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Challenges from North to South
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Full-scale Pile Loading Tests on Instrumented Concrete Piles in Clay Till, in Edmonton, Alberta

Xiaobo Wang, Robin Tweedie, Renato Clementino,
Thurber Engineering Ltd., Edmonton, Alberta, Canada

ABSTRACT
Cast-in-place concrete belled piles founded in clay till are commonly used for support of heavily loaded foundations in the Edmonton area. Full-scale pile loading test on instrumented piles not only verifies load-carrying behavior and performance of the piles but also allows for the break-down of shaft and toe resistances for the pile design. Based on the results of the pile loading test, an increase of geotechnical resistance factor from 0.4 to 0.6 can be applied in the geotechnical resistance design. For projects with a large number of piles required, the overall benefit gained from the increase in design reliability and the reduction in total piling costs can be substantial. This paper presents the results of three full-scale pile loading tests on instrumented cast-in-place concrete belled piles constructed in three separate project sites in the downtown area of Edmonton, Alberta. The shaft diameters of the test piles ranged from 0.9 m to 1.2 m, bell diameters from 1.8 m to 2.7 m, and test loads were up to 9,000 kN. Embedded strain gauges and tell-tales were installed in the test piles, which provided a reliable assessment on the shaft and toe resistances. Considerable variability in the test results was identified at the test sites and the design implications are discussed in the paper.

RÉSUMÉ
L'utilisation des pieux forés à base évasée comme fondation profonde pour les structures assujetties à des charges importantes est une pratique courante dans la région d'Edmonton dans les tills argileux. Des essais de chargements pratiqués sur des pieux instrumentés à échelle réelle permettent non seulement d'analyser le méchanisme du transfert de la charge, mais aussi de décortiquer les composantes de la résistance le long du fût et à la pointe lors de la conception. Les données obtenues lors de ces essais permettent aussi d'accroître le facteur de la résistance géotechnique de 0,4 à 0,6 lors de la détermination de la résistance géotechique du pieu. Dû à l'accroissement de la fiabilité lors de la conception, pour les projets de grande envergure comportant beaucoup de pieux, ces essais de chargement présentent un avantage considérable, ainsi qu'une réduction potentielle importante du coût total des pieux. Cet article présente les résultats de trois essais de chargement, réalisés à pleine échelle, sur des pieux tubulaires à base évasée (béton coulé en place) instrumentés sur trois sites distincts au centre-ville d'Edmonton. Ces pieux d'essais ont été construits avec des fûts à diamètre variant de 0,9 m à 1,2 m, des bases variant de 1,8 m à 2,7 m de diamètre et soumis à des chargements d'essais allant jusqu'à 9000 kN. Des jauges de déformation et des tassomètres ont été placés dans le béton des pieux, ce qui a permis d'établir une interprétation fiable de la résistance le long du fût ainsi qu'à la pointe. Cet article comporte aussi une discussion sur la variation considérable des données recueillies, ainsi que les implications engendrées sur la conception des pieux de production.

1 INTRODUCTION

With proper design and construction, cast-in-place concrete piles can provide a highly effective foundation option for circumstances where conventional spread footings may not satisfy the design requirements. Although analytical methods used in pile design and the application of new technologies to pile construction have advanced substantially in recent decades, the estimate of bearing capacity in actual pile design practice still relies heavily on empirical correlations. The performance of cast-in-place piles is highly dependent upon the geotechnical properties of foundation soils and details of construction procedures. This makes it difficult to accurately predict the bearing capacity of the piles at the Ultimate Limit States and their load-displacement behavior at the Serviceability Limit States.

For situations where ground conditions are complex, cost and consequence of foundation failure are significant, or there is little experience of similar piling work, pile loading tests are essential to validate the design before construction. While fulfilling its traditional role in design validation, pile load testing also provides opportunities to improve the accuracy of the prediction of pile performance and to optimize the foundation design. Full-scale pile loading test performed on actual field conditions significantly reduces the risk of pile design by investigating uncertainties associated with ground conditions, piling contractor's experience, and construction methodology. The use of higher geotechnical resistance factors is therefore justified when pile loading tests are performed at the project site. Based on the results of the pile loading tests, an increase of geotechnical resistance factor from 0.4 to 0.6 can be used to calculate factored geotechnical resistance in pile design as per the National Building Code (2010).

Pile load testing on instrumented test piles provides load transfer information along the pile shaft and toe. The shaft and end-bearing resistances can be back-calculated to validate the geotechnical parameters and to optimize the pile design. The test pile is typically instrumented with telltales and stain gauges located at several pre-determined depths to measure the axial strains and pile compression under the applied load. Load distribution along the pile can be determined from the strain measurements.

This paper presents a summary of three full-scale pile load tests on instrumented cast-in-place concrete belled piles founded in clay till in Edmonton downtown area. The shaft diameters of the test piles ranged from 0.9 m to 1.2 m, bell diameters from 1.8 m to 2.7 m, and test loads were up to 9,000 kN. The paper describes the soil conditions at each test site and provides interpretation and discussion of the test results.

2 SUBSURFACE CONDITIONS

2.1 General Geological Setting

The regional geology of the Edmonton area has been well described by Kathol and McPherson (1975). The main stratigraphic units in the area comprise a succession of glacial deposits over Empress Formation sands overlying bedrock of Edmonton Formation. Glaciolacustrine clay and silt, and glacial till form the glacial deposits. The glacial till is classified as the over-consolidated sandy or silty clay and contains numerous sand and silt lenses, coal, shale fragments, and stones of sizes varying from fine gravel to boulder. Empress sands are of preglacial age consisting of very dense clayey sand and sandy gravel. The bedrock of Edmonton Formation comprises interbedded clay shale, siltstone, and sandstone. Coal and thin bentonite seams are often encountered within the bedrock strata.

The three pile load test sites are located in the central area of Edmonton, as shown on Figure 1. The results of the site-specific geotechnical investigation programs indicated that the main soil strata at the three test sites are typical of those encountered at the central Edmonton area. Based on findings of the geotechnical investigations and pile construction reports, the subsurface conditions at each test location are briefly described in Sections 2.2 to 2.4.

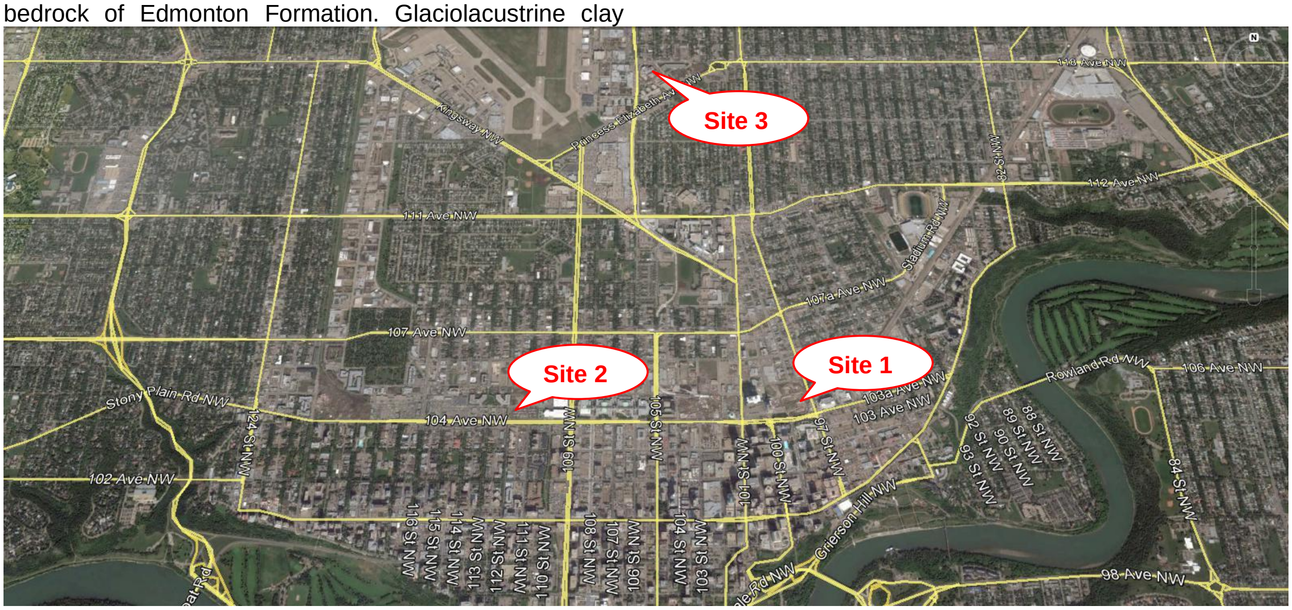

Figure 1. Site plan showing approximate test pile locations

2.2 Site 1 – 97 Street and 103A Avenue

Site 1 was located on the northwest corner of 97 Street and 104 Avenue. The subsurface conditions at the test pile location consist of 0.5 m thick clay fill over glaciolacustrine clay extending to a depth of 5 m from the ground surface. The clay was underlain by glacial clay till that extended to a depth of 18 m. Empress Formation sands and gravels were encountered underneath the clay till and extended to the top of the bedrock at a depth of about 28 m. The material properties of clay and clay till are summarized in Table 1.

The short term groundwater monitoring at Site 1 indicated that the groundwater table at the test pile location was relatively deep and below 20 m from the ground surface.

2.3 Site 2 – 112 Street and 104 Avenue

Site 2 was located at the northeast corner of 112 Street and 104 Avenue. At the test pile location, the subsurface conditions consist of 2.6 m thick clay fill over glaciolacustrine clay extending to a depth of 10.2 m from the ground surface. The clay was underlain by glacial clay till. The material properties of clay and clay till are summarized in Table 1.

Base on the short term groundwater monitoring at Site 2, the groundwater table was about 6 to 7.5 m blow the ground surface.

2.4 Site 3 – 103 Street and 108 Avenue

Site 3 was located on the southwest corner of 103 Street and 118 Avenue. At the test pile location, the subsurface conditions consist of 2 m thick gravel and clay fill over glaciolacustrine clay extending to a depth of 6.2 m from the ground surface. The clay was underlain by glacial clay till that extended to a depth of 34.3 m. The clay till was underlain by dense to very dense sand of Empress Formation. The material properties of clay and clay till are summarized in Table 1.

Short term groundwater monitoring at Site 3 indicated that the groundwater table was relatively shallow and varied from 3.5 to 6 m below the ground surface.

Table 1 Material Properties of Clay and Clay Till

Location	Soil Type	Depth (m)	Water Content (%)	SPT N (average)
Site 1	Clay Clay till	0.5 - 5.0 5.0 - 18	24 - 40 9 - 22	6 – 13 (9) 23 – 42 (32)
Site 2	Clay Clay till	2.6 -10.2 10.2 – 13.2	34 - 37 19 - 21	8 – 10 (9) 16 – 29 (23)
Site 3	Clay Clay till	2 - 6.2 6.2 - 34.3	35 - 40 15 - 25	6 – 7(7) 14 – 42 (26)

3 PILE INSTALLATION AND TEST PROCEDURE

3.1 Test Pile Installation

Static axial compressive tests were conducted on prototype cast-in-place concrete belled piles founded in clay till at eat test site. The shaft diameters of the test piles ranged from 0.9 m to 1.2 m, and bell diameters from 1.8 m to 2.7 m. The length of test piles varied from 12 m to 17 m.

Embedded strain gauges were installed at selected depths in each test pile to measure strains along the pile length. The strain measurements were used to determine load distribution along the pile depth and also the amount of load transferred to the pile base. Two telltales were installed in each test pile to estimate pile concrete compression under test load applied at the pile top. Coupled with displacement readings at the pile top, the measurements from the telltales were used to determine the displacement at the pile base.

Schematic sections of the as-built test piles are presented on Figures 2 to 4. In addition to pile dimensions, subsurface conditions observed during pile installation, locations of strain gauges and telltale are also presented on the diagrams.

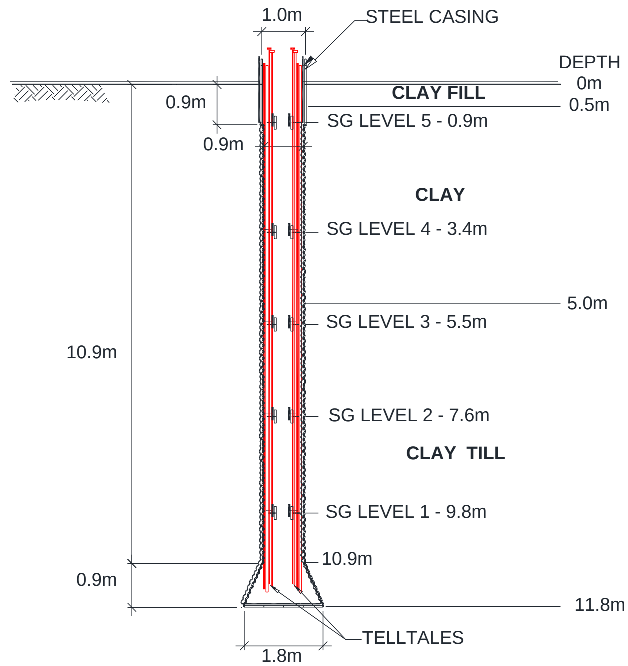

Figure 2 Schematic section of test pile at Site 1

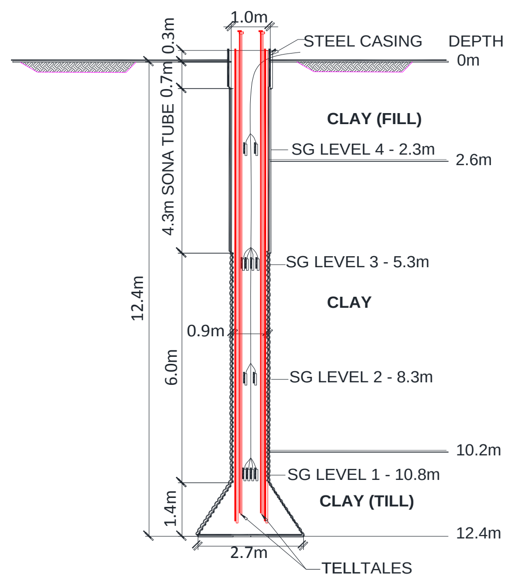

Figure 3 Schematic section of test pile at site 2

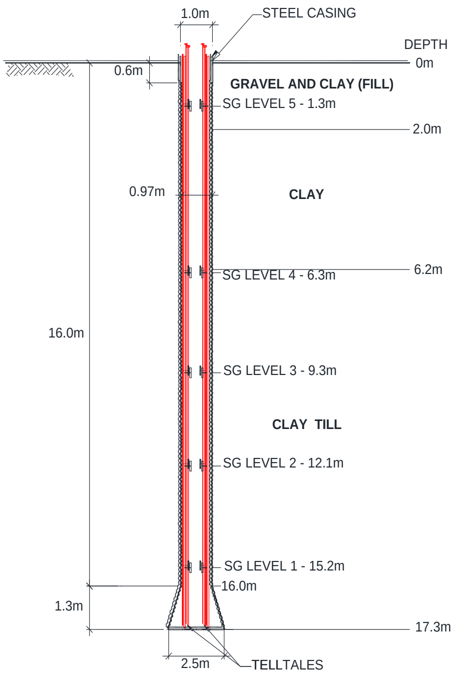

Figure 4 Schematic section of test pile at Site 3

3.2 Test Procedure

The load tests at three sites were performed following Procedure A – Quick Test in accordance with ASTM D1143/D1143M - 07 Standard Test Method for Deep Foundations under Static Axial Compressive Load. The load was applied in increments of 5 percent of the anticipated ultimate capacity of the test pile. Each load increment was held for 8 to 15 minutes. After reaching the maximum test load, the load was removed in 5 to 6 equal decrements. At each load increment or decrement, pile movement indicators were read at 0.5, 1, 2, 4, 8, and 15-minute intervals while the load was held constant. The data logger automatically recorded the strain gauge readings at 30 second intervals throughout the test duration.

4 LOAD TEST RESULTS

4.1 Load-Displacement Behavior

Plots of load versus displacement measured from pile loading tests provide the most useful information for foundation designers to evaluate the overall pile performance. The load-displacement curves for the three pile loading tests are presented on Figures 5 to 7.

As shown on Figure 5, the load-displacement curve for the loading test at Site 1 was nearly linear when load increased from about 2,250 kN to the maximum test load of 9,000 kN. At the maximum test load, the test pile had not reached failure condition. A relatively softer toe response was observed at early stage of the test (test load up to about 2,250 kN). This may be the result of the excavation disturbance to the foundation soils near the pile toe.

In comparison, Figures 6 and 7 clearly indicated that test piles at Sites 2 and 3 were loaded reached geotechnical failure at the maximum test loads.

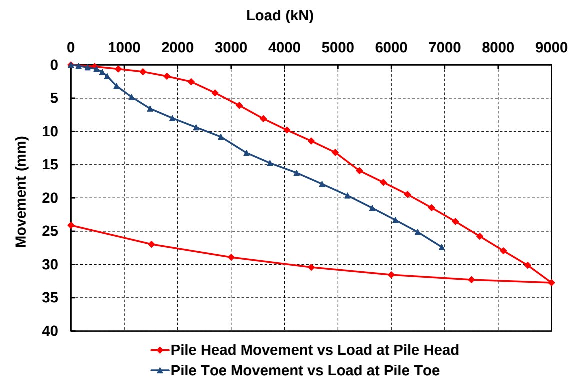

Figure 5 Load displacement curve for test pile at Site 1

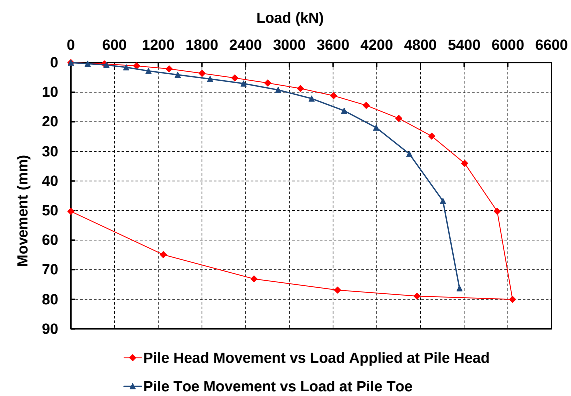

Figure 6 Load displacement curve for test pile at Site 2

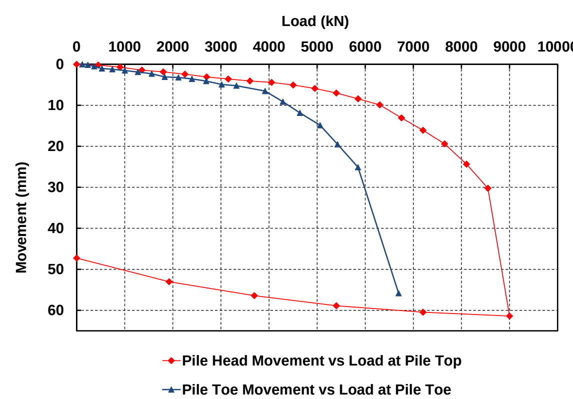

Figure 7 Load displacement curve for test pile at Site 3

4.2 Load Distribution in Test Pile

In order to separate shaft and toe resistances, readings of strain gauges were analyzed to estimate shaft resistance mobilized at each loading level. Conversion of the strain data to loads requires an assessment of pile stiffness (elastic modulus of the pile times its cross sectional area). It has been found that elastic modulus of concrete piles is strain-dependent and using a constant pile modulus to compute shaft resistance often produces an unreasonable load distribution (Fellenius 1989, Lam and Jefferis 2011). Therefore, the tangent modulus approach proposed by Fellenius (1989, 2001) was used to determine load distributions along the pile shaft from strain gauge measurements. The shaft resistance mobilized at each load increment is presented on Figures 8 to 10.

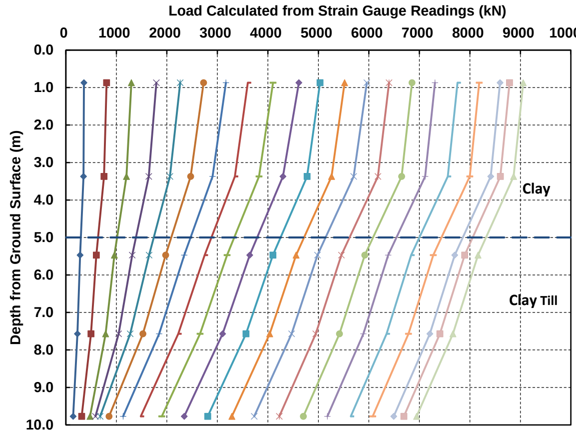

Figure 8 Load distribution along test pile at Site 1

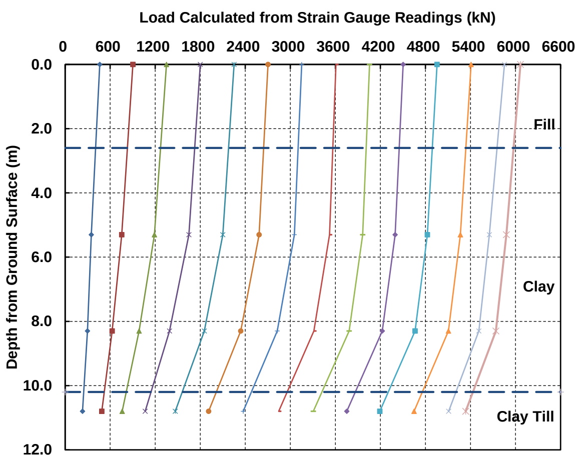

Figure 9 Load distribution along test pile at Site 2

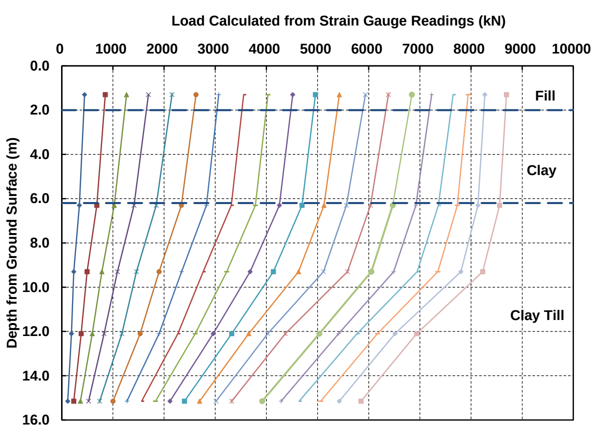

Figure 10 Load distribution along test pile at Site 3

5 ANALYSES AND DISCUSSION

5.1 Shaft and Toe Resistances Mobilized

Shaft and toe resistances were calculated using the strain gauge data collected during the pile loading tests. As shown on Figures 8 to 10, the shaft load reduces with depth in a top-down pile load test due to the shaft resistance provided by the soil. The load difference between two consecutive levels of strain gauges is the amount of shaft resistance over the corresponding pile length. The mobilized unit shaft resistance can be estimated from the load distribution diagram and the diameter of the test pile. The resistance developed at the pile toe can be estimated based on the measurements of the strain gauges installed at the lowest level minus shaft resistance between the top of bell and the lowest strain gauges. The shaft resistance from the top to the bottom of the bell is considered to be negligible. The unit toe resistance can be determined with the cross sectional area of at pile toe.

The mobilized unit shaft resistances computed based on strain gauge measurements from the three pile loading tests are presented on Figures 11 to 13.

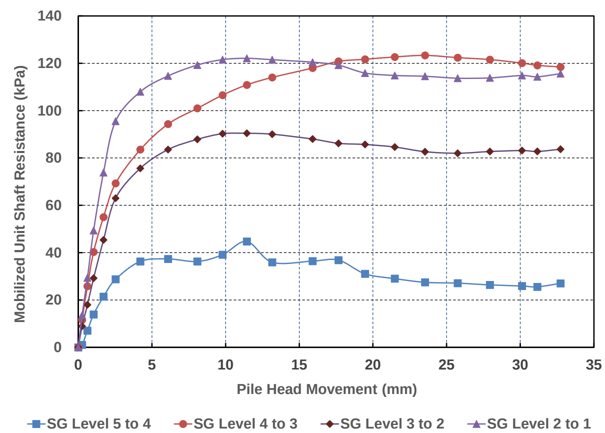

Figure 11 Mobilized unit shaft resistance from pile loading test at Site 1

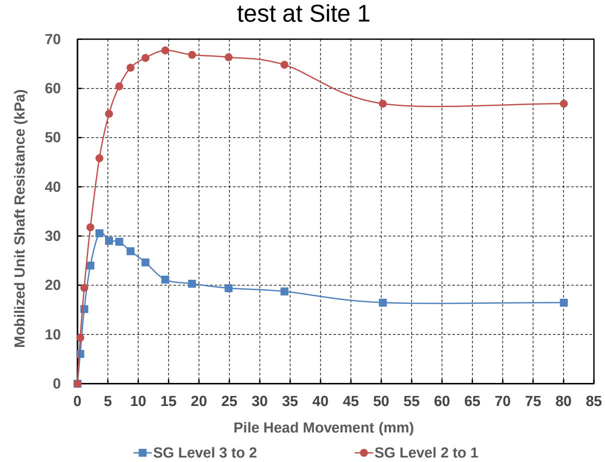

Figure 12 Mobilized unit shaft resistance from pile loading test at Site 2

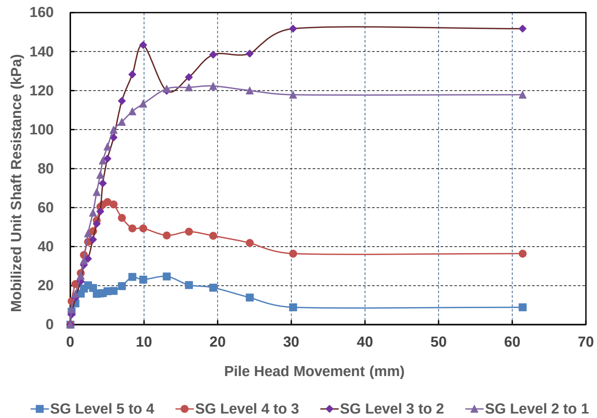

Figure 13 Mobilized unit shaft resistance from pile loading test at Site 3

The average unit shaft and toe resistances are summarized in Table 2 and 3, respectively. The average SPT values are also provided in the tables for comparison.

Table 2 Ultimate Shaft Resistances Estimated from Pile Loading Tests

Location	Depth (m)	Soil Type	Ultimate Shaft Resistance (kPa)	SPT N (Average)
Site 1	0.9 – 3.4	Clay	37	9
	3.4 – 5.5	Clay Clay till	123	-
	5.5 – 7.6	Clay till	90	-
	7.6 – 9.8	Clay till	122	37
Site 2	5.3 – 8.3	Clay	38	9
	8.3 -10.8	Clay	68	10
Site 3	1.3 – 6.3	Clay	20	7
	6.3 – 9.3	Clay till	63	21
	9.3 – 12.1	Clay till	152	34
	12.1 – 15.2	Clay till	121	23

Table 3 Toe Resistances Mobilized in Pile Loading Tests

Location	Toe Resistance (kPa)	SPT N (Average)
Site 1	2,730	31
Site 2	925	23
Site 3	1,190	28

* average SPT values within two bell diameter below pile toe with the exception of Site 2, where only two SPTs were recorded at depth below the pile toe.

As indicated on Figures 11 to 13, ultimate shaft resistance of stiff to hard clay and clay till is generally developed within 5 to 10 mm of pile movement. However, substantial pile movement is required to fully mobilize the ultimate resistance at the pile toe. Figures 6 and 7 indicated that pile movements of about 50 mm and 30 mm were measured at Sites 2 and 3, respectively, when the test piles were loaded to failure.

The shaft resistance estimated from the pile load test at Site 1 is considered to be representative of the ultimate resistance along the pile shaft, while the mobilized resistance at the pile toe is less than the ultimate toe resistance since the test pile had not reached failure at the maximum test load. Plot of load displacement for the test at Site 1 (Figure 5) indicated that the pile head and toe movements at the maximum test load were about 33 mm and 27 mm, respectively. The pile head movement was about 1.8 percent of the pile toe diameter. According to Brown et al. (2010), geotechnical failure load of a drilled shaft founded in cohesive soils can be estimated as the combination of the ultimate shaft resistance and the toe resistance corresponding to a pile top displacement of 4 percent of the shaft diameter, if plunging failure cannot be achieved. The toe diameter of the test pile at Site 1 was 1.8 m, indicating that the pile movement up to 72 mm may be required to fully mobilize toe resistance. It is, therefore, anticipated that the ultimate toe resistance for this pile is considerably greater than the toe resistance mobilized during the test.

The toe resistances estimated from pile loading tests at Sites 2 and 3 were based on the loads at failure conditions. The mobilized resistances are considered to be the ultimate shaft and toe resistances.

5.2 Discussions

Common local practice for the design of end-bearing belled piles founded in stiff clay and clay tills is to adopt the total stress approach as outlined in Canadian Foundation Engineering Manual (CFEM 2006). The ultimate shaft resistance is estimated from the undrained shear strength multiplied by a shaft resistance coefficient (α). The ultimate toe resistance is estimated from the average undrained shear strength of foundation soils within 2 times toe diameter below the pile toe multiplied by a bearing capacity coefficient (N_t). The bearing capacity coefficient is dependent on the pile toe diameter and varies from 6 to 9 (CFEM 2006). In the case that the strength of foundation soils is not measured directly from laboratory or in situ tests, the undrained shear strength is often estimated from the empirical correlations based on the SPT blow counts.

The ultimate shaft and toe resistances measured from the pile load tests are used to calibrate the coefficients required in the total stress pile design approach. The following correlation equation is frequently used to estimate undrained shear strength from SPT N values.

$$s_u/p_r = 6N \qquad [1]$$

where s_u is the undrained shear strength, p_r is a reference pressure and equals to 1 kPa, and N is SPT blow counts.

Based on shaft and toe resistance measured form the pile load test and undrained shear strength estimated from SPT values using equation [1], the pile design coefficients α and N_t are back-calculated and presented in Table 3. Based on the same undrained shear strength values, the α values were also estimated using the equation recommended in CFEM (2006). Figure 14 shows the shaft resistance coefficient α versus undrained shear strength estimated from the SPT values in Table 2.

Table 3 Shaft Resistance Factor (α) and Bearing Capacity Coefficient (N_t) Estimated from Pile Loading Tests

Location	**Soil Type**	**Coefficients in Total Stress Pile Design**	
		α	N_t
Site 1	Clay	0.69	-
	Clay Till	0.55	> 14
Site 2	Clay	0.73 – 1.13	-
	Clay Till	-	6.7
Site 3	Clay	0.48	-
	Clay Till	0.50 – 0.88	7.1

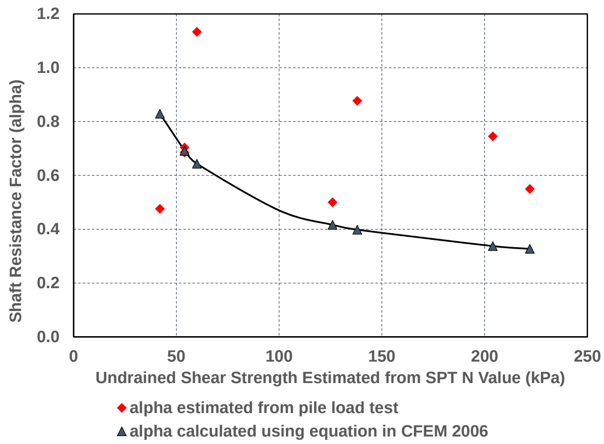

Figure 14 Shaft resistance coefficient vs undrained shear strength

The back-calculated α values from the three pile load tests ranged from 0.48 to 1.13. As shown on Figure 14, the α values back-estimated from the results of pile load tests are generally greater than those recommended in CFEM.

For pile toe diameter greater than 1 m, an N_t value of 6 is recommended in CFEM. In comparison, the coefficient N_t values back-calculated from the pile loading tests at Sites 2 and 3 is about 7. An N_t value of 9 was reported by Ruban and Kort (2011) for a static pile load test on an end bearing pile founded in similar clay till in the Edmonton area. An N_t value greater than 14 was back-calculated from the results of the pile load test at Site 1, indicating that equation [1] may underestimate undrained shear strength of over-consolidated hard clay till.

5.3 Conclusion

Three full-scale pile loading tests were undertaken on instrumented cast-in-place concrete belled piles in Edmonton downtown area. Using readings of strain gauges embedded in the test piles, soil resistances along shaft and at pile toe mobilized at each load level were determined. The full response of the pile to load could established from the results of the test on an instrumented pile.

The load test results indicate that ultimate shaft resistance of stiff to hard clay and clay till is generally mobilized with a pile head movement of 5 to 10 mm. However, considerable movements are required to fully mobilize toe resistance.

None of the three pile load tests had laboratory tests to estimate the strength of foundation soils. Undrained shear strength were estimated from SPT values using a commonly adopted empirical equation. The estimated shaft resistance coefficient α values are generally greater than those recommended in the literature. An N_t value of about 7 was back-estimated from two pile load test where ultimate toe resistance were measured.

The geotechnical parameters developed from pile loading tests on prototype piles are based on the actual ground conditions and construction procedures. The measured shaft and toe resistance from pile loading tests are considered to be applicable for the pile design. With the increase of geotechnical resistance factor from 0.4 to 0.6, the factored geotechnical resistance may increase substantially, resulting in significant cost savings to the project.

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