Pile load testing of concrete belled pile and rock socket pile using the Osterberg load cell

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ABSTRACT
Cast-in-place concrete piles are a commonly used foundation type in northern Alberta due to the cohesive soils predominant in the area. Although cast-in-place concrete piles are common, limited pile load testing has been undertaken on these foundation types to confirm shaft friction (side shear) and end bearing parameters. In order to optimize pile foundation design and permit use of higher soil resistance factors for Limit States design, full-scale pile load tests were undertaken on two different pile foundation types. One load test was undertaken on a belled pile end bearing in glacial clay till and a second load test on a straight pile socketed in weak clay shale bedrock. An Osterberg load cell was used in both load tests to provide end bearing and shaft friction parameters for the clay, clay till and clay shale bedrock. In addition to confirming ultimate design parameters, the results of the load test provided valuable insight to the load settlement performance exhibited by these piles and enabled a more economical pile design.

RÉSUMÉ
Coulé en place des pieux en béton sont un type de fondation couramment utilisés dans le nord de l'Alberta en raison de la cohésion des sols qui sont prédominants dans la région. Bien que les pieux en béton coulé sur place sont communes, limitée essais de chargement de pieux ont été entrepris sur ces types de fondation pour confirmer frottement sur le fût et les paramètres de la résistance de pointe. Pour la conception de pieux de fondation d'optimiser et de permettre l'utilisation de la hausse des facteurs de résistance du sol pour les états limites de conception, les essais sur l'échelle de chargement de pieu ont été réalisées sur deux types différents de fondation sur pieux. Un test de charge a été menée sur un palier extrémité de la pile grelot dans l'argile du till et un test de charge secondes sur un tas droite encastrés dans le roc de schiste argileux faibles. Une cellule de pesage Osterberg a été utilisé dans les deux essais de charge pour fournir un cap fin et les paramètres de frottement sur le fût de l'argile, l'argile et d'argile jusqu'à ce roc de schiste. En plus de confirmer les paramètres de conception finale, les résultats du test de charge fourni de précieux renseignements à la performance règlement des charges exposées par ces piles et a permis une conception de pieux plus économique.

1 INTRODUCTION

Anthony Henday Drive (AHD) is a perimeter ring road that is being constructed around the city of Edmonton, Alberta. To date, the southwest and southeast legs of AHD have been constructed. The northwest leg of AHD is currently under construction and is scheduled to be completed in the fall of 2011. Northwest AHD (NWAHD) comprises 21 km of new, divided, limited access highway including 8 new interchanges and 29 separate grade separation structures. The west end of the alignment starts at Yellowhead Trail, and the east end terminates at Manning Drive in the northeast part of Edmonton (Figure 1).

EBA, A Tetra Tech Company (EBA), has provided geotechnical engineering for the design-build contractor's engineering team consisting of Flatiron Constructors, Graham Infrastructure and Parsons Construction. Early in the design stages of the project, the design team was tasked with providing innovative solutions for optimizing foundation design for the project. One option that was proposed included performing pile load tests at select bridge locations. As the bridge design was undertaken in accordance with Limit States Design, the advantages of conducting a load test included the ability to adopt a soil resistance factor of 0.6 for the design of the bridge foundations, compared to a soil resistance factor of 0.4 if no testing was undertaken. There was also the possibility of increasing the ultimate soil strength parameters, depending on the results of the load test.

During the preliminary design, cast-in-place concrete piles were identified as a feasible pile foundation type for many of the bridge structures. Cohesive soils, including lacustrine clay, clay till and clay shale, are common across Alberta, which has led to the use of cast-in-place concrete piles as a cost-effective pile foundation type. After review of the subsurface conditions at all the bridge sites, two locations were identified as being favourable candidates for performing pile load tests.

The two locations selected for pile load tests included the proposed Campbell Road and 127 Street interchanges. Both sites are identified on Figure 1. Site-specific subsurface conditions for each location are outlined in the following sections.

2 SUBSURFACE CONDITIONS

2.1 Regional Geology

The surficial geology of the Edmonton area has been well documented by Kathol and McPherson (1975). Prior to glaciation, the surface of the Edmonton area consisted of

Figure 1. Site plan and test pile locations

a well-developed drainage system, which is similar to that of today. The preglacial valleys were generally wide and drained to the northeast. Alluvial sediments deposited by preglacial rivers are known locally as the Saskatchewan Sands and Gravels.

During glaciation, till sheets were deposited over the Saskatchewan Sands and Gravels and the bedrock (where exposed). As the glaciers retreated to the northeast, they stagnated north of Edmonton resulting in the formation of a large proglacial lake called glacial Lake Edmonton. Meltwater from the ice sheet deposited sediment into this glacial lake. The glacial lake deposits extend over most of the Edmonton area and comprise fine-grained glaciolacustrine sand, silt and clay.

2.2 127 Street Interchange

Drilling at this site was undertaken in the summer of 2008 using a truck-mounted dry auger drill equipped with both solid and hollow stems augers. Sampling comprised disturbed auger samples supplemented by Standard Penetration Tests (SPT) at 1.5 m intervals. Relatively undisturbed Shelby tube samples were taken at select depths within the overburden sediments.

Drilling was undertaken at the proposed abutment and pier locations. Due to grading activities at the site, the proposed test pile location was situated near the toe of the north abutment. One borehole was drilled within a few metres of the proposed test pile location. Open standpipe piezometers were installed in all borehole locations except the test pile location, where two vibrating wire (VW) piezometers were installed.

The general subsurface stratigraphy comprised glaciolacustrine clay and silt to a depth of 8.5 to 9.9 m, overlying glacial clay till that extended to a depth of 30 to 33 m, which was underlain by Edmonton Formation clay shale and sandstone bedrock. At the test pile location, the high plastic clay was silty, stiff, moist with numerous layers and lenses of interbedded silt and sand. SPT blow counts varied from 4 to 7 blows per 300 mm. Three unconsolidated undrained triaxial tests from the clay indicated compressive strengths of 86 to 113 kPa.

The clay till is typically silty, sandy, damp to moist, medium plastic, with traces of gravel and coal particles. SPT values within the clay till generally ranged from 20 to 24 blows per 300 mm at the test pile location. Three unconsolidated undrained triaxial tests from the till strata indicated compressive strengths of 263 to 318 kPa. Sand and coal layers were identified randomly throughout the clay till.

Rafted layers of clay shale bedrock varying from less than 1 m to 8.6 m were encountered in the glacial clay till. At the test pile location, rafted clay shale was identified at a depth of 22.1 m. The test pile location borehole was terminated in rafted clay shale at a depth of 25.7 m. The stratigraphy at the test pile location including blow counts (N values) is presented on Figure 2.

Based on the VW piezometers, the groundwater table within the clay was at a depth of 1.2 m and within the clay till at a depth of 3.0 m.

2.3 Campbell Road Interchange

The same general drilling and sampling procedure as described in Section 2.2 was implemented for the proposed Campbell Road interchange. A total of four boreholes were drilled at this site. Due to site grading at the time of test pile installation, the test pile was located at the toe of the south abutment.

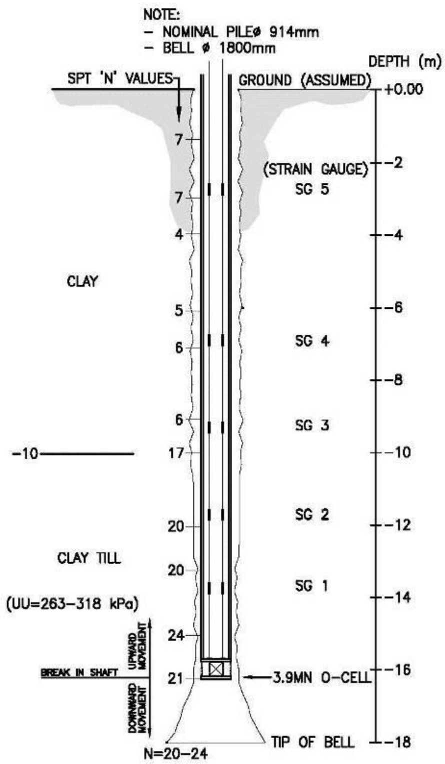

Figure 2. Test Pile Instrumentation – 127 Street

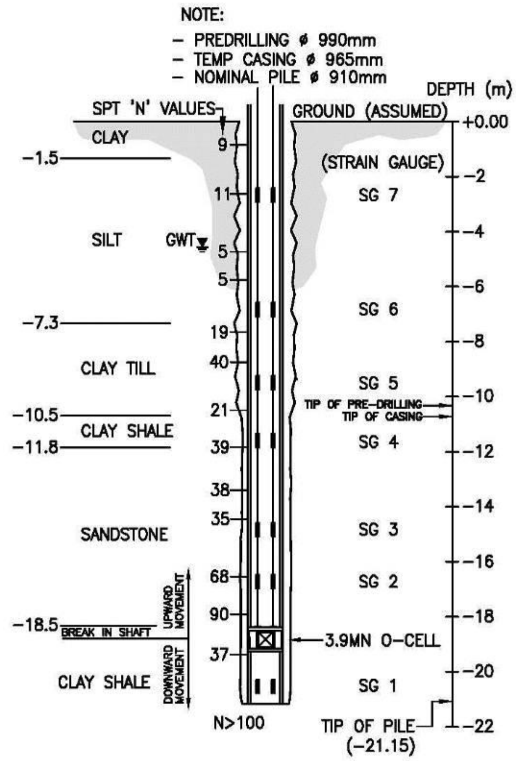

Figure 3. Test Pile Instrumentation – Campbell Road

The subsurface stratigraphy at the test pile comprised glaciolacustrine clay and silt to a depth of 7.5 m, overlying glacial clay till that extended to a depth of 10.5 m, which in turn was underlain by Edmonton Formation clay shale and sandstone bedrock. High plastic firm to stiff clay (N = 9) was encountered near the surface, grading into a loose silt with SPT N values of 5 to 11 per 300 mm.

The upper bedrock was weathered from 11.5 to 20 m with SPT N values ranging from 35 to 90 blows per 300 mm. Below a depth of 20 m, the bedrock was relatively competent (for the local geological conditions) with N values over 100 blows per 300 mm.

Based on the VW piezometers, the groundwater table within the clay was at a depth of 4.2 m and within the clay shale bedrock at a depth of 5.8 m.

3 TEST PILE INSTALLATION

3.1 127 Street Interchange

The test pile at the 127 Street site comprised a 914 mm diameter cast-in-place concrete pile with an 1,800 mm diameter bell. The pile bore was started by pre-drilling with a 990 mm diameter auger prior to inserting a 965 mm diameter temporary steel casing to a depth of 11.7 m. The casing was installed to control seepage and sloughing from the glaciolacustrine clay and silt. A 914 mm diameter auger was used to drill to a depth of 18.0 m and a belling tool was used to construct the nominal 1,800 mm diameter bell and clean the base.

After cleaning the base, the carrying frame with attached Osterberg (O-cell) assembly (see Photo 1) was inserted into the pile bore and temporarily supported from the casing. Concrete was delivered by pump through a tremie pipe to the base of the pile until the top of the concrete reached near the ground surface. The contractor removed the temporary casing immediately after concrete placement. Some water had accumulated in the pile base; however, the water was displaced by the concrete tremied into the pile. Test pile installation was completed in November 2008.

Photo 1. Osterberg Load Cell

3.2 Campbell Road Interchange

The test pile at the Campbell Road site comprised a 914 mm diameter cast-in-place concrete rock socket pile drilled to a depth of 21.15 m. The pile was started by pre-drilling with a 990 mm diameter auger prior to inserting a 965 mm diameter temporary steel casing to a depth of 10.7 m. The casing was installed to control seepage and sloughing from the glaciolacustrine clay and silt. A 914 mm diameter auger was used to drill to a depth of 21.15 m, and a belling tool was used to clean the base.

Similar to the procedure used for the belled pile at 127 Street, the base was cleaned, the O-cell assembly inserted and temporarily suspended from the surface with the O-cell positioned at the desired elevation. Concrete was then placed through a tremie pipe lowered to the tip of the pile bore. The contractor removed the temporary casing immediately after concrete placement. The pile bore remained dry during the entire installation procedure. Test pile installation was completed in November 2008.

4 OSTERBERG CELL TESTING

Details of the pile load test procedure and test results are presented in two reports prepared by LOADTEST Inc. (2008a, 2008b). The following presents a summary of both pile load tests.

4.1 Instrumentation - 127 Street Interchange

The loading assembly consisted of a 3.9 MN, 330 mm diameter O-cell located 1.52 m above the tip of pile. O-cell testing instrumentation included three Linear Vibrating Wire Displacement Transducers (LVWDTs) positioned between the lower and upper plates of the O-cell assembly to measure expansion. Two telltale casings (nominal 12 mm steel pipe) were attached to the carrying frame, diametrically opposed, extending from the top of the O-cell assembly to beyond the top of concrete.

Strain gauges were used to assess the side shear load transfer of the pile above the O-cell assembly. Five levels of two diametrically opposed vibrating wire embedment strain gauges were installed in the pile above the base of the O-cell assembly. Two lengths of steel pipe were also installed, extending from the top of the pile to the top of the bottom plate, to vent the break in the pile formed by the expansion of the O-cell. Figure 2 presents a schematic of the test pile installation.

Pile compression was measured using 6 mm telltales installed in the 12 mm steel pipes and monitored by LVWDTs. Two automated digital survey levels (Leica NA 3000 Series) were used to monitor the top of pile movement from a distance of approximately 4.7 m.

A Bourdon pressure gage and electronic pressure transducers were used to measure the pressure applied to the O-cell at each load interval. The transducers were used for automatically setting and maintaining loads, for data analysis and real-time plotting.

All instrumentation was connected through a data logger to a laptop computer allowing data to be recorded and stored automatically at 30-second intervals and displayed in real time. The same laptop computer synchronized to the data logging system was used to record the survey data.

4.2 Pile Instrumentation - Campbell Road Interchange

A similar arrangement as described above was used for the Campbell Road site. A loading assembly comprising a 3.9 MN, 330 mm O-cell was located 2.27 m above the pile tip. Six levels of two diametrically opposed embedment strain gauges were installed in the pile above the base of the O-cell assembly, and one level of two strain gauges was installed below the O-cell.

Pile compression and top of pile movement were recorded with 6 mm telltales monitored by LVWDTs and automated digital survey levels, respectively. Pressure applied to the O-cell was monitored with a Bourdon pressure gauge and electronic pressure transducers. All instrumentation was connected through a data logger to a laptop computer allowing data to be recorded and stored automatically.

4.3 Load Test Procedures

A similar testing procedure was used for both sites. Load increments were applied using the Quick Load Test method for Individual Piles, in accordance with ASTM D1143, holding each load increment for 8 minutes. Each load increment was approximately 5% of the estimated ultimate pile capacity. The data logger automatically recorded the instrument readings every 30 seconds.

The 127 Street test pile was loaded in 17 increments to a bi-directional gross load of 3.28 MN. The loading was halted because the O-cell was approaching its maximum stroke. The pile was then unloaded in 4 decrements and the test concluded.

The Campbell Road test pile was loaded in 23 increments to a bi-directional gross load of 4.43 MN. The load test was partially interrupted as high winds blew away the temporary test hoarding. The pile was partially unloaded, then reloaded and the load taken to 5.23 MN. The pile was then unloaded in 4 decrements.

5 LOAD TEST RESULTS AND ANALYSIS

The load-displacement behavior recorded during the test was analyzed both in its individual components and in its recombined state to assess the pile performance as an integrated whole. The loads applied by the O-cell act in two opposing directions, resisted by the capacity of the pile above and below. Theoretically, the O-cell does not impose an additional upward load until its expansion force exceeds the buoyant weight of the pile above the O-cell. Therefore, net load, which is defined as gross O-cell load minus the buoyant weight of the pile above, is used to determine side shear resistance above the O-cell and to construct the equivalent top-loaded load-settlement curve.

Throughout this paper the term “side shear” is used to describe the mobilized shaft resistance along the pile perimeter rather than the more common term “skin friction”. This terminology is considered more technically correct as the resistance along the perimeter of shaft is composed of components of not only the friction at the interface of the soil and concrete but also the shear strength of the soil itself.

In order to assess the side shear resistance of the test pile, loads are calculated based on the strain gauge data and estimates of composite pile modulus. Calculating load from the strain measurements proved to be complicated as applying a constant pile modulus to the strain measurements produced an unreasonable load distribution. Therefore, an approach to determine a strain-dependent pile modulus recommended by Fellenius (2001) was adopted to convert strain to load.

5.1 127 Street Interchange

For the load test undertaken at 127 Street, the maximum upward applied net load to the upper side shear was 3.07 MN. At this loading, the upward movement of the O-cell top was 18.75 mm. Results of the load displacement curve for side shear and end bearing resistance are presented on Figure 4.

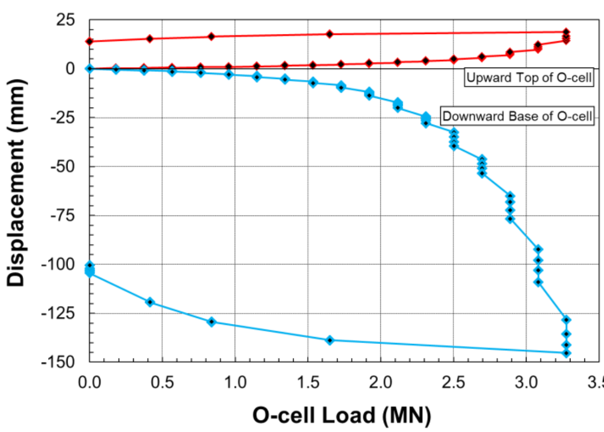

Figure 4 Osterberg Cell Load-Displacement for Belled Pile

Net unit side shear curves developed from the strain gauges readings are presented on Figure 5 and mobilized side shear resistance values are presented on Table 1.

Table 1. Mobilized Side Shear Resistance – 127 Street

Load Transfer Zone	Soil Strata	Unit Side Shear (kPa)
0 to 3 m	Clay	41
3 to 7 m	Clay	33
7 to 9.5 m	Clay	60
9.5 to 12 m	Till	55
12 to 14 m	Till	96
14 to 16.5 m	Till	119

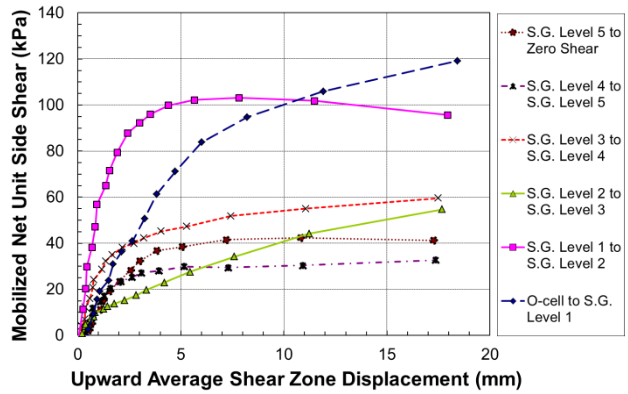

Figure 5. Mobilized Side Shear Resistance for Belled Pile

The maximum O-cell load applied to the combined side shear and end bearing of the pile below the O-cell was 3.28 MN. At end of pile loading, the average downward movement of the O-cell base was 145 mm. Assuming the entire applied load was transferred to the base without any side shear resistance, the unit end bearing on the pile base was calculated to be 1287 kPa. The unit end bearing curve for the belled pile is illustrated on Figure 6.

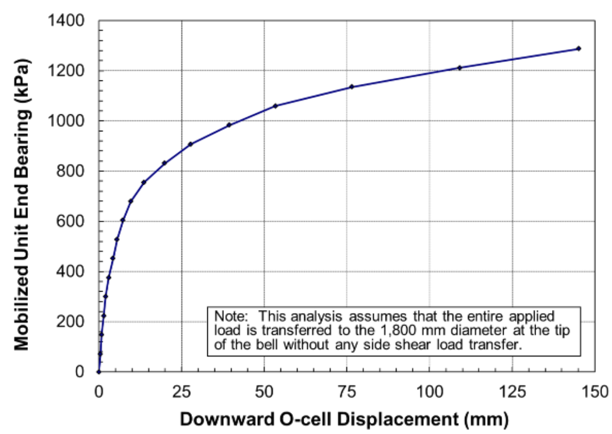

Figure 6. Mobilized End Bearing for Belled Pile

The results of the side shear resistance from above the O-cell and end bearing resistance from below the O-cell were combined to generate an equivalent top loaded load-displacement curve, which is depicted on Figure 7. The total displacement curve has been adjusted for the additional elastic compression of the pile that would occur if the load applied below the O-cell were applied at the pile head. Separate curves illustrating the development of side shear and end bearing resistance with pile head displacement are also presented on Figure 7. To demonstrate the ultimate pile top load capacity, the load-displacement data from the pile section above the O-cell has been extrapolated to the maximum measured end bearing displacement of 150 mm.

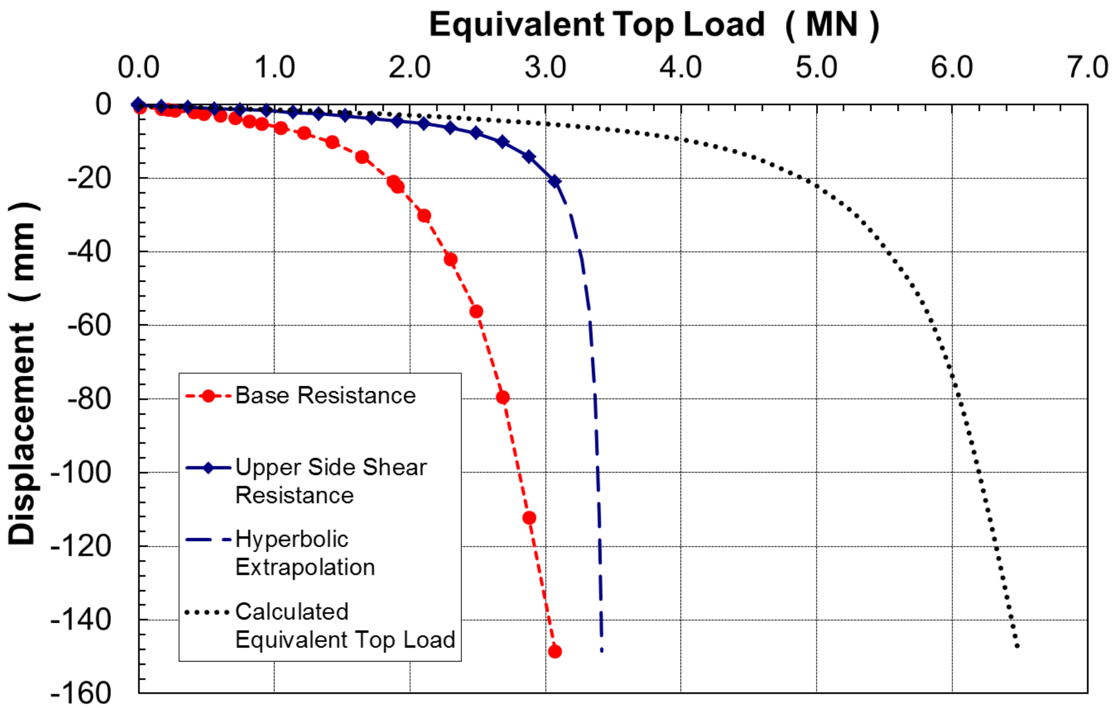

Figure 7. Load/Displacement Curve for Belled Pile

5.2 Campbell Road Interchange

At the Campbell Road site, the maximum O-cell load applied to the combined end bearing and lower side shear was 5.26 MN. At this load, the average downward movement of the O-cell base was 85 mm. Deducting the side shear resistance mobilized below the O-cell, the applied load to the pile tip was 4.15 MN. The unit end bearing resistance at the base of the pile was calculated to be 6,041 kPa. Results of the load displacement curve for side shear and end bearing for the rock socket pile are presented on Figure 8.

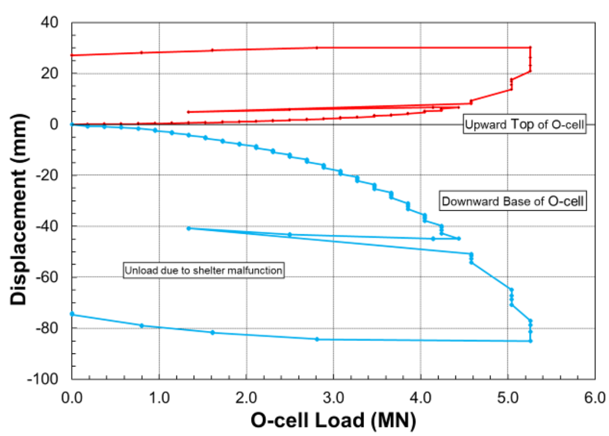

Figure 8. O-Cell Load-Displacement for Rock Socket Pile

The maximum upward applied net load to the Campbell Road Interchange test pile was 5.03 MN. At this load, the upward movement of the O-cell top was 30 mm. Mobilized unit side shear values are presented on Table 2.

Table 2. Mobilized Side Shear Resistance – Rock Socket Pile

Load Transfer Zone	Soil Strata	Unit Side Shear (kPa)
0 to 3 m	Clay	48
3 to 7 m	Clay	40
7 to 10 m	Till	82
10 to 12 m	Weathered Bedrock	120
12 to 15 m	Weathered Bedrock	152
15 to 17 m	Weathered Bedrock	124
17 to 19 m	Weathered Bedrock	95
19 to 20.5 m	Weathered Bedrock	166

Figure 9 presents the equivalent top load displacement curve for the rock socket pile along with the separate curves illustrating the development of side shear and end bearing resistance. As with the belled pile, the upper pile section load displacement data was extrapolated to demonstrate the top load performance to the maximum measured end bearing displacement of 90 mm.

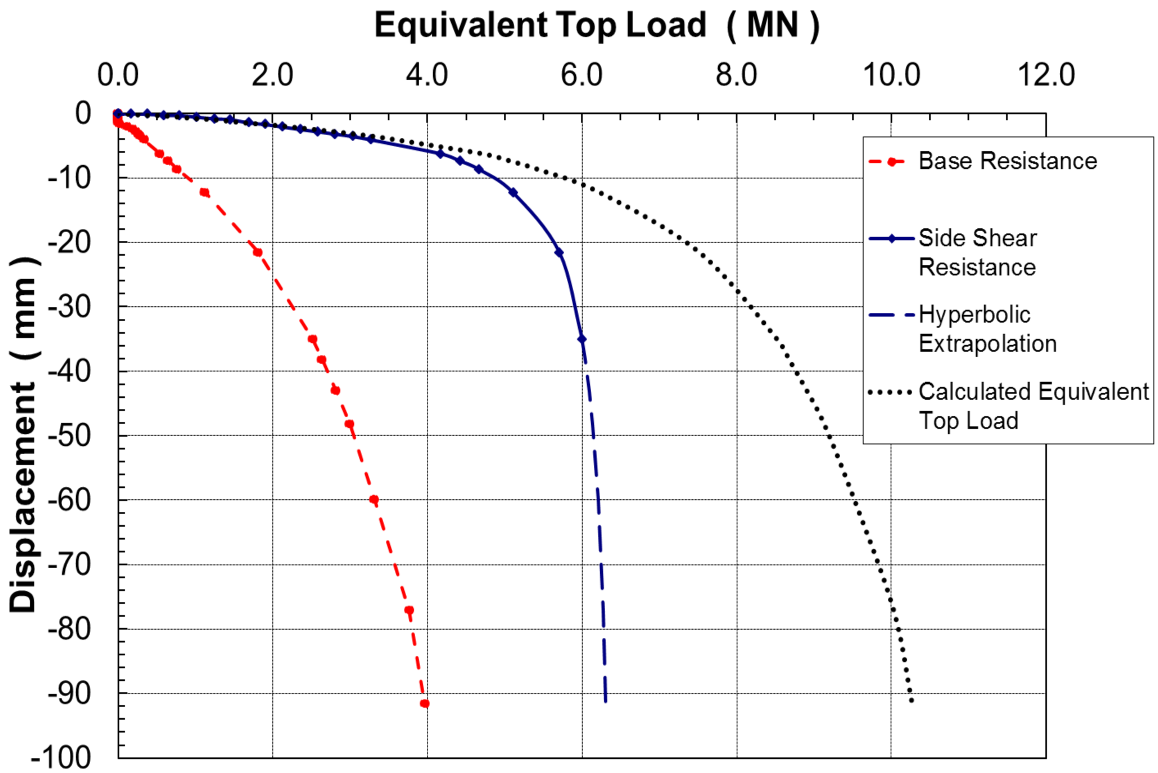

Figure 9. Load/Displacement Curve for Rock Socket Pile

6 DISCUSSION

6.1 Mobilization of Resistance

Some of the more interesting elements of the analysis of both data sets are centered on the development of pile component resistance with displacement. Reviewing Figure 5, it is noted that, while full side shear resistance is developed within 5 to 10 mm of pile displacement, significant pile displacement is required to fully develop end bearing resistance (Figure 6).

The different rates of component resistance mobilization provide insight into how service loads would be carried in the production piles. Reviewing Figure 7, it is of interest to note that the majority of the side shear resistance was fully developed with a pile head movement of 20 mm, while 150 mm of pile deformation was required to develop the same level of resistance in end bearing. At 5 mm of pile head movement, the shaft provided 70% of the pile resistance. At 10 mm of deformation, the shaft provided 66% of the total pile resistance. By 20 mm of movement, the shaft still provided 62% of the total pile capacity.

Figure 9 (for the rock socket pile) illustrates the same general trend as the belled pile: that the side shear resistance develops much more rapidly than the end bearing. At 6 mm of pile settlement, 90% of the total load is carried in the shaft. At 12 mm, the shaft is carrying 82% of the total pile capacity. With 20 mm of pile head movement, the shaft provides 76% of the total pile resistance.

6.2 Design Implications

Data from both tests were used to define ultimate pile capacities and enable design optimization. From Figure 7, the ultimate pile capacity can be estimated to be approximately 6.5 MN. Given a soil resistance factor of 0.6, the factored geotechnical resistance for this pile would be 3.9 MN. At this load level, the estimated pile settlement would be approximately 8 mm, which is considered tolerable for most structures. Most piles rarely experience a condition where they are subjected to the full factored load. In most cases, piles rarely exceed the unfactored design load, which for this pile would be approximately 2.9 MN. At this stress level, the pile deformation would be less than 5 mm.

A similar trend is displayed with the rock socket pile (Figure 9). The ultimate pile capacity is estimated to be 10.2 MN. With a soil resistance factor of 0.6, the factored geotechnical resistance for this pile would be 6.1 MN. From the total load settlement curve, the corresponding settlement would be approximately 12 mm. The unfactored load on this pile would be approximately 4.5 MN, which corresponds to a pile settlement of approximately 6 mm.

Common local practice for end bearing design of belled piles in till is to adopt an undrained analysis approach as per the Canadian Foundation Engineering Manual (2006). Utilizing an average undrained shear strength of 145 kPa from the UU tests and an N_t value of 9, the estimated ultimate end bearing for the clay till is

1300 kPa. This value is a good agreement of the ultimate end bearing determined from the belled pile load test. It should be noted that CFEM recommends use of an N_t value of 6 for pile diameters greater than 1 m; however, this load test indicates that use of an N_t value of 9 is applicable for the local clay till.

6.3 Comparison with Other Tests

A similar straight shaft bored pile into the bedrock was load tested on the Southeast Anthony Henday Drive (SEAHD) project (Skinner et al., 2008) in Edmonton. The results of the load test that extended into bedrock indicated an ultimate end bearing of approximately 2,000 kPa, which is substantially less than the end bearing determined from the current study. It is speculated that the difference in end bearing between the current study and the SEAHD project is due to the fact that the pile tip was bearing in the upper weathered clay shale and did not extend into the underlying more competent bedrock.

To the author's knowledge, only one other pile load test on an instrumented rock socket pile has been performed in the Edmonton area. Results from that pile load test had indicated an ultimate end bearing in the competent bedrock of approximately 5 MPa. The load test from this project has provided a greater understanding of pile performance and has confirmed that design values historically used for end bearing design of rock socket piles in the local more competent clay shale bedrock.

7 CONCLUSION

Limited numbers of pile load tests have been conducted for rock socket piles or belled concrete piles in the Edmonton area. Occasionally, pile load tests may have been undertaken; however, instrumentation of the piles has rarely been incorporated. Therefore, the development of the side shear and end bearing resistance in those pile tests was not well understood. The two O-cell load tests discussed herein have provided valuable insight into the performance of these pile types.

The load test results illustrate that for these types of cast-in-place concrete piles, minimal pile deformation results in significant pile resistance. Under typical service or unfactored loading conditions, pile deformation for piles with relatively long shafts is expected to be in the range of 5 to 6 mm, which is favourable from a performance perspective. Shorter piles would engage a larger proportion of end bearing and therefore would develop a greater amount of settlement under design load conditions.

As a result of performing the pile load tests, the structural designers were able to use a soil resistance factor of 0.6 for pile design, a full 50% increase over the 0.4 factor used if the load testing had not been performed. This resulted in a significant cost saving to the contractor for this project.

The results of the belled pile load test has confirmed that using an undrained analysis approach for calculating end bearing capacity of belled piles in the local clay till is an acceptable design methodology.

ACKNOWLEDGEMENTS

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