Pile bearing capacity in glacial till

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ABSTRACT
Pile load testing was conducted during a recent project undertaken at the University of Saskatchewan Campus in Saskatoon, Saskatchewan. The pile testing program included vertical axial load testing of both drilled, cast-in-place friction piles and end bearing belled caissons. The subgrade soils at the site consisted of 2 to 4 metres of clay overburden underlain by very stiff glacial till that became hard with depth. A geotechnical investigation was undertaken at the subject site complete with SPT, CPTu and standard laboratory testing to determine the mechanical properties of the subsurface deposits. A summary of the site characterization and pile testing program has been presented and the results are compared with conventional empirical methods of estimating pile capacity.

RÉSUMÉ
L'essais de chargement sur pieux a été conduite pendant un projet récent entrepris à l'Université de Saskatchewan. Le programme d'essais de pieux a inclus l'essais de chargement vertical sur les pieux de friction coulés en place, et les pieux de caissons élargies de résistance à la pointe combinés. Les sols de sous-catégorie au site ont consisté en 2 à 4 mètres d'argile surchargent sous-jacent par till glaciaire très raide avant que ce ne soit devenu dur avec la profondeur. Une enquête géotechnique a été entreprise au site complet avec SPT, CPTu et l'examen de laboratoire standard pour déterminer les propriétés mécaniques des depots soussuperficiels. Un résumé de la caractérisation de site et le programme d'essais de pieux a été présenté et les résultats sont comparés avec les méthodes empiriques conventionnelles d'évaluer la capacité de pieux.

1 INTRODUCTION

A pile load testing program was carried out at the University of Saskatchewan Campus in Saskatoon, Saskatchewan at the site of the proposed Intervac Facility. The purpose of the pile load testing was to determine the load carrying characteristics of the founding subgrade soils and provide a cost effective foundation. The proposed foundation system for the building consisted of a combination of drilled, cast-in-place concrete straight shaft piles and belled caissons.

The soil conditions at the subject site consisted of a thin clay overburden underlain by an extensive deposit of very stiff to hard glacial till. Glacial till soils, like those encountered at the Intervac site, have historically provided sound bearing support for numerous large scale projects within the City of Saskatoon region. However, since the glacial till soils are considered a competent bearing material, full scale load testing is rarely done. As a result, the full carrying capacity of the glacial till soils is not well known. The pile load testing program has provided a rare opportunity to compare the measured pile capacities with estimates determined utilizing standard empirical pile design methods. This paper has been prepared to summarize the results of the pile load testing along with the field and lab testing program undertaken.

2 GEOLOGY

The geology of the University of Saskatchewan Campus and surrounding area is comprised of 45 to 60m of stratified drift (i.e., glaciolacustrine clays) and glacial till overlying Bearpaw Shale (Fredlund 1970). Four Distinct glacial events appear to have occurred in the Saskatoon area. The sediments above the bedrock have been subdivided into the Sutherland Group and Saskatoon Group (Christiansen 1968, 1992). The general stratigraphy of the campus area has been shown in Figure 1.

<table>
<tr><td>Stratified Drift</td><td rowspan="3">Saskatoon Group</td><td>Glacial Lake Deposits</td></tr>
<tr><td rowspan="5">Glacial Till</td><td>Battleford Formation</td></tr>
<tr><td>Floral Formation</td></tr>
<tr><td rowspan="3">Sutherland Group</td><td>Warman Formation</td></tr>
<tr><td>Dundurn Formation</td></tr>
<tr><td>Mennon Formation</td></tr>
<tr><td colspan="3">Bedrock</td></tr>
</table>

Figure 1. Stratigraphic Chart

The tills from the younger Battleford formation are considered soft to stiff, massive, silty clays whereas tills from the older Floral Formation and Sutherland Group are stiff to hard, fissured, silty clays. The tills from the Sutherland Group also have higher plasticity as compared to the upper tills.

3 SITE CHARACTERIZATION

The soil conditions at the location of the test pile arrangement consisted of 4m of clay overburden underlain by very stiff to hard glacial till to a depth of at least 15m below existing ground surface (i.e., the maximum depth explored with the geotechnical investigation at this site). The glacial till can be described as a silty clay containing some sand and trace gravel. The till was brown in colour and became grey below about 7m below existing ground surface. A cobble/boulder layer was encountered in the till at a depth of about 7m in some areas of the site. The groundwater table was recorded at a depth of about 3m below existing ground surface. A summary of the laboratory index testing undertaken on soil samples recovered from the glacial till stratum has been summarized in Table 1. The test results from the Battleford and Floral units were combined due to their similarity in composition.

Table 1. Glacial Till Index Test Results

Soil Index Property	Battleford & Floral Tills		Sutherland Till	
	Range	Avg.	Range	Avg.
W (%)	7 – 16	12	14 - 21	18
W_L (%)	23 - 26	24	36 - 61	48
W_P (%)	11 - 14	12	14 - 20	17
γ (kN/m^3)	21 - 23	22	20 - 22	21
Sand (%)	39 - 50	43	21 - 28	26
Silt (%)	35 - 43	38	40 - 55	46
Clay (%)	12 – 19	16	18 - 36	27

An examination of Table 1 revealed that the till units, on average, contained between 16 to 27 percent clay. Although the predominant grain size is silt and sand, the tills are considered cohesive due to the dominant behaviour of the clay portion.

The primary test methods for determining the mechanical properties of the glacial till consisted of standard penetration testing (SPT), piezocone penetration testing (CPTu) and unconsolidated undrained triaxial testing (UU). The test results gathered from a test hole and two CPTu soundings undertaken adjacent the test piles have been shown in Figure 2.

The cone sounding, CPTu-2, and the SPT and UU tests were undertaken in March 2008 after completion of the pile loading tests and excavation of the basement level of the building. The cone readings for CPTu-2 were artificially high at the 4m depth as the soil was frozen at the time of the investigation. CPTu-1 was conducted in November 2007 prior to excavation of the basement.

Based on the results of the in-situ and laboratory testing, the glacial till stratum was subdivided into its geologic units as shown on Figure 2. The Battleford and Floral Till units were relatively thin, measuring approximately 1 to 1.5m in thickness. The Sutherland Till was extensive and extended to the full depth explored.

An examination of Figure 2 revealed that the measured CPTu tip resistance (q_c) in each of the three identified till units was quite consistent and measured approximately 2.9 MPa in the Battleford Till, 6 MPa in the Floral Till and 4.5 MPa in the Sutherland Till. The SPT and UU test results followed a similar trend with higher values encountered in the lower tills. The cone soundings identified a weaker layer of soil at the transition from the Floral to Sutherland Till between approximately 6.6 to 8m below existing grade. The measured tip reading in this zone was in the order of 2.6 MPa.

In addition to performing compression tests in the lab, the strength of the subgrade soils can be estimated utilizing the in-situ test results. The relationship between undrained shear strength (s_u) and q_c is commonly expressed using the following equation:

$$s_u = q_c/N_k \qquad [1]$$

where N_k is a cone factor.

The undrained shear strength can also be related to SPT N values in the manner proposed by Stroud (1989) using the relationship:

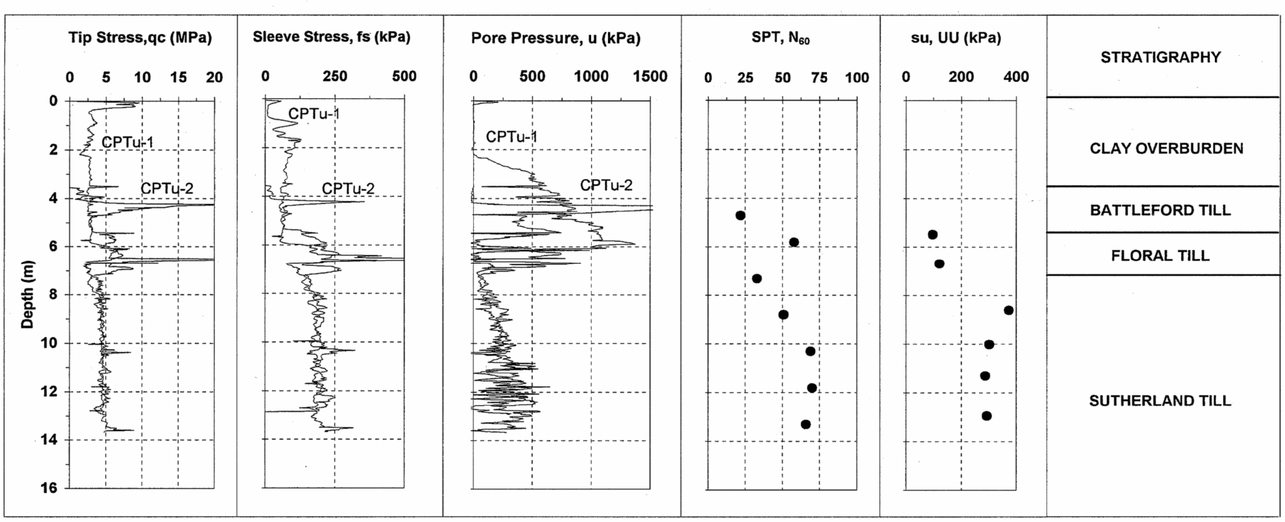

Figure 2. Soil Conditions at Test Pile Arrangement

$$s_u = f_1 N_{60} \quad [2]$$

where f_1 is a constant.

When compared to the UU test results (primarily below 7m), a reasonably good fit was observed when using N_k=14 and f_1=5. The results are shown plotted in Figure 3. The tip readings from 3.5 to 4.6m were inferred from CPTu-1 to eliminate the artificially high measurement recorded in CPTu-2 due to frozen ground.

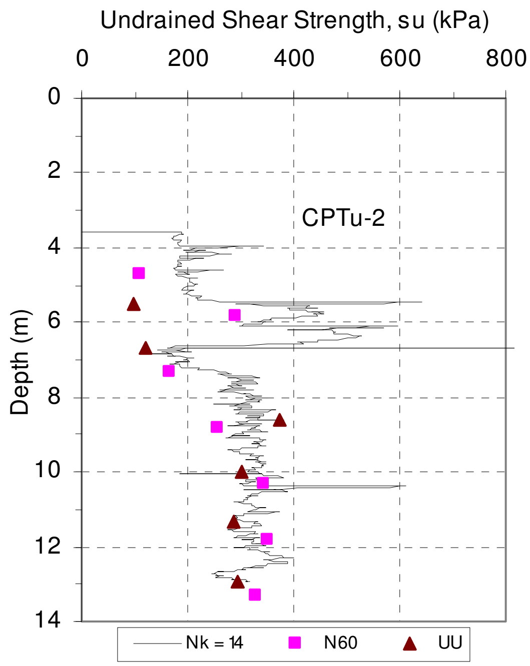

Figure 3. Undrained Shear Strength vs. Depth

The profile presented in Figure 3 shows that the undrained shear strength was fairly uniform below a depth of about 7.4m and varied in strength from approximately 300 to 375 kPa with an average of about 325 kPa. The strength of the weaker zone encountered at 7m was in the order of 185 kPa.

The SPT f_1 factor of 5 is in agreement with that proposed by Stroud for boulder clays (i.e., glacial till). The Nk factor was also in the range of that found by others (Adam 1985; Powell and Quarterman 1988; as discussed by Trenter 1999). When using Nk = 14, a poor fit is realized when compared to the limited SPT and UU data from 4 to 6.5m. The poor fit could be attributed to the different composition of the upper soils (i.e., sandier/siltier) as compared to the lower till unit. Additional testing is needed to properly characterize the strength of this zone.

The soil conditions encountered within the depth explored on Figure 3 likely represent 3 glacial events. It is possible that the results of further lab testing could indicate a different Nk value is valid in each till unit.

4 PILE LOAD TESTING PROGRAM

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 installation and testing of two drilled, cast-in-place friction piles and two belled caissons. A fifth test pile, which was not loaded to failure, will not be discussed herein. The test pile layout along with the test hole and piezocone locations have been presented in Figure 4.

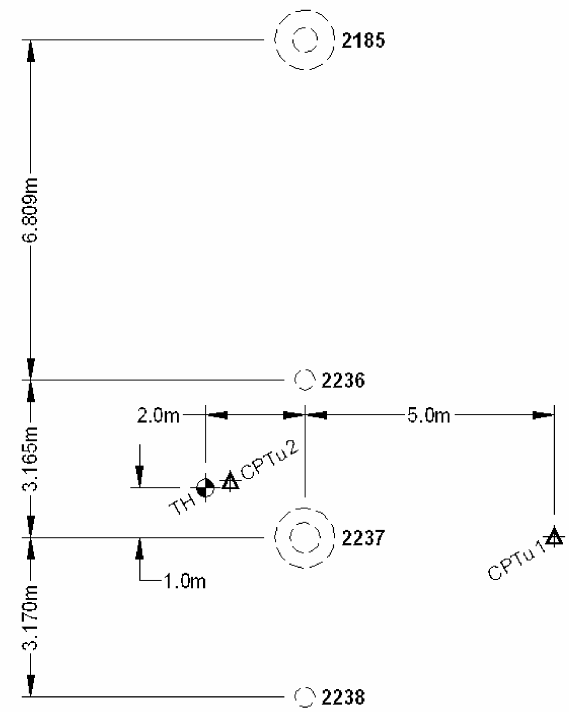

Figure 4. Test Pile Layout

It was attempted to construct the test piles such that the majority, if not all, of the pile capacity was developed in either skin friction or end bearing. Styrofoam pellets and clay backfill were placed at the bottom of the friction piles to eliminate end bearing while the shafts of the belled caissons were formed with casing inside an oversized hole to eliminate shaft friction. The pile construction details for Piles Nos. 2185 and 2236 have been presented in Figure 5.

The static load testing procedure was undertaken in accordance with the Quick Testing Method as described in the Canadian Foundation Engineering Manual (4th Edition, 2006). The test piles were loaded in 17 or more equal 5 minute load increments to failure.

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 shafts under axial compressive, tensile and lateral loading (L1-L2 Method).

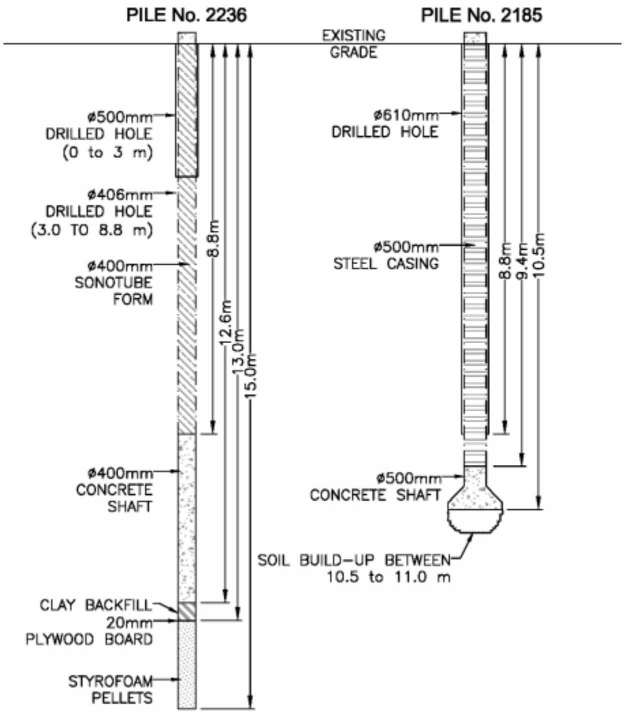

Figure 5. Pile Construction Details

Drilled 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 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 in Figure 6 and Figure 7. The interpreted failure capacities, as determined by the L1-L2 Method, have been summarized in Table 2.

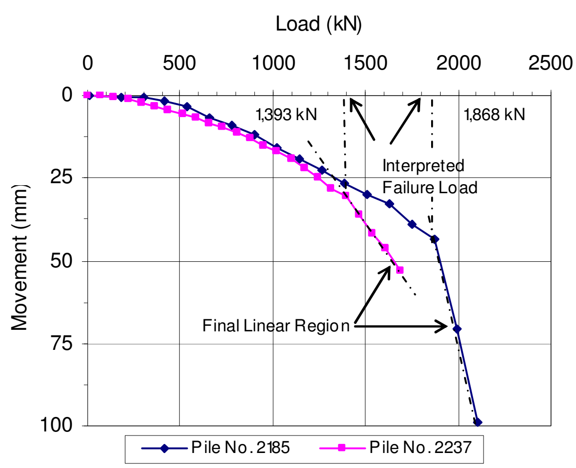

Figure 6. Belled Pile – Load vs. Movement Curves

As mentioned previously, it was attempted to construct the piles such that the load was carried entirely by the pile toe or pile shaft. The high capacity interpreted from the load test results from Pile No. 2236 led to some questioning of how successful this was. As shown on Figure 5, the upper 8.8m of the pile shaft was formed in sonotube in an attempt to eliminate skin friction in this zone. However, a closer examination of the pile construction details revealed that the drilled hole was only marginally larger than the sonotube form below a depth of about 3m (i.e. 406mm auger vs. 400mm sonotube). This was confirmed after excavation and inspection of the upper 4m of the pile shaft revealed that the sonotube form was in intimate contact with the surrounding soil below the 4m depth.

The sonotube form utilized was light weight “Econo-Tube”. It is believed that the sonotube either expanded under the weight of the concrete and/or the pile hole relaxed, or a combination of both, to close the gap between the hole and pile shaft. The load carrying capacity of the sonotubed portion of the pile shaft should be less as compared to concrete cast directly against a rough pile hole. According to Kulhawy (1991), the use of a permanent casing in a bored hole will reduce the shearing resistance of the soil-shaft interface for cast-in-place concrete from unity to something in the order of 0.7. Hence, a reduction of 0.7 was applied to the carrying capacity of the cased portion of the pile shaft in our analysis.

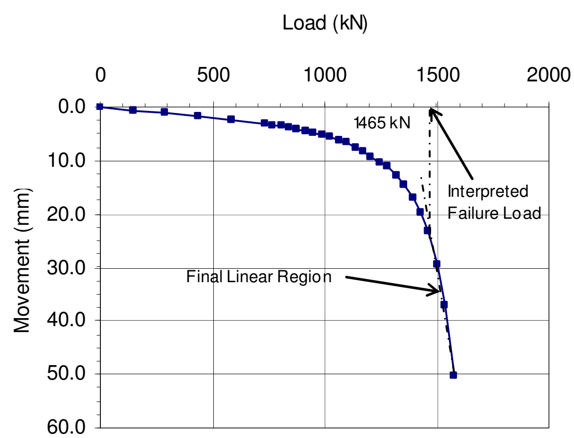

Figure 7. Friction Pile No. 2236 - Load vs. Movement Curve

Table 2. Test Pile Dimensions and Interpreted Capacities

Pile No.	Pile Type	Shaft/Bell Diameter (mm)	Skin Friction Zone/Bell Depth (m)	Interpreted Failure Capacity (kN)
2185	Belled	1,200	10.5	1,868
2236	Friction	400	3 to 12.6	1,465
2237	Belled	1,200	7.8	1,393
2238	Friction	400	3 to 6.4	256

Based on the method of construction, the applied load carried by the belled piles was not all in end bearing. As shown in Figure 5, the bottom-most 1m or so of the pile shaft for Pile No. 2185 was in contact with the surrounding soil. Pile No. 2237 was also constructed in a similar fashion. The piles were constructed in this way to prevent concrete from entering and filling the gap between the casing and over-sized hole. The load movement curves shown in Figure 6 suggest that some of the load was carried in shaft friction as the initial part of the curves were relatively flat. The transition to end bearing appears to have occurred somewhere between an applied load of 300 to 400 kN for Pile No. 2185 and 100 to 200 kN for Pile No. 2237.

Approximately 500mm of soil build-up was measured beneath the pile toe during formation of the bell for Pile No. 2185. The disturbed soil did not appear to impact pile performance as the shape of the load movement curve was similar to that of Pile No. 2237, which had a clean bell. It is believed that the disturbed soil was in a compact state as a result of the belling process. According to Poulos (2005), the debris left at the base of a bored pile as the result of inadequate cleaning is unlikely to fail, but will be expected to deform considerably. In this case, the amount of additional settlement was not excessive and may have been equal to the difference in movement recorded at failure between the two piles (i.e., 14mm).

5 AXIAL PILE CAPACITY

The ultimate capacity of a single pile is equal to the summation of load carried by the pile shaft and pile toe. The pile shaft and toe capacity in cohesive soils is commonly estimated utilizing total stress methods. The alpha method, which relates shaft friction to the average undrained shear strength of the soil over the depth occupied by the pile, is expressed as

$$r_s = \alpha s_u \qquad [3]$$

where r_s is the unit shaft resistance and α is a coefficient that is dependant on the soil type, pile material and method of installation. The toe capacity is determined based on the average undrained shear strength within 1 to 2 bell diameters below the base level of the pile and is given by the equation

$$r_t = s_u N_c \qquad [4]$$

where r_t is the unit toe resistance and N_c is a bearing capacity factor commonly set at 9 (Skempton 1951, Meyerhof 1976).

Axial pile capacity is also commonly determined utilizing the CPTu results directly. The measured cone resistance (q_c) is generally related to shaft and toe resistance in the following format

$$r_s = c_s q_c \qquad [5]$$

and

$$r_t = c_t q_c \qquad [6]$$

where c_t and c_s are constants that depend on soil and pile type. Other methods have been proposed that relate the measured cone sleeve friction (f_s) directly to pile shaft friction (Schmertmann 1978). However, the use of q_c is preferred as it has been found to be more reliable (Salgado 2006).

Values of α, c_s, and c_t as determined from the results of the pile load testing have been presented in Table 3 and Table 4 along with some common values found in the literature.

The bearing capacity factor, Nc, as determined from the test results from Pile No. 2185 was 4.1. This is based on an average undrained shear strength of 325 kPa below the pile toe and 1518 kN load carried by the pile toe at failure (assuming 350 kN of the applied 1868 kN loading was carried by the pile shaft). When applying Nc = 4.1 to the results from Pile No. 2237, s_u = 268 kPa, which is approximately equal to the average strength of the weak layer situated just above the pile toe and the soil within a depth of one bell diameter below the pile toe (i.e., [185+325]/2 = 255 kPa). The pile shaft was assumed to carry 150 kN of the 1393 kN failure load.

6 DISCUSSION OF RESULTS

An examination of Tables 3 and 4 revealed the following.

1. The shaft friction in the zone 3 to 7.2m was best predicted utilizing the total stress method presented by Kulhawy and Jackson or the direct CPT methods by Eslami and Fellenius or Bustamante and Gianeselli. Reasonable agreement was found utilizing Weltman and Healy's results.
2. The method of DeRuiter and Beringen grossly underpredicted the shaft friction in this zone. The method is likely intended for use in clean sands and not silt/sand/clay mixtures as encountered at this site.
3. Below 8m, the shaft friction was under-predicted by all methods. The closest agreement was found when utilizing the CPT Method of DeRuiter and Beringen.
4. The toe constant determined from the Eslami and Fellenius method was in good agreement with the results from this site.
5. None of the methods had good agreement with the results when compared over the full length of pile and soil tested.

Of all the predictive methods presented, Weltman and Healy's was the only method specifically intended for glacial till soils. As shown in Tables 3 and 4, it performed no better or worse than the other methods.

The Nc factor of 4.1 determined from the load test results was considerably lower than the commonly accepted value of 9. Similarly, the results of pile load testing conducted by Tweedie et al. (1990) in a fissured glacial till deposit in Lloydminster, Saskatchewan, revealed an Nc value of about 5.4. As was suggested in their case, the most likely reason for the low results was due to the fissured nature of the glacial till soils.

Table 3. Shaft and Toe Constants, c_s and c_t, for CPT Prediction of Axial Pile Capacity

Pile Component	Zone/ Depth (m)	CPTu Soil Behaviour Type[3]	Interpreted Ultimate Bearing Pressure (kPa)	Source			
				Intervac Site	DeRuiter & Beringen (1979)	Bustamante & Gianeselli, LCPC Method (1982)	Eslami & Fellenius (1997)
Shaft, c_s	3 to 6.4	silty sand to sandy silt	60	0.016 (0.018)[4]	0.003	0.017	(0.015)
	Below 8	clayey silt to silty clay	195	0.043 (0.045)	0.038	0.025	(0.025)
Toe, c_t[2]	7.8	clayey silt to silty clay	1,099	0.24[1] (0.25)[1]	0.36	0.35	(0.28)
	10.5		1,342	0.29[1] (0.30)[1]			

Notes:
1. Toe constant based on averaging procedure proposed by LCPC Method (i.e., 1.5b above and below toe, where b is the toe diameter).
2. Analysis based on reducing the ultimate capacities presented in Table 2 for Pile Nos. 2185 and 2237 to 1518 kN and 1243 kN, respectively, to reflect some of the load being carried by the pile shaft.
3. CPTu Soil Behaviour Type as per Robertson 1990.
4. (0.018) Applicable to Eslami and Fellenius Method which uses an apparent effective cone stress, q_E, to correct for pore pressure acting against the cone shoulder; $q_E = q_t - u_2$

Table 4. Alpha Factor, α, for Prediction of Axial Shaft Friction Capacity

Zone (m)	Interpreted Ultimate Bearing Pressure (kPa)	Source			
		Intervac Site	Kulhawy & Jackson (1989)	Reese & O'Neill (1989)	Weltman & Healy (1978)
3 to 6.4	60	0.36[1]	0.37	0.54	0.43
Below 8	195	0.60[2]	0.29	0.45	≈0.36

Notes:
1. Based on utilizing s_u = 100 kPa from 3 to 5.4 and s_u = 325 kPa from 5.4 to 6.4., resulting in an average s_u value of 166 kPa along the entire pile shaft.
2. An average undrained shear strength of 325 kPa was used in the analysis.

Most glacial till soils contain directional voids such as fissures, joints and foliations (Trenter 1999). These features may not be present in small samples tested in the lab or at the small scale of the cone penetrometer and SPT sampler. As such, it is likely that the shear strength interpreted from these methods is more characteristic of the intact strength of the soil mass and not the representative strength below that of a foundation. Meyerhof (1983) established an empirical reduction factor for bored piles in fissured clays which decreases with increasing diameter of the pile toe. The reduction factor is given by

$$Rc = \frac{b+1}{2b+1} \leq 1 \qquad [7]$$

where Rc is a reduction factor to be applied to the intact undrained shear strength and b is the pile toe diameter in metres. Based on a pile diameter of 1.2m, the reduction factor according to Meyerhof is 0.65. For ease of discussion, the reduction factor will be applied to the Nc factor in our case.

Another issue that could have affected the results was the depth of embedment of the pile toe into the bearing stratum. Typically, the pile toe should be embedded into the bearing stratum by at least four diameters to mobilize the full strength of that layer (CFEM 2006). To account for partial embedment, Fleming et al. (1985) recommended that Nc = 6 for the case where the pile toe just penetrates the bearing stratum and Nc = 9 when the pile toe penetrates the bearing stratum by 3 diameters or more. Linear interpretation is recommended for intermediate values.

The depth of embedment into the bearing stratum for Pile No. 2185 was approximately 2 pile diameters. According to the relationship presented by Fleming, the Nc factor should be reduced to a value of 8. Application of the scaling factor proposed by Meyerhof reduces Nc to 5.2, which is in reasonable agreement with our results.

The lower test results from Pile No. 2237 imply that the pile capacity was affected by the weak layer situated directly above the pile toe. Based on an Nc of 6 to reflect the minimal embedment of the pile toe into the bearing stratum and Meyerhof's reduction of 0.65, the resulting Nc = 3.9, which is in good agreement with the results found in this study. Although the average undrained shear strength within 1 to 2 bell diameters below the pile toe is typically utilized, a better fit is realized in our case when using the average strength of the soil directly above and below the pile toe.

Alternately, the Canadian Foundation Engineering Manual (CFEM) recommends utilizing a range of values for the bearing capacity factor depending on the size of the pile toe. According to the CFEM and a toe diameter of 1.2m, the resulting Nc = 6. Application of Meyerhof's reduction to account for the fissured nature of the soil results in Nc = 3.9, which agrees well with Nc = 4.1 interpreted for both test piles.

The ultimate end bearing capacity as determined by the CPT was in good agreement with that predicted by Eslami and Fellenius. Similarly to the CFEM, Eslami and Fellenius incorporated a scaling factor into their method to account for the size of the foundation (i.e., the larger the pile toe size, the greater the reduction). Although the adjustment factor was incorporated to reflect the reduction in "usable" toe resistance with increasing pile toe diameter (i.e., larger movements required to mobilize toe resistance), and not the fissured nature of the soil, it served the same purpose in our case.

The amount of movement recorded at failure as a percentage of the bell diameter was 2.5% and 3.6% for Pile Nos. 2237 and 2185, respectively. Relatively good agreement was found when comparing the results with the normalized drilled pile load movement curves prepared by Reese and O'Neill (1989). According to their chart depicting load transfer in toe bearing vs. settlement, a displacement of about 4% of the toe diameter is predicted at failure.

Unlike toe resistance, the ultimate unit shaft friction of bored or driven piles in fissured clays is practically independent of the pile diameter (Meyerhof 1983). This is because the unit shaft resistance is dependant on the shear strength of a relatively thin layer of remoulded soil directly adjacent the pile shaft. The ultimate shaft friction can thus be estimated utilizing the intact shear strength of the soil using conventional total or effective stress analysis. This is substantiated by the relatively high shaft friction interpreted from our results below a depth of 8m.

An examination of the results suggests that the 0.7 reduction applied to the cased portion of the pile shaft appeared to be justifiable. When utilizing the derived ultimate skin friction value of 195 kPa, the resulting load carried by the 1.1m of pile shaft in contact with the soil above the bell for Pile No. 2185 was approximately 350 kN. This is in good agreement with the range estimated based on the shape of the initial portion of the load movement curve.

The use of the piezocone on this project provided invaluable information which would likely have been missed using conventional methods. The piezocone properly identified the occurrence of a weak layer from 6.2 to 8m which was not identified during the initial borings. The lower than expected test results from Belled Pile No. 2237, combined with the piezocone information helped to properly identify inadequate embedment into the bearing stratum as one of the reasons for the low results. A second pile was tested at a lower depth which confirmed that a higher capacity could be realized with deeper embedment into the bearing stratum.

As can be seen from the wide range of values presented in Table 3 and Table 4, the use of commonly found values presented in the literature should be done with caution as they may not apply to the soil conditions encountered locally. Where warranted, pile load testing should be undertaken to optimize foundation design. In-situ and laboratory testing should accompany the pile load testing in order to get the full benefit of the results.

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