CPT MEASUREMENTS NEAR DRILLED DISPLACEMENT PILES

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ABSTRACT

Drilled displacement piles (also known as augered cast-in-place displacement piles or augured, pressure-injected displacement piles) are installed by the displacement of soil and subsequent placement of 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. Cone penetration testing was performed at a beachfront site in Myrtle Beach, South Carolina. Baseline data were collected for the natural ground conditions and compared to data collected adjacent to a single pile and within groups of displacement piles. The data, which are presented as a ratio of the post-installation measurements to the pre-installation measurements, support that the installation of drilled displacement piles in granular soil results in a substantial increase in tip resistance and sleeve friction as measured in the CPT. As may be expected, measurements near a single pile show that the measured post-installation tip resistance reduces as the distance from the pile increases. Within pile groups, the post-installation tip resistance in looser sands increases as the pile spacing decreases.

INTRODUCTION

As described by NeSmith (2002), drilled displacement piles are installed by the displacement of the soil within the pile volume. The drilling tool (shown in Figure 1) used to install displacement pile consists of a bottom auger section with a length of 0.9 m (3 feet), a displacement section that is equal to the nominal diameter of the pile, and a few flights of reverse auger above the displacement section. As this tool is advanced, the soil in the pile volume is displaced horizontally. Depending on the soil grain-size characteristics, *in situ* soil density, pile spacing, and pile diameter, the installation process can result in measurable densification and increased lateral stress. It is intuitive that the conditions near drilled displacement piles are likely to be similar to the conditions surrounding other types of displacement foundations. Meyerhoff (1959) examined the conditions surrounding a driven circular pile and concluded that pile installation induced compaction and an increase in the principal stress ratio. Nataraja and Cook (1982) observed an increase in N-value between driven displacement piles. Several researchers and practicing engineers (Solymar *et al.*, 1986; Barksdale, 1987; Shamoto *et al.*, 1997) have described the improved conditions after the installation of sand piles. Chen & Kulhawy (2001) compiled field data showing a substantial increase in relative density in granular soils surrounding pressure-injected footings.

In an effort to characterize the improved soil conditions after installation of drilled displacement piles, the authors performed cone penetration testing at a site in North Myrtle Beach, South Carolina. The initial testing was performed prior to the installation of drilled displacement piles. Subsequent testing was performed adjacent to a single pile and between groups of three and four piles within one or two days of pile installation.

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 feet) 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 2 presents a CPT profile of the beach sands and shell hash prior to installation of any displacement piles. Note that the majority of the profile classifies as silty sands (SBT=7) and sands (SBT=8 or 9) according to the sleeve friction based system proposed by Robertson *et al.* (1986). There are thin clay layers at depths of 4.5 m to 5.5 m and 7 m to 8 m.

The testing was performed in the area of planned 4 x 3 pile cap of 406 mm (16 in) diameter drilled displacement piles. The piles were installed with a Bauer BG 25 drilling platform. 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. As 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 confirm that this process results in a very uniform cross-section with a diameter equivalent to that of the displacement tool.

DATA PRESENTATION

The influence of displacement pile installation is represented by the ratio of the post-installation data compared to the pre-installation data. For the cone tip resistance and sleeve friction, the ratios are expressed as follows:

Tip resistance ratio: $$R_q := \frac{q_{t\_post\_installation}}{q_{t\_pre\_installation}}$$

Sleeve friction ratio: $$R_{fs} := \frac{fs_{post\_installation}}{fs_{pre\_installation}}$$

R_q and R_{fs} were computed at each depth increment of 50 mm (2 in). Only soils described by a SBT of 7, 8 or 9 by the pre-installation testing were considered in computing R_q and R_{fs}. Also, only data below 1.5 m (5 ft) were considered because it is believed that the lack of confinement (as illustrated by the observed soil displacement) precludes densification very near the 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 = 0.52. For this exception, the CPT location was shifted to one side of the triangular pile arrangement.

The data is presented in terms of normalized corrected tip resistance, q_{t1}, which is defined as:

Normalized corrected tip resistance: $$q_{t1} := \left(\frac{q_t}{p_a}\right)\left(\frac{p_a}{\sigma'_{vo}}\right)^{0.5}$$

where σ'_{vo} is the vertical effective stress and p_a is atmospheric pressure.

RESULTS

Testing adjacent to a single pile (with a diameter, D, of 406 mm) was performed at distances of 1.5D, 2.5D, 3.5D, and 4.5D from the center-of-pile. The data from these tests are graphically illustrated in Figures 3, 4, and 5. Figure 3 compares q_t versus depth for both the pre-installation and post-installation measurements. Figures 4 and 5 illustrate the variation of R_q and R_{fs}, respectively, with the pre-installation normalized corrected cone tip resistance (q_{t1}). These plots show that R_q and R_{fs} range up to approximately 4 and 3.5, respectively and that the higher values represent soils with a q_{t1} of less than 50. As may be expected, there is a moderate reduction in R_q as the distance away from the pile increases, but this trend is less clear for R_{fs}.

Testing was performed within the pile group configurations shown in Figure 6. 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 (A_p) divided by the tributary area (A) 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 determined by graphically determining the total area and pile area bounded by the pile groups.

The data collected within pile groups are presented in Figures 7 and 8. Figures 7 and 8 illustrate the variation of R_q and R_{fs}, respectively, with the pre-installation q_{t1} for the different values of a_s. In an effort to identify useful trends, Table 1 presents the mean and standard deviation of R_q and R_{fs} within select ranges of q_{t1}. The analysis represented by Table 1 shows that the highest mean values of R_q and R_{fs} represent soil with a q_{t1} of 50 or less and, for this range of q_{t1}, the mean R_q and mean R_{fs} increase with increasing a_s. The relationships between a_s and the mean values of R_q and R_{fs} do not appear proportional at higher q_{t1}. When q_{t1} is 50 or less, the mean R_q is generally greater than the mean R_{fs}. When q_{t1} is greater than 50, the mean R_{fs} is generally greater than the mean R_q.

Table 1. Summary of R_q and R_{fs} for Conditions within Pile Groups

	Range of q_{t1}	**Mean Ratios (Standard Deviation)**				
		a_s = .013	**a_s = .027**	**a_s = .034**	**a_s = .068**	**a_s = .09**
R_q	**0 to 50**	3.0 (1.0)	3.4 (1.2)	4.1 (2.1)	5.1 (2.4)	5.3 (1.5)
	50 to 100	1.8 (0.8)	2.1 (0.9)	1.9 (0.7)	2.5 (1.1)	2.2 (0.8)
	100 to 150	2.4 (0.2)	2.3 (0.5)	1.7 (0.4)	2.2 (0.4)	1.9 (0.1)
	150 to 200	2.2 (0.4)	2.2 (0.6)	1.4 (0.3)	2.1 (0.3)	1.7 (0.3)
	200 to 250	1.9 (0.2)	1.7 (0.6)	1.2 (0.3)	1.8 (0.24)	1.3 (0.1)
R_{fs}	**0 to 50**	2.9 (1.2)	3.7 (1.7)	3.5 (1.4)	4.5 (2.0)	4.4 (1.7)
	50 to 100	2.6 (1.6)	3.0 (1.6)	2.5 (1.1)	3.1 (1.6)	2.5 (0.9)
	100 to 150	2.9 (1.6)	2.8 (1.0)	1.9 (0.6)	2.9 (0.9)	2.0 (0.2)
	150 to 200	3.0 (0.8)	3.0 (1.2)	1.5 (0.6)	2.8 (0.8)	2.0 (0.5)
	200 to 250	3.0 (0.6)	2.3 (1.1)	1.5 (0.6)	2.8 (0.9)	1.6 (0.4)

DISCUSSION

The results of this study show that the installation of drilled displacement piles in granular soil results in a substantial increase in tip resistance and sleeve friction as measured by the CPT. Furthermore, there is a clear trend that increases in R_q and R_{fs} are proportional to the area replacement ratio for soils exhibiting a pre-installation q_{t1} resistance of 50 or less. At higher values of q_{t1}, there is no clear trend with either R_q or R_{fs} with area replacement ratio. It is the authors' belief that the results are influenced by the variation in soil density through the depth being considered. Particularly, looser soils (represented by lower q_{t1} values) experience significant densification during pile installation while the densification in denser soils is slightly more modest because the denser soils tend to transfer stress to nearby looser soils. The authors' anticipate that granular soils of more uniform density would exhibit less variation in R_q and R_f throughout the profile.

There are at least two important issues that the authors' have not addressed with respect to the application of this data in engineering analysis. First, while it is recognized that the CPT is well established as useful measurement of ground improvement (Lunne *et al.*, 1996; Dove *et al.*, 2000;

Shaefer & White, 2004; Mackiewicz & Camp, 2007), it is also known that its results are influenced by changes in both density and stress (Meyerhoff, 1959; Masmood & Mitchell, 1993; Salgado *et al.*, 1997). In some analyses, it may be useful to determine the relationship between the R_q, R_{fs}, density, and *in situ* stress state. Second, the testing did not attempt to establish the influence of time on R_q, R_{fs}, and R_{Vs}. A number of field studies show that the tip resistance continues to increase for some time after the application of various soil densification techniques (Mitchell & Solymar, 1984; Schmertmann, 1987; Mesri *et al.*, 1990; Charlie *et al.*, 1992). However, the effect of aging on the behavior of sands is not well understood as illustrated by the laboratory study by Baxter & Mitchell (2004) that was unable to reproduce the increase in tip resistance over time that is typical observed in the field. Until the aging of sands is better understood, it seems prudent to establish the influence of time on a project-by-project basis if no specific historical data are available.

It is proposed that the increases in tip resistance due to the installation of drilled displacement piles in sands of varying density are conservatively represented by the values of R_q presented in this paper. That is, the testing for this study was performed within one- to two days of pile installation and the tip resistance is expected to increase with time. This does not also apply to side friction as Charlie *et al*. (1992) reported a decrease in normalized local friction with time. Furthermore, the testing within pile groups was performed near or at the midpoint of the pile configuration which is believed to provide added conservatism to the application of these values as an estimate for the entire soil volume.

CONCLUSIONS

Cone penetration testing was performed at a beachfront site in Myrtle Beach, South Carolina. Baseline data were collected for the natural ground conditions and compared to data collected adjacent to a single pile and within groups of drilled displacement piles. The results of this study support that the installation of drilled displacement piles in granular soil results in a substantial increase in tip resistance and sleeve friction as measured by the CPT. The measurements near a single pile show that the post-installation tip resistance reduces as the distance from the pile increases. Analysis of the data collected within pile groups revealed that increases in R_q and R_{fs} (values representing the increases in the tip resistance and sleeve friction as defined herein) are proportional to the area replacement ratio for granular soils exhibiting a pre-installation q_{t1} (normalized corrected tip resistance) of 50 or less. At higher values of q_{t1}, there is no clear trend with either R_q or R_{fs} with area replacement ratio. It is proposed that the increases in tip resistance due to the installation of drilled displacement piles in sands of varying density are conservatively represented by the values of R_q presented in this paper.

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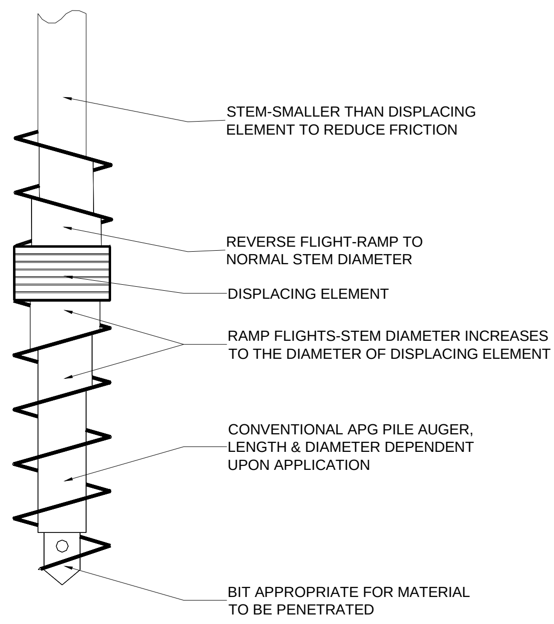

Figure 1. Berkel Displacement Tool

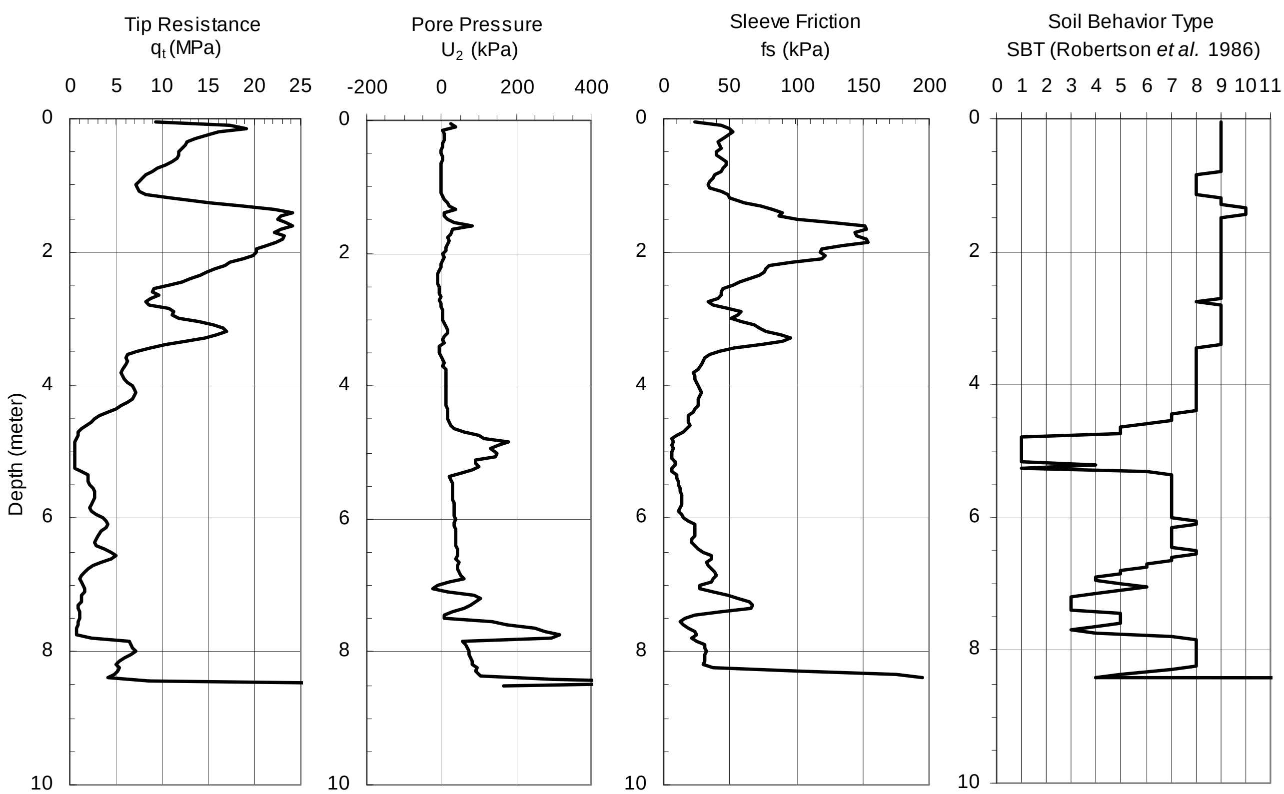

Figure 2. CPT Profile for Site Conditions

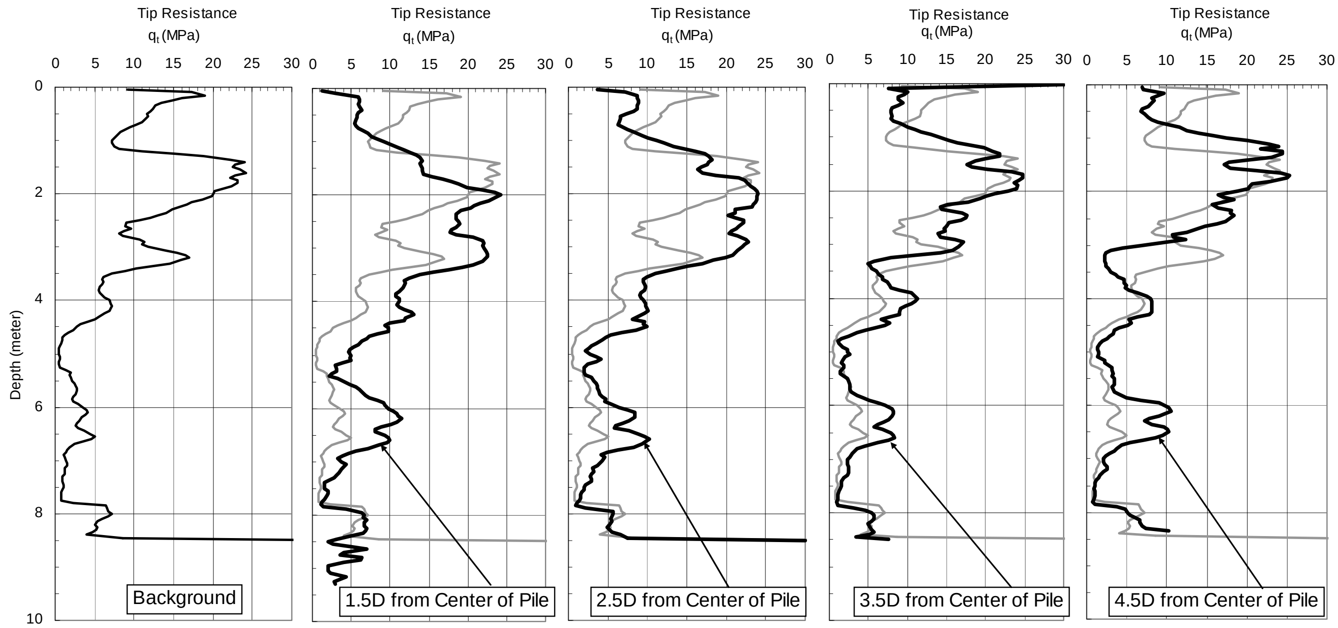

Figure 3. Tip Resistance Versus Depth Near Single Displacement Pile

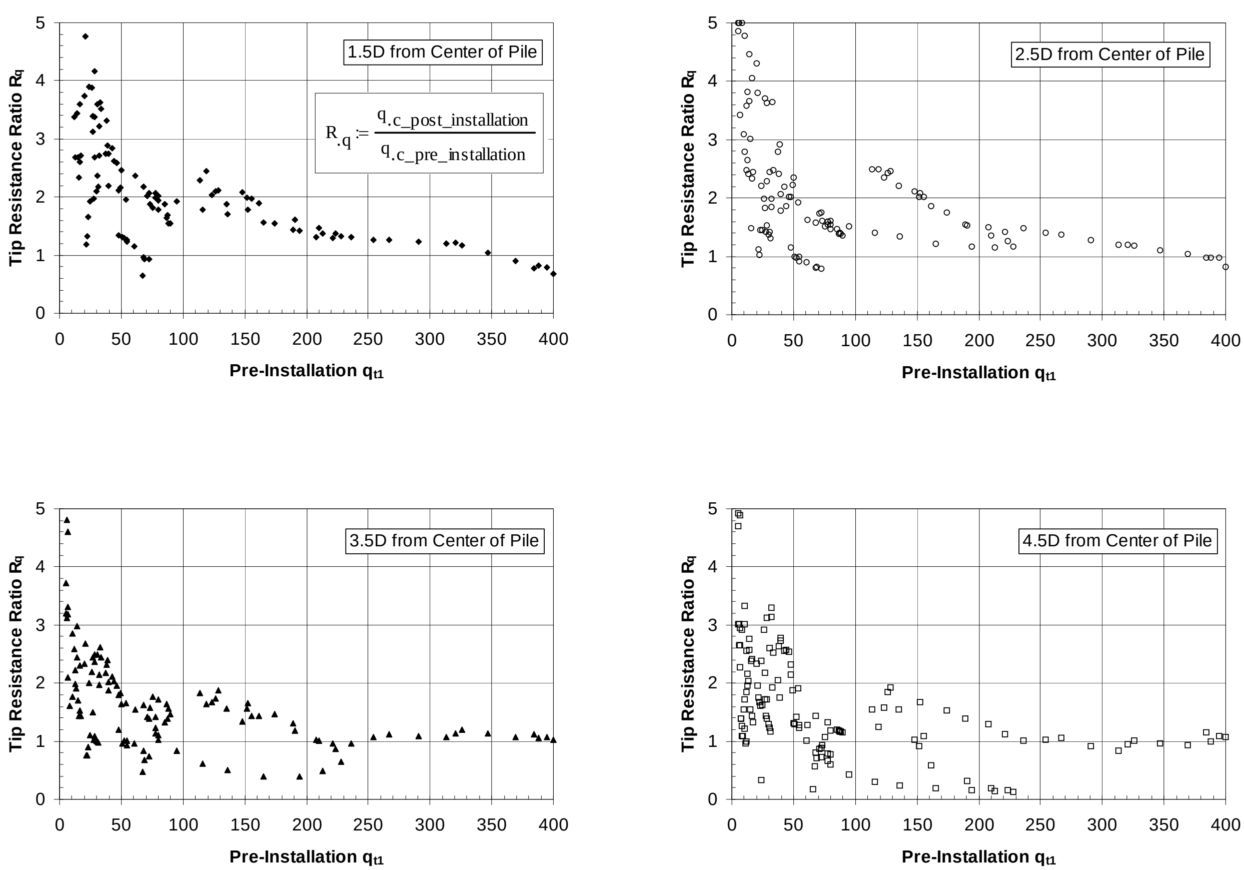

Figure 4. Tip Resistance Ratio Near Single Drilled Displacement Pile

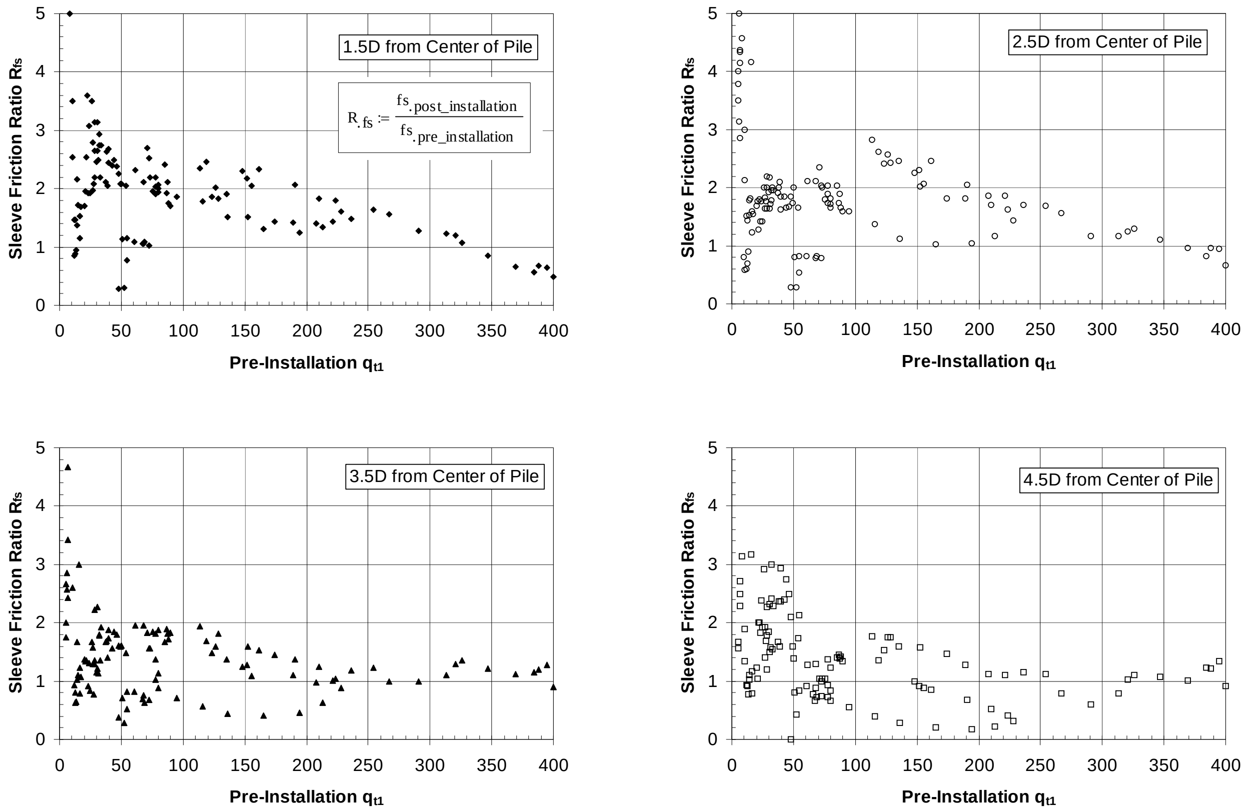

Figure 5. Sleeve Friction Ratio Near Single Drilled Displacement Pile

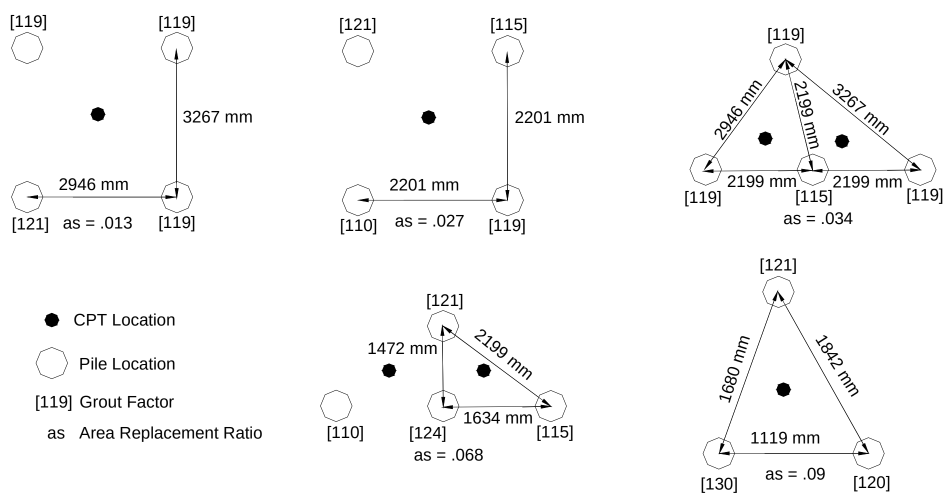

Figure 6. Configurations of Displacement Pile Groups and Relative CPT Locations

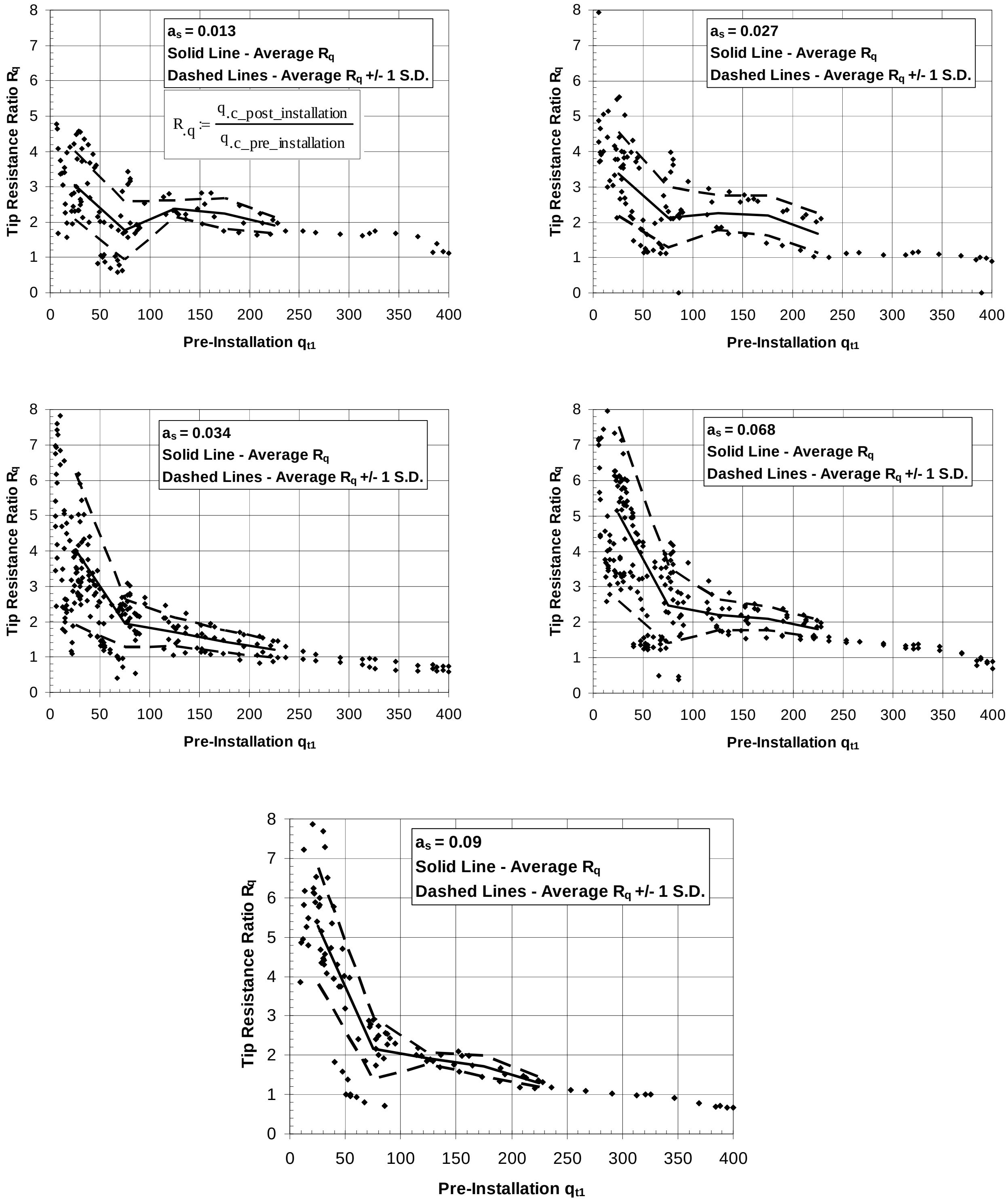

Figure 7. Tip Resistance Ratio for Various Area Replacement Ratios

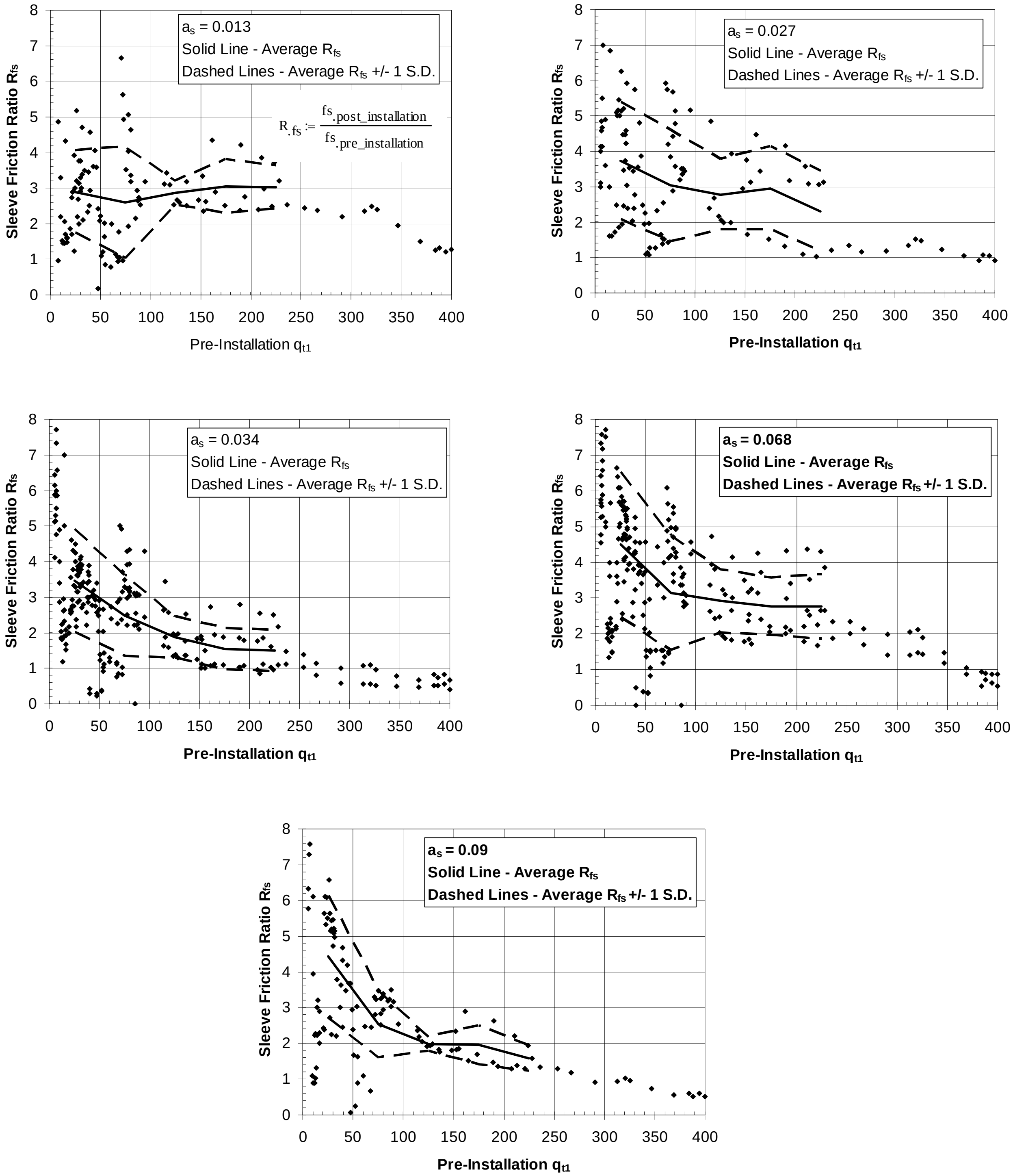

Figure 8. Sleeve Friction Ratio for Various Area Replacement Ratios