Derivation of CFA pile capacity in a silty clay soil using CPTu

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ABSTRACT: Conventional axial pile load testing was undertaken on two full-scale CFA piles installed in silty clay soils in Prince Albert, Saskatchewan, Canada. Prior to installation of the test piles, piezocone penetration tests (CPTu) were undertaken at the proposed centre location of each pile. Pile capacity estimates utilizing two direct CPT methods and two indirect methods (with CPT derived soil parameters) are presented. An examination of the results revealed that the indirect method of Coleman and Arcement (2002) provided the closest agreement between the measured (Q_M) and predicted (Q_P) pile capacity for both test piles, with a value of Q_P/Q_M of 1.09 and 0.96, respectively. The LCPC (1982) and Eslami-Fellenius (1997) direct methods also performed well for one of the test piles with a predicted capacity within approximately 4% of measured. Overall, the agreement between Q_P and Q_M was good as all predictions were within approximately 19 percent of the measured pile capacity.

1 INTRODUCTION

Unstable soil conditions were identified in the geotechnical investigation undertaken for a site in Prince Albert, Saskatchewan, Canada that warranted the use of an alternative piling system to conventional drilled piles. Continuous Flight Auger Piles (CFA) were considered to be a practical alternative and were chosen as the foundation system. To the author's knowledge, this was the first project in the Prince Albert area to use CFA piles. Due to the lack of knowledge on the behaviour of CFA piles in this area, pile load testing was undertaken to optimize the design of the piles.

Piezocone penetration tests (CPTu) were conducted on August 18, 2008 at the location of each proposed test pile. The CFA piles were installed on August 26, 2008 and load tested 9 and 10 days after installation, respectively.

CPTu testing and CFA piles are relatively new technology to this region of Canada. This project provided a good opportunity to compare how well empirical pile prediction methods pertain to Saskatchewan soils. CPTu derived pile capacity estimates as well as the results of the pile load testing are presented.

2 SITE CHARACTERIZATION

2.1 Soil Stratigraphy

The soil conditions encountered at the site within the depth of interest of the test piles (i.e. 16 metres below existing grade) consisted of a glacio-lacustrine deposit of silty clay. Sand/silt layers were encountered within the clay at varying depths. The clay was medium to highly plastic and generally stiff in consistency. The water table was situated at a depth of approximately 3 to 4 metres below existing grade. Glacial till was encountered below the bearing elevation of the piles at approximately 22 metres below existing grade.

2.2 Piezocone Penetration Testing

To assist in the interpretation of the pile load test results, a piezocone penetration test (CPTu) was conducted at the centre of each test pile prior to pile installation. A standard piezocone with a 60° tip, 10 cm^2 base area and a 150 cm^2 friction sleeve was used on this project. The filter element for measurement of pore pressure was located behind the cone in the u_2 position. The test plots generated during the cone soundings, including the interpreted soil behaviour type (SBT) profile according to the soil profiling method of Robertson (1990), have been presented on Figures 1 and 2. For comparison, the soil conditions encountered in a Test Hole drilled adjacent to TP2 is shown on Figure 2 along with the SBT of Eslami-Fellenius (1997).

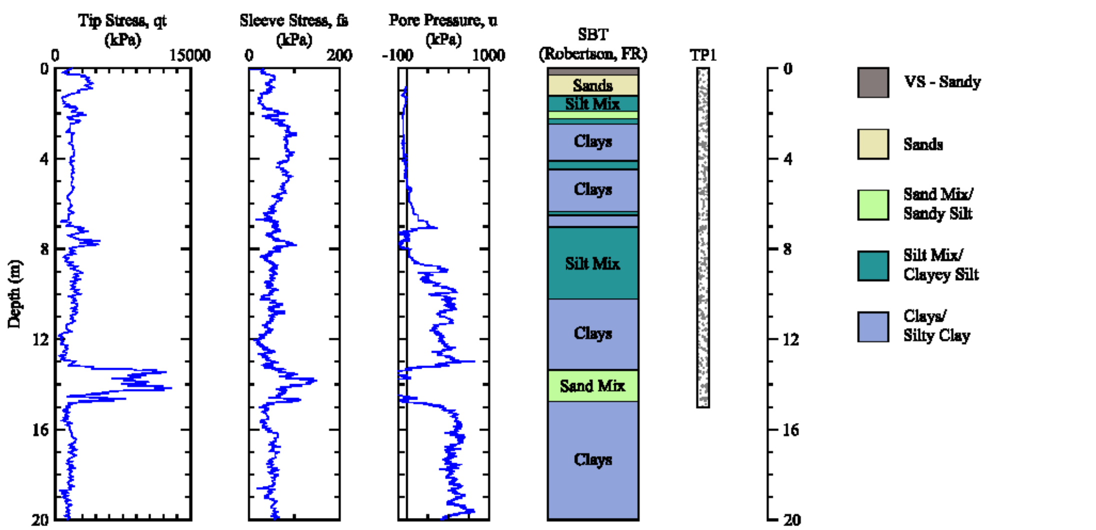

Figure 1. CPTu results at TP1.

An examination of the figures revealed that measured tip readings (q_t) in the clay within the depth of interest ranged from approximately 700 to 3000 kPa with an average of about 1800 kPa. In general, higher tip readings were encountered at the location of TP2. Based on Eq. 7 presented in Section 3.2, the average CPTu derived undrained shear strength of the clay soils within the depth of interest was approximately 85 kPa at TP1 and 120 kPa at TP2.

The elevated tip and negative pore pressure readings measured at TP1 indicate the presence of silt/sand layers at depths of 7 to 8 metres and 13 to 14.5 metres. A thin sand/silt layer was interpreted at the location of TP2 between about 5 to 6 metres from ground surface.

Contrast to the interpreted SBT, the soil conditions observed in test holes drilled at the site were predominantly cohesive. The implications of this on the pile capacity estimates are discussed further in Section 5.

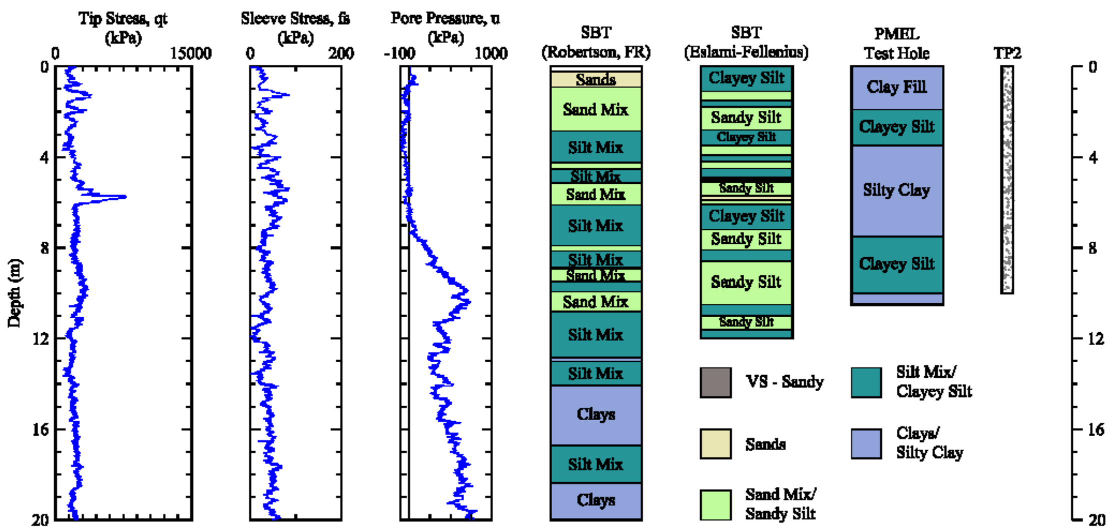

Figure 2. CPTu results at TP2.

3 AXIAL PILE CAPACITY PREDICTIVE METHODS

3.1 *Introduction*

As with other pile types, the ultimate capacity of a single CFA pile (R_{ult}) is equal to the summation of load carried by the pile shaft (R_s) and pile toe (R_t) and is given by

$$R_{ult} = R_s + R_t = \int C_s r_s dz + A_t r_t \tag{1}$$

where C_s = circumferential area of the pile shaft
r_s = unit shaft resistance over depth dz
A_t = pile toe area
r_t = unit toe resistance

There are two general types of methods for estimating pile capacity, those that are based on soil parameters and those that are based on in-situ tests. Soil parameter methods (otherwise known as "indirect" when using CPT results) determine pile capacity using classical soil mechanics principals whereas in situ test methods correlate pile capacity with SPT or CPT test results directly. A discussion of the "direct" and "indirect" pile predictive methods used in this study is presented in the following sub-sections.

3.2 *Indirect Methods*

The soil conditions encountered at the site consisted predominantly of silty clay. For piles installed in cohesive soils, it is common practice to determine the ultimate pile capacity utilizing a total stress approach which relates undrained shear strength to shaft and toe capacity through the use of coefficients. Unit shaft and toe resistance are commonly determined using the following relationships

$$r_s = \alpha s_u \tag{2}$$

$$r_t = N_c s_u \tag{3}$$

where s_u is undrained shear strength, α is an adhesion coefficient and N_c is a bearing capacity factor commonly set at 9.

Recently, Brown et al (2007) published a state-of-the-practice manual regarding the design and construction of continuous flight auger piles. In their report, predictive methods were identified that appeared to generally provide reliable and accurate estimates of static axial capacity of single CFA piles. Two total stress methods identified in their report were utilized in the analysis for this study. They were the methods presented by Coleman & Arcement (2002) and O'Neill & Reese (1999), otherwise known as the FHWA 1999 method.

Research on the results of load tests conducted on instrumented drilled piles has shown that α is not constant and varies with the magnitude of undrained shear strength. Coleman & Arcement suggest the following relation for determination of α.

$$\alpha = 56.192 s_u^{-1.0162} \qquad \text{for } 0.35 \le \alpha \le 2.5 \tag{4}$$

Similarly, O'Neil & Reese also suggest α varies with magnitude of undrained shear strength according to the following:

$$\alpha = 0.55 \qquad \text{for } s_u \le 150 \text{ kPa} \tag{5}$$

$$\alpha = 0.55 - 0.001(s_u - 150) \qquad \text{for } 150 \text{ kPa} \le s_u \le 250 \text{ kPa} \tag{6}$$

The magnitude of undrained shear strength was determined based on the results of the CPTu testing and the following empirical equation

$$s_u = \frac{q_t - \sigma_v}{N_{kt}} \tag{7}$$

where q_t is corrected cone tip resistance, σ_v is total overburden stress and N_{kt} is an empirical cone factor. Based on local experience, a value of $N_{kt} = 17$ was assigned to the clay stratum at this site.

The β Method, according to Coleman & Arcement (2002), was utilized to estimate the unit skin friction in the silt/sand layers interpreted between 7 to 8m and 13 to 14.5m at TP1 and from 5 to 6m at TP2. Otherwise the soils were treated as cohesive.

3.3 *Direct Methods*

Two direct CPT methods, that utilize the measured cone tip readings directly for deriving pile capacity, have also been included in the analysis. These are the methods of Eslami and Fellenius (1997) and Bustamante and Gianeselli (1982), otherwise known as the LCPC method.

Most CPT based equations for deriving pile capacity relate the cone resistance to pile shaft and toe resistance using constants that vary with soil and pile type. The general format of the CPT derived equations are as follows

$$rs = csqc \tag{8}$$

$$rt = ctqc \tag{9}$$

where c_s and c_t are constants, and, q_c is measured cone tip resistance. The Eslami and Fellenius method differs slightly from Equation 8 and 9 above in that the method uses an "effective" cone stress ($q_E = q_t - u_2$) in place of q_c for determination of unit shaft and toe resistance.

Full details of the appropriate shaft and toe constants to apply to both methods, in addition to the averaging procedures to obtain a characteristic q_c or q_E beneath the pile toe are discussed elsewhere by Lunne et al. (1997) and Fellenius (2009). It should be noted, the maximum limits presented for the LCPC Method for unit shaft resistance were not applied in this study.

4 PILE LOAD TESTING PROGRAM

CFA piles are installed by drilling a continuous flight, hollow stem auger (that is plugged at the base) into the ground, followed by pressure injection of concrete through the stem and simultaneous extraction of the hollow stem auger. For a detailed overview of the construction sequence and their benefits/limitations for use, the reader is referenced to the publication prepared by Brown et al (2007).

Conventional non-instrumented pile testing, with the load applied at the pile head, was undertaken during this project. The load testing program consisted of the testing of two CFA piles. The piles were nominal 406 mm in diameter and extended to depths of 10 metres and 15 metres below existing grade, respectively. The static load testing procedure was undertaken in accordance with the Quick Testing Method as described in the Canadian Foundation Engineering Manual (4^{th} Edition, 2006).

The failure load of each pile was determined based on the interpretation method presented by Hirany and Kulhawy (1989). The method was developed specifically for the interpretation of load test results conducted on drilled foundations under axial compressive, tensile and lateral loading (L1-L2 Method).

Axial pile compression and tension load movement curves generated during a load test normally have three distinct regions: initial linear, transition and final linear. According to the L1-L2 Method, the interpreted failure load (Q_{L2}) is determined graphically and is defined qualitatively as the load beyond which a small increase in load produces a significant increase in movement (i.e., transition point to final linear region). An example of the method is illustrated on Figure 3 for the load movement curve generated for TP1.

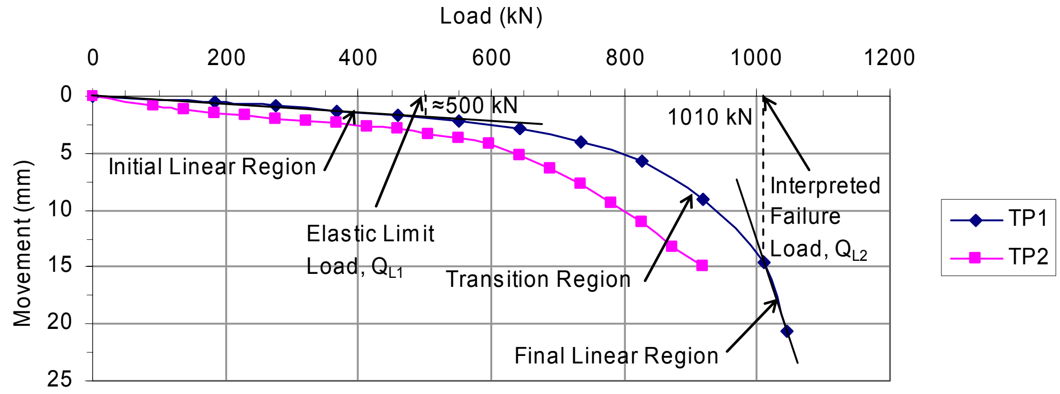

Figure 3. Load–movement curves for TP1 and TP2.

Examination of Figure 3 revealed that the shape of the load movement curve for TP2 had only 2 distinct regions, initial linear and transition. A distinct final linear region is not obvious, which makes it difficult to define the failure load for this pile according to the L1-L2 Method. Although the determination of a failure load for TP2 is not obvious, the shape of the load movement curve can assist in determining a safe working load and thus an estimate of what the failure capacity may be.

As discussed by Hirany & Kulhawy (2002), the maximum design load should be located within the initial linear region of the load-movement curve, preferably less than or equal to the load at which the shape of the load-movement curve changes from the initial linear region to the transition region (i.e. elastic limit load, Q_{L1}). This ensures that the pile-soil behaviour will be essentially elastic and pile displacements at working loads will be small. Application of a factor of safety (FS) of 2 or greater to Q_{L2}, will, on average, give a safe working load equal to or less than Q_{L1} (Hirany & Kulhawy, 2002). In essence, doubling Q_{L1} would provide an approximation of the value of Q_{L2}. Based on this rational and the shape of the load-movement curve for TP2, $Q_{L1} \approx 450$ kN, which results in $Q_{L2} \approx 2(450) \approx 900$ kN.

Based on the work done at Cornell University on a database of load tests undertaken on CFA piles (Kulhawy, 2004), Q_{L2} was found to occur at an average displacement of approximately 3.4% B (where B=pile diameter). For this study, Q_{L2} occurred at a pile displacement of 3.6% B and 3.5% B for TP1 and TP2, respectively. The good agreement between the load-movement behaviour of the piles in this study with the general framework presented for the L1-L2 Method suggests that $Q_{L2} = 900$ kN is a reasonable estimation of the failure load for TP2.

A summary of the interpreted failure capacity and resulting average unit shaft resistance for both test piles has been presented in Table 1.

Table 1. Load test interpreted pile capacities.

Pile number	Shaft Diameter mm	Shaft length m	Interpreted failure capacity kN	*Average unit shaft resistance kPa
TP1	406	15	1010	49
TP2	406	10	900	57

*Based on deducting toe bearing contribution (i.e. $9s_u$) from interpreted failure capacity.

5 RESULTS, DISCUSSION AND CONCLUSIONS

The CPTu derived pile capacity estimates (Q_P) have been shown normalized in Figure 4 with respect to the measured pile capacity (Q_M). Although not determined during the load testing program, the contribution of the pile shaft and toe to the total capacity for each predictive method has been shown on the figure. A discussion of the results is presented below.

1. The indirect method of Coleman & Arcement agreed very well with the results of the load testing with a value of Q_P/Q_M of 1.09 and 0.96, respectively.
2. The LCPC and Eslami-Fellenius direct methods had excellent agreement with the results from TP1 with a predicted capacity within approximately 4% of measured.
3. Better overall agreement was attained between the predicted and measured capacities for TP1 as all 4 methods were within approximately 10% of the interpreted pile capacity.
4. Both direct methods underpredicted capacity for TP2.
5. In the analysis, the LCPC prediction utilized the SBT soil profiling method of Robertson (1990) whereas the Eslami-Fellenius Method used its own profiling chart. Examination of Figure 2 shows that both profiling methods indicated the presence of sandy silt layers within the depth of the test pile. The soil conditions in the test hole drilled adjacent to TP2 consisted of deposits of silty clay and clayey silt. The under prediction by both methods could be attributed to the disparity between the soil conditions recorded at the test hole and the SBT determined by the CPT.
6. The coefficient utilized for determination of unit shaft resistance for sand/silt soils is approximately 30 to 40 percent smaller as compared to the value used for silt/clay soils for both direct CPT methods. If one assumes the SBT for both methods for TP2 to be predominantly silt/clay, the predicted total capacities come into close agreement with the measured pile capacity (i.e within 1% - Eslami-Fellenius and 8% - LCPC).

The findings of this study demonstrate that when used directly or indirectly with the four predictive methods presented, the CPTu was well suited to predicting the capacity of CFA piles installed in a silty clay deposit at this study site. The lower agreement between Q_P and Q_M for TP2 (when using the direct methods) demonstrates the importance of the accuracy of the CPTu interpreted soil type as compared with the actual soil conditions. Conventional drilling and sampling should be used in conjunction with the CPTu to assist in the interpretation of the CPTu results. Nonetheless, the predicted pile capacities for TP2 were still good and within 19% of measured.

The results of the pile load testing were successful in optimizing the design of the CFA foundation piles at this site. Combined with utilizing a lower factor of safety (as a result of conducting the pile load testing), the allowable skin friction utilized for design increased by about 70 percent as compared to the initial geotechnical recommendations.

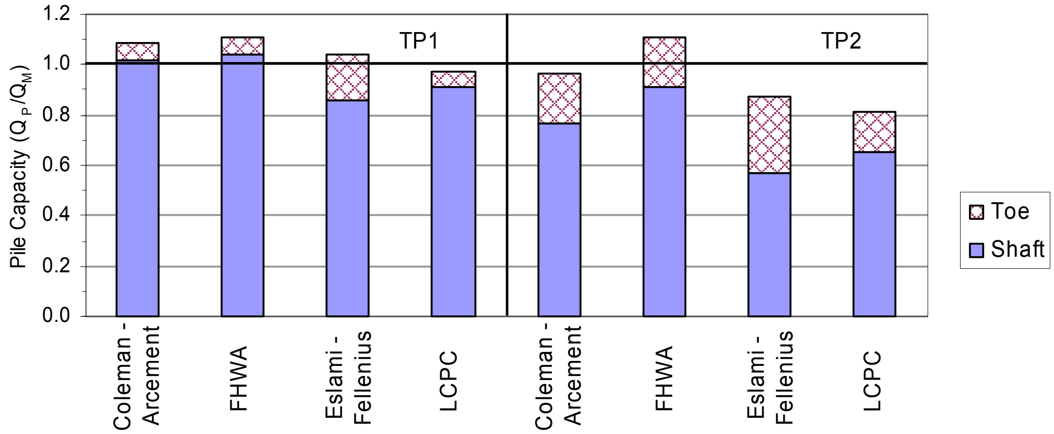

Figure 4. Predicted vs. measured pile capacity for TP1 and TP2.

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