al. (1997)
SPT- $(N1)_{60}$	$(N_1)_{60} = \frac{N_{60}}{({\sigma_v}'/{p_a})^{0.5}}$	Das (2007)
Internal friction angle ϕ'	$\phi' = \sqrt{15.4(N_1)_{60}} + 20$	Das (2007)
Pre-installation Ko	$K_0 = 1 - sin\phi'$	Lade and Lee (1976)
OCR	For sand $OCR = (K_{0(ov)}/K_{0(NC)})^{\frac{1}{1-sin\phi'}}$ For clay $OCR = (K_{0(ov)}/K_{0(NC)})^{\frac{1}{0.65}}$	Huang and Mayne (2008)
Undrainded shear strength, S_u	$S_u = \frac{q_c - \sigma_{vo}}{N_k}$	Lunne and Kleven (1981)
Young's modulus E	Depends of the soil types Typical values: $E = 2.5q_c$	Schmertmann (Fang, 1997)

Standard Penetration Tests (SPTs)

SPTs are performed both before and after installation to investigate soil improvement. SPTs are more effective in granular deposits than cohesive soil. In usual, the corrected blow count (N_{60}) is used to estimate the improvement.

It is possible to evaluate the simple correlation between the SPT and CPT on the same site which was performed both the SPTs and CPTs. The correlation of SPT N-values and CPT results of sands was shown as follow:

$$\frac{q_{c/p_a}}{N} = 4 \qquad (20)$$

Where, p_a= atmosphere pressure.

Modulus Load Tests

Modulus load test performs to safety confirmation of the DAP. The modulus load test measures the top (S_t) and bottom deformation (S_b) and records the applied stress (σ_t). Stiffness is defined as the ratio of applied stress divided by the amount of top deformation (σ_t/S_t). Figure 10 shows a schematic of a typical modulus load test. Normally, one test takes about several hours. Any record in the settlements takes about every 20 minutes.

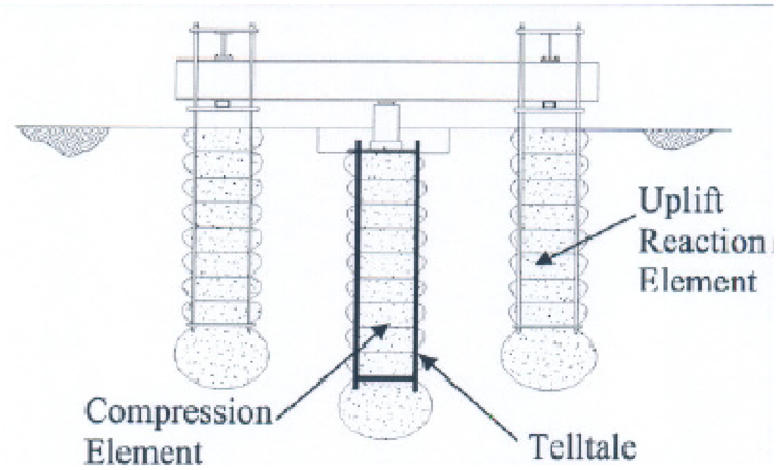


Figure 14. Schematic of the modulus load test (from Wissmann 1991)

The results of full scale testing of the aggregate pier using load cells are shown in Figure 15. The results show that the vertical stress decreases to less than 20% on the RAP element at a depth of about 3 diameters. That means the most vertical applied stress is distributed to the matrix soil at the depth of about 3 diameters. The ability of stress distribution in matrix soil is strongly related with the characteristics of the matrix soil in this range of depth.

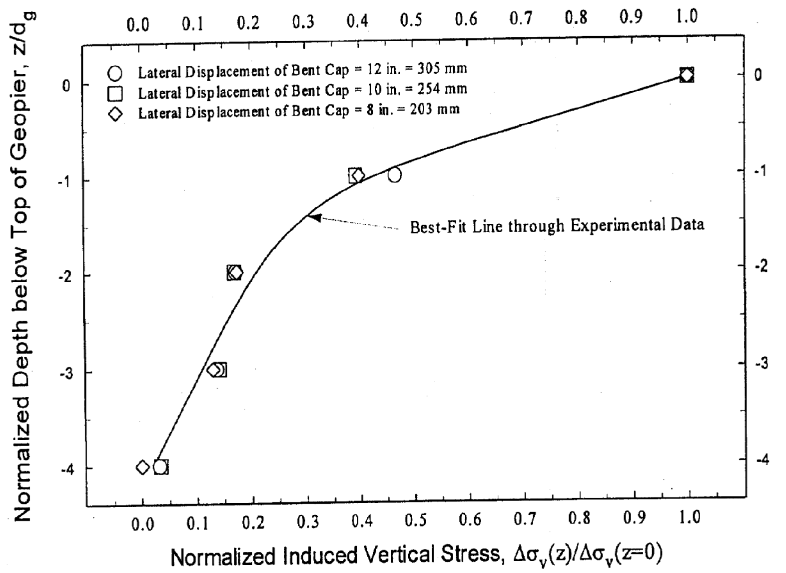


Figure 15. Distribution of compressive vertical stress within middle rammed aggregate pier (from Fox and Cowell, 1998)

CHAPTER 3: CASE HISTORIES

The Geopier Foundation Company ™ (GFC) provided case study data from 16 sites where displacement aggregate piers were installed. The data for each case that is presented in this chapter were extracted from the data provided by GFC and prepared so they were consistent between the cases. Each case history includes a description of the project, the subsurface conditions at each site, the pier system installed at each site, a description of the in situ tests that resulted in the study data, and the results and preliminary analysis of the data from these tests. This chapter has two main parts, the case histories and the all of the data from the case studies. Table 11 to Table 13 summarizes the brief information of all cases.

INDIVIDUAL CASE STUDIES

Salinas, CA

Project Description

The project involved the construction of cinemas in Salinas, CA. A DAP system was selected to support the foundation footing with a bearing capacity of 18.33 ksf (0.88MPa) and a foundation up-lift resistance of 40 kips (178 kN).

Subsurface Conditions

The results from two pre-installation SPTs showed that the matrix soil strata at the site consisted of soft to firm clay and firm silt and sand fill to a depth of around 17 ft (5.2 m), underlain by soft to firm silt to a depth of about 22 ft (6.7 m), and then underlain by soft to medium dense sand (Table 20b).

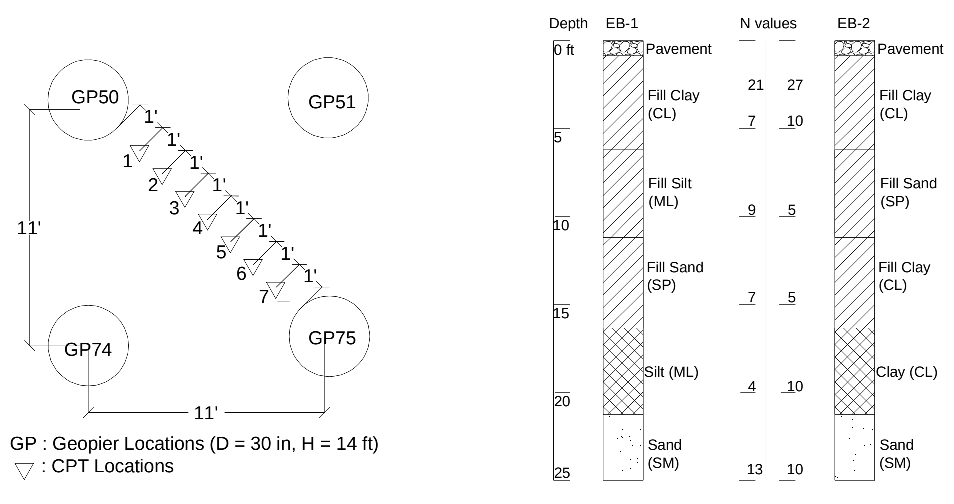
The laboratory tests show the fines content (passing #200 sieve) varied from 17% to 34% in the sand layers. The groundwater table depth was around 29 ft (8.8 m).

Pier System

Geopiers were installed on 11 ft (3.4 m) centers. The Geopier diameter was approximately 30 inches (0.76 m) and the height was approximately 14 ft (4.27 m).

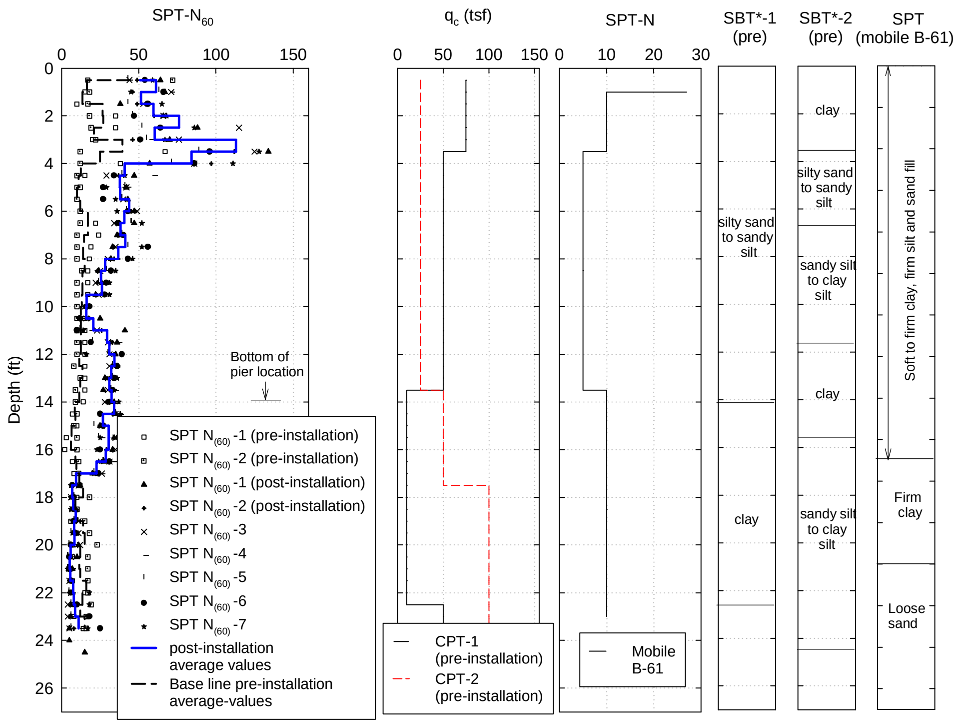
Tests and Results

SPTs were performed before installation, and CPTs and modulus load tests were conducted after installation. Seven CPTs were performed within one pier group; the CPT locations are shown in Figure 16a, and CPT results are show in Figure 17. The average N_{60} value (calculated from CPT data) increased from 15 to 40 after Geopier installation. The soil seems not improved at a depth of about l0 ft (3 m). A modulus load test and and uplift test were also performed. Total settlement and telltale settlement at design stress was about 0.2 in. (0.5 cm) and 0.02 in. (0.05 cm), respectively. The calculated stiffness modulus was 639 pci (173 kN/cm^3) at design stress. The pier moved up about 0.16 in (0.41 cm) at design uplift load of 40 kips (178 kN). These results are shown in Figure 18



(a) (b)

Figure 16. Salinas, CA: The plan layout of CPT locations (a) within a pier group and (b) subsoil profiles from SPT



Note: SPT $N_{(60)}$ values correlated from CPT data(not electronic data provided). Pre-installation CPT data and soil behavior types profiles are estimated from hard copy. Boring log (moblie B-61) profile located at the vicinity CPT locations.

Figure 17. Salinas, CA: SPT-N_{60} values calculated from CPT data, pre-installation q_c, SPT-N, and soil type profiles

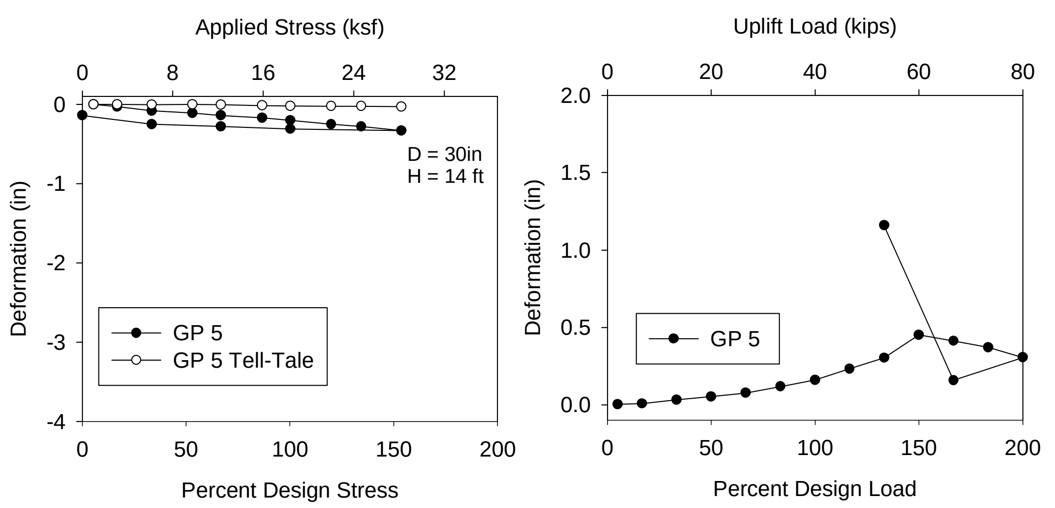


Figure 18. Salinas, CA: Applied stress and deformation results from the modulus load test and uplift test

Minneapolis, MN

Project Description

The project involved the construction of a 12,500 ft^2 (1161 m^2) single story, slab on grade facility. The project consisted of IRAP elements that were installed beneath column and wall footings. The column loads varied from 5 to 80 kips (22 kN to 356 kN). Three pier types—IRAPs, TMRAPs, and Geopiers—were installed for research purposes.

Subsurface Conditions

The soil stratum consisted of silty to clayey sand with organics to a depth of about 2.5 ft (0.76 m), underlain by very loose to loose, poorly graded sand to depth of around 15 ft (4.5 m), and then underlain by medium dense sand. The SPT N-values in the sand ranged from 2 to 16. No groundwater was encountered. Figure 20 shows the CPT results and SPT soil profiles.

Pier System

IRAPs were designed to support the columns and walls for the entire project. Three types of DAPs—IRAPs, TMRAPs, and Geopiers—were installed for research purposes. The design capacity of the DAPs was 40 kips (178 kN). The IRAPs and Geopiers were installed by the conventional methods. After installation, one of the three test TMRAPs did not have additional compation. The top of the second pier was compacted for 15 seconds using a standard RAP hammer. For the third pier, the final two lifts were compacted using a modified (non-tapered) mandrel.

Tests and Results

Only SPTs were performed before the piers were installed. CPTs and modulus load tests were performed after the piers were installed. The dimensions, types, and locations of the piers and locations of the CPTs are shown in Figure 19.

Modulus load tests were performed on three TMRAPs (TP-1, TP-2, and TP-3) where three different top compaction methods were used; modulus load tests were also performed on one IRAP and one Geopier. The modulus load tests were performed one day after the installation of TMRAPs and two days after the installation of the IRAP and the Geopier. The tell-tale of TP-1 was damaged during the modulus load test.

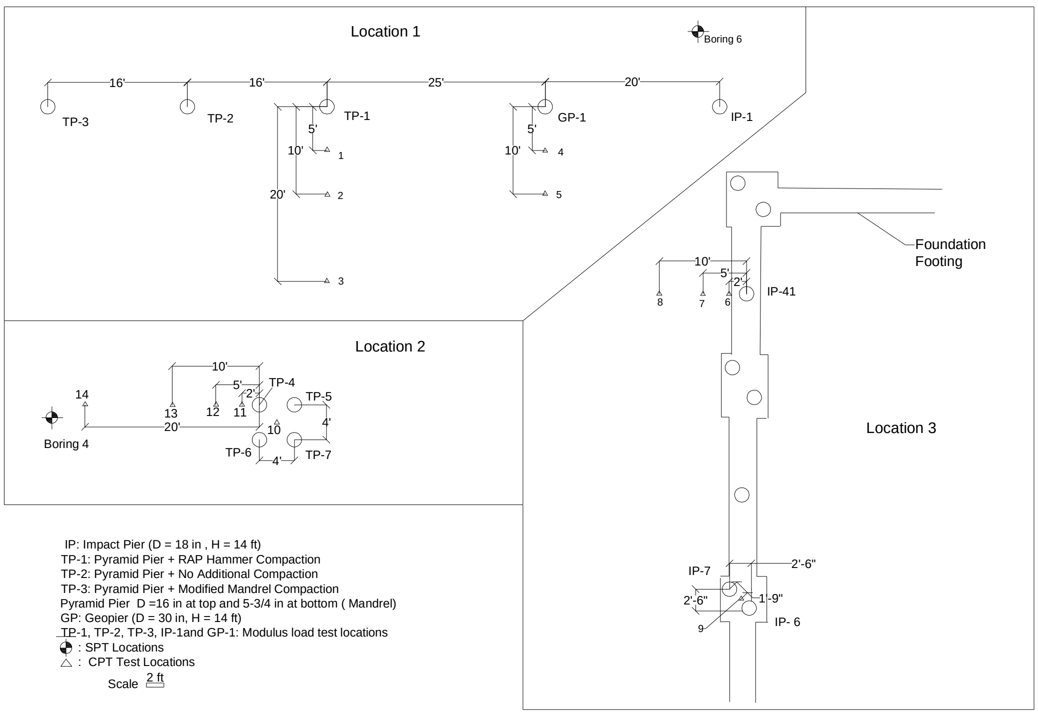
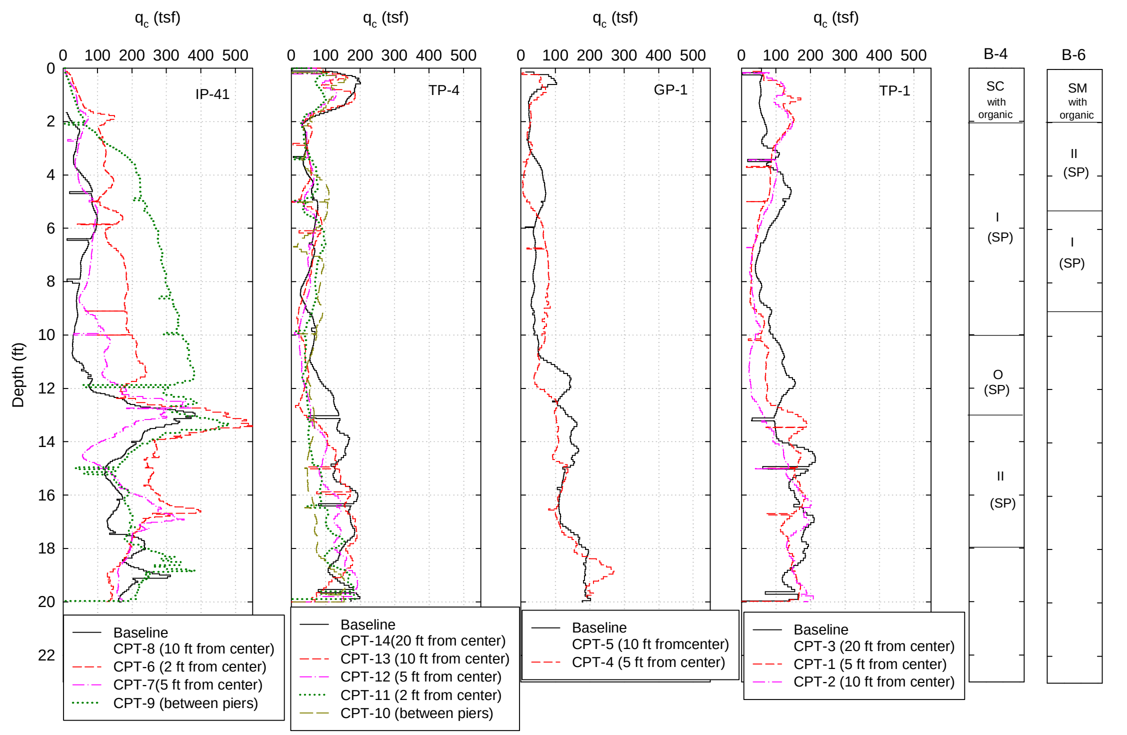


Figure 19. Minneapolis, MN: IRAP, TMRAP, and Geopier layout and CPT and SPT locations

The CPT results indicated soil improvement within 5 ft (1.6 m) of the center of the piers (Figure 20). The mean CPT tip resistance values of the matrix soil within the pier group were almost 1.5 times greater than those of the matrix soil outside group. The modulus load test results are shown in Figure 21 and summarized on Table 6. Since the design stress values of the DAPs were unknown, the assumed values are given on Table 7. The CPT tip resistance results indicated that the additional RAP hammer compaction and modified mandrel compaction did not significantly increase the stiffness of TMRAPs at the assumed design stress levels. The tell-tale deflection of the TP-2 was 0.1 inches (0.25 cm) while the top deflection was 0.5 inches (1.25 cm) under the assumed design stress. The stiffness of the IRAP was the highest, whereas the Geopier stiffness was the lowest.



O: Very loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29), III: Dense sand (N = 30 to 49), IV: Very dense sand (N: greater than 50)

Figure 20. Minneapolis, MN: CPT tip resistance profiles and SPT soil profiles

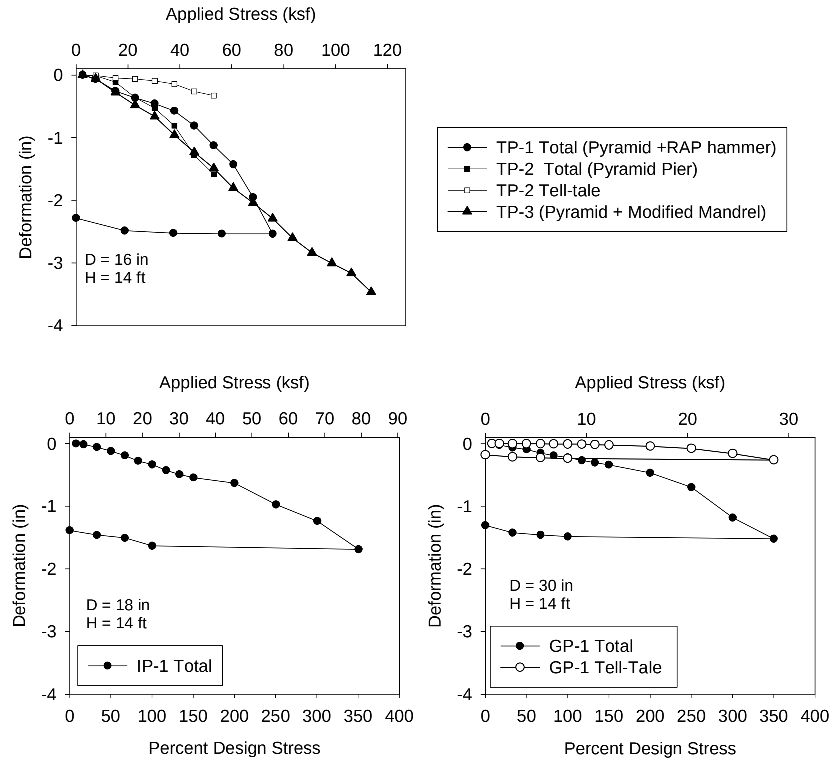


Figure 21. Minneapolis, MN: Applied stress and deformation results from modulus load tests

Table 6: Summary results from modulus load tests

Pier ID	Days after pier installation	Diameter, D (in)	Height, H (ft)	Design Stress (ksf)	k_1 (pci)
TP-1	1	16	14	22.7 (assume)	433
TP-2	1	16	14	22.7 (assume)	441
TP-3	1	16	14	22.7 (assume)	330
IP-1	2	18	14	22.65	469
GP-1	2	30	14	8.15	255

Lacrosse, WI

Project Description

The project consisted of a six-story building located on the east side of the Mississippi River in La Crosse, WI. The design load for the columns was between 330 kips (1468 kN) and 880 kips (3914 kN). The allowable soil bearing pressure for the footings was designed as 6 ksf (0.29 MPa), and the design stress was 22 ksf (1.05 MPa) for the IRAPs elements.

Subsurface Conditions

The soil stratum consisted of loose sand fill with construction debris (concrete and gravel fragments) to depths of about 10 ft (3 m) to 16 ft (5 m), underlain by loose to medium dense sand. The SPT N-values ranged from 2 to 12 between the depths of 10 ft to 15 ft (Figure 23). The groundwater table was at approximately 10 ft (3 m) to 15 ft (4.5 m).

Pier System

The IRAPs were designed with a capacity of 60 kips (267 kN), an allowable pressure of 6 ksf (0.29 MPa), and with a stiffness modulus of 150 pci (554 MN/m^3). The pier penetration depth was 30 ft (9 m), and the piers were spaced on 3 ft (1 m) centers within the footing. And the footings were supported by up to seventeen IRAPs.

Aggregate with nominal diameter of 2 to 2.5 in. (5 to 6.7 cm) was too large to flow freely from the bottom of the mandrel. Aggregate with a nominal diameter of 1 in. (2.5 cm) had an acceptable withdrawal rate of 0.2 ft/s.

Tests and Results

SPTs and CPTs were performed before and after the IRAP installation. SPT and CPT and were conducted at the vicinity piers group. The design parameters of the IRAPs, footing size, and SPT and CPT locations are shown in Figure 22.

A modulus load test was conducted on a group of five piers after the CPTs and four days after the IRAPs were installed. Another modulus load test was conducted on a single pier near the pier group.

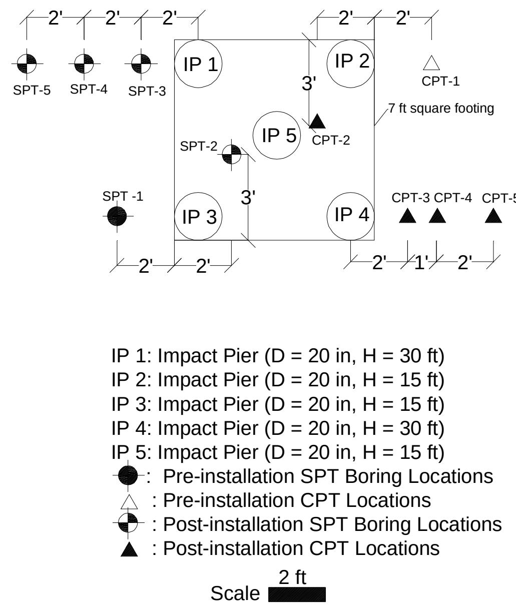
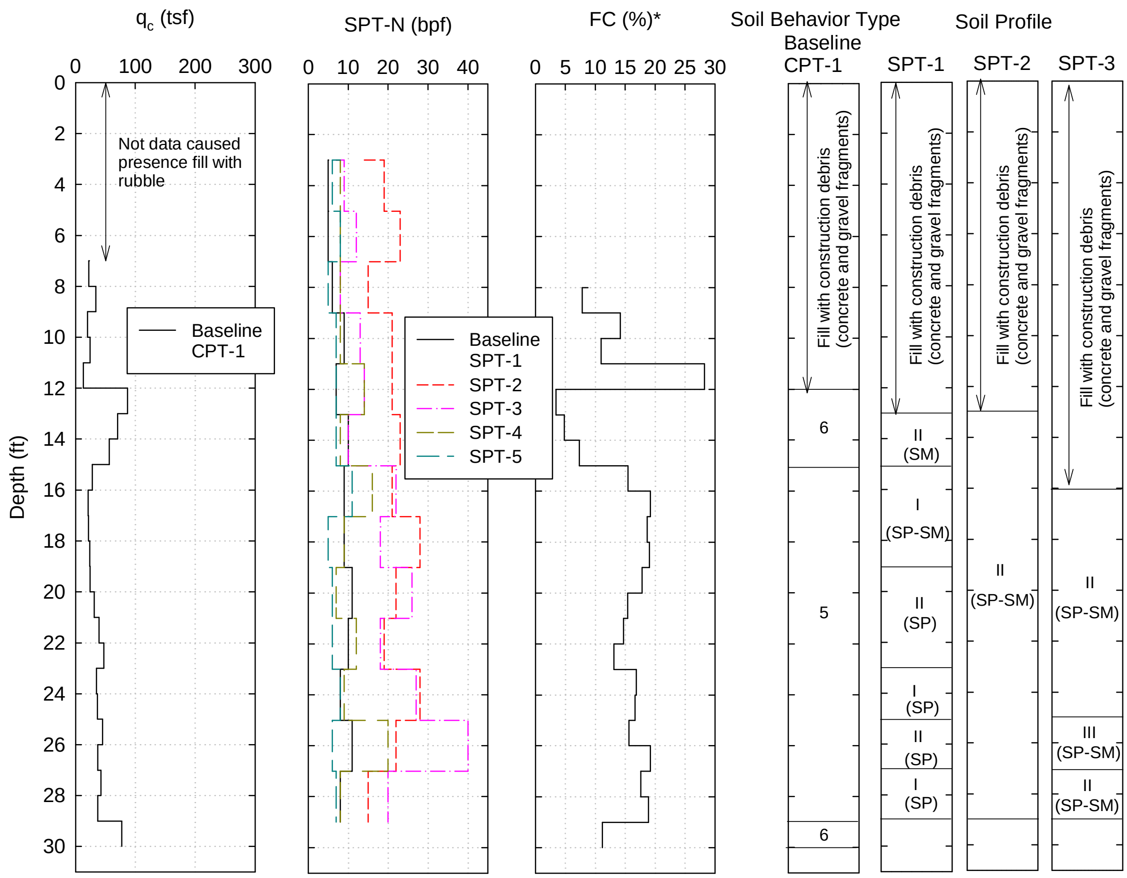


Figure 22. Lacrosse, WI: The plan layout of the 7 ft x 7 ft (2.1 m x 2.1 m) footing and locations of pre-and post-installation SPTs and CPTs

The pre-installation SPT profiles and the post-installation CPT profiles are shown in Figure 23 and Figure 24, respectively. The average SPT N-values increased from 8 to 17 at a distance of 2 ft away from the pier group after the piers were installed. The average SPT N-value within the pier group was 21. In CPT-2, the tip resistance reached refusal. The average tip resistance values increased from 36 tsf (1.7 MPa) to 88 tsf (4.2 MPa) and 57 tsf (2.7 MPa) at distances of 2 ft (0.6 m) and 5 ft (1.5 m) away from the group, respectively.

The single pier and pier group modulus load tests were performed to confirm acceptable levels of performance. Figure 25 shows the results of both the single pier and full scale footing load test results. Based on this, it is indicated that the stiffness modulus of 609 pci

(2247 MN/m^3) was obtained at the design stress level of 22.2 ksf (1.1MPa). The full scale footing load tests results indicated the deflection of the footing was 0.2 inches (0.5 cm) at the design stress level of 6 ksf.



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)
O: Very loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29), III: Dense sand (N = 30 to 49), IV: Very dense sand (N: greater than 50)

Figure 23. Lacrosse, WI: SPT and calculated fines content profiles

The CPTs showed that the fines content in this site was approximately 5% and 25%. Tip resistance values increased at a depth of 13 ft where the transit layers of fill material and native soil were found. Figure 26 indicates that the lower fines content has larger tip resistance increment.

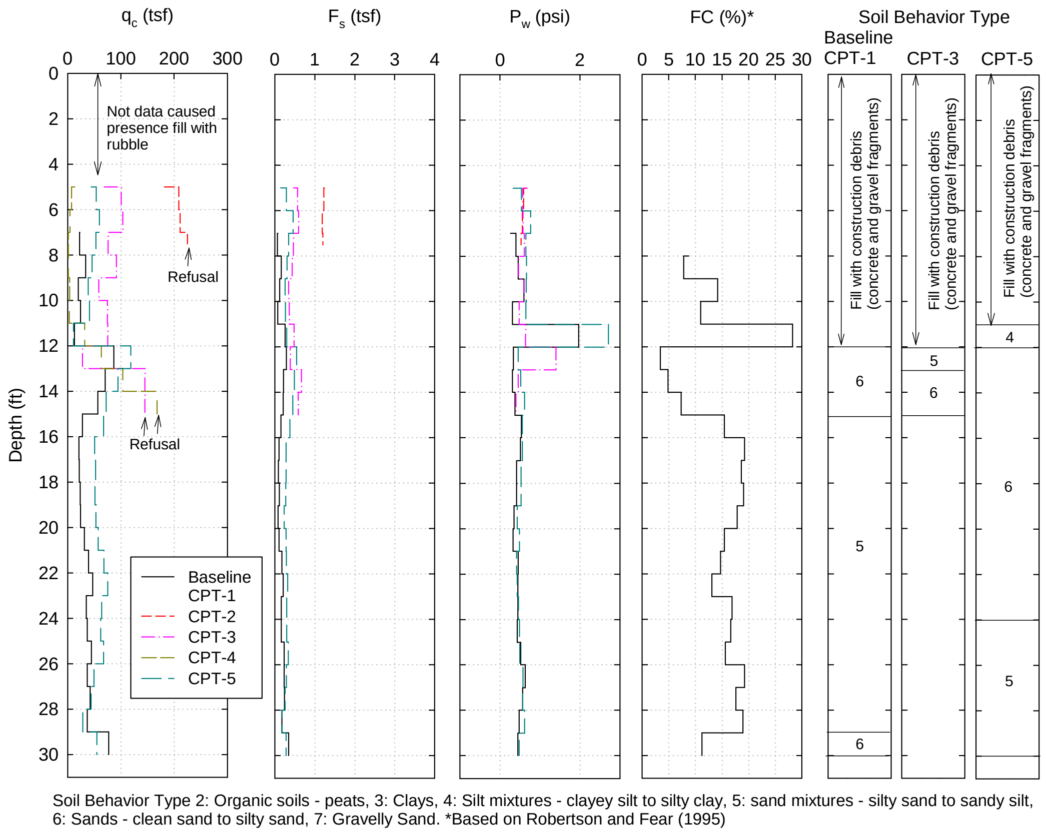


Figure 24. Lacrosse, WI: CPT baseline profiles and SPT profiles

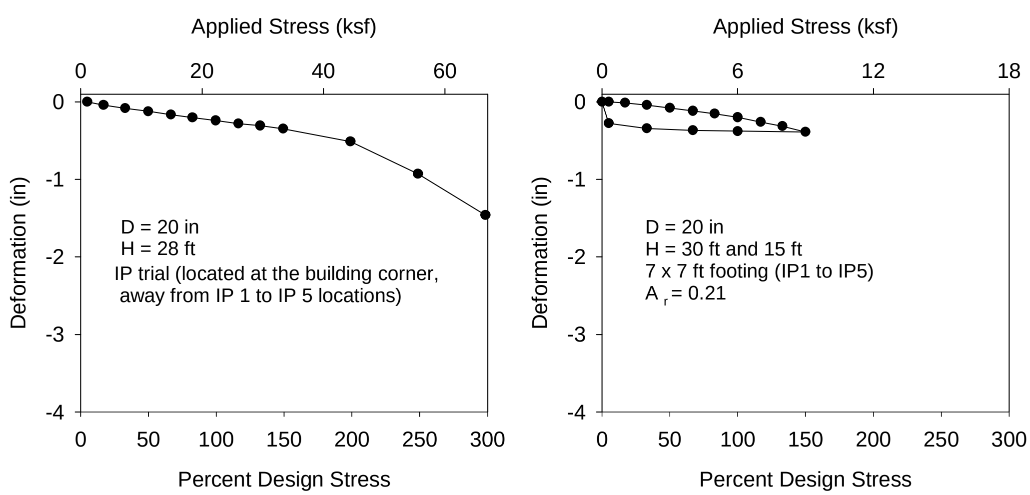


Figure 25. Lacrosse, WI: Modulus load test results for an individual pier and a pier group

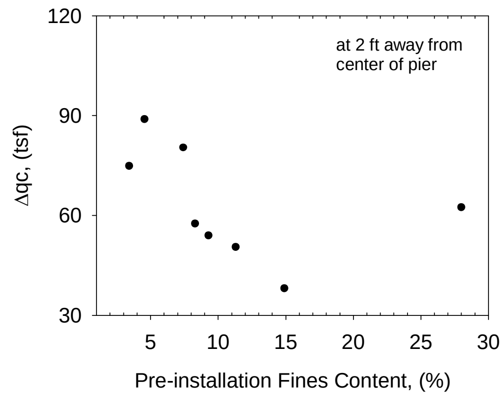


Figure 26. Lacrosse, WI: Fines content from CPT compared with increasing tip resistance

Manalapan, NJ

Project Description

The project was designed for highway embankment subsoil improvement in Manalapan, NJ. The improvement area was about 250 ft (76.2 m) by 60 ft (18.3 m). IRAP system was designed for the soil improvement and supported the keystone retaining wall and concrete retaining wall. Approximately around total 800 IRAPs were installed in this site. Sheetpile was constructed after the DAP installation.

Subsoil Conditions

The SPT soil exploration indicated that the subsoil conditions were consisted of very loose sand trace silt to silty sand to a depth of about 15 ft (4.5 m), underlain by loose to medium dense sand. The water table was varied from 1 ft to 3 ft (0.3 m to 0.9 m) from test locations surface. The soil exploration profiles are shown in Figure 29.

Pier System

Pyramid RAPs and IRAPs were designed to install in this site. The dimensions of the pyramid mandrel were about 24 inches (0.65 m) diameter at the top and 8 inches diameter at the bottom. The design penetration depth of the pier was 14 ft (4.3 m). The spacing of the piers was around 8 ft (2.4 m) on centers. The pier layout was considered as parallelogram. The two construction methods, which were pyramid hammer compaction in upper 5 ft (1.5 m) and blunt tip compaction in upper 4 ft (1.2 m), were conducted for research purpose.

Tests and Results

CPT, SPT, and modulus load tests were performed at the vicinity of the project locations. SPT was performed before the IRAP installation. CPT was performed after the IRAP installation. The locations of the tests are shown in Figure 28. The CPT results indicated that the soil was not improved much by IRAP (Figure 29)

The modulus load tests results pointed out the settlement as 2 inches at the design stress level of 18 ksf (0.86 MPa) and 15 ksf (0.72 MPa) for the pyramid compaction and the blunt tip compaction, respectively (Figure 30). The pier stiffness modulus was 59 pci (217 MN/m^3) and 51 pci (188 MN/m^3) at the design stress level for the pyramid compaction method and blunt tip compaction method, respectively.

The test results indicated that the initial very loose sandy ground did not have too much improvement by IRAP.

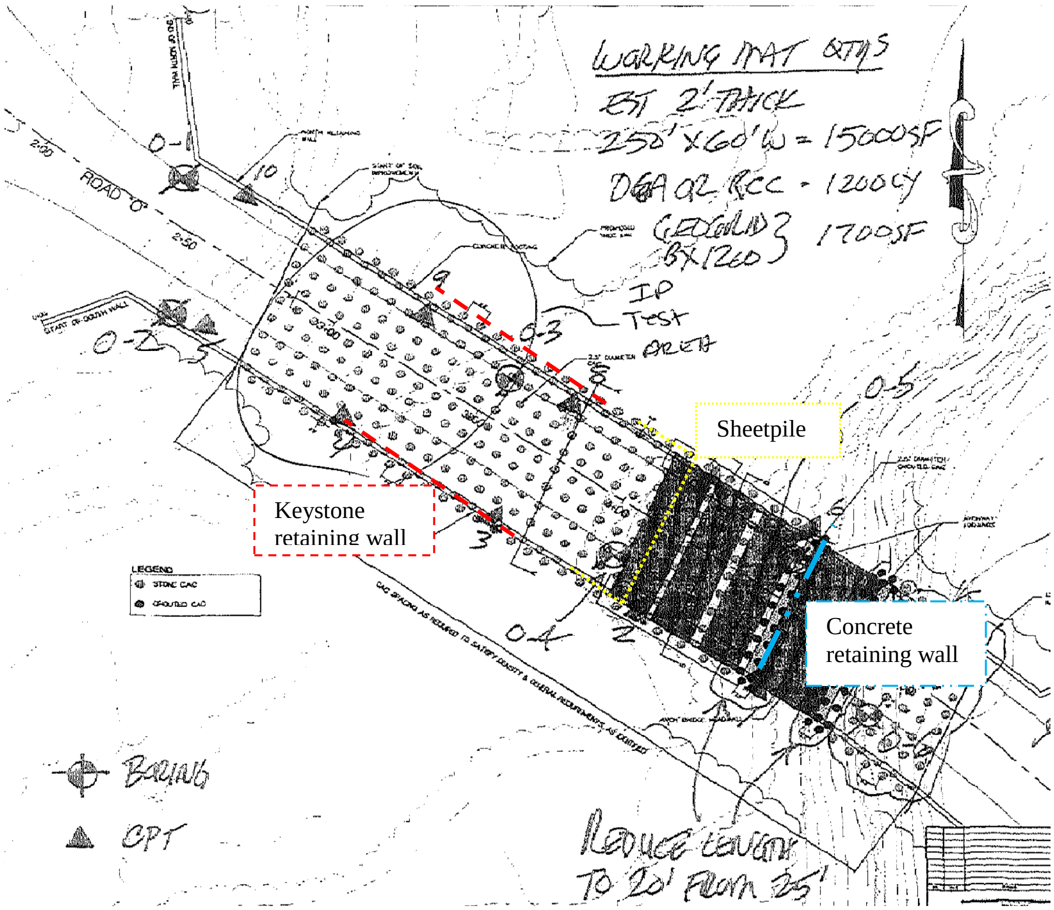


Figure 27. Lacrosse, NJ: Project plan layout (French Parrello Company)

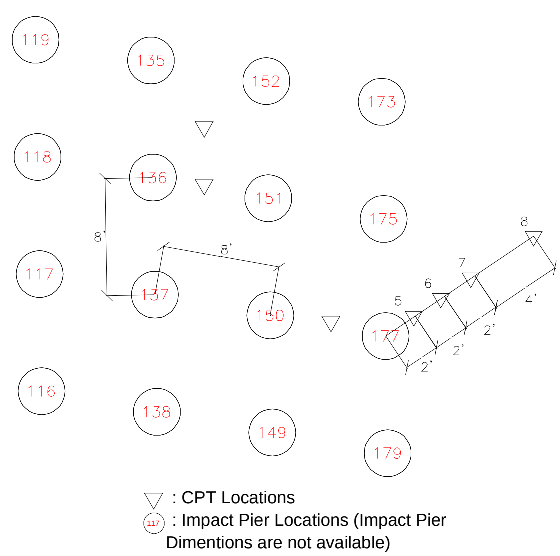
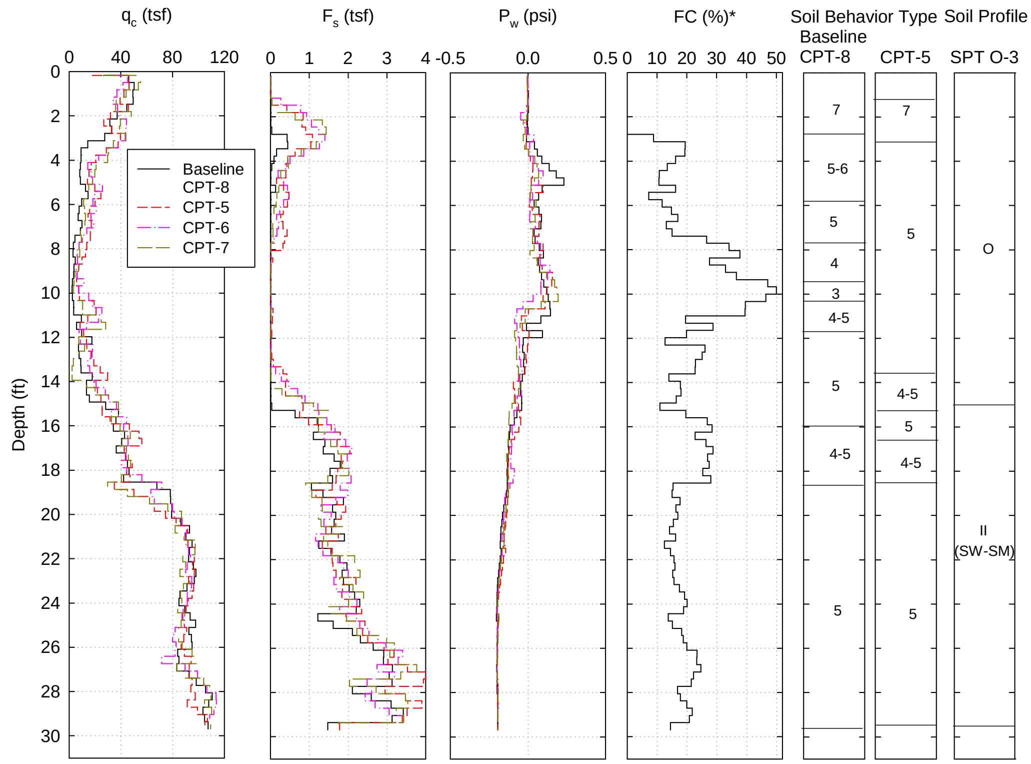


Figure 28. Manalapan, NJ: Plan layout of the CPT locations



Soil Behavior 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)
O: Very loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29), III: Dense sand (N = 30 to 49), IV: Very dense sand (N: greater than 50)

Figure 29. Manalapan, NJ: The CPT profiles for tests performed outside pier groups (CPT 5 to 8) and pre-installation SPT profile (SPT located in the vicinity of the CPT locations)

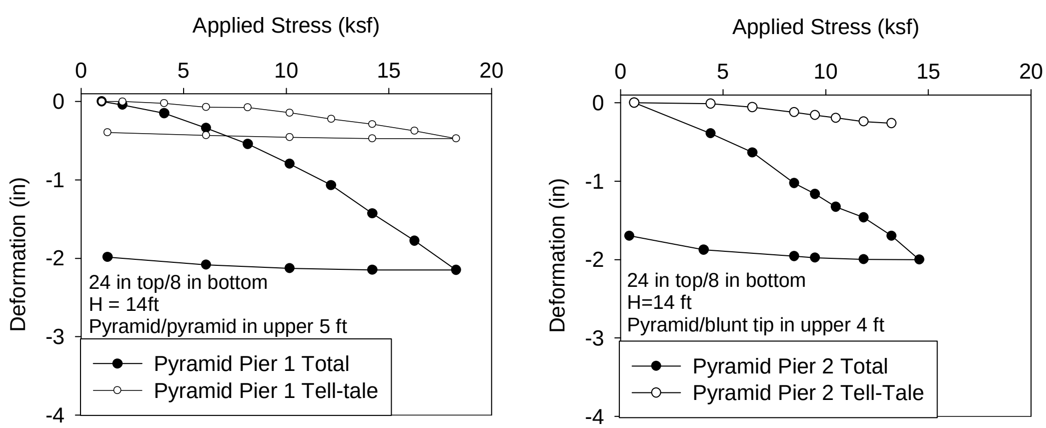


Figure 30. Manalapan, NJ: Applied stress and deformation results from modulus load tests

Reynolds, IN

Project Description

The project consisted of providing TMRAP to support a 105 ft (32 m) diameter grain storage bin with an 8.25 ft (2.51 m) wide concrete ring wall footing. The design maximum slab pressure was 5 ksf (0.24 MPa). The design upper zone settlement and total settlement was 2 inches (5 cm) and 2.5 inches (6.4 cm), respectively. The in situ tests locations and proposed footing locations are shown in Figure 31.

Subsoil Conditions

The subsoil conditions are shown in Figure 32, and generally were consisted of 5 ft (1.5 m) medium stiff to stiff clay fill overlying loose to medium dense sand to a depth of 17 ft (5.2 m), underlain by the stiff to hard glacial till. The groundwater table was about 5 ft (1.5 m) from the ground surface.

Pier System

The TMRAP was designed and selected to facilitate the construction of the displacement RAPs for the project site. The test piers were constructed at 4.5 ft (1.4 m) center to center distance. The dimensions of the RAPs were 24 inches (0.6 m) top diameter and 16.6 ft (5 m) height.

Tests and Results

The in situ tests were consisted of SPT and modulus load tests. SPT were performed before and after the DAP installation. The in situ test locations and pier layout are shown in Figure 31. The modulus load tests were performed on the piers within the confining piers (TP-1) and without confining piers (TP-2). SPT-5 and SPT-6 were performed on the matrix soil within the piers group. SPT-8 and SPT-9 were performed outside the pier group.

The SPT results indicated that soil was significantly improved to a depth of about 10 ft (3 m) (Figure 32). The average SPT value was increased from 12 to 22 in piers group for upper 10 ft (3 m). Soil was not improved below the depth of about 10 ft (3 m). Load test results indicated that the pier with confining piers did not significantly increase the stiffness and reduce the settlement. Tell-tale settlement of TP-1 show large settlement at the pier bottom during load test. But the tell-tale settlement of TP-2 was zero. The tell-tale may be damaged during the construction.

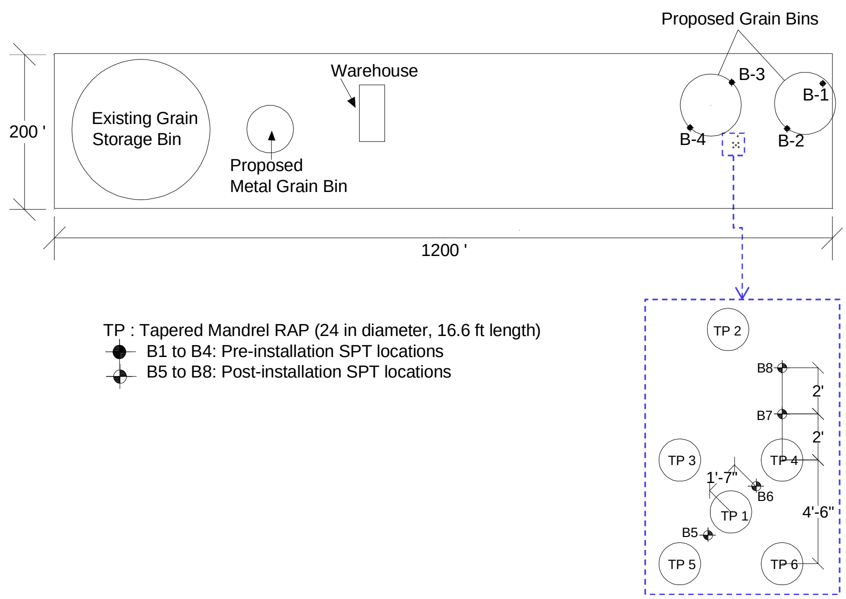


Figure 31. Reynolds, IN: Pier and SPT locations plan layout

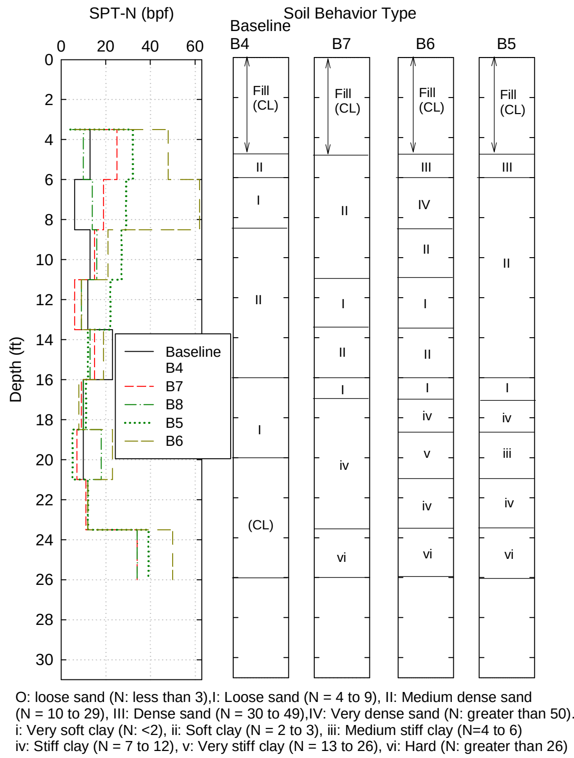


Figure 32. Reynolds, IN: SPT profiles for tests performed outside pier groups (B-7 and B-8) and within pier groups (B-5 and B-6)

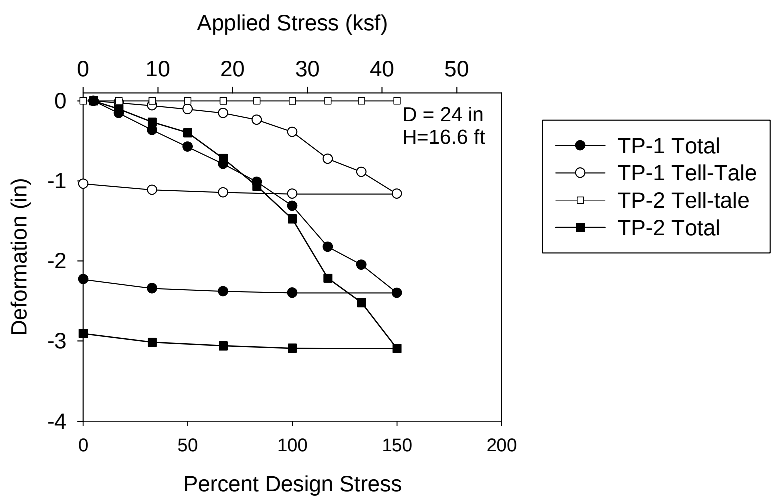


Figure 33. Reynolds, IN: Applied stress and deformation results from modulus load tests

Tampa, FL

Project Description

The project was to design and support a gasoline tank located in Tampa, FL. The diameter was designed about 120 ft (36.4 m). The site presented a tight work environment.

Subsoil Conditions

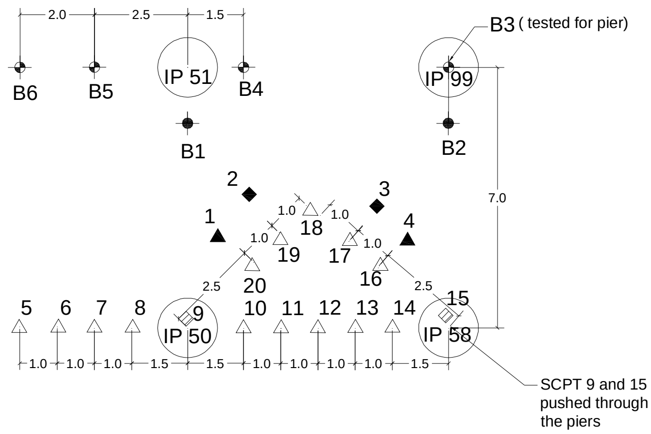
The subsoil profiles consisted of sand stone fill to a depth of 5 ft (1.5 m), underlain by sand with clay and shell fragments to a depth of 15 ft (4.6 m), underlain by silty to clayey sand. Groundwater was encountered at a depth of about 3 ft (1 m). The soil profiles are shown in Figure 35.

Pier System

IRAPs with 20 in. top diameter and 20 ft height were designed to support the gasoline tank footing foundations and matrix soil improvement. The IRAP elements were designed between 5 ft to 7 ft (1.5 to 2.1 m) center to center distance. It was reported that the IRAP work continued effectively in the ground conditions without creating excess water or spoils that would require disposal during construction.

Tests and Results

Both CPTs and SPTs were conducted before and after the IRAP installations to investigate the matrix soil improvement. SCPTs (Seismic CPT) were performed during the pre-installation and post-installation in the matrix soil. The additional SCPTs were performed through the IRAP elements to investigate the DAP stiffness and strength. The locations of CPTs, SCPTs and SPTs are shown in Figure 34. Load modulus load test was performed in the project area, and identified the nearest boring (T-08-01) is shown in the Figure 35.



IP : Impact pier (D = 20 in, H = 20 ft)
:Pre-installation SPT locations
:Post-installation SPT locations
:Pre-installation CPT locations
:Post-installation CPT locations
:Pre-installation Seismic CPT locations
:Post-installation Seismic CPT locations

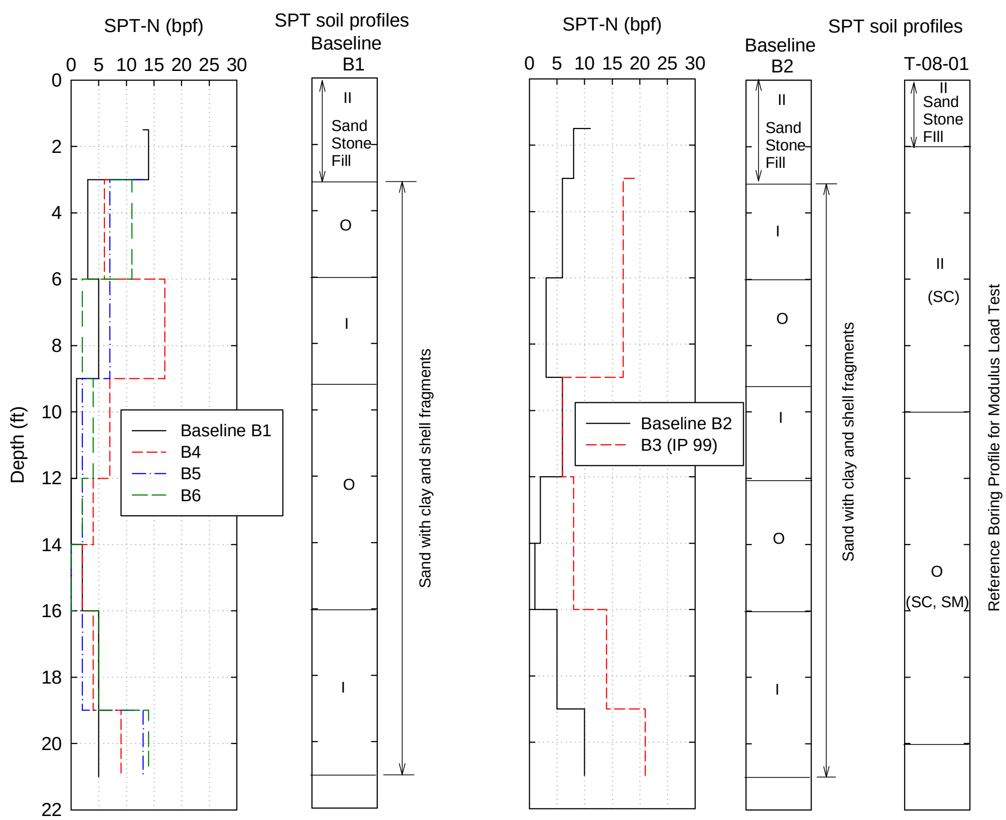
Figure 34. Tampa, FL: Pier and CPT locations plan layout

The CPT results indicated that the ground was improved in the sand stone fill layer which was indentified upper about 5 ft (1.5 m) from the ground surface (Figure 36). Matrix soil was not significantly improved for the very loose to loose clayey sand layer. The SPT results were similar to the CPT results (Figure 35).

The results of the CPT and SPT which were performed through the IRAP elements indicated the pier stiffness resistance from CPT that was highly related to the stiffness of the matrix soil. The CPT tip resistance through the pier elements was approximately 2 to 3 times greater than through the loose to medium dense clayey sand soil layers. However, the tip resistance of the pier element tended to equalize to the matrix soil for very loose clayey sandy layers.

The modulus load test results were obtained at approximately 0.35 inches (0.89 cm) from the top deformation of the pier at the design stress level of 18 ksf (0.86 MPa) (Figure 37), which indicated that the stiffness modulus of the pier was 374 pci (1380 MN/m^3). No tell-tale settlement during loading may be due to:

- Budging deformation domination during the load test
- The telltale was damaged during the construction



O: Very loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29)

Figure 35. Tampa, FL: SPT N-values and soil exploration profiles

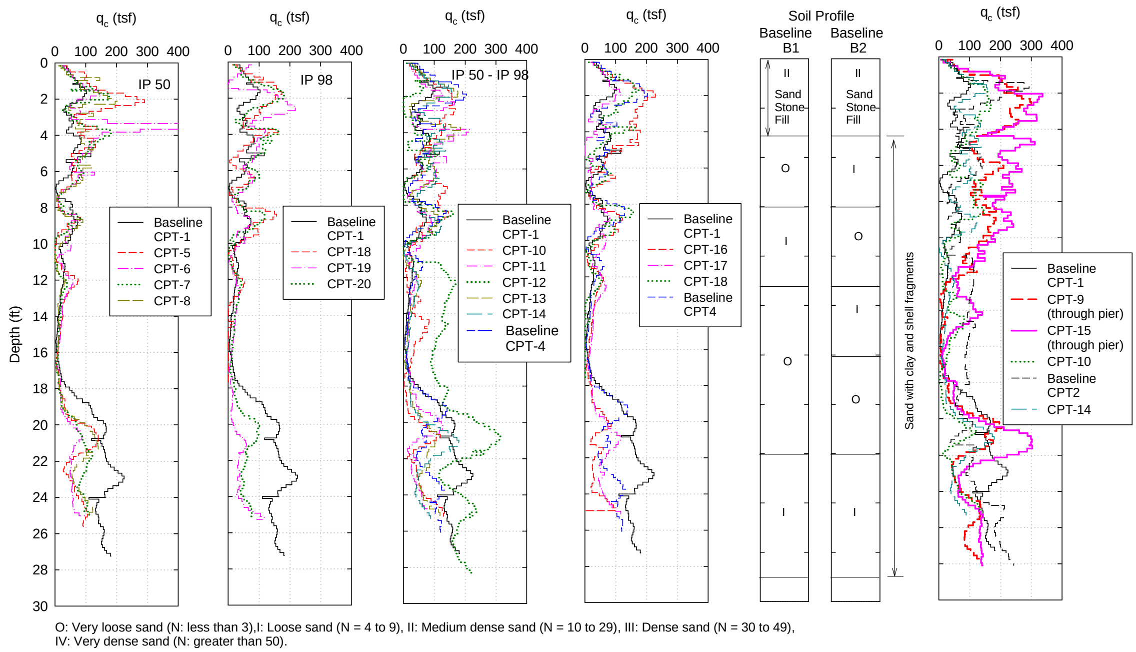


Figure 36. Tampa, FL: SPT N-values and soil exploration profiles

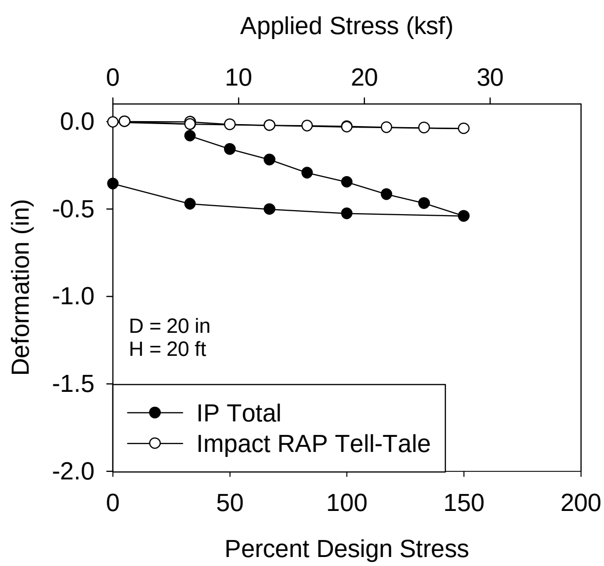


Figure 37. Tampa, FL: Applied stress and deformation results from modulus load tests

(IRAP located in the project area, identified nearest boring profile: T-08-01)

Seattle, WA

Project Description

The project was designed the construction of a 64,000 ft^2 (5946 m^2) high bay warehouse facility. The column load was designed up to 250 kips (1112 kN) and floor loads up to 1000 psf (47.9 kPa). Site liquefaction mitigation was required. The total and differential settlement was required less than 1 in. (2.5 cm) and 0.5 in. (1.7 cm), respectively.

Subsoil Conditions

Subsurface conditions were consisted of 10 ft (3 m) to 12 ft (3.6 m) of existing sand fill over soft clay that extended to approximately 30 ft (9.1 m) underlain by loose to medium dense sand to a depth of about 60 ft (18.2 m). Groundwater was encountered at a depth of about 5 ft (1.5 m). The results of soil behavior types classifications from CPT supported the explored soil information.

Pier System

IRAP system was selected to install in this site. The IRAP was designed as 5 ft (1.5 m) center on the center distance. The top diameter and height was designed to be 20 inches (0.51 m) and 39 ft (11.9 m), respectively.

Tests and Results

The in situ tests included the CPT and modulus load tests. The pre-installation and post-installation CPT locations are shown in Figure 38. Two modulus load tests were performed on IP-1 and IP-2. The IP-1 that was located at the south area of the site was performed after 16 days later the pier installation. The IP-2 which was located at the northeast area was performed after 9 days later the pier installation.

The pre-installation and post-installation CPT results are shown in Figure 40 and Figure 40. The results indicated that the IRAP was effective to improve the sand fill layer which consisted of 12 ft (3.6 m) depth. And it is not effective to improve the soft clay layer that extent from a depth of 12 ft (3.6m) to a depth of about 28 ft (8.5m). Figure 41 shows that soil with less than 20% fines content was effective to improve.

The isolated IRAP placed by 1 ft (0.3 m) thick concrete cap was used to modulus load test. Figure 42 shows the modulus load test results. The top deflections of IP-1 and IP-2 were approximately 0.21 inch (0.53 cm) and 0.15 inches (0.38 cm) at a designed stress of about 18 ksf (0.86 MPa), indicating an impact stiffness modulus of 619 pci (2284 MN/m^3) and 873 pci (3221 MN/m^3), respectively. The results indicated that the pier increased the stiffness due to the aging effect.

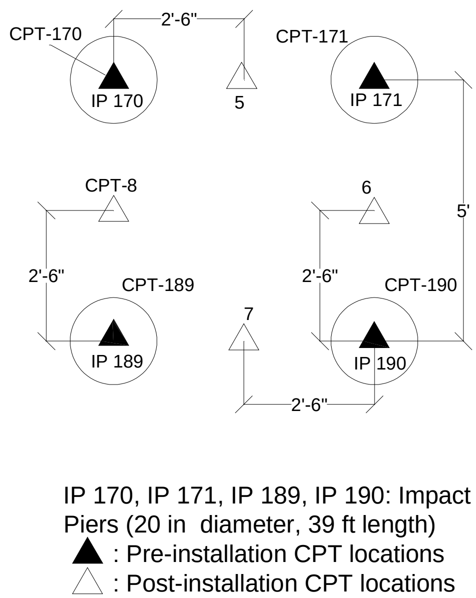
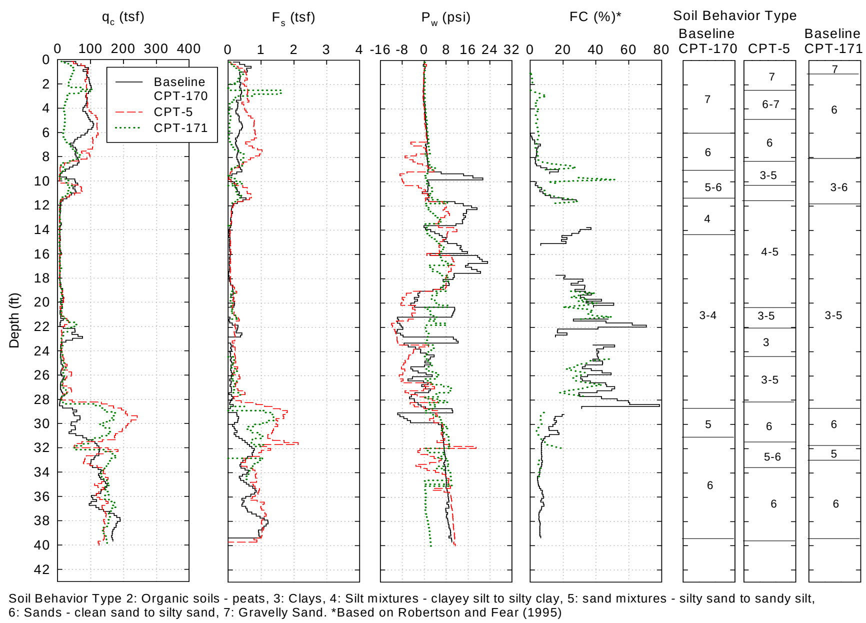
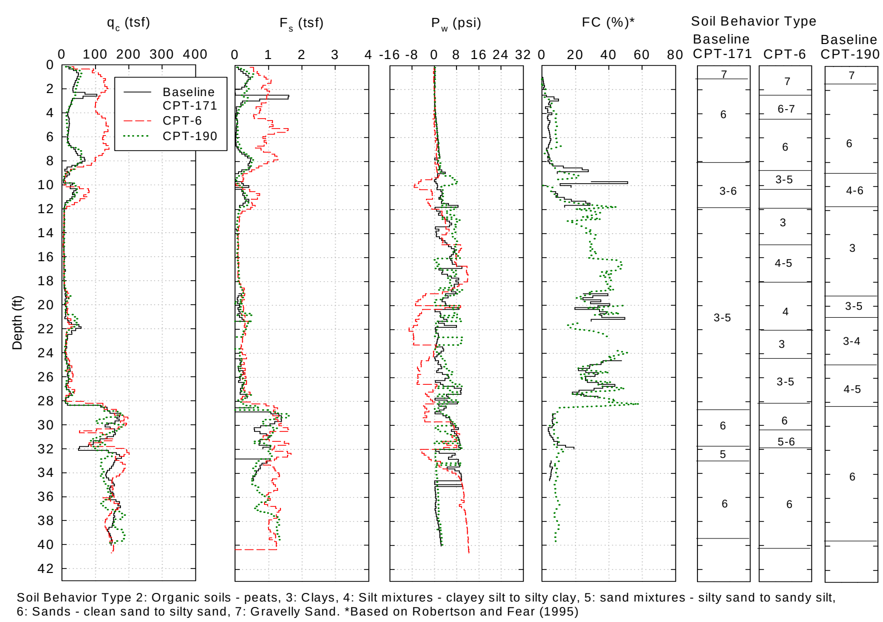


Figure 38. Seattle, CA: Pier and CPT locations plan layout

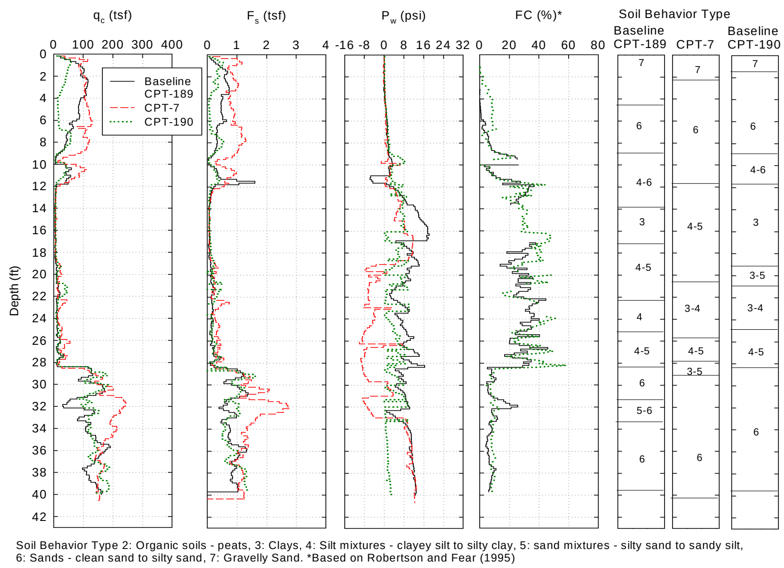


(a)

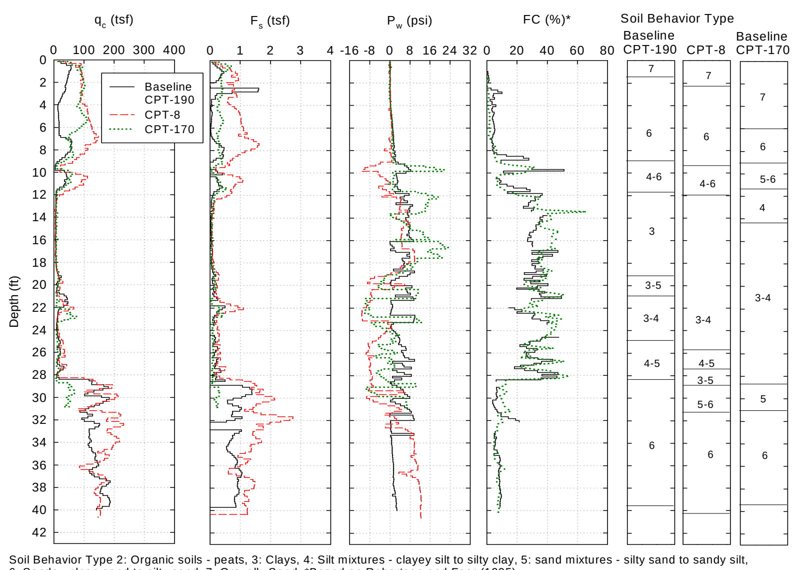


(b)

Figure 39. Seattle, CA: Pre-installation and post-installation CPT results, fines content and soil behavior types profiles between (a) pier-170 and pier-171, and (b) pier-171 and pier 190



(c)



(d)

Figure 40. Seattle, CA: Pre-installation and post-installation CPT results, fines content and soil behavior types profiles (c) pier-189 and pier-190, and (b) pier-190 and pier 170

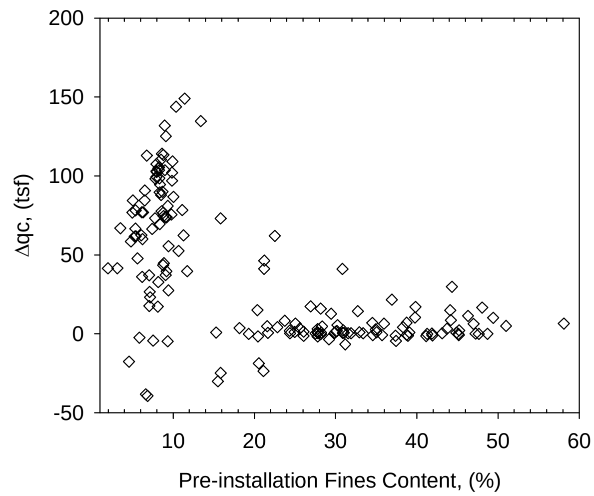


Figure 41. Seattle, CA: The fines content versus increasing tip resistance

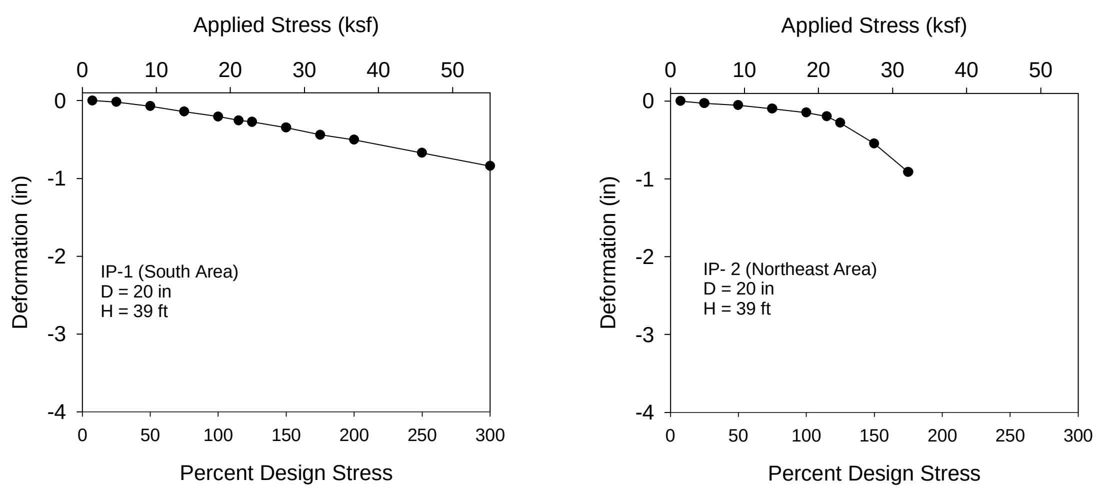


Figure 42. Seattle, CA: Applied stress and deformation results from modulus load tests

Springfield, MA

Project Description

The testing plan was located in the Church of God site in Springfield, MA. During the construction, the difficulties with flow of the 3/8 inches (1 cm) to 3/4 inches (2 cm) aggregate were observed within the sand fill layers above the peat soil during the construction. It was suspended that clogging of the mandrel might occur. The introduction of water was used after the flow restriction. The purpose of this project was to verify that the strength and stiffness of the piers constructed prior to the usage of water in the mandrel that was adequate in the sand fill layer above the peat.

Subsurface Conditions

The subsoil consisted of loose to dense sand fill materials to a depth of about 14 ft (4.3 m), underlain by peat to a depth of about 19 ft (5.8 m), and then underlain by loose to dense sand.

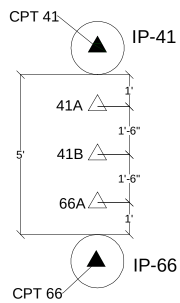
Pier System

IRAPs were selected to install in the project site. The piers were designed to a height of 25 ft (7.6 m).

Tests and Results

CPTs were conducted to test the stiffness and strength of the piers and matrix soils. Ten of the thirteen CPTs were performed at the center of the IRAP elements. The other three CPT soundings were performed in the matrix soil which is shown in Figure 43 (CPT 41A, 41B and 66A).

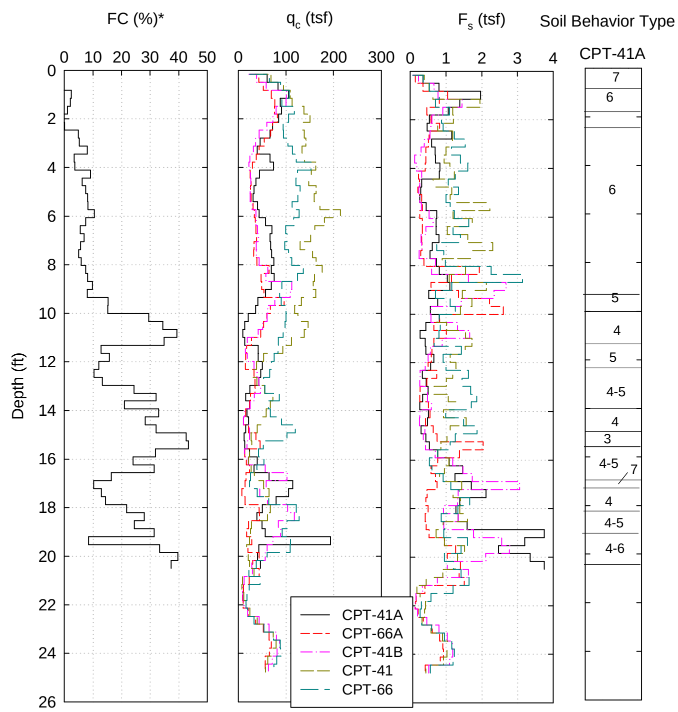
CPT results are shown in Figure 44. The tip resistance varied from 100 tsf to 200 tsf (9.6 MPa to 19.2 MPa) in pier element, was usually 2 to 3 times greater than that of matrix soil in the sand fill layer. The tip resistance of the piers in the peat zone seemed to be equal to the tip resistance in the matrix soil. The pier strength and stiffness in the upper fill layer verified that the construction of the piers prior to adding water was acceptable.



IP Number	CPT Number
177	177
280	280
438	438
318	318
319	319
341	341
342	342
542	542
543	543
535	543

IP-66: Impact Pier(D = Unknown,
H = 25 ft)
▲ : CPT through center of piers
△ : CPT in matrix soil
Note: Except CPTs 41A, 41B and 66A are test in matrix soil, all CPTs through at center of piers)

Figure 43. Springfield, MA: The locations of CPT soundings



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand.
*Based on Robertson and Fear (1995)

Figure 44. Springfield, MA: CPT profiles performed on center on piers and matrix soil (only CPT 41A performed in the matrix soil)

Prince George County, MD

Project Description

The IRAP system was designed for alternative footing foundation for chalk point plant located in Prince George County., MD. The construction equipment is used both the Liebherr hammer and ABI hammer.

Subsurface Conditions

The subsoil conditions consisted of the sand and silt mixture to a depth of about 35 ft (10.6 m). The groundwater table was at the depth of 25 ft (7.6 m) based on the CPT results.

Pier System

IRAP was design spacing at about 9 ft (2.7m) in the 160 ft x 90 ft (48.5 m x 27 m) project area.

Tests and Results

The pre-installation and post-installation CPT locations are shown in Figure 45. From the test results (Figure 46), the average tip resistance values outside the individual piers and within the piers group were 142 tsf (13.6 MPa) and 162 tsf (15.5 MPa), respectively. Figure 47 shows the results of fines content and the tip resistance increasing. The results indicated that soil with less than 20 percent fines content showed a better improvement.

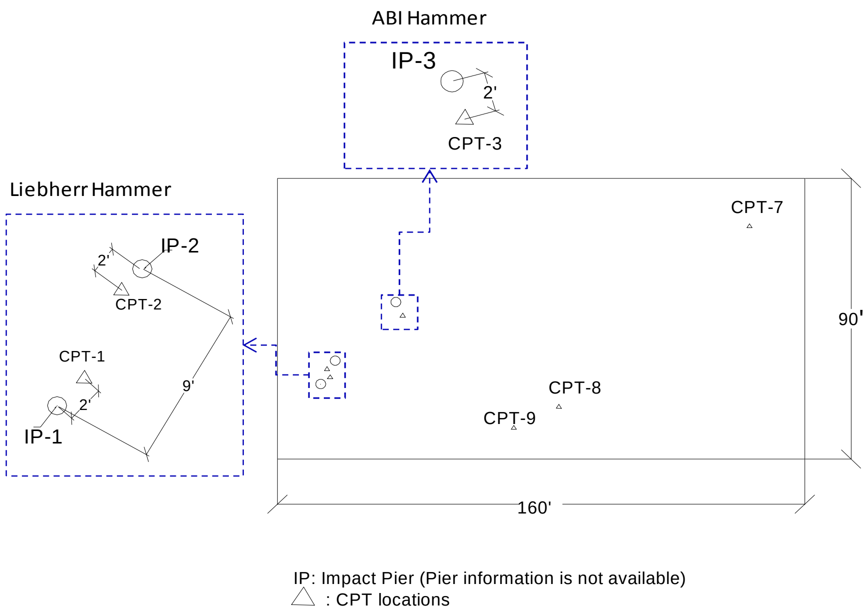
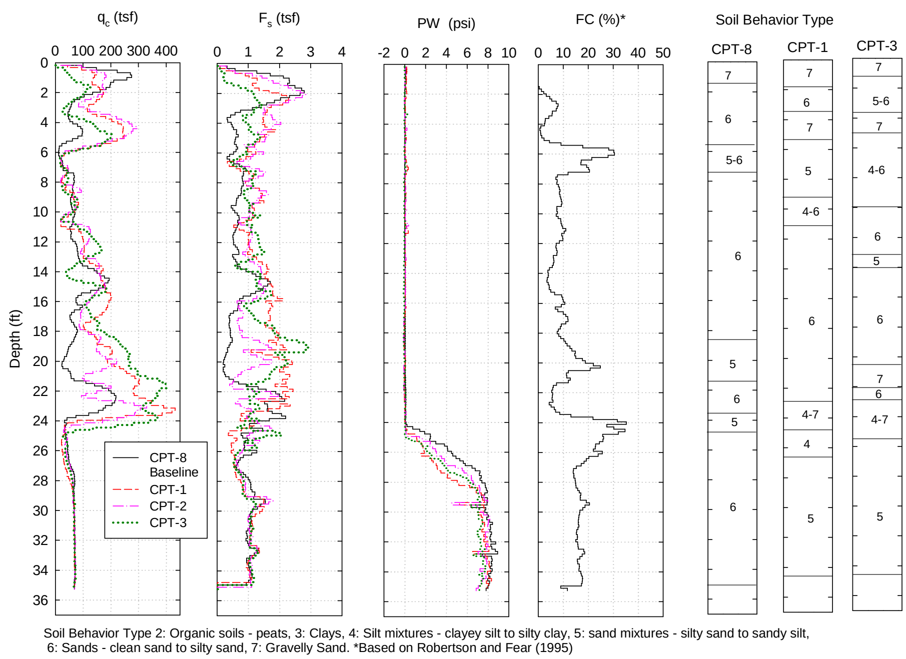
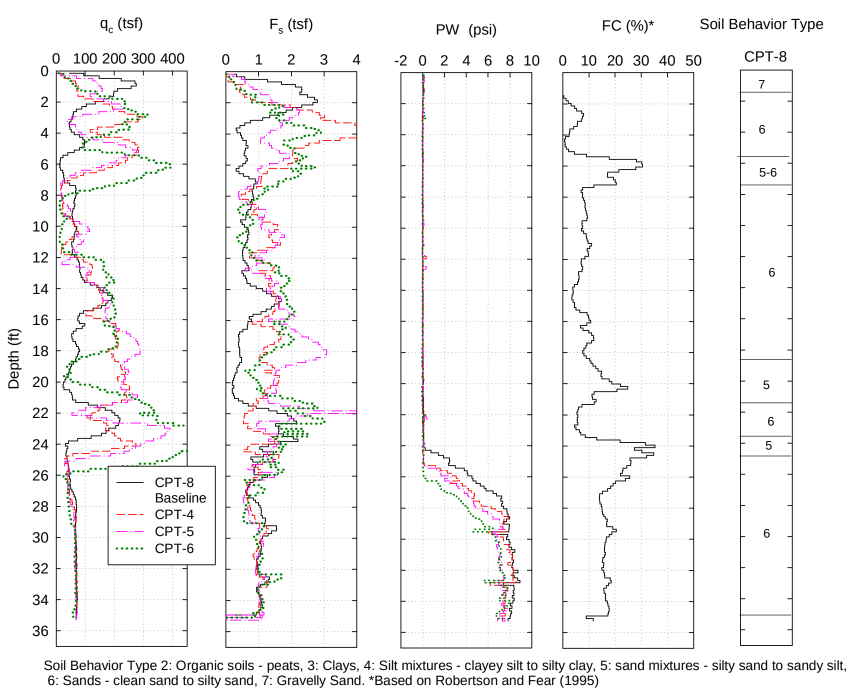


Figure 45. Prince Geoge County, MD: Plan layout of CPT locations (CPTs in pier groups located at the vicinity project locations)



(a)



(b)

Figure 46. Prince Geoge County, MD: CPT results profiles for individual pier (a) and pier groups (b)

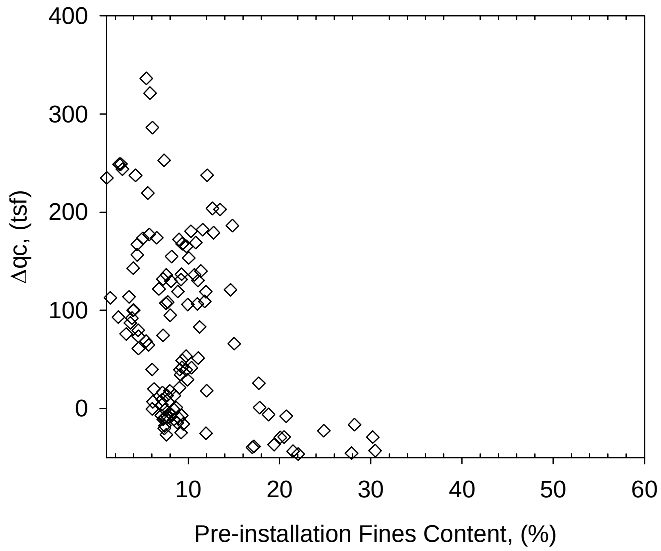


Figure 47. Prince Geoge County, MD: The fines content versus the tip resistance increasing

Waterloo, IA

Project Description

The project was located at the Wagner Road test site in Waterloo, IA. The purpose of this project was to study the construction methods, materials, individual and pier groups' effects. Additionally, the modulus load tests investigated the aging effect of the stiffness of a single pier. The piers' information and locations and in situ test locations are shown in Figure 48.

Subsurface Conditions

The SPT borings profile near the CPT locations indicated that the subsoil consisted of granular alluvium which was composed of poorly graded sand with varying amounts of fines and gravel. The fine sand trace clay and organics to a depth of 2.5 ft (0.75 m), underlain by fine to medium sand trace clay and gravel to a depth of about 25 ft (7.5 m). The groundwater table was observed to be between 9 and 12 ft (2.7 m to 3.7 m) based on the recorded pore water pressure.

Pier System

The types of DAP consisted of IRAP, rampact pier (RP) and chain mandrel (CM). The materials included sand and aggregate. The diameters of the DAP varied between 20 inches (0.51 m) and 24 inches (0.61 m) The height of the piers was designed to be between 7 ft (2.1 m) and 14 ft (4.2 m). The results indicated that the nominal diameter of the IRAP system varied from 21.6 inches to 24 inches (0.55 m to 0.6 m).

Tests and Results

The in situ tests consisted of CPTs and modulus load tests. CPTs were performed both pre-installation and post-installation of DAP. Modulus load tests were performed on every type of pier, including one group of four IRAPs. Another four piers were performed modulus load tests on the different days after installation to investigate the aging effect. The CPT results are shown in Figure 49 to Figure 52. The modulus load test results are shown in Figure 53 and Figure 54. The summaries of modulus load test results are shown in Table 7 and Table 8.

The total average tip resistance increased from 58 tsf to 126 tsf (5.55 MPa to 12.1 MPa) in pier group.

The orders of average tip resistance values of the matrix soil at 2 ft (0.6 m) from center of piers are: IP-S > RP > RP-S > CM > IP. The orders of the stiffness of piers at designed stress level are: IP-S > RP > RP-s > CM > IP. The results show that the larger tip resistance values of the matrix soil indicated larger stiffness of the piers at the design stress level.

The pier stiffness increment may be due to the aging (time) effect. The design stiffness modulus might increase to 1.5 times in two months.

The modulus load test results of IP-1 and IP-2 indicated that the stiffness modulus and bearing capacity were not significantly different from the single pier test of 7 ft (2.1 m) height and 14 ft (4.3 m) height.

For aggregate material, the stiffness modulus of impact processes and rampact processes were not significantly different. But the stiffness modulus of rampact processes was 1.3 times greater than that of the impact processes for sand material.

Figure 55 shows the fines content is not a significant factor to influence the matrix soil improvement for the soils with fines content less than 15% in this test site.

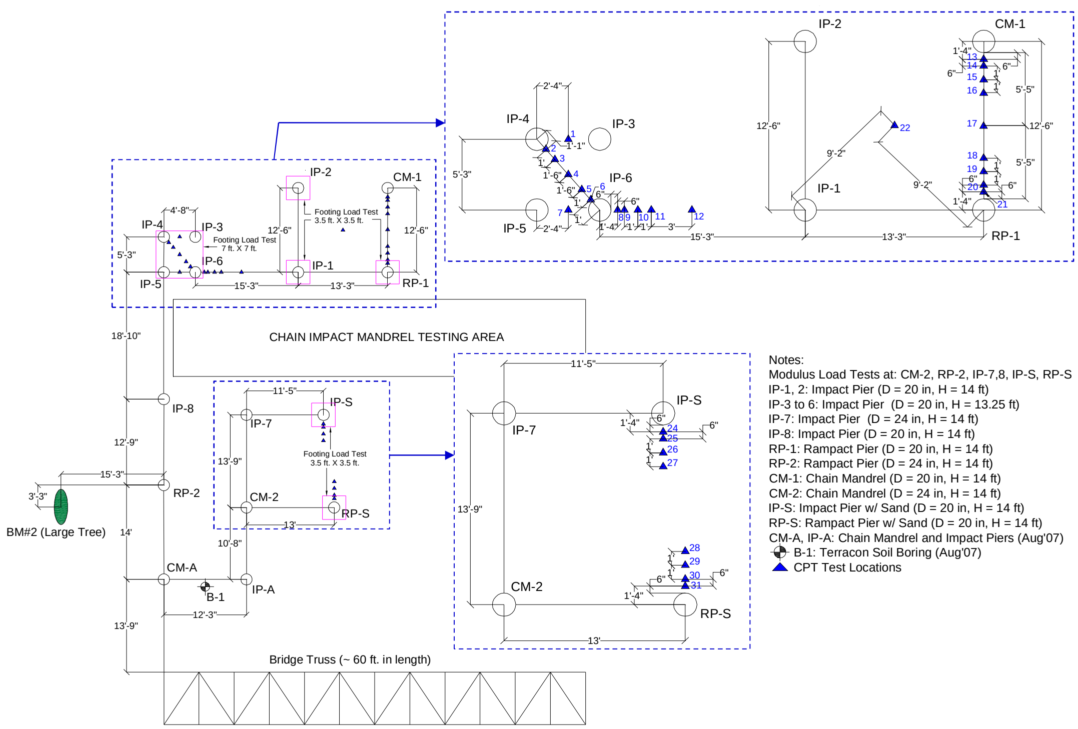
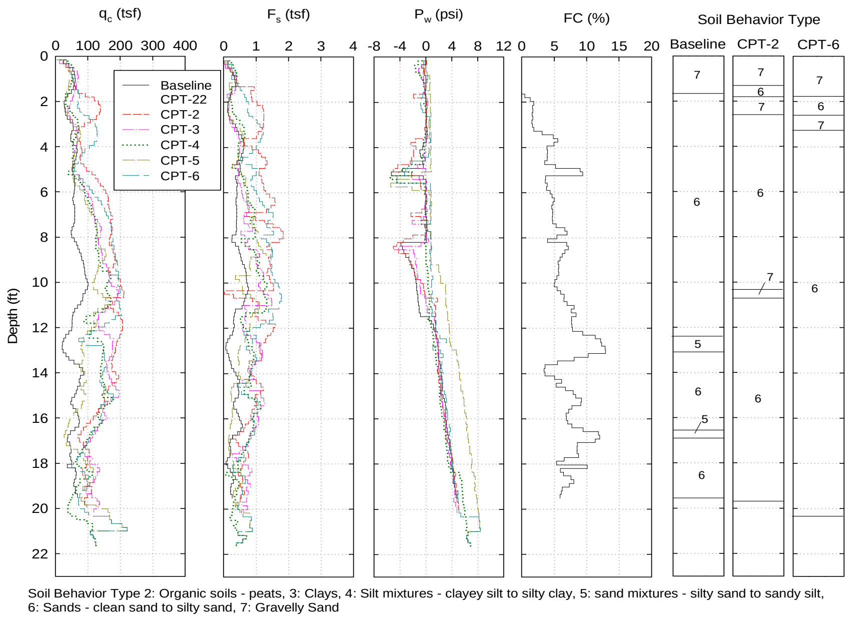
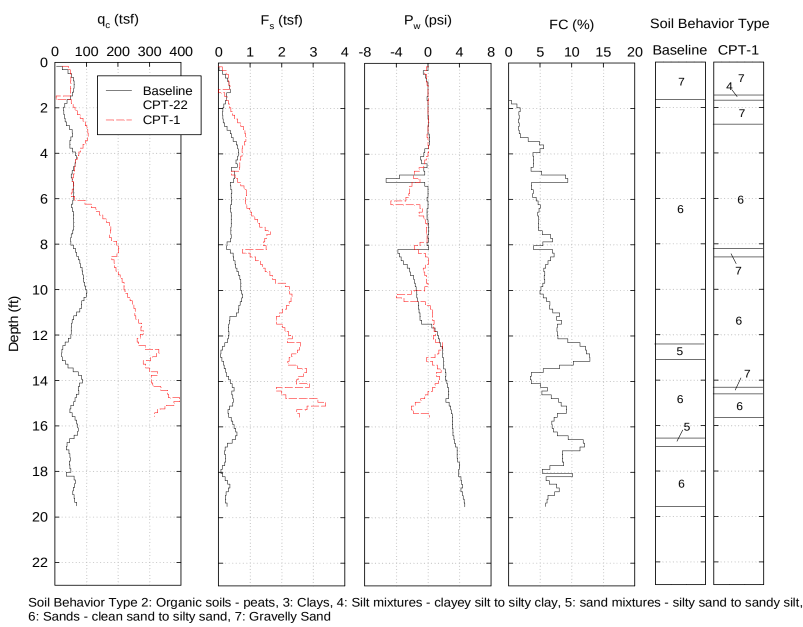


Figure 48. Waterloo, IA: Pier and CPT locations plan layout

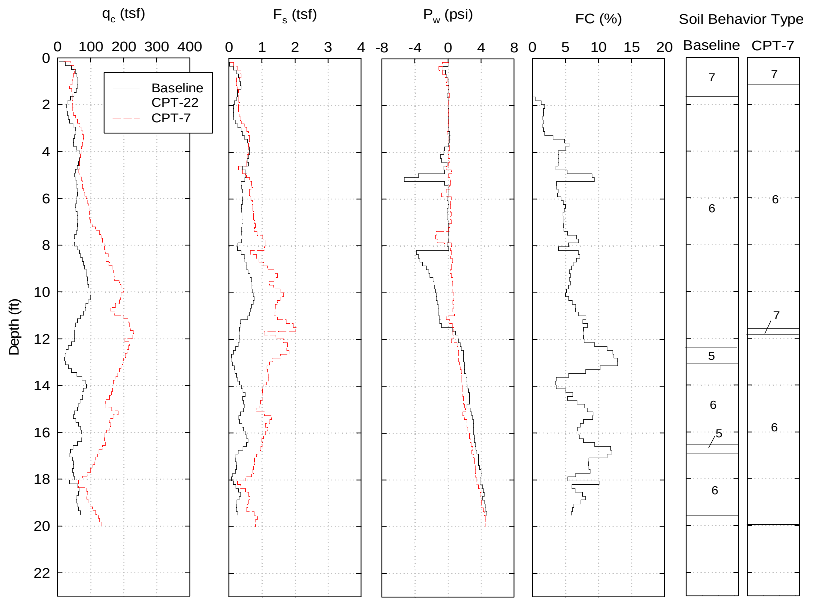


(a)



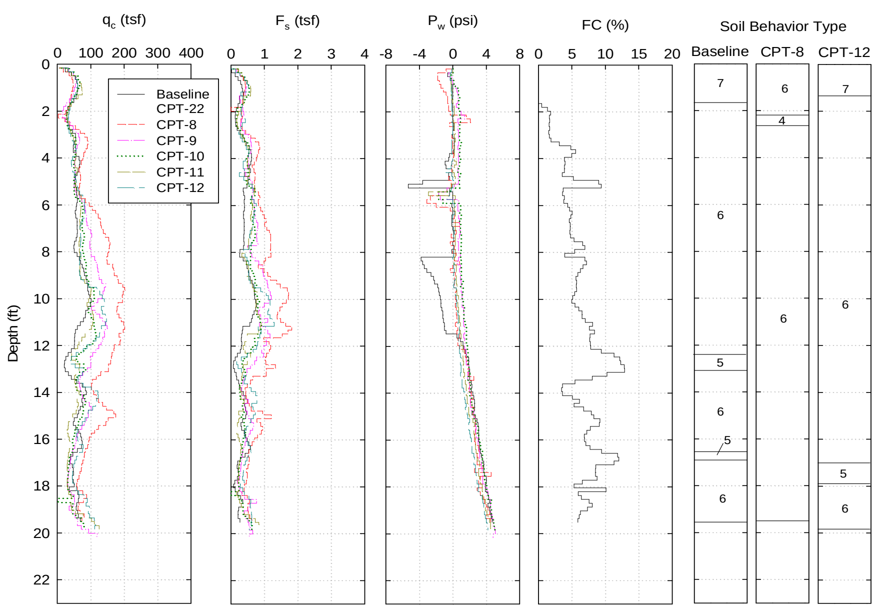
(b)

Figure 49. Waterloo, IA: CPT profiles for tests performed near the (a)IP-3 and (b)IP-4



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand

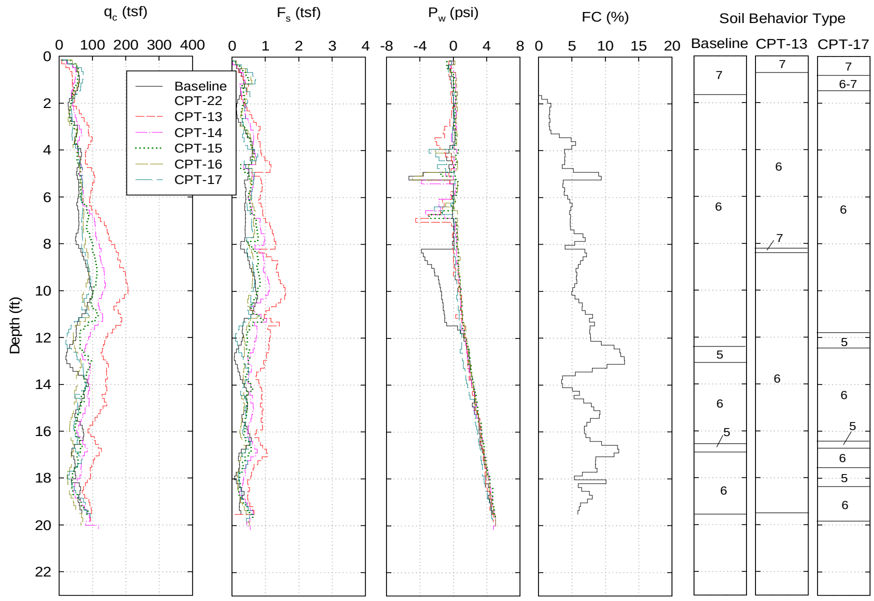
(c)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand

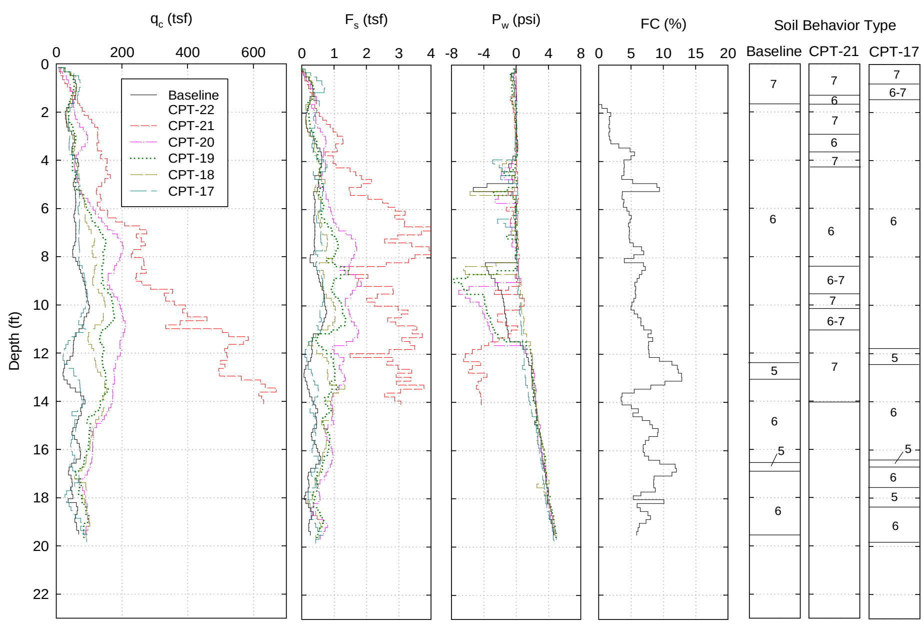
(d)

Figure 50. Waterloo, IA: CPT profiles for tests performed near the (c) IP-5 and (d) IP-6



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand

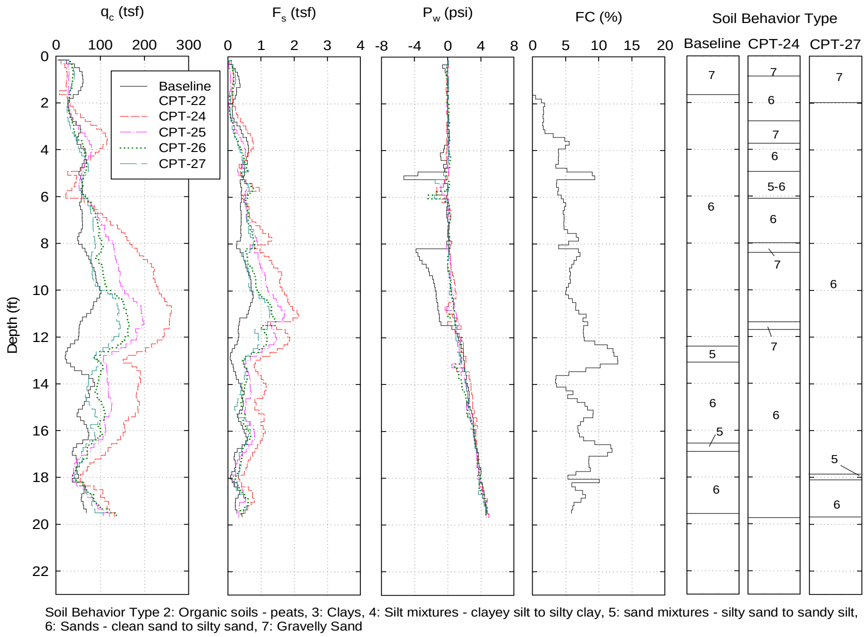
(a)



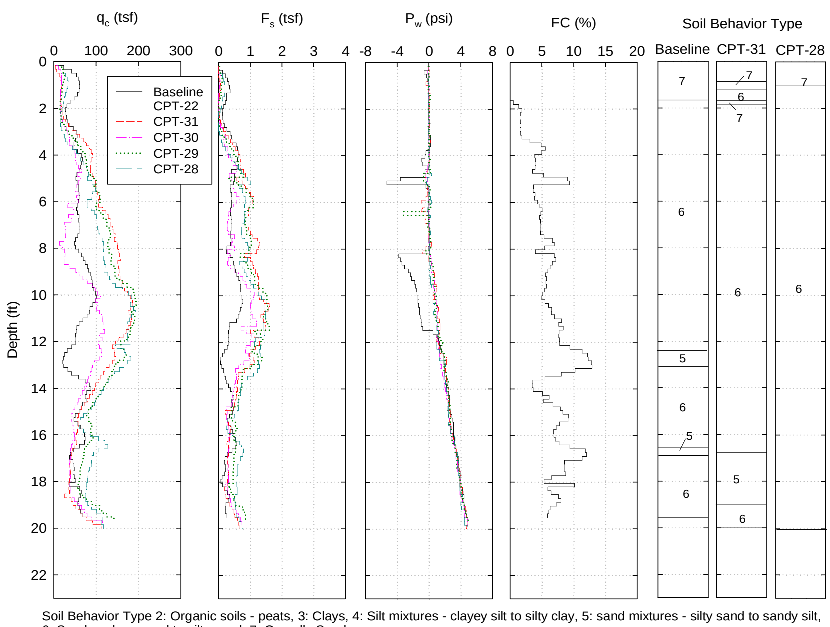
Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand

(b)

Figure 51. Waterloo, IA: CPT profiles for tests performed near (a) CM-1 and (b) RP-1



(c)



(d)

Figure 52. Waterloo, IA: CPT profiles for tests performed near (c) IP-S and (d) RP-S

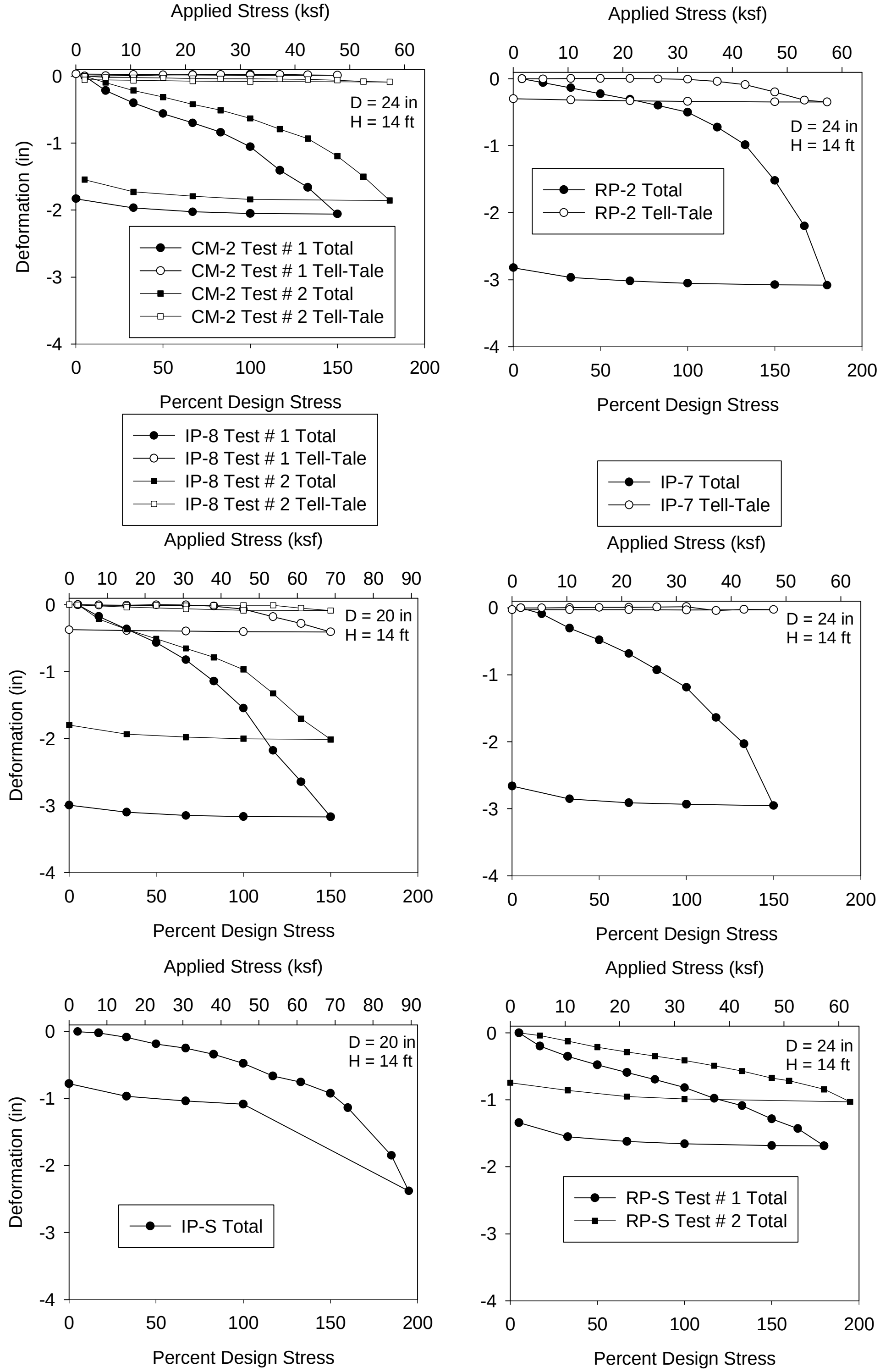


Figure 53. Waterloo, IA: Applied stress and deformation results from modulus load tests for individual piers

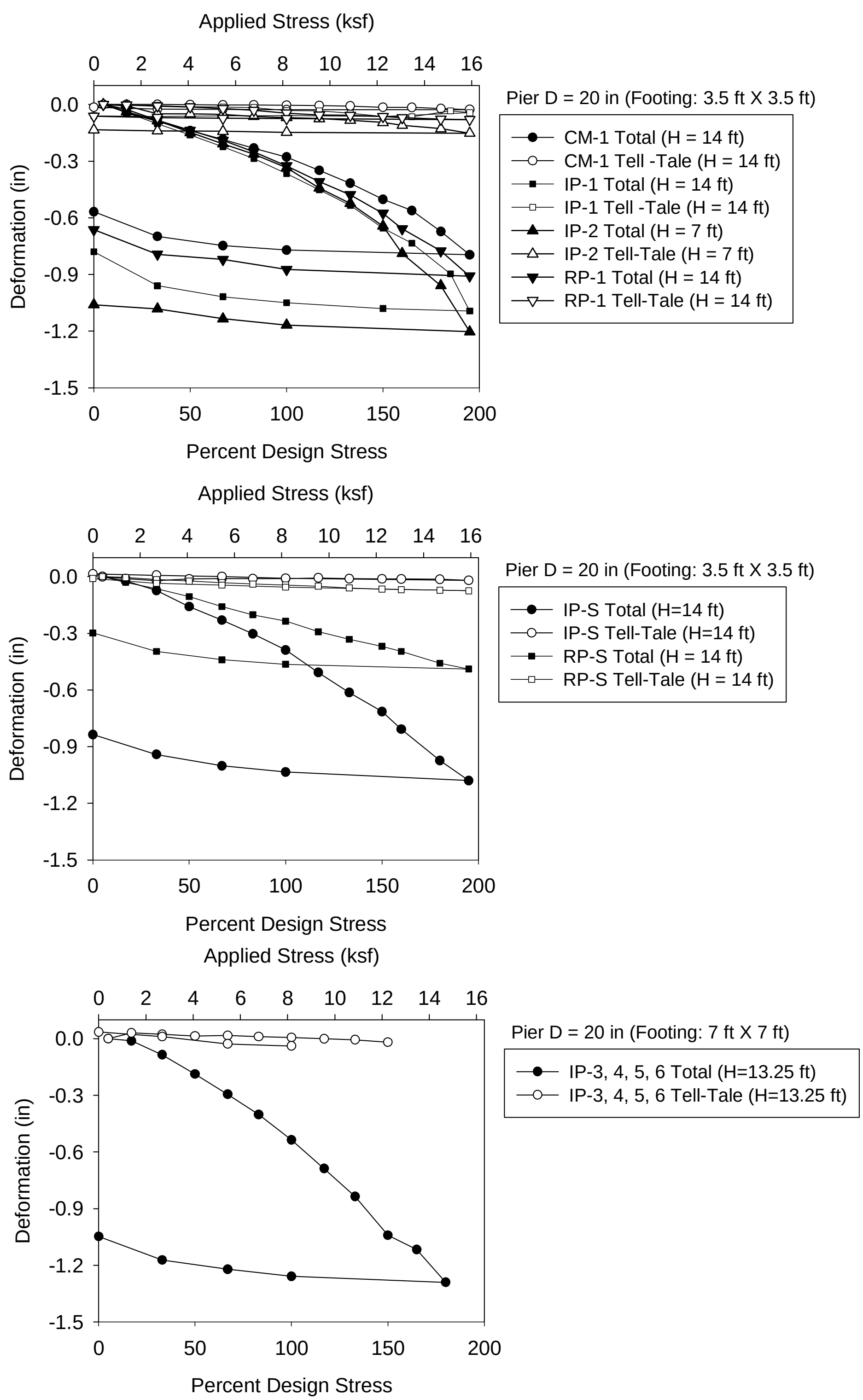


Figure 54. Waterloo, IA: Applied stress and deformation results from modulus load tests for footings

Table 7: Waterloo, IA: Summary results from modulus load tests for individual pier

Pier ID	Days after pier installation	Diameter, D (in)	Height, H (ft)	Design Stress (ksf)	$k_{100\%}$ (pci)
CM-2 Test # 1	14	24	14	31.83	210
CM-2 Test # 2	21	24	14	31.83	350
RP-2	Not Given	24	14	31.83	442
IP-8 Test # 1	13	20	14	45.85	206
IP-8 Test # 2	20	20	14	45.85	329
IP-7	Not Given	24	14	31.83	186
IP-S	Not Given	20	14	45.85	671
RP-S Test # 1	12	24	14	31.83	271
RP-S Test # 2	57	24	14	31.83	539

Table 8: Waterloo, IA: Summary results from footing load tests

Pier ID	Days after pier installation	Diameter, D (in)	Height, H (ft)	Footing Size (ft X ft)	Area Replacement Ratio, Ar = Ag/AF	Design Stress (ksf)	k100% (pci)
CM-1	Not Given	20	14	3.5 X 3.5	0.18	8.163	204
IP-1	0	20	14	3.5 X 3.5	0.18	8.163	155
IP-2	0	20	9	3.5 X 3.5	0.18	8.163	169
RP-1	Not Given	20	14	3.5 X 3.5	0.18	8.163	174
IP-S	Not Given	20	14	3.5 X 3.5	0.18	8.163	146
IP-3, 4, 5, 6	0	20	13.25	7.0 X 7.0	0.18	8.163	106

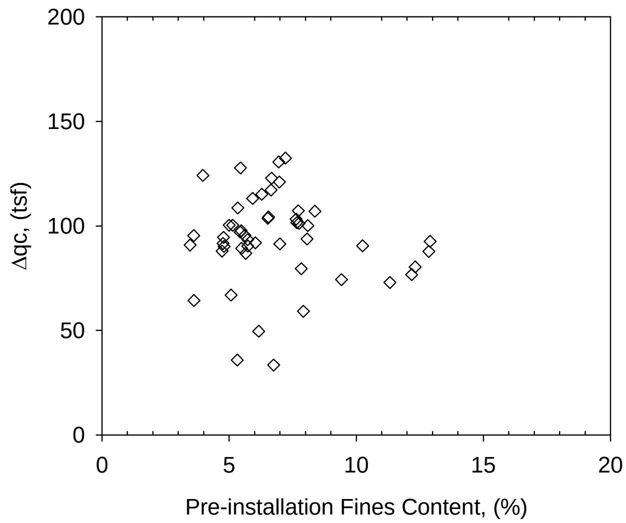


Figure 55. Waterloo, IA: Fines content and tip resistance increasing

Lynn Haven, FL

Project Description

This project involved the construction of the alternative footing foundation of the elementary school facility which was located in Lynn Haven, FL. IRAP was selected to support the columns and walls' footings for increasing the bearing capacity, decreasing the settlement and reducing the construction cost. Depending on the column loads, 2 to 6 IRAPs were designed beneath the column footings. The distance between piers beneath the wall footings was varied from 7 ft (2.1 m) to 12.8 ft (3.9 m).

Subsurface Conditions

The SPT soil explored vicinity of the project location indicated that the soil profile consisted of clayey fine sand to medium fine sand to a depth of 18 ft (5.4 m), underlain by silty sand with mica.

Pier System

Several hundred IRAPs were designed and installed in the construction area. The piers were designed as 20 in. (0.51 m) diameter and 6 to 7 ft (1.8 to 2.1 m) height.

Tests and Results

The in situ tests were consisted of SPT, CPT and modulus load test. SPT was performed before the pier installation and located at the vicinity of the CPT locations. The CPTs were conducted both before and after the pier installation where its locations are shown in Figure 56. The modulus load test was performed on the IP-1 (Figure 56).

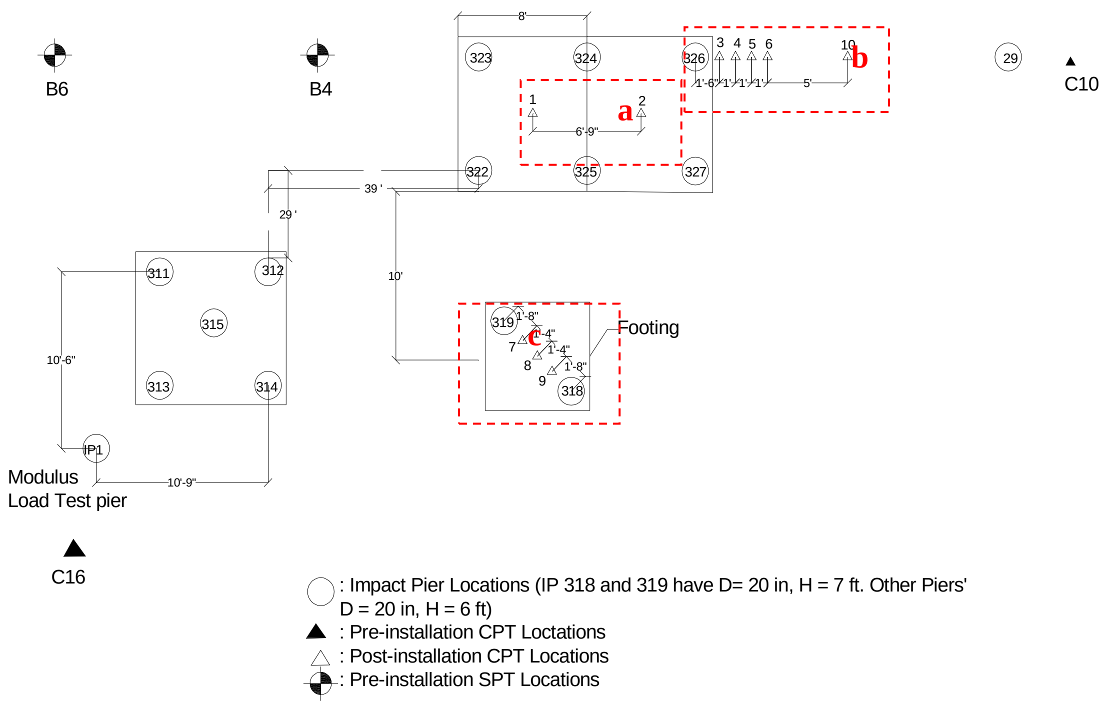


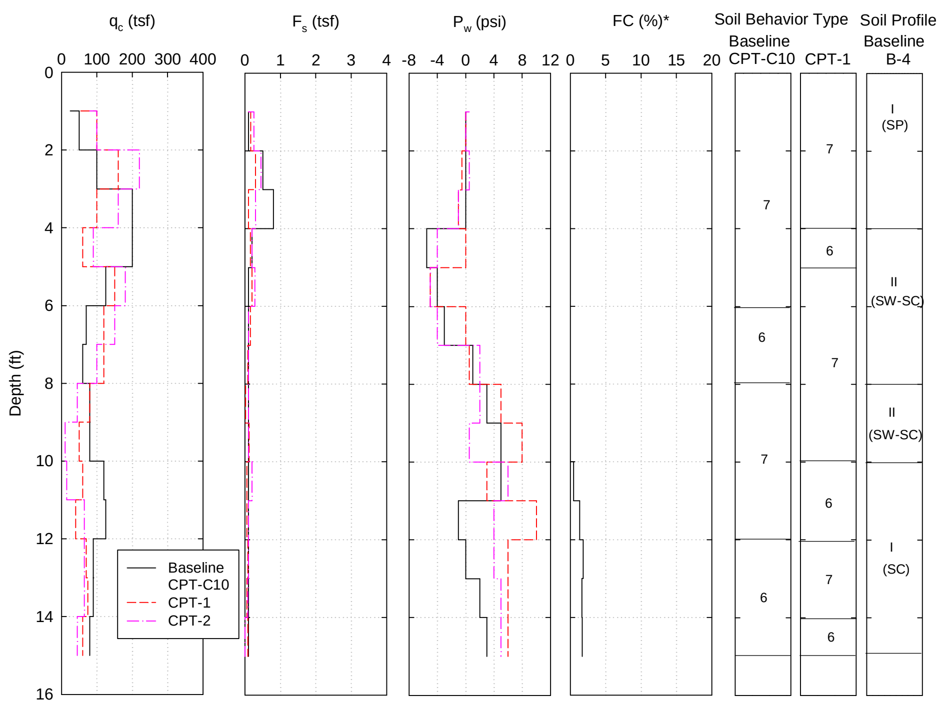
Figure 56. Lynn Haven, FL: Pier and CPT locations plan layout

Figure 57 shows the CPT results which the values were estimated at 1 ft (0.3 m) intervals from hard copies. The CPT results indicated that the soil densification in this site was difficultly quantifiable. The possible reasons might be due to the following cases:

- Significant amount of fines content existing on the construction site

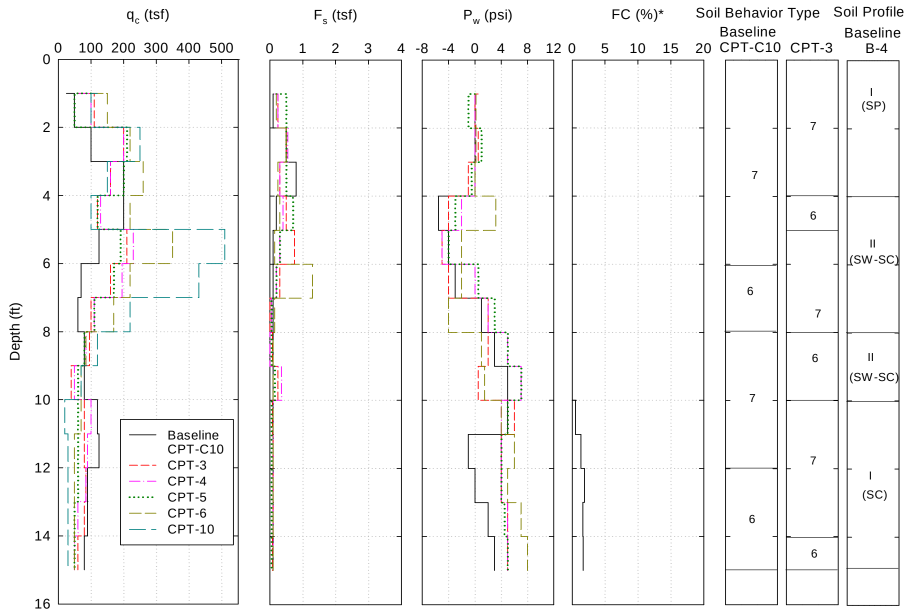
- Clayey soil types of the fine-grained soil may significantly reduce the effectiveness of the improvement
- The soil stratum were complicated

The deformation was about 0.2 inches at the top of the design stress level of 18 ksf (0.86 MPa) indicating 663 pci (2446 MN/m^3) of the stiffness modulus from load test. The results of the modulus load test are shown in Figure 58.



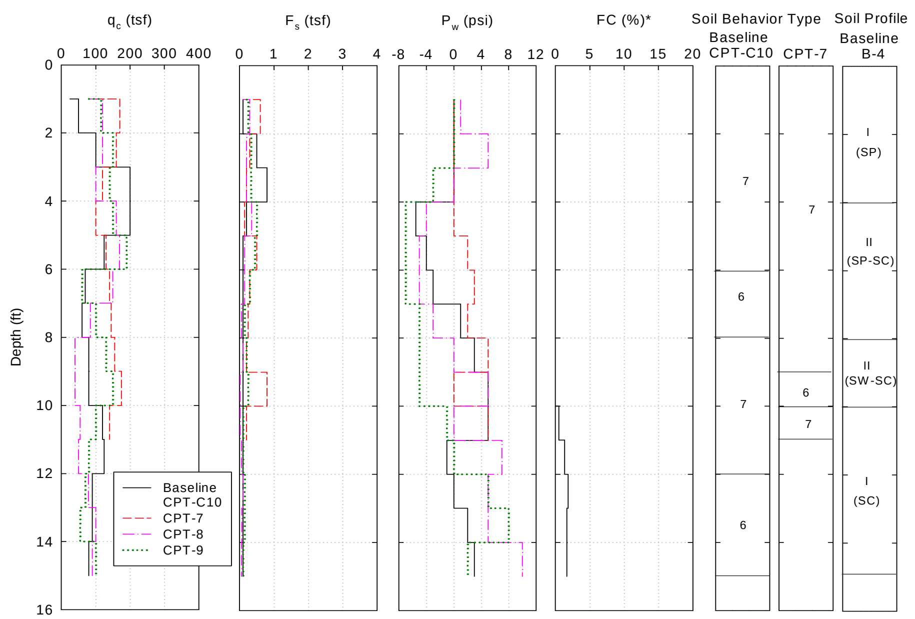
Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995);
O: loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29), III: Dense sand (N = 30 to 49),IV: Very dense sand (N: greater than 50)

(a)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995);
O: loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29), III: Dense sand (N = 30 to 49),IV: Very dense sand (N: greater than 50)

(b)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995);
O: loose sand (N: less than 3),I: Loose sand (N = 4 to 9), II: Medium dense sand (N = 10 to 29), III: Dense sand (N = 30 to 49),IV: Very dense sand (N: greater than 50)

(c)

Figure 57. Lynn Haven, FL: CPT and SPT profiles within the three different groups [group (a), group (b) and group (c)]

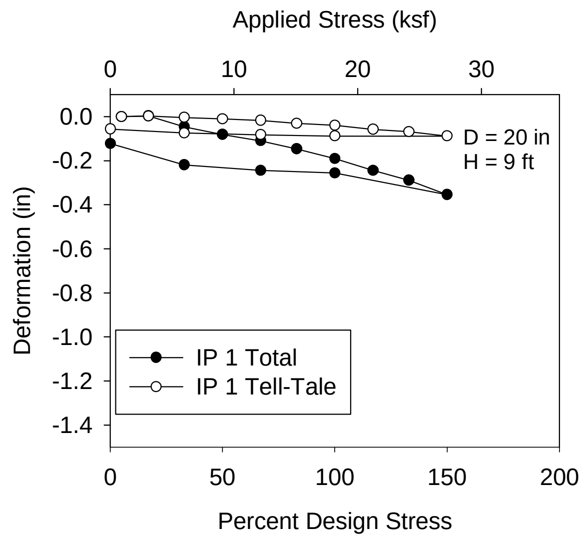


Figure 58. Lynn Haven, FL: The modulus load test results on IP-1

Jacksonville, FL

Project Description

IRAPs were introduced as an alternative foundation of JEA Kennedy Generating Station located in Jacksonville, FL. IRAPs were designed to support the footings consisted of buildings, demineralization tanks and GSU transformer. The SPTs and full scale modulus load test were conducted in the project site.

Subsurface Conditions

The soil profile consisted of the loose to dense fine sand to a depth of 35 ft, underlain by the weather limestone to a depth of 45 ft, underlain by stiff to hard clay. Figure 60 shows the subsoil profiles.

Tests and Results

The in situ tests included SPT and modulus load tests. The SPTs were performed both before and after the piers installation. The locations of SPTs are indicated in Figure 59.

The impact mandrel was observed to refusal during the installation at a depth of about 10 ft to 13 ft (3.1 m t0 4 m). SPT results profiles indicated that IRAP was effective in improving the soil around 2 diameters depth beneath the piers bottom in this site. The average SPT values of pre-installation (B-2, B-4 and B-6) and post-installation (B-1, B-3 and B-5) are 8 and 19 within the piers depth, respectively.

The modulus load test results are shown in Figure 61. The total deflection at the design stress of 28.2 ksf (1.35MPa) was about 0.21 inches (0.53 cm). The calculated stiffness modulus was 913 pci (3369 MN/m^3).

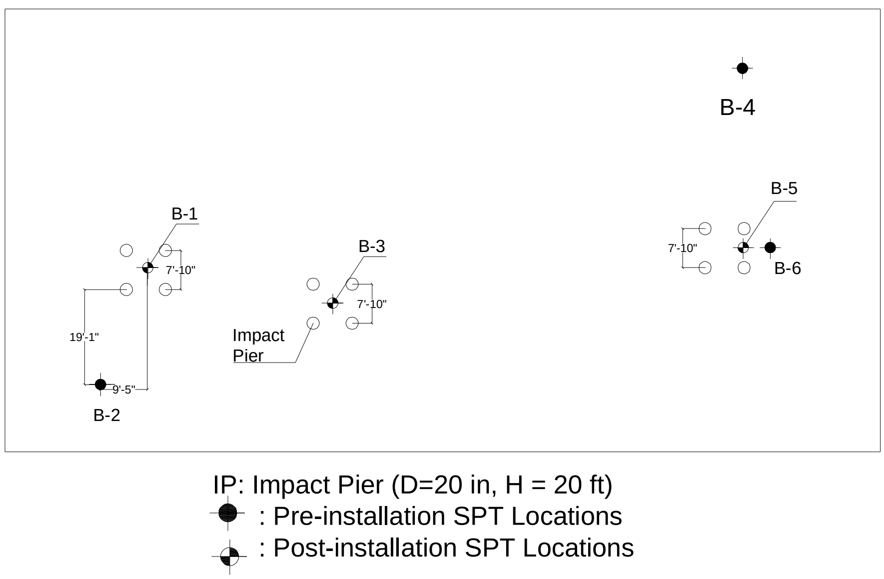


Figure 59. Jacksonville, FL: Piers and SPT locations plan layout

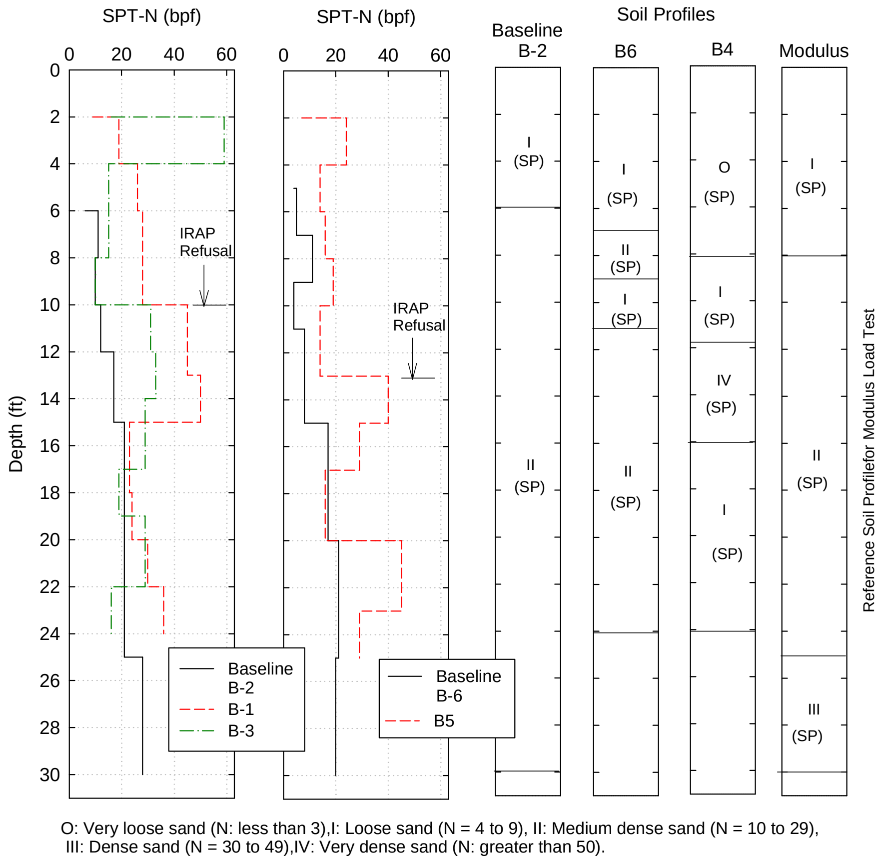


Figure 60. Jacksonville, FL: Pre-installation and post-installation SPT profiles

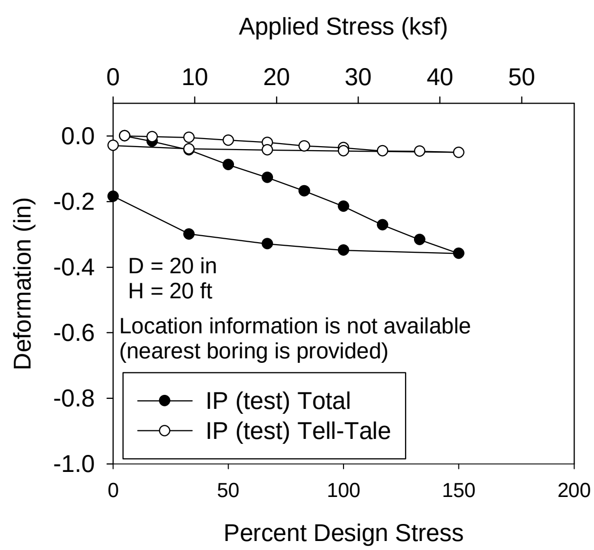


Figure 61. Jacksonville, FL: Modulus load test results on trial pier at vicinity B-2 location

Westminster, CA

Project Description

More than 2500 IRAPs were installed for the matrix soil improvement of the Moran Asian Garden project located in Westminster, CA. The design piers were at 10 ft x 7.4 ft (3.1 m x 2.3 m); for research purposes, piers were at 3 ft x 6 ft (0.9 m x 1.8 m). Twenty four CPTs were performed to compare the matrix soil improvement effectiveness. Twelve CPTs were performed before the IRAP installation, and another 12 CPTs were performed after the IRAP installation. The locations of the CPTs and IRAPs are shown in Figure 62. The IRAPs in "Test area A" were installed by the single-pass process and the IRAPs in "Test area B" were installed by double-pass process.

Subsurface Conditions

The subsoil conditions from the previous drill investigation indicated that the site was generally interbedded layers of loose to medium dense sand (SP, SP-SM, and SM), firm to very stiff non-plastic silt (ML), and firm to very stiff clay (CL). The lab tests showed that the fines content at the site contain approximately 58% to 82% for the fine grain soil layer. The silts were found to be non-plastic. The groundwater table was approximately 2 to 3 ft (0.76 to 1 m) below the ground surface.

Pier System

The diameters of the IRAP were about 24 in.(0.62 m) in "Test area A" locations and 30 in.(0.76m) in "Test area B" locations. The piers were installed to a depth of 25 ft (7.6 m) in testing area.

Tests and Results

CPT was conducted to investigate the soil improvement. The CPT locations are shown in Figure 62.The CPTs were performed for 2 weeks after the piers installation. Thus, the pore water pressure induced during the construction was considered fully dissipate.

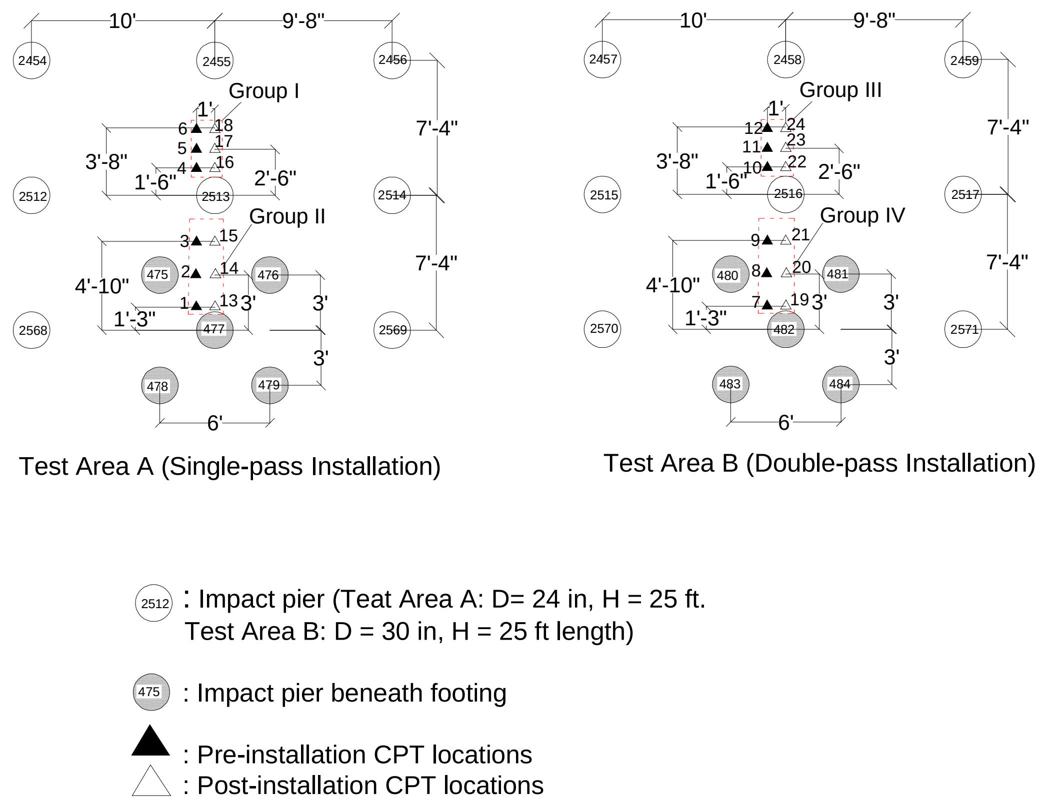
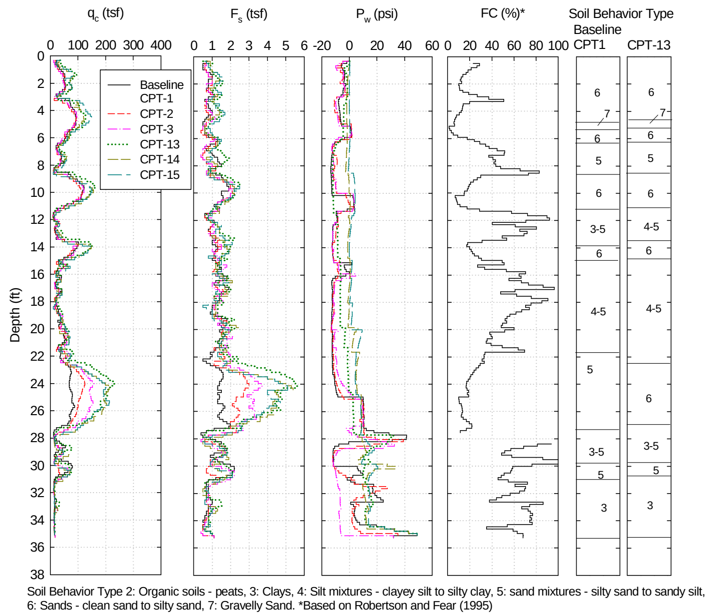


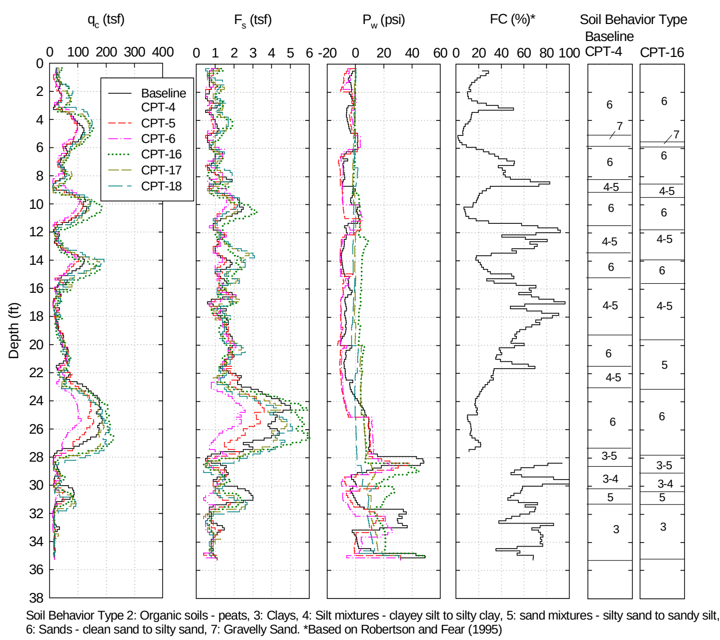
Figure 62. Westminster, CA: Plan layout of test pier, CPT locations

Figure 63 and Figure 64 show the CPT results. Based on the CPT sounding profiles, the ground may be reasonably defined as interbedded of soft layers and stiff layers. Many researchers indicated that the tip resistance of stiff soil layer could be underestimated if the stiff layer was less than 2.5 ft (0.76 m) in such interbedded layers (Vreugdenhil et al. 1994). Selecting the peak tip resistance in each layer to evaluate the matrix soil improvement in the sandy layer could represent the in situ conditions. The peak values for the stiff layers are summarized on Table 9. The soft layers were not shown to improve from the test results.

The average values of the peak tip resistance ratio of the double-pass and single-pass methods were 1.5 and 1.32, respectively. The average values of the peak tip resistance ratio of the smaller pier spacing and larger pier spacing were 1.42 and 1.45, respectively. The results indicated that the double-pass construction method had benefit for the matrix soil improvement. The pier spacing is in the range between 6 ft (1.8 m) and 7.3 ft (2.3 m) did not significantly influence the improvement in this site.

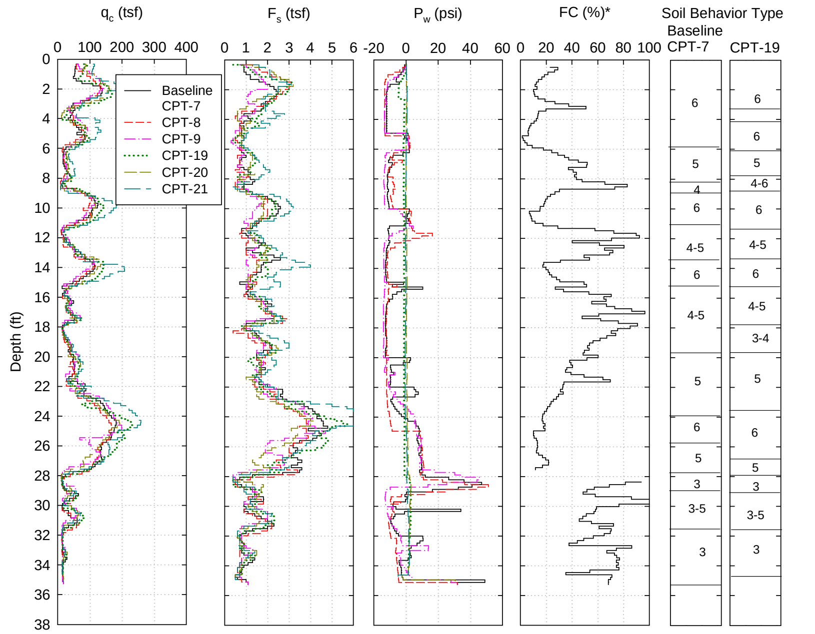


(a)



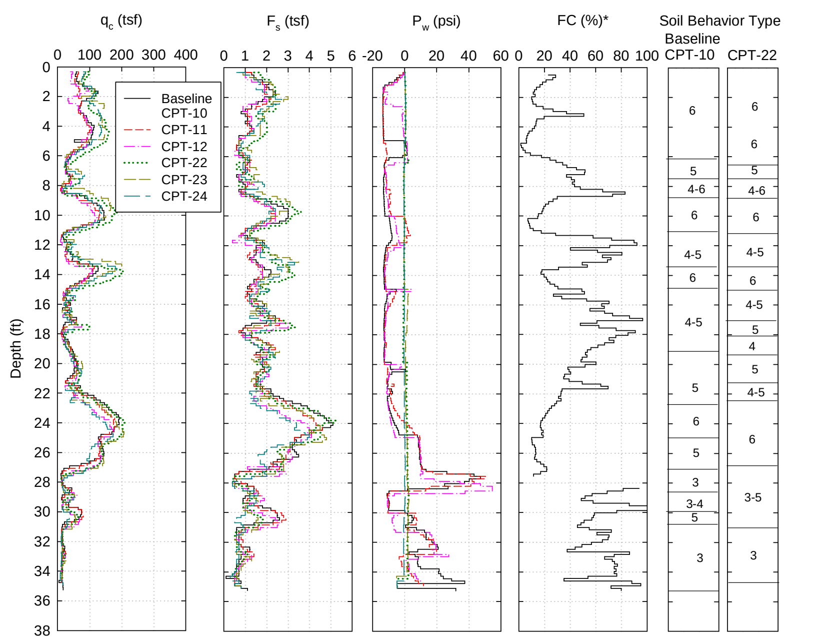
(b)

Figure 63. Westminster, CA: CPT profile before and after installation of piers in Group II (a) and Group I (b)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)

(c)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)

(d)

Figure 64. Westminster, CA: CPT profile before and after installation of piers in Group IV (c) and Group III (d)

Table 9: Westminster, CA: Summary results of peak tip resistance

Group II			
Layer	CPT 13/CPT 1	CPT 14/CPT 2	CPT 15/CPT 3
1	1.35	1.3	1.55
2	1.33	1.34	1.24
3	1.47	1.48	1.5
4	2.6	1.71	1.35
Group I			
Layer	CPT 16/CPT 4	CPT 17/CPT 5	CPT 18/CPT 6
1	1.27	1.43	1.52
2	1.39	1.21	1.39
3	1.43	1.67	2.07
4	1.2	1.23	2.03
Group IV			
Layer	CPT 19/CPT 7	CPT 20/CPT 8	CPT 21/CPT 9
1	1.13	1.07	1.43
2	1.16	1.12	1.56
3	1.24	1.19	1.93
4	1.26	1.15	1.3
Group II			
Layer	CPT 22/CPT 10	CPT 23/CPT 11	CPT 24/CPT 12
1	1.39	1.46	1.29
2	1.27	1.27	1.14
3	1.61	1.75	1.69
4	1.09	1.15	0.99

Summary	
Catalogs	Post/pre peak qc ratio
Single-pass installation	1.32
Double-Pass installation	1.5
larger spacing of piers	1.45
smaller spacing of piers	1.42

Oakland, CA

Project Description

The project was designed for alternative foundation footing for restaurant depot located in Oakland, CA. CPTs were conducted to investigate the matrix soil improvement. Based on the provided information, there were not CPTs conducted before the pier installations. Six CPTs were performed within the pier groups.

Subsurface Conditions

Subsoil conditions which were estimated from the CPT results and consisted of sandy soil to a depth of 5 ft (1.5 m), underlain by clayey soil to a depth of about 24 ft (7.3 m), and then underlain by interbedded sandy layers and clayey/silty layers.

Pier system

The IRAP was designed to matrix soil improvement. The spacing of the piers was 7 ft (2.3 m).

Tests and Results

Since the pre-installation CPTs were not conducted in this site, it is not possible to compare the soil improvement between pre-installation and post-installation. Figure 65 shows the locations of the pos-installation CPTs.

Figure 66 shows the CPT results of the post-installation CPT profiles. it was difficult to quantify the soil improvement. But it was noted that the pore water in clayey layer for CPT-1 from the depth of about 12 ft (3.7 m) to a depth of 24 ft (7.3 m) was significantly larger than the other two. It might be due to the distance between the CPT-1 and the IRAPs that was greater than that of the CPT-2 and CPT-3. This induced the pore water need take longer time to dissipate. The tip resistance values of the CPT-5 were significantly greater than that of the

CPT-4 from the depth of 12 ft (3.7 m) to 28 ft (8.5 m). The reasons are probably due to the original soil stratum or the ground disturbed.

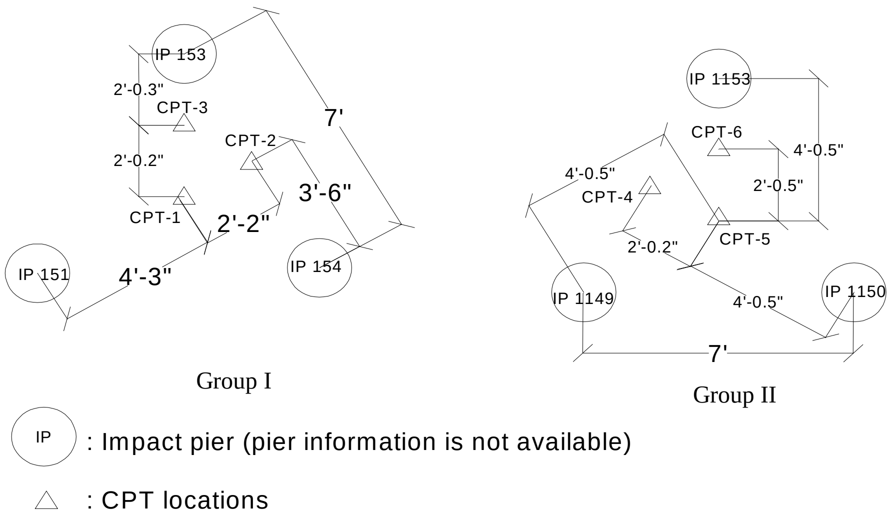
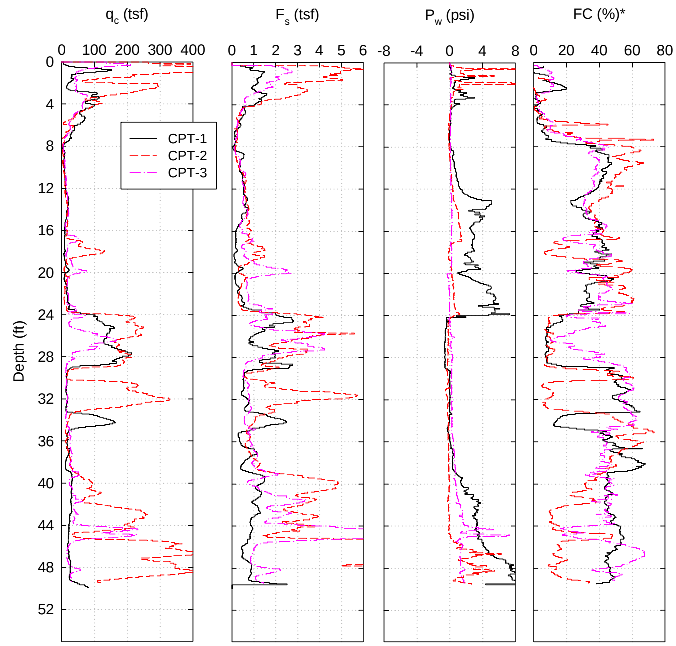
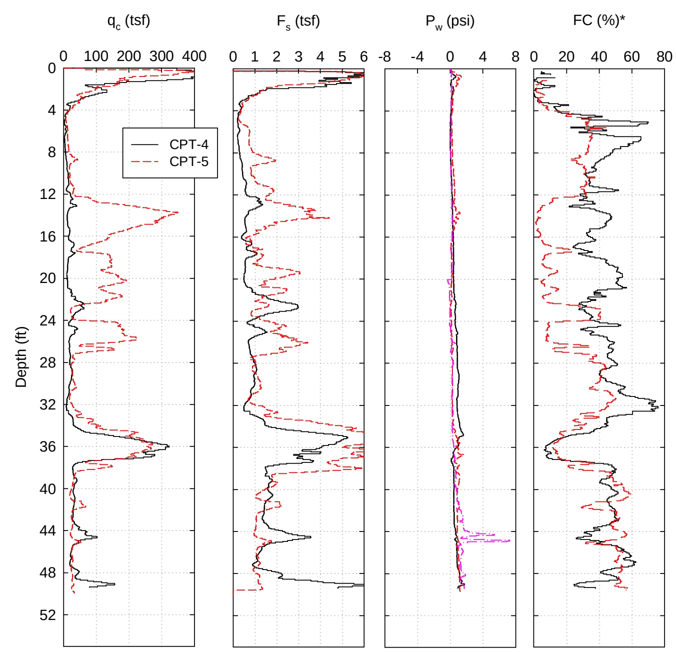


Figure 65. Oakland, CA: Pier and CPT locations plan layout



*Based on Robertson and Fear (1995)

(Group I)



*Based on Robertson and Fear (1995)

(Group II)

Figure 66. Oakland, CA: Pier and CPT locations plan layout

Waterloo, IA (Liquefaction)

Project Description

The project was to investigate the soil densification based on the different construction methods. The equipment, construction methods and CPT locations are illustrated in Figure 67. The installation time was recorded for quality control for each pier installation. After the pier installation, the average of 4'/3' method took an average of 5 minutes. The 4'/4' and 3'/2' methods took an average 15 minutes, while the pull-drive 4'/3' method took an average of 10 minutes. The averages of the diameters of the piers for the above three methods were 25.3 in. (0.65 m), 29.9 in. (0.76 m), and 28.7 in. (0.73 m).

Subsurface Conditions

The SPT borings profile near the CPT locations indicated that the subsoil consisted of granular alluvium which was composed of poorly graded sand with varying amounts of fines and gravel. The fine sand trace clay and organics to a depth of 2.5 ft (0.76 m), underlain by fine to medium sand trace clay and gravel to a depth of about 25 ft (7.6 m). The groundwater table was observed to be between 9 and 12 ft (2.7 m and 3.7 m) based on the recorded pore water pressure.

Pier system

IRAPs were constructed on the project area with a spacing of 8 ft. The diameters of the piers varied from 22 to 30 in. (0.56 m to 0.76m). The height of the piers was designed as 15 ft (4.5 m) and 20 ft (6m).

Tests and Results

The pre-installation in situ tests included SPT boring logs and CPT. Only CPTs were conducted after IRAP installations. The CPT locations were shown in Figure 67.

Figure 68 to Figure 70 show the CPT results profiles. The general trends from the tests results indicated that the soil profiles in the upper about 5 ft (1.5 m) were not significantly improved. The average values of the tip resistance of the three construction methods are summarized on Table 10. The results indicated that the pull-drive 4’/3’ method did not provide any benefit. The 4’/4’and 4’/3’ method generated the densest method however it was time consuming. The average tip resistance of soil profiles using Liebherr hammer and ABI hammer were 130 tsf (12.4 MPa) and 134 tsf (12.8 MPa), respectively.

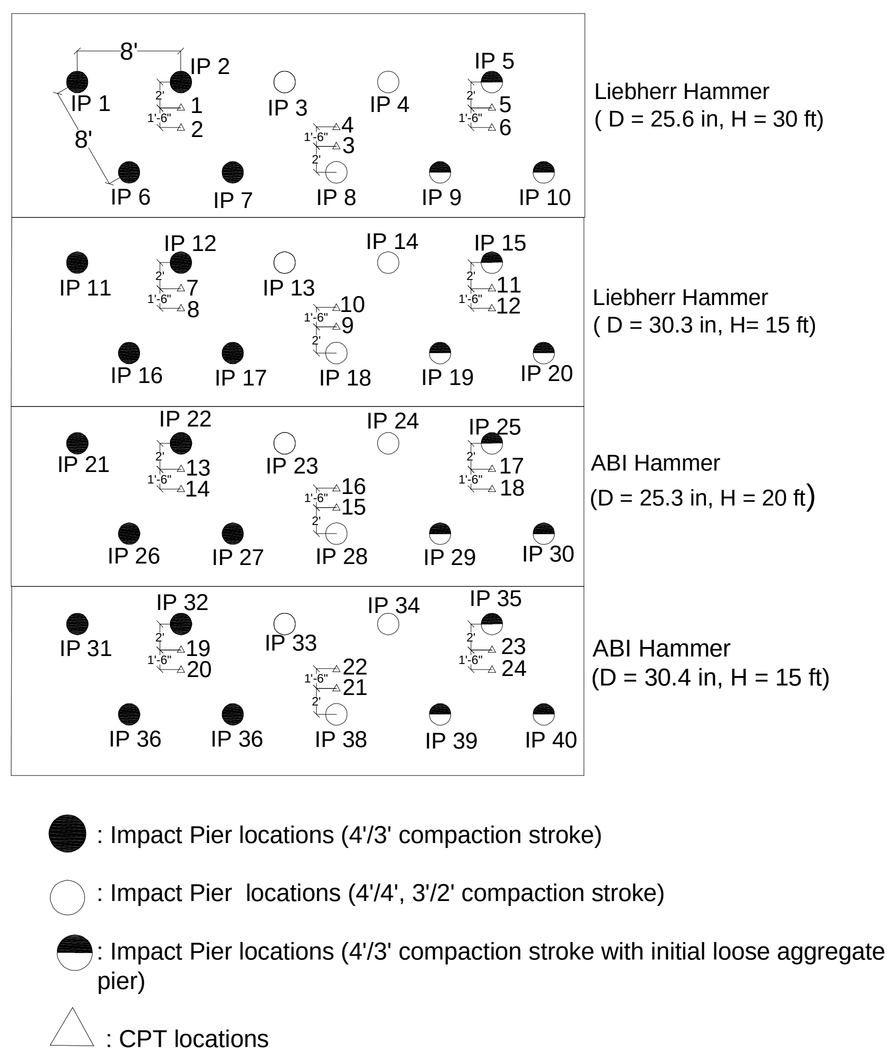
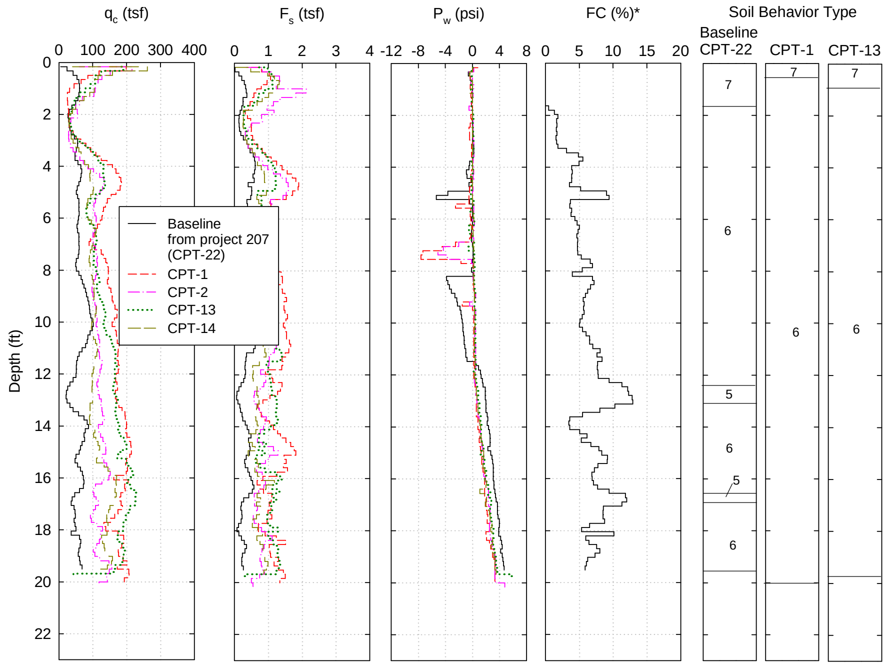
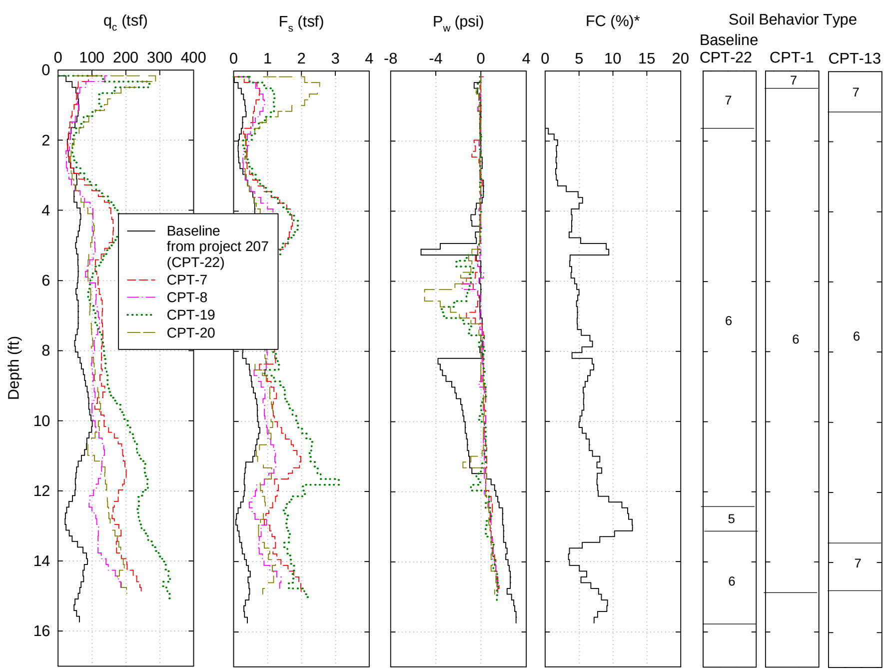


Figure 67. Waterloo, IA (Liquefaction): Pier and CPT locations plan layout

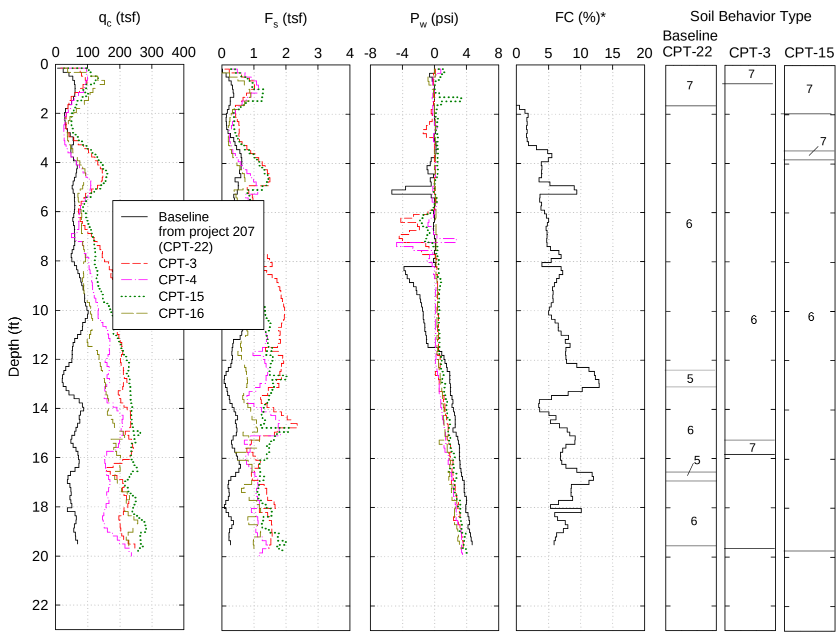


Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)



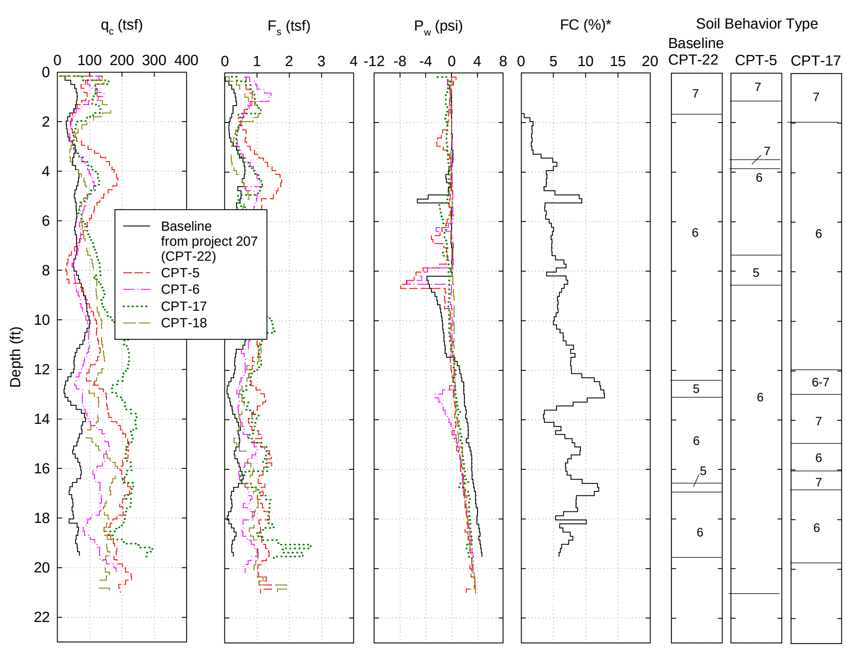
Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)

Figure 68. Waterloo, IA (Liquefaction): CPT profiles for tests performed by 4'/3' method using 20 ft (top) and 15 ft mandrel (bottom) (Waterloo, IA, CPT - 22 – Baseline)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)

Figure 69. Waterloo, IA (Liquefaction): CPT profiles for tests performed by 4'/4'and 4'/3' method using 20 ft mandrel (Waterloo, IA, CPT - 22 – Baseline)



Soil Behavior Type 2: Organic soils - peats, 3: Clays, 4: Silt mixtures - clayey silt to silty clay, 5: sand mixtures - silty sand to sandy silt, 6: Sands - clean sand to silty sand, 7: Gravelly Sand. *Based on Robertson and Fear (1995)

Figure 70. Waterloo, IA (Liquefaction): CPT profiles for tests performed by pull-drive 4'/3' method using 20 ft mandrel(Waterloo, IA, CPT - 22 – Baseline)

Table 10: Waterloo, IA (Liquefaction): Summary results for three methods

Construction methods	Construction time (minutes)	Average q_c, tsf
4’/3’ method	5	128
4’/4’and 4’/3’ method	15	145
Pull-drive 4’/3’	10	123

Rochester, NH

Project Description

The project involved the design of IRAP to support the New Hampshire Route 16 Ramp F MSE wall located in Rochester, NH. The subsoil profiles were selected the boring logs (W2-125, W2-126 and W2-127) where the relative locations are illustrated in Figure 71. The locations of the post-installation boring logs are also shown in Figure 71. The boring W2-126, which was the closest to the post-installation boring log profiles, was selected to compare with the post-installation results. The piers were constructed on approximately 4.5 ft (1.5 m).

Subsurface Conditions

The subsoil conditions within the pier installation elevations, which were approximately from 148 ft to 168 ft (45 m to 51 m), were consisted of loose to medium dense fine sand trace silt. The SPT varied between 5 and 28 and had average value of 10.

Pier system

Hundreds of piers were installed to ground improvement. The diameter and height of the pier were designed about 21in. (0.53 m) and 24 ft (7.2 m), respectively. The spacing of the pier was 4.5 ft (1.5 m).

Tests and Results

The locations of the post-installation boring logs are also shown in Figure 71. The boring W2-126, which was the closest to the post-installation boring log profiles, was selected to compare with the post-installation results.

The results indicate that soil was significantly improved by the IRAP installation. The average N values increased from 10 to 24. Figure 72 shows the in situ SPT results.

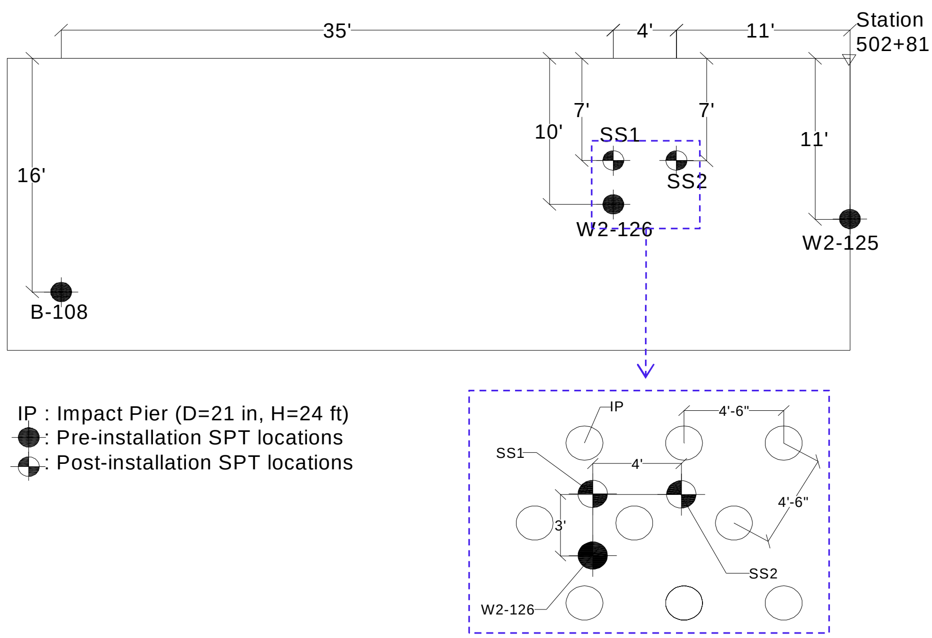


Figure 71. Rochester, NH: Piers and SPTs locations plan layout

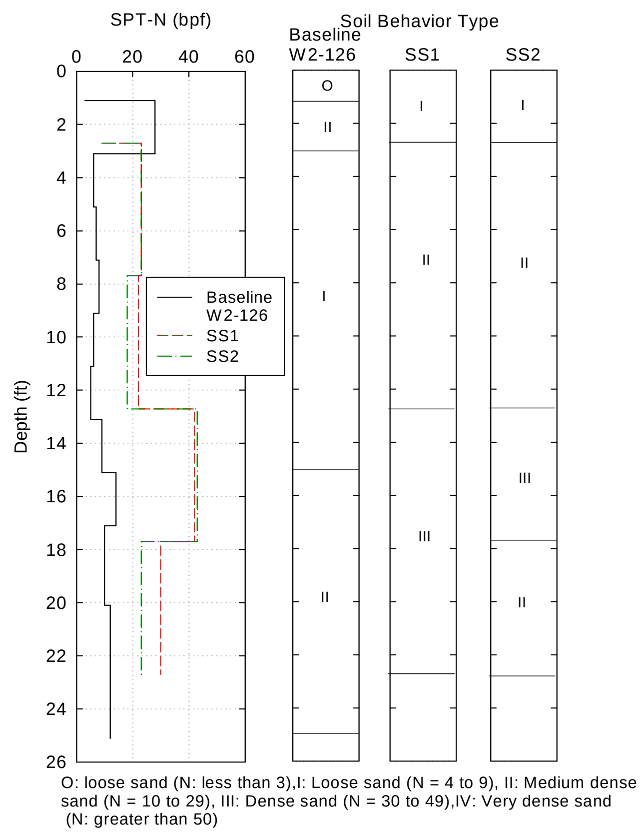


Figure 72. Rochester, NH: Pre-installation and post-installation SPT profiles

SUMMARY OF THE CASE STUDIES

The projects are including recently six years and from 11 states. The subsoil conditions varied from clean sands to clayey soils. Total six types of DAPs were conducted to the projects sites, especially, the IRAPs were constructed in total 14 projects sites. Diameters of piers varied from 20 in. to 30 in. (0.51m to 0.76 m). Heights of piers varied from 7 ft (2.1 m) to 39 ft (11.9 m). Spacing of piers varied from 4.2 ft (1.3 m) to 11 ft (3.4 m). And calculated area replacement ratio varied from 0.035 to 0.2. Several construction methods were used in the piers installations.

Modulus load tests were performed at ten project locations. SPTs were performed at eleven project locations before piers were installed and at five locations after piers were installed. CPTs were performed at six project locations before piers were installed and at twelve project locations after piers were installed. Table 11 summarizes the dates and subsurface conditions of the project sites; Table 12 summarizes the pier types, dimensions, and spacing of the projects; and Table 13 summarizes the kinds of modulus load tests, and pre-installation and post-installation tests by project.Table 14 summarizes the construction methods and tips during construction.

Table 11: Summary of dates and subsurface soil conditions by project

Project Location	Year	Subsurface Soil Conditions
Salinas, CA	2002-2004	Soft to firm clay, firm silt and sand fill to a depth of about 17 ft, underlain by soft to firm silt to a depth of about 22 ft, underlain by loose to medium dense sand
Minneapolis, MN	2004	Silty to clayey sand with oganics to a depth of about 2.5 ft, underlain by very loose to loose sand to depth of about 15 ft, underlain by medium dense sand
Lacrosse, WI	2005	Loose sand fill with construction debris (concrete and gravel fragments) to depths of about 10 to 16 ft, underlain by loose to medium dense sand
Manalapan, NJ	2005	Very loose sand to depth of about 15 ft, underlain by loose to medium dense sand
Reynolds, IN	2006	Clay fill to a depth of about 5 feet, underlain by loose to medium dense sand trace gravel to depth of about 17 ft, underlain by stiff to hard glacial till
Tampa, FL	2006	Sand stone fill to a depth of about 5 ft, underlain by sand with clay and shell fragments to a depth of about 15ft, underlain by silty to clayey sand
Seattle, WA	2006	Gravel sand layer to a depth of about 6 ft underlain by silt sand medium to a depth of about 14.5 ft, underlain by clays to silt medium and sand medium to sand to a depth of about 39.5 ft
Springfield, MA	2006	Loose to dense sand fill to a depth of about 14 ft, underlain by peat to a depth of about19 ft, underlain by loose to dense sand
Prince George County, MD	2007	Sand and silt mixture to a depth of about 35 ft
Waterloo, IA	2007	Fine sand trace clay and organics to a depth of about 2.5 ft, underlain by fine to medium sand trace clay and gravel to a depth of about 25 ft
Lynn Haven, FL	2008	Clayey fine sand to medium fine sand to a depth of about 18 ft, underlain by silty fine sand with mica
Jacksonville, FL	2008	Loose to dense fine sand to a depth of about 35 ft, underlain by weathered limestone to a depth about 45 ft, underlain by stiff to hard clay
Westminster, CA	2008	Silty clay to clayey silt with interbedded layers of loose to medium dense sand to a depth of about 35 ft
Oakland, CA	2008	Sandy soil to a depth of about 5 ft, underlain by clay layer to a depth of about 12 ft, underlain by sandy soil
Waterloo, IA (Liquefaction)	2008	Fine sand trace clay and organics to a depth about 2.5 ft, underlain by fine to medium sand trace clay and gravel to about 25 ft
Rochester, NH	2008	Loose to medium dense fine sand trace silt
Legend: 1 ft = 0.31 m.		

Table 12: Summary of pier types, dimensions, and spacing by project

Project Location	Pier Types	Pier Dimensions and Spacings
Salinas, CA	Geopier	D = 30 in, H=14 ft, about 11 ft on-center spacing (Ar*=0.04)
Minneapolis, MN	Pyramid pier, Geopier	D = 16 in at top and 5-3/4 in at bottom (Pyramid Pier), D = 18 in (IRAP), D=30 in (Geopier), H = 16 ft, and 4 ft on-center spacing (Ar=0.11)
Lacrosse, WI	IRAP	D = 20 in, H =15 to 30 ft, and 3.5 ft on-center spacing (Ar = 0.21)
Manalapan, NJ	Pyramid pier IRAP	D = 24 in at top and 8 in at bottom, H = 14 ft, and 8 ft on-center spacing (Ar =0.057)-[Pyramid pier]; 8 ft center spacing for impact pier
Reynolds, IN	TM RAP	D = 24 in, H = 16.6 ft, 4.5 ft on-center spacing (Ar=0.15)
Tampa, FL	IRAP	D = 20 in, H = 20 ft 6 ft 7 in to 6 ft 11 in on-center spacing (Ar=0.045)
Seattle, WA	IRAP	D = 20 in, H = 39 ft, and 6.3 ft on-center spacing (Ar = 0.072)
Springfield, MA	IRAP	H = 16 to 30 ft, 4.5 ft on-edge spacing (pier D information not available)
Prince George County, MD	IRAP	9.5 ft to 25.7 ft on-center spacing (pier D and H information not available)
Waterloo, IA	IRAP Rampact RAP Chain Mandrel	D = 24 in and 20 in, H = 7 to 14 ft, 4ft 8 in to 5 ft 3 in on-center spacing for group piers, and 12 ft 6 in to 18 ft 10 in on-center spacing for single piers (Ar = 0.089)
Lynn Haven, FL	IRAP	D = 20 in, H=10 ft, 4.5 to 10 ft on-center spacing (Ar = 0.048)
Jacksonville, FL	IRAP	D=20 in, H= 20 ft, 6 ft 7 in to 7 ft 10 in on-center spacing (Ar=0.035)
Westminister, CA	IRAP	D = 24 in and 30 in, H= 25 ft, 4.2 ft to 6 ft on-center spacing (Ar = 0.144 and 0.225)
Oakland, CA	IRAP	7 ft on-center spacing (pier D and H information not available)
Waterloo, IA (Liquefaction)	IRAP	D = 22 to 30 in, H=15 ft and 20 ft, 8 ft on-center spacing (Ar=0.065 and 0.089))
Rochester, NH	IRAP	D = 21+ in, H = 24 ft, and 4 ft 6 in on-center spacing (Ar=0.15)

Legend: D = Diameter. H = Height. Ar = area replacement ratio. 1 ft = 0.31 m; 1 in. = 2.54 cm.

Table 13: Summary modulus load tests, pre-installation, and post-installation tests by project

Project Location	Modulus Load Test	Pre-installation	Post-installation
Salinas, CA	Yes (UL)	SPT	CPT
Minneapolis, MN	Yes (TT)	SPT	CPT [0]
Lacrosse, WI	Yes (Compared to Full Scale Footing Load Test)	SPT, CPT	SPT, CPT [0-1]
Manalapan, NJ	Yes (TT)	SPT	CPT [U]
Reynolds, IN	Yes (TT)	SPT	SPT [1]
Tampa, FL	Yes (TT)	SPT	SPT [U]
Seattle, WA	Yes	CPT	CPT [U]
Springfield, MA	No	SPT	CPT [U]
Prince George County, MD	No	CPT	CPT [U]
Waterloo, IA	Yes (TT) (Compared to Full Scale Footing Load Test)	CPT	CPT [19]
Lynn Haven, FL	Yes (TT)	SPT	CPT
Jacksonville, FL	Yes (TT)	SPT	SPT
Westminster, CA	No	CPT	CPT [14]
Oakland, CA	No	—	CPT [U]
Waterloo, IA (Liquefaction)	No	CPT SPT	CPT [12]
Rochester, NH	No	SPT	SPT [U]
Legend: UL = up lift test; TT = telltale; U = number of days post-installation is unknown			

Table 14. Summary of IRAP construction characteristics

Project Name	Construction characteristics
Minneapolis, MN	• The number of lifts to construct pyramid piers in a group (4 piers) was related with the construction order. • Construction of the first pier required 7 lifts and spent 2.5 minutes penetration. • Construction of the second pier required 4 lifts and spent 49 seconds penetration. • Construction of the third pier required 3 lifts and spent 36 seconds penetration. • Construction of the forth pier required 2 lifts and spent 15 seconds penetration.
Lacrosse, WI	• The nominal size of open graded aggregate (2 to 2.5 inches) is too large to free flow through the mandrel and hopper to the bottom of mandrel. • 1 inch size open graded aggregate was satisfied to flow rate for IRAP construction. • After construction of several piers in group, the subsequent piers may be difficult to penetrate to the design depth.
Springfield, MA	• During constructing of IRAP, the difficulties with the flow of ¾ inches to 3/8 inches aggregate was observed within a sand fill layer above the peat soil at the site. • Using water through the mandrel to help the aggregate to flow to the mandrel bottom. • It is verified that the pier strength and stiffness was acceptable by prior adding water.
Westminster, CA	• Single pass (3 ft up and 2 ft down) and double pass (3 ft up, 3ft down, and then 3 ft up and 2 ft down) were employed to research purpose. • The double pass methods induce larger diameter (30 inches) than single pass method (24 inches). • The improvement efficiency of double pass method is slightly greater than that of single pass method.
Waterloo, IA (Liq.)	• Construction methods included: 4’/3’; 4’/4’ then 3’/2’ double compaction; 4’/3’ with initial loose aggregate. • 4’/3’ method took about 5 minutes to construct and achieve design requirement. • 4’/4’ and 3’/2’ method took about 15 minutes to construct and exceeded design requirement. • 4’/3’ drive-pull and re-drive method took about 10 minutes and provided no benefit compared to the single 4’/3’ method.

CHAPTER 4: RESULTS

As descript in the back ground chapter, matrix soil improvement and soil engineering properties can be estimated from the CPT data and SPT data. Results show that matrix soil improvement is affected by fines content, initial relative density and pier spacing. Based on the individual project test results, this chapter will descript the analysis of the data and a summary of the results that support these findings.

CPT Data

Most of the CPT data base provided the tip resistance, sleeve friction and pore water pressure (Table 15). It can be seen from the Table 15 that some projects have a large number of CPT profiles while other projects were limited too few CPT profiles due to the differential pier height and the interval of CPT data. The analysis results biased toward the projects contain more data. The tip resistance values were sensitive for small variations in stratigraphy and misalignment. That means that the more complicate soil stratum has larger bias, such as multiple soil layers, differential elevations. The average value of the CPT tip resistance in one foot depth increment to represent one data was employed to reduce these biases for some analysis. However, the heights of the piers are different for different projects. The results biased toward longer piers.

Table 15: Summary results of CPTs

Project Name	**CPT Soundings**	**Intervals (ft)**	**Depth of pier (ft)**	**No. of data / profile (before average)**	**No. of data / profile (after average)**
Salinas, CA	Corrected N_{60}	0.5	17	34	17
Minneapolis, MN	q_c	0.065	14	215	14
Lacrosse, WI *	q_c, f_s and u	1	28	8 (refusal)	8
Manalapan, NJ *	q_c, f_s and u	0.32	14	44	14
Tampa, FL *	q_c, f_s and u	0.16	20	125	20
Seattle, WA *	q_c, f_s and u	0.16	39	244	39
Prince Geo. Co,, MD *	q_c, f_s and u	0.16	24	150	24
Waterloo, IA *	q_c, f_s and u	0.16	14	88	14
Lynn Haven, FL	q_c	1	9	9	9
Westminster, CA *	q_c, f_s and u	0.16	25	156	25
Waterloo. IA (Liq.) *	q_c, f_s and u	0.16	15	93	15

Legend: q_c= CPT tip resistance. f_s= sleeve friction, u = pore water pressure; * = can be obtained matrix soil fines content

Notes and Observations from CPT Results

To reduce the bias of the data analysis, the following methods were used in the data collection and analysis.

- The same depths from the CPT profiles were used to analyze the improvement with distance. The elevation of some tests had to be adjusted to provide matching results (Figure 73a).
- If the CPT data did not include a pre-installation test, the test profiles which were influenced least by the DAPs were used as the baseline. If two CPTs had a relative large distance (more than 5 ft), elevations were considered by matching soil strata (Figure 73b).

- In some cases, the CPT tip resistance values were lower than expected. This may be due to ground disturbance or possible adjacent CPT hole. It has been reported that tip resistance values will be influenced if the distance between two tests is closer than 10–20 cone diameters (1–2 ft) (Lunne et al. 1997). In some cases, the tip resistance values increase due to penetration of the edge of the pier. (Figure 73c and d)
- In some cases, for highly variable interbedded soil profiles with stiff layers thinner than 2.5 ft, the tip resistance values may be underestimated due to the transition from one layer to another layer (Vreugdemhil et al. 1994). The peak values of the layers are less influenced by the transition. It is better to use the peak values to represent the stiff layers conditions.

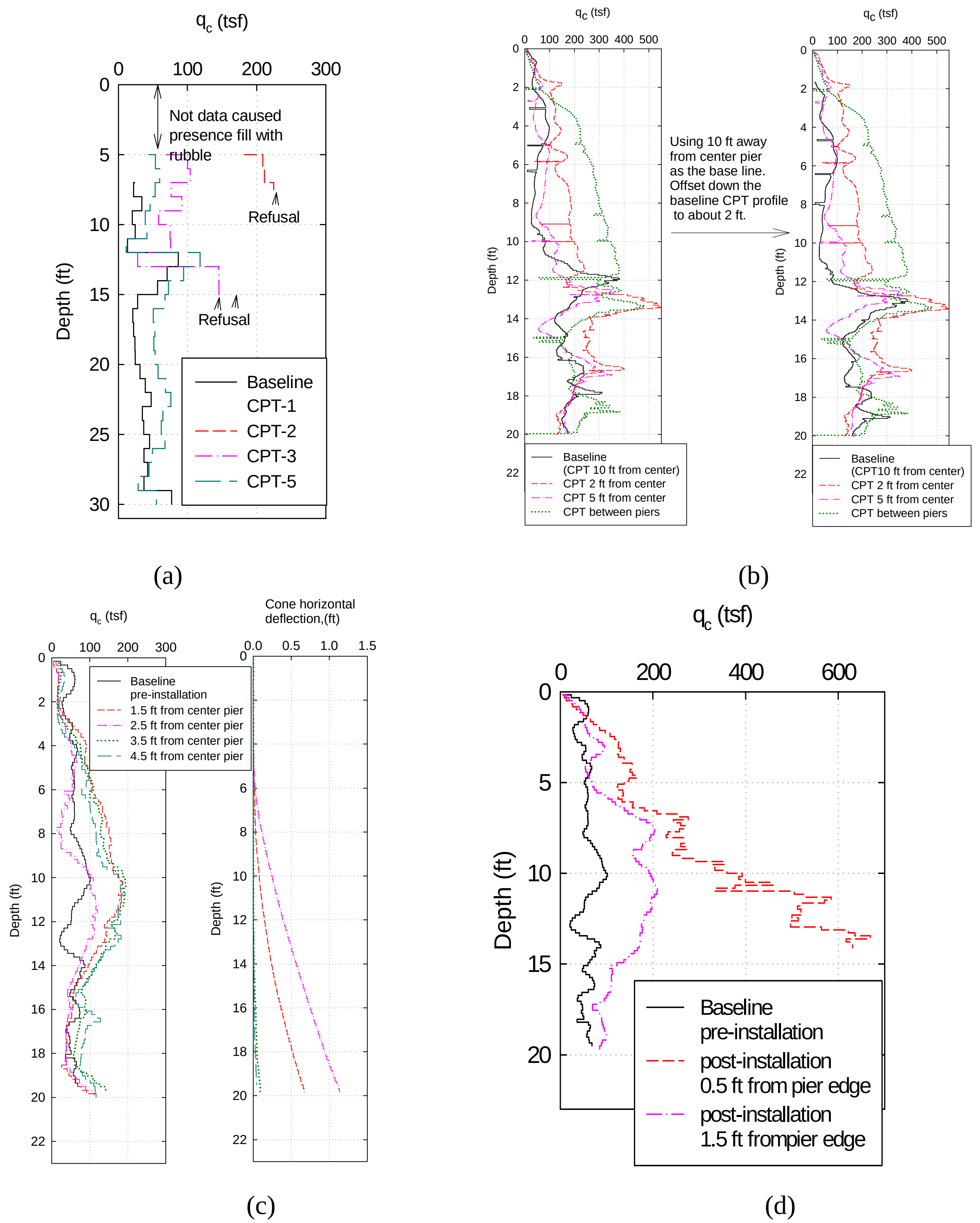


Figure 73. Illustrations of data collection: (a) same depth data comparison; (b) adjusted for elevation; (c) penetrating verticality; (d) potential penetration of the edge of a pier

DATA ANALYSIS

General Analysis

Overall, the key observations from case histories are listed as follow: (1) several projects indicate that the surface soils to the depth of about 1–2 pier diameters did not have much improvement; (2) most projects indicate the overburden pressure or depth was not a significant factor to influence the matrix soil improvement at elevations deeper than 1 to 2 diameters; (3) some projects show the soil improvement can be achieved as deep as 2 diameters beneath the pier bottom.

Summary of CPT and SPT Data

SPTs were performed at five of the sixteen project sites before and after piers were installed to investigate the matrix soil improvement. Figure 74 shows the SPT N-values profile which was combined the pre-installation and post-installation SPT N-values profiles from the five sites. The post installation SPT N-values profiles were selected within the pier groups. The mean values of SPT N-values increased from 9 to 20 after DAPs were installed. The mean values of SPT N_{60} values, which were calculated from CPT data, increased from 11 to 21 after DAPs were installed. Figure 75 shows the statistical summary of the SPT N-values and calculated SPT N_{60}-values. The cumulative curves in Figure 75a are approximately parallel starting from N-values around 5. That means the N-values increased approximately equal from N-values larger than 5. It similar trends were observated for N_{60} values larger than 7 from Figure 75b.

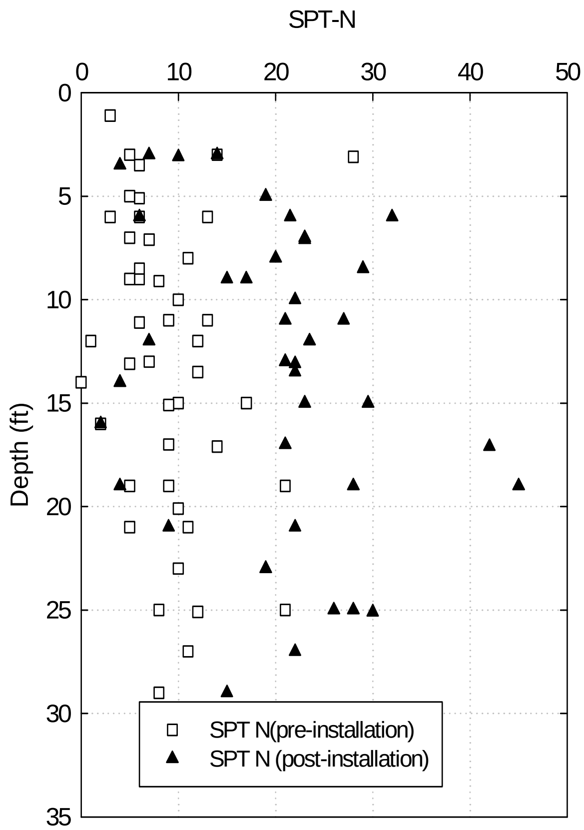


Figure 74. SPT N-values profiles from several project sites (Lacrosse, WI; Reynolds, IN; Tampa, FL; Jacksonville, FL; and Rochester, NH)

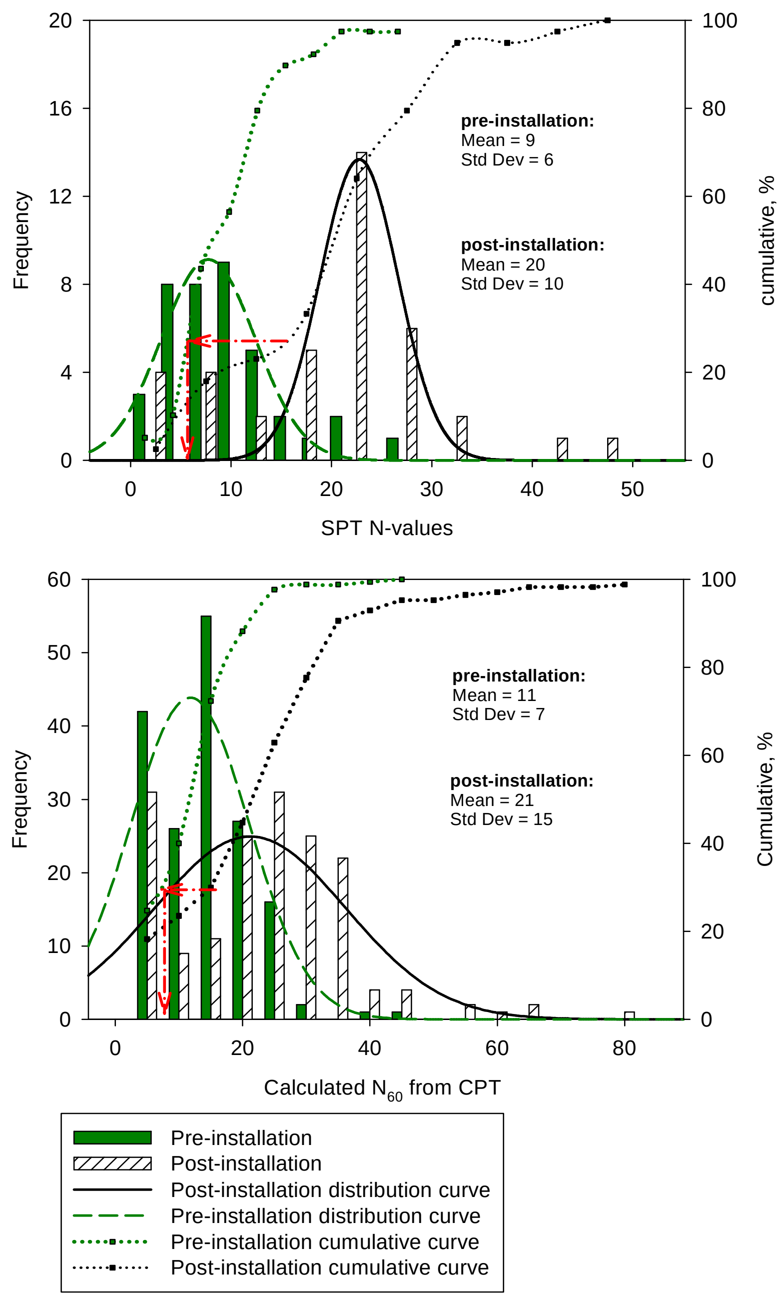


Figure 75. Summary of SPT N-values and calculated N_{60} values from CPT for all available case histories

Determining the Effective Soil Improvement Zone

CPTs were performed in matrix soils at only 11 out of 16 sites. Table 16 summerizes the CPT information for these 11 sites. Three sites did not have sleeve friction measurement and at one site the cone tip refusal was encountered. Two of the cases were performed at the same site (Waterloo, IA) and were later combined. The CPT data from the resulting six sites were used to predict and generate the relationship between soil types and the effective improvement zones for piers groups (Table 16). Figure 76shows the summary of the tip resistance and friction ratios in pier groups with 2 ft (0.65 m) between individual piers. Figure 77a shows the initial CPT data for two catalogues, which are effective improvement and non effective improvement, on the Robertson's soil classification chart. The effective improvement is defined as the Δq_c values larger than 1 MPa (11tsf) after DAPs installation. Figure 77b shows the effective improvement zone that was generated from Figure 77a.

Table 16: Summary results of CPTs from 11 sites

Project Name	CPT Soundings	No. of data / profile	Comments
Salinas, CA	Corrected N_{60}	34	Not CPT data profiles (only within group)
Minneapolis, MN	q_c	215	CPTs were performed within and outside group
Lacrosse, WI	q_c, f_s and u	8 (refusal)	CPTs were performed within and outside group, but CPTs were refusal within group
Manalapan, NJ *	q_c, f_s and u	44	CPTs were performed within and outside group
Tampa, FL *	q_c, f_s and u	125	CPTs were performed within and outside group
Seattle, WA *	q_c, f_s and u	244	CPTs were performed within group only
Prince Geo. Co,, MD *	q_c, f_s and u	150	CPTs were performed within and outside group
Waterloo, IA * +	q_c, f_s and u	88	CPTs were performed within and outside group
Lynn Haven, FL	q_c	9	CPTs were performed within and outside group, but it is the estimated values
Westminster, CA *	q_c, f_s and u	156	CPTs were performed within group only
Waterloo. IA (Liq.) +	q_c, f_s and u	93	CPTs were performed within group only.
Legend: * : CPT data are used to the determine the effective improvement zone; +: have the same initial soil profile			

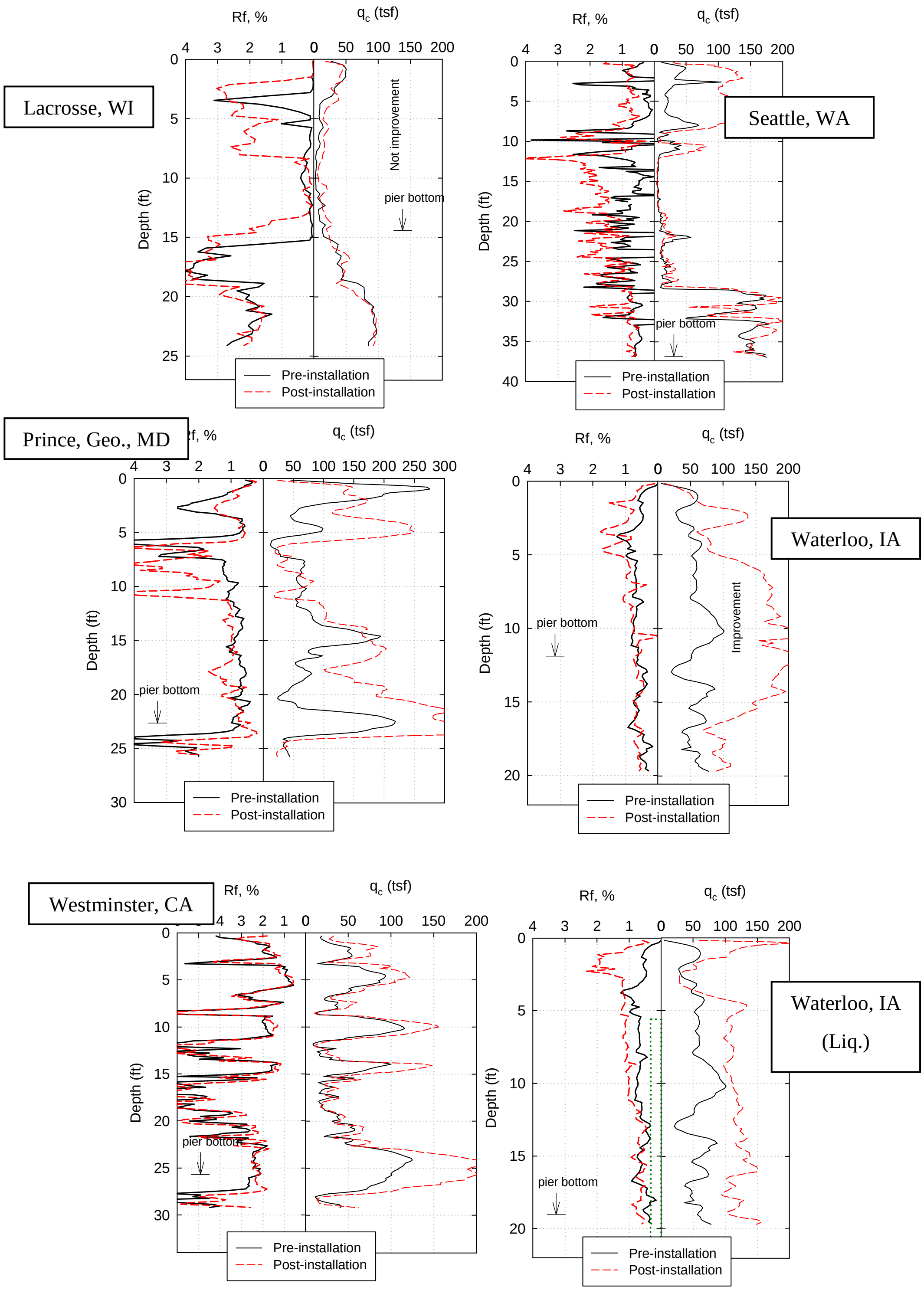
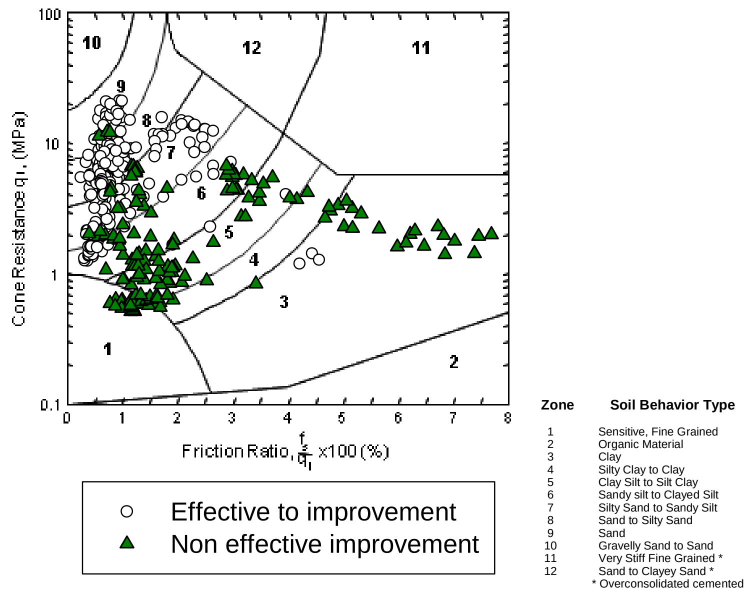
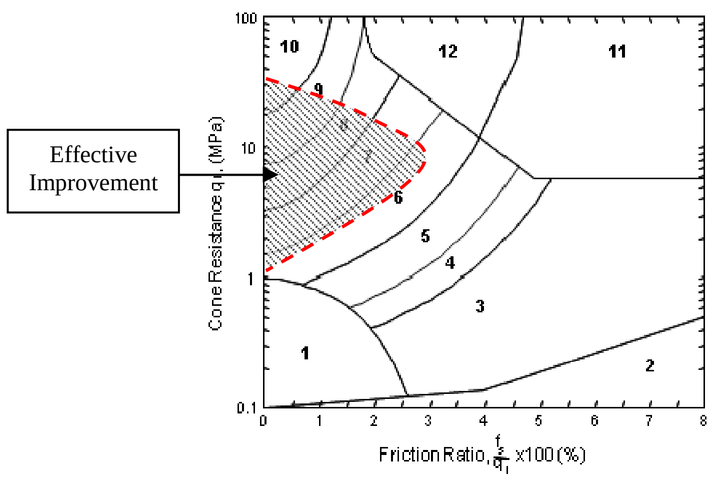


Figure 76. Summary of friction ratios and tip resistance as illustrations of soil improvement or nonimprovement



(a)



(b)

Figure 77. The effective improvement data and non effective improvement data shown in Robertson's classification chart (a) and the effective improvement region (b)

Determining Matrix Soil Improvement from Friction Ratios

Figure 77 indicates that the effective improvement soil is likely to relate with both the tip resistance and friction ratio. However, most of the effective improvement soils are concentrated in low friction ratio zones. Friction ratios may be used to distinguish the effective improvement soils from non effective improvement soils. Figure 78 shows the correlation between increasing tip resistance and the friction ratio. The negative values of increment of tip resistance may be due to the differential elevations and ground modification. The results indicate that the soils with friction ratios that are less than 1% can be significantly improved.

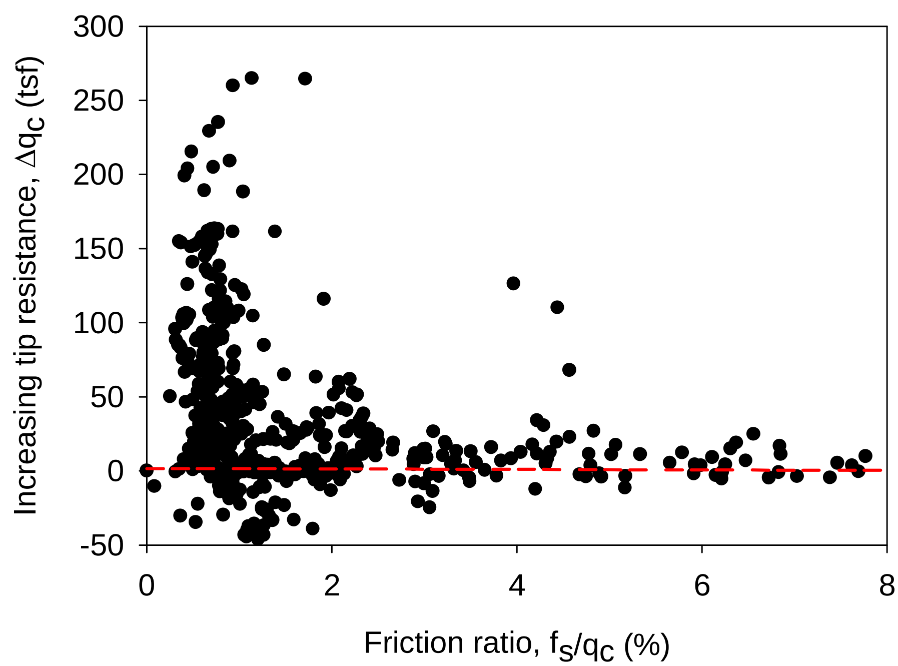


Figure 78. The increasing tip resistance with regard to friction ratio for all available case histories

Determining Matrix Soil Improvement from Fines Content (FC)

Previous research has indicated that the FC of the matrix soil influences the degree of improvement for compaction and displacement methods. For example, Hussin and Ali reported that the effective FC for vibro-technologies was less than 12 % (1983), and Aboshi et al.(1991) reported that the soil improvement was reduced with FC greater than 20%, but when sandy soil had less than 20% FC it was not possible to estimate the reduction in soil improvement (Aboshi et al. 1989). Figure 79 shows the relationship between SPT N-values and FC from the study of Aboshi et al. (1991). Slocombe et al. (2000) found that FC affects soil radial densification, and for the deep compact method, soils show no obvious improvement for FC greater than 15%.

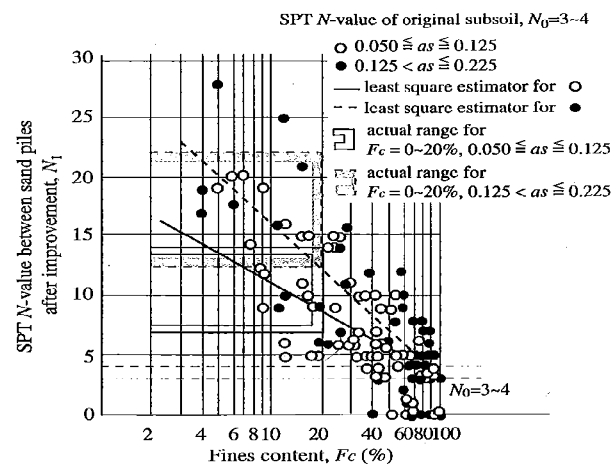


Figure 79. Relationship between SPT N-values and fines content (from Aboshi et al. 1991)

The results from this study indicate that the soil is significantly improved for FC of less than 20%, and that soil improvement will reduce when the FC is greater than 20%. However, in sandy soil with less than 20% FC, it is difficult to estimate the reduction in improvement. In some of the cases in this project where the FC was less than 20%, ground conditions induced different amounts of matrix soil improvement are listed as follow:

- Although the FC is less than 20%, the CPT results indicate the tip resistance in the very loose sandy layer does not increase as much as in denser layer. That indicates that the initial relative density somewhat affects the degree of improvement.

- It appears that the sandy soil with clay does not significantly improve in some cases, which means that the type of fines affects the degree of improvement.
- The calculated FC was underestimated for the unimproved soil layers.

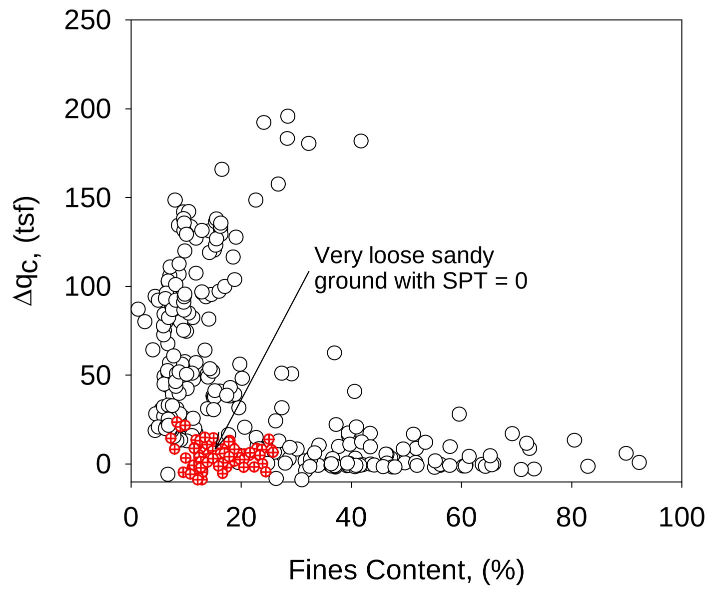


Figure 80. The increase tip resistance versus calculated fines content from CPT for all available case histories

Soil Behavior Types

The method for classifying soil behavior types proposed by Jefferies and Davis (1997) was used in this study. The soil behavior types in the cases in this study ranged from 3 (Clays) to 6 (Clean Sand to Silty Sand). The soil behavior types were classified based on CPT soundings obtained before IRAPs were installed. In this study, the amount of increasing tip resistance (Δq_c) tended to decrease as the soil behavior type index increased, and it appeared that the clean sand to silty sand soils were significantly improved; silty sand to sandy silt soils were improved to a lesser degree; and clayey soils showed no improvement. (Figure 81).

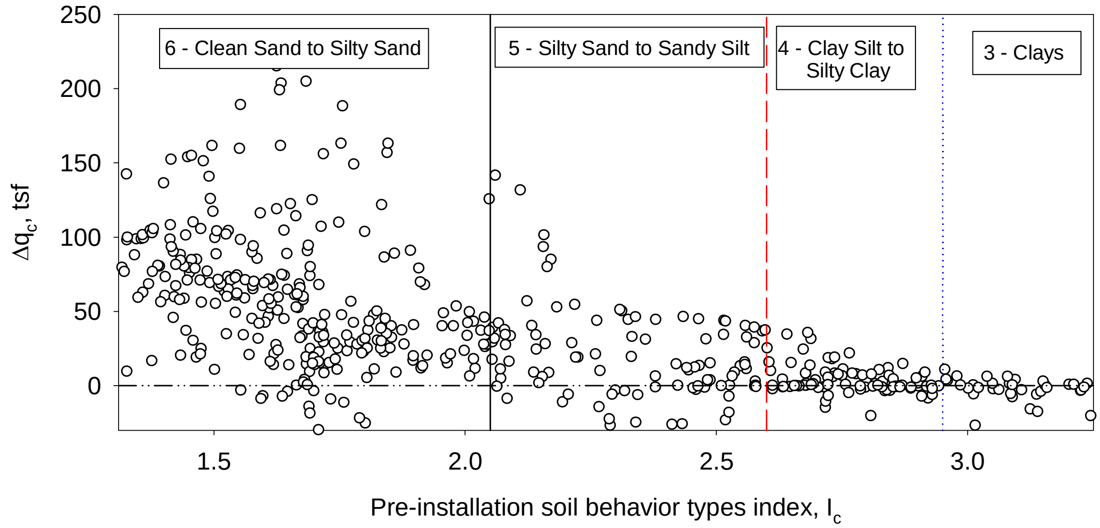


Figure 81. Increases in tip resistance compared with soil behavior types in pier groups for all available case histories (following classification method proposed by Jefferies and Davis 1997)

Statistical Summary of Sandy Soil (FC<20%) Improvement

Previous studies indicated that the sandy soils can be effective to improvement. This category will use statistical tools to summarize the improvement in sandy soils (FC<20%) from the case histories data base.

Improvement Index

The direct estimate of the soil improvement can be obtained by comparing pre- and post-installation SPT-N vales or CPT tip resistance. Using CPT tip resistance, Dove et al. (2000) defined an improvement index I_d, as:

$$I_d = \frac{q_{c,after}}{q_{c,before}} - 1 \qquad (20)$$

Schaefer and White (2004) indicated that this improvement index could alternatively be based on any in situ quality control measurement technique, where the specific soil property is measured before and after improvement. Using SPT N-values, I_d will be defined as:

$$I_d = \frac{N_{after}}{N_{before}} - 1 \tag{21}$$

Figure 82 shows the statistic summary of the sandy soils (FC<20%) improvement index values from CPT results. The peak value of the predicted normal distribution curve of the improvement index of the sandy soils with FC less than 20% is 0.62. Figure 83 shows the statistic summary of the improvement index values from SPT results. The peak value of the predicted normal distribution curve of improvement index from SPT is 1.2. The sandy soils improvement indices for individual projects are summarized in Table 17.

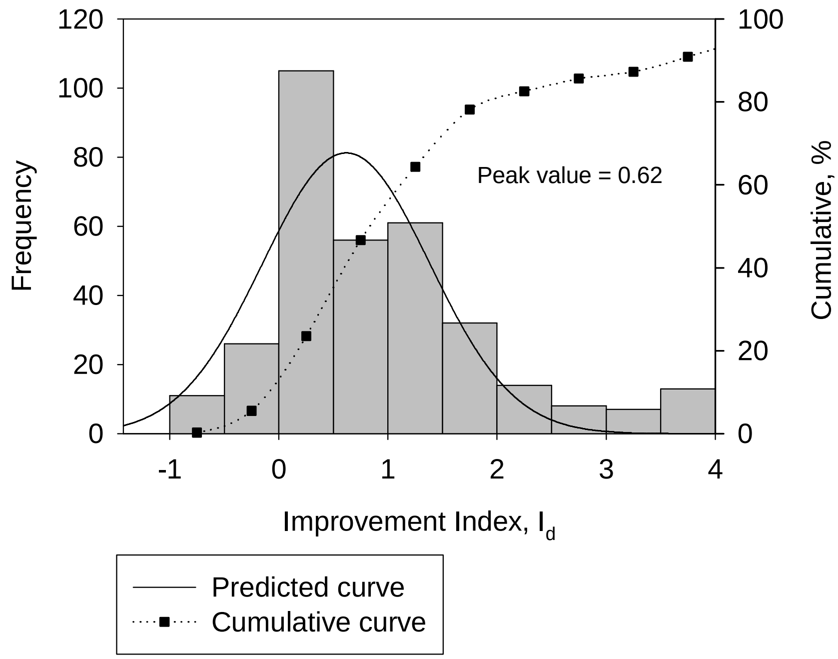


Figure 82. Statistic summary of improvement index (I_d) from (q_c) for sandy ground with calculated fines content less than 20% for available case histories

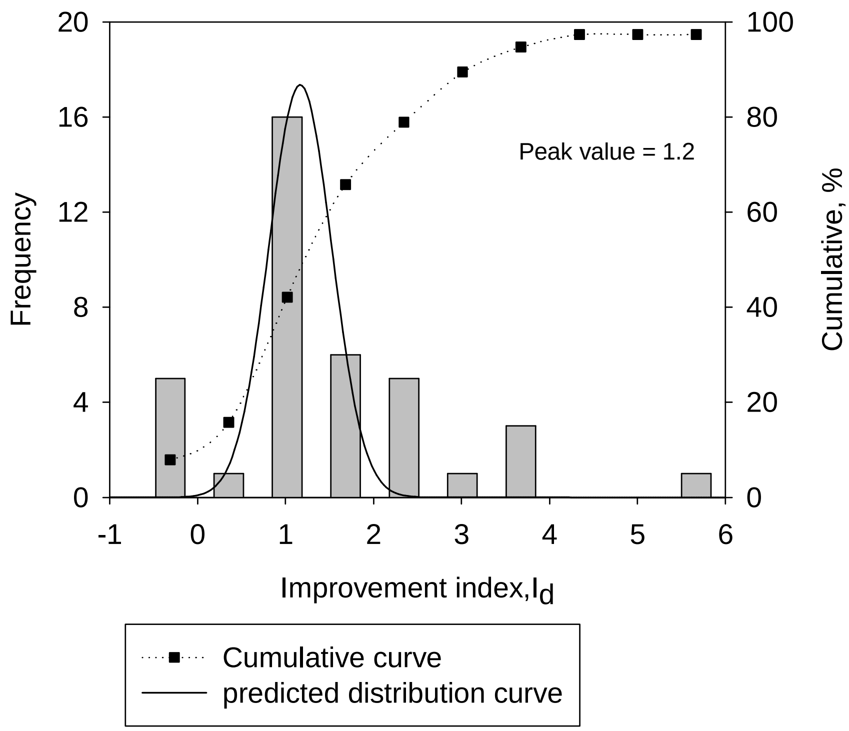


Figure 83. Statistic summary of improvement index (I_d) from SPT N-values for sandy soils for all available case histories

Group Effective Factors

CPTs were performed on the matrix soil both within and outside pier groups in some projects sites to estimate the group effective factor for those sites. The group effective factor (f_g) is defined as:

For CPT,

$$f_g = \frac{\Sigma\, q_{c,in\ group\ pier}}{\Sigma\, q_{c,at\ same\ distance\ outside\ group\ pier}} \text{ or } \frac{q_{c,in\ group\ pier}}{q_{c,at\ same\ distance\ outside\ group\ pier}} \quad (22)$$

For SPT,

$$f_g = \frac{\Sigma\, N_{,in\ group\ pier}}{\Sigma\, N_{\ at\ same\ distance\ outside\ group\ pier}} \text{ or } \frac{N_{,in\ group\ pier}}{N_{\ at\ same\ distance\ outside\ group\ pier}} \quad (23)$$

Table 17 summarizes the improvement index and group effective factors for individual project sites. The group efficiency factors varied from 1.14 to 1.8 and had a average value of 1.35.

Table 17. Summary of soil improvement index and group effective factor in sandy layers

Project Location	Soil Information	Improvement Index (I_d)	Group effective factor (f_g)
Cinemas, CA *	Loose to medium dense sand to silty sand — SM	1.67	—
Minneapolis, MN	Loose to medium dense sand — SP	1.35	1.25
Lacrosse, WI *	Loose to medium dense sand — SP (FC: 5%-25%)	1.45	1.2
Manalapan, NJ	Very loose silty sand — SP-SM (FC: 10%-35%)	0	—
Reynolds, IN *	Medium dense sand trace gravel — SW-SP	1.6	1.8
Tampa, FL	Very loose to loose silty sand — SM	0.25	—
Seattle, WA	Loose to medium dense sand – SP (FC: 5% - 20%)	2.4	—
Prince Geo. Co., MD	Loose to medium dense sand – SP-SM	0.57	1.14
Waterloo, IA	Loose to medium dense sand — SP (FC: 5% to 15%)	1.16	1.34
Lynn Haven, FL	Loose to medium dense sandy soil — SW-SP-SC	0	—
Jacksonville, FL *	Medium dense sand — SP	0.6	—
Westminster, CA	Loose to medium dense sand — SP (FC: 10%-80%)	0.6	—
Waterloo. IA (Liq.)	Loose to medium dense sand — SP (FC: 5% to 15%)	1.04	—
Rochester, NH *	Loose to medium dense sand trace silt – SP-SM	1.4	—

Legend: * : From SPT N values or calculated N_{60} values from CPT

Radial Densification

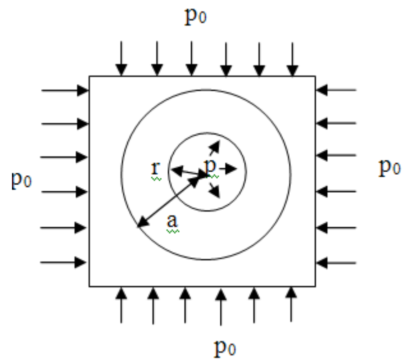


Figure 84. Cavity expansion modes

From the cavity expansion theory, either the radial stress or tangential stress reduces with the radial distance away from pier. For the elastic model, Yu (2000) reported the radial stress of cavity expansion could be represented in the form:

$$\sigma_r = -p_0 - (p - p_0)(\frac{a}{r})^2 \qquad (24)$$

$$\sigma_\theta = -p_0 + (p - p_0)(\frac{a}{r})^2 \qquad (25)$$

Similarly, the redial densification using CPT tip resistance could be expressed in the form:

$$q_{c1} = q_{c0} + q_{c0} \cdot (\frac{a}{r})^{-2} \qquad (26)$$

Where, q_{c1} = the post installation tip resistance values in matrix soil; q_{c0} = the pre-installation tip resistance values,

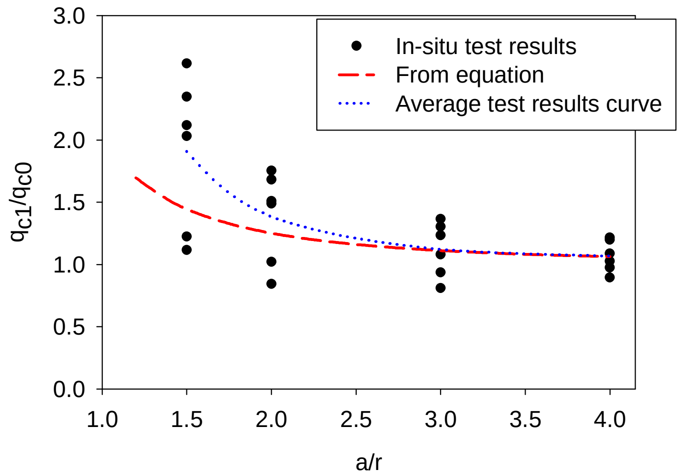


Figure 85. Radial densification from modified equation for sandy soils, in situ test results and in situ test average curves (Manalapan, IN and Waterloo, IA)

Relative Density

The relative density is an important parameter for sandy soil. It's relative with the sandy soil strength and liquefaction. CPT can be used to estimate the in situ relative density of sands. However, there are many factors will influence the calculated relative density results, such as overconsolidation ratio, particle size and aging (Skempton 1986, Kulhawy and Mayne 1990). Based on the comparison of the several equations provided on Table 4, the equation developed by Jamiokowski et al is used in calculations.

$$D_r = -98 + 66\log_{10}\frac{q_c}{(\sigma'_{vo})^{0.5}} \tag{27}$$

The effective overburden pressure (σ'_{vo}) of this empirical equation is determined by the unit weight, depth and groundwater table elevations. The relative density increases due to the decreased voids ratio. The voids ratio is relative large in the very loose to loose sand which is easy to lateral compress and reduce the void ratio. Since the aggregate is considered as placing the constant volume for each lift procedure, the very loose to loose sands induce same among densification by impacting same volume aggregate in some range of small initial relative density. The larger diameters of DAPs have been found in the loose sandy ground than in the dense ground (Figure 86). Because the aggregate tends to be more difficult to be impacted as many volumes in the dense sands as in loose sands due to the much higher lateral pressure required to change the same amount of void ratio (Figure 87). That may induce some volumes of aggregate to push back to the mandrel. Figure 88 shows the relationship of relative density between before and after DAPs were installed. The results indicates that the relative density increase about 20% for the sand with initial relative density less than about 40% (Figure 88: A to B). The relative density increases less in the dense sand than in the loose sand (Figure 88: B to C).

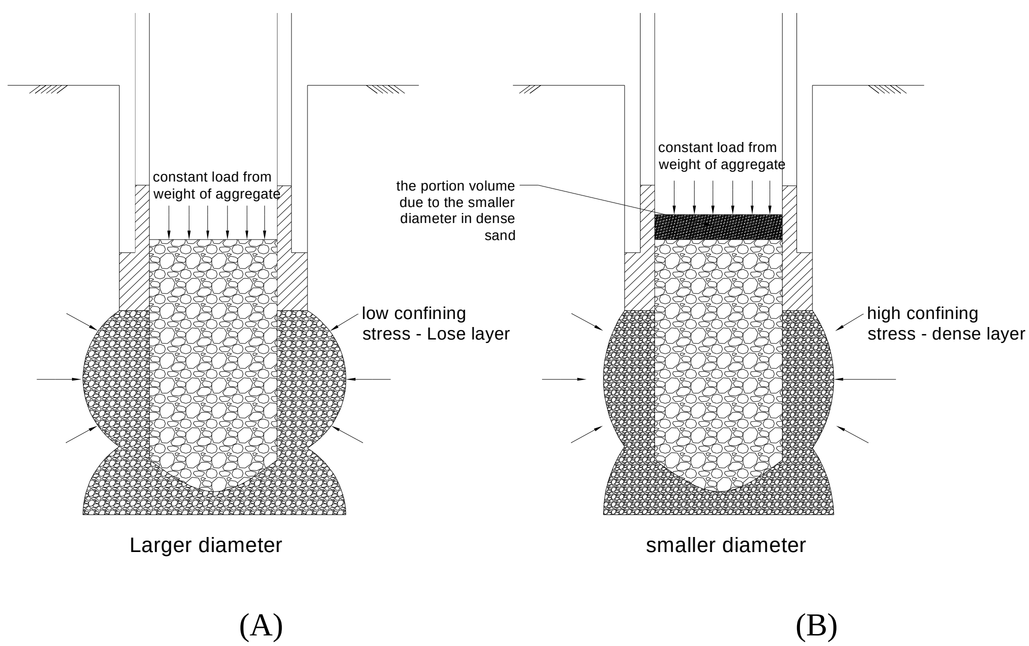


Figure 86. The construction profiles of constructing pier in the loose sand (A) and dense sand (B)

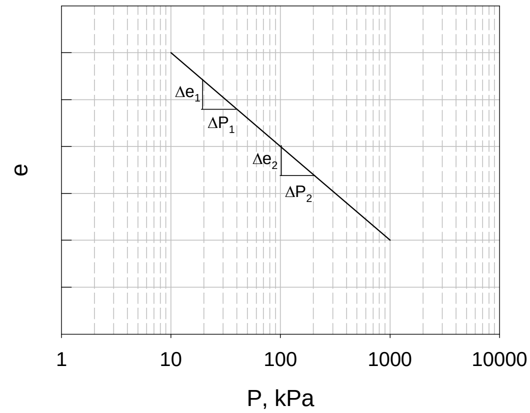


Figure 87. Typical e-log p curve of sand

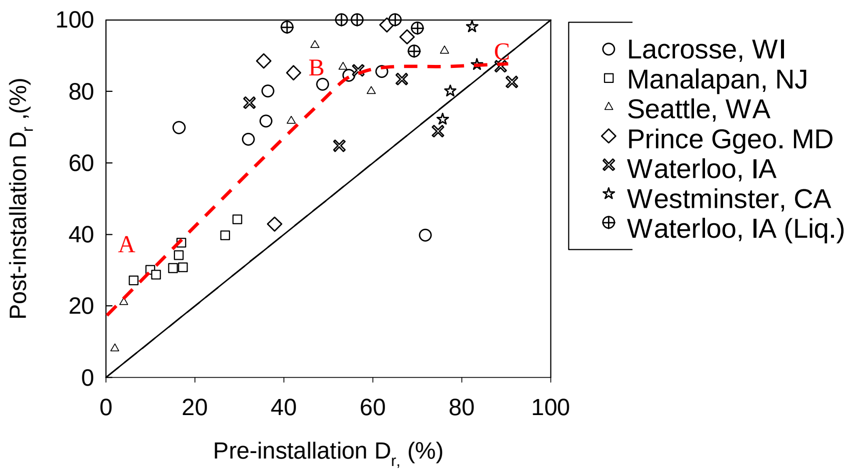


Figure 88. Comparisons of the relative density changes between pre-installation and post-installation at seven sites

Load Test Settlement

The load test results are related to the soil types and strength within depths of the upper 3 to 4 times diameters (Fox and Cowell 1998).Table 18 lists of the top soil types and the load test results are pointed out. The relationships between the settlements of individual IRAP and the ratios of design stress to average tip resistance within the top 3 diameters depth of matrix soil were shown in Figure 89 . This figure combines the design stress, stiffness of the matrix soil and deformation of individual pier under design stress. The results indicate the pier settlement was related with the matrix soil stiffness with the top 3 diameters depth. It may be used to give a quick approaximation of the design stress by giving settlement or predict the individual pier settlement under design stress.

Typically, bulging and tip movement during the load test contributes to the deformation of a single aggregate pier. Because the tip movement deformation can be easily obtained by using tell-tale, the amount of bulging deformation can be determined from the total deformation and tell-tale deformation. The term of settlement index is introduced here, which is defined as:

$$I_s = \frac{s_{i+1} - s_i}{S_{i+1} - S_i} \quad (28)$$

Where, s_i = total telltale settlement during ith load, s_{i+1}= total telltale settlement during (i+1)th load; S_i = total settlement during ith load; S_{i+1}= total settlement during (i+1)th load.

Increasing applied stress on the top of pier induces the total settlement increase. Assume the amount of total settlement increase from i^{th} loading to $(i+1)^{th}$ loading equals to the amount of tell-tale settlement (bottom settlement) from i^{th} loading to $(i+1)^{th}$ loading, the pier element will not contribute any settlement. The settlement index equals to 1 in this situation. Conversely, if the tell-tale settlement equals to zero, the total settlement will be totally contributed by the pier element. Between these two extreme conditions, the total settlement is contributed both bulging and tip movement.

Figure 90 shows the settlement index values with the applied stress for the case histories. The results indicated that the deformations of piers in the most sites were predominated contribution by bulging. The trends of the bulging and tip movement during loading appear to be to wave change for most case histories.

Table 18. Summary of modulus load test results

Project Locations	**Soil type with depth equal to 3 times pier diameter**	**Settlement at design load (in inches)**	**Settlement at 150% design load (in inches)**	**Position of load test pier**	**Comments**
Salinas, CA	Sand to silty sand — SM ($\overline{N_{60}}$ = 16)	0.2	0.33	—	Geopier
Minneapolis, MN	Sand ($\overline{q_c}$=65 tsf)	TP - 0.4	0.77	Individual	TP-pyramid pier; IP-impact pier; GP-Geopier
		IP - 0.34	0.54		
		GP - 0.22	0.34		
Lacrosse, WI	Sand, gravel fragment fill ($\overline{q_c}$=32 tsf)	0.24	0.35	Individual	—
		0.2	0.39	Pier group	
Manalapan, NJ	Silty sand ($\overline{q_c}$=11 tsf)	2.15	—	—	Assumed typical design stress :18.3 tsf
Reynolds, IN	Sandy soil ($\overline{N}$=8)	1.47	3.1	Individual	Design stress = 28 ksf
		1.31	2.4	Within group	
Tampa, FL	Silty sand ($\overline{q_c}$=59 tsf)	0.35	0.54	Individual	Design stress = 18.6 ksf
Seattle, WA	Silty sand to sandy silt ($\overline{q_c}$=91 tsf)	0.21	0.35	Individual	Design stress =18.3 ksf
Waterloo, IA	Sand ($\overline{q_c}$=50 tsf)	1.54	3.17	Individual	Design stress = 45.85ksf
Lynn Haven, FL	SW-SP-SC ($\overline{q_c}$=100 tsf)	0.19	0.35	individual	Design stress = 18.6 ksf
Jacksonville, FL	Sand ($\overline{N}$=10)	0.21	0.36	individual	Design stress = 28.2 ksf

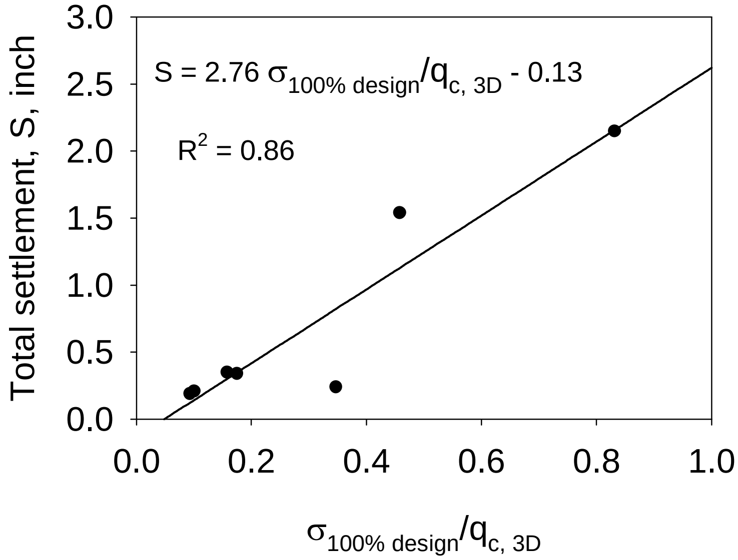


Figure 89. Relationship between total settlement and the ratio of design applied stress and pre-installation average $q_{c,3D}$ values ($\sigma_{design}/q_{c,3D}$) within the top 3 times diameters depth

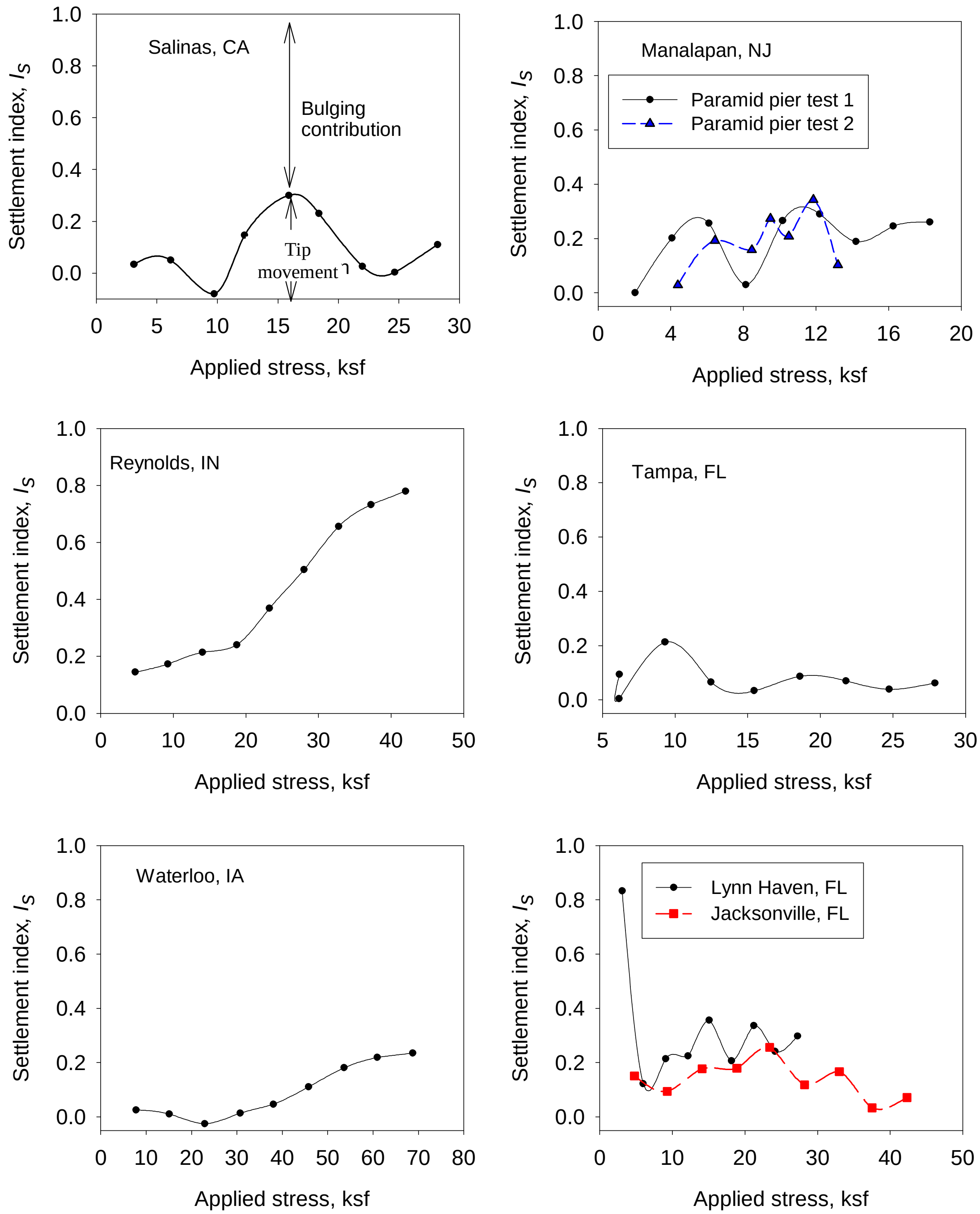


Figure 90. The settlement index values with the applied stress

CHAPTER 5: DISCUSSION OF RESULTS

This chapter is arranged into two parts, a discussion of matrix soil improvement and a discussion of the characteristics that influence the stability of DAPs during vertical loading.

MATRIX SOIL IMPROVEMENT

This section will present the conclusions of matrix soil improvement according to construction methods, area replacement ratios, pier group efficiency, and soil engineering parameter changes before and after pier installation.

Area Replacement Ratio

The area replacement ratio (R_a) was defined as the ratio of the sectional area of the DAP to the hypothetical cylindrical area. The DAP to footing ratio (R_f) was defined the ratio of the total pier area beneath the supporting footing to the footing area. Figure 91 indicated the different between area replacement ratio and the DAP to footing ratio. The area replacement ratio involved with the matrix soil improvement, however the DAP to footing ratio involved with the footing bearing capacity. The area replacement ratio was another factor to affect the efficiency of matrix soil improvement. Figure 92 shows the correlations between matrix soil improvement and the area replacement ratio.

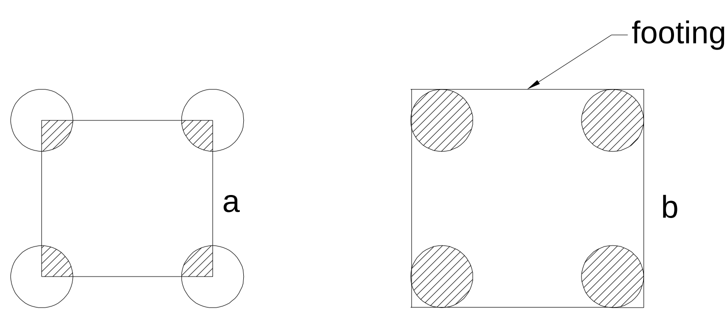
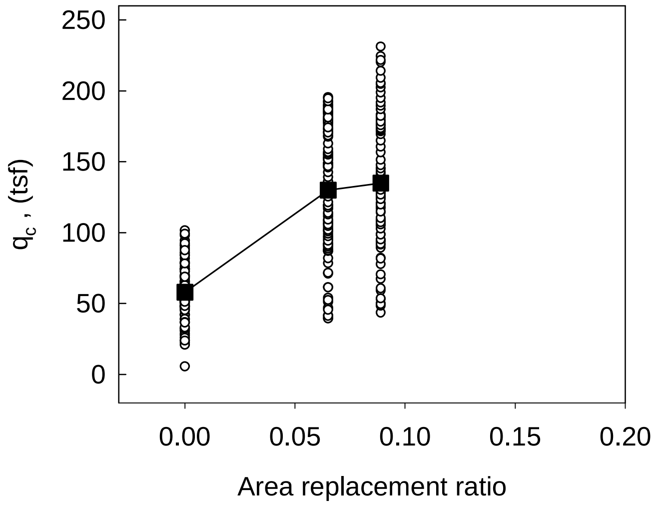
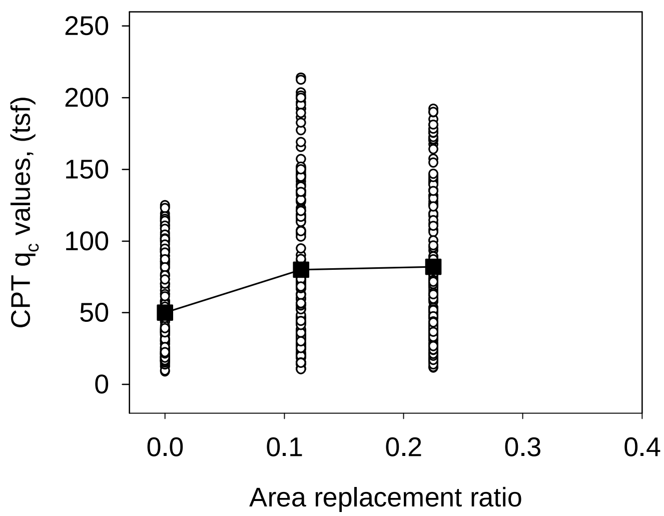


Figure 91. Schematic(a) the area replacement ratio and (b) the DAP to footing ratio



(a)



(b)

Figure 92. Illustration of the matrix soil improvement and area replacement ratio – (a) Waterloo, IA and (b) Westminster, CA (1 tsf = 0.096 MPa)

Several results may be induced by increasing area replacement ratio:

- Larger area replacement ratio may induce the greater ground heave.
- Larger area replacement ratio will increase the matrix soil excess pore water pressure that will against the partially lateral pressure during ramming action.

- Larger area replacement ratio may cause greater soil disturbance and remolding of the interbeded soil strata.

Matrix Soil Improvement within Pier groups

Overall, the sandy matrix soil improvement has been discussed as the improvement index in Chapter Four; however, the different locations in the group have the different efficiency of the degree of improvement. Figure 93 shows the improvement index and the locations within and outside the pier groups. Different ground conditions showed different efficiency improvement. The heterogeneous soil strata are more difficult to observe the improvement (Figure 93a and b) than relative homogeneous soil strata (Figure 93c). Figure 94 shows the idealized contour improvement field of the Waterloo, IA site, which consists of relative homogeneous fine sand soil profiles. The descriptions indicated the soil in the group is more improved than outside group, and also the soil element near the piers have larger degree of improvement. But this trend is difficult to identify for the heterogeneous soil strata.

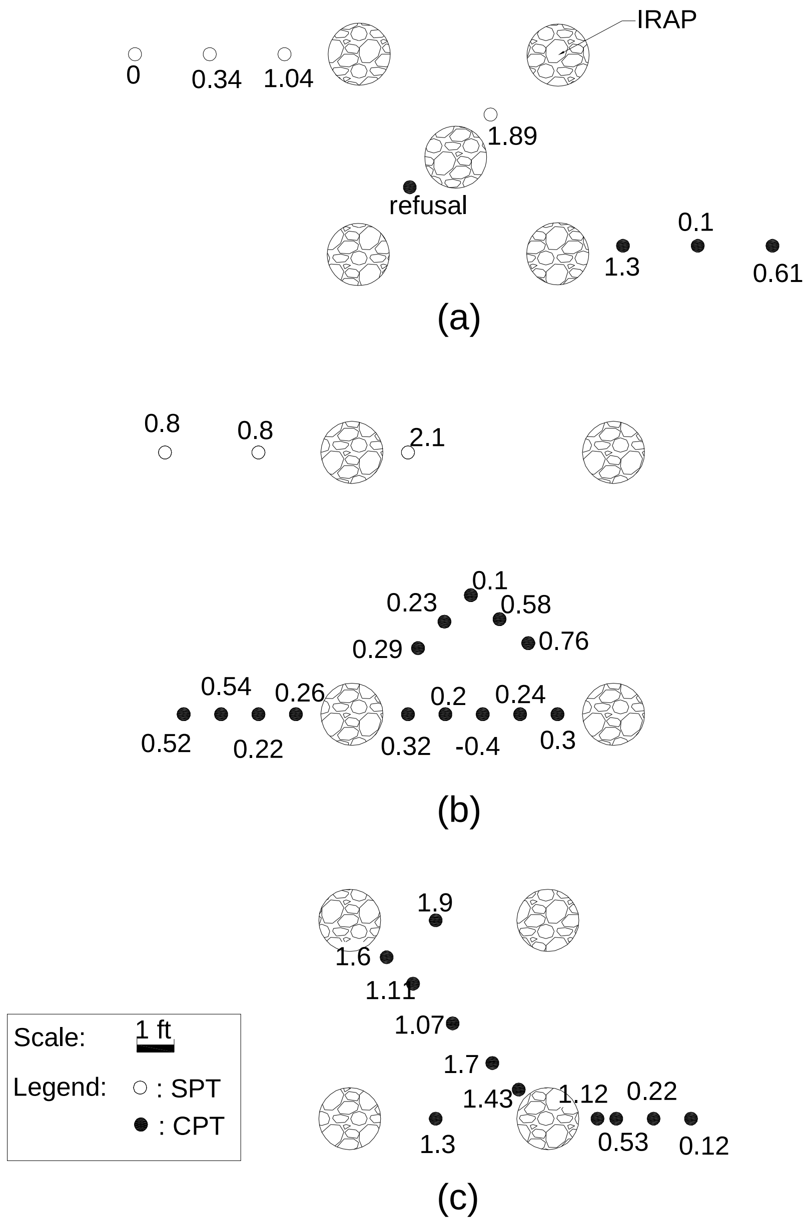
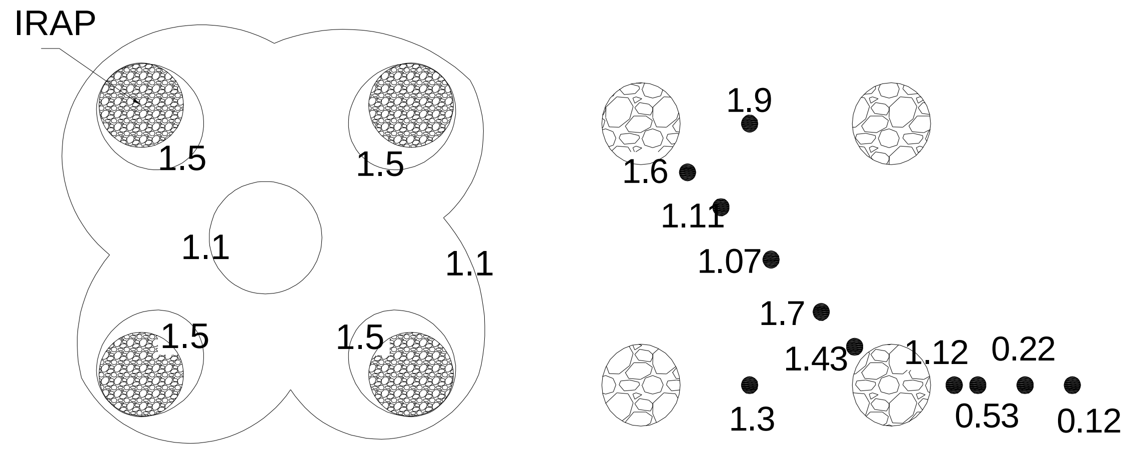


Figure 93. The improvement index (I_d) values of sandy soils at the vicinity of the pier groups in (a) Lacrosse, WI; (b) Prince George County, MD; and (c) Waterloo, IA (Note:I_d=$q_{c,after}/q_{c,before}$-1)



(a) (b)

Figure 94. (a) The idealized contour field of improvement index (I_d) of sandy ground and (b) the actural test results in Waterloo, IA (Note:I_d=$q_{c,after}$/$q_{c,before}$-1)

Soil engineering parameter changes between pre- and post-installation

To gain further insight into the matrix soil densification of DAPs, soil parameters of pre-installation and post-installation are used to compare between the pre-installation and post-installation in this section. The soil parameter profiles, which were calculated from CPT results, are attached in the Appendix II. The average values of several soil engineering parameters at the mid-depth and greater depth are summarized on Table 19. The assumption of normal consolidated soils with 120 pcf (1.92kg/liter) unit weight is used in the calculations. The results indicate that the friction angle of the sandy soils increase varied from 2 to 6 degree after the DAP installation. The undrained shear strength and friction angles of clayey soils were not significant increasing after DAP installation. Lateral stress coefficient and OCR increased for both sandy soils and clayey soils.

Table 19. Summary the matrix soil parameters for pre-installation and post-installation

Case	$\emptyset'$		S_u (tsf)		K_o		*OCR*
	Pre-	Post-	Pre-	Post-	Pre-	Post-	Post-
Riverside Center	31.5	37	-	-	0.48	1.21	6.3
Road O Crossing	25.5	27.5	-	-	0.57	0.83	3.94
Washington Liquor*	27.9	27.3	1.6	1.63	0.54	0.58	-
Chalk Point P.P.	32.3	38	-	-	0.52	2.3	65
Wagner Road	34	39	-	-	0.55	2.4	18
Moran Asian*	28	31	1.8	1.8	0.56	0.59	2.7
Wagner Road Liqf.	34	40	-	-	0.55	2.5	21

Note: *:obtained from silty or clayey soils

THE STABILITY OF DAPS DURING LOAD TEST

The DAP tends to lateral bulge out to the matrix soil and vertical tip movement into the ground during loading. The stiffness of matrix soil are affected the stability of the DAP. The first subsection is to discuss the stiffness of DAP and matrix soil from CPT data. And then, the performances of the single pier and group pier during loading are discussed. Finally, the effect of pore water pressure is analyzed in this section.

Stiffness of DAP and Matrix Soil from CPTs data

The various construction methods used to install DAPs affect the matrix soil improvement during mandrel penetration before ramming. The matrix soil improvemen can be separated to two phases. The first phase is similar to pile driving. The second phase is the lateral enhance by ramming action. Both phases will increase the preconsolidation pressure and reinforcing the matrix soil. The CPT and SPT are introduced to investigate the strength of piers and the strength of matrix soil. The method is performing the CPT and SPT on the DAP shaft and the surrounding matrix soil. Figure 95 shows the results of tip resistance of DAP shaft and matrix soil. The results indicate that the tip resistance of the piers is generally 2 to 3 times greater than the matrix soil, but the tip resistance of the piers in the peat zone is approximately equal to that of the matrix soil.

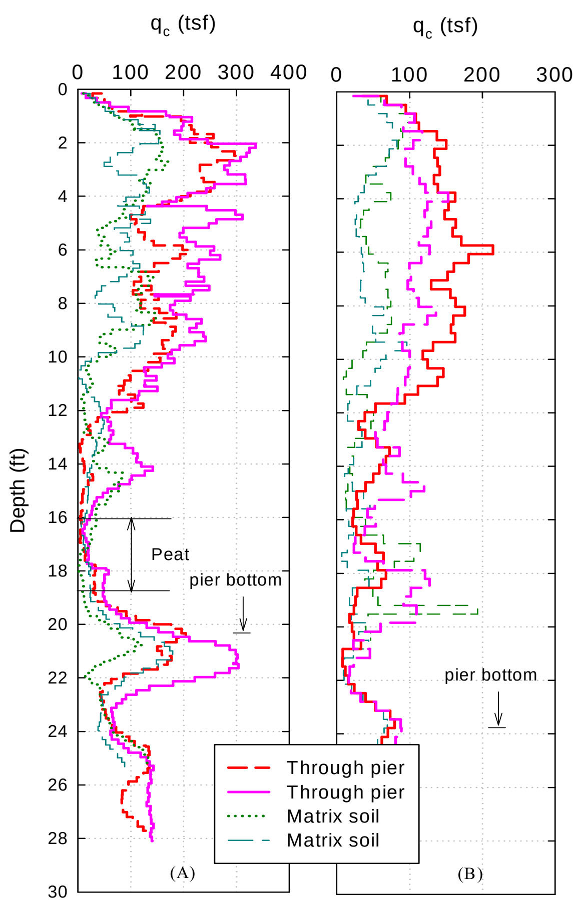


Figure 95. The CPT tip resistance results of the RAPs and matrix soil in (a) Tampa, FL and (b) Springfield, MA

Deformation of Single DAP during Loading

Since no tensile strength of the DAP during loading, the pier tends to lateral compress the soil and bulge in response to the load. The radial stress (σ_θ) of the soil increases and contacts against the aggregate pier lateral compression and bulging. The vertical compression stress is quickly decreasing along the pier. As discussion in the background, the most vertical stress will be dissipated to the soil within 3 diamters depth from ground surface for RAP. Figure 96 shows the settlement results and the soil strength from CPT tip resistance in the top 3 diameters deep.

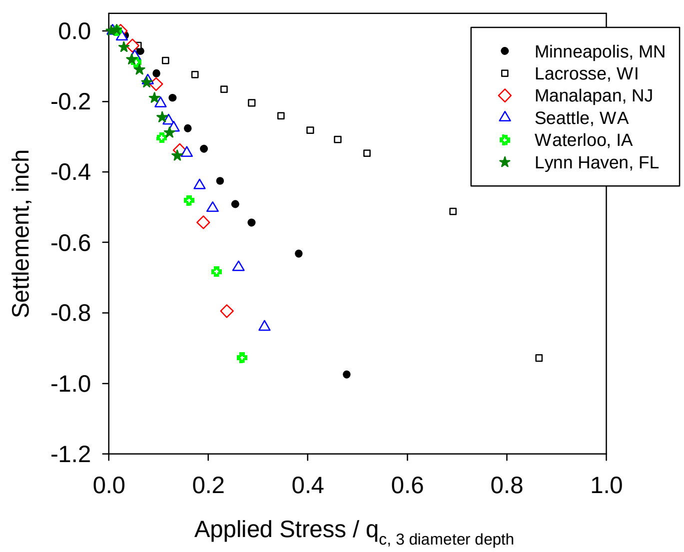


Figure 96. The results of combination in situ CPT results in matrix soil and single IRAP load test results

Deformation of Pier Groups

Both the single pier and pier groups load tests were conducted in the several project sites. The matrix soil under loading induces the additional lateral stress which may increase the resistance of the DAP. The interaction between the soil and pier element are marked. Stress cell recorded in another site indicated the stiffness ratio of the piers and the matrix soil tends to increase during load increasing (White et al. 2001). Comparison the results between full scale small size footing and large size footing load test results may recognize the difference between the larger scale group and small scale group. Figure 97 shows the load test results of single pier with cap, single pier footing and pier groups footing. The results indicate that the footing contained matrix soil will increase the bearing capacity. The pier unit in group tends to reduce the bearing capability.

The difference between the single pier and the single pier footing may be due to the matrix soil sharing partial load and the pier stiffness increasing caused by increasing the confining pressure. The load capacity of single pier with matrix soil in groups is lower than single pier footing, one reason may be due to that the uneven confining pressures, which tend the pier to slide, and the boundary conditions are different in group.

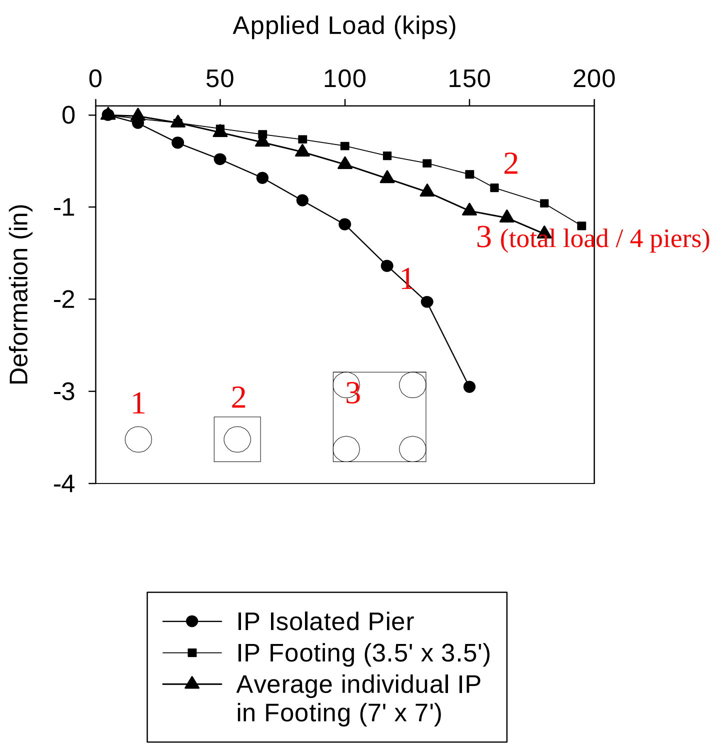


Figure 97. Load tests results indicating single pier unit in Waterloo, IA site

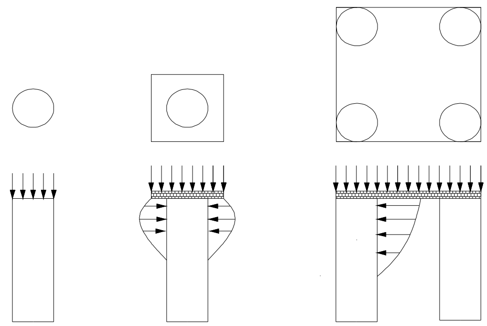


Figure 98. Illustration the pier-soil interactions beneath the footing

Pore Water Pressure in Piers and Matrix Soil

High pore water pressure will be generated in the surrounding soil during the ramming action that may induce the temporary liquefaction of saturated soil (Handy and White, 2006). After the pore water dissipates, the total stress decreases as the effective pressure increases, which may increase the interaction between the pier and matrix soil and increase the stiffness of the pier (Figure 99). The relatively greater permeability of aggregate piers is beneficial because they can provide drainage in constant low permeability soil layers and sand-clay-sand interbeded soil layers. Further, because the drainage path from the point in the middle of the space between piers, more time is required to dissipate pore water pressure (Figure 100). However, pore water pressure in DAPs appears to be highly related with the matrix soil conditions (Figure 101).

In situ CPT soundings indicate that negative pore water pressures are generated in low permeability layers during initial penetration. One reason for this negative pressure may be that initial unsaturated silty or clayey soils have negative air pressure. Another reason for negative pore water pressures may be the combination of soil remolding, reduction, and pore water being forcing away from the piezometer by air. Figure 101 shows the differences in pore water pressures before and after DAP installation, differences that may be the result of ground modification and remolding after the installation.

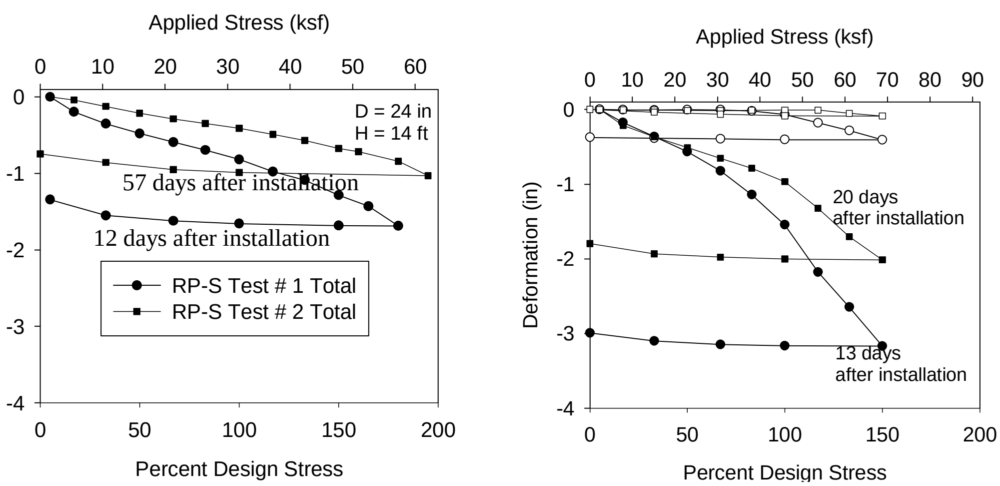


Figure 99. Comparison modulus load test results for time effect for Waterloo, IA site (1 tsf = 0.096 MPa)

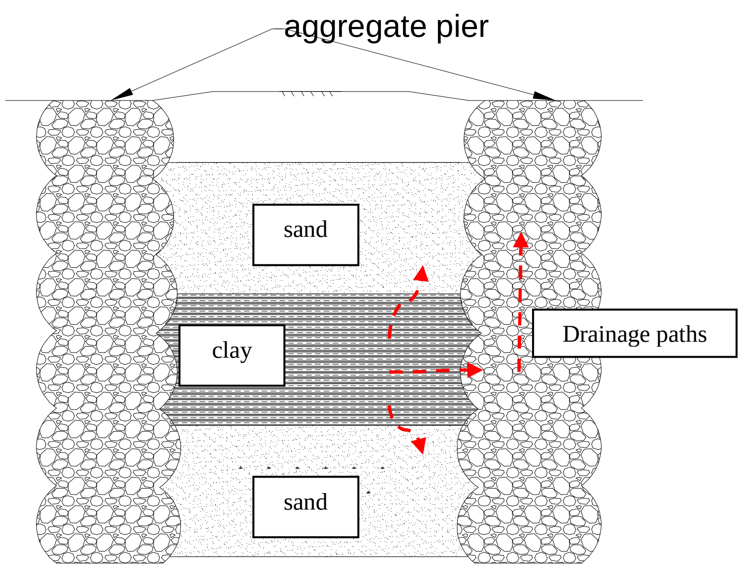


Figure 100. Schematic of the drainage paths of the sand-clay-sand interbedded soil layers after DAP installation

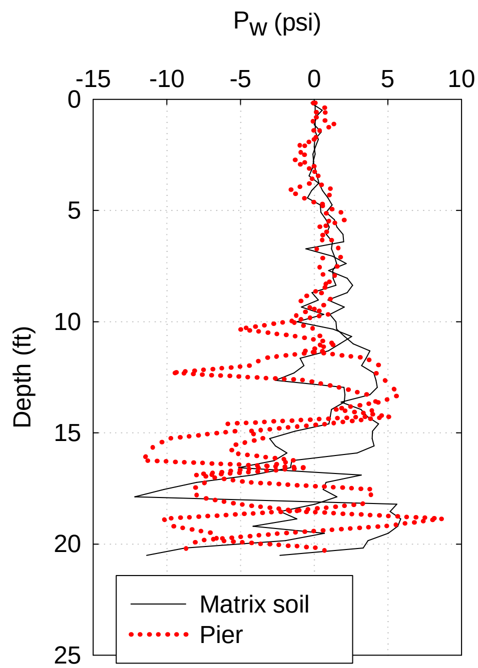


Figure 101. Pore water pressure in matrix soil and pier element from CPT of Springfield, MA

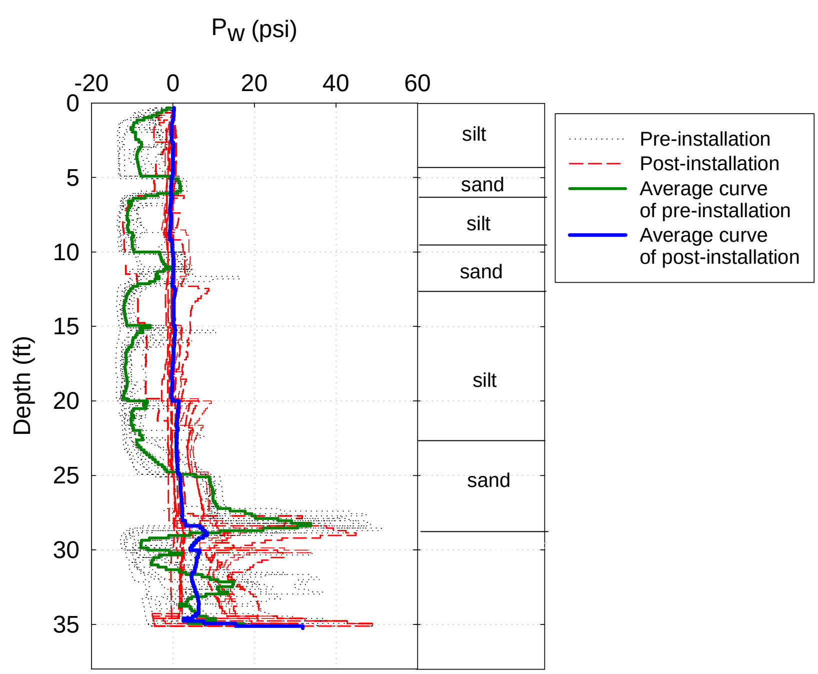


Figure 102. Pre-installation and post-installation Pore water pressure in interbeded soil layers from CPT for Westminster, CA site

PROJECTED MATRIX SOIL IMPROVEMENT TABLES

This section provides two tables that can be used to predict soil improvement after DAPs are installed. These tables are based on tip resistance data and friction ratios obtained from CPTs performed both before and after DAPs were installed at the case sites. The soil classifications used in the tables are based on the Jefferies and Davis classification system (1997), and Figure 101 shows the distribution of increases in CPT tip resistance values with respect to the pre-installation soil index. Because clayey soils did not show improvement (Figure 103), only clean sand to silty sand soils and silty sand to sandy silt soils were used to construct the two design tables.

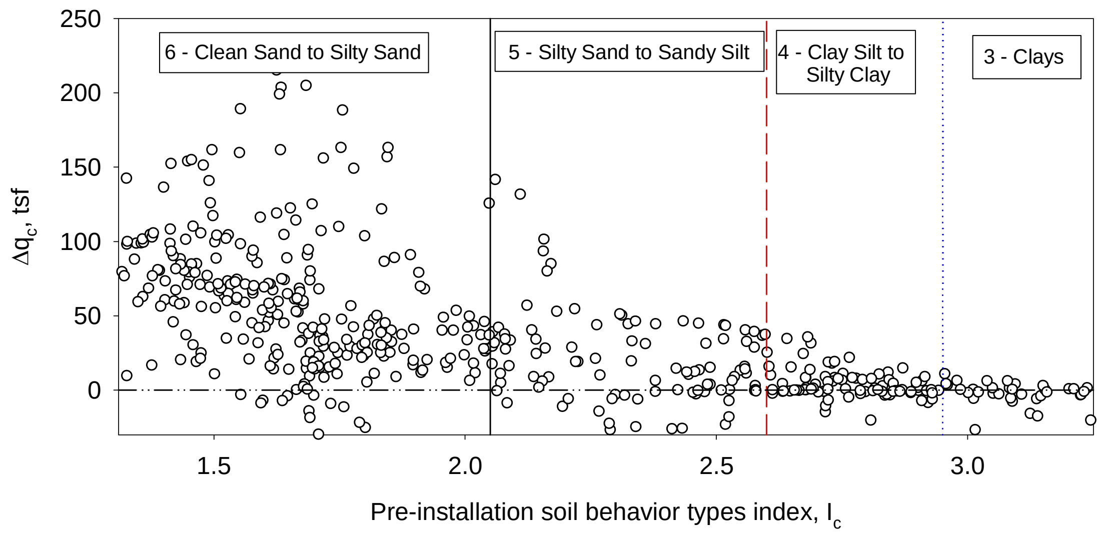


Figure 103. Increases in tip resistance with respect to pre-installation soil behavior types in pier groups (following classification method proposed by Jefferies and Davis 1997)

Table 20 summarizes the predicted matrix soil improvement for sandy soils and silty soils within pier groups. Table 21 summarizes the predicted matrix soil improvement for sandy soils and silty soils measured at 2 ft from the center of an individual pier not in a group of piers.

Table 20. Predicted matrix soil improvement for sandy soils and silty soils within pier groups

Clean sand to silty sand		Silty sand to sandy silt	
Pre-installation tip resistance, q_c (tsf)	Δq_c (tsf) 0 – 25 – ave. – 75 – 100	Pre-installation tip resistance (tsf)	Δq_c (tsf) 0 – 25 – ave. – 75 – 100
10–30	5–30–60	5–20	5–25–45
30–60	15–55–80	20–40	5–15–35
60–100	20–55–85	40–80	5–25–55
100–160	30–60–85	80–100	–
Legend: 1 tsf = 0.096 MPa			

Table 21. Predicted matrix soil improvement for sandy soils and silty soils measured at 2 ft from the center of an individual pier not in a group of piers

Clean sand to silty sand		Silty sand to sandy silt	
Pre-installation tip resistance (tsf)	Δq_c (tsf) 0 – 25 – ave. – 75 – 100	Pre-installation tip resistance (tsf)	Δq_c (tsf) 0 – 25 – ave. – 75 – 100
10–30	5–20–40	5–20	5–10–20
30–60	5–40–70	20–40	5–15–35
60–100	5–30–85	40–60	–
100–160	20–60–85	60–80	–
Legend: 1 tsf = 0.096 MPa			

The soils can be classified several groups which have different amount improvement after group DAPs are installed on Robertson et al.'s classification chart. The results are shown in Figure 104.

Comparing the design table and Figure 104, the former is used to the known soil types and initial tip resistance, the later is applicable to for the soil profiles with tip resistance and friction ratio. Both methods have quite large variance, but the results are still achieved to the purposes of this study.

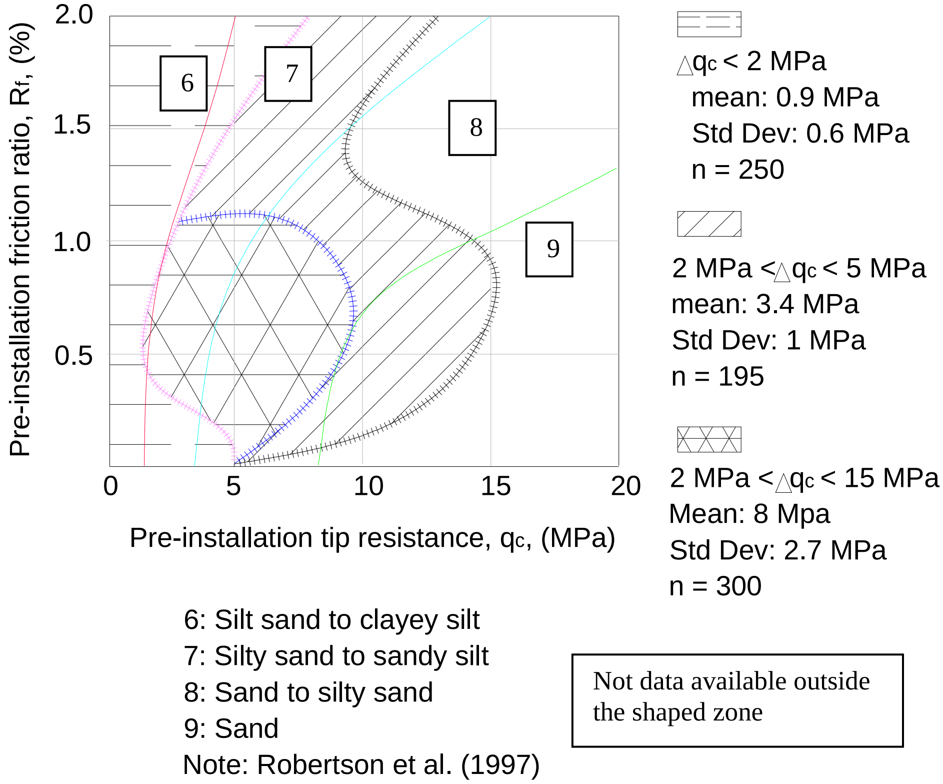


Figure 104. Soil improvement projected from initial tip resistances and friction ratios (the amount increment respected to matrix soils within pier groups; the spacing of the piers within the range of 4 ft and 7 ft)

CHAPTER 6: CONCLUSIONS

Conclusions based on the data from the 16 cases provided by the Geopier Foundation Company resulted in the following conclusions. These conclusions are grouped into these categories: matrix soil improvement and soil-pier interactions. The final two sections of this chapter offer suggestions for future research and implications for constructing DAPs.

MATRIX SOIL IMPROVEMENT

- The following methods can be used to identify the pre-installation effective improvement and non effective improvement soils:
  - CPT friction ratio less than 1%
  - Fines content less than 20%
  - Effective improvement chart (Figure 77).
- The types of fines may affect the soil improvement. Clay types may reduce the degree of improvement.
- Based on soil behavior index, clean sand to silty sand soils, sandy silt soils, and silty clay to clay soils show major improvement, minor improvement and no improvement, respectively.
- The initial relative density affects the degree of improvement for clean sand to silty sand soil. The very loose sandy soils (for example, SPT=0) indicate much less improvement than dense sand after IRAP installation.
- Overall, the average SPT N-vales increased from 9 to 20 after DAPs installation in groups; The average SPT N_{60}-values, which were calculated from CPT results, increased from 11 to 21 in the groups.
- Typically, the surface soils to the depth of 1 to 2 DAP diameters do not have much improvement.
- The overburden pressure (or depth) was not a significant factor to influence the matrix soil improvement at elevations deeper than 1 to 2 diameters.
- Soil improvement can be achieve as deep as 2 diameters beneath the pier bottom, but strongly depends on the soil types and soil strata.

- The average values of improvement index of clean sand to silty sand soils (FC<20%) from CPT and SPT were approximately 0.62 and 1.2, respective.
- The average group effective factor was approximately 1.35 for sandy soils.
- The relative density of the sandy soils increased after DAPs installation.
- CPT results indicated the effective improvement zone of single pier to be within the 5 ft from center.
- DAP may induce the subsoil disturbance and modifications which may reduce the tip resistance values at some points.

SOIL-PIER INTERACTIONS

- The pier stiffness and strength from the tip resistance tend to strongly relate with the stiffness and strength of the matrix soil. The tip resistance values of the pier are normally 2 to 3 times greater than the matrix soil composed of sand layers.
- DAPs provide a vertical drainage to dissipate the pore water pressure in clayey layers.
- The CPT results indicated that the pore water pressure in DAP was related to the matrix soil conditions.
- The interactions, which are represented by the stiffness, between the pier and soils increase after the pore water pressure dissipates.
- Settlement index was introduced to study the settlement of DAPs.

SUGGESTIONS FOR FUTURE RESEARCH

It is recommended to future researchers to use and update the design tables and figures.

Numerical analysis of the single pier and pier groups settlement in sand is recommended. The sliding effect may be considered for pier groups simulation.

The temporary liquefaction may be occurred during ramming compaction in sands. The correlations of sandy soil densification with the critical voids ratio, ground heave, dilatants and drainage path are not fully understood. It is recommended to future research on this area.

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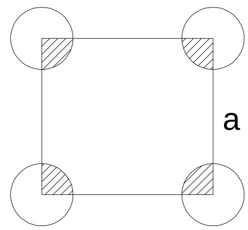
APPENDIX I: SAMPLE CALCULATIONS

Sample calculations of area replacement ratio



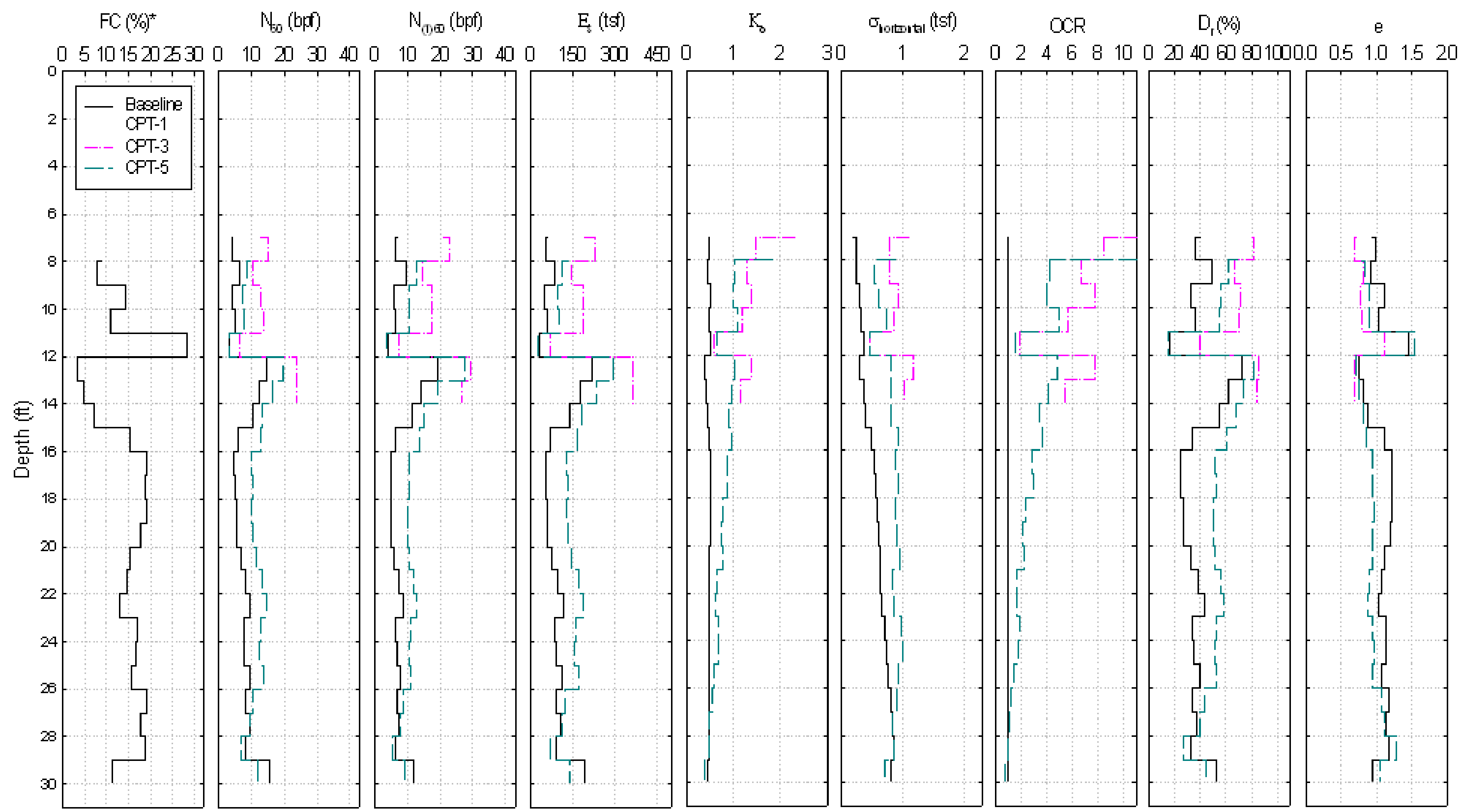
Assume the diameter of piers = D, Ar = area replacement ratio

$$A_r = \frac{\pi D^2}{4a^2}$$

APPENDIX II: SAMPLE SPREAD SHEET AND SOIL PARAMETERS PROFILES

1	2	3	4	5	6	7	8	9	10	11
Depth, m	ft	Qc, tst	Fs, tsf	Pw, psi	n (degrees)	Overburdent (psf)	FR (%)	Qt	Ic	Soil Type
0.05	0.16	5.60	0.00	0.00	0.00	19.68	0.00	568.11		
0.10	0.33	23.90	0.02	0.00	0.00	39.36	0.08	1213.43	0.41	7.00
0.15	0.49	41.90	0.13	-0.60	0.01	59.04	0.31	575.78	1.01	7.00
0.20	0.66	53.50	0.22	-0.30	0.00	78.72	0.41	876.98	0.99	7.00
0.25	0.82	58.90	0.29	-0.20	0.00	98.40	0.49	925.33	1.04	7.00
0.30	0.98	61.00	0.33	0.00	0.01	118.08	0.54	1032.20	1.06	7.00
0.35	1.15	61.20	0.35	0.00	0.00	137.76	0.57	887.50	1.11	7.00
0.40	1.31	59.70	0.37	0.00	0.00	157.44	0.62	757.38	1.17	7.00
0.45	1.48	56.20	0.26	0.00	0.00	177.12	0.46	633.60	1.11	7.00
0.50	1.64	50.20	0.26	-0.10	0.01	196.80	0.52	474.45	1.23	7.00
0.55	1.80	38.80	0.22	0.00	0.01	216.48	0.57	357.46	1.34	6.00
0.60	1.97	31.10	0.17	0.00	0.00	236.16	0.55	262.38	1.42	6.00
0.65	2.13	27.50	0.14	0.00	0.01	255.84	0.51	213.98	1.47	6.00
0.70	2.30	28.60	0.13	0.00	0.01	275.52	0.46	206.61	1.45	6.00
0.75	2.46	30.50	0.14	0.00	0.00	295.20	0.46	205.64	1.46	6.00
0.80	2.62	32.20	0.15	0.10	0.00	314.88	0.47	213.28	1.45	6.00
0.85	2.79	36.50	0.19	0.10	0.01	334.56	0.52	226.97	1.46	6.00
0.90	2.95	46.30	0.27	0.00	0.00	354.24	0.59	260.40	1.44	6.00
0.95	3.12	54.90	0.37	0.00	0.01	373.92	0.68	292.65	1.45	6.00
1.00	3.28	55.50	0.41	0.20	0.01	393.60	0.74	303.20	1.47	6.00
1.05	3.44	51.10	0.46	0.20	0.01	413.28	0.90	264.74	1.57	6.00
1.10	3.61	47.50	0.56	0.20	0.01	432.96	1.18	233.98	1.70	6.00
1.15	3.77	47.70	0.60	0.10	0.01	452.64	1.26	216.66	1.74	6.00
1.20	3.94	56.20	0.62	-0.40	0.01	472.32	1.11	211.22	1.71	6.00
1.25	4.10	65.40	0.62	-0.50	0.01	492.00	0.95	231.04	1.63	6.00
1.30	4.26	67.80	0.60	-0.90	0.01	511.68	0.89	210.65	1.64	6.00
1.35	4.43	65.70	0.55	-0.80	0.01	531.36	0.84	202.41	1.63	6.00
1.40	4.59	63.70	0.59	-0.10	0.01	551.04	0.93	224.34	1.63	6.00
1.45	4.76	59.40	0.41	-0.50	0.01	570.72	0.69	183.95	1.61	6.00
1.50	4.92	54.80	0.51	-0.40	0.01	590.40	0.94	168.22	1.72	6.00
1.55	5.08	51.90	0.51	-3.60	0.01	610.08	0.99	91.44	1.94	6.00
1.60	5.25	55.20	0.48	-5.30	0.01	629.76	0.87	78.80	1.96	6.00
1.65	5.41	58.70	0.37	-0.40	0.01	649.44	0.63	165.13	1.62	6.00
1.70	5.58	58.80	0.39	0.00	0.01	669.12	0.67	174.75	1.61	6.00
1.75	5.74	59.70	0.42	0.00	0.01	688.80	0.71	172.34	1.63	6.00
1.80	5.90	60.00	0.40	0.00	0.01	708.48	0.67	168.38	1.63	6.00
1.85	6.07	58.50	0.41	-0.10	0.01	728.16	0.71	156.58	1.66	6.00
1.90	6.23	56.70	0.41	0.00	0.01	747.84	0.73	150.64	1.69	6.00
1.95	6.40	53.50	0.38	0.00	0.01	767.52	0.72	138.41	1.71	6.00
2.00	6.56	55.90	0.38	-0.10	0.01	787.20	0.68	138.49	1.70	6.00
2.05	6.72	59.40	0.40	-0.10	0.01	806.88	0.68	143.67	1.68	6.00
2.10	6.89	59.20	0.40	-0.10	0.12	826.56	0.68	139.81	1.69	6.00
2.15	7.05	59.50	0.40	0.00	0.12	846.24	0.68	139.62	1.69	6.00
2.20	7.22	60.00	0.40	0.10	0.12	865.92	0.67	139.91	1.69	6.00
2.25	7.38	59.60	0.39	0.10	0.13	885.60	0.66	135.81	1.69	6.00
2.30	7.54	57.00	0.39	0.10	0.13	905.28	0.69	126.95	1.73	6.00
2.35	7.71	51.70	0.40	0.00	0.14	924.96	0.78	110.79	1.81	6.00
2.40	7.87	49.20	0.38	0.10	0.14	944.64	0.78	104.76	1.83	6.00
2.45	8.04	51.20	0.27	-0.10	0.14	964.32	0.53	103.64	1.74	6.00
2.50	8.20	59.10	0.26	0.10	0.14	984.00	0.44	120.89	1.64	6.00
2.55	8.36	65.80	0.37	-3.80	0.15	1003.68	0.57	84.21	1.83	6.00
2.60	8.53	69.70	0.46	-3.60	0.15	1023.36	0.66	89.75	1.84	6.00
2.65	8.69	74.40	0.48	-3.40	0.15	1043.04	0.65	96.41	1.81	6.00
2.70	8.86	79.10	0.52	-3.10	0.16	1062.72	0.66	104.13	1.79	6.00
2.75	9.02	81.60	0.54	-2.60	0.15	1082.40	0.67	111.28	1.77	6.00
2.80	9.18	86.00	0.58	-2.30	0.15	1102.08	0.68	119.24	1.75	6.00
2.85	9.35	88.80	0.64	-2.20	0.15	1121.76	0.73	122.68	1.75	6.00
2.90	9.51	90.60	0.68	-1.80	0.16	1141.44	0.76	128.55	1.75	6.00
2.95	9.68	91.30	0.70	-1.70	0.15	1161.12	0.77	129.05	1.75	6.00
3.00	9.84	94.30	0.70	-1.60	0.15	1180.80	0.75	132.81	1.73	6.00
3.05	10.00	98.10	0.71	-1.50	0.31	1200.48	0.73	137.66	1.72	6.00
3.10	10.17	101.60	0.74	-1.40	0.30	1220.16	0.73	142.06	1.71	6.00
3.15	10.33	99.20	0.77	-1.40	0.30	1239.84	0.78	136.78	1.74	6.00
3.20	10.50	92.30	0.74	-1.30	0.30	1259.52	0.81	126.73	1.77	6.00
3.25	10.66	85.30	0.69	-1.20	0.30	1279.20	0.82	116.61	1.80	6.00
3.30	10.82	83.90	0.65	-1.20	0.30	1298.88	0.78	113.14	1.80	6.00
3.35	10.99	78.80	0.62	-1.10	0.30	1318.56	0.79	105.81	1.83	6.00
3.40	11.15	69.00	0.57	-1.00	0.30	1338.24	0.83	92.20	1.89	6.00
3.45	11.32	60.10	0.36	-1.00	0.30	1357.92	0.61	79.13	1.86	6.00

12	13	14	15	16	17	18	19	20	21	22
Fines Content	N60	Es	effective stress	N(1)60	Friction angle	Ko(pre)	stress (tsf)	fs	Dr	e
	0.62	14.00	0.01	6.28	29.83	0.50	0.00	1.00	51.66	0.79
-3.60	2.92	59.75	0.02	20.80	37.90	0.39	0.01	1.00	83.32	0.63
-1.92	5.96	104.75	0.02	42.48	45.58	0.29	0.01	1.00	99.41	0.59
-2.02	7.57	133.75	0.07	28.08	40.79	0.35	0.01	1.00	87.69	0.63
-1.70	8.47	147.25	0.06	34.29	42.98	0.32	0.02	1.00	92.97	0.61
-1.60	8.80	152.50	0.06	34.91	43.19	0.32	0.02	1.00	93.37	0.61
-1.26	8.96	153.00	0.06	36.88	43.83	0.31	0.02	1.00	94.53	0.61
-0.76	8.91	149.25	0.07	33.93	42.86	0.32	0.03	1.00	91.61	0.63
-1.25	8.23	140.50	0.08	29.34	41.26	0.34	0.03	1.00	87.96	0.64
-0.30	7.61	125.50	0.09	25.57	39.84	0.36	0.04	1.00	83.04	0.66
0.81	6.08	97.00	0.11	18.72	36.98	0.40	0.04	1.00	73.13	0.72
1.81	5.01	77.75	0.11	15.21	35.31	0.42	0.05	1.00	66.44	0.76
2.42	4.49	68.75	0.12	13.07	34.19	0.44	0.06	1.00	61.66	0.78
2.18	4.64	71.50	0.13	12.99	34.14	0.44	0.06	1.00	61.64	0.78
2.23	4.96	76.25	0.14	13.36	34.35	0.44	0.06	1.00	62.42	0.78
2.12	5.22	80.50	0.15	13.59	34.47	0.43	0.07	1.00	62.99	0.77
2.24	5.94	91.25	0.15	15.32	35.36	0.42	0.07	1.00	66.33	0.76
2.08	7.50	115.75	0.16	18.75	36.99	0.40	0.07	1.00	72.24	0.73
2.19	8.92	137.25	0.18	21.19	38.07	0.38	0.07	1.00	75.67	0.72
2.44	9.07	138.75	0.19	20.98	37.97	0.38	0.08	1.00	75.21	0.73
3.96	8.64	127.75	0.18	20.22	37.65	0.39	0.08	1.00	73.19	0.75
6.09	8.37	118.75	0.19	19.09	37.15	0.40	0.09	1.00	70.34	0.79
6.92	8.53	119.25	0.20	18.98	37.10	0.40	0.09	1.00	69.75	0.80
6.23	9.93	140.50	0.22	21.21	38.07	0.38	0.09	1.00	73.29	0.78
4.88	11.26	163.50	0.26	21.88	38.36	0.38	0.09	1.00	74.91	0.75
4.98	11.70	169.50	0.28	22.03	38.42	0.38	0.10	1.00	75.05	0.75
4.90	11.32	164.25	0.32	19.99	37.55	0.39	0.10	1.00	72.31	0.77
4.91	10.98	159.25	0.32	19.30	37.24	0.40	0.11	1.00	71.31	0.77
4.46	10.14	148.50	0.28	19.08	37.14	0.40	0.11	1.00	71.22	0.77
6.54	9.74	137.00	0.32	17.18	36.26	0.41	0.12	1.00	67.08	0.81
11.31	9.97	129.75	0.32	17.51	36.42	0.41	0.12	1.00	65.40	0.88
11.78	10.67	138.00	0.56	14.21	34.79	0.43	0.14	1.00	59.22	0.92
4.63	10.06	146.75	0.70	12.05	33.62	0.45	0.14	1.00	57.96	0.83
4.55	10.06	147.00	0.35	16.92	36.14	0.41	0.14	1.00	67.73	0.78
4.91	10.29	149.25	0.33	17.79	36.55	0.40	0.14	1.00	68.96	0.78
4.79	10.31	150.00	0.34	17.57	36.45	0.41	0.14	1.00	68.68	0.78
5.45	10.19	146.25	0.35	17.12	36.23	0.41	0.15	1.00	67.56	0.79
5.84	9.95	141.75	0.37	16.32	35.86	0.41	0.15	1.00	65.99	0.81
6.29	9.46	133.75	0.37	15.47	35.44	0.42	0.16	1.00	64.22	0.82
6.06	9.85	139.75	0.38	15.89	35.64	0.42	0.16	1.00	65.11	0.81
5.78	10.41	148.50	0.40	16.44	35.91	0.41	0.17	1.00	66.22	0.80
5.97	10.41	148.00	0.41	16.24	35.82	0.42	0.17	1.00	65.78	0.81
5.95	10.46	148.75	0.42	16.13	35.76	0.42	0.18	1.00	65.58	0.81
5.90	10.54	150.00	0.42	16.20	35.79	0.42	0.18	1.00	65.73	0.81
5.99	10.48	149.00	0.43	16.07	35.73	0.42	0.18	1.00	65.45	0.81
6.66	10.15	142.50	0.44	15.38	35.39	0.42	0.19	1.00	63.85	0.83
8.30	9.47	129.25	0.45	14.19	34.78	0.43	0.20	1.00	60.73	0.86
8.71	9.07	123.00	0.46	13.34	34.33	0.44	0.21	1.00	58.77	0.88
6.79	9.14	128.00	0.47	13.40	34.36	0.44	0.21	1.00	59.83	0.85
4.97	10.20	147.75	0.49	14.58	34.98	0.43	0.21	1.00	63.22	0.81
8.68	12.13	164.50	0.48	17.42	36.38	0.41	0.20	1.00	66.43	0.84
9.01	12.92	174.25	0.78	14.67	35.03	0.43	0.22	1.00	61.35	0.87
8.33	13.63	186.00	0.77	15.53	35.46	0.42	0.22	1.00	63.30	0.85
7.84	14.38	197.75	0.77	16.42	35.90	0.41	0.22	1.00	65.14	0.83
7.39	14.72	204.00	0.75	16.94	36.15	0.41	0.22	1.00	66.26	0.82
7.01	15.41	215.00	0.73	18.05	36.67	0.40	0.22	1.00	68.27	0.81
7.16	15.95	222.00	0.72	18.84	37.04	0.40	0.22	1.00	69.42	0.81
7.06	16.25	226.50	0.72	19.16	37.18	0.40	0.23	1.00	69.94	0.80
7.15	16.40	228.25	0.70	19.60	37.37	0.39	0.23	1.00	70.55	0.80
6.78	16.83	235.75	0.70	20.07	37.58	0.39	0.23	1.00	71.42	0.79
6.41	17.39	245.25	0.71	20.70	37.86	0.39	0.23	1.00	72.50	0.78
6.24	17.96	254.00	0.71	21.34	38.13	0.38	0.23	1.00	73.45	0.78
6.83	17.72	248.00	0.71	21.01	37.99	0.38	0.24	1.00	72.71	0.79
7.53	16.69	230.75	0.72	19.66	37.40	0.39	0.25	1.00	70.45	0.81
8.17	15.59	213.25	0.72	18.33	36.80	0.40	0.26	1.00	68.13	0.82
8.14	15.33	209.75	0.73	17.99	36.64	0.40	0.26	1.00	67.61	0.83
8.74	14.54	197.00	0.74	16.95	36.15	0.41	0.27	1.00	65.62	0.84
10.13	13.01	172.50	0.74	15.14	35.27	0.42	0.28	1.00	61.76	0.88
9.55	11.23	150.25	0.74	13.05	34.17	0.44	0.30	1.00	57.75	0.89



*Based on Robertson and Fear (1995)

Figure 105. Calculated soil parameter profiles from CPT for Lacrosse, WI

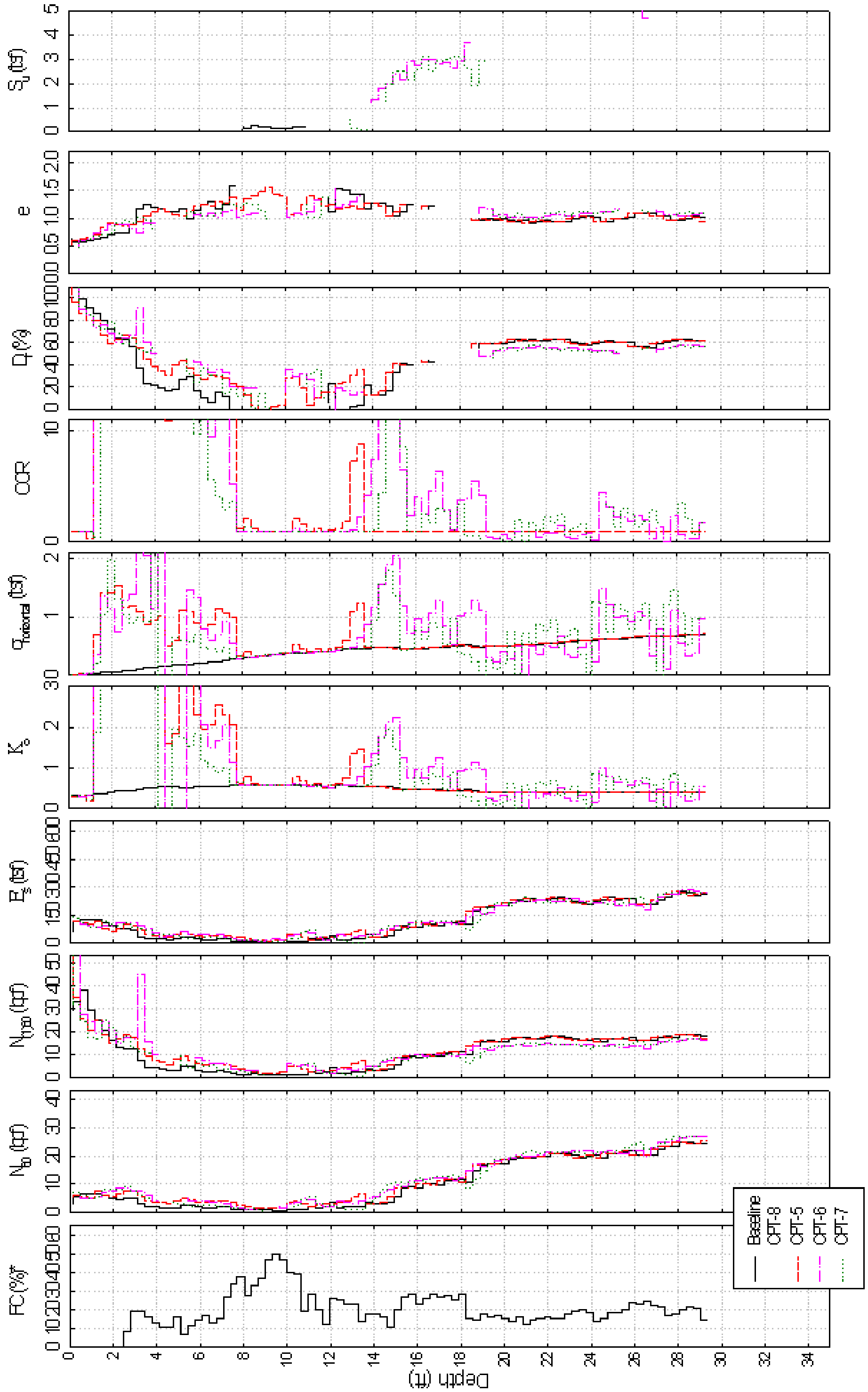


Figure 106. Calculated soil parameter profiles from CPT for Manapana, NJ

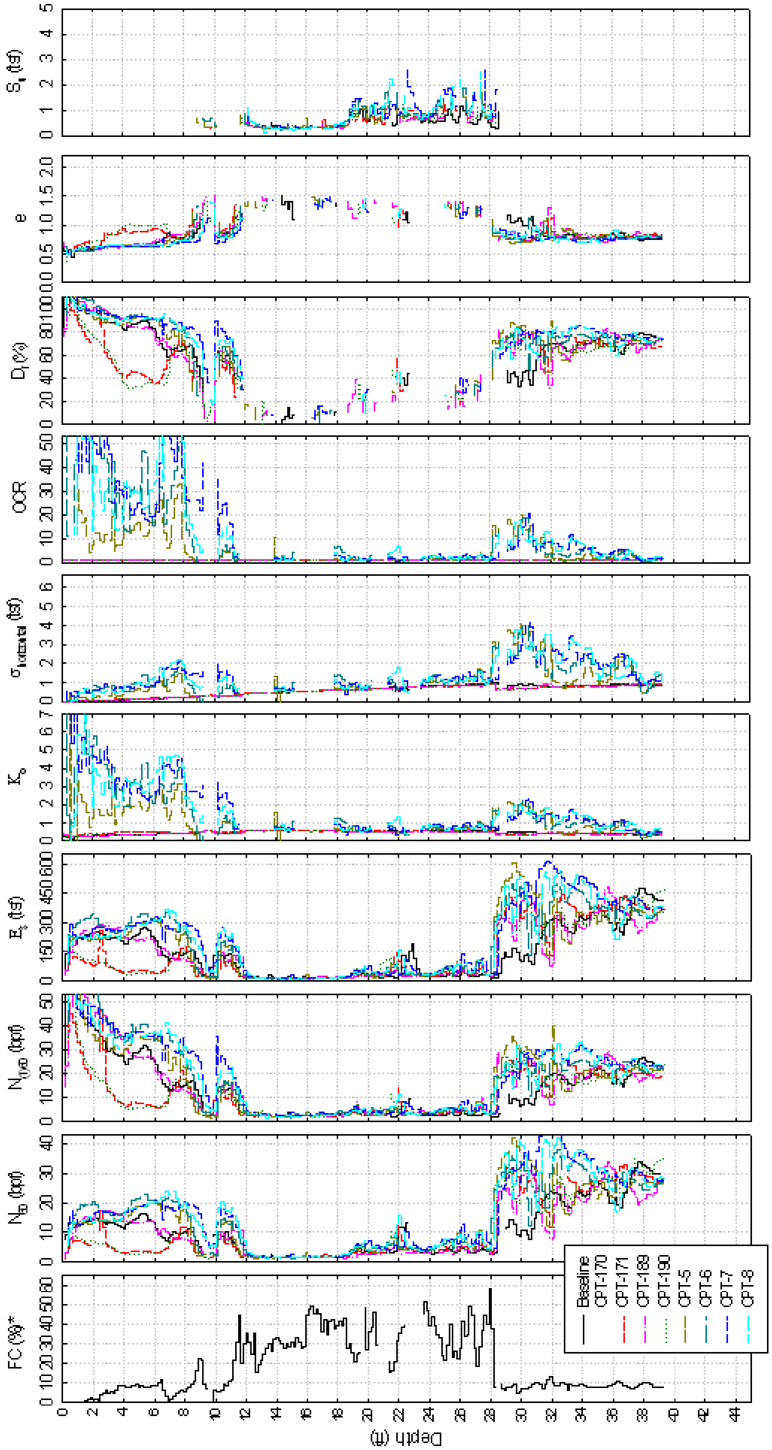


Figure 107. Calculated soil parameter profiles from CPT for Seattle, WA

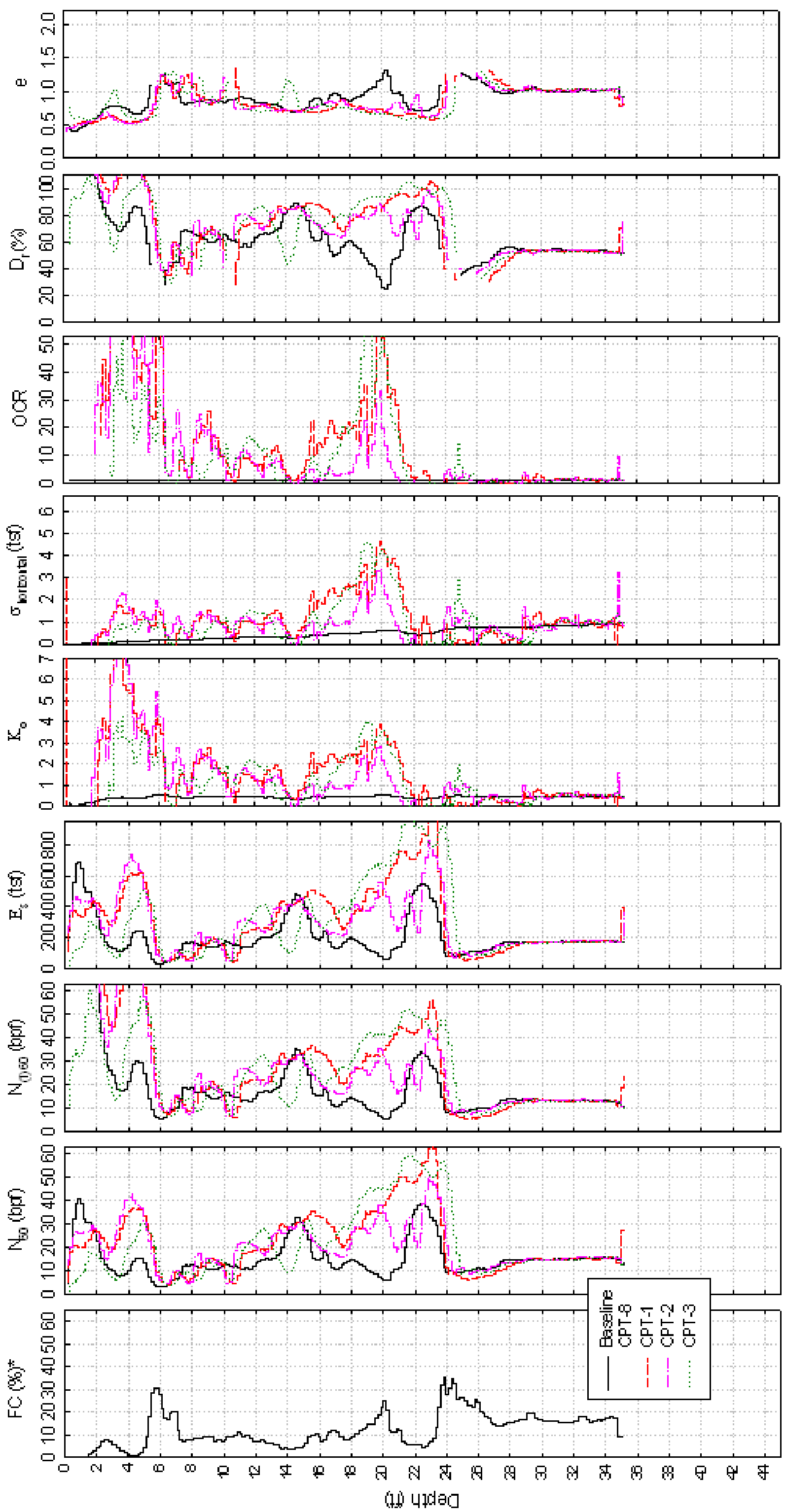


Figure 108. Calculated soil parameter profiles from CPT for Lynn Haven, FL

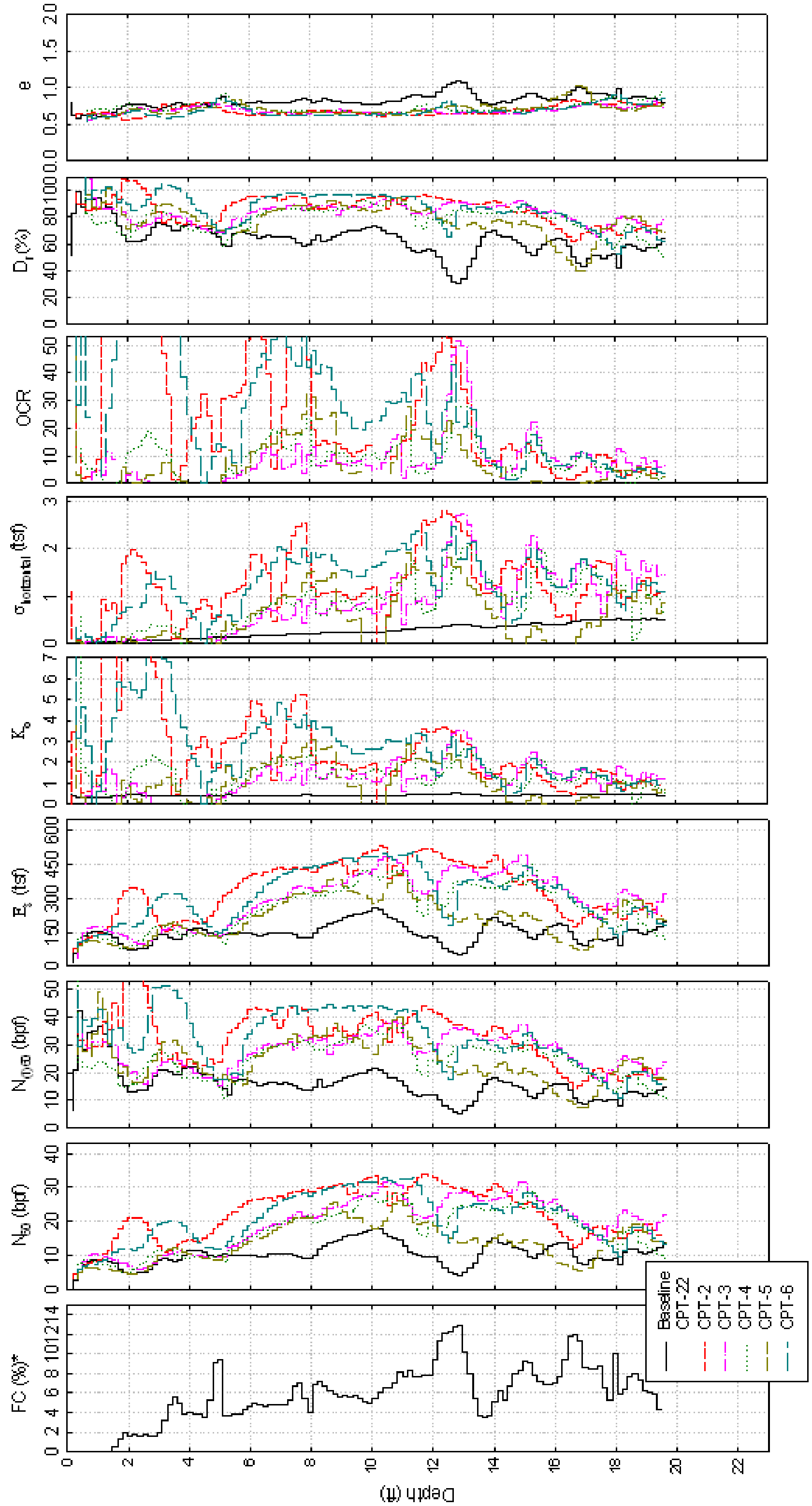


Figure 109. Calculated soil parameter profiles from CPT for Waterloo, IA

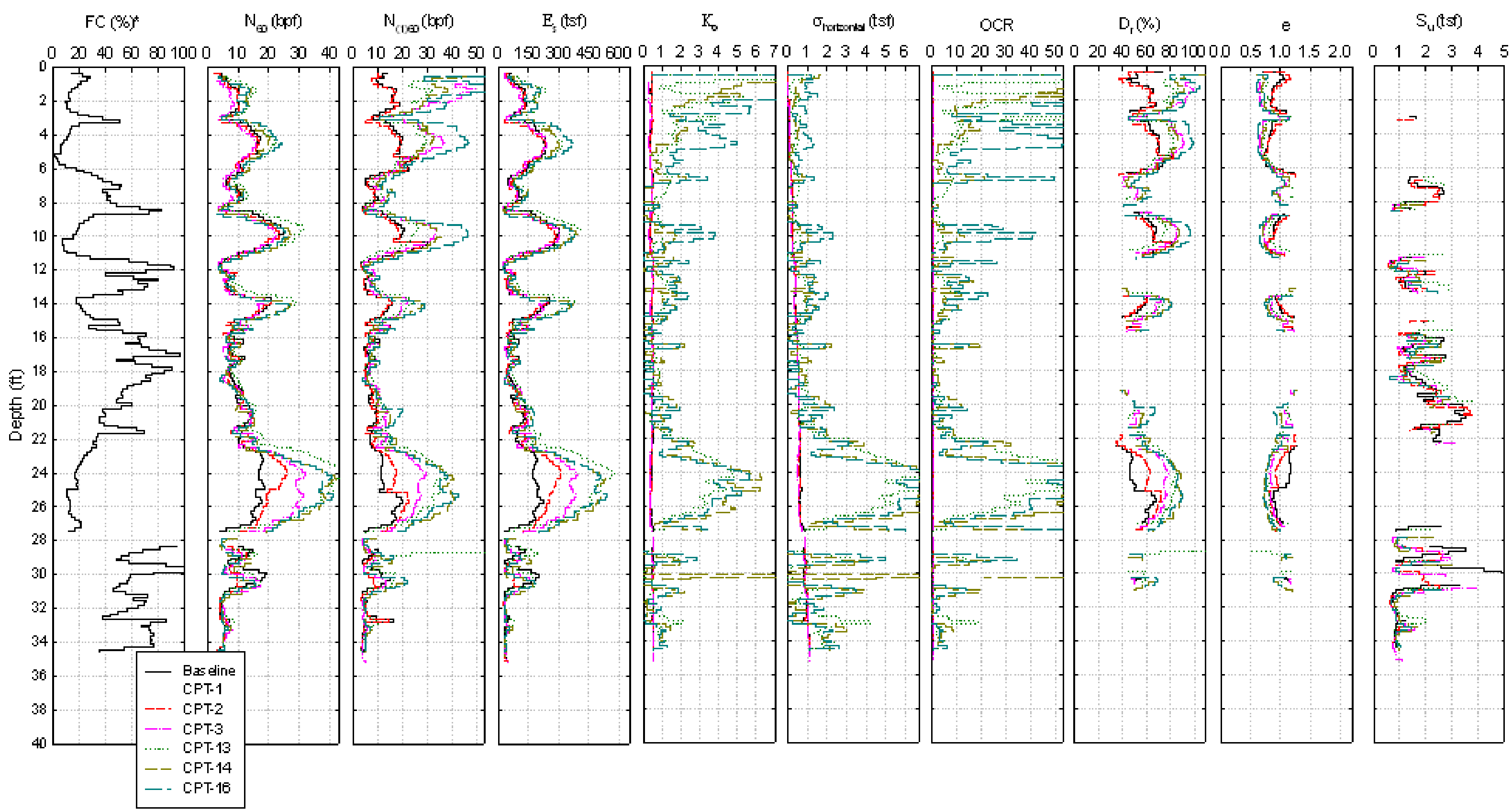


Figure 110. Calculated soil parameter profiles from CPT for Lynn Westminster, CA

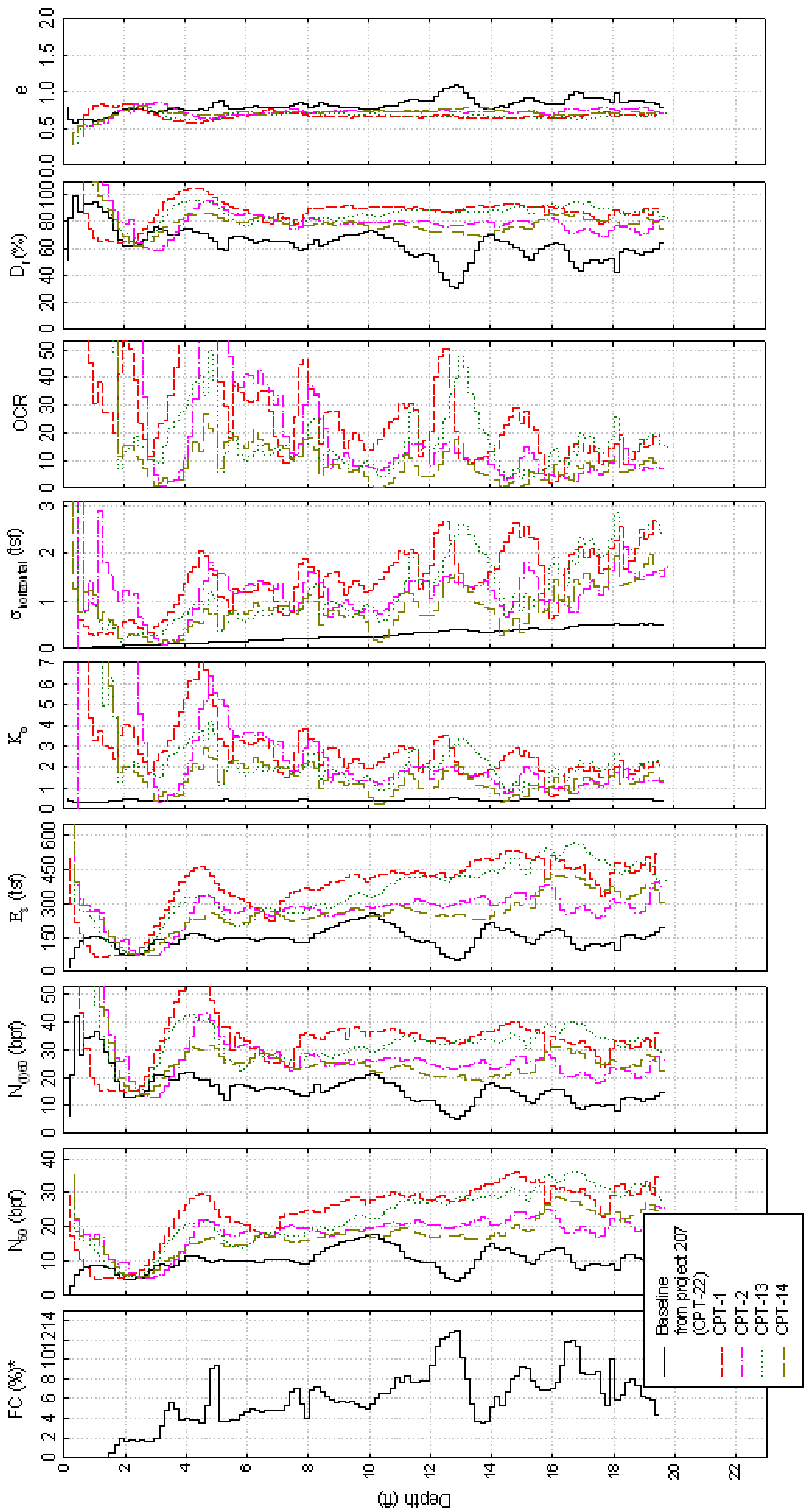


Figure 111. Calculated soil parameter profiles from CPT for Waterloo, IA (Liq.)

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