In-situ loading tests of small-diameter helical piles and evaluation of design methods

Weidong Li, Lijun Deng
Department of Civil and Environmental Engineering – University of Alberta, Edmonton, Alberta, Canada
Richard Schmidt, Baocheng Li
Almita Piling Inc., Edmonton, Alberta, Canada

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ABSTRACT
The present research investigated the behaviour of three types of single-helix piles. The research conducted axial loading tests of helical piles installed in cohesive and cohesionless soils located in Alberta. Subsurface conditions of testing sites were investigated using cone penetration tests (CPT). Correlations between the axial capacity and final installation torque were proposed based on the testing results. It was observed that, for the two smaller piles, the torque factor varied with the amplitude of axial capacities while the biggest piles had relatively stabilized torque factor values. In addition, CPT-based method and several indirect methods were used to predict the pile capacities. It is shown that the CPT-based method provides adequate agreement with the test results despite that modified end bearing coefficients for helical piles were adopted for capacity prediction. It is therefore recommended that CPT-based method could be used directly to predict the pile axial capacity.

RÉSUMÉ
La présente étude s'est penchée sur le comportement de trois types de pieu à vis unique. Des essais de chargement axial ont été menés sur des pieux à vis installés dans des sols cohérents et pulvérulents situés en Alberta. A l'endroit des essais, le sous-sol est examiné par des essais au pénétromètre statique (CPT, Cone Penetration Test). A partir des résultats expérimentaux, il est proposé des corrélations entre la capacité de charge axiale et le couple mesuré en fin d'installation. Il est observé que pour les deux pieux les plus petits, le facteur de couple varie selon la capacité axiale du pieu, tandis que pour les pieux les plus gros, les valeurs du facteur de couple sont relativement constantes. Aussi, une méthode basée sur l'essai CPT et plusieurs méthodes indirectes sont utilisées pour estimer la capacité des pieux. La méthode basée sur l'essai CPT se montre en accord avec les résultats expérimentaux, malgré la modification des coefficients pour la résistance de pointe du pieu à vis, afin d'en estimer la capacité. La méthode basée sur l'essai CPT peut donc être directement utilisée pour estimer la capacité axiale du pieu.

1 INTRODUCTION

A helical pile is a deep foundation system consisting of a square or circular shaft and a varying number of helices affixed to the shaft. Helical piles can be categorized into single-helix pile and multi-helix pile. Helical piles are increasingly used in engineering applications because of the ease of installation and uninstallation. The specific applications of helical piles include pipelines, transmission towers, commercial buildings, and offshore structures. Another notable application of helical piles is the building remediation, especially in urban areas where the space may be inaccessible to large equipment (El Sharnouby and El Naggar 2012).

There are two methods for predicting the axial capacity of helical piles: theoretical method and empirical method. A number of theoretical studies have been conducted to approach pile capacity through several soil parameters and pile geometry. However, in the industry, a simplified empirical solution leading to pile capacity directly from installation torque is preferred (Hoyt and Clemence 1989, Ghaly et al. 1991, Tsuha and Aoki 2010).

The currently recognized theoretical methods are based on individual plate bearing model and cylindrical shearing model. For single-helix pile, only individual plate bearing method is appropriate. Using this method, the axial capacity of helical pile is assumed to be the sum of shaft friction and helical plate bearing against pile movement (Meyerhof and Adams 1968). Numerous conventional theoretical methods based on soil parameters have been proposed by researchers including Terzaghi (1943), Meyerhof (1951, 1976), and Burland (1973). The empirical methods are represented by LCPC method (Bustamante and Gianeselli 1982) and the torque factor method. The LCPC method was developed by Bustamante and Gianeselli, based on 197 pile load tests and raw CPT results. Pile category selection is vital for capacity prediction using the LCPC method. However, helical pile has not been added to the categories of LCPC method. Zhang (1999) suggested modified factors for multi-helix piles using the LCPC method. The torque factor method assumes the axial pile capacity to be installation torque (T) multiplied by torque factor (K_T), i.e., $Q_u=K_T \times T$. The installation torque refers to the final torque applied to the helical pile during installation. Livneh and El Naggar (2008) used the average installation torque over the final 1 meter of installation to deal with potential abrupt change of installation torque.

The present study investigates the axial behaviour of three single-helix piles. Thirty-two full-scale piles were installed and tested in three types of soils in Western Canada, including 18 axial compressive tests and 14 axial tensile tests. The study summarizes the methods of predicting the axial capacities of helical piles and evaluates the methods using the results of field tests..

The main objective of present study is to understand the behaviour of three types of small-diameter single-helix piles in cohesive and cohesionless soils under axial static loading. Theoretical methods and empirical methods mentioned in prior discussions are used to predict the pile capacities. The specific objectives are to: (*i*) evaluate axial capacities of single-helix piles, (*ii*) correlate axial capacities to installation torques, (*iii*) validate LCPC method for helical piles, and (*iv*) investigate the effect of complex soil profile on the pile behaviour.

2 BACKGROUND: DESIGN METHODS FOR HELICAL PILE CAPACITY

There are three design methods to calculate the ultimate axial capacities of helical piles, including the indirect theoretical method, the direct CPT-based LCPC method, and the torque factor method. This section provides a review of the current design methods for helical piles.

2.1 Theoretical Axial Pile Capacity

In the present study on single-helix piles, only individual bearing is appropriate. The capacity of single-helix pile is generally assumed to be the sum of helix bearing capacity and shaft friction capacity. Specifically, when considering uplift capacity of helix embedded in soil, a distinction has to be made between deep and shallow embedment. Different failure modes are considered depending on embedment depth of helical plate. Critical embedment ratio, the critical helix embedment depth over its diameter, is widely used to distinguish shallow from deep failure. Narasimha et al. (1993) suggested 4.0 for anchors embedded in cohesive soil based on a series of pullout tests on multi-helix screw anchors; Meyerhof and Adams (1968) established a correlation between critical embedment ratio and friction angle (Table 1).

Table 1. Critical embedment ratio varying with friction angle (After Meyerhof and Adams 1968)

Friction Angle φ (°)	Critical Embedment Ratio H/D_{cr}
25	3
30	4
35	5
40	7
45	9
48	11

Bearing capacities of the single circular helical plate in cohesive and cohesionless soils are estimated using Equation 1 and 2, respectively.

$$Q_b = A(N_c S_u + \sigma') \quad [1]$$

$$Q_b = A N_q \sigma' \quad [2]$$

where:

A	=	projected area of helix plate
N_c	=	bearing capacity factor for cohesive soil
S_u	=	undrained shear strength of cohesive soil
N_q	=	bearing capacity factor for cohesionless soil
σ'	=	effective stress at helix plate

CGS (2006) suggests that N_c is approximated to be 9.0 for helix diameter less than 0.5 m. Meyerhof values of N_q for drilled piles in cohesionless soil vary with friction angle (Table 2).

Table 2 . Bearing factor values for driven and drilled piles (After Meyerhof 1976)

Friction Angle, φ (°)	N_q, for Driven Piles	N_q, for Drilled Piles
20	8	4
25	12	5
28	20	8
30	25	12
32	35	17
34	45	22
36	60	30
38	80	40
40	120	60
42	160	80
45	230	115

Uplift capacity of individual helical plate is estimated using Equation 3 for cohesive soil and Equation 4 for cohesionless soil,

$$Q_u = A(N_c S_u + \sigma') \quad [3]$$

$$Q_u = A F_q \sigma' \quad [4]$$

where:

F_q = breakout factor for cohesionless soil

Meyerhof (1976) suggests that N_c values for the uplift capacity calculation vary with the embedment ratio of the helix plate (Equation 5).

$$N_c = 1.2H/D \leq 9.0 \quad [5]$$

For cohesionless soil, Das (1990) summarized the breakout factor, F_q, that is related to the friction angle and embedment ratio. However, the value suggested by Das (1990) was for driven piles only. In order to revise the breakout factor for drilled piles, the numerical relationship between Meyerhof's end bearing factors of driven piles and drilled piles (Table 2) is assumed to be valid for breakout factors as well.

The shaft-soil interaction, or friction, also accounts for the ultimate limit capacity of helical piles, and the shaft friction per unit length can be estimated using Equations 6 and 7 for cohesive soil and cohesionless soil, respectively:

$$q_s = \pi d \alpha s_u \quad [6]$$

$$q_s = \pi d \beta \sigma' \quad [7]$$

where:

d	=	shaft diameter
α	=	adhesion coefficient
β	=	a combined shaft resistance factor

Tomlinson (1957) calibrated the adhesion coefficient for steel shafts in clay (Figure 1). The combined shaft resistance factor, β, is recommended by CGS (2006).

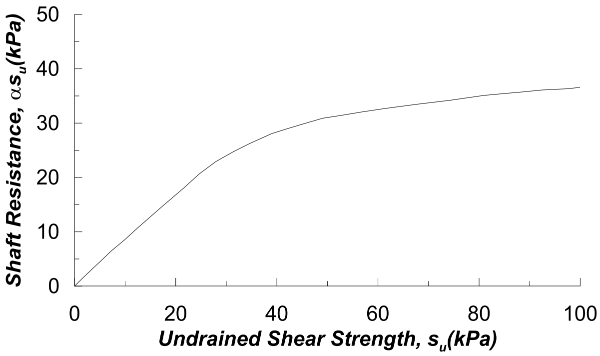


Figure 1. Adhesion (αs_u) of steel pile against undrained shear strength of clay (After Tomlinson 1957)

The effective length of pile shaft for friction resistance evaluation is simplified as the helix embedment (Narasimha Rao and Prasad 1993).

A frequent scenario in a project, that a weaker layer overlain by a thin layer of stronger soil at the bottom of helix, has to be considered. Meyerhof (1974) discussed the mechanism of footings punching through sand layer into the underlying clay. An assumption was made by Meyerhof and Adams (1968) that the ultimate bearing capacity of a footing punching through sand into thick clay bed could be approximated considering the failure as an inverted uplift problem. For the case in present testing, Equations 8 and 9 are appropriate to estimate the ultimate bearing capacity weakened by underlying clay. Michalowski and Shi (1995) proposed a series of modified design charts for evaluating the effect of weaker clay layer underlying, and the results came out that Meyerhof (1974) underestimated the ultimate bearing capacity by 17 percent roughly.

$$Q_b = A(N_c S_u + 2P_p \sin\delta/D + \gamma H) \quad [8]$$
$$P_p = 0.5\gamma t^2(1+2H/t)K_p/\cos\delta \quad [9]$$

where:

- N_c = bearing capacity factor = 5.14
- P_p = lateral passive earth pressure on vertical failure plane
- δ = average friction coefficient along vertical failure plane = $2\varphi/3$
- t = thickness of sand between helix and clay bed
- K_p = coefficient of passive earth pressure

2.2 Direct Method

The CPT-based LCPC method (Laboratoire Central des Ponts et Chausees) was developed by Bustamante and Gianeselli (1982), based on 197 pile load tests and raw CPT results. A wide range of pile types were tested and calibrated to cover as many conditions as possible. CGS (2006) has adopted and outlined the details of the LCPC method.

LCPC method introduced two important factors, end bearing coefficient (k_c) and friction coefficient (a), to approach the ultimate capacity of axially loaded piles. The scaling of end bearing coefficient and friction coefficient are based on soil type, cone resistance values and pile types. Equations 10 and 11 are respectively used to calculate end bearing and unit shaft friction.

$$Q_b = A k_c q_{ca} \quad [10]$$
$$q_s = \pi d(q_c/a) \quad [11]$$

where:

- k_c = end bearing coefficient
- q_{ca} = equivalent average cone resistance near pile end (helix)
- a = friction coefficient
- q_c = cone resistance of CPT

It is noticeable that the LCPC method is currently not capable of predicting the uplift capacity, since the method relies on compression tests and the calibration.

2.3 Torque Factor Method

Hoyt and Clemence (1989) proposed a simple relationship between final installation torque and pile capacity (Equation 12):

$$Q_u = K_t T \quad [12]$$

where:

- K_t = torque factor
- T = installation torque at the end of installation

The torque factor may range from 5 m^{-1} to 15 m^{-1} depending on the pile shaft geometries and loading directions. Although there is a lack of theory behind the torque method, Equation 12 is one of the most common design method used by the helical pile industry.

3 SITE INVESTIGATION

The present research conducted in-situ tests of three single-helix piles subject to axial compression and axial tension loads. Three sites were selected for the load tests. Site 1 at the University Farm is located in central Edmonton, Alberta, Canada, Site 2 at the Sand Pit is located about 7.5 km north to Bruderheim, Alberta, Canada, and Site 3 is located in the backyard of Almita Piling Inc. in Ponoka, Alberta, Canada.

Cone penetration tests (CPT) were performed to a minimum depth of 7.0 m to develop the soil profile. The CPT results show that at Site 1, the top 5.0 m layer consists of uniform clay, underlain by interbedded silty clay and clayey silt from 5.0 m to 7.0 m. The ground water table is 4.8 m deep. At Site 2, the top soils are interbeded clean sand and silty sand to a depth of 4.4 m, underlain by clayey silt to silty clay from 4.4 m to 5.6 m, underlain by a mixture of sand to silty sand from 5.6 m to 6.2 m; below 6.2 m, the soil is a mixture of silty clay to clay. The ground water table is 3.0 m deep. At Site 3, an organic clay sheet of 2 cm thickness covers the entire yard, underlain by a clayey silt to silty clay layer which is about 0.2 m thick.

Then a uniform soft clay deposit takes up the following profile from 0.2 m to 2.2 m. Between 2.2 m and 4.6 m is interbedded silty sand , sandy silt, and clayey silt. Deeper than 4.6 m is a deposit of sand.

The CPT test results are shown in Figures 2 to 4. The soil parameters were simplified according to the soil behaviour type for the calculations of ultimate capacities.

4 IN-SITU TESTING PROGRAM

4.1 Pile Dimensions

There are three types of piles with shaft diameters increasing from 7.3 cm to 11.4 cm, helix diameters from 0.305 m to 0.406 m, and pile lengths from 2.44 m to 4.57 m. The detailed pile dimensions and sketches are shown in Table 3 and Figure 5 respectively.

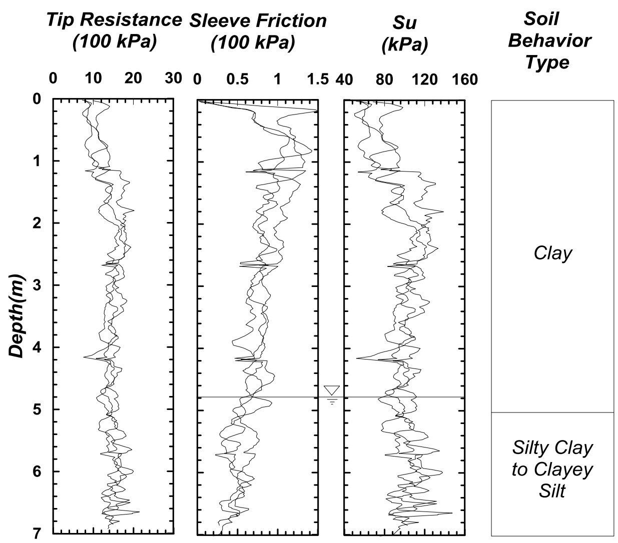


Figure 2. CPT profile of Site 1 at the University Farm

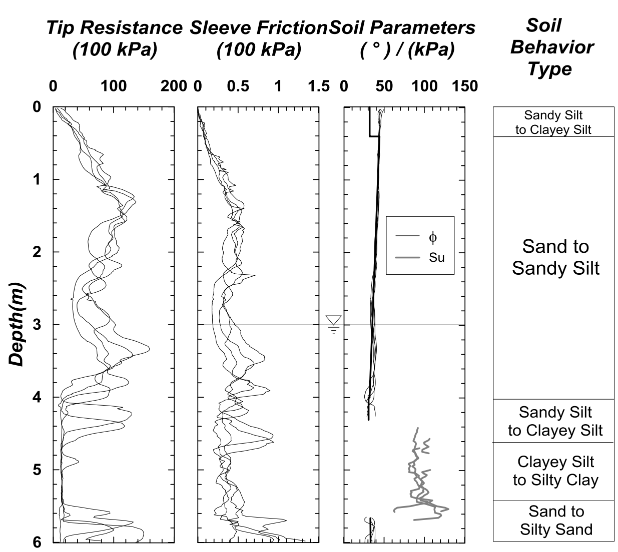


Figure 3. CPT profile of Site 2 at the Sand Pit in Bruderheim

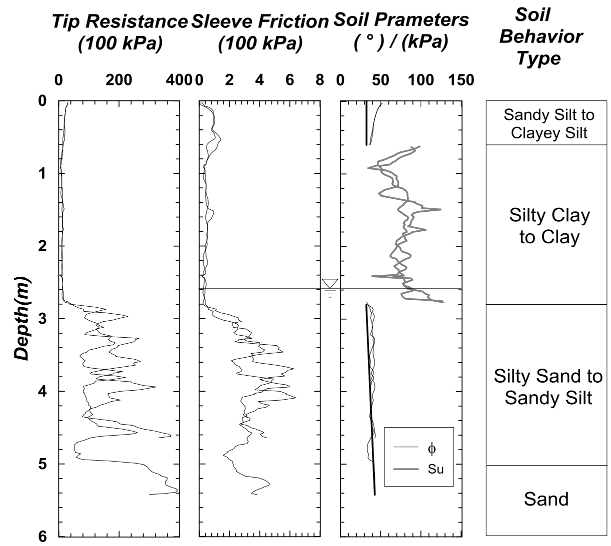


Figure 4. CPT profile of Site 3 at the Almita Yard in Ponoka

Table 3. Pile geometries of all types

Pile Type	L (m)	d (cm)	D (m)	H (m)	P (cm)
1	2.44	7.3	0.305	1.83	7.6
2	3.05	8.9	0.356	2.44	7.6
3	4.57	11.4	0.406	3.96	7.6

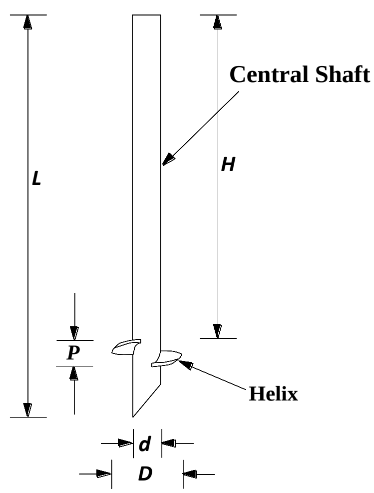


Figure 5. Sketch of tested single-helix pile

4.2 Testing Procedure

The in-situ loading tests were conducted according to ASTM standards D1143-81 (ASTM 1981) for compressive loads and D3689-90 (ASTM 1990a) for tensile loads.

For compressive tests following ASTM D 1143-81 (ASTM 1981), each pile was loaded to ultimate failure at an increment of 5% of the predicted design capacity. Constant time interval of 5 minutes was adopted to allow adequate time for pile mobilizing and data reading. Load increments were added until “failure” defined as the pile settlement reached 10% of the helix diameter. This maximum load was suspended for 15 minutes and then the unloading was started. Unloading stages adopted a decrement of 25% the maximum load. At the meantime, the constant time interval increased to 10 minutes. Figure 6 shows the setup of compression test.

Figure 6. Setup of axial compression tests

The tension tests followed the standard ASTM D3689-90 (ASTM 1990a). The testing procedure was similar to that in compressive tests despite the setup of equipment and devices. Figure 7 shows the setup of axial tension tests.

Figure 7. Setup of axial tension tests

5 TEST RESULTS

A large number of axial load vs. displacement curves are obtained from the axial compression and tension tests. An example axial load vs. displacement curve is shown in Figure 8. The ultimate axial capacity of each pile is interpreted from load-displacement curves at the displacement corresponding to 10% of the helix diameter. The measured ultimate capacities are summarized and presented in Table 4 and used as denominators under predicted capacities by direct and indirect methods for comparison.

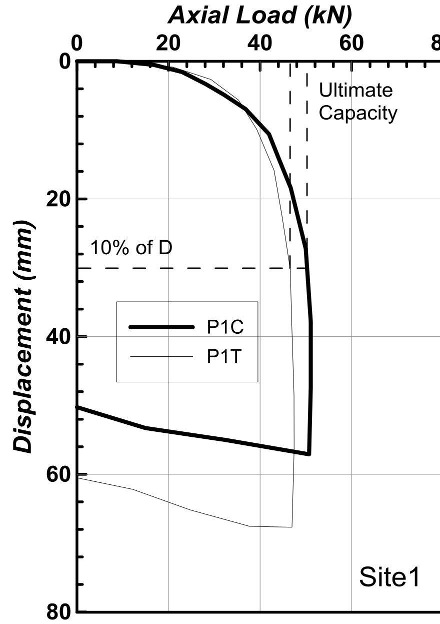


Figure 8 Load-Displacement curves of Type1 piles (P1) under compression (P1C) and tension (P1T) loading at Site1

Table 4. Measured and predicted pile capacities

Pile Type	Site Code	Compression/ Tension	Measured Capacities/kN	Predicted/kN Theo	LCPC
1	1	C	52,56,48	77	53
		T	48,44	64	53
	2	C	104,96	113	250
		T	80,73	108	250
	3	C	256	252	292
		T	160	159	292
2	1	C	75,70,73,72	117	82
		T	84,76	109	82
	2	C	126,134	113	328
		T	108,93	106	328
	3	C	254	276	442
		T	269	214	442
3	1	C	112,110	109	97
		T	100	154	97
	2	C	128,114	119	279
		T	178,164	169	279
	3	C	468	437	624
		T	329	275	624

5.1 Theoretical predictions

The soil strength parameters are estimated using CPT interpretations after Robertson and Cabal (2012). The ratio of each predicted axial capacity over the corresponding value observed in field testing is presented in Figure 9. The results showed that theoretical method overestimated the pile capacity in stiff clay. The average over-prediction was 48 percent, and the highest prediction was 67 percent greater than the testing result. However, in the varying soil conditions, i.e., Site 2 and Site 3, theoretical methodology was giving more reasonable predictions, with an average overestimation by 9%. And the highest over-prediction of 55 percent, which occurred to the Type 3 piles under compression in Site 2, was suspected to be the result of weaker clay layer underlying close to the bottom of helix.

After Michalowski and Shi (1995), the ultimate bearing capacity predictions of the two Type 3 piles were revised to be 119 kN, which fell within ±8 percent off testing results. During this revision, two assumptions were made: *i*) the thickness of sand layer between helix and clay bed, T, was selected to be 0.3 m since all four CPT logs displayed varying T from 0.2 m to 0.4 m; *ii*) the prediction developed on the basis of shallow footing also works for the present deep foundation type since both of them were undergoing punch shearing failure.

5.2 Direct predictions (LCPC)

Although the LCPC method is designed for bearing capacity, uplift capacity can be achieved assuming the bearing and uplift capacities of individual helix are identical. The pile category system provides a similar pile type of "cast screwed pile" for the present calculation. The predicted results are also plotted in Figure 9 the same way as in theoretical predictions.

It is notable that CPT logs are combined and the lowest value at any specific depth is selected to process the calculation for conservatism. The plot shows a perfect agreement between testing results and predictions for stiff clay at Site 1 with an average over-prediction of 5 percent. All predicted capacities of 14 helical piles tested in Site 1 falls within an error of 17 percent of testing results.

The predictions at Site 2 showed much greater error (averaging 159 percent higher than testing results) than expected, which was unacceptable. For Site 3, the prediction errors also reached up to 90 percent over the real capacities on the unsafe side.

Figure 9b shows that the LCPC method agrees with the test results in uniform clay in Site 1 but significantly overestimates the capacities of piles installed in cohesionless soils in Site 2 and 3. The observation is due to the fact that Bustamante and Gianeselli (1982) conducted the field testing in cohesive soils; the observation was also concluded by Tappenden (2007). The coefficients and factors summarized in LCPC method are based on soil types, cone tip resistance of CPT test, and pile types. However, there are too many types of soils in the nature and it is inappropriate to define the soil types simply by cone tip resistance. In the present study, the clay deposits in Site 1 are concise and uniform, therefore good predictions are easy to obtain. However, in Site 2 and 3, the CPT logs showed the variation of soil types versus the depth. Thus there is high probability that neither of the soil type of Site 2 and 3 has been tested explicitly by Bustamante and Gianeselli (1982).

5.3 Torque factors

The torque factors were estimated from the test results and presented in Figure 10. Torque factors were classified by pile types and loading direction. Generally, the torque factors for tension capacity were smaller than the torque factors for compression capacity. And for Type 1 and Type 2 piles, the torque factors decreased along with the increasing installation torques. For instance, the torque factor of Type 1 pile subject to compressive loading decreased from 34 m^{-1} to 21 m^{-1} when the installation torque increased from 678 Nm up to 10847 Nm. But for the biggest pile, Type 3, the measured pile capacities showed an approximately linear and constant relation to the corresponding installation torques. Additionally, the bigger pile had lower torque factor.

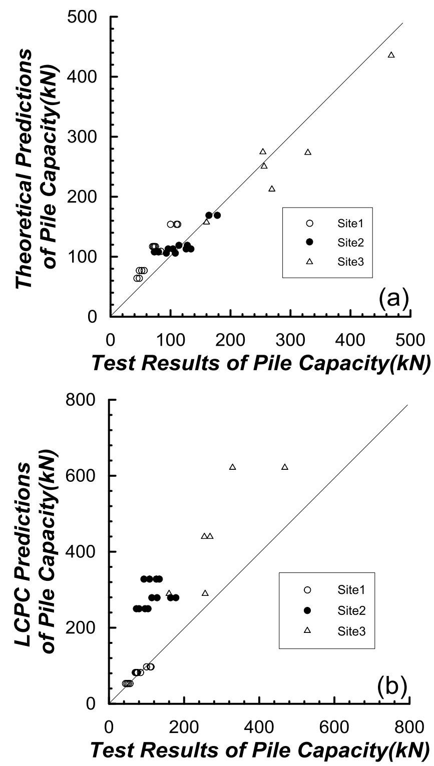
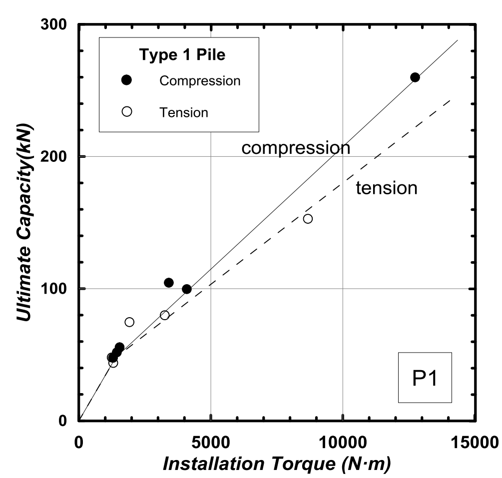
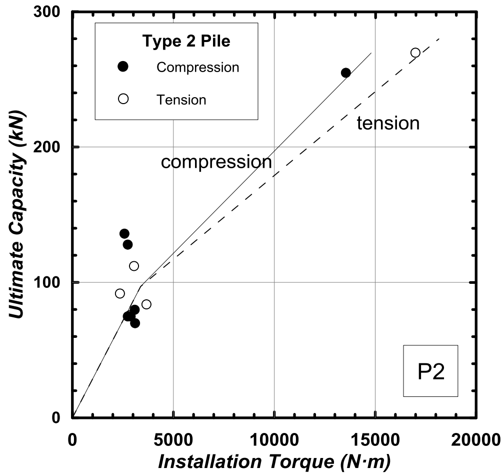


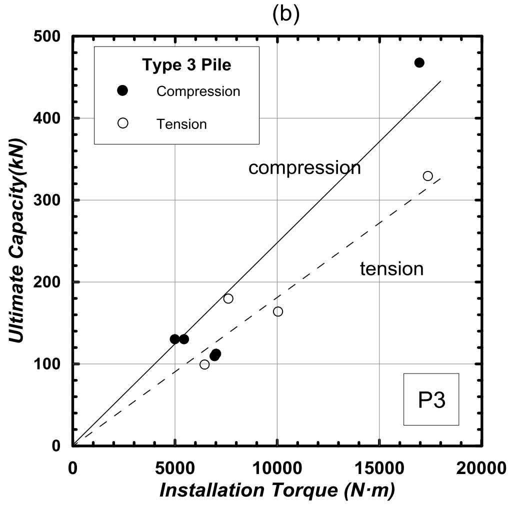
Figure 9. Comparison of predicted pile capacities and measured pile capacities using (a) theoretical method and (b) LCPC method



(a)



(b)



(c)

Figure 10. Torque factor design chart for tested piles

CONCLUSIONS

The axial capacities of 32 single-helix piles installed in Western Canada were obtained from static loading tests in cohesive and cohesionless soils. The sites are characterized by CPT logs. The testing results are presented in the paper and the current design methods for helical piles were evaluated through comparing the predictions to the testing results. The following conclusions may be drawn.

Theoretical method gave good predictions for cohesionless soils, especially when the ultimate capacity was affected by underlying weak layers. There are relative studies to help solve more specific and complicated problems introduced by complex soil conditions. Most of the predicted capacities fell within 15 percent off the capacities obtained from testing results. However, the theoretical method overestimated the axial capacity of cohesive soil using a commonly recommended bearing capacity factor of 9.0. The present study recommends that in-situ testing is necessary to obtain the bearing capacity factor in clay.

The LCPC method could predict the ultimate axial capacity of helical piles in the uniform cohesive soil with less than 17 percent error. The LCPC method overpredicted the pile capacities in cohesionless soils mixed with several cohesive soils by up to 3.5 times greater than measured. The LCPC method may not be accurate due to the complexity of soil deposits, and good predictions are achievable only for the soil profiles explicitly tested during the development of LCPC method.

A series of torque factors were proposed for the piles tested. The torque factors were found to vary with helical pile dimensions and loading directions. The bigger pile had a more linearized but lower torque factor generally.

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