Short Term Decrease in Undrained Shear Strength due to Helical Pile Installation

Gustavo Padros
ARKK Engineering Corp., Sherwood Park, Alberta, Canada
Richard Schmidt
Almita Piling Inc., Edmonton, Alberta, Canada
Jose Antonio Padros
IPESA, Mexico City, Mexico

ABSTRACT
The results of a test program to measure the short term decrease in undrained shear strength due to the installation of helical piles in stiff to very stiff cohesive soils is presented. The test program comprised the installation of 2 helical piles with single helix and double helices and undertaking a total of 17 Cone Penetration Tests (CPT), distributed inside and outside the helix footprint. Two CPT's were pushed prior to helical pile installation, allowing the determination of native soil conditions. The ratio of the uncorrected tip resistance (q_c) measured in the CPT tests pushed after and before helical pile installation (q_cCPTa/q_cCPTb) was considered indicative of the loss in undrained shear strength due to soil disturbance. The results for the single helix pile indicated q_cCPTa/q_cCPTb ratios ranging from 0.6 to 0.8 inside the helix footprint and from 0.75 to 0.95 outside the helix footprint. In the case of the double helix pile, the ratios ranged from 0.35 to 0.75 inside the helix footprint and from 0.65 to 1.0 outside the helix footprint. It was concluded that the decrease in undrained shear strength at the test site due to helical pile installation was smaller than what may have been initially considered. The potential increase in soil shear strength with time was not analyzed in the present investigation.

RÉSUMÉ
Les résultats d'un programme d'essai visant à mesurer la diminution à court terme de la résistance au cisaillement non drainée due à l'installation de pieux hélicoideaux dans des sols cohésifs rigides à très rigides sont présentés. Le programme d'essai comprenait l'installation de 2 pieux vissés à simple hélice et à double hélice et un total de 17 tests de pénétration de cône (CPT), répartis à l'intérieur e tà l'extérieur de l'empreinte de l'hélice. Deux CPT ont été poussés avant l'installation du pieu vissé, ce qui a permis de déterminer les conditions du sol indigène. Le rapport de la résistance de pointe non corrigée (qc) mesurée dans les essais CPT poussés après et avant l'installation de la pile hélicoidale (qcCPTa/qcCPTb), a été considéré comme indicateur de la perte de résistance au cisaillement non drainée due à la perturbation du sol. Les résultats pour l'empilement d'une suele hélice indiquent des rapports qcCPTa/qcCPTb allant de 0.6 à 0.8 dans l'empreinte de l'hélice et de 0.75 à 0.95 à l'extérieur de l'empreinte de l'hélice. Dans le cas de la pile à double hélice, la perturbation du sol à l'intérieur de l'empreinte de l'hélice variait de 0.35 à 0.75 et à l'extérieur de l'empreinte de l'hélice variait de 0.65 à 1.0. Il a été conclut que la diminution de la résistance au cisaillement non drainé sur le site d'essai en raison de l'installation de pieux hélicoïdeaux était inférieure à ce qui aurait pu être initialement envisagé. L'augmentation potentielle de la résistance au cisaillement du sol avec le temps n'a pas été analysée dans le présente étude.

1 INTRODUCTION

Installation of deep foundations in cohesive soils produces disturbance in the soil structure, comprising remolding, the alteration of the state of stresses in the vicinity of the pile and the temporary increase in pore water pressures. The undrained shear strength of cohesive soils typically decreases as a result of disturbance. Upon pore water dissipation and depending on the clay mineralogy, some amount of shear strength initially lost may be recovered with time. However, as the result of soil disturbance, the pile compressive and tension resistance decreases and greater horizontal displacements may be expected in the lateral load pile performance.

The installation procedure of a deep foundation will have an influence in the amount of disturbance produced in the soil. For the case of helical piles, the rotation of the helices in combination with the downward penetration of the pile will disturb the soil in the vicinity of the pile.

Accurate estimation of the reduction in shear strength thus becomes very important when deep foundations are designed. Soil disturbance may be estimated in the laboratory using sensitivity tests (Padros et.al. 2012-1). Given their method of installation, helical piles in particular are considered by some engineers as a type of foundation that produces high soil disturbance. However, when the soil shear strength parameters are back-calculated from compressive, uplift and lateral load tests undertaken in helical piles installed in stiff to very stiff cohesive soils, the results often indicate the amount of disturbance was moderate (Padros et.al. 2012-2). Furthermore, installation effects on low sensitivity soils may be negligible.

The objective of this paper is to measure the reduction in undrained shear strength produced in stiff to very stiff cohesive soils following the installation of two helical piles, based on the results from Cone Penetration Tests (CPT) located in the vicinity of the pile, pushed prior and after helical pile installation.

2 TEST PROCEDURE

2.1 Methodology

In order to determine the disturbance produced in a cohesive soil by helical pile installation, the following methodology was applied:

(a) Selection of a site where the subsurface conditions consisted mostly of stiff to very stiff clay. A site west of Ponoka, AB was selected (test location coordinates 52º41.509'N and 113º59.153'W). The site map is shown in Figure 1. The subsurface conditions are discussed in Section 2.2.

(b) Selection of pile sizes to be installed: Single and double helix helical piles were selected, designated SP-1 and SP-2, respectively. Both piles had the same, relatively large helix diameter (914 mm). The piles spacing is shown in Figure 2. The helical pile sizes and embedment depths are shown in Figure 3.

Figure 1. Site map

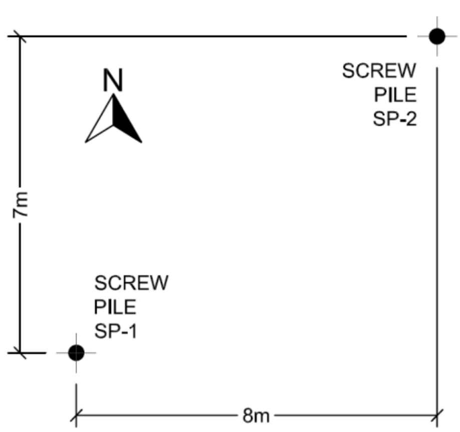

Figure 2. Helical pile spacing

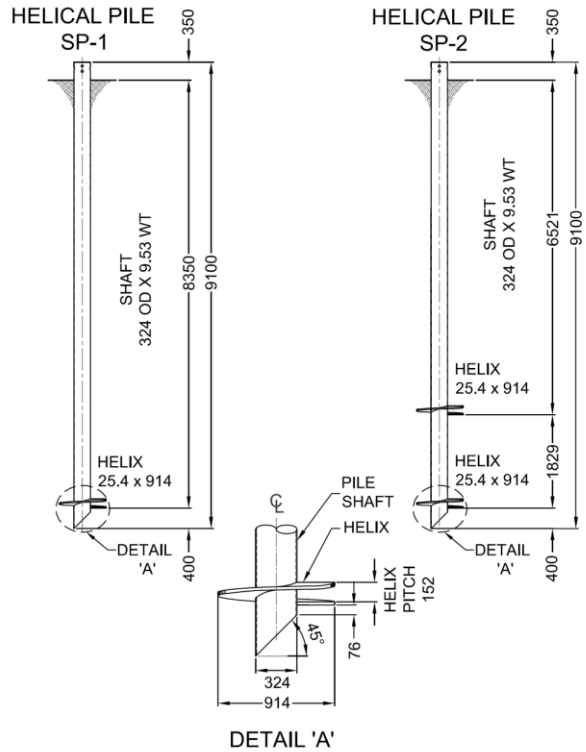

Figure 3. Helical pile sizes and embedment depth (mm)

(c) On October 9, 2013, a test program was undertaken, comprising the installation of helical piles SP-1 and SP-2 and pushing a total of 17 CPT's to depths ranging from 6.0 m to 11.8 m. The CPT's depth and their distance to the pile centre are indicated in Table 1.

Table 1. CPT Depth and Distance to Pile Centre

CPT	Depth (m)	Distance to Pile Centre (mm)	Adjacent to Helical Pile
CPT1	11.8	133	SP1
CPT2	11.8	133	
CPT3	11.8	135	SP2
CPT4	11.6	135	
CPT5	8.2	302	SP1
CPT6	8.1	302	
CPT7	8.1	297	
CPT8	10.0	612	SP1
CPT9	9.6	612	
CPT10	8.6	312	SP2
CPT11	6.0	312	
CPT12	6.2	312	
CPT13	9.6	612	SP2
CPT14	6.3	612	
CPT15	6.3	612	
CPT16	6.3	612	
CPT17	10.0	612	

(d) The sequence of events (including time) is summarized in Table 2. The CPT's location is shown in Figures 4 and 5, from where it may be noted that some CPT's were located within the helices footprint, whereas other CPT's were located slightly outside the helices footprint. In particular, CPT-10 location allowed it to be pushed between the interior edges of the uppermost helix and penetrate between the helices towards the lowermost helix. CPT-1 and -3 were intended to measure native soil conditions prior to helical pile installation and hence serve as basis for comparison.

(e) In order to investigate the disturbance produced by helical pile installation, a comparison between the values of the uncorrected tip resistance (qc) measured in the CPT tests was undertaken. The series of comparisons carried out and their purpose are summarized in Table 3.

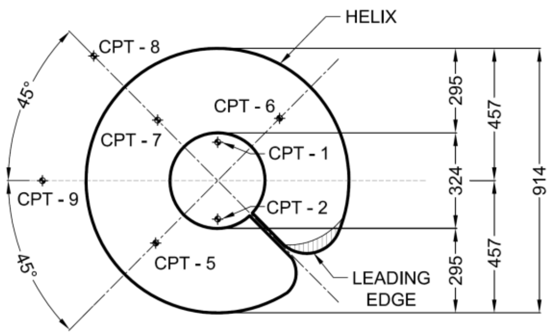

Figure 4. CPT's pushed near SP1

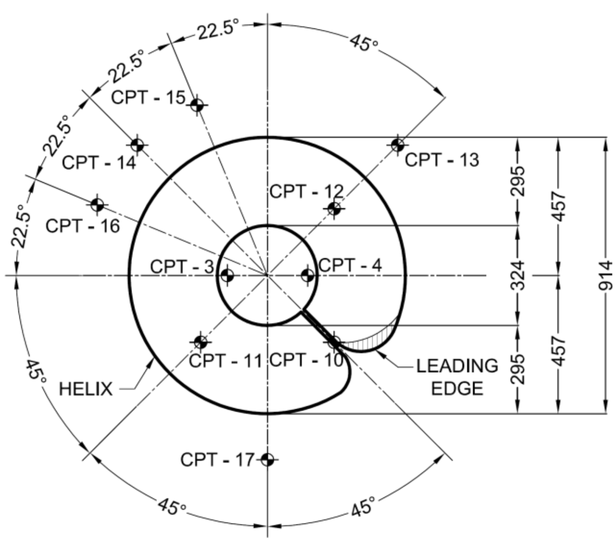

Figure 5. CPT's pushed near SP2

(f) The rationale is that q_c ratios greater than 1 are associated to soil intrinsic variability (heterogeneous nature), whereas q_c ratios smaller than 1 are associated to soil disturbance and soil variability.

Table 2. Sequence of events (October 9, 2013)

Time	Description
7:10 – 7:50	Push CPT-1 and -2 near the centre of SP1
7:50 – 8:30	Push CPT-3 and -4 near the centre of SP2
8:40 – 8:55	Install SP-1
9:00 – 9:15	Install SP-2
9:00 – 10:30	Push CPT-5 to -9 adjacent to SP1
10:30–15:00	Push CPT-10 to -17 adjacent to SP2

Table 3. CPT Comparison and Purpose

Comparison	Purpose	Fig
q_cCPT3 / q_cCPT1	Assess variability of native soil	8
q_cCPT5 / q_cCPT1 q_cCPT6 / q_cCPT1 q_cCPT7 / q_cCPT1	Determine disturbance inside helix footprint at SP1 location	9
q_cCPT8 / q_cCPT1 q_cCPT9 / q_cCPT1	Determine disturbance outside helix footprint at SP1 location	10
q_cCPT10 / q_cCPT3 q_cCPT11 / q_cCPT3 q_cCPT12 / q_cCPT3	Determine disturbance inside helix footprint at SP2 location	11
q_cCPT13 / q_cCPT3 q_cCPT14 / q_cCPT3 q_cCPT15 / q_cCPT3 q_cCPT16 / q_cCPT3 q_cCPT17 / q_cCPT3	Determine disturbance outside helix footprint at SP2 location	12

(g) CPT-2 was located about 270 mm c/c from CPT-1. Similarly, CPT-4 was located about 270 mm c/c from CPT-3. Neither CPT-2 nor CPT-4 were used in the comparisons as their results indicated relatively high soil disturbance due to their proximity with the CPT's pushed first.

2.2 Subsurface Conditions

The subsurface conditions encountered at the site are described next, based on the results obtained in CPT-1 and CPT-3:

(a) A dense granular fill about 0.5 m to 0.6 m thick was present in the surface, overlying:

(b) A stiff to very stiff silty clay layer extending to the termination of the CPT's at about 11 m. The tip resistance ranged from 1 MPa to 2 MPa, which based on correlations (Mayne 2007) corresponded to an undrained shear strength ranging from about 60 kPa to about 120 kPa. Thin sand lenses ranging in thickness from about 0.1 m to about 0.2 m were encountered interbedded in the clay deposit at about 7.7 m depth in CPT1 and at about 2.4 m and 3.4 m in CPT2.

(c) The groundwater level was not determined, but based on information from local residents it appeared to be around 2 m depth at the time the test program was undertaken.

The results of the undrained shear strength of the clay (S_u) are plotted in Figure 6 and are summarized in Table 4. The undrained shear strength was computed using Equation 1:

$$S_u = (q_c - p) / N_k \quad [1]$$

where q_c is the uncorrected tip resistance measured by the CPT's, p is the overburden pressure, computed assuming a unit weight of 19 kN/m^3, and N_k is the cone bearing factor, assumed equal to 15. The results of the tip resistance measured in CPT-1 and CPT-3 are shown in Figure 7.

Table 4. Undrained Shear Strength

CPT	Depth (m)	S_u (kPa)
1	0 – 0.5	Granular fill
	0.5 – 0.8	95 – 100
	0.8 – 1.6	80 – 85
	1.6 – 2.2	60 – 65
	2.2 – 3.2	95 – 100
	3.2 – 5.2	110 – 120
	5.2 – 7.2	75 – 80
	7.2 – 7.7	80 – 85
	7.7 – 7.8	Sand lens
	7.8 – 10.0	80 – 85
2	0 – 0.4	Granular fill
	0.4 – 0.7	65 – 70
	0.7 – 1.1	95 – 100
	1.1 – 2.3	80 – 85
	2.3 – 2.4	Sand lens
	2.4 – 3.3	95 – 100
	3.3 – 3.5	Sand lens
	3.5 – 4.9	120 – 125
	4.9 – 5.2	105 – 110
	5.2 – 7.6	65 – 70
	7.6 – 10.0	55 – 60

2.3 Helical Pile Installation Torque Measurement

The torque measured during helical piles installation is summarized in Table 5.

Table 5. Helical Pile Installation Torque Measurement

Depth (m)	Torque at SP-1 (kNm)	Torque at SP-2 (kNm)
1.0	84.4	91.8
2.0	87.8	92.1
3.0	62.5	75.8
4.0	72.0	84.2
5.0	77.4	92.6
6.0	83.1	97.8
7.0	86.0	108.0
8.0	94.2	110.5
8.75	92.5	110.0

The soil depth inside the shaft (plug depth) was 2.6 m and 3.1 m inside SP-1 and SP-2, respectively.

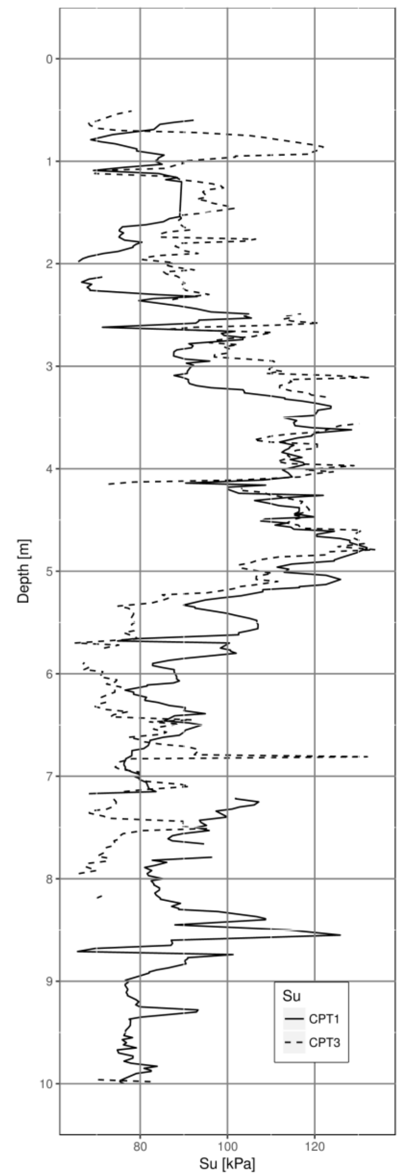

Figure 6. Undrained shear strength at CPT1 and CPT3

3 TIP RESISTANCE COMPARISON RATIOS

The tip resistance measured in CPT1 and CPT3 is depicted in Figure 7. The results of the comparisons described in Table 3 are presented in Figures 8 to 12. The results from the upper 2 m depth were left out of the comparisons due to their high variability, possibly associated to the different thickness of the granular fill and the high overconsolidation stress of the clay at shallow depth. Furthermore, in the design of helical pile foundations the shear strength of the upper 2 m soils is typically neglected due to potential soil shrinkage away from the pile.

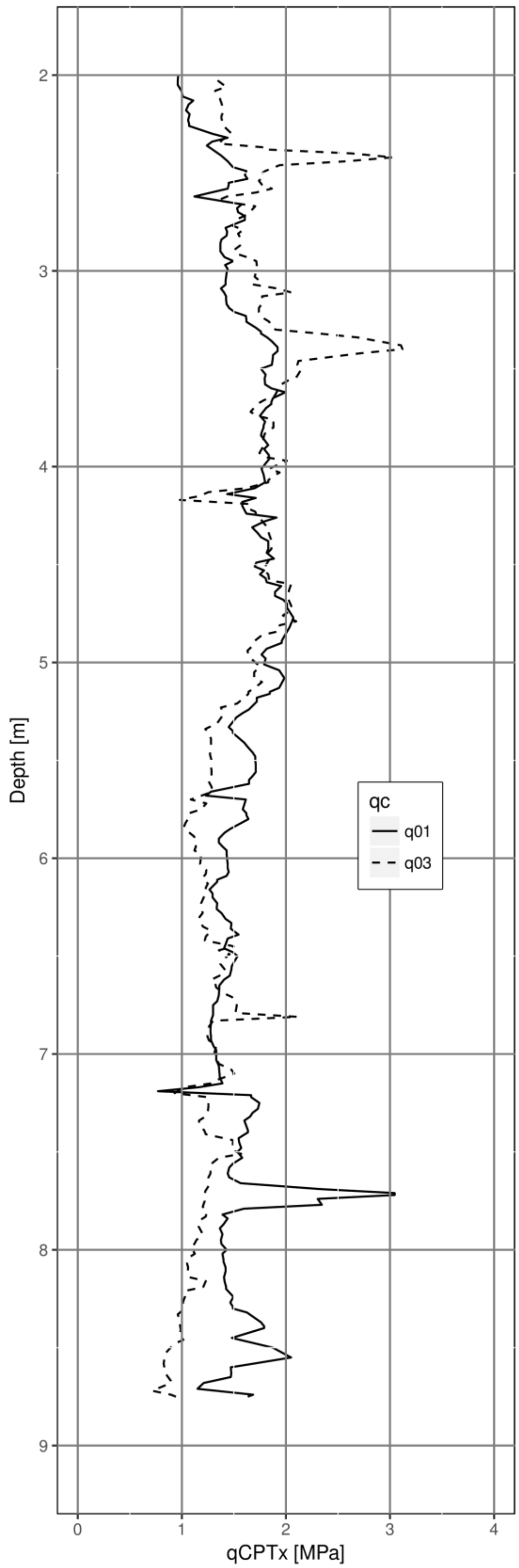

Figure 7. Tip resistance measured in CPT1 and CPT3

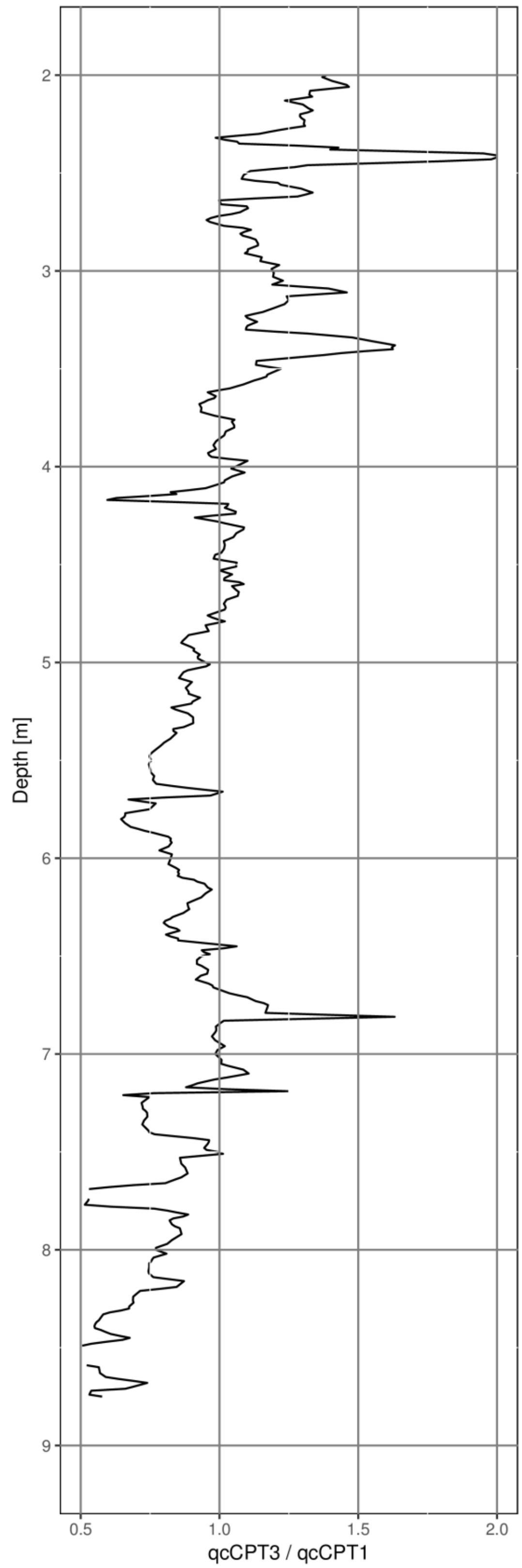

Figure 8. Native soil variability ratio q_cCPT3 / q_cCPT1

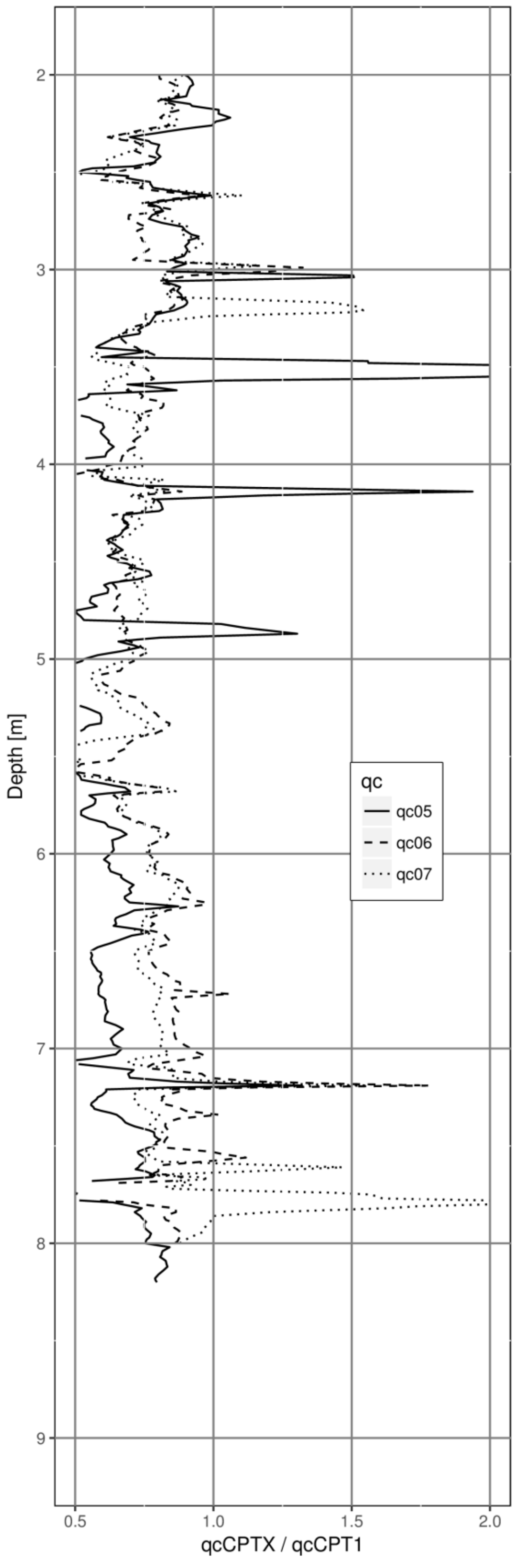

Figure 9. Soil disturbance inside helix footprint at SP1

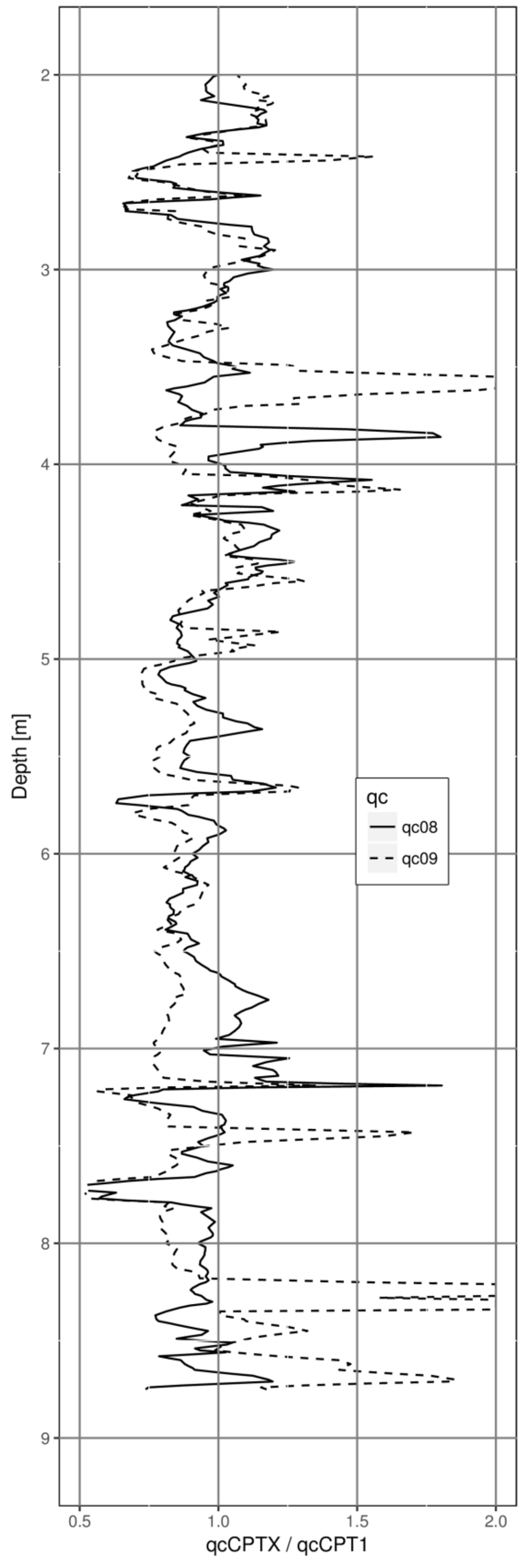

Figure 10. Soil disturbance outside helix footprint at SP1

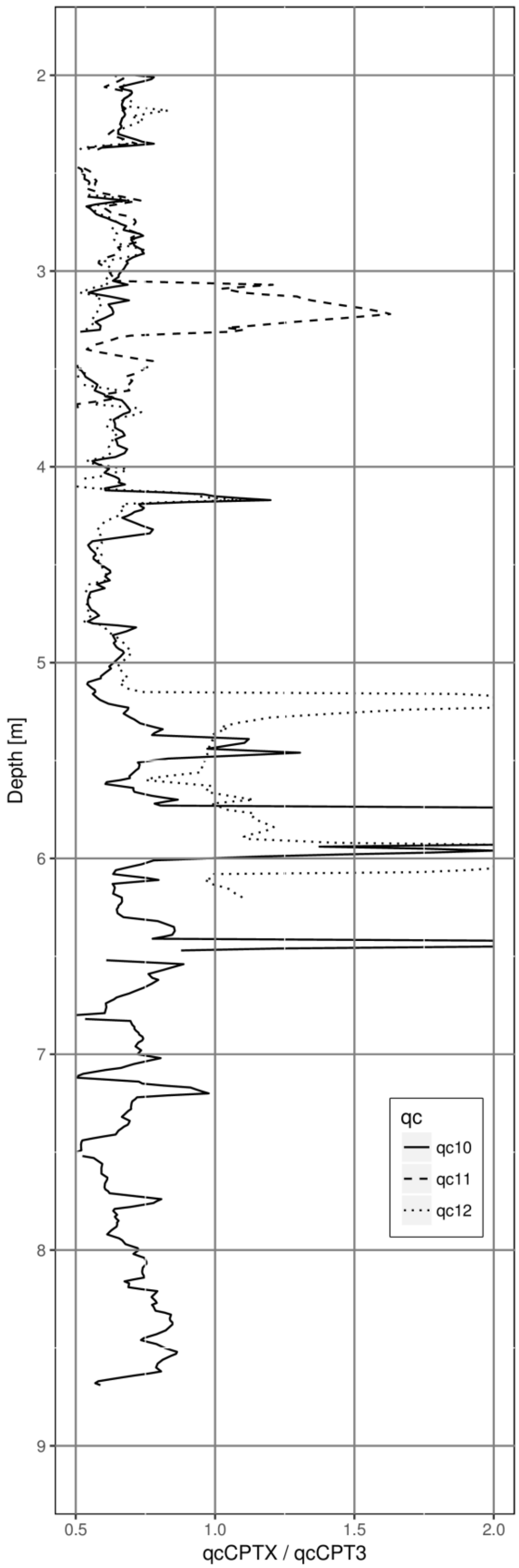

Figure 11. Soil disturbance inside helix footprint at SP2

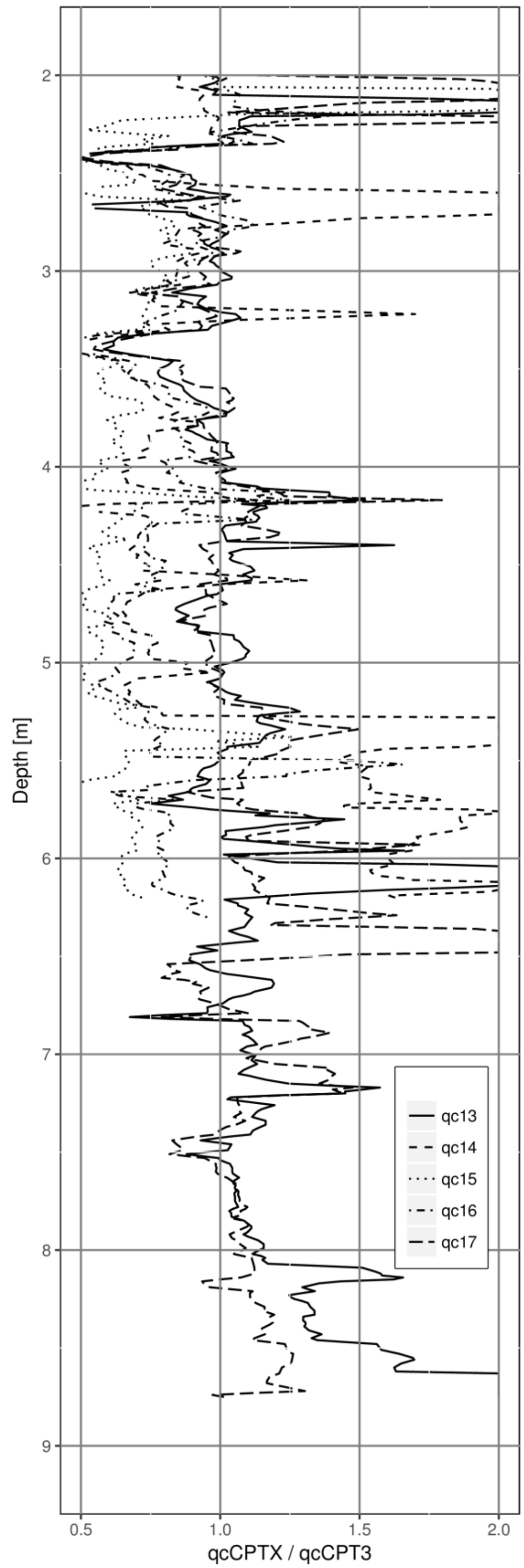

Figure 12. Soil disturbance outside helix footprint at SP2

4 DISCUSSION

The following comments may be derived from the comparisons described in Table 3 and illustrated on Figures 8 to 12:

(a) The results of q_cCPT3 / q_cCPT1 shown in Figure 8 ranged from about 0.7 to about 1.35 (neglecting the occasional very high and low pics), which is an indication the variability of the undrained shear strength of the native soil prior to helical pile installation.

(b) For practical purposes, the qc ratios smaller than 1 had the following approximate ranges:
- Inside the helix footprint of SP1: 0.6 to 0.8 (average of 0.73);
- Outside the helix footprint of SP1: 0.75 to 0.95 (average of 0.88);
- Inside the helix footprint of SP2: 0.35 to 0.75 (average of 0.62);
- Outside the helix footprint of SP2: 0.65 to 1.00 (average of 0.90);

(c) The soil disturbance produced by SP2 was greater than that produced by SP1. This statement is applicable to the disturbance inside the helix footprint as well as to the disturbance in the vicinity of the pile, outside the helix footprint. Therefore the number of helices had an impact on the amount of disturbance produced.

The results from Figures 9 to 12 may be also used as follows: Considering that the q_c measured by the CPT's was recorded at about 2 cm to 4 cm depth intervals, then each of these measurements is a statistical sample. Designating "N" as the total number of CPT measurements (hence the total number of statistical samples), then the soil disturbance measured inside and outside the helix footprint can be divided in fractions of N, as summarized in Table 6.

Table 6. Soil Disturbance Ratios

N	Soil Disturbance Ratios (inside or outside the helix footprint)			
	SP1		SP2	
	Inside	Outside	Inside	Outside
0.95 N	0.46 – 1.21	0.67 – 1.64	0.20 – 2.42	0.51 – 2.40
0.8 N	0.57 – 0.90	0.78 – 1.18	0.25 – 0.99	0.62 – 1.39
0.6 N	0.63 – 0.85	0.82 – 1.10	0.49 – 0.75	0.71 – 1.14
0.5 N	0.65 – 0.83	0.84 – 1.06	0.54 – 0.72	0.75 – 1.04

Some observations that may be drawn from Table 6 are the following:

(a) If 60% of the statistical samples (0.60N) is considered, the soil disturbance developed outside the helix footprint for both SP1 and SP2 is smaller than the variability of the native soil. This may be an indication that disturbance outside the helix footprint was negligible for 60% of the pile embedment depth (discarding the upper 2 m).

(b) For any given fraction of statistical samples, the soil disturbance produced by the double helix helical pile was greater than that produced by the single helix helical pile.

5 HELICAL PILE SOIL DISTURBANCE MITIGATION

5.1 Disturbance due to helix rotation

Soil disturbance is typically greater near the surface, and tends to decrease with depth until a constant rate of pile penetration is achieved, which typically occurs at a depth of about half a helix diameter. Further deep, the disturbance produced by the helix typically consists of a spiral shape cut through the soil. Ideally, as the helix rotates, it cuts the soil at regular intervals equal to the pitch (i.e., spacing between the upper end and the lower end of the helix). As was determined in the test program discussed hereto, multiple helices increase disturbance. However, this disturbance can be decreased when the following conditions are met: (a) The helix thickness is small and the leading edge is sharpened; (b) The helix plates are manufactured as true helices (i.e., when the angle at which the helix plate is welded to the shaft is constant along the pitch and the helix radius intersects the shaft at 90 degrees), as these helices cut spirally as they descend below the surface; (c) The spacing between the helices is designed as a multiple of the pitch size; (d) A slow, constant rate of pile penetration is maintained during installation.

5.2 Disturbance due to shaft penetration

Given that helical piles are typically installed open-ended, the disturbance produced by the shaft penetration is small at shallow depth, as the soil is cored inside the pile and forming a soil column (plug). Soil disturbance gradually increases as the pile installation proceeds, due to the increase in length of the soil column, developing more friction against the inside wall of the shaft, allowing less soil to enter through the pile tip and displacing more soil to the sides, until a full plug is developed inside the pile. Once the full plug is formed, the maximum soil disturbance is achieved, remaining constant during the rest of the pile installation. If the cohesive soils are saturated, excess pore pressures will generate as a result of the shear stresses and disturbance produced on the soil.

Mitigation alternatives against disturbance caused by shaft penetration include the use of greater shaft diameters, smaller pile lengths, slow rate of pile penetration and continued removal of the soil plug inside the shaft.

6 CONCLUSIONS

a. The variability of the undrained shear strength of the native soil prior to helical pile installation was measured by means of the ratio q_cCPT3 / q_cCPT1 and ranged from about 0.7 to about 1.35.

b. The decrease in the undrained shear strength due to soil disturbance produced by helical pile installation in a stiff to very stiff silty clay was investigated using CPT's distributed inside and outside the helix footprint, pushed prior and after helical pile installation.
c. The values of the tip resistance measured in the CPT tests were taken as basis for comparison. The ratios investigated were indicated in Table 3.
d. The q_c ratios greater than 1 are associated to the variability of the undrained shear strength of the native soil prior to helical pile installation, whereas q_c ratios smaller than 1 are associated to soil disturbance but also include soil variability.
e. Following helical pile installation, the q_c ratios smaller than 1 had the following approximate ranges:
    - Inside the helix footprint of SP1: 0.6 to 0.8 (average of 0.73);
    - Outside the helix footprint of SP1: 0.75 to 0.95 (average of 0.88);
    - Inside the helix footprint of SP2: 0.35 to 0.75 (average of 0.62);
    - Outside the helix footprint of SP2: 0.65 to 1.00 (average of 0.90);

f. The number of helices had an impact on the amount of soil disturbance produced. The soil disturbance produced by the double helix pile (SP2) was greater than the disturbance produced by the single helix pile (SP1). This conclusion is applicable to the disturbance inside the helix footprint as well as the disturbance in the vicinity of the pile, outside the helix footprint.
g. Alternatives to mitigate the soil disturbance produced by helical piles comprise geometric considerations and installation procedures.
h. Based on the results of the test program, it is considered that the decrease in the undrained shear strength due to soil disturbance produced by helical pile installation at this site is similar to the soil disturbance that would be expected from driven pile installation.
i. The potential increase in soil shear strength with time was not analyzed in the present investigation.

ACKNOWLEDGEMENT

The Authors are grateful to Hector Ibarra, P. Eng. with Almita Piling Inc. for preparing the figures, and to CPT Services Inc for undertaking the CPT testing program.

REFERENCES

Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manua*l, Third Edition, Vancouver.

Mayne, Paul. 2007. Geotechnical Site Characterization by Seismic Piezocone Tests. CPT Seminars, Edmonton, AB.

Padros, G., Liu, J., Schmidt, R. and Oki, U. 2012-1. Assessment of Helical Pile Lateral Load Performance in Cohesive Soils Considering Disturbance due to Installation. Canadian Geotechnical Conference, Winnipeg, MA.

Padros, G., Liu, J., Schmidt, R. and Oki, U. 2012-2. Predicted and Measured Compressive Capacity and Torque of Large Helix Diameter Helical Piles in Stiff to Very Stiff Cohesive Soils. Canadian Geotechnical Conference, Winnipeg, MA.