PREDICTING THE AXIAL CAPACITY OF SCREW PILES INSTALLED IN CANADIAN SOILS

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ABSTRACT
The results of 26 full-scale static axial load tests are presented for screw piles installed in Alberta and British Columbia since 1998, and the effectiveness of three design methods are evaluated for predicting the axial capacity of screw piles in cohesive and cohensionless soils. Theoretical formulations for capacity calculations are examined alongside the LCPC direct pile design method, and an empirical relationship correlating the installation torque to the ultimate screw pile capacity.

RÉSUMÉ
Les résultats de 26 essais de chargement statiques sont présentés pour des pieux vissés installés en Alberta et en Colombie Britannique depuis 1998 ainsi que l'évaluation de trois méthodes de conception utilisées pour prédire la capacité axiale des pieux vissés en sols pulvérulents. Les formules théoriques utilisées pour fins de calcul de la capacité sont examinées en parallèle avec la méthode du LCPC et une relation empirique établissant la corrélation avec la capacité ultime d'un pieu vissé.

1 INTRODUCTION

This paper presents three design methodologies for calculating the axial capacity of screw piles loaded in uplift or compression in cohesive and cohesionless soils. An indirect design approach based on theoretical formulations of bearing capacity and skin friction is presented, followed by the LCPC direct pile design method (based on the cone penetration test), and an empirical relationship correlating the installation torque to the axial screw pile capacity. The results of 26 static load tests conducted in tension and compression on screw pile installations in cohesive and cohesionless soils are summarized. Each of the capacity prediction methods are applied to the screw piles tested, and the effectiveness of the methods are discussed in light of discrepancies between the measured and predicted capacities.

1.1 General

Screw piles, also known as helical piles or screw anchors, are structural, deep foundation elements used to provide stability against compressive, tensile, and lateral loads. Screw piles consist of one or more circular, helical plates affixed to a central shaft of smaller diameter.

Screw piles are embedded into the soil by the application of a turning moment to the pile head, causing the helices to penetrate the ground in a "screwing" motion without creating vibration or spoil. Installation of screw piles is typically accomplished using a torque head affixed to the arm of a backhoe or a trailer-mounted hydraulic boom. Screw piles are frequently installed to depths of less than 10 m, and are favoured for their rapid installation, typically requiring about 30 minutes per pile using a two person crew.

Screw piles are not well-suited for installation into very hard, dense, or gravelly soils, where shallow refusal or damage to the steel helical plates could occur. However, commercial screw piles have been fabricated with helices of up to 25 mm thick to offset the likelihood of structural damage during installation in adverse conditions.

2 SHALLOW VERSUS DEEP FAILURE IN UPLIFT

For screw piles loaded in uplift, a distinction must be made between shallow and deep embedment. For shallow screw piles in uplift, the bearing failure zone above the uppermost helix will extend to the ground surface, as evidenced by tensile cracking and surface heave (Narasimha Rao et al., 1993). For deep screw piles in uplift, the bearing failure zone above the uppermost helix will be contained below the surface. The demarcation between the shallow and deep failure condition is expressed as the critical embedment ratio, H_{crit}, which is equal to the embedment depth of the uppermost helix divided by its diameter.

In cohesionless soils, the critical embedment ratio, H_{crit}, beyond which the deep failure condition prevails, is dependent upon the soil friction angle, as given in Table 1 (after Meyerhof and Adams 1968). For screw piles installed in cohesive soils, Narasimha Rao et al. (1993)

suggest that a critical embedment ratio of about 4.0 be used, on the basis of laboratory uplift tests on model screw piles installed in clay. Whether an embedded screw pile may be considered as shallow or deep will affect the parameters chosen in the calculation of the axial capacity.

Table 1: Variation of critical embedment ratio with soil friction angle (after Meyerhof and Adams 1968)

Soil friction angle, Φ	Critical Embedment Ratio, H_{crit}
25°	3
30°	4
35°	5
40°	7
45°	9

3 FAILURE MODELS FOR AXIALLY LOADED SCREW PILES

There are two primary failure models established in the literature for describing the behaviour of screw piles under axial load—these are the cylindrical shear model and the individual plate bearing model (Narasimha Rao et al. 1993). The choice of the most appropriate failure model is considered to be dependent on the screw pile geometry. For multi-helix screw piles, the value of the inter-helix spacing ratio, which is equal to the spacing (S) between adjacent helical plates divided by their average diameter (D), is the parameter which determines the use of either model.

3.1 Individual Plate Bearing Model

The individual plate bearing model is applicable to single-helix screw piles and multi-helix screw piles with an inter-helix spacing ratio equal to or greater than 3.0 (CFEM 2006). The model assumes that bearing failure occurs above or below each individual helix in tension or compression, with negligible interference between multiple failure zones.

The ultimate axial capacity given by the individual plate bearing model will be the summation of the bearing capacity above/below each helix; additionally, skin friction along the section of pile shaft between the uppermost helix and the ground surface may be considered for compressive loading or for uplift loading under the deep failure condition, but should be neglected for uplift under the shallow failure condition (Narasimha Rao et al. 1993; Mitsch and Clemence 1985).

3.2 Cylindrical Shear Model

The cylindrical shear model is applicable to multi-helix screw piles with inter-helix spacing ratios less than 3.0. The model assumes that during axial loading, a frictional, cylindrical failure surface is formed between the top and bottom helices. The failure surface is attributed to installation disturbance and interference between closely-spaced helical plates.

The ultimate screw pile capacity calculated as per the cylindrical shear model will be the sum of the bearing capacity below the bottom helix in compression, or above the top helix in tension, plus the skin friction acting along the cylinder of soil circumscribed between the top and bottom helices. Skin friction acting along the section of pile shaft between the uppermost helix and the ground surface may be considered for compressive loading, or for uplift loading under the deep failure condition, but should be neglected for uplift under the shallow failure condition (Narasimha Rao et al. 1993; Mitsch and Clemence 1985).

4 CALCULATION OF AXIAL SCREW PILE CAPACITY

Screw pile axial capacity calculations may be performed using indirect (theoretical) methods, direct methods, or empirical methods. Indirect or theoretical methods, although involving some degree of empiricism, are based on established equations of bearing capacity and skin friction used to describe the behaviour of piles and buried anchors in compression and uplift. The equations depend on the accurate estimation of characteristic soil parameters. Direct methods eliminate the intermediate calculation of soil parameters by relating in-situ test results, such as from a cone penetration test (CPT), to the unit bearing resistance and skin friction of a pile. Empirical methods for predicting the axial capacity of a screw pile attempt to directly relate the torque measured during installation to the ultimate pile capacity; this concept is analogous to the relationship between pile driving effort and capacity. A detailed overview of the three design methodologies is given in the following subsections.

4.1 Theoretical Methods

A summary of the appropriate theoretical equations for calculating ultimate unit bearing and frictional resistance of embedded screw piles is given below. The ultimate screw pile capacity is determined by application of the equations in conjunction with the appropriate failure model, i.e., the cylindrical shear model or the individual plate bearing model.

4.1.1 Bearing

The ultimate bearing resistance, Q_b, mobilized beneath a screw pile helix is given by Equation [1] for cohesive soils, and Equation [2] for cohesionless soils.

$$Q_b = \frac{\pi (D^2 - d^2)}{4}\left(N_c C_u + \gamma' H\right) \qquad [1]$$

$$Q_b = N_q^* \gamma' H \frac{\pi (D^2 - d^2)}{4} \qquad [2]$$

Where:
H = depth of helix below ground surface (m)
D = diameter of helix (m)
d = diameter of pile shaft (m)
N_c = bearing capacity factor for cohesive soil
C_u = undrained shear strength of cohesive soil (kPa)
N_q^* = bearing capacity factor for cohesionless soil
γ' = effective soil unit weight (kN/m^3)

For calculating the bearing capacity under the lowermost helix, the variable 'd' may be omitted from Equations [1] and [2], assuming the formation of a soil plug within the base of the screw pile shaft.

The value of the bearing capacity factor in cohesive soil, N_c, is typically taken as 9.0 (CFEM 2006). In cohesionless soil, values for the bearing capacity factor N_q^*, such as those recommended by Vesic (1963) may be used (Figure 1).

4.1.2 Uplift

The ultimate uplift resistance, Q_u, mobilized above the helix of a screw pile in tension is given by Equation [3] for cohesive soils, and Equation [4] for cohesionless soils.

$$Q_u = \frac{\pi(D^2 - d^2)}{4}\left(N_c C_u + \gamma' H\right) \quad [3]$$

$$Q_u = F_q \gamma' H \frac{\pi(D^2 - d^2)}{4} \quad [4]$$

Where:
F_q = breakout factor for cohesionless soil

The bearing capacity factor for cohesive soil, N_c, to be used for uplift should be determined by Equation [5], as dependant on the embedment depth of the uppermost helix, H_1, divided by the helix diameter, D_1 (after Meyerhof 1976).

$$N_c = 1.2\left(\frac{H_1}{D_1}\right) \leq 9 \quad [5]$$

Values for the breakout factor, F_q, to be used in cohesionless soils have been determined by Das (1990) based on Mitsch and Clemence's (1985) theory, and are shown in Figure 2 for shallow screw piles ($H_1/D_1 \leq H/D_{crit}$) and Figure 3 for deep screw piles ($H_1/D_1 > H/D_{crit}$).

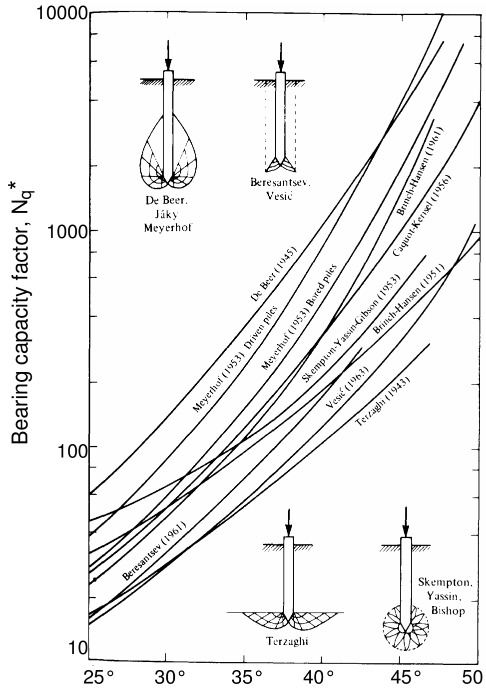

Figure 1: Variation of bearing capacity factor, from various authors (after Winterkorn and Fang 1975)

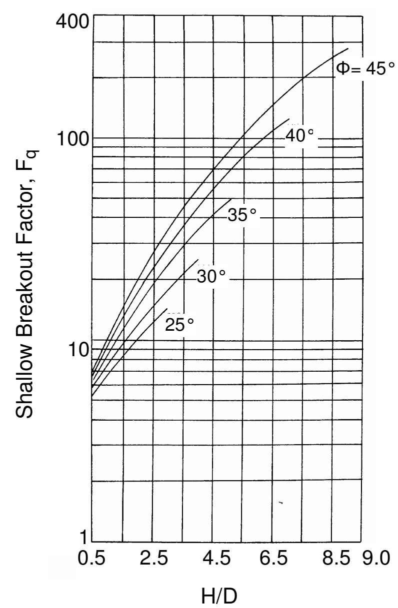

Figure 2: Variation of breakout factor, shallow condition (after Das, 1990)

4.1.3 Shaft Friction

Skin friction may be considered to act along the length of the screw pile shaft between the uppermost helix and the

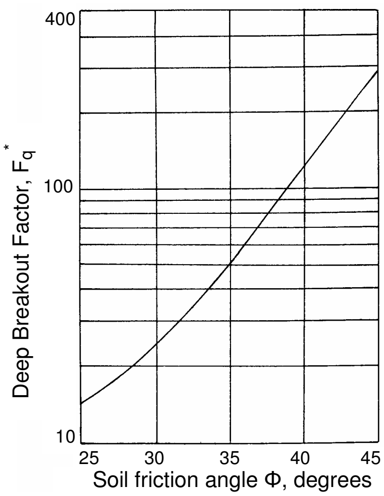

Figure 3: Variation of breakout factor, deep condition (after Das, 1990)

ground surface for screw piles in compression, or for deep screw piles in uplift. However, a distance equivalent to one helix diameter should be subtracted from the available shaft length, to account for the effect of bearing disturbance above the uppermost helix when in tension, and for the void-forming or 'shadowing' effect above the uppermost helix when acting in compression (Zhang 1999). The effective shaft length, H_{eff}, over which the ultimate skin friction may be mobilized, is then as defined in Equation [6].

$$H_{eff} = H_1 - D_1 \quad [6]$$

For a screw pile embedded in cohesionless soil, the ultimate unit shaft friction, Q_S, may be calculated by Equation [7] for compressive loading, and by Equation [8] for uplift loading.

$$Q_s = \pi \cdot d \cdot H_{eff} \cdot \left(\frac{1}{2} \gamma' H_1 K_s \tan\phi \right) \quad [7]$$

$$Q_s = \pi \cdot d \cdot H_{eff} \cdot \left(\frac{1}{2} \gamma' H_1 K_u \tan\phi \right) \quad [8]$$

Where:
Φ = effective soil friction angle
K_s = lateral earth pressure coefficient in compression
K_u = lateral earth pressure coefficient in uplift

Mitsch and Clemence (1985) have proposed values for the parameter K_u on the basis of laboratory scale tests on model screw piles under uplift in sand, as shown in Table 2.

Table 2: Recommended K_u values for screw piles (after Mitsch and Clemence 1985)

Soil friction angle, Φ	Lateral earth pressure coefficient in uplift, K_u
25°	0.70
30°	0.90
35°	1.50
40°	2.35
45°	3.20

The K_u values suggested by Mitsch and Clemence (1985) are approximately 40 percent less than the K_u values proposed by Meyerhof and Adams (1968) for the uplift capacity of buried foundations. The lower coefficients for screw piles are attributed to the disturbance created by the churning action of the helices during installation; Meyerhof and Adams' (1968) values, however, were based on essentially undisturbed soil (Mitsch and Clemence 1985).

For compressive loading, values for the parameter K_S may be calculated by Equation [9].

$$K_s = \frac{\beta}{\tan\delta} \quad [9]$$

Where:
β = skin friction design parameter for displacement piles in sand (CFEM 2006)
δ = interface friction angle, equal to 0.6Φ for steel embedded in sand (Kulhawy 1984)

In the case of a screw pile embedded in cohesive soil, the ultimate shaft friction mobilized under either compressive or uplift loading may be calculated by Equation [10].

$$Q_s = \pi \cdot d \cdot H_{eff} \cdot C_u \cdot \alpha \quad [10]$$

Where:
α = adhesion factor

Values for the adhesion factor, α, such as those given in the Canadian Foundation Engineering Manual for grouted anchors installed in cohesive soils, may be used (CFEM 2006).

4.1.4 Cylindrical Friction

Under the cylindrical shear failure model, frictional resistance is mobilized along the circumscribed cylinder of soil bounded by the top and bottom helices of the screw pile. Equations [11] and [12] may be applied in cohesionless soil to calculate the ultimate cylindrical friction, Q_{cyl}, mobilized in compression and uplift, respectively. In cohesive soil, Equation [13] should be used for both uplift and compression.

$$Q_{cyl} = \pi \cdot D \cdot \left(\frac{1}{2}\gamma' K_s \tan\phi\right) \cdot \left(H_n{}^2 - H_1{}^2\right) \quad [11]$$

$$Q_{cyl} = \pi \cdot D \cdot \left(\frac{1}{2}\gamma' K_u \tan\phi\right) \cdot \left(H_n{}^2 - H_1{}^2\right) \quad [12]$$

$$Q_{cyl} = \pi \cdot d \cdot C_u \cdot \alpha \cdot \left(H_n - H_1\right) \quad [13]$$

Where:

H_n = depth of lowermost helix below ground (m)

H_1 = depth of uppermost helix below ground (m)

4.2 Direct Methods (LCPC Method)

The LCPC method attempts to calculate the ultimate unit skin friction and end bearing of a pile on the basis of scaling coefficients applied to an in-situ CPT profile of tip resistance. The method is documented by Bustamante and Gianeselli (1982), and described in the Canadian Foundation Engineering Manual (CFEM 2006)

The scaling coefficients used in the LCPC method are determined on the basis of soil type and magnitude of the measured CPT tip resistance. The method was derived from the interpretation of 197 static load tests conducted on piles, mostly of the bored or driven type, installed in France under a variety of soil conditions.

The LCPC method may be used to calculate components of unit skin friction and bearing to be applied to the screw pile failure surface determined by the cylindrical shear model or the individual plate bearing model. However, it should be noted that the majority of the conventional pile load tests on which the LCPC method was based were conducted in compression, in soil types described as sand, clay, silt, and chalk.

4.3 Empirical Methods

Empirical relationships correlating the required installation torque to the ultimate axial screw pile capacity have been proposed by several authors (Ghaly and Hanna 1991; Hoyt and Clemence 1989; Narasimha Rao et al. 1989; Perko 2000). The concept of an empirical torque to capacity correlation is analogous to the relationship of pile driving effort to pile capacity (Hoyt and Clemence 1989).

The simplest form of the empirical torque relationship directly relates the capacity, Q, of a screw pile to the installation torque, T, by means of an empirical scaling factor, K_t, as shown in Equation [14].

$$Q = K_t \cdot T \quad [14]$$

In an analysis of 91 screw pile load tests in uplift from the published literature and the authors' private files, Hoyt and Clemence (1989) obtained good approximations of ultimate uplift capacities using K_t equal to 33 m^{-1} for all square-shaft screw piles and round-shaft screw piles less the 89 mm in diameter, 23 m^{-1} for round shaft piles 89 mm in diameter, and 9.8 m^{-1} for round, 89-mm-diameter piles with 219 mm diameter extension shafts (extending from the top helix to the ground surface). Hoyt and Clemence (1989) averaged the installation torque over the final distance of penetration equal to three times the largest helix diameter, and all piles were multi-helix screw piles loaded in uplift under the deep failure condition. As a preliminary estimate, the Canadian Foundation Engineering Manual (CFEM 2006) suggests that a K_t factor of 10 m^{-1} be used in the absence of any site-specific correlations established by pile load testing. As the torque relationship may only be used to predict the screw pile capacity after installation has taken place, it is best-suited for on-site production control rather than for the actual design of piles in the office (Hoyt and Clemence 1989).

5 FIELD TESTING PROGRAM

As part of the current research program, load-displacement data was compiled for 29 static axial load tests performed on full-scale screw piles installed at nine different sites in Western Canada. The load test sites were located in or near Bruderheim, Edmonton, Ft. McMurray, Ft. Saskatchewan, and Hythe, Alberta, and Ft. St. John, British Columbia. All screw pile load tests were conducted in accordance with the respective ASTM standards for individual piles loaded in compression and tension (ASTM Designation: D1143 1981; ASTM Designation: D3689 1990). The incremental application of load to the test piles was as per the *'Quick Test'* procedure described in the above standards.

5.1 Investigation of Load Test Sites

Geotechnical field investigations of varied scope were conducted at the screw pile load test sites. Cone penetration tests (CPT's) were performed at seven of the nine test sites, supplemented by test borings and standard penetration tests (SPT's) at six of the seven sites. The investigations at the remaining two load test sites were comprised of test borings supplemented by SPT's.

The surficial geologies of the test sites, although varied, predominantly consisted of glacially-derived soils such as lacustrine clay, clay till, sand, and sand till. Additionally, near-surface weathered clay shale bedrock was encountered at two of the test sites.

Table 3 summarizes the locations of the nine load test sites, the field investigation methods and predominant soil conditions at each site, and details of the corresponding screw pile load tests conducted in tension or compression.

Table 3: Load Test Sites, Screw Pile Geometries and Ultimate Axial Capacities

Test Site No.	Vicinity of Test Site	Predominant Soil Conditions	Soil Investigation Technique(s)	Load Test Designation (Tension, T, or Compression, C)	Pile Length (m)	No. of Helices	Helix Diameter(s) D_1 (cm)	D_2 (cm)	D_3 (cm)	Shaft Diameter d (cm)	Inter-Helix Spacing Ratio S/D	Installatio Torque (kN·m)	Ultimate Axial Capacity (kN)
1	Edmonton, AB	Stiff silty clay	Test Holes, SPT, CPT	C1	5.0	3	35.6	35.6	35.6	21.9	1.5	20.3	180
				C2	3.0	3	35.6	35.6	35.6	21.9	1.5	15.6	160
				C3	5.0	2	35.6	35.6	--	21.9	3.0	19.5	210
				T1	5.0	3	35.6	35.6	35.6	21.9	1.5	22.1	210
				T2	3.0	3	35.6	35.6	35.6	21.9	1.5	20.3	140
				T3	5.0	2	35.6	35.6	--	21.9	3.0	22.9	210
2	Bruderheim, AB	Loose to compact silty sand	Test Holes, SPT, CPT	C4	5.0	3	35.6	35.6	35.6	21.9	1.5	44.7	470
				C5	3.0	3	35.6	35.6	35.6	21.9	1.5	40.7	420
				C6	5.0	2	35.6	35.6	--	21.9	3.0	44.7	380
				T4	5.0	3	35.6	35.6	35.6	21.9	1.5	50.8	360
				T5	3.0	3	35.6	35.6	35.6	21.9	1.5	42.7	190
				T6	5.0	2	35.6	35.6	--	21.9	3.0	47.9	360
3	Ft. Saskatchewan, AB	Stiff silty clay	Test Holes, CPT	C7	4.6	1	45.7	--	--	17.8	--	25.6	212
				C8	4.6	1	45.7	--	--	21.9	--	34.8	268
				C9	5.5	2	50.8	45.7	--	17.8	3.1	31.5	372
4	Lamont, AB	Hard clay till over weathered clay shale	Test Holes, CPT	C10	9.3	2	50.8	45.7	--	24.4	3.2	118.6	1177
5	Ft. McMurray, AB	Hard clay till	Test Holes, SPT, CPT	C11	5.9	1	76.2	--	--	27.3	--	85.4	1094
				C12	6.0	2	76.2	76.2	--	27.3	3.0	97.6	1375
				T7	5.9	1	76.2	--	--	27.3	--	81.3	800
				T8	6.0	2	76.2	76.2	--	27.3	3.0	122.0	1325
6	Ft. McMurray, AB	Very dense sand till	Test Holes, SPT, CPT	T9	4.9	1	76.2	--	--	40.6	--	257.6	2025
7	Hythe, AB	Firm to stiff clay till over weathered clay shale	Test Holes, SPT, CPT	C13	7.5	1	40.0	--	--	21.9	--	120.9	1075
8	Ft. St. John, BC	Firm to stiff silty clay	Test Holes, SPT	C14	10.4	2	91.4	91.4	--	32.4	1.8	79.0	634
				C15	6.1	3	50.8	50.8	50.8	14.0	3.0	19.7	270
9	Ft. St. John, BC	Stiff silty clay	CPT	C16	5.0	2	45.7	45.7	--	11.4	3.3	13.5	245
				C17	4.0	1	45.7	--	--	11.4	--	8.0	169

5.2 Axial Load Test Results

The ultimate axial capacity of each of the test piles was interpreted from the load-deflection curve using the Brinch-Hansen 80% Failure Criterion (Fellenius 1990). The interpreted ultimate axial capacities of the screw piles are summarized in Table 3. Additionally, the specific geometry of each screw pile and the required installation torques at the finished pile depths are also shown in Table 3.

6 PILE CAPACITY PREDICTIONS

Class 3 ultimate axial capacity predictions were made for each of the 26 test piles using the indirect theoretical methodology described above, the LCPC direct pile design method, and the empirical torque correlation. The failure surfaces were approximated by the cylindrical shear model for multi-helix screw piles having an inter-helix spacing ratio (S/D) less than 3.0, and by the individual plate bearing model for single-helix screw piles and multi-helix piles with S/D ≥ 3.0.

Soil strength parameters used in the theoretical methodology were estimated on the basis of published correlations to SPT and CPT data (Lunne et al. 1997, Peck et al. 1974; Stroud and Butler 1975). Capacity predictions by the LCPC method were accomplished for 23 of the 26 test piles, as CPT's were not included in the geotechnical investigations at test site numbers 7 and 8. A K_t factor of 9.2 m^{-1} was derived for use in the empirical torque correlation by linear regression of the measured screw pile capacities versus the required installation torques, with an R^2 value equal to 0.96.

6.1 Theoretical Predictions

Consistently accurate capacity predictions were accomplished using the theoretical methodology, as shown in Figure 4. The ultimate screw pile capacities predicted by the theoretical method were plotted as a ratio of the measured capacities, and exhibited a maximum over-prediction of 11 percent. Of the 26 capacity predictions made, 21 fell within 20 percent of the measured capacities, and the remaining five capacity predictions outside this boundary erred on the side of conservatism, with a maximum underestimation of 42 percent.

6.2 LCPC Predictions

Using the LCPC method, capacity predictions for the test piles showed significant variation from the measured capacities, as shown in Figure 4. While good predictions within 20 percent of the measured capacities were obtained for the screw piles in firm to stiff clay deposits, significant over predictions of 2.25 to 4.75 times the measured capacity occurred in the hard and very dense glacial till materials.

The use of the LCPC method for screw piles installed in glacial till soils is hindered by the limited empirical scaling factors defined within the method. The scaling factors are grouped into categories based on the soil type, whether clay, silt, sand, or chalk, and the soil stiffness indicated by the cone penetration tip resistance at the appropriate depth (CFEM 2006). For clay soils, as concerns piles T7, T8, C11, and C12, installed in clay till, the available scaling factors are for *'compact to stiff clay and compact silt'* (Bustamante and Gianeselli 1982) having tip resistance readings greater than 5000 kPa. The clay till soils into which piles T7, T8, C11, and C12 were installed exhibited tip resistance readings in the order of 10,000 to 20,000 kPa at the depth of the piles.

Similarly, test pile T9 was installed into very dense sand till, and the scaling factors available for use within the LCPC method were for *'compact to very compact sand and gravel'* (Bustamante and Gianeselli 1982), exhibiting CPT tip resistance values of 12,000 kPa or greater. Tip resistance values of 15,000 to 60,000 kPa were recorded in the sand till at the T9 test site.

On the basis of the capacity predictions shown in Figure 4 for piles C11, C12, T7, T8, and T9, it is concluded that the LCPC method is not directly applicable to glacial till soils, as the scaling factors defined within the method can only be accurately applied to those soils for which they were explicitly intended, i.e. clay, silt, sand, or chalk.

Additionally, the uplift capacity of pile T5 was overestimated by approximately 150 percent using the LCPC method. This test pile was installed in loose to compact silty sand, at a depth corresponding the shallow failure condition. Based on the preceding theoretical discussions, the capacity of a helix installed in sand will be significantly lower in uplift than in compression, especially in the case of a shallow failure condition. As the LCPC method was developed primarily on the basis of compression tests, it may not be directly applied to piles loaded in tension in cohesionless soils.

However for 'deep' screw piles in cohesive soils, the bearing capacity and skin friction in uplift will be essentially equivalent to that in compression, and therefore the LCPC method may be applied with some degree of confidence.

6.3 Torque Predictions

Of the 26 ultimate capacity predictions made using the empirical torque relationship with a K_t factor of 9.2 m^{-1}, 23 fell within 30 percent of the measured screw pile capacities, as shown in Figure 4.

The ultimate capacities of test piles C16 and C17 were underestimated by 50 and 60 percent, respectively, using the torque method. These discrepancies could be due to the fact that the two piles were fabricated with the smallest shaft diameter of any of the test piles, 11.4 cm. There may be justification for the use of a larger K_t factor for screw piles of relatively small shaft diameter, as suggested by Hoyt and Clemence (1985). Based on the two load test results for piles C16 and C17, a K_t factor of 18 m^{-1} would provide reasonable predictions of the ultimate pile capacities for screw piles with shaft diameters of 11.4 cm. However, use of a K_t factor in excess of 10 m^{-1}, as is recommended by the Canadian Foundation Engineering Manual (CFEM 2006), should be cautioned, on the basis of the limited number of small-shaft screw piles examined under the current investigation.

One significant over prediction of 110 percent occurred using the torque method for test pile T5, installed in sand and loaded in uplift under the shallow failure condition. It should be noted that he use of a single K_t factor within the empirical torque relationship does not capture the difference between the screw pile capacity in uplift as opposed to compression. This discrepancy is especially significant for screw piles loaded in uplift under the shallow failure condition, as the bearing capacity above a shallow buried plate in uplift will be significantly less than above a deep plate in uplift or below a buried plate in compression.

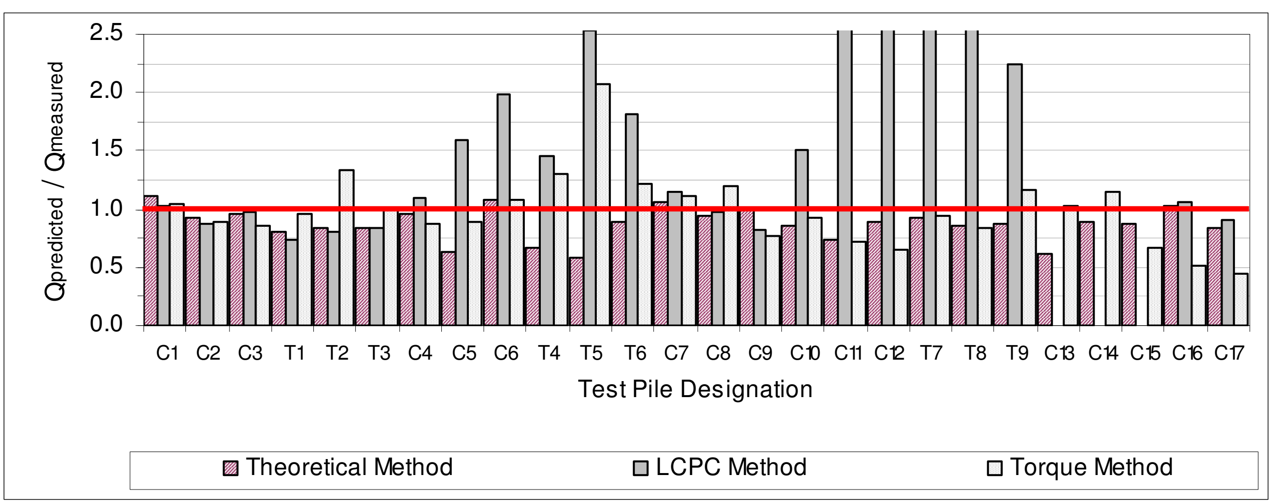

Figure 4: Ratio of predicted to measured ultimate axial screw pile capacities

7 CONCLUSION

The axial capacities of 26 full-scale screw piles installed in Alberta and British Columbia since 1998 were interpreted on the basis of static load test results. The effectiveness of three design methods were evaluated for predicting the ultimate axial capacity of the test piles, installed in cohesive and cohensionless soils. The failure surfaces were approximated by the cylindrical shear model for multi-helix screw piles having an inter-helix spacing ratio (S/D) less than 3.0, and by the individual plate bearing model for single-helix screw piles and multi-helix piles with S/D ≥ 3.0.

Good predictions of the ultimate screw pile capacities in uplift and compression were obtained using theoretical formulations for appropriate components of bearing capacity and friction. Most of the capacity predictions made by the theoretical method fell within 20 percent of the measured capacities, with a small number of predictions outside the 20 percent range erring on the side of conservatism. Capacity predictions based on the LCPC direct pile design method, using the results of static cone penetration tests, were good for screw piles under uplift and compression in clay, but significant deviations from the measured capacities occurred for piles installed in sand and glacial till materials. A direct empirical relationship between the installation torque and the ultimate axial screw pile capacity was applied using a K_t factor of 9.2 m^{-1} derived by linear regression of the data set. Capacity predictions made using the torque method typically fell within 30 percent of the measured capacities for deep screw piles in uplift and compression. The torque relationship significantly over-predicted the uplift capacity of a shallow screw pile installed in sand.

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