Field behaviour of screw micropiles subject to axial loading in cohesive soils

<table>
<thead>
<tr>
<th>Journal:</th>
<th>Canadian Geotechnical Journal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manuscript ID</td>
<td>cgj-2017-0109.R1</td>
</tr>
<tr>
<td>Manuscript Type:</td>
<td>Article</td>
</tr>
<tr>
<td>Date Submitted by the Author:</td>
<td>30-Apr-2017</td>
</tr>
<tr>
<td>Complete List of Authors:</td>
<td>Guo, Zhengyang; University of Alberta, Civil and Environmental Engineering Deng, Lijun; University of Alberta, Department of Civil and Environmental Engineering</td>
</tr>
<tr>
<td>Keyword:</td>
<td>field test, tapered, installation torque, screw micropile, axial behavior</td>
</tr>
</tbody>
</table>
Field behaviour of screw micropiles subject to axial loading in cohesive soils

Zhengyang (Moira) Guo¹ and Lijun Deng²

Corresponding Author:

Lijun Deng

Department of Civil & Environmental Engineering

University of Alberta

Edmonton, Canada T6G 1H9

Phone: (780) 492 – 6210

Email: ldeng@ualberta.ca

¹ MSc student, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada, T6G 1H9

² Assistant Professor, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada, T6G 1H9, ldeng@ualberta.ca, 1-780-4926210
ABSTRACT

Field tests of full-scale screw micropiles with a diameter varying from 76 to 114 mm and a length varying from 1.6 to 3 m were undertaken to investigate the axial pile capacities, load-transfer mechanism, and end installation torques of the piles in cohesive soils. Forty tests were performed on piles subjected to axial compressive and tensile loads. Six tests were instrumented with strain gauges on the pile shaft. Results showed the piles reached the limit state before the displacement exceeded 10% of the shaft diameter. The majority of axial load was transferred to the threaded segment. The adhesion coefficient of the top smooth shaft at the limit state was less than 0.1. The failure mode along the cylindrical threaded shaft was cylindrical shearing along the edge of the threads; the threads increased the axial capacities of the segment. Axial capacities of the threaded tapered segment were 43% on average greater than that of a cylindrical segment with the equivalent volume. Compressive capacities of all test piles were estimated and the results agreed reasonably well with the measured capacities. A theoretical torque model was proposed to estimate the end installation torques based on the cone penetration test results; the theoretical results matched the measured end torques very well.

Key words: field test, screw micropile, tapered, axial behaviour, installation torque
INTRODUCTION

Micropiles are piles with small diameters, normally ranging between 100 mm and 300 mm. Micropiles were introduced to underpin historic buildings damaged during World War II in Italy (Bruce 1988). Applications of micropiles include supporting light axial and lateral structural loads, replacing or underpinning existing foundations, reinforcing embankment, slope and landslide, and seismic retrofitting. Current practice of micropile installation includes the following steps: drilling to a targeted depth, placing steel reinforcement, and placing or pressurizing the grout (FHWA 2005). Micropiles have advantages when there are difficult ground conditions such as limited access to construction sites due to natural or manmade obstruction. Another advantage is that the pile capacity is often attainable by full-scale loading tests due to the relatively small size of the pile (Coduto 2001).

Screw micropiles, a new foundation type, have been recently introduced to the foundation construction industry of North America. This pile type consists of the smooth cylindrical segment on the top, threaded cylindrical segment in the middle, and threaded tapered segment at the bottom. Unlike conventional micropiles, this pile type is made of a steel tubular shaft with continuous spiral threads. This pile type is screwed into the ground, which minimizes the ground heave and fully displaces the soil to maximize the lateral earth pressure on the shaft.

Although the screw micropile may potentially be useful in many applications, research on the axial behaviour of this pile type is currently unavailable. The geotechnical design of this pile type largely depends on site-specific load tests, which are costly although attainable. There is no acceptable design guideline for this pile type or similar in the literature. Current literature suggests that pile shape (tapered vs. straight shaft, Wei and El Naggar 1998), shaft roughness (smooth vs. threaded shaft, Cuthbertson-Black 2001), and installation methods (drilled, driven,
or screwed-in, Salgado 2008) have significant impacts on the pile axial behaviour. Due to the unique characteristics, screw micropiles may perform differently from conventional piles.

The closed-end tapered segment of screw micropile is similar to the configuration of a tapered pile. The axial behaviour of tapered piles was characterized by a limited number of researchers. Rybnikov (1990) conducted field tests of bored cast-in-place tapered piles in sandy soil and found that the specific capacity of a tapered pile was 30% greater than that of a cylindrical pile, where the specific pile capacity is defined as the axial capacity per unit shaft volume. Kodikara and Moore (1993) developed a theoretical model for piles with taper angles of 1° to 5° in cohesive soils, and concluded that the shaft resistance of tapered piles is substantially greater than that of cylindrical piles of equal shaft volume. El Naggar and Sakr (2000) investigated the effects of taper on the axial compressive capacity and proposed an analytical approach for the design of tapered piles in sand. Despite these, however, there is a lack of field experimental research in the capacities of the threaded, tapered piles that are installed into cohesive soils by torsion.

Hence, the first objective of the present research is to investigate the load-transfer mechanisms along various segments of micropiles subject to axial compressive loadings and to develop a feasible method of predicting the axial capacities of the piles.

The installation method of screw micropile is similar to that of helical piles. The installation torque is of primary importance in the evaluation of axial capacities of helical piles in the design practice. Sakr (2015) proposed a theoretical torque model to estimate the installation torque of round-shaft helical piles in cohesionless soils. Empirical torque–capacity correlation is being used for design purpose (e.g., Hoyt and Clemence 1989; CFEM 2006; Ghaly and Hanna 1991). Since helical piles and the screw micropiles are both screwed into the ground, the installation
torque may also be useful in predicting the axial capacities of micropiles and an understanding of the torsion resistance during micropile installation is vitally important. Therefore, the second objective is to develop a theoretical torque model based on in-situ soil properties and pile characteristics and to verify the model using measured installation torques.

To fulfill the preceding objectives, field load tests were carried out to characterize the performance of screw micropiles under mainly axial compressive loads. Full-scale screw micropiles of six sizes were tested at a cohesive soil site located in Edmonton, Canada. Site investigation consisting of cone penetration tests (CPT), Shelby tube sampling, and laboratory tests of intact soil samples was carried out to obtain the profiles of soil strength and physical properties. At least three compression and three tension tests were conducted on the same pile type. Axial loading, displacement, and the installation torque were recorded. Four research piles were instrumented with strain gauges at five stations and the results are presented in detail. Although axial tension tests were conducted, the present paper will focus on the axial compression performance of piles based upon attained high-quality strain gauge records.

Based on the results of instrumented piles, the adhesion coefficient along the smooth segment was determined, the failure mechanisms of the threaded cylindrical shaft were characterized, and effects of the tapered tip on the axial capacities were quantified. The axial capacities at the limit state of all test piles were estimated and compared with the field-measured capacities. A theoretical torque model was proposed using the remolded soil strength based on the sleeve friction from CPT soundings; the model was validated by the measured end installation torques of all test piles.
PILE DESCRIPTION

Screw micropiles (shown in Figure 1) in the present research consist of a hollow tube with continuous spiral threads welded on the lower shaft. This pile type is manufactured in different lengths and diameters, commonly used sizes range from 66 mm to 140 mm in diameter and 0.8 m to 5 m in length. This pile type is made of structural steel coated with galvanization for corrosion resistance. This pile type is installed under torsion. Screw micropiles may have the following advantages over conventional piles. They are quick to install, easy to dismantle, reusable, and immediately loadable. No earth work is required prior to installation and the terrain remains undamaged after the piles have been removed. Because of the relatively small size and the installation method, the equipment required for installation is small and can be useful where the access is limited.

In present research, six types of screw micropiles were tested under axial compression and tension. Test pile types varied in length \((L)\), diameter \((D)\), and the tip shape. The configuration of each pile type is summarized in Table 1 and shown in Figure 1. The spacing between threads \((s_{th})\) of all test piles is 50 mm. The thread width \((w_{th})\) and thickness \((t_{th})\) of all test piles are 12 mm and 2 mm, respectively. The thickness of the smooth shaft of all test piles is 3.6 mm.

Due to the tapered shape at the tip of the piles, the shaft diameter changes within the tapered segment. The diameters in Table 1 are referring to the outer shaft diameter of the smooth shaft. Table 1 also lists the average taper angle \((\theta)\) of the closed-end tapered segment, as shown in Figure 1. The pile shaft and threads are made of structural steel having a Young’s modulus of 210 GPa and a yield strength of 248 MPa. The pile material conforms to German standards DIN EN 10219-1 (DIN 2006a) and DIN EN 10219-2 (DIN 2006b).

SITE INVESTIGATION
A cohesive soil site is selected for the field tests of full-scale piles. Figure 2 shows the site located at the University of Alberta South Campus Farm, Edmonton, Canada. This site has been traditionally used for field tests of piles because it represents typical soil profiles observed in greater Edmonton area. The soils at this site are mainly glaciolacustrine sediments as part of the Glacial Lake Edmonton deposits formed about 8,000 year ago (Edwards 1993). Site investigation includes CPT, Shelby tube borehole sampling beside CPT locations, and laboratory tests of intact Shelby tube samples. Five CPTs were conducted on the test site as shown in Figure 3. Two Shelby tube sampling boreholes (BH-3 and BH-4) of 5.1 cm diameter were drilled and soil samples were obtained between 0.6 m and 2.7 m below ground surface.

CPT-0 through CPT-2 were conducted in the summer of 2014, and CPT-3 and CPT-4 in the summer of 2016 when the field tests were carried out. Results of CPT-3 and CPT-4 dissipation tests showed that the groundwater table was 5.3 m below ground surface, which is deeper than the layer of interest. As shown in Figure 4, CPT results suggest that the site is composed of approximately 1 m of silty organic clay underlain by a highly-plastic, stiff clay layer of 4.5 m thickness.

The CPT results were used to estimate the undrained shear strength $s_u$ of in-situ soil (as shown in Figure 4c) using Equation 1 (Robertson and Cabal 2012):

$$s_u = \frac{q_t - \sigma_v}{N_{kt}}$$

where $q_t$ is the corrected cone resistance, $\sigma_v$ is the total vertical stress, and $N_{kt}$ is the site-specific cone factor. In the present research, $N_{kt}$ of 14 is adopted based on previous experience with the test site (Tappenden 2007; Zhang 1999); this selection is confirmed by the laboratory strength tests. The $s_u$ of intact Shelby tube samples was measured using the unconfined compressive strength (UCS) tests and vane shear tests. Lab-measured and estimated $s_u$ are compared in
Figure 4c. It appears that the lab-measured $s_u$ was fairly consistent with depth, with values ranging between 50 kPa and 110 kPa. The $s_u$ estimated from CPT results was overestimated in the upper, approximately 1 m thick soil; this discrepancy is likely due to the high cone resistance of the desiccated, organic crust. For soils below 1 m depth, $s_u$ estimated from CPT was consistent with the lab-measured values. At 2.5 m, $s_u$ converges to an average value of approximately 100 kPa. Because the sleeve resistance $f_s$ is a reasonable estimation of the remoulded undrained shear strength (Robertson and Cabal 2012), the sensitivity of the clay can be determined by the ratio of $s_u$ to $f_s$. Results shows that the sensitivity ranges between 1 and 2, which implies that the clay has a low sensitivity (Skempton and Northey 1952) and which is compatible with the geological history of the South Campus Farm site (Bayrock and Hughes 1962; Edwards 1993).

Figure 5 shows the moisture contents and Atterberg limits of samples at BH-3 and BH-4. The in-situ moisture content was close to or less than the plastic limit. Figure 6 shows that the soils are classified as CH. Void ratio of the intact soil samples ranged from 0.7 to 1.1 and the degree of saturation ranged from 80% to 95%.

**FIELD TEST PROGRAM**

Carried out in the summer of 2016, the field test program consists of 22 axial compression and 18 tension tests on 6 pile types. A minimum of three compression and three tension tests were carried out on each pile type. The piles P1, P3, P5, and P6 were instrumented with strain gauges that were used to estimate the axial forces. The field test program is described below.

**Loading frame**

A loading frame was developed following ASTM D1143 (ASTM 2007a) and D3689 (ASTM 2007b). The spacing between any two piles (either test or reaction piles) was kept at least five
times the greater shaft diameter to minimize the pile-to-pile interaction. The spacing was less than 2.5 m, which is the minimum recommendation of ASTM (2007a, 2007b); however, given the small diameter of the present micropiles, it is not crucial to keep a minimum of 2.5 m spacing. Figure 7 shows the reaction system consisting of a reaction beam and two reaction pile groups on the ends of the beam and connectors. The reaction beam of 4.2 m length is a wide-flange W360 × 179 I-beam, which is strengthened with two additional 7.9-mm-thick steel plate stiffeners (Figure 7) that are welded to flanges to increase the bending and torsional rigidity of the reaction beam. The reaction beam can accommodate two loading tests at one setup station, as shown in Figure 7a. For each reaction pile group, there were four identical 2.1 m long screw piles with a diameter of 0.14 m; the reaction pile groups were sufficiently stiff such that there was no noticeable axial displacement to groups during the loading tests. The beam was connected to the reaction pile groups by steel plates and threaded steel bars. The load was supplied by a two-way hydraulic jack connected to the load cell and fixed on the reaction beam. The two-way hydraulic jack allowed the loading frame to serve for both compressive and tensile loading conditions using the same configuration. The loading frame was designed with an overall factor of safety of 2.5, if the allowable load of 400 kN is applied in the middle of the reaction beam.

Test procedure

Test procedure conformed to ASTM D1143 (ASTM 2007a) for axial compressive load test and ASTM D3689 (ASTM 2007b) for axial tensile load test. Procedure A: Quick Test in the ASTM standard was adopted. The capacity of the test pile was estimated based on the previous load tests of similar pile types. The load was applied at an increment of 5% of the anticipated pile capacity at plunging failure; if the pile did not fail at the anticipated capacity, the load was
applied in the same increment until failure. During each load interval, the load was maintained constant for 5 min. After reaching failure, the pile was unloaded in four equal decrements. In the unloading stage, the load cell reading was normally very stable, so the time interval was revised from 5 min to 2 min. Pile setup time, which is the waiting period between the installation and the testing, was at least 48 hours for all piles.

**Instrumentation**

For all the tests, two linear potentiometers (LP) of a maximum stroke of 150 mm were used and the average of the two readings was taken as the displacement of the pile head. We used a cylindrical load cell (model 1240 manufactured by Interface) that has a capacity of 900 kN and an accuracy of 0.07%; the load cell can measure both tensile and compressive load. The installation torque was recorded at the last cycle of the torque head rotation using a pressure differential gauge integrated into the installation equipment. All instruments were calibrated prior to the field tests.

Six pile load tests were instrumented with strain gauges. Strain gauges consisting of a full-bridge Wheatstone circuit were applied onto the external pile shaft to measure the internal axial loads. The strain gauges were manufactured by Vishay Precision Group with the model number CEA-06-250UT-350 and can measure a strain range of ±5%. Five stations were located on the instrumented piles: one beneath the pile cap, one above the threads, one on the tip and the other two between the threads. Figure 1 shows the locations of the strain gauge stations and Table 1 lists the spacing between adjacent stations for all instrumented piles. Protective coatings were used to prevent the wires and gauges from being torn off during pile installation. Three layers of coating were applied: a layer of epoxy, a thin layer of rubber coating, and a thick layer of aluminum foil tape. The edge of the aluminum foil tape was sealed with rubber coating to
prevent soil and water from entering the circuit. Lastly, the stations were wrapped by plastic tapes and a steel sheet cover to protect the gauges from physical damage. The steel sheets of 0.79 mm thickness was wrapped around gauges and were welded onto the pile shaft. Despite the protective measures, several strain gauges were still damaged during the pile installation or uninstallation and therefore only the results of instrumented piles with complete strain gauge records are presented and discussed.

RESULTS AND DISCUSSION

After tests, uninstalled piles were examined thoroughly to check the integrity of structural steel; no visible damage was observed on the pile shaft or threads. Figure 3 shows the layout of all test piles and the locations of instrumented piles P1 (P1-C4 and P1-C5), P3 (P3-C3), P5 (P5-C4 and P5-C5), and P6 (P6-C1), of which the results are presented and discussed. The pile test ID (e.g. P1-C4) implies the pile type, test type (C for compression), and the repetition number.

Load vs. normalized displacement curve

Figure 8 shows the load \((Q)\) vs. normalized displacement \((w/D)\) curve, where \(w\) is the axial displacement and \(D\) is the smooth shaft diameter, for selected tests of each pile type subject to axial compression and tension. The selected tests were conducted in the western region of the test site (Figure 3) where the soil characteristics have been very similar according to CPT-0, CPT-1, and CPT-2 soundings (Figure 4). As such, it is substantiated to compare the \(Q\) vs. \(w/D\) curves of the selected tests.

The pile capacity at the limit state, \(Q_L\), is defined as the maximum load a test pile can sustain without reaching the plunging failure (Salgado 2008). Figure 8a shows that \(Q_L\) of P1, P3, and P5 in compression were 158, 138, and 94.3 kN, respectively, and \(Q_L\) of P2, P4, and P6 were 39.3, 36.1, and 23.6 kN, respectively; it appears that \(Q_L\) of longer piles are 3.8 to 4.0 times \(Q_L\) of
shorter piles of the same \(D\). Figure 8b shows that \(Q_L\) of P1, P3, and P5 in tension were 130, 90, and 89.1 kN, respectively, and \(Q_L\) of P2, P4, and P6 were 31.4, 13.6, and 25.6 kN, respectively; it appears that \(Q_L\) of longer piles are 3.5 to 6.6 times \(Q_L\) of shorter piles of the same \(D\). Most piles reached the limit state before \(w/D\) reached 0.1; this means that for the present micropiles installed in cohesive soils, the limit state capacity will still govern the axial design, as would be anticipated for other piles in cohesive soils (Salgado 2008). A softening behaviour was observed after the piles reached the limit state, when the resistance dropped with an increasing displacement. The softening behaviour can be explained by the fact that the soil is lightly overconsolidated and that the post-failure remolded \(s_u\) is less than the peak \(s_u\). The CPT \(f_s\) and \(s_u\) profiles (Figure 4) confirm that the remolded \(s_u\) is less than the peak \(s_u\).

The load-transfer and failure mechanisms along each segment during axial compression tests are quantified in detail in subsequent subsections.

**Axial load distribution**

For instrumented piles, the internal shaft load \(Q\) at a strain gauge station is calculated using Equation 2:

\[
Q = \varepsilon EA
\]  

(2)

where \(\varepsilon\) is the measured strain, \(E\) is the Young’s modulus of the steel, and \(A\) is the cross-sectional area at the gauge station. In the tapered segment, \(A\) is revised to account for the change in wall thickness.

Figure 9 shows the load distribution of piles P1 and P5 at different stages of axial loading, while the last curve represents the load distribution at the limit state. The load distribution curves show that the smooth shaft resistance was almost negligible and the majority of axial load was carried by the threaded segment. The internal force is close to zero at the pile tip, which implies
that the screw micropiles behave as a friction pile. Coduto (2001) also suggested that for micropiles, due to the small diameter, the toe bearing capacity is normally neglected.

These instrumented piles are divided into three segments: smooth segment (between SG-1 and SG-2), straight threaded segment (between SG-2 and SG-4), and tapered threaded segment (between SG-4 and SG-5). Pile P1 has a unique shape where the diameter of the shaft changes twice along the length; strain gauges were only installed at the top and bottom of the first tapered segment, as a result the load transfer within the second tapered segment was not captured. For P1, the smooth segment is between SG-1 and SG-2, the straight threaded segment is between SG-2 and SG-3, and the tapered segment is between SG-3 and SG-4; the purpose of SG-5 was to measure the end bearing at the pile tip. In order to understand the load transferred to the soil at various segments at the limit state, the unit shaft resistance, $q_{sl}$, is calculated using Equation 3:

$$q_{sl} = \frac{Q_{Li} - Q_{Lj}}{S_{ij}}$$

where $q_{sl}$ is the unit shaft resistance between Station $i$ and Station $j$, $Q_{Li}$ is the measured internal load at the limit state at Station $i$, and $S_{ij}$ is the surface area of the shaft segment between Station $i$ and Station $j$. Note that for threaded segments, the thread width was accounted for when calculating the surface area of the pile.

Figure 9 shows the calculated $q_{sl}$ distribution of test piles for which a complete set of strain gauge results were obtained. It is observed that $q_{sl}$ is the least along the smooth segment and the greatest in the tapered shaft. Figure 9 suggests that the presence of threads have increased $q_{sl}$ by comparing the load transfer of the smooth shaft to the threaded segment. In the tapered segments, the unit shaft resistance is much greater than that of the straight threaded segment with the same diameter, which is apparently due to the tapered shape.
Load transferred to smooth shaft

Load transferred to the smooth segment, $Q_{sm}$, can be calculated using Equation 4:

$$Q_{sm} = \alpha s_u \pi D L_{sm}$$  \hspace{1cm} (4)

where $\alpha$ is the adhesion coefficient between soil and smooth shaft, $L_{sm}$ is the length of the smooth segment. The term $\pi DL_{sm}$ represents the contact area between the smooth segment and surrounding soil.

To determine $\alpha$ at the limit state, measured $q_{sl}$ for three compression tests of piles with a diameter of 76 mm was compared with the average $s_u$ over the span of smooth segment, as shown in Figure 10. A linear trend line shows a slope of 0.0926, which equals $\alpha$ at the limit state. This $\alpha$ is substantially less than the adhesion coefficients suggested in literature: Randolph and Murphy (1985) suggested a lowest $\alpha$ of 0.73 or driven piles in clay having a strength ratio $(s_u/\sigma_v')$ greater than 1, and Tomlinson (1957) recommended an adhesion coefficient of 0.58 for driven piles in clayey soils. Due to the torsional installation and the fact that the diameter of smooth segment is less than the external diameter ($D$ plus $2w_{th}$) of threaded segment, the soil surrounding the smooth shaft might have been expanded and disturbed during the installation, leaving a annular cavity between the pile and soil, thus causing a very small $\alpha$ value. This annular cavity was often visible after pile installation. Because the soil and pile were not in a firm contact, adhesion along the smooth shaft was not sufficiently mobilized. In conclusion, the smooth shaft is not a key contributor to the capacity of micropiles in cohesive soils.

Load transferred to cylindrical threaded shaft

In order to predict the limit shaft resistance of the threaded segment, the soil failure mode shall be determined. At the limit state, it is hypothesized that two failure modes may be applicable to the straight threaded shaft. Figure 11a and b show the possible cylindrical shear mode (CSM)
and individual bearing mode (IBM) for soil failure. Because the pile load test is in accordance with ASTM D1143 (ASTM 2007a) Quick Test, there is not enough time for excess pore pressure dissipation when the failure occurs; therefore, the soil is considered in undrained condition and the soil resistance is calculated using $s_u$.

For the CSM shown in Figure 11a, the soil between the threads and pile is assumed to behave as one complete unit, and the shear plane is a cylindrical surface on the outer edge of the threads. When the pile is displaced, the resistance arises from the $s_u$ of the shaft-soil cylinder. The limit load transferred to the straight threaded segment under the CSM hypothesis, $Q_{th}$, was estimated using Equation 5:

$$Q_{th} = (s_u)_{avg} \pi (D + 2w_{th})L_{th}$$  \hspace{1cm} (5)

where $(s_u)_{avg}$ is the CPT-based average undrained shear strength along the threaded segment, $D$ is the smooth shaft diameter, $w_{th}$ is the thread width (11 mm), and $L_{th}$ is the length of the threaded segment. The term $\pi (D + 2w_{th})L_{th}$ is essentially the contact area between the soil and pile shaft that considers the thread width. For piles P3, P5 and P6, as shown in Figure 1 and Table 1, $L_{th}$ equals $(L_2 + L_3)$, and for P1, $L_{th}$ equals $L_2$.

For the IBM shown in Figure 11b, each thread is considered an individual bearing plate, and the smooth shaft between two threads provides upward shaft resistance. The limit load transferred to the threaded segment $Q_{th}$ under the IBM hypothesis was estimated using Equation 6a:

$$Q_{th} = \sum_{i=1}^{m} (N_i (s_u)_i + (\sigma_v)_i)\pi (D + w_{th})w_{th} + \alpha (s_u)_{avg} \pi DL_{th}$$ \hspace{1cm} (6a)

where $m$ is the number of threads within the threaded segment, $N_i$ is the bearing capacity coefficient that equals 9 in present study (CFEM 2006; Salgado 2008), $s_u$ and $\sigma_v$ are the CPT-
based undrained shear strength and total stress at the depth of individual threads. Equation 6a considers the annular threads as individual deeply-embedded strip footings. The term

\[ \pi(D + w_{th})w_{th} \]

is the projected area of each thread and the term \( \alpha(s_u)_{avg} \pi DL_{th} \) is the adhesion around the pile shaft. Because the adhesion coefficient is very small as observed in preceding subsection, the contribution of the shaft resistance is insignificant. Hence, Equation 6a could be expressed as:

\[
Q_{th} = \sum_{i=1}^{m} \left( N_i (s_u)_i + (\sigma_v)_i \right) \pi (D + w_{th})w_{th} \quad (6b)
\]

where the resistance of the smooth shaft within the cylindrical threaded segment is neglected.

Figure 12 shows the estimated \( Q_{th} \) based on two possible failure modes vs. the measured load transferred to threaded shaft for six instrumented piles in compression tests, following Equations 5 and 6b respectively. It appears that the CSM assumption, although not perfect, better predicts the axial load in the threaded shaft than the IBM assumption. The IBM assumption overestimates \( Q_{th} \) by about 200%, even though the pile shaft adhesion has been neglected. For all test piles, the ratio of thread spacing \( s_{th} \) to thread width \( w_{th} \) is 50/12 or 4.2. The literature on helical piles (Narasimha Rao et al. 1991; Elsherbiny and El Naggar 2013) suggested that the ratio has a significant effect on the failure mode; smaller ratio leads to CSM for helical piles. Hence it is concluded that the ratio of 4.2 of present test piles is small enough to mobilize the CSM in present stiff cohesive soils. In conclusion, the failure mechanism along the threaded segment is cylindrical shearing of the surrounding soil, as suggested in Figure 11a. The presence of threads pushes the shear failure surface outward to the edge of the threads, increases the “adhesion coefficient” to 1 and thereby enhances the pile capacities.
Load transferred to tapered threaded shaft

When studying the compressive capacities of tapered piles, one approach is to normalize the pile capacity by the pile shaft volume (termed the specific capacity). Rybnikov (1990) found that the specific capacity of a tapered pile in sand was 1.3 times that of a cylindrical pile. Wei and El Naggar (1998) showed that under different confining stresses, the specific shaft resistance of a tapered pile in sand was 1.13 to 1.65 times that of a cylindrical pile.

The present study investigates the effects of the tapered shape on pile capacities. We begin with comparing the measured limit axial load of the tapered shafts with the estimated axial load of a cylindrical shaft of equal volume. If we assume an equivalent, cylindrical shaft that has the same shaft volume as the tapered shaft, then we can adopt the CSM for the equivalent shaft based on the preceding observation on cylindrical threaded shafts. The limit load transferred to the equivalent shaft \( Q_{tp_{\text{eq}}} \) can be calculated using Equation 7:

\[
Q_{tp_{\text{eq}}} = (s_u)_{avg} \pi (D_{avg} + 2w_h)L_{tp}
\]

where \( D_{avg} \) is the average shaft diameter of the tapered shaft, \( L_{tp} \) is the total length of the tapered segment, and \((s_u)_{avg}\) is the average \( s_u \) of the soil surrounding the tapered segment obtained from CPT strength profiles. The term \((D_{avg} + 2w_h)\pi L_{tp}\) is essentially the slip surface area of the equivalent shaft. For piles P5 and P6, \( L_{tp} \) equals \( L_4 \), and for P1, \( L_{tp} \) equals \( L_3 \). Refer to Figure 1 and Table 1 for pile dimensions.

Figure 13 shows the estimated \( Q_{tp_{\text{eq}}} \) versus the measured \( Q_{tp} \) for four compression tests of two instrumented piles for which strain gauge records were attained at the segment. From the limited number of available data points, it appears that the measure \( Q_{tp} \) is greater than \( Q_{tp_{\text{eq}}} \) by 3 to 77%. A linear trend line is plotted to show an empirical relationship between measured and estimated values. It is observed that for the four tests the actual load transferred to the tapered
segment is greater than $Q_{\text{tp, eq}}$ by 43% on average. The average taper angle of 5.2° and 5.5° was fairly large and thus may have increased the axial capacities significantly. As discussed in preceding research (e.g., Wei and El Naggar 1998), the reason for the capacity enhancement is that additional lateral pressure from the soil was induced and resulted in an increase in shear stresses across the surface of tapered segment when the taper expands the adjacent soils.

**Test pile axial capacities: measured vs. estimated**

Now that the failure mechanisms of each segment have been preliminarily established from instrumented piles, the capacity of non-instrumented piles at the limit state, $Q_L$, can be estimated using Equation 8 in order to verify the preceding assumptions:

$$Q_L = Q_{\text{sm}} + Q_{\text{th}} + 1.43(Q_{\text{tp, eq}})$$

(8)

where $Q_{\text{sm}}$, $Q_{\text{th}}$, and $Q_{\text{tp, eq}}$ are obtained from Equations 4, 5, and 7, respectively. For Equation 4, adhesion coefficient $\alpha$ equals 0.09 for the soil-pile interface as previously discussed. The coefficient of 1.43 in the $Q_{\text{tp, eq}}$ term is empirically obtained from Figure 13. Average $s_u$ along each segment was taken from the CPT strength profiles.

Figure 14a shows the estimated and measured limit capacities for all pile types. To account for the site heterogeneity when estimating $Q_{L,e}$, we considered all of the CPT strength profiles as shown in Figure 4 and then obtained the maximum, minimum, and average values. $Q_{L,e}$ is shown in Figure 14a as error bars. It is seen that the majority of the measured capacities fall within or close to the estimated range except for two data points from P5. From Figure 14a, it is observed that longer piles appeared to exhibit a wider variation in the estimated pile capacity. For instance, the minimum $Q_{L,e}$ for P1 is about 80 kN while the maximum is about 150 kN; this is because the pile length, especially threaded length, plays a central role in contributing to the shaft resistance. The effect of heterogeneity of the soil on the pile capacity is amplified by the threaded length.
Figure 14b shows the average $Q_{Le}$ versus the average $Q_{Lm}$ for all six piles and the linear-fit trend line. It is observed that the average $Q_{Le}$ is about 12% less than the average $Q_{Lm}$. The reason for underestimation could be due to underestimated $\alpha$, $Q_{tp, eq}$, or $s_u$ because of the limited number of CPT soundings. Given the complexity of field tests and the heterogeneity of soils, however, a discrepancy of 12% on average is considered acceptable when estimating the pile axial capacities. As noted in Randolph (2003), “we may never be able to estimate the axial capacity in many soil types more accurately than about ±30%.” Hence, it is reasonable to conclude that $Q_{Le}$ based on Equation 8 is a good approximation to the measured $Q_{Lm}$ and the failure mechanisms proposed in preceding subsections are valid.

**End installation torque: measured vs. estimated**

End installation torque was recorded at the last cycle of rotation during the installation process. At the last cycle of rotation, the pile does not advance downward but rather rotates in-place. At this moment, the soil surrounding the shaft has experienced large shear strain and is at the remolded state. Note that the remolded soil strength was not appropriate for the pile capacity calculation using Equation 6 because of the soil setup. Although the soil is being sheared in the horizontal tangential direction, the soil resistance to pile rotation is considered similar to the sleeve friction recorded in the CPT soundings, because the soil is at the remolded state (refer to Robertson and Cabal for soil’s remolded shear strength). Therefore, a theoretical torque model based on CPT sleeve frictions is proposed and examined using the measured end torques.

Figure 11c illustrates the proposed torque model, where the applied torque is counterbalanced by the torque produced by sleeve friction $f_s$ acting on the pile shaft and the threads. The end installation torque, $T$, is estimated using Equation 9:

$$T = 2 \sum_{i=1}^{m} \left( f_s \right)_i \pi \left( \frac{D_i + w_{th}}{2} \right)^2 w_{th} + f_s (L - L_t) \pi \frac{D^2}{2}$$  \hspace{1cm} (9)
where $f_s$ is the sleeve friction measured from CPT, $D$ is the diameter of smooth shaft, $L$ is the total length of pile, $L_1$ is the length of smooth shaft, $m$ is the total number of threads, $D_i$ is the smooth shaft diameter at each thread location. Equation 9 assumes that the soil shear resistance at the soil-pile interface is equal to the remolded shear strength, which is equal to the sleeve friction $f_s$ according to the CPT guide (Robertson and Cabal 2012). Equation 9 also assumes that the total torque is the sum of the torque produced by the shear resistance on the top and bottom surfaces of each thread (i.e., the first term in Equation 9) and the torque produced around the smooth shaft between threads (i.e., the second term in Equation 9). Because the upper smooth shaft of the pile have shown not be in a firm contact with the soil and have a very low adhesion coefficient, the torque from the smooth shaft adhesion is neglected in Equation 9.

To account for the effect of the site heterogeneity on installation torques, $f_s$ from all five CPT soundings were used when estimating the end torques using Equation 9. Figure 15a shows the average, maximum, and minimum estimated torques $T_e$ for each pile type and six measured torques $T_m$ (from both compression and tension tests) for each pile. The effect of site heterogeneity is implied in Figure 15a as the maximum and minimum error bars. It is seen that the majority of measured $T_m$ fall within or close to the estimated ranges, despite several outliers for the pile type P2. The estimated and measured torques are averaged and compared as shown in Figure 15b. It appears that the average $T_e$ shows a good linear correlation with the average $T_m$. The trend line shows that the average $T_e$ is only 2% lower than average $T_m$. The results shown in Figure 15b validates the proposed torque model based on the CPT $f_s$ profiles.

The significance of the torque model is that it correlates the soil strength parameter $f_s$ to the installation torques. Once the installation torque is obtained, one can back-calculate $f_s$ from
Equation 9 and therefore estimate, with the proper soil sensitivity, the undrained shear strength at the pile location; finally, the limit pile capacity can be predicted.

CONCLUSIONS

Forty full-scale load tests were conducted on six types of screw micropiles in the field at a cohesive soil site. The micropiles consist of a hollow steel shaft with threads welded onto the lower segment and a tapered pile tip. The diameter of test piles ranges from 76 mm to 114 mm, and the length ranges from 1.6 m to 3 m. Comprehensive site investigation was carried out and six pile tests were instrumented with strain gauges to investigate the load transfer mechanisms.

Following conclusions may be drawn:

1. For screw micropiles, the adhesion coefficient between the smooth shaft and soil is less than 0.1 on average, because the torsional installation process expands the soils and creates annular gaps between pile shaft and surface soils. The small adhesion coefficient means that the axial resistance of the smooth shaft could be neglected when the pile capacity is estimated.

2. For screw micropiles, the majority of the axial load was carried by the straight threaded and the tapered threaded segment. Soils around the straight threaded segment fail in the cylindrical shear mode at the limit state. The ratio of thread spacing to thread width is 4.2 for all test piles. The presence of threads pushes the shear failure plane outward to the edge of the threads and thereby substantially increases the limit capacity of the threaded segment. The estimated limit capacities of the threaded segment based on the CSM assumption well predicted the measured capacities of instrumented piles.

3. Taper shape increases the shaft resistance. For the instrumented test piles with 5.2° and 5.5° taper angle at the tip, the limit shaft resistance of the tapered segment, is 43% on average greater than that of a cylindrical, equivalent straight shaft with the same shaft volume.
4. Limit capacities of all test piles under compression were estimated using the proposed adhesion coefficient, failure mechanism, and empirical relationship between the tapered segment capacity and equivalent straight segment capacity. The estimated limit capacities of piles were 12% on average less than the measured capacities. Given the complexity of field tests and soil heterogeneity, the discrepancy may be considered acceptable.

5. A theoretical torque model was proposed to estimate the end installation torque at the last cycle of rotation. The torque model considers the torque induced by the remolded shear strength acting on the shaft and the threads below the smooth segment. The torque model adopts the CPT-based sleeve friction and accounts for the site heterogeneity. The estimated torques of all test piles were 2% on average less than the measured torques, which confirms the validity of the proposed torque model.

ACKNOWLEDGMENT

The research is funded by Natural Sciences and Engineering Research Council of Canada (NSERC) under the Collaborative R&D program (CRDPJ 469600-14). The authors are also grateful to Krinner Canada Inc. for the financial and technical support. The first author appreciates the support of NSERC–Industrial Postgraduate Scholarship. We are grateful to Benoit Trudeau, Dennis Lechot, and William Carney of Workonthat Structure Inc. for the assistance in performing the pile load tests, and Harra Vardeleon, Mujtaba Khidri, and Chunhui Liu, students at the University of Alberta, in helping with the field investigation and laboratory soil tests.
REFERENCES


German Institute for Standardization (DIN). 2006a. DIN EN 10219-1, Cold formed welded structural hollow sections of non-alloy and fine grain steels-Part 1: Technical delivery conditions.

German Institute for Standardization (DIN). 2006b. DIN EN 10219-2, Cold formed welded structural hollow sections of non-alloy and fine grain steels-Part 1: Tolerances, dimensions and sectional properties.


Tappenden, K. M. 2007. Predicting the Axial Capacity of Screw Piles Installed in Western Canadian Soils. MSc thesis, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada.


Figure Captions

Figure 1. Schematic of test piles. Note: black strips on the shaft denote the locations of full-bridge strain gauge stations; L1 to L4 denote the spacing between adjacent strain gauge stations.

Figure 2. Map of test site located in Edmonton, Canada. Global coordinate of test site: N 53.498394, W 113.532756.

Figure 3. Layout of CPT, Shelby tube boreholes (BH), and test piles.

Figure 4. CPT test results and undrained shear strength measured in laboratory.

Figure 5. Atterberg limit and natural moisture content (mc) of intact soil samples: (a) BH3 (b) BH4.

Figure 6. Plasticity chart for undisturbed soil samples from BH-3 and BH-4.

Figure 7. Setup of the field load tests: (a) a drawing of the system showing two piles tested in a sequence; and (b) a photo of the load test apparatus.

Figure 8. Selected axial load (Q) versus normalized axial displacement (w/D) curves for: (a) compression tests and (b) tension tests. w: axial displacement; D: diameter of smooth shaft. The selected tests were located in the western region of the test site, where the soil characteristics are very similar.

Figure 9. Axial loads transferred to pile shaft and unit shaft resistance at limit state (q_{sl}) for tests: (a) P1-C5, (b) P5-C4, and (c) P5-C5.

Figure 10. Measured q_{sl} vs. average s_u of the smooth shaft segment of selected instrumented piles. The shaft diameter of the piles P5 and P6 is 76 mm.

Figure 11. Hypothesized theoretical modes of axial failure in the threaded segment: (a) cylindrical shear mode (CSM) along the edge of threads, and (b) individual bearing mode (IBM) under each thread; and (c) proposed theoretical torque model, in which the applied torque is counterbalanced by the shear resistance from the threads and the shaft.

Figure 12. Estimated limit capacity Q_{th} vs. measured Q_{th} in the threaded shaft of selected instrumented piles. The estimation are based on the CSM and IBM hypotheses.

Figure 13. Measured limit capacity Q_{tp} vs. estimated Q_{tp_eq} of the threaded tapered segment of selected instrumented piles.

Figure 14. Limit capacities of all test piles: (a) estimated Q_{Le} of P1 through P6 vs. the measured Q_{Lm}, and (b) average estimated Q_{Le} vs. average measured Q_{Lm}. The estimated Q_{Le} is the sum of the limit capacities of the smooth segment, threaded segment, and the tapered
threaded segment. The error bars in (a) show the minimum and maximum $Q_{Le}$ due to the site heterogeneity as shown in the CPT $s_u$ profiles.

Figure 15. End installation torque $T$ of all test piles: (a) estimated torques $T_e$ of P1 through P6 vs. the measured torques $T_m$, and (b) average $T_e$ vs. average $T_m$. The error bars in (a) show the minimum and maximum $T_e$ due to the site heterogeneity shown in the CPT $f_s$ profiles.
<table>
<thead>
<tr>
<th>Pile ID</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>L₁ (mm)</th>
<th>L₂ (mm)</th>
<th>L₃ (mm)</th>
<th>L₄ (mm)</th>
<th>θ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>114</td>
<td>3000</td>
<td>762</td>
<td>1148</td>
<td>229</td>
<td>612</td>
<td>5.5</td>
</tr>
<tr>
<td>P2</td>
<td>114</td>
<td>1600</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>5.5</td>
</tr>
<tr>
<td>P3</td>
<td>89</td>
<td>3000</td>
<td>660</td>
<td>890</td>
<td>1030</td>
<td>254</td>
<td>5.5</td>
</tr>
<tr>
<td>P4</td>
<td>89</td>
<td>1600</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>5.5</td>
</tr>
<tr>
<td>P5</td>
<td>76</td>
<td>3000</td>
<td>1753</td>
<td>457</td>
<td>483</td>
<td>254</td>
<td>5.2</td>
</tr>
<tr>
<td>P6</td>
<td>76</td>
<td>1600</td>
<td>520</td>
<td>375</td>
<td>318</td>
<td>210</td>
<td>5.2</td>
</tr>
</tbody>
</table>

Note: Symbol definitions are labelled in Figure 1. The spacing between threads ($s_{th}$) of all test piles is 50 mm. The thread width ($w_{th}$) and thickness ($t_{th}$) of all test piles are 12 mm and 2 mm, respectively. The thickness of the smooth shaft of all test piles is 3.6 mm.
Figure 1. Schematic of test piles. Note: black strips on the shaft denote the locations of full-bridge strain gauge stations; $L_1$ to $L_4$ denote the spacing between adjacent strain gauge stations.
Figure 2. Map of test site located in Edmonton, Canada. Global coordinate of test site: N 53.498394, W 113.532756.
Figure 3. Layout of CPT, Shelby tube boreholes (BH), and test piles.
Figure 4. CPT test results and undrained shear strength measured in laboratory.

289x202mm (300 x 300 DPI)
Figure 5. Atterberg limit and natural moisture content (mc) of intact soil samples: (a) BH3 (b) BH4.
Figure 6. Plasticity chart for undisturbed soil samples from BH-3 and BH-4.

215x279mm (300 x 300 DPI)
Figure 7. Setup of the field load tests: (a) a drawing of the system showing two piles tested in a sequence; and (b) a photo of the load test apparatus.

103x113mm (300 x 300 DPI)
Figure 8. Selected axial load ($Q$) versus normalized axial displacement ($w/D$) curves for: (a) compression tests and (b) tension tests. $w$: axial displacement; $D$: diameter of smooth shaft. The selected tests were located in the western region of the test site, where the soil characteristics are very similar.
Figure 9. Axial loads transferred to pile shaft and unit shaft resistance at limit state ($q_{sL}$) for tests: (a) P1-C5, (b) P5-C4, and (c) P5-C5.

201x288mm (300 x 300 DPI)
Figure 10. Measured $q_{sl}$ vs. average $s_u$ of the smooth shaft segment of selected instrumented piles. The shaft diameter of the piles P5 and P6 is 76 mm.
Figure 11. Hypothesized theoretical modes of axial failure in the threaded segment: (a) cylindrical shear mode (CSM) along the edge of threads, and (b) individual bearing mode (IBM) under each thread; and (c) proposed theoretical torque model, in which the applied torque is counterbalanced by the shear resistance from the threads and the shaft.
Figure 12. Estimated limit capacity $Q_{th}$ vs. measured $Q_{th}$ in the threaded shaft of selected instrumented piles. The estimation are based on the CSM and IBM hypotheses.
Figure 13. Measured limit capacity $Q_p$ vs. estimated $Q_{p,eq}$ of the threaded tapered segment of selected instrumented piles.
Figure 14. Limit capacities of all test piles: (a) estimated $Q_{le}$ of P1 through P6 vs. the measured $Q_{Lm}$, and (b) average estimated $Q_{le}$ vs. average measured $Q_{Lm}$. The estimated $Q_{le}$ is the sum of the limit capacities of the smooth segment, threaded segment, and the tapered threaded segment. The error bars in (a) show the minimum and maximum $Q_{le}$ due to the site heterogeneity as shown in the CPT $s_u$ profiles.
Figure 15. End installation torque $T$ of all test piles: (a) estimated torques $T_e$ of P1 through P6 vs. the measured torques $T_m$, and (b) average $T_e$ vs. average $T_m$. The error bars in (a) show the minimum and maximum $T_e$ due to the site heterogeneity shown in the CPT $f_s$ profiles.