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CPT EVALUATION OF LIQUEFACTION MITIGATION WITH STONE COLUMNS IN INTERBEDDED SOILS

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16. Abstract

The liquefaction/lateral spread mitigation program at the Hinckley Drive (SR-79) overpass near Roy, Utah, USA required vibro-replacement stone columns. Cone penetration (CPT) testing was used to verify the soil improvement. Previous Utah Dept. of Transportation (UDOT) mitigation plans had used nearly continuous Standard Penetration (SPT) testing and sampling for improvement verification, which required additional laboratory testing. Because of interbedded layers with high fines content, UDOT specified the CPT for evaluating soil improvement to more easily eliminate thin clay and silt layers from consideration without the need for extensive laboratory testing as in previous projects. This report discusses data obtained from this project and utilizes the CPT results in evaluating several approaches for assessing stone column treatment effectiveness and applicability in soils with higher fines content. In particular, statistical analysis was performed to determine if the soil behavior type index, I_c, of 2.6 prescribed for the project as a lower-bound limit for soil improvement was appropriate for soils with higher fines contents.

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UNIT CONVERSION FACTORS

Units used in this report and not conforming to the UDOT standard unit of measurement (U.S. Customary system) can be found in the table below with their U.S. Customary equivalents.

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. (Adapted from FHWA report template, Revised March 2003)

1.0 INTRODUCTION

The Utah Department of Transportation (UDOT) planned to make significant improvements to Hinckley Drive (SR-79) near Roy, Utah, USA. The project included a new grade separated crossing at the Union Pacific Rail Road (UPRR) and Utah Transit Authority (UTA) rail corridors. The design called for a new single span steel girder bridge with a span of approximately 91 m in length. Approaches to the bridge were planned to include embankment fills of up to 12 m with wrap-around MSE walls at the abutments. A geotechnical exploration and design program, initiated in April, 2007 by the UDOT Geotechnical Division, indicated that liquefaction and lateral spreading would develop for the design earthquake. Therefore, stone column treatment was recommended to mitigate the hazard.

Because of the interbedded layers with high fines content, UDOT specified the cone penetrometer for evaluating soil improvement in the hope that thin clay layers could be more easily eliminated from consideration without the need for extensive laboratory testing as in previous projects. This report provides a case history for this project and utilizes the CPT results in evaluating several approaches for assessing stone column treatment effectiveness and applicability.

2.0 GEOLOGY AND GEOTECHNICAL SITE CHARACTERIZATION

2.1 Site Overview and Geologic Setting

The project site is located within the Basin and Range Province and is characterized by ancient Lake Bonneville sediment deposition. More recent flood plain deposits from the Pleistocene age comprise the surficial soil profile. The Weber Segment of the Wasatch Fault lies approximately 8 km to the east of the project which is capable of producing a 7 to 7.2 M earthquake at a recurrence interval of approximately 1600 years. Historically, the project site was used for farming and grazing and slopes gradually from east to west.

2.2 Subsurface Exploration and Conditions

The UDOT Geotechnical Division conducted the initial subsurface exploration program at the new bridge and embankment area which consisted of four Standard Penetration Test (SPT) borings and three Cone Penetration Test (CPT) soundings. The SPT borings were performed with a CME-850 drill rig using rotary wash methods. Soil samples were collected using California and Shelby tube samplers at 0.5 to 1.5 m intervals. The CPT soundings were conducted by ConeTec, Inc. (Salt Lake City office) and included pore pressure dissipation and shear wave velocity testing.

The exploration program revealed that interlayered loose to medium dense silty sands, non-plastic silts, and clays were present to a depth of approximately 12 to 20 m below existing ground surface. Below these layers, medium dense silty sand layers were encountered to the termination of the borings (approximately 29 m). Groundwater was encountered between 2 and 3.6 m. A typical borehole log is shown in Figure 1 along with the SPT $(N_1)_{60}$ values. Interbedded layers are evident from about 6 to 12 m. The $(N_1)_{60}$ is the raw blowcount, N, corrected to a hammer energy of 60% of the theoretical free-fall energy and an overburden pressure of 1 kg/cm^2 using procedures specified by Youd et al (2001).

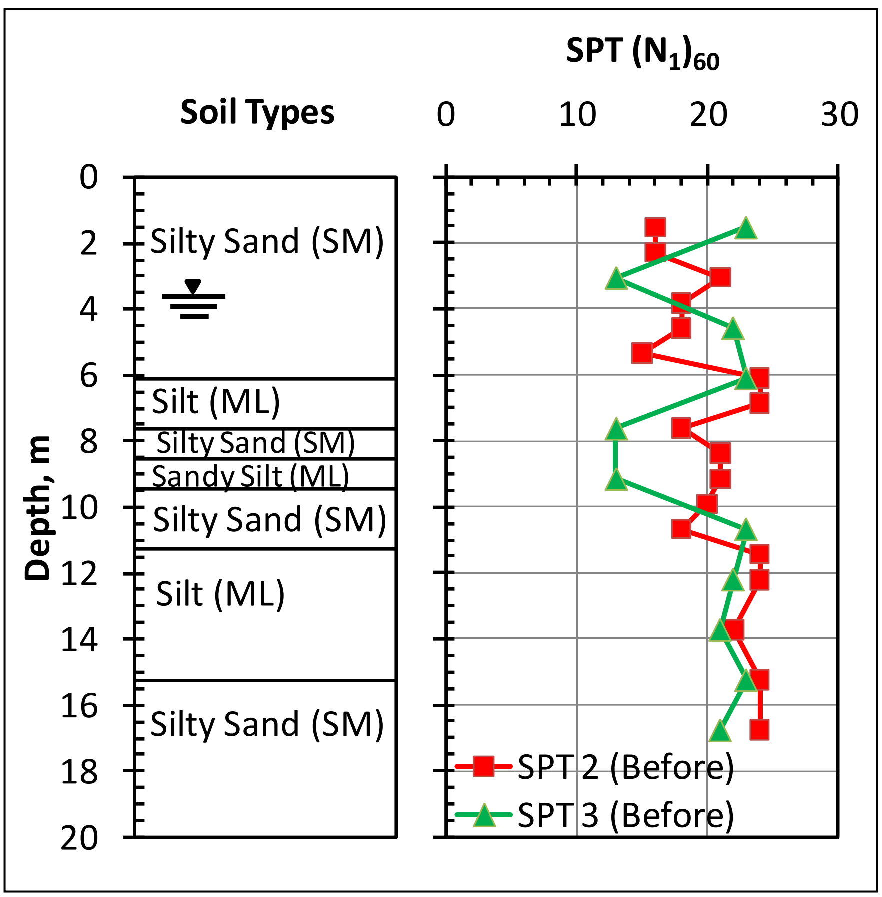

Figure 1: Typical soil profile and SPT blow counts prior to treatment at Hinckley Drive site.

A profile of $q_{c1n\text{-}cs}$ obtained from CPT 10 prior to treatment is also provided in Figure 2. The $q_{c1n\text{-}cs}$ value is the raw cone tip resistance q_c corrected to an overburden pressure of 1 kg/cm^2 and a clean sand condition using procedures specified by Youd et al (2001). The variation in $q_{c1n\text{-}cs}$ clearly indicates that the profile consists of even thinner interbedded layers than suggested by the soil profile obtained from the SPT boring. A profile of the running average $q_{c1n\text{-}cs}$ over a 0.6 m length is also provided in Figure 2. The averaging process effectively eliminates the peaks and troughs in the profile which facilitates comparisons with post-treatment profiles as discussed subsequently.

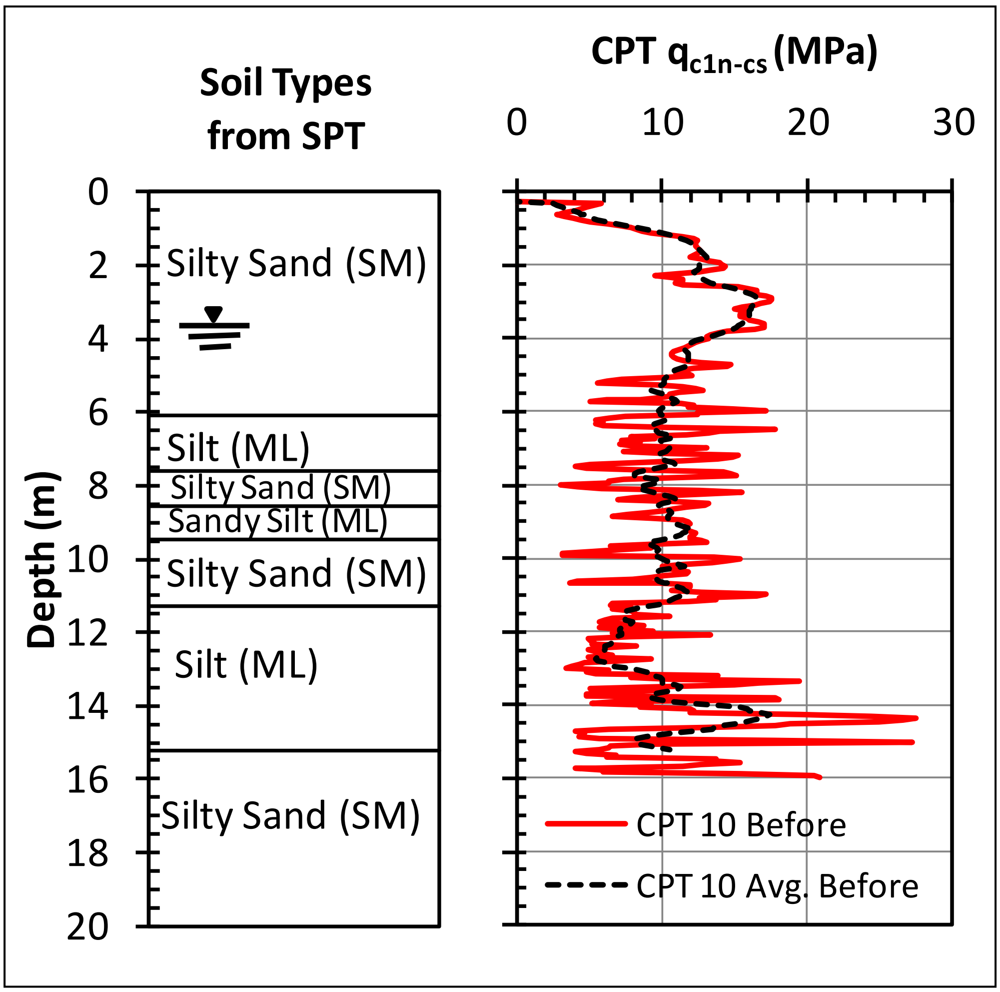

Figure 2: Typical soil profile and CPT 10 cone tip resistance for averaged (0.6 m interval) and non-averaged conditions before treatment at Hinckley Drive site.

2.3 Geotechnical Recommendations

Based on the subsurface exploration program, UDOT made recommendations to install deep foundations for support of the bridge and a surcharge program for the proposed embankments. A liquefaction analysis was performed for the project site which indicated a potential for both liquefaction and lateral spread to occur during a seismic event. A liquefaction mitigation program was proposed and was detailed in UDOT Special Provision 02243S.

3.0 LIQUEFACTION MITIGATION PROGRAM

The UDOT Geotechnical Division recommended that soil improvement by vibro-replacement be used to densify the in-situ soils to prevent detrimental effects from the potential liquefaction hazard.

The mitigation program specified a maximum column spacing of 2.44 m (8 ft) and an improved equivalent clean sand CPT tip resistance (q_{c1n-cs}) of 11.5 MPa (120 tons/ft^2) for soils with a behavior type index (I_c) of 2.6 or less. Soils with an I_c greater than 2.6 were not considered to be improvable and either not liquefiable or not susceptible to lateral spreading if liquefied (Youd et al 2001, 2009). Post treatment verification required CPT testing between the stone columns.

Nicholson Construction Co. was retained to execute the liquefaction mitigation program. Based on a small test program and previous experience, Nicholson elected to install 0.76 m (30 inch) diameter stone columns using a dry bottom feed method in a triangular arrangement at 2.44 m (8 ft) spacing. This spacing represents an area replacement ratio (A_r) of about 10%. This replacement ratio required 260 and 290 columns on the east abutment and west abutments, respectively. The stone columns typically extended to depths of about 12.2 to 13.7 m (40 to 45 ft) and consisted of crushed stone with a maximum diameter of 19 mm. Although UDOT had used wick drains in concert with stone column treatment to densify liquefiable sands with high fines content in past projects (Rollins et al. 2006, 2009), this approach was not considered necessary in this case because the initial q_{c1n-cs} values were relatively close to the required post-treatment values.

During construction, Nicholson monitored the volume of injected stone as a function of depth. Based on these measurements, the actual average column diameter was 0.91 m which leads to an A_r of 14%. Amperage and treatment time were also monitored as a function of depth to help ensure consistent energy per length of treatment. A typical profile showing column diameter along with average amperage and treatment time vs. depth is provided in Figure 3.

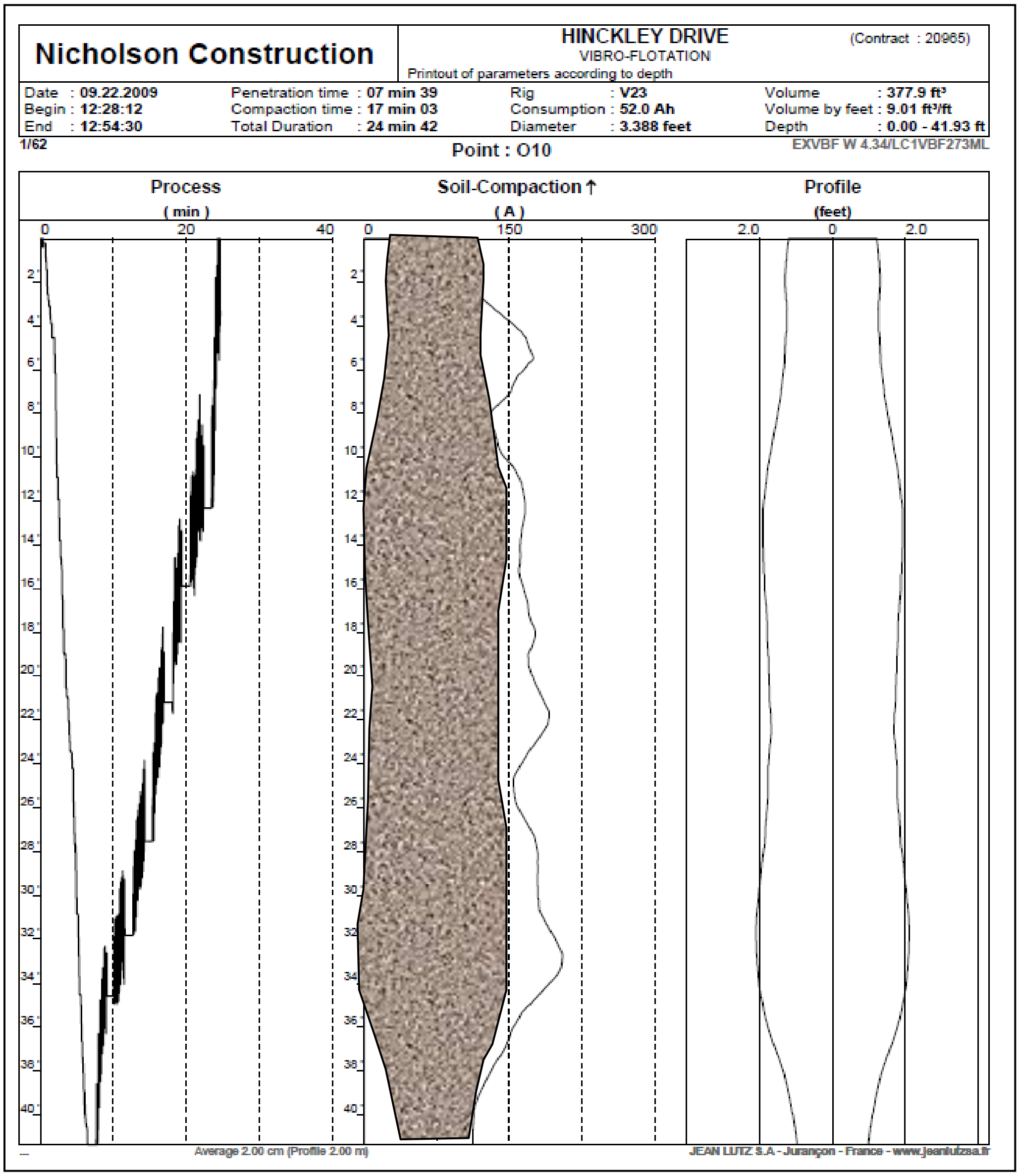

Figure 3: Profile showing measured treatment time, average amperage and column diameter as a function of depth.

4.0 EVALUATION OF IMPROVEMENT

4.1 In-situ Testing

Post treatment testing consisted of seven CPT soundings on the east abutment and six soundings on the west abutment. Soundings extended to a depth of approximately 15 m (50 ft) which was beyond the treatment depth. In addition, five SPT bore holes were performed adjacent to several of the CPT holes to confirm the soil classification types where cone tip resistance values were low and I_c did not exceed 2.6. The layout of the CPT soundings and SPT bore holes relative to the stone columns and abutment location at the east abutment is provided in Figure 4. Typically the CPT soundings were located within the center of three stone columns. The CPT soundings were typically completed about 2 weeks after treatment. However, CPT 2b was performed about 7 weeks after treatment to evaluate potential changes in cone tip resistance with time.

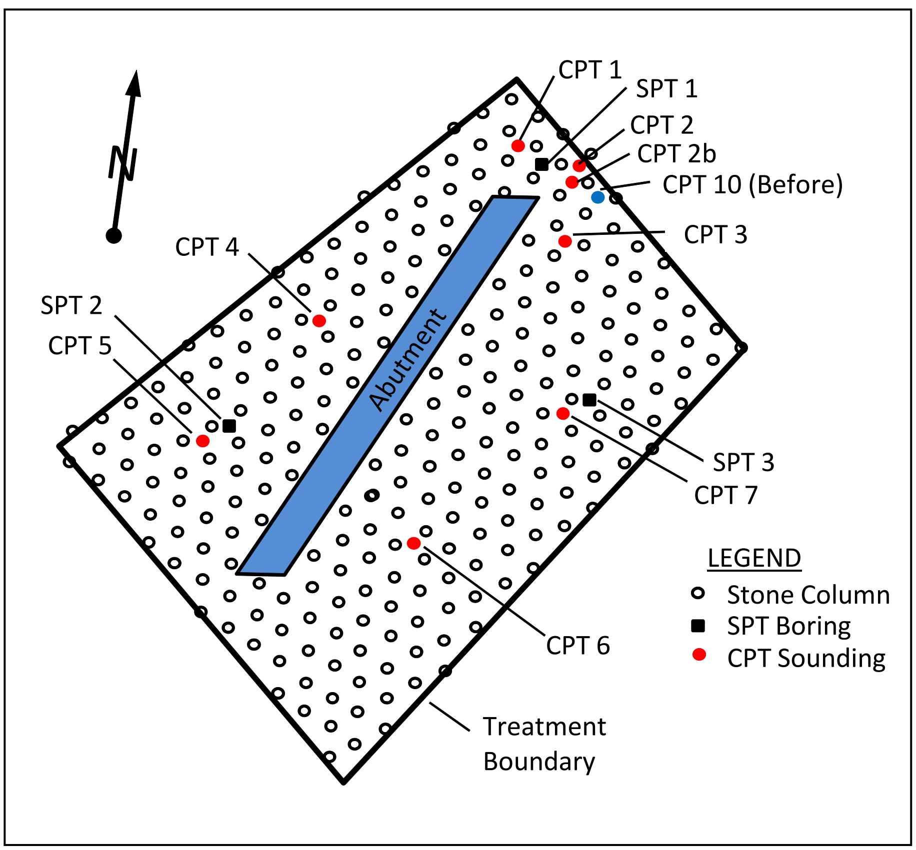

Figure 4. Layout of SPT and CPT holes relative to the stone columns at east abutment.

4.2 Evaluation for Construction Acceptance

To minimize the influence of thin clay layers, UDOT specified that the $q_{c1n\text{-}cs}$ values and I_c values be computed with a running average over a 0.6 m (2 ft) length interval prior to evaluation. The $q_{c1n\text{-}cs}$ values and I_c values were computed using recommendations by Youd et al (2001). However, no correction factors were applied to account for thin layers in the profile. In addition, no correction was made to the I_c to account for potential changes in the friction ratio before and after treatment. A typical plot of $q_{c1n\text{-}cs}$ and I_c versus depth from CPT 1 is shown in Figure 5. Generally, the average $q_{c1n\text{-}cs}$ values after stone column treatment were greater than the required minimum value of 11.5 MPa (120 tsf). However, as I_c increased, the $q_{c1n\text{-}cs}$ values typically decreased. Moreover, as I_c increased above 2.6, indicating the presence of a clay layer, the $q_{c1\text{-}cs}$ values typically decreased below the 11.5 MPa limit. However, as indicated previously, these zones were not considered problematic in terms of liquefaction.

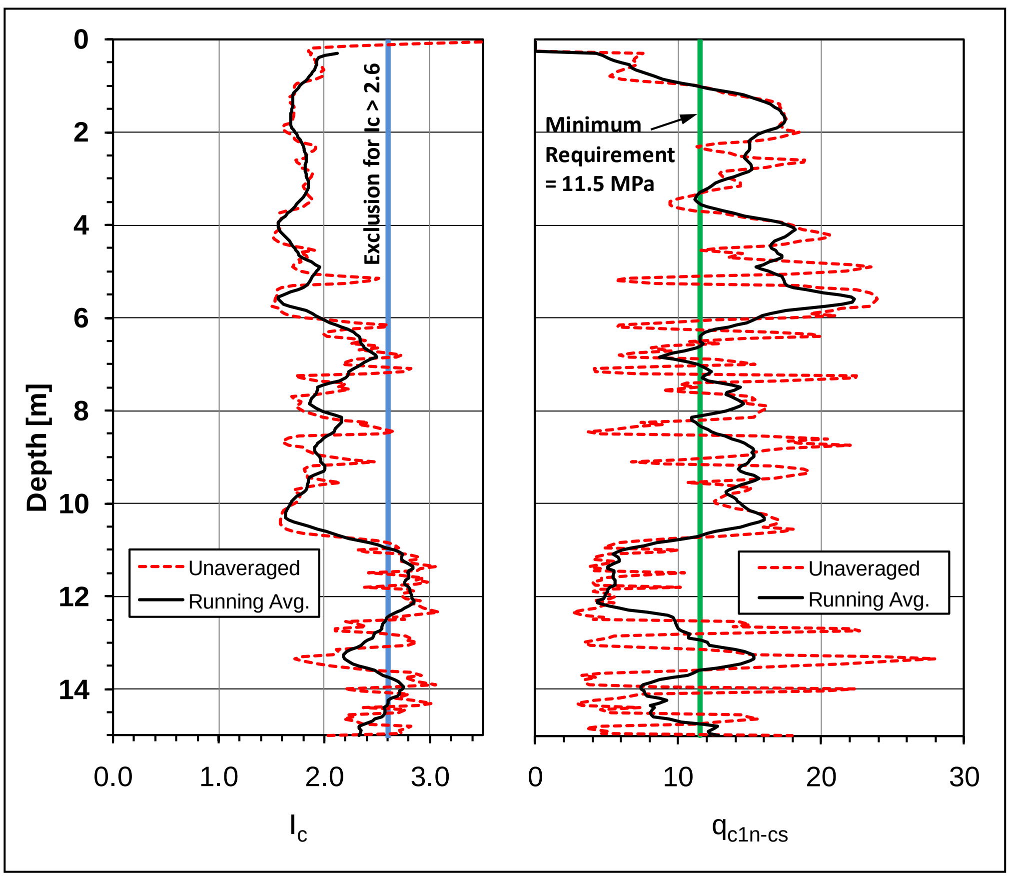

Figure 5. Profile showing $q_{c1n\text{-}cs}$ and I_c values versus depth relative to the required values.

Because of the highly variable q_{c1-cs} profiles produced by the thin interbedded layering, it was generally impractical to make direct comparisons of the improvement produced by stone column treatment as a function of depth. However, using running average q_{c1-cs} profiles some comparisons are possible.

For example, Figure 6 provides a comparison of the running average q_{c1-cs} profiles for CPTs 2 and 3 after treatment in comparison with a profile for nearby CPT 10 before treatment. Typically, improvement in q_{c1-cs} can be observed for the silty sand layers, however, in the silt layers there is relatively little improvement and in some cases the tip resistance actually decreases.

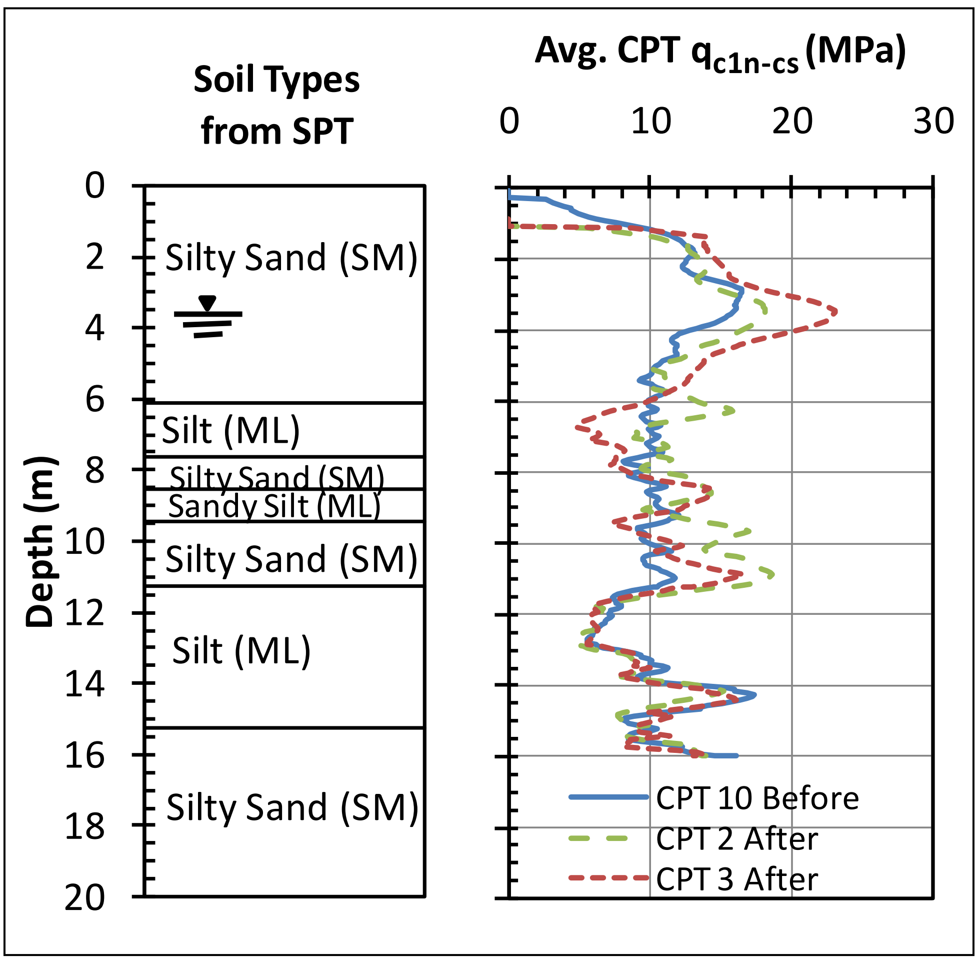

Figure 6. Plots of running average q_{c1n-cs} vs. I_c before and after stone column treatment along with soil profile from SPT.

Because of concerns about the accuracy of the CPT in assessing cohesive layers, SPT borings were performed at several CPT holes. When the I_c value was above 2.6, the soil layer usually classified as ML or CL-ML or was identified as having silty clay lenses and layers. The fines content was measured on SPT samples at 40 locations and these values are plotted against the I_c values at adjacent CPT soundings in Figure 7 along with a best-fit curve. The relationship between I_c and fines content used by Robertson and Wride (1998) for liquefaction evaluation is also shown in Fig. 7 for comparison. In nearly all cases, the measured fines content for a given I_c value was higher than predicted using the Robertson and Wride relationship. For example, an Ic of 2.6 corresponds to a fines content of 35% for the Robertson and Wride relationship, while it was closer to 100% for this data set. Similar findings were noted by Pease (2010) for sites in Nevada and California.

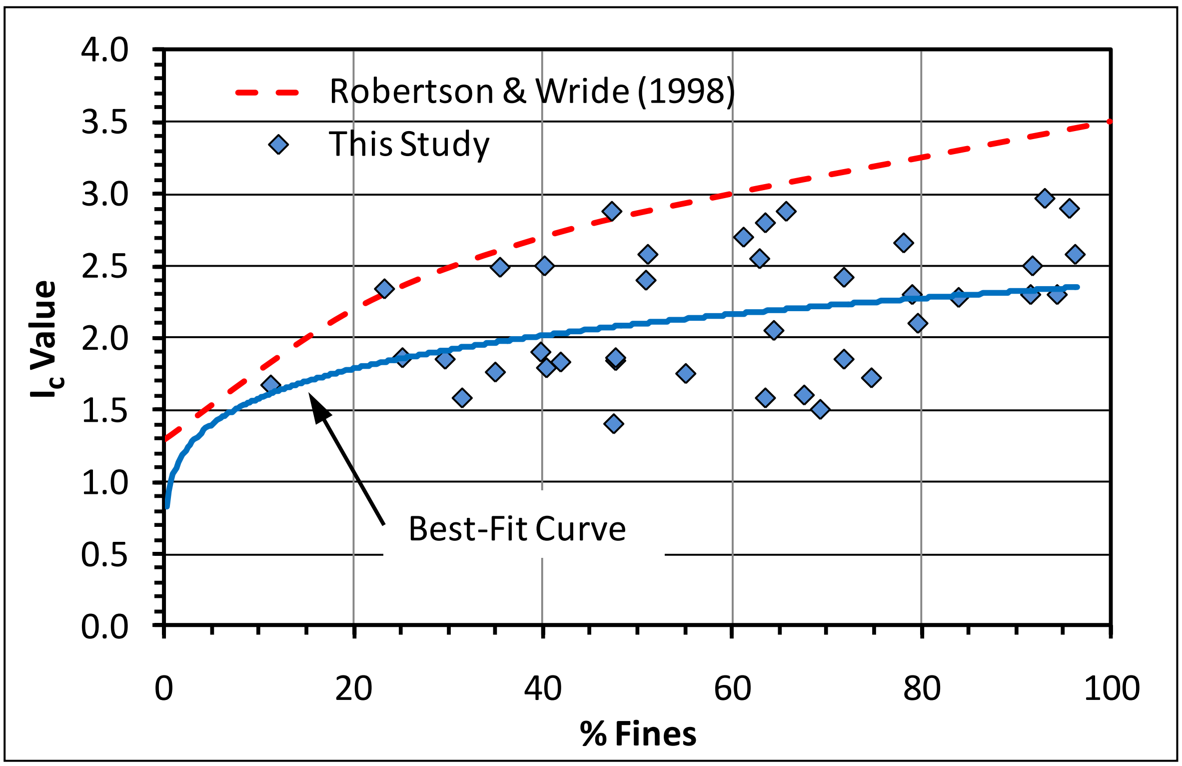

Figure 7. I_c vs. fines content data from the Hinckley Drive site in comparison with relationship used by Robertson & Wride (1998) for liquefaction evaluation.

4.3 Improvement Evaluation Based on q_{c1n-cs} vs. I_c

Because of the variation in the soil profiles for CPT soundings before and after treatment, it was not possible to obtain a reliable direct side-by-side comparison of the improvement at the site. However, to provide an indication of the improvement that was produced by the stone column treatment, plots of q_{c1n-cs} vs. I_c have been produced using all the CPT data before and after treatment. This format is a convenient form because I_c can serve as a proxy for both fines content and plasticity index which are known to influence both q_{c1n-cs} and the efficiency of vibratory compaction. Because of space constraints, data will only be presented for the east abutment. Data points for unaveraged q_{c1n-cs} vs. I_c are shown in Figure 8 while data points for averaged data are shown in Figure 9. There are 386 data points prior to treatment and 1086 data points after treatment.

For all data sets, there is a clear trend for the q_{c1n-cs} values to decrease as I_c increases. This is consistent with the fact that the cone tip resistance typically decreases as the fines content increases and as the soil behaves more like clay.

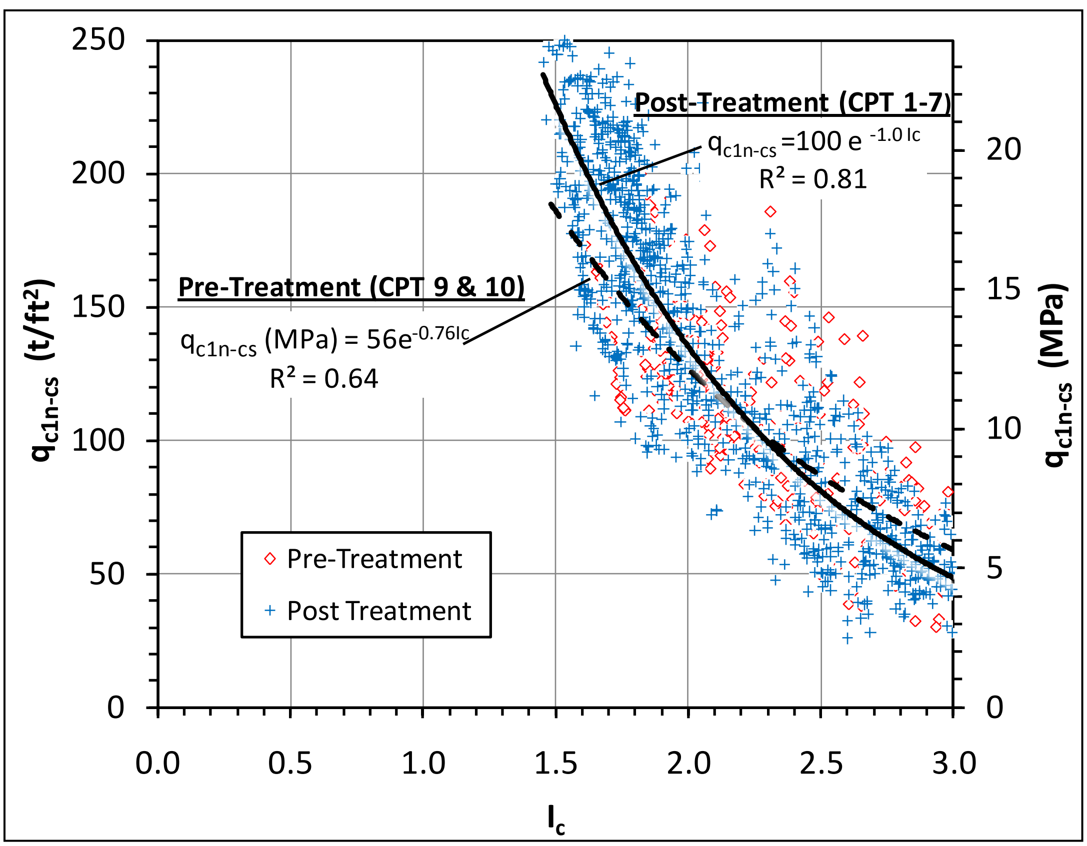

Figure 8. Plots of unaveraged q_{c1n-cs} vs. I_c before and after stone column treatment.

The best-fit curves for each data set were obtained using an exponential relationship. Correlation coefficients for post-treatment data were reasonably high with values around 0.80; however, the pre-treatment correlation coefficients were somewhat lower. Lower values may be attributable to the smaller data set involved and the fact that the soundings are relatively far apart. The equations and R^2 values for conditions before and after treatment are shown in Figures 8 and 9. As the I_c values increase above 1.5, the difference decreases between the best-fit curves before and after treatment. Generally, the two curves intersect at an I_c value of about 2.3 to 2.4, beyond which the post-treatment tip resistance is less than prior to treatment.

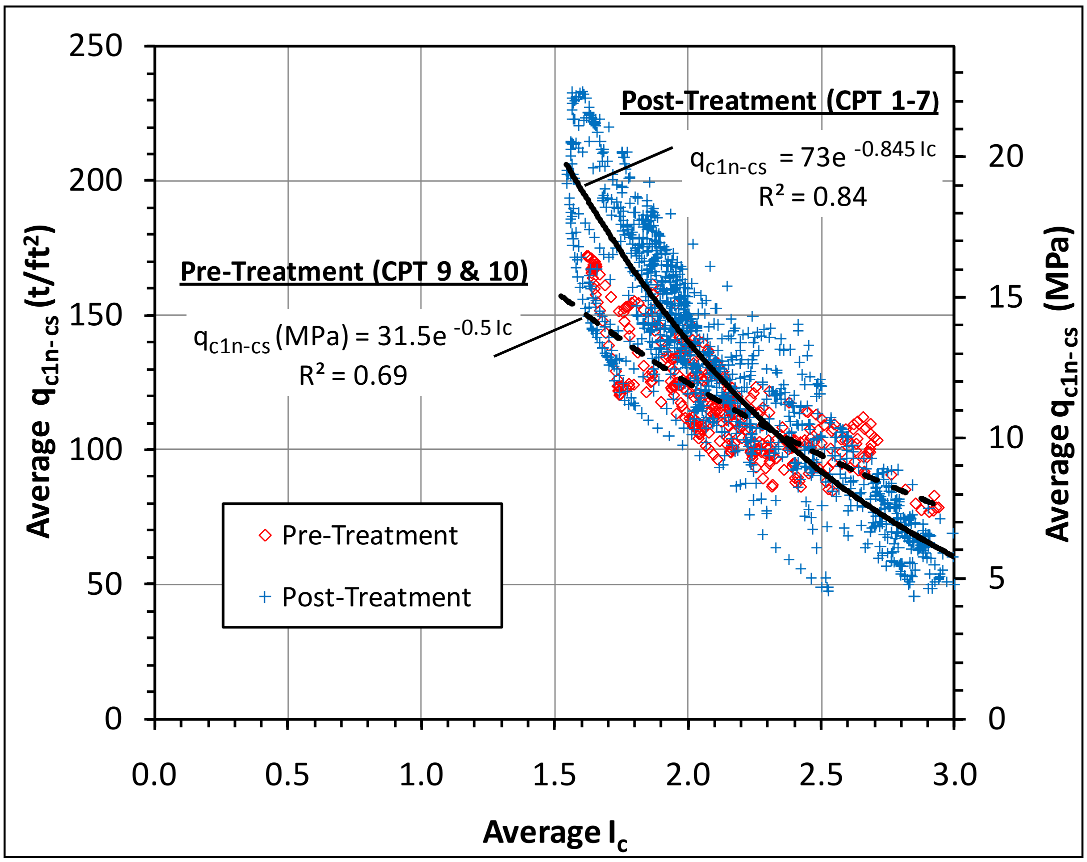

Figure 9. Plots of running average $q_{c1n\text{-}cs}$ vs. I_c before and after stone column treatment.

Using the best-fit curves shown in Figures 8 and 9, curves showing the percent improvement in $q_{c1n\text{-}cs}$ with I_c have been plotted in Figure 10. For I_c values of 1.5, the improvement in cone tip resistance was as high as 20 to 30%. However, for I_c values around 2.2 to 2.3 there was no improvement. Tip resistance decreased more than 10% when I_c values exceeded 2.6. The results in

Figures 8 through 10 are consistent with previous experience (Mitchell, 1982; Rollins et al. 2006) which indicates that the efficiency of vibratory compaction methods decreases as the fines content and plasticity of the soil increase.

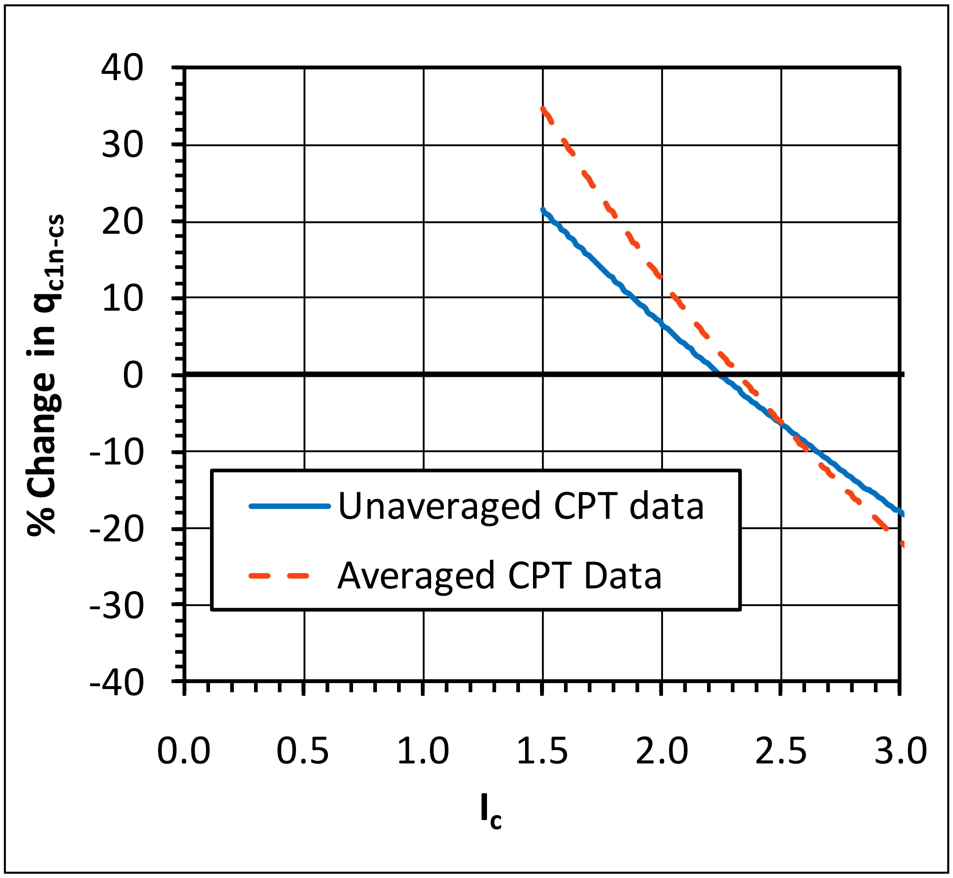

Figure 10. Plot showing the percent change in $q_{c1n\text{-}cs}$ after stone column treatment for averaged and unaveraged values.

A number of investigators have noted that penetration resistance often increases as a function of time after treatment for various soil improvement techniques (Mitchell and Solymar 1984, Mesri et al. 1990, Schmertmann 1991). To evaluate these time rate effects at this site, CPT 2b was performed about 5 weeks after CPT 2, which was performed about 2 weeks after stone column treatment.

To help sort out the effects of the fines on the improvement, the average $q_{c1n\text{-}cs}$ was plotted versus I_c for these two CPT soundings as shown in Figure 11. Exponential trend lines, with equations, are also shown along with the data points in Figure 11. The plots in Figure 11 show that there is a clear increase in the $q_{c1n\text{-}cs}$ values obtained 7 weeks after treatment relative to the values 2 weeks after treatment. However, the increase appears to be less substantial as the I_c value increases. Based on the two best-fit trend lines in Figure 11, the average percent increase in $q_{c1n\text{-}cs}$ seven weeks after treatment has been determined relative to the value two weeks after treatment. The percent increase is plotted versus I_c in Figure 12. The percent increase in $q_{c1n\text{-}cs}$ decreases almost linearly with I_c. Increases in $q_{c1n\text{-}cs}$ are 20% to 30% for I_c values from 1.5 to 2.3, respectively. However, for I_c values greater than 2.6 the percent increase in $q_{c1n\text{-}cs}$ was less than 16%.

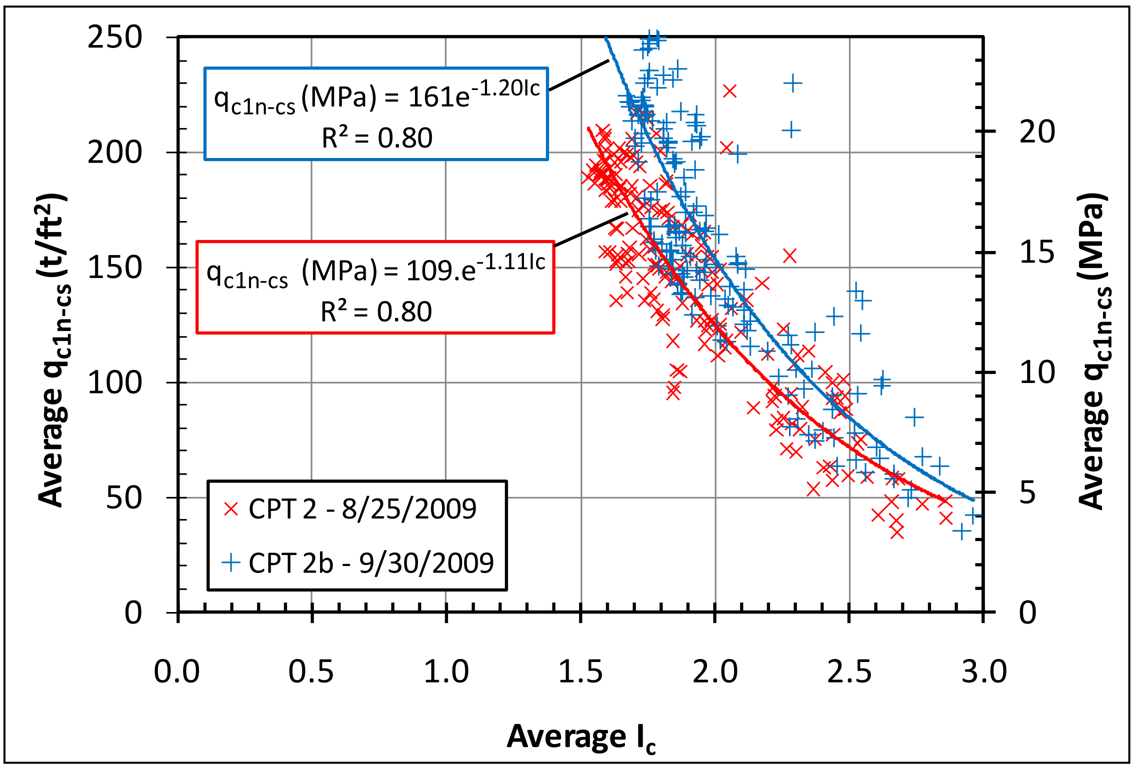

Figure 11. Plots of running average $q_{c1n\text{-}cs}$ vs. I_c two weeks after stone column treatment (CPT 2) and seven weeks after treatment (CPT 2b).

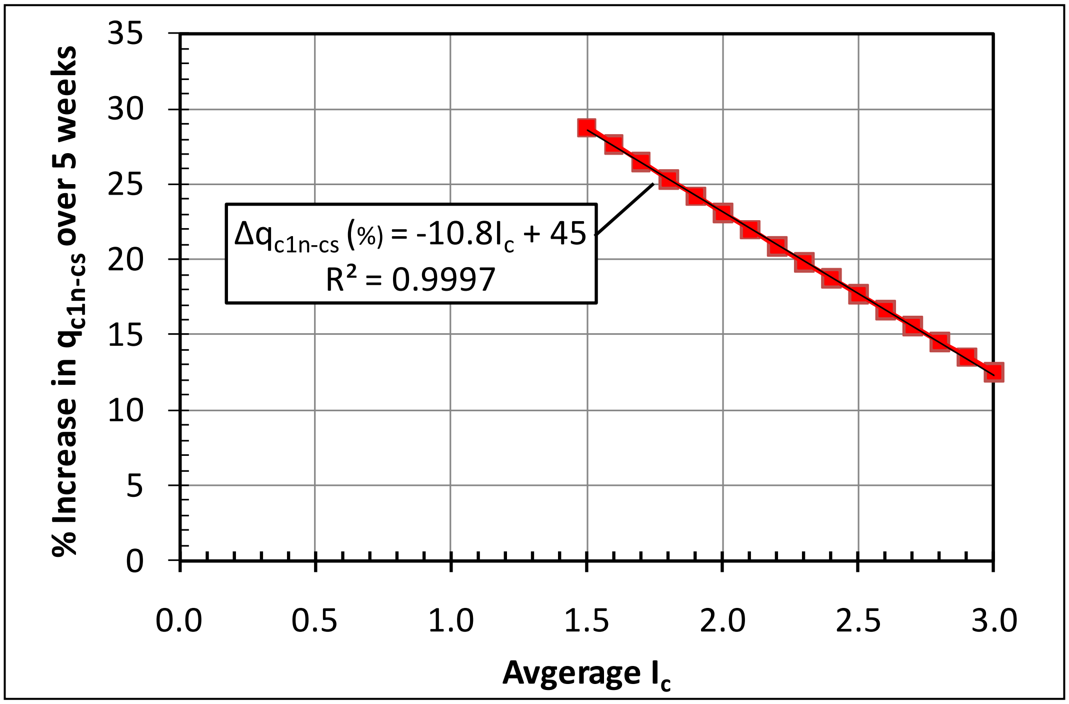

Figure 12. Plots of the percent increase in $q_{c1n\text{-}cs}$ vs. I_c seven weeks after stone column treatment relative to the value obtained two weeks after treatment.

4.4 Improvement Evaluation Based on q_c and F_r

Massarch (1991) proposed that the compactability of a soil could be defined as “compactable”, “marginally compactable” and “not compactable” based on the cone tip resistance and friction ratio (F_r) as shown in Figure 13. Of course, these classifications were developed without considering the use of wick drains in connection with the compaction.

Data points before and after treatment were separated out based on the three classifications and average values before and after treatment are plotted in Figure 13 for each CPT sounding. CPTs 9 and 10 were performed prior to treatment and are shown with solid symbols while CPTs 1-7 were performed after treatment and have open symbols. A visual review of the average data points in Figure 13 indicates that the improvement in q_c clearly decreases as the data points move from the “compactable” to the “not compactable” categories. However, the improvement for the “marginally compactable” category was about the same as that for the “compactable” category. These results indicate that the “not compactable” boundary is appropriate while the “marginally compactable” boundary is somewhat conservative.

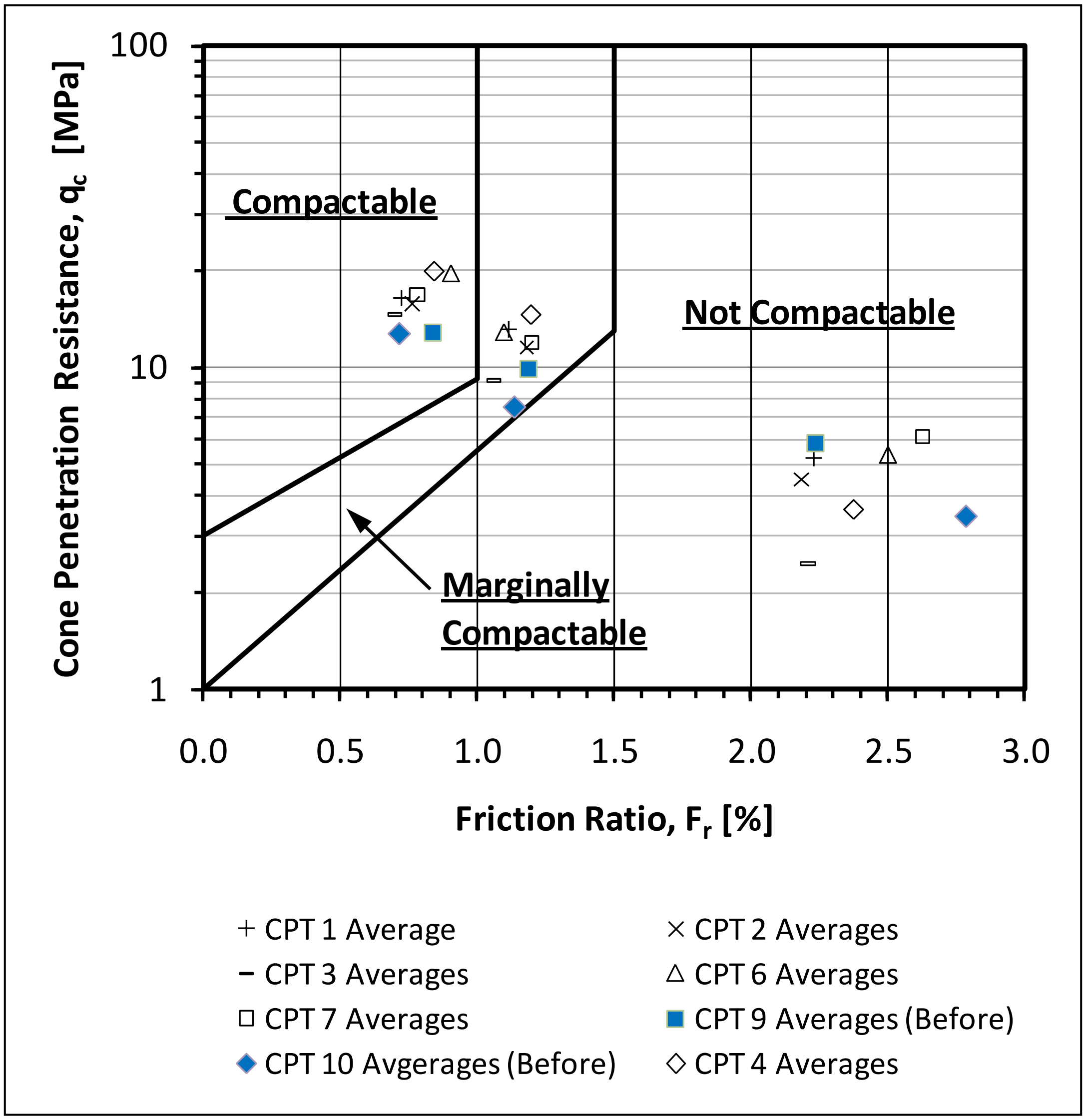

Figure 13. Compactability classification for deep vibratory compaction based on CPT data (After Massarch, 1991).

Table 1 shows the average q_c before and after treatment for the data points which fall within the three compactability categories along with the percent improvement for each category. The percent improvement was about the same for the "compactable" and "marginally compactable" categories, while a small decrease in q_c was reported for the "not compactable" category.

Table 1. Summary of average q_c values in three compactability categories and percent improvement owing to stone column treatment at Hinckley Drive site.

Compactability Category	Avg. q_c Before Treatment (MPa)	Avg. q_c After Treatment (MPa)	Change in q_c (%)
Compactable	12.84	17.46	36.0
Marginally Compactable	9.34	13.57	45.0
Not Compactable	5.07	4.65	-9.0

5.0 CONCLUSIONS

Based on the results of the field testing and the analysis of the test data, the following conclusions have been developed:

1. Evaluation of stone column treatment with the CPT provided a continuous record of tip resistance and I_c which facilitated the identification of cohesive layers not likely subject to improvement.
2. For the soils at this test site, an I_c greater than 2.6 provided a strong indication that the soil classified as a silt or clay and had a higher fines content than predicted by the Robertson and Wride (1998) relationship for these low-plasticity soils.
3. The results from this site indicated that the densification produced by stone column treatment with A_r of 14% was minimal for I_c values greater than about 2.3.
4. CPT soundings showed an increase in penetration resistance over a five week period after treatment. However, the percent increase in q_{c1n-cs} decreased almost linearly with increasing I_c. Increases in q_{c1n-cs} were 20% to 30% for I_c values from 1.5 to 2.3, respectively, but were less than 16% for I_c values greater than 2.6.
5. Correlations between q_{c1n-cs} and I_c before and after densification provide a useful means of evaluating the effectiveness of stone column treatment. I_c can serve as a proxy for the fines content and plasticity index which both influence compaction efficiency.

6. The compactability criteria defined by Massarch (1991) based on q_c and F_r appears to provide a reasonable estimate of the "Not Compactable" boundary for this case history. However, the boundary for "Marginally Compactable" was somewhat conservative.

6.0 REFERENCES

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