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Large Diameter Helical Pile Capacity - Torque Correlations

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ABSTRACT

Large diameter helical piles are utilized increasingly to support heavy structures. Both the magnitude of the required installation torque and the pile capacity can be directly attributed to the soil shearing resistance developed over the embedded area of the pile including the shaft and helical plates. Hence, the pile capacity can be correlated to installation torque. Such correlations are widely used in the helical pile industry as a means for quality control/quality assurance. In the current study, a total of 10 test piles, were installed while monitoring the installation torque continuously with depth. The recorded installation torque profiles were demonstrated to be accurate and repeatable. Field pile load tests were conducted and their results were analyzed to determine the interpreted ultimate capacity of the test piles. The results demonstrate that the ultimate capacity of large diameter helical piles can be interpreted from pile load tests data employing the failure criteria proposed by Elkasabgy and El Naggar (2015) and Fuller and Hoy (1970). The measured installation torque and corresponding ultimate capacity values were employed to define torque - capacity correlation (K_t) based on embedded pile area. It was demonstrated that the proposed K_t is suitable for large diameter helical piles.

Keywords

Helical pile, screw pile, installation torque, capacity-to-torque, torque factor, glacial till.

INTRODUCTION

Large diameter helical piles are used increasingly to support large compressive and tension loads. Installation torque applied to a helical pile is required to overcome the soil resistance as the pile advances into the soil. As the embedded surface area of the installed pile increases so does the soil resistance and the required installation torque. The rate of change in required installation torque depends on the change in soil strength/stiffness.

Helical piles are installed by applying torque to the pile head in conjunction with an applied vertical downward pressure “crowd”, which enables the helices to advance the pile into the soil. The applied torque is provided via a driving head (hydraulically powered rotary motor). Torque motors used for helical piles installation range in torque output, varying from 110 kN-m to 350 kN-m (Ramsey Industries 2014).

Hydraulic pressure gauges and/or electronic pressure transducers are situated in line with the hydraulic system in order to measure the forward acting pressures, reverse acting pressures and/or differential pressure (forward minus reverse). These pressure measurements are then converted into a torque via calibrated conversion factor based upon the combined hydraulic efficiency of the machine and torque motor, i.e. (Perko 2009):

$$[1] \quad T = P \times K_p$$

where T is the installation torque (in units of kN.m or ft.lb), P is the differential hydraulic pressure (kPa or psi), and K_p is the calibration factor for specific hydraulic machine and torque motor combination.

Torque - Capacity Correlation (K_t)

Installation torque is often used as the quality assurance and quality control parameter governing as-built design specifications. For small diameter helical piles, the empirical torque – capacity correlation (K_t) is traditionally used for verification of axial capacity. However, it is not as well established for large diameter helical piles, and better understanding and evaluation are required to rationally apply it for large diameter helical piles.

Theoretical models have been developed by Perko (2001, 2009) and Ghaly and Hanna (1991) to describe the relationship between torsional resistance of soils and the tensile geotechnical helical pile load carrying capacity. Similarly, Sakr (2014) developed a theoretical model for compression loading. More often however, empirical relationships have been developed to correlate torque and load carrying capacity, as in the study by Hoyt and Clemence (1989) whereby the initial proposed capacity-torque (K_t) relationship aimed at correlating applied torque to the tensile capacity of helical anchors. Zhang(1999) and Tappenden (2007) continued with

empirical studies and developed further K_t factors that can be used to estimate large diameter helical pile capacity from installation torque records, applicable to both tension and compression loads.

Hoyt and Clemence (1989) analyzed 91 tensile load tests at 24 different sites, involving small helical pile shaft sizes of 38 mm to 89 mm. They provided a simple correlation between the pile ultimate capacity, P_u , and the installation torque T averaged over the last the final three times the diameter of the largest helix or one meter of installation, i.e.

$$[2] \quad P_u = K_t \times T$$

Ghaly and Hanna (1991) conducted a laboratory investigation on small model helical piles. They concluded that several factors affect the installation torque, including: general pile configuration (i.e. single pitch helix, multi pitch helix, tapered); shaft and helix diameters; helix thickness, pitch, angle shape of leading helical edge; shape of the pile toe (i.e. flat, tapered, conical); and helical pile material surface roughness. They correlated the installation torque and the pile capacity as:

$$[3] \quad \left[\frac{P_u}{\gamma A H} \right] = 2 \left[\frac{T}{\gamma A H p} \right]^{1.1}$$

where T is the installation torque, γ is the unit weight of sand, A is the surface area of the helical plate, H is the installation depth, and p is the anchor pitch.

Zhang (1999) investigated helical piles with diameters of 219 mm to 356 mm and proposed a torque factor, K_t , to correlate the pile capacity to its installation torque. The suggested range of K_t is $6.8 - 10.7 \text{ m}^{-1}$ for piles installed in clay and $4.4 - 10.5 \text{ m}^{-1}$ for piles installed in sand. Tappenden (2007) also developed a set of K_t factors and compared them with Ghaly and Hanna's non-dimensional K_t formulation. He concluded that Ghaly and Hanna's K_t consistently overestimated the pile ultimate capacity by 132 to 858%; hence it is deemed to be inappropriate for large size helical piles capacity predictions. Ghaly and Hanna's non dimensional torque factor is heavily dependent upon: the helix area, pile embedment depth, soil unit weight, and helical pitch. Conversely, in more recent studies such as Perko (2009) it has since been found K_t

to be most significantly dependent upon the diameter of the shaft and consequentially the embedded area of the shaft.

Perko (2001) proposed K_t based on an energy model; however, it requires many parameters, some of which are not easily measurable during pile installation, such as the crowd force. Perko (2009) proposed correlation between K_t and the effective shaft diameter (d_{eff}) based on exponential regression analysis of over 300 load tests, i.e.:

$$[4] \quad K_t = \frac{\lambda_k}{d_{eff}^{0.92}}$$

where; λ_k is curve fitting factor equal to 1433 mm^{0.92}/m (22 in^{0.92}/ft).

Torque factors given by Eq. 4 were found to be in good agreement with previous research presented by Hoyt and Clemence (1989).

Interpreted ultimate load criteria

Static pile load tests are used to evaluate the pile performance under applied loads and determine the pile ultimate capacity. The applied force and resulting displacement at the pile head are recorded to produce a static load-displacement response curve. From this data, the pile performance can be evaluated, including design capacity, ultimate load capacity and global stiffness response (Kyfor, Schnore, Carlo, & Baily, 1992). Three regions can be identified within the load displacement curve: an initial linear elastic region with high stiffness (large slope), a non-linear region with gradually decreasing stiffness (decreasing slope), and a final linear region with a small residual stiffness (small slope). A suitable interpreted failure load criterion is employed to determine the pile capacity, which ideally should fall within the non-linear region. There are a few graphical methods to determine the interpreted failure load that do not impose a certain settlement limit, such as the Brinch-Hansen and the Chin Failure Criteria (Perko, 2009). However, the Brinch-Hansen method does not work if a recognizable change in slope is not observed, and the Chin method usually overestimates the ultimate capacity of the pile.

Some of the widely used interpreted failure load criteria, include: the Fuller and Hoy (1970) method, the Davisson's offset method (Davisson, 1973), the slope and tangent method (Butler &

Hoy, 1977), and the O'Neil and Reese method (O'Neill & Reese, 1999). The Davisson criterion leads to conservative values of the ultimate loads (Kulhawy & Hirany, 2009). On the other hand, the O'Neill and Reese's method, in which the ultimate capacity corresponds to displacement at the pile head equal to 5% of pile diameter, tends to overestimate the pile capacity. Elsherbiny and El Naggar (2013) evaluated the ultimate capacity of large diameter helical piles from field tests and numerical study. It was noted that for piles installed in clay the failure criteria of 5%D falls within the nearly linear rapid failure region of the curve, which could slightly over estimate the pile's capacity. Fuller and Hoy (1970) defined the failure load as the minimum load for a rate of total settlement of 0.15 mm/kN using a tangent to the load-settlement curve sloping at 0.15 mm/kN. This method is recommended for application along with the quick load test procedure. These aforementioned methods for establishing failure were initially proposed for use with compressive capacity testing and as such may therefore have mixed results when applied to the common tensile loading upon helical anchors. In addition, the load transfer mechanism for helical piles is different than that for straight shaft driven piles or drilled shafts. Therefore, it is necessary to evaluate their suitability for helical piles under both compression and uplift loading.

Some interpreted failure load criteria were proposed specifically for helical piles. For example, Livneh and El Naggar (2008) proposed an interpreted failure load criterion for small diameter helical piles based on the results of field load tests conducted on slender helical piles with a solid square shaft of 44.5 mm and a lead helix diameter ranging from 200 to 300 mm. In their method, the pile head settlement under the ultimate load is given by:

$$[5] \quad S_p = \frac{P_u \times L}{E_p \times A} + 0.08 \times D$$

Where S_p is settlement at ultimate load, E_p is elastic modulus of pile; L pile length and D is the largest helix diameter. Elsharnouby and El Naggar (2012a and b) demonstrated the applicability of this criterion for small diameter grouted helical piles. For large diameter helical piles, Elkasabgy and El Naggar (2015) revised the Livneh and El Naggar criterion, i.e.

$$[6] \quad S_p = \frac{P_u \times L}{E_p \times A} + 0.035 \times D$$

In the current study, both criteria by Fuller and Hoy (1970) and Elkasabgy and El Naggar (2015) are used to determine the ultimate capacity of the tested large diameter helical piles.

RESEARCH OBJECTIVES AND SCOPE OF WORK

The objectives of this research are twofold: to investigate the significant parameters that affect the installation torque and to investigate the relationship between installation torque and ultimate load capacity in tension and compression of large diameter helical piles.

This study evaluated the factors that affect installation torque, including: pile configuration (i.e. pile shaft size and shape, number and diameter of helices); soil conditions before and during installation; accuracy of torque measurements; and installation procedures such as applying down-pressure (crowd) on the pile and use of pre-drilling process. A custom load pin was fabricated and incorporated onto a helical pile drive head to accurately measure the installation torque. Seventeen piles were installed while continuously monitoring the installation torque with pile embedment depth. Axial pile load tests were subsequently conducted on ten helical piles. The installation torque, and load settlement measurements were collected and analyzed in order to evaluate torque - capacity correlations for large diameter helical piles. The torque – capacity correlations are investigated by soil type and loading condition (i.e. compression or tension).

EXPERIMENTAL TESTING PROGRAM

Test site

The test site was the yard of Helical Pier Systems Inc. (HPS) pile manufacturing facility, located in Lamont near Edmonton, Alberta. The soil in this area is generally glacial till, predominantly comprised of unsorted clay, silt and sand with interlayering of gravels (Shetsen, 1990).

A site investigation was conducted to characterize the soil layers and to establish their shear strength profile, which included three cone penetration test (CPT) soundings. The CPT results included cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u) at regular intervals of 0.02 m. The testing was conducted to a depth of approximately 9 m for one CPT;

164 however, due to hard/stiff soil conditions, the other two CPT tests were terminated at
165 approximately 5.7 m because the push rod apparatus was nearing its capacity.

166 The results from CPT soundings are presented in Figure 1. Values of q_c ranged from 2 to 10 MPa
167 (Fig. 1a) and f_s ranged from 30 to 600 kPa (Fig. 1b). The friction ratio R_s (f_s/q_c) is presented in
168 Figure 1c. It seems saturation loss occurred during CPT1 and CPT2, thus the results of pore
169 pressure are not presented (Robertson, 2009). CPT3 achieved and maintained saturation,
170 providing the pore pressure profile as shown in Figure 1d.

171 *Soil properties*

172 Cone tip resistance (q_c) and the friction ratio (f_s/q_c) values can be used to determine soil type by
173 using the soil behavior type (SBT) chart proposed by Robertson (1990). The corresponding
174 profiles of normalized cone tip resistance and friction ratio are presented in Figure 1e.
175 Normalized friction ratio values ranged from 3 to 9% with the exception of the first one meter.
176 Relatively high friction ratios combined with high cone tip resistance indicates the soil is highly
177 over-consolidated and consists primarily of clay, silt and sand. The normalized tip resistance and
178 normalized friction ratio indicate the top 3 m of soil fall within zones 4, 5, 6 (clay, sand, silt
179 mixtures) and the underlying soils fall within zones 11, 12 (very stiff over consolidated fine
180 grained material).

181 Lunne et al. (1997) provided estimates for the soil unit weight based on the SBT zones as shown
182 in Table 1. For top 3 m of soil (zones 4, 5, 6), the unit weight, $\gamma_s = 18 \text{ kN/m}^3$, and for the
183 underlying soils (zones 11, 12), $\gamma_s = 21 \text{ kN/m}^3$.

184 Considering the measured relatively large cone tip resistance and the existence of stiff over-
185 consolidated materials, it is recommended to use uncorrected values of cone tip resistance to
186 characterize soil shear resistance (Robertson, 1990). The undrained shear strength (S_u) can be
187 estimated using the total cone resistance, i.e. (Lunne et al.(1997)):

$$188 \quad [7] \quad S_u = \frac{q_c - \sigma_{vo}}{N_k}$$

189 where σ_{vo} is the total in-situ vertical stress, and N_k represents the cone factor.

Lunne et al. (1997) suggested that, generally, cone factors range from 15 to 20, while Meigh (1987) reported that the typical cone factor for glacial clays ranges from 14 to 22 with an average of 18. Thus, N_k was assumed to be 18 and the resulting undrained shear strength profiles calculated using Equation 7 are provided in Figure 2.

It is assumed that the remolded shear strength is equal to the lesser of the already estimated shear strength and the measured sleeve friction. Both peak and remolded shear strength profiles are shown in Figure 2. The peak and remolded shear strength values used for further analysis are presented in Table 2 as averaged within one meter intervals.

Test pile configurations and instrumentation

Five pile configurations were chosen to evaluate the influence of pile diameter and helical plates' configuration on the installation torque and the ultimate load carrying capacity. Prior to installation, all test and reaction piles were marked along their length in order to indicate embedment depth every 300 mm. These markings were utilized to provide manual recording of depth with time coinciding with installation torque measurements.

Pile configurations

Pile configurations utilized in the testing program are detailed in Table 3. Pile IDs include a letter, a number then a letter. The first letter denotes the loading mode, whereby "C" refers to compressive loading, "T" refers to tension (uplift) loading, and "RP" refers to reaction pile within the loading test setup. The number, 6, 8 or 10, refers to the pile diameter, namely 6-5/8" (168.3 mm), 8-5/8" (219.1 mm) and 10-3/4" (273 mm), respectively. These diameters are some of the most commonly used sizes of large diameter helical piles. The last letter is either an "S" or a "D", which refers to a single helix or double helices, respectively.

The helix diameter was approximately three times the pile diameter, i.e., the 168.3, 219.1, and 273 mm piles were fitted with 457.2, 609.6 and 762 mm helical plates, respectively. For piles with double helices, the inter-helix spacing was equal to three times the helix diameter. A schematic drawing of the test pile configurations is provided in Figure 3.

Installation Procedure and Layout

All test and reaction piles were spaced centre-to-centre at 2.75 m in a semi grid formation as shown in Figure 4. All 168.3 and 219.1 mm diameter test piles were arranged to have two-reaction pile loading system. The 273 mm diameter piles were arranged to have four-reaction pile loading system. The reaction piles (RP) were arranged to be utilized in testing multiple piles as shown in Figure 4.

Installation Torque Measurement

The current-state-of-practice is to install helical piles with a hydraulic powered rotational drive head. First, the helical pile is affixed to the drive head; a vertical pressure (crowd) is applied to advance the lead helical plate into the soil; and finally, torque is applied sufficient enough to engage the pitch of the helical plates within the soil thereby producing an advancing force effectively pulling the pile into the soil.

Installation torque is most commonly measured by recording the hydraulic pressure in line with the rotary hydraulic drive head. This measured pressure is either a direct forward acting pressure or, in some cases, a differential pressure (forward minus reverse). Torque measurement can be conducted at intervals throughout the installation process to produce the profile of torque with depth. The final torque and/or average torque measured over a distance equal to 3 times the largest helix diameter (i.e. last 3D) is usually recorded and used as quality control via K_t .

Pile Load Tests

The experimental investigation comprised six compression and four tension load tests. Each pile configuration was tested in both compression and tension (uplift), with the exception of the 273 mm diameter pile configurations. The load tests were conducted following a quick maintained load test procedure, in accordance with ASTM (2007a) standard D1143-81 for compression and ASTM (2007b) standard D3689-07 for uplift.

The compression or tension loads were applied to the pile head while simultaneously monitoring the pile movement. In addition, seven of the test piles were instrumented with strain gauges to enable observations of load transfer mechanisms.

Load test setup

The test site layout was configured to minimize the required number of reaction piles. For all 168.6 and 219.1 mm diameter piles, a system of two reaction piles and a single reaction beam was used. In the case of the 273.0 mm test piles, a four-reaction pile arrangement was employed.

The compressive and tension loads were applied by using a hydraulic jack with maximum capacity of 2,530 kN. A pneumatic pump was utilized to control the load increment. The load was measured employing two methods: using a calibrated load cell with a maximum capacity of 4,000 kN situated between the reaction beam and the hydraulic jack; and using a pressure transducer with a maximum capacity of 2,530 kN, which was mounted in line with the hydraulic jack. Figure 5a and 5b shows the arrangements for the hydraulic jack and load cell arrangements for both the compression and uplift loading, while Figure 5c and 5d demonstrates the 2-pile and 4-pile reaction frame arrangements.

Both vertical and horizontal pile head movements were monitored during loading. Two linear variable displacement transducers (LVDT's) were utilized to measure the vertical settlement of the piles. The LVDTs were mounted on the pile head, diametrically opposite each other, and were bearing against stationary independent reference steel beams. Three manual gauges were similarly mounted to the pile head to provide redundancy. In addition, two manual gauges were arranged orthogonally to one another in the horizontal plane to measure the lateral movement of the pile head. The LVDTs and manual gauges provided accurate measurement to the nearest 0.0254 mm. All load test data, with the exception of pile strain gauge readings, were recorded at one second intervals via the data acquisition module Graphtec midi logger GL200A.

Procedure

The load was applied in increments of 50 kN (5 % of the anticipated failure load). For each load increment, the load was maintained at an almost constant level for 5 minutes, as set out in ASTM Standard D1143 (ASTM 2007a). Once the rate of pile head movement increased and failure was approached, or the testing apparatus was at its limit, the final load increment was maintained for a period of 10 minutes. Following the maximum applied load, the load was removed in approximately 200 kN increments while maintaining each increment for 5 minutes. The final unloading of the pile head was monitored for an additional 10 minutes.

RESULTS

The results of the testing program included measurements of installation torque profiles during installation of all test and reaction piles and pile load-displacement curves for all piles tested under compression and uplift loading. These results were used to establish representative installation torque and ultimate pile capacity values. The installation torque and pile capacity values were then used to establish useful torque – capacity correlations.

Installation Torque Profiles

Livneh and El Naggar (2008) stated that the installation torque is a measure of the energy required to overcome the shear strength of the soil and hence is directly related to the soil shear strength and the pile capacity. The installation torque depends on the embedded surface area of the pile (Sakr, 2013; Perko, 2001; Rogers, 2012). Thus, it is expected that installation torque increases as the depth increases, especially for piles installed in soil whose strength increases with depth. The rate of increase in installation torque would correspond to the pile surface area embedded in the soil and the change in soil strength.

The torque-depth profiles constructed for all reaction piles (273.0 pile diameter and single 762 mm-diameter helix) are presented in Figure 6, while Figure 7 compares the torque profiles for all 168.6, 219.1 and 273.0 mm diameter tests piles (both single and double helices). Both figures confirm that piles with the same geometry displayed similar torque profiles and consistent torque values. As expected, double helix piles generated larger final torque compared to single helix piles because the second helix increases soil resistance and hence installation torque.

In order to investigate the effect of the pile diameter on the installation torque, the torque profiles for piles with different diameters are presented in Figure 8a and Figure 8b for single and double helix piles, respectively. It is clearly evident that the increase in pile shaft diameter has a more significant effect on the required installation torque than that of the second helix. For example, the addition of second helix resulted in an increase of the final torque by 5 - 10 kN.m, whereas the increase in the pile diameter from 168.6 to 219.1 mm (and from 219.1 to 273.0 mm) increased the installation torque by 20 - 30 kN.m. These findings confirm that the installation torque is proportional to the total pile embedded surface area and the soil shear strength.

Average and final installation torque

Three measures of installation torque were evaluated as follows: overall average torque weighted over the entire embedment depth; average torque weighted over the last installed depth equal to 3D; and the final maximum installation torque measured at final embedment. The final torque and the torque averaged over the last 3D exhibited similar trends, i.e., they increased as the embedded area of the pile increased. The difference between the two values increased as the diameter of the pile/helix increased. The final installation torque measurement has a tendency to include short spikes not indicative of the major soil strata relied upon for bearing at the lead helix. Therefore, the average torque weighted over the last installed depth equal to 3D are utilized to establish the K_t factors within this study.

Pile Load Test Results

The results from the axial pile load testing program are presented herein in terms of load-settlement curves and load transfer diagrams. The load-settlement curves are interpreted to determine the ultimate pile capacity values. The determined pile capacity values are then used along with average installation torque measured over a depth of the last three times the largest helical diameter, to establish a torque - capacity correlation (K_t) factor that can be used for the prediction of the ultimate pile capacity.

Interpreted ultimate capacity Criteria

All static axial load tests were conducted according to the quick maintained load test procedure and, as such, appropriate interpretation methodologies were employed. Four methods were utilized for interpreting the tests results to determine the interpreted ultimate pile capacity, including: the Davisson's offset method (Davisson 1973); the method proposed by Elkasabgy and El Naggar (2015), which defines the ultimate load as the load corresponding to net settlement equal to 3.5% of the largest helix diameter (not including elastic settlement of the pile itself); the method proposed by Fuller and Hoy (1970); and the plunging failure (if occurred) taken as the maximum load occurring.

327 *Compressive Load Tests*

328 Figure 9 presents the load-settlement curve for Pile C6S. It exhibits typical plunging failure with
329 a failure load of 644 kN, which occurred at settlement of 25 mm. The interpreted failure criteria
330 produced ultimate load varying from 430 to 630 kN, with Davisson' criterion providing the
331 lowest value while Elkasabgy and El Naggar was the closest to the failure load. These loads
332 corresponded to settlements varying from 7.2 mm to 19.3 mm. Similarly, the load-settlement
333 curve for Pile C6D clearly demonstrates that the pile experienced plunging failure, which
334 occurred at 1144 kN with a settlement of 27.4 mm. The interpreted failure criteria predicted
335 ultimate load capacity varying from 896 kN to 1090 kN, with Davisson' criterion providing the
336 lowest value while Elkasabgy and El Naggar was the closest to the failure load. The
337 corresponding settlement values varied between 12.6 and 27.0 mm. It should also be noted that
338 the capacity of the double helix pile C6D is much higher than the capacity of single helix pile
339 C6S.

340 Piles C8S and C8D exhibited the same trends as can be noted from the results presented in
341 Figure 10. They experienced plunging failure at 1064 and 1516 kN, respectively, which occurred
342 at settlements of 39.4 and 34.0 mm. Similarly, the interpreted failure criteria provided lower
343 loads corresponding to lower settlement; the interpreted ultimate load using the Elkasabgy and El
344 Naggar criterion was the closest to the actual failure load and the Davisson's criterion provided
345 the lowest capacity. Figure 11 presents the results for piles C10S and C10D. Both piles exhibited
346 plunging failure, with failure loads 1445 kN and 1822 kN. It is also noted that both piles
347 experienced significant creep settlement. As can be noted from the figures, the onset of failure
348 occurred at 58.0 mm and 85.1 mm, respectively, but the creep settlement reached 77.3 and more
349 than 100mm. The load test was finally stopped due to the excessive displacement that exceeded
350 the capacity of the loading system.

351 *Uplift Load Tests*

352 Figure 12 shows the load-displacement of Pile T6S, which exhibited clear failure with a quickly
353 terminating non-linear transition region. Failure occurred at 870 kN, while the interpreted
354 ultimate capacities ranged between 720 and 837 kN, which occurred at displacements of 10.7 -
355 31.7 mm. Similarly, Pile T6D displayed recognizable failure at load of 982 kN. The interpreted

ultimate capacities ranged between 870 – 982 kN, and the corresponding displacements ranged between 15.4 and 22.5 mm. Figure 13 presents the results for piles T8S and T8D. They show the same trends with failure loads of 1100 and 1380 kN, while the interpreted ultimate capacity varied from 970 to 1020 for T8S and from 1053 to 1276 kN. It is noted from the tension load tests that failure occurred at relatively smaller displacements. Consequently, the interpreted failure loads were much closer to the actual failure loads.

Comparison of interpreted failure load criteria

The ultimate capacity of all tested piles determined from the different interpreted failure criteria are summarized in Figure 14 and in Table 4. As can be noted from Table 4 and Figure 14, the ultimate capacity determined by the method proposed by Elkasabgy and El Naggar (2015) provided the closest capacity values to the plunging failure load, while the Davisson method provided the most conservative. In addition, the Fuller and Hoy (1970) method provided reasonable estimates of the pile ultimate load capacity.

It is also noted from Table 4 that the pile settlement corresponding to the plunging failure load ranged from 15 to 85 mm. In many cases, the capacity of the loading system and/or the range of settlement measurement devices do not allow the loading to proceed up to such large settlement. Therefore, the plunging failure may not be attained in many practical test setups. On the other hand, the settlement for the Elkasabgy and El Naggar and Fuller and Hoy criteria ranged from 19.0 to 3.04 and 11.0 to 46.0 mm, respectively. Thus, the interpreted failure criteria proposed by Elkasabgy and El Naggar and Fuller and Hoy appear to be more appropriate for the determination of the capacity of large diameter helical piles. It is noted that the Fuller and Hoy criterion produces a more conservative estimate of the ultimate pile capacity, but it is based on the actual pile performance during the pile load test and not just the pile geometrical properties.

TORQUE-CAPACITY CORRELATIONS

Undoubtedly, plunging failure is universally accepted method to determine the pile ultimate capacity. However, as discussed above, plunging failure may not be attained because of test setup limitations and/or significant creep displacement of the test pile. In this case, it is necessary to select a suitable interpreted failure criterion for determining the pile ultimate capacity values to be used to establish K_t factors. An interpreted failure criterion that utilizes a suitable

settlement tolerance (e.g. Elkasabgy and El Naggar) may be employed. However, settlement criteria may not always be valid for varying pile geometry. Alternatively, criteria based on the actual pile performance during the load test (e.g. Fuller and Hoy) are applicable to different pile geometry, and may be more appropriate for varying soil conditions.

K_t Factors

The ultimate capacity of the tested piles determined from the plunging failure and the interpreted failure criteria were used to evaluate the K_t factors. The calculated values are presented in Table 5. It can be noted from Table 5 that the K_t factors varied from 6.7 to 16.4 for Davisson, 13.0 to 21.0 for Elkasabgy and El Naggar, 13.8 to 21.0 for Plunging, and 10.2 to 19.4 for Fuller and Hoy criteria. It is also noted that the K_t factors for double helix piles were slightly higher than those for the single helix piles for the same pile diameter. Finally, there is no significant difference between the K_t factors for piles in tension versus compression, perhaps because all tension piles were installed under deep embedment condition.

Given the closeness of the pile ultimate capacity and K_t factors determined using the Fuller and Hoy with those obtained from the plunging failure, it is suggested to use the Fuller and Hoy criterion to establish the pile capacity values from the load test data. These ultimate capacity values are then used to establish the K_t factors.

Capacity-Torque Correlation Curve Fitting

The K_t data obtained in this study is based on all tension and compressive interpreted failure loads determined using the Fuller and Hoy criterion and the torque measured over the last 3D. In addition, these results are augmented by the pile ultimate capacity and installation torque values reported by Tappenen (2007) for large diameter helical piles installed in similar soil profile (i.e. sand/glacial till). This helped increase the data set used to establish a suitable K_t relation for helical piles installed in glacial tills.

Figure 15 presents the K_t factors established by directly correlating the pile capacity to its installation torque. The obtained correlation for all experiments regardless of testing mode and soil type provided K_t equal to 10.3 with a coefficient of determination of 0.84. This K_t factor appears to give slightly conservative predictions of helical piles installed in glacial till but

perhaps appropriate for sandy soils. Correlation for compressive and tension tested provided slightly different value of K_t equal to 11.40 and 9.3 with coefficients of determination of 0.88 and 0.84, respectfully.

Figure 16 presents the direct torque - capacity factors plotted vs the pile diameter, and the curve fitting of the data used the pile diameter as a fitting parameter. The lines of best fit for glacial till, sand, and all data compiled are used to establish a K_t relationship incorporating the pile diameter as a curve fitting parameter. For the purpose of comparison, the K_t relationship provided by Perko (2009) is plotted in Figure 16. It is observed from the figure that there is close agreement between the best fit for both compression and tension data and Perko (2009), especially for larger diameter piles. This agreement suggests that the Perko relationship can be used to predict the capacity of helical piles in different types of soils and loading conditions. It should also be noted that the difference in load capacity for the same torque value shown in Figures 15 and 16 is primarily due to loading condition and pile geometry, which lead to different load capacity. The lower capacity values are for tension loading due to reduced strength of disturbed glacial till above the helix, which does not have the same effect on compression load capacity

Proposed Torque-Capacity Correlation Using Pile Embedded Area

The main limitation of the above formulations is that they do not account fully for the helical pile configuration (i.e. pile diameter, helix diameter, number of helices). Utilizing the total embedded pile area as a curve fitting parameter would enable accounting for the pile diameter, number and diameter of helices, and depth of installation in curve fitting. Therefore, the use of pile embedded area as a curve fitting parameter in order to establish K_t relationship is explored herein. This offers the option to subtract the surface area of the pile embedded within expected zones of very soft layers, which can even enhance the accuracy of K_t relationship.

Figure 17 shows K_t factors plotted against the pile embedded area. The data is curve fitted considering the pile embedded area as a fitting parameter. Four best fit lines are attempted, one to fit all data, one to fit tension data, one to fit compression data and one to fit only glacial till data. As can be observed from Fig. 17, curve fitting all data underestimates K_t for the glacial till data points. On the other hand, as expected, curve fitting only the glacial data represents the

glacial till data points reasonably well. Furthermore, correlations for compression only were not found to be significantly better represented, likely a result of ultimate capacities relying upon underlying soils not affected by installation torque. The case of tension loading data, curve fitting provided an interesting result: whereas the coefficient of determination is the highest of any other data set, which signifies that as embedded area increases the torque-capacity factor increases. This is contrary to the relationship between torque-capacity factors and pile diameter. This observation may account for the limits of shallow failure criteria. Accordingly, it is suggested to use the equation that represents the K_t factor for helical piles installed in glacial till, i.e.

$$[11] \quad K_t = 29.2 A_e^{-0.5} \quad (R^2 = 0.26)$$

where K_t (m^{-1}) is the torque to capacity factor, and A_e (m^2) is the total embedded area. It is noted that the correlation given by Equation 11 has low R^2 value, which indicates poor correlation. This is attributed to two factors. First, the equation is developed using a limited data set. Secondly, the correlation is particularly off for the smaller diameter piles (i.e. C6S and C6D), which affect the value of R^2 significantly and will be discussed further later. It is recommended to expand the data set through testing additional pile configurations installed in different soil types.

To further understand the effect of pile embedded area on its capacity, the variations of installation torque and pile capacity are presented in Figures 18 a and b, respectively. As can be noted from Figure 18, both installation torque and pile capacity are highly correlated with the pile embedded area as indicated by the high R^2 values. It is also noted that the correlations are relatively complex polynomials with very different coefficients for both correlations. This partially explains why the correlation is poor when considering a simpler form for the torque factor as a function of the pile embedded area. Adding more data points will eventually aid in developing an appropriate torque factor as a function of the pile embedded area.

Torque – Capacity Predictions

The pile capacity was predicted employing six different correlations that use K_t factors proposed by: Tapenden (2007), i.e. $K_t = 9.2 \text{ m}^{-1}$; Hoyt and Clemence (1989), i.e. $K_t = 9.8 \text{ m}^{-1}$; Perko (2009), i.e. $K_t = 1433 \times D^{0.92} \text{ m}^{-1}$; and present study, i.e. $K_t = 10.3 \text{ m}^{-1}$, $29.2 \times A_e^{-0.5} \text{ m}^{-1}$, and

221.05 $\times D^{-0.536} \text{ m}^{-1}$. As shown in Fig. 19, most capacity predictions were below the measured capacity (i.e. conservative). The average predicted pile capacity, Q_p , is 0.71-0.98 P_u , and the coefficient of variation ranged from 0.01 to 0.03. The results demonstrate the suitability of K_t method for design confirmation of large diameter helical piles as it gives consistently conservative and reasonably accurate prediction in comparison with theoretical calculations and CPT correlations.

In addition, inspecting the results in Figure 19 suggests that embedded area can be an accurate means of evaluating torque-capacity correlation as it provided very close predictions of axial capacity for all piles except for the small diameter piles in compression (i.e. C6S and C6D). It should be noted that the capacity of the “smaller” diameter helical pile in compression is primarily due to the helical plates bearing on undisturbed soil, while the contribution of slender shaft to the capacity is minimized due to the soil disturbance along the shaft. This effect is also manifested in the K_t factor for compression being greater than for tension cases as shown in Figure 15 ($K_t = 11.4 \text{ m}^{-1}$ for compression and $K_t = 9.2 \text{ m}^{-1}$ for tension). Therefore, it is possible that the torque factor as a function of the embedded area is not suitable for the small diameter helical piles in compression due to the relatively small contribution of the slender shaft to the total capacity but large embedded shaft area compared to the helical plates area. This is not the case for piles subjected to tension loading, as both contributions of the helical plates and the pile shaft are affected by the soil disturbance. However, additional load test data should be collected and separate correlations should be established for “smaller diameter” pile and “large diameter” pile and also for tension and compression to account for effect of reduced strength of glacial till above the top helix. In the meantime, it is suggested to use Eq. 11 for conservative evaluation of K_t factor for large diameter (i.e. $d \geq 200 \text{ mm}$) helical piles installed in glacial till.

A closer look at the results in Figure 19 reveals predictions of capacity for C6S and C6D using different K_t factors are generally poor (Q_p as low as 0.47 and as high as 1.4). On the other hand, the predictions of capacity of large diameter piles are excellent, especially using correlations that account for the pile geometry ($Q_p = 0.82\text{-}0.98 P_u$). It is also noted that the predictions of capacity for piles subjected to tension are excellent, even for the small diameter piles. For example, Equation 11 predicted $Q_p = 0.90\text{-}0.99 P_u$ for the tension piles, as shown in Figure 19. Furthermore, the site specific K_t factor that accounts for the pile embedded area proposed in the

current study provided enhanced accuracy for “larger” diameter piles and piles subjected to tension.

SUMMARY AND CONCLUSIONS

In this study, a total of 10 helical piles with varying configurations were installed at a site consisting of primarily over-consolidated glacial till. All pile installations were monitored and the variations of installation torque, vertical crowd, and installation rate with depth were recorded. Six static compression load tests and four static tension load tests were conducted to establish the piles ultimate capacity. The following conclusions may be drawn.

1. The installation torque measurements obtained by using the fabricated torque pin were demonstrated to be accurate and repeatable.
2. Based on the measured pile capacities and installation torque obtained in the current study, two different K_t factors can be suggested for large diameter helical piles installed in sand and/or glacial till under conditions of compression and/or tension. Direct correlation for sand and glacial till: $K_t = 10.3$. For helical piles subjected to tension loading or helical piles with diameter $\geq 200\text{mm}$ installed in glacial till and subjected to compression, $K_t = 29.2A_e^{-0.5}$ ($R^2 = 0.26$).
3. The ultimate capacity of large diameter helical piles can be determined from the pile load test data employing the interpreted ultimate failure loads using the Elkasabgy and El Naggar (2015) and Fuller and Hoy (1970). Both criteria provided reasonable predictions for both compressive and tensile static pile capacity.

Experimental investigations similar to the current study should be attempted in different soil types and varying soil strength profiles. The results from such studies can be used to confirm the findings from this study (e.g. the method to calculate installation torque), and to calibrate the proposed K_t factor as a function of embedded pile area. Also, it would be interesting to conduct installations while intentionally changing the crowd at the same depth and record the corresponding torque to better evaluate the effect of the crowd on the generated torque. Additionally, further experimental installation should be attempted whereby the applied crowd is held constant and/or minimized.

528

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537 **REFERENCES**

- 538 Abdelghany, Y. 2008. *Monotonic and Cyclic Behaviour of Helical Screw Piles Under Axial and*
539 *Lateral Loading. Ph.D. thesis: The University of Western Ontario. London, ON.*
- 540 *ASTM Designation: D1143* 1981. *Standard test method for piles under static axial compressive*
541 *load.* (ASTM), American Society for Testing and Materials.
- 542 Bradka, D. T. 1997. *Vertical Capacity of Helical Screw Anchor Piles.* M.E.Sc.: University of
543 Alberta. Edmonton, AB.
- 544 Bradka, T., and Kasprick, B. 2013, May 24. VP of Engineering, Operations (HPS). (J. Harnish,
545 Interviewer)
- 546 Bustamante, M., and Gianselli, L. 1982. Pile Bearing Capacity by Means of Static Penetrometer
547 CPT. *Proceeding of the @nd European Symposium on Penetration Testing.* 2, pp. 493-
548 500. Amsterdam: Balkema Publisher.
- 549 Butler, H., and Hoy, H. 1977. *Users Manual for the Texas Quick-Load Method for Foundation*
550 *Load Testing.* Washington, D.C.: US Department of Transportation, Federal highway
551 Administration.

- 552 Canadian Geotechnical Society. 2006. *Canadian Foundation Engineering Manual 4ed.*
553 Richmond, B.C: BiTech Publishers Ltd.
- 554 Davisson, M. T. 1973. High Capacity Piles. *Proceeding of the Lecture Series, Innovaions in*
555 *Foundations Construcion* (p. 52p). Illinois: ASCE.
- 556 Elkasabgy, M. and El Naggar, M.H. 2015. Axial compressive response of large-capacity helical
557 and driven steel piles in cohesive soil. *Canadian Geotechnical Journal*, Vol. 52, No. 2, pp. 224-
558 243.
- 559 El Sharnouby, M.M. and El Naggar, M.H. 2012a. Field investigation of axial monotonic and
560 cyclic performance of reinforced helical pulldown micropiles. *Canadian Geotechnical*
561 *Journal*, Vol 49, No. 5, pp. 560-573.
- 562 El Sharnouby, M.M. and El Naggar, M.H. 2012b. Axial monotonic and cyclic performance of
563 fibre-reinforced polymer (FRP) – steel fibre–reinforced helical pulldown micropiles
564 (FRP-RHPM). *Canadian Geotechnical Journal*, Vol. 49, No. 12, pp. 1378-1392.
- 565 Elsherbiny, Z. and El Naggar, M.H. 2013. Axial compressive capacity of helical piles from field
566 tests and numerical study. *Canadian Geotechnical Journal*, Vol. 50 (12), 1191-1203.
- 567 Fuller, F., and Hoy, H. 1970. *Pile Load Tests Including Quick-load Test Method Conventional*
568 *Methods and Interpretations*.
- 569 Ghaly, A., and Clemence, S. 1998. Pullout Performace of Inclined Helical Screw Anchors in
570 Sand. *Journal of geotechnical and Geoenvironmental Engineering*, 617-627.
- 571 Ghaly, A., and Hanna, A. 1991. Experimetnal and theoretical studies on installation torque of
572 screw anchors. *Canadian Geotechnical Journal*, 353-364.
- 573 Hoyt, R. M., and Clemence, S. P. 1989. Uplift Capacity of Helical Anchors in Soil. *12th*
574 *Internation Conference on Soil Mechanics and Foundation Engineering*, (pp. 1-12). Rio
575 de Janiero, Brazil.
- 576 Kulhawy, F. H. 2004. On the axial behaviour of drilled foundations. *American Society for Civil*
577 *Engineering: Geo Support*.

- 578 Kulhawy, F. H., and Hirany, A. 2009. Interpreted Failure Load for Drilled Shafts via Davisson
579 and L1- L2. *2009 International Foundation Congress and Equipment Expo:
580 Contemporary Topics in Deep Foundations*, (pp. 127-135).
- 581 Kyfor, Z., Schnore, A., Carlo, T., and Baily, P. 1992. *Static Testing of Deep Foundations*.
582 Washington: U.S. Department of Transportation: Federal Highway Administration.
- 583 Livneh, B., and El Naggar, M. H. 2008. Axial testing and numerical modeling of square shaft
584 helical piles under compressive and tensile loading. *Canadian Geotechnical Journal*,
585 1142-1156.
- 586 Lunne, T., Robertson, P. K., and Powell, J. 1997. *Cone Penetration Testing : In Geotechnical
587 Practice*. London UK: Blackie Academic and Professional.
- 588 Meigh, A. C. 1987. *Cone Penetration Testing: methods and Interpretation*. Letchworth U.K.:
589 Adlard & Sons Ltd.
- 590 Meyerhof, G. 1951. The Ultimate Bearing Capacity of Foundations . *Geotechnique*.
- 591 Meyerhof, G. G. 1976. Bearing Capacity and Settlement of Foundations. *Journal of Geotechnical
592 and Geoenvironmental Engineering*, 195-228.
- 593 Mitsch, M., and Clemence, S. 1985. The Uplift Capacity of Helix Anchors in Sand. . *Uplift
594 Behaviour of Anchor Foundations in Soil*, (pp. pg 26-47). Michigan.
- 595 Mooney, J. S., Clemence, S. P., and Adamczak . 1985. Uplift Capacity of helix Anchors in Clay
596 and Silt. *American Society of Civil Engineering*, 48-72.
- 597 Narasimha Rao, S., and Prasad, Y. 1993. Estimation of uplift capacity of helical anchors in
598 clays. *Journal of Geotechnical Engineering*, 352-357.
- 599 O'Neill, M., and Reese, L. 1999. *Drilled Shaft: Construction, procedures and design methods*.
600 FHWA-IF-99-025.
- 601 Perko, H. A. 2001. Energy method for Predicting Installation Torque of Helical Foundation and
602 Anchors. *GeoDenver: Geotechnical Special Publications*. Reston: ASCE Press.

- 603 Perko, H. A. 2009. *Helical Piles: A Practical Guide to Design and Installation*. New Jersey: John
604 Wiley & Sons.
- 605 Prakash, S., and Sharma, H. 1990. *Pile Foundation In Engineering Practice*. Toronto: John
606 Wiley & Sons.
- 607 Radhakrishna, H. 1976. *Helix Anchor Tests in Sand*. Ontario Hydro Research Division.
- 608 Ramsey Industries. 2014. *Eskridge - Gear Drive, Brakes, Diggers and Anchor Drives*. Retrieved
609 January 25, 2014, from Diggers: Product Specifications:
610 <http://www.eskridgeinc.com/diggers/diggerprodspecs.html>
- 611 Roberston, P., Campanella, R., Gillespie, D., and Greig, J. 1986. Use of piezometer cone data.
612 *Proceedings of the ASTM Specialty Conference In Situ '86: Use of in situ tests in*
613 *geotechnical engineering* (pp. 1263-1280). ASCE.
- 614 Robertson, P. K. 1990. Soil classification using the cone penetration test. *Canadian Geotechnical*
615 *Journal*, 151-158.
- 616 Robertson, P. K. 2009. Interpretation of cone penetration tests - a unified approach. *Canadian*
617 *Geotechnical Journal*, 1337-1355.
- 618 Rogers, W. 2012. *Theoretical Installation Torque for Helical Pipe Piles - Part 1: Single Helix -*
619 *Homogeneous Soils*. Quality Anchor Products Inc.
- 620 Sakr, M. 2013. Relationship between Installation Torque and Axial Capacities of Helical Piles in
621 Cohesive Soils. *The Journal of the Deep Foundation Institute*.
- 622 Sakr, M. 2014. Relationship between installation torque and axial capacities of helical piles in
623 cohesionless soils. *Canadian Geotechnical Journal*, 52, 747-759.
- 624 Shetsen, I. 1990. Quaternary geology of Central Alberta. *Alberta Geological Survey*. Alberta
625 Research Council.
- 626 Skempton, A. W. 1951. The bearing Capacity of Clays. *Proceedings of the Building Research*
627 *Congress*, (pp. 180-189).

- 628 Tappenden, K. M. 2007. *Predicting the Axial Capacity of Screw Piles Installaed in Western*
629 *Canadian Soils*. Edmonton: The University of Alberta.
- 630 Terzaghi, K. 1943. *Theoretical Soil Mechanics*. New York: John Wiley & Sons.
- 631 Transportation Research Board. 2007. *National Cooperative Highway Research Program: Cone*
632 *Pentration Testing*. Washington.
- 633 Zhang, D. 1999. *Predicting Capacity of Helical Screw Piles in Alberta Soils, M.S. Thesis. Dept*
634 *of Civil and Environmental Engineering*. University of Alberta.
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- 636

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637 List of Symbols

- 638 A : surface area of the helical plate
- 639 A_e : total embedded area
- 640 D : diameter of the pile toe
- 641 d_{eff} : effective shaft diameter
- 642 E_p : young's modulus of the pile material
- 643 F_t : torque factor
- 644 f_s : sleeve friction
- 645 H : installation depth
- 646 K_p : pressure to torque calibration factor
- 647 L : pile length
- 648 N_k : cone factor
- 649 N_{qu} : uplift capacity factor
- 650 p : helical plate pitch.
- 651 P : differential hydraulic pressure
- 652 P_u : ultimate load carrying capacity
- 653 Q : ultimate load carrying capacity
- 654 q_c : cone tip resistance
- 655 S_p : total pile head settlement
- 656 T : installation torque
- 657 γ : unit weight
- 658 λ_k : is curve fitting factor equal to $1433 \text{ mm}^{0.92}/\text{m}$ ($22 \text{ in}^{0.92}/\text{ft}$)
- 659 σ_{vo} : total in-situ vertical stress

Figures

Figure 1: CPT –Data: a) Cone tip resistance (q_c); b) Sleeve friction (f_s); c) friction ratio; d) Pore pressure (u_2 & u_o); e) Normalized tip resistance; f) Normalized friction ratio; g) SBT - CPT #1; h) SBT - CPT #2; and i) SBT - CPT #3

Figure 2: a) Estimated undrained shear strength, S_u ; b) Design shear strength peak (S_{up}) and remolded (S_{ur})

Figure 3: Test pile drawing

Figure 4: Locations of test and reaction piles as well as CPT soundings

Figure 5: Load test setup: a) Compression test setup; b) Tension test setup; c) Two reaction pile setup; and d) Four reaction pile setup

Figure 6: Torque profile for reaction piles (RP1-8)

Figure 7: Torque depth profile for test piles: a) C6S, C6D, T6S T6D; b) C8S, C8D, T8S, T8D; and c) C10S, C10D.

Figure 8: Torque depth profile for: a) single helix piles; b) Double helix piles

Figure 9: Load settlement curves for test piles: C6S and C6D

Figure 10: Load settlement curves for test piles: C8S and C8D

Figure 11: Load settlement curves for test piles: C10S and C10D

Figure 12: Load settlement curves for test piles: T6S and T6D

Figure 13: Load settlement curves for test piles: T8S and T8D

Figure 14: Comparison of ultimate failure load from different criteria

Figure 15: Direct torque – capacity correlation

682 Figure 16: Torque - capacity vs shaft diameter

683 Figure 17: Torque - capacity vs embedded area

684 Figure 18 variation of: a) installation torque with embedded area; and b) capacity and embedded
685 area

686 Figure 19: Torque - capacity predictions

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Tables

Table 1: Unit Weight Estimate based on SBT

Zone	Approximate Unit Weight (kN/m³)
1	17.5
2	12.5
3	17.5
4	18
5	18
6	18
7	18.5
8	19
9	19.5
10	20
11	20.5
12	19

Table 2: Peak shear strength (S_{up}) and remolded shear strength (S_{ur}) values averaged over 1 m intervals.

Depth (m)	S_{ur} (kPa)	S_{up} (kPa)
1	83.4	112.6
2	70.3	73.7
3	96.5	104.1
4	175.4	199.8
5	238.8	246.6
6	249.3	421.8
7	267.8	332.2
8	264.1	552.2
9	307.2	597.8

Table 3: Pile Configuration and Testing Summary

Pile ID	Length (m)	<u>Pile Shaft</u>		No. of Helices	<u>Helix</u>		Spacing Ratio (S/D)	Axial Load Testing	Strain Gauge
		Embedment (m)	Diameter (m)		Diameter (m)				
C6S	7.62	6.858	0.1683	1	0.4572	-	-	Compression	YES
T6S	7.62	6.858	0.1683	1	0.4572	-	-	Tension	NO
C6D	7.62	6.858	0.1683	2	0.4572	3	3	Compression	NO
T6D	7.62	6.858	0.1683	2	0.4572	3	3	Tension	NO
C8S	7.62	6.858	0.2191	1	0.6096	-	-	Compression	YES
T8S	7.62	6.858	0.2191	1	0.6096	-	-	Tension	YES
C8D	7.62	6.858	0.2191	2	0.6096	3	3	Compression	YES
T8D	7.62	6.858	0.2191	2	0.6096	3	3	Tension	YES
C10S	7.62	6.248	0.273	1	0.762	-	-	Compression	YES
C10D	7.62	6.248	0.273	2	0.762	3	3	Compression	YES
RP1-8	7.62	6.248	0.273	1	0.762	-	-	NA	NO

Table 4: Ultimate capacity of tested piles

Pile ID	Average Install Torque - 3D (kNm)	Davisson		Elkasabgy & El Naggar		Plunging		Fuller & Hoy	
		Load (kN)	Set (mm)	Load (kN)	Set (mm)	Load (kN)	Set (mm)	Load (kN)	Set (mm)
C6S	46.7	430	7.2	605	19.3	644	25	475	11.4
C6D	54.5	896	12.6	1090	23.5	1144	27.4	1060	23.0
C8S	77.1	800	11.3	1000	29.1	1064	39.4	890	19.8
C8D	97.8	1067	11.5	1516	34.0	1516	34	1390	26.9
C10S	105.4	930	11.6	1233	34.1	1445	58	1220	34.9
C10D	121.8	822	11.3	1332	31.2	1822	85.1	1425	46.9
T6S	49.7	728	10.7	837	24.2	870	31.7	720	11.8
T6D	61.7	874	12.4	982	22.5	982	22.5	870	15.4
T8S	71.7	971	11.7	-	-	1020	15.2	970	14.1
T8D	92.6	1053	13.1	1276	31.3	1380	35.8	1175	21.4

Table 5: Summary of correlation of torque to capacity factors

Pile ID	Average Install Torque - 3D (kN*m)	Davisson		Elkasabgy & El Naggar		Plunging		Fuller & Hoy	
		Load (kN)	K_t (m^{-1})	Load (kN)	K_t (m^{-1})	Load (kN)	K_t (m^{-1})	Load (kN)	K_t (m^{-1})
C6S	46.7	430	9.2	605	13.0	644	13.8	475	10.2
C6D	54.5	896	16.4	1090	20.0	1144	21.0	1060	19.4
C8S	77.14	800	10.4	1000	13.0	1064	13.8	890	11.5
C8D	97.8	1067	10.9	1516	15.5	1516	15.5	1390	14.2
C10S	105.4	930	8.8	1233	11.7	1445	13.7	1220	11.6
C10D	121.8	822	6.7	1332	10.9	1822	15.0	1425	11.7
T6S	49.7	728	14.6	837	16.8	870	17.5	720	14.5
T6D	61.7	874	14.2	982	15.9	982	15.9	870	14.1
T8S	71.7	971	13.5	-	-	1020	14.2	970	13.5
T8D	92.6	1053	11.4	1276	13.8	1380	14.9	1175	12.7

Figures

Figure 1:

Figure 2:

Helical Pile Schedule

Helical Pile Type	Shaft Dia (O.D) (d) (mm)	Pile Wall Thickness (w.t) (mm)	Helix Dia (D1) (mm)	Helix Dia (D2) (mm)	Pitch (P) (mm)	Helical Thickness (t) (mm)	Pile Length (m)	Inter Helical Spacing (S/D) (mm)	Embedment Depth (m)
RP 1-7	273	9.3	762	NA	152	19	7.62	NA	6.248
C/T 6S	168	7.1	457	NA	152	19	7.62	NA	7.315
C/T 6D	168	7.1	457	457	152	19	7.62	1372	7.315
C/T 8S	219	8.2	610	NA	152	19	7.62	NA	6.706
C/T 8D	219	8.2	610	610	152	19	7.62	1829	6.706
C/T 10S	273	9.3	762	NA	152	19	7.62	NA	6.248
C/T 10D	273	9.3	762	762	152	19	7.62	2286	6.553

Figure 3:

Figure 4:

Figure 5:

Figure 6: Torque profile for reaction piles (RP1-8)

Figure 7:

Figure 8:

Figure 9:

Figure 10:

Figure 11:

Figure 12:

Figure 13

Figure 14:

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Figure 15:

Figure 16:

Figure 17:

a)

b)

Figure 18

Figure 19: