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Increase in Cyclic Liquefaction Resistance of Sandy Soil Due to Installation of Drilled Displacement Piles

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ABSTRACT: Drilled displacement piles are installed by the displacement of soil and subsequent placement of fluid cement grout within the evacuated volume. Depending on the soil grain-size characteristics, soil behavior, *in situ* soil density, pile spacing, and pile diameter, the installation process can result in measurable densification and an increase in lateral stress. Thus, it follows that the drilled displacement process may be used to mitigate cyclic liquefaction in sandy soils. In an effort to quantify the increase in cyclic liquefaction resistance resulting from drilled displacement piles, the authors performed pre- and post-installation cone penetration testing at a site in North Myrtle Beach, South Carolina. The results support that the installation of drilled displacement piles generally increases the liquefaction resistance of the surrounding soil. For a specific area replacement ratio, the trend is that improvement decreases with increasing pre-installation $(q_{t1N})_{cs}$ and approaches unity at very high values of pre-installation $(q_{t1N})_{cs}$. The improvement can be significant for values of pre-installation $(q_{t1N})_{cs}$ generally as high as 250. Overall, the degree of improvement is greater for higher area replacement ratios.

INTRODUCTION

As described by NeSmith (2002), drilled displacement piles (also known as augered cast-in-place displacement or augered, pressure grouted displacement piles) are installed by the displacement of soil and subsequent placement of fluid grout within the evacuated volume. Siegel *et al.* (2007a; 2007b) showed that the installation process can result in ground improvement as characterized by measurable increases in cone tip resistance and sleeve friction in sandy soils. Thus, it follows that the potential for liquefaction can be intentionally reduced by the installation of drilled displacement piles in sandy soils.

This paper examines the reduction in cyclic liquefaction potential due to the ground improvement within groups of drilled displacement piles. To characterize the soil conditions, the authors' performed cone penetration testing (CPT) prior to and following the installation of drilled displacement piles at a site in North Myrtle Beach, South Carolina. Using the CPT data, the normalized (equivalent) cone tip resistance for clean sands [$(q_{t1N})_{cs}$] is used to graphically represent the reduction in cyclic liquefaction potential in sandy soils resulting from the installation of drilled displacement piles.

INSTALLATION PROCESS

The Berkel tool (Figure 1) used in the drilled displacement process consists of a bottom auger section with a length of 0.9 m (3 ft), a displacement section that is equal to the nominal diameter of the pile, and few flights of reverse auger above the displacement section. As this tool is advanced to the pile tip depth, the soil in the pile volume is displaced horizontally. Once the tool has reached the pile tip depth, then fluid cement grout is pumped downward through the hollow stem and tool, and introduced into the hole through the grout port at the tip of the tool. Once a sufficient amount of grout has been pumped to fill the volume between the tool and the hole, then the tool is extracted in a controlled manner while grout is pumped at a rate sufficient to fill the hole. The drilled displacement process typically results in somewhat lower grout factors (*i.e.*, the ratio of the actual grout volume to the theoretical hole volume) than typically used for conventional augered, cast-in-place piles.

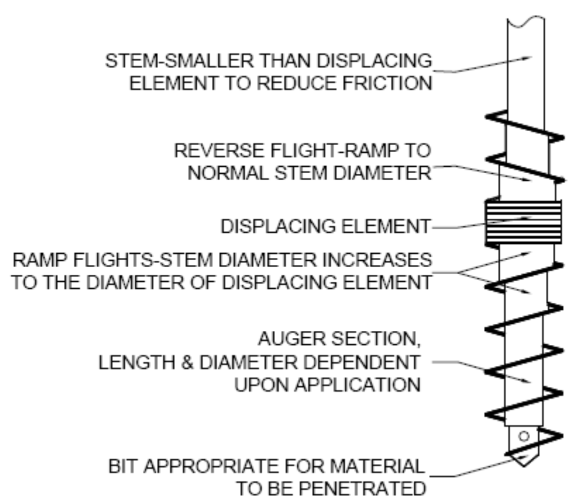

Figure 1. Berkel Drilled Displacement Pile Tool

The installation platform (Figure 2) is typical of those used in European continuous flight auger (CFA) applications. It includes a vertical mast with an attached turntable capable of producing 25 meter-tons (180,000 ft-lbs) of torque and a cabling system that allows a downward force (or crowd) of 356 kN (40 tons). Pile lengths greater than 17 m (56 ft) with diameters of up to 457 mm (18 inches) are routinely installed with this system. The system is also adaptable to larger equipment that can increase the maximum pile length and the practical pile diameter.

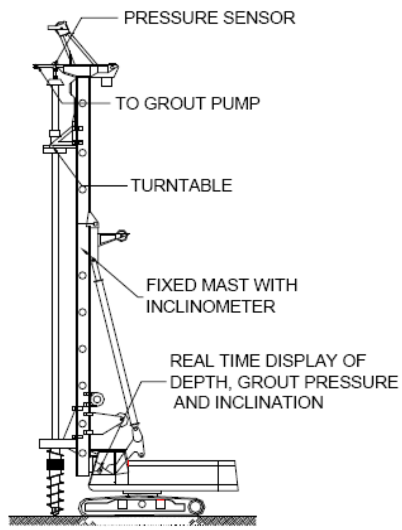

Figure 2. Installation Platform for Drilled Displacement Piles

NORTH MYRTLE BEACH (SC) TEST SITE

The test site is a beachfront development in North Myrtle Beach, South Carolina, which is located within the Coastal Plain Physiographic Province. The upper 8.5 m (28 ft) of the subsurface profile consist of beach sands and shell hash that are Pleistocene age deposits. Locally, these sands and shell hash are underlain by the Pee Dee Formation. The testing at this site focused on the conditions in the upper beach sands and shell hash because the presence of limestone lenses within Pee Dee Formation was expected to present significant difficulties with respect to testing and interpretation. Figure 3 presents a CPT profile of the beach sands and shell hash. Note that the majority of the profile for the pre-installation data classifies as sands and sand mixtures (I_c < 2.60) according to the classification system by Robertson and Wride (1998). There are thin clay layers at depths of 4.5 m to 5.5 m (14.8 ft to 18 ft) and 7 m to 8 m (23 ft to 26.2 ft).

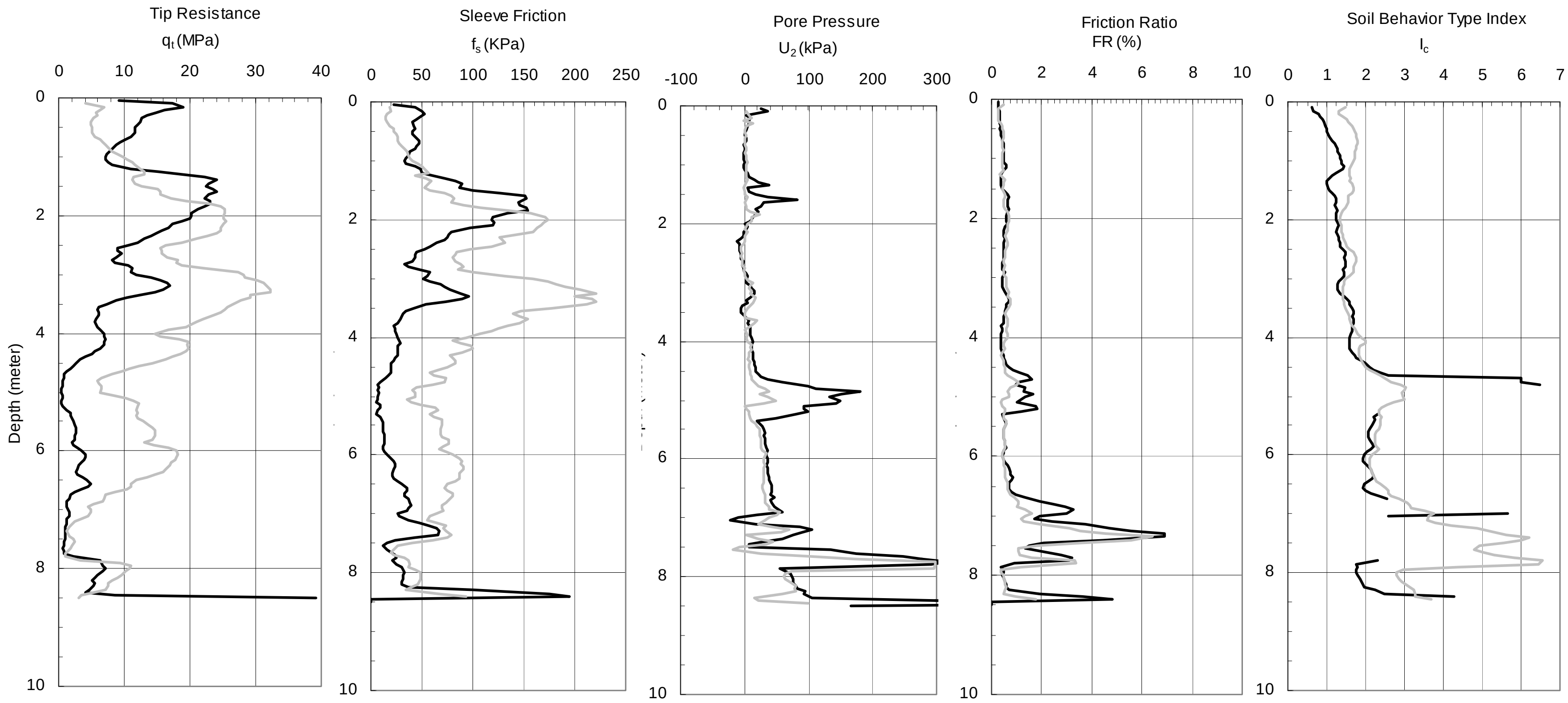

Figure 3. CPT Profiles for Original Site Conditions (Dark Lines) and for a Pile Group with a_s=0.068 (Light Lines)

The testing was performed in the area of planned 4 x 3 pile cap of 406 mm (16 in) diameter drilled displacement piles. Figure 4 shows the various pile group configurations included in this study. The piles were installed with a Bauer BG 25 drilling platform. As previously discussed in detail, the installation involves advancing the displacement tool to the design depth and then extracting the displacement tool at a slow forward rotation while pumping a pressurized grout through a port at the tip of the displacement tool. Typical of this displacement system, the upper soils (approximately 1.5 m) are displaced upward during the initial penetration of the displacement tool and otherwise the spoil generation is negligible. Observations by the authors of extracted piles from other sites confirm that this process results in a very uniform cross-section with a diameter equivalent to that of the displacement tool.

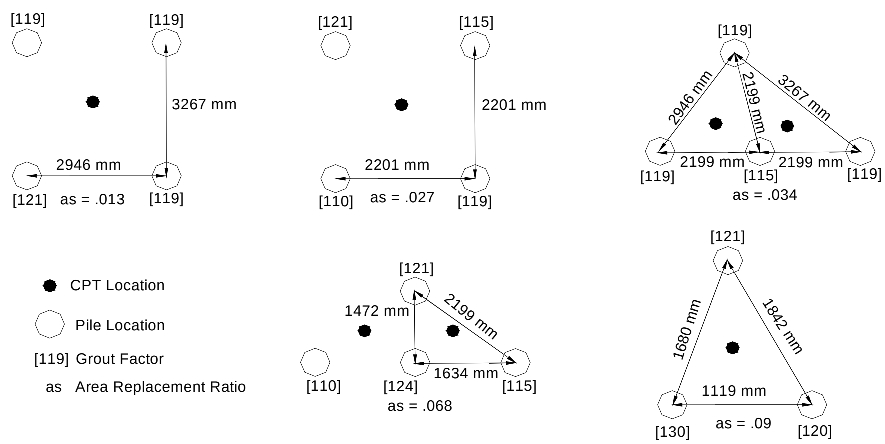

Figure 4. Configurations of Drilled Displacement Pile Groups and Relative CPT Locations

DATA ANALYSIS

Liquefaction is the soil behavior phenomenon in which saturated sand softens and loses strength due to the development of high pore water pressures during strong ground shaking (Seed and Idriss; Silver and Seed, 1971). In this study, the reduction in cyclic liquefaction potential is quantified using the normalized (equivalent) cone tip resistance for clean sands $(q_{t1N})_{cs}$ computed using the CPT-based cyclic liquefaction prediction method by Robertson and Wride (1998) and the recommendations by NCEER (Youd *et al.*, 2001). The advantages of using $(q_{t1N})_{cs}$ are that it considers both tip resistance and sleeve friction in a single value, it may be calculated for soils with a range of fines content, and it may be used to directly estimate the cyclic resistance ratio (CRR) from published charts (Robertson and Wride, 1998; Youd *et al.*, 2001).

DATA PRESENTATION

The influence of displacement pile installation is represented by the ratio post-installation $(q_{t1N})_{cs}$ to the pre-installation $(q_{t1N})_{cs}$. The Improvement Ratio [$R_{(qt1N)cs}$] is defined by the following expression:

$$\text{Improvement Ratio:} \quad R_{(qt1N)cs} = \frac{(q_{t1N})_{cs\_post\_installation}}{(q_{t1N})_{cs\_pre\_installation}}$$

The Improvement Ratio, $R_{(qt1N)cs}$, was computed at each measurement depth increment of 50 mm (2 in). The water table was assumed to be at the ground surface in computing $R_{(qt1N)cs}$. Only data collected below 1.5 m (5 ft) were considered because the lack of confinement (as illustrated by the observed soil displacement previously described by the authors) precludes densification very near the working ground surface. Note that the testing was performed in the center of the pile group with the exception of the configuration with an a_s = 0.068. For this exception, the CPT location was shifted slightly to one side of the triangular pile arrangement.

The relationship between pile cross-sectional area and pile spacing are represented by the area replacement ratio (a_s) which is defined as the cross-sectional area of the pile divided by the tributary area for each pile. Because this is an actual project, the pile group configurations were dictated by the design. The area replacement ratios, which range from 0.013 to 0.09, were computed by graphically determining the total area and pile area bounded by the pile groups.

RESULTS AND DISCUSSION

The plots presented in Figure 5 summarize the results of this study and show the pre-installation $(q_{t1N})_{cs}$ versus the Improvement Ratio, $R_{(qt1)cs}$ for the various area replacement ratios. The plots indicate that installation of drilled displacement piles generally results in an increase in liquefaction resistance. For a specific area replacement ratio, the trend is that $R_{(qt1N)cs}$ decreases with increasing pre-installation $(q_{t1N})_{cs}$ and approaches unity at very high values of pre-installation $(q_{t1N})_{cs}$. The improvement can be significant for values of pre-installation $(q_{t1N})_{cs}$ generally as high as 250. Overall, the degree of improvement is greater for higher area replacement ratios.

It is proposed that the increases in liquefaction resistance, in terms of the Improvement Ratio, due to the installation of drilled displacement piles in sandy soils are conservatively represented in Figure 5. The testing within the pile groups was performed near or at the midpoint of the pile configurations which is believed to provide conservatism to the application of these values as an estimate for the entire soil volume. Recent field observations and numerical analyses by Martin and Olgun (2006) support that stiff inclusions were effective at mitigation liquefaction-induced settlement as a result of a high vertical composite stiffness. This suggests that there is a beneficial reinforcement aspect not considered by the Improvement Ratio.

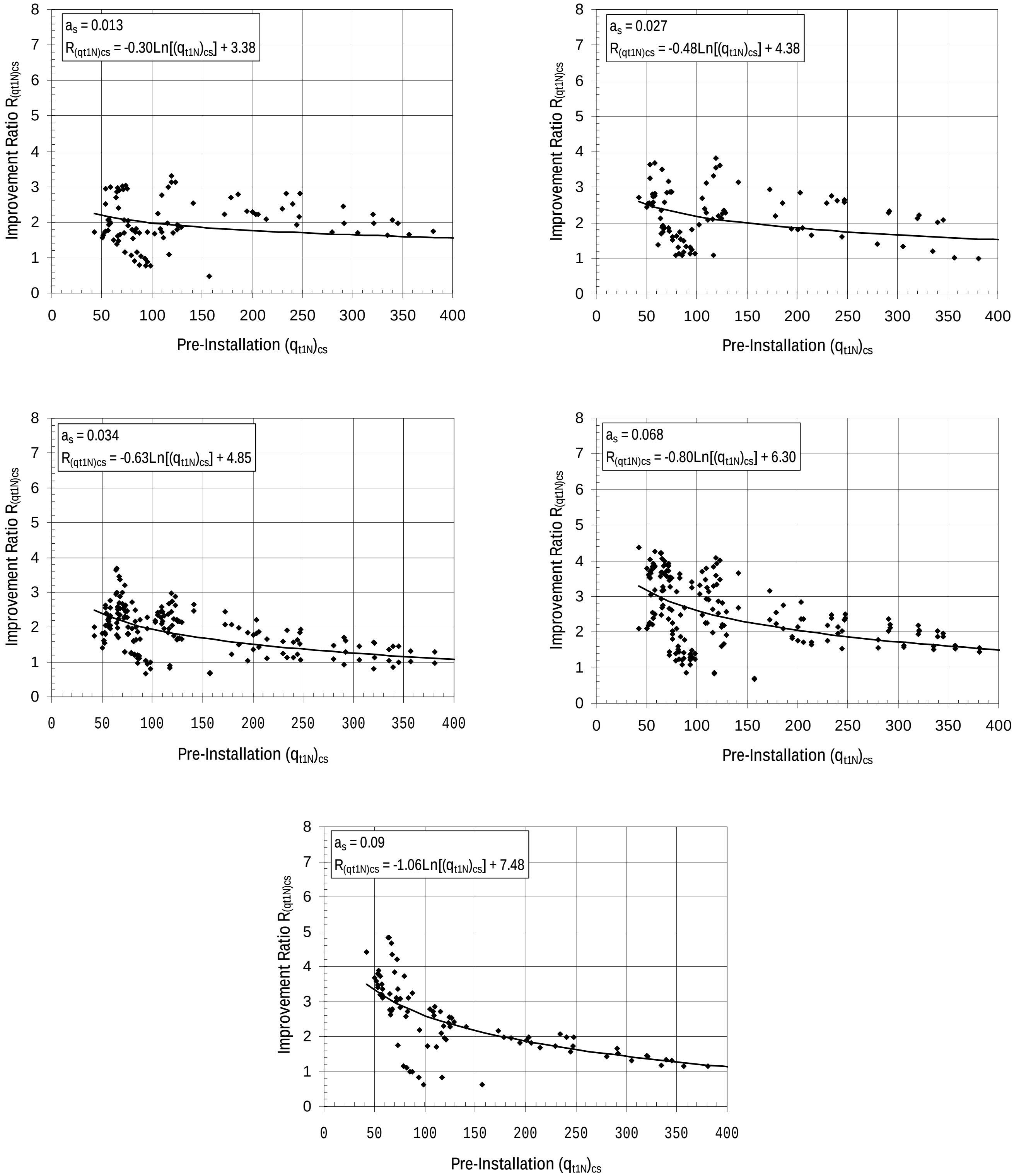

Figure 5. Improvement Ratios for Various Area Replacement Ratios

In the application of the plots presented in Figure 5, it may be helpful to discuss the limitations of the CPT-based liquefaction methodology. While a number of field studies (Mitchell and Solymar, 1984; Schmertmann, 1987; Mesri *et al*., 1990; Charlie *et al*., 1992) have shown that the tip resistance continues to increase for some time after the application of a variety of ground improvement techniques, the effect of time on the CPT resistances is not well understood. The data presented in this paper is based on testing performed within one or two days after installation and there has been no adjustment made for the influence of time on the cone penetration test data.

As illustrated in Figure 3, this study shows that there is a measurable reduction in friction ratio between the pre-installation data and the post-installation data. Greater changes in friction ratio have been reported after ground improvement by dynamic compaction (Tan *et al.*, 2007). This raises a pertinent question as to appropriate consideration where the Soil Behavior Type Index (I_c) changes based on the post-installation (or post-improvement) CPT data. A strict evaluation of I_c indicates that post-installation soils have a higher fines content. It is more likely that there is actually no change in soil type, but that the post-installation soil conditions are not well calibrated with the friction ratio-based classification systems. While this study used both pre- and post-installation values of I_c computed from the respective testing, such an approach may lead to inappropriate conclusions regarding liquefaction potential where there is a more dramatic change in I_c.

The CPT-based liquefaction prediction and the associated calculation of $(q_{t1N})_{cs}$ does not consider effects of aging, overconsolidation, geologic setting and depositional conditions (Pyke, 2003; Youd *et al.*, 2003). In general, the absence of such considerations is conservative; however, this conservatism may be reduced or eliminated by the disturbance induced by the ground improvement process. Therefore, the results of this study are most appropriately considered along with all of the other pertinent factors when determining whether to implement the drilled displacement process to increase the liquefaction resistance at a particular site.

CONCLUSIONS

Cone penetration testing was performed at a beachfront site in North Myrtle Beach, South Carolina. Baseline data were collected for the natural ground conditions and compared to data collected within groups of drilled displacement piles. The reduction in cyclic liquefaction potential was quantified using the normalized (equivalent) cone tip resistance for clean sands $(q_{t1N})_{cs}$ computed using the CPT-based cyclic liquefaction prediction method. The results support that the installation of drilled displacement piles generally increases the liquefaction resistance of the surrounding soil. For a specific area replacement ratio, the trend is that improvement decreases with increasing pre-installation $(q_{t1N})_{cs}$ and approaches unity at very high values of pre-installation $(q_{t1N})_{cs}$. The improvement can be significant for values of pre-installation $(q_{t1N})_{cs}$ generally as high as 250. Overall, the degree of improvement is greater for higher area replacement ratios.

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