Evaluation of Full-Scale Pile Load Testing Using Osterberg Cell ® in till and Georgian Bay Shale in Southern Toronto

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Abstract. A full scale, axial Pile Load Test (PLT) using an Osterberg Cell ® (O-Cell) assembly installed in a test shaft, and a lateral PLT using the same test shaft and a reaction shaft were carried out to inform caisson (drilled shaft) design for a project in Toronto near Lake Shore Blvd. Under supervision of Golder (now WSP), Loadtest (a division of Fugro USA Land Inc.) performed a bi-directional axial load test on the test shaft installed within a nominal 1676 diameter rock socket in the Georgian Bay shale, and a nominal 1829 mm diameter shaft in the overburden soils. Subsequently to the axial load test, a lateral load test was performed using the same test shaft and a reaction shaft. The results of the axial testing were compared with literature and the results of other PLTs carried out in the Southern part of the Greater Toronto Area (GTA) to assess local relationships/correlations between Uniaxial Compressive Strength (UCS) and the geotechnical resistance of rock sockets installed in Georgian Bay shale. Using RSPile (by Rocscience Inc.), the results of the lateral testing were used to refine the strain ratio (ε_{50}) parameter for the site's predominantly native silty clay to silty clay till soils, originally estimated from literature, to create a better model of pile/soil response to lateral forces.

Keywords: Pile Load Test · Osterberg Cell ® · Rock Socket

1 Introduction and Site Description

This site is located in the southern part of Toronto, Ontario near Lake Shore Blvd. And the Don River. Part of the development at the site will consist of a structure supported by rock-socketed caisson foundations advanced into the shale bedrock, the surface of which is approximately 12 m below the proposed floor elevations. The preliminary design for the project includes a large number of 1.8 m diameter caissons (through the overburden), with a 1.65 m diameter rock socket, with the caissons varying from approximately 11 m to 15 m in length.

A full scale, axial Pile Load Test (PLT) using an Osterberg Cell ® (O-Cell) assembly installed in a test shaft, and a lateral PLT using the same test shaft and a reaction shaft were carried out to inform the caisson (drilled shaft) design. Excavations at the test pile site were carried out so that the test pile was advanced from a ground surface that is approximately equal to the elevation of the bottom of the proposed pile cap.

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2 Site Investigation and Subsurface Conditions

2.1 Investigation Methods

The initial geotechnical investigation for the project was carried out in 2021 with 29 boreholes advanced to support design. Samples of the shale bedrock were obtained using a 'HQ'-size triple-tube rock core barrel at all borehole locations to generally 5 m below the bedrock surface. In-situ Standard Penetration Testing (SPT) and field vane shear testing was carried out in the overburden. Three Pressuremeter Tests (PMT) were carried out in varying soil strata in a separate hole adjacent to five boreholes (i.e., a total of fifteen tests), and Cone Penetration Testing (CPT) was carried out adjacent to eight of the boreholes. Samples of the cohesive soils were obtained using 76 mm outside diameter (O.D.) thin-walled 'Shelby' tubes (ASTM D1587-08, Standard Practice for Thin-Walled Tube Sampling) at select locations for relatively undisturbed samples. Standpipe piezometers were installed in eighteen boreholes to permit monitoring of groundwater level.

Classification testing (i.e., water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. In addition, consolidation (oedometer) tests were carried out on selected samples of the silty clay to silty clay till deposit. Uniaxial Compressive Strength tests (to assess Uniaxial Compressive Strength (UCS), Young's modulus, and core density) were carried out on 48 selected specimens of the bedrock core samples.

2.2 Subsurface Conditions

Overburden. Based on geotechnical investigation, the subsurface conditions generally consist of predominantly near surface granular fill soils, overlying a thick silty clay to silty clay till stratum, underlain by shale bedrock. In some locations, the silty clay till deposit is underlain by a 0.3 m to 1.2 m thick layer of residual soil (soil that has weathered from bedrock).

Bedrock. Shale bedrock of the Georgian Bay formation was encountered below the silty clay / silty clay till or residual soil deposits at depths ranging from 9.7 m to 16.8 m (i.e., at about Elevations 66.8 m to 68.9 m). A highly weathered bedrock zone, up to 2.1 m thick (typically closer to 0.5 m thick), was encountered at the bedrock surface in most of the boreholes advanced to bedrock and is underlain by a generally moderately to slightly weathered bedrock.

A summary of the UC test results completed is provided in Table 1. Based on the laboratory Unconfined Compression (UC) tests carried out on forty-eight samples of the bedrock from the current investigation, and twenty-one samples of the bedrock from previous investigations at the site, and in accordance with Table 3.5 in the Canadian Foundation Engineering Manual [2], the following were observed:

- shale bedrock generally classified as very weak (R1, 1 MPa $<$ UCS $<$ 5 MPa) to weak (R2, 5 MPa $<$ UCS $<$ 25 MPa); and,
- contains limestone interlayers classified as medium strong (R3, 25 MPa $<$ UCS $<$ 50 MPa) to very strong (R5, 100 MPa $<$ UCS $<$ 250 MPa).

Table 1. Summary of Unconfined Compression (UC) Test Results.

Lithology	Bulk Density (g/cm^3) (average)	Uniaxial Compressive Strength (MPa) (average)	Young's Modulus (MPa) (average)
Siltstone and Shale	2.3 to 2.8 (2.6)	2.2 to 17.1 (10.2)	0.1 to 4.5 (1.3)
Siltstone, Shale and Limestone	2.6 to 2.8 (2.7)	1.3 to 42.4 (28.2)	1.3 to 7.4 (3.5)
Limestone	2.7 (2.7)	87.2 to 177.7 (131.3)	21.6 to 48.2 (38.5)

Note: The testing summarized above was completed on the moderately to slightly weathered portion of the bedrock samples. Intact specimens of the highly weathered bedrock zone could not be obtained for testing

Groundwater. The design high water table elevation is estimated to range between Elev. 78.5 m and Elev. 78.0 m (1 m to 2 m below existing ground surface).

3 Preliminary Pile Design in RSPile by Rocscience Inc.

Prior to the PLT, a preliminary analysis of the axial and lateral geotechnical resistance of the proposed caissons was carried out using the commercially available program RSPile (Version 3.005), developed by Rocscience Inc. The pile behavior is modelled in RSPile using the differential equation for a beam-column, as derived by Hetenyi (1946). The soil response models (P-y curves) used in the lateral resistance analyses were based on the API Method for sand [1] in the non-cohesive fill soils; the soft clay solution by Matlock [3] in the soft to firm cohesive soils ($s_u = 30$ kPa to 50 kPa); and the modified stiff clay without free water solution by Welch & Reese [9] in the firm to very stiff cohesive soils ($s_u = 50$ kPa to 140 kPa). An elastic-perfectly plastic model was used to model the shale bedrock.

3.1 Original Parameters

Overburden Parameters. For non-cohesive soils, the unit weight and effective friction angle parameters employed in the axial and lateral analyses were estimated from correlations based on the in-situ Standard Penetration Tests (SPT) "N"-values as proposed by Peck et al (1974), and U.S. Navy (1986). The initial stiffness was estimated from a correlation based on relative density and the effective friction angle as proposed by API [1]. The parameters, as estimated from the correlations, were adjusted using engineering judgment based on precedent experience in similar soil conditions, where appropriate.

For cohesive soils, total stress parameters were employed in the axial and lateral analyses assuming short-term, undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength (s_u)) for the cohesive soils were estimated from the in-situ field vane tests (adjusted using Bjerrum's correction method, where applicable), from the laboratory oedometer tests (following the correlation proposed by

Mesri, (1975)), and from the CPT results (based on Mesri (1975), Demers and Leroueil (2002), and Lunne et al. (1997) using Nkt = 15.5).

Strain ratio and initial stiffness for cohesive deposits were estimated from correlations based on the s_u as proposed by Meyer and Reese (1979) and Reese, et al., (1975), respectively. Average and lower-bound strain ratios were considered in the analysis as shown in Table 3 in Sect. 5 below.

Summary plots of the laboratory and in-situ testing data used to assess the s_u, accompanied by design lines for s_u and pre-consolidation stress, as well as water contents and Atterberg limits, are shown below in Fig. 1.

Bedrock Parameters. When considering axial deflection of the caisson, the horizontal bedding of the sedimentary shale bedrock plays a major role in the stiffness of the rock mass because the load acts perpendicularly to the bedding. In the vertical direction, the stiffness of the rock mass is mostly controlled by the compression of the bedding, especially if clay seams are present in the bedrock (N.B., the presence of clay seams was identified when logging the rock core at the site). In a horizontally layered rockmass, such as the Georgian Bay Shale at site, the lateral (horizontal) modulus is mostly controlled by the intact rock strength and not affected by the bedding. If vertical or sub-vertical jointing is widely spaced, the lateral modulus of the rockmass will approach intact values.

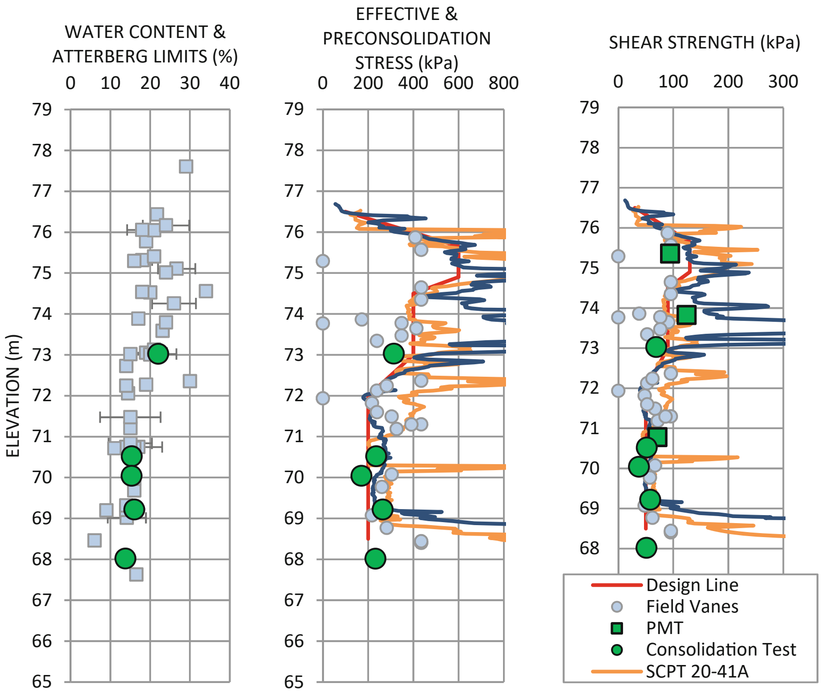


Fig. 1. Summary plots of the laboratory and in-situ testing data used to assess the shear strength, accompanied by design lines for shear strength and pre-consolidation stress, as well as water contents and Atterberg limits.

Table 2. Bedrock Material Parameters Employed in Geotechnical Resistance Analysis.

Bedrock Weathering	Uniaxial Compressive Strength (MPa)	Vertical Elastic Modulus (MPa)	Horizontal Elastic Modulus (MPa)	Lateral Spring Constant (MN/m/m)	Subgrade Reaction Modulus (kN/m^3)
Highly Weathered	10	370	370	400 to 480	222,200 to 266,700
Moderately Weathered to Fresh	10	370	1060	1100 to 1290	611,100 to 716,700

Note: Subgrade Reaction Modulus increases with depth as the distance from the bedrock surface increases

For the shale bedrock, the vertical and horizontal elastic deformation moduli employed in the analysis were estimated based on the design UCS value, an intact Hoek-Brown parameter (m_i) of 7, and a geological strength index (GSI) of 50 and 80, respectively, following the correlation proposed by Hoek-Diederichs.

The subgrade reaction modulus for the bedrock was estimated from the horizontal elastic deformation modulus as proposed by Poulos and Davis (1980). The coefficient of horizontal subgrade reaction modulus is given by:

$$k_h = \frac{K}{B} \tag{1}$$

Where k_h is the Coefficient of Horizontal Subgrade Reaction (kPa/m), K is the Subgrade Reaction Modulus (kN/m^2), or initial slope of P-y curve, and B is the pile diameter/width (m). Summarized in Table 2 is the simplified bedrock stratigraphy and the associated material parameters employed in the geotechnical resistance analyses for the bedrock at the site.

The ultimate unit shaft friction and end bearing stiffnesses for the axial analysis for the caissons embedded (i.e., socketed) into the rock was initially evaluated based on previous static PLT carried out on caissons in the Georgian Bay Shale undertaken at various locations within the Greater Toronto Area (GTA). The preliminary ultimate unit shaft friction (t-z curves) and end bearing stiffnesses (Q-z curves) are shown below in Figs. 4 and 5.

3.2 Geotechnical Capacity Estimates

Assuming a 1.8 m diameter caisson through the overburden, a 1.65 m diameter rock socket extending 3.5 m below bedrock surface (i.e., 3.0 m into fair to excellent quality bedrock, based on RQD), 11 m to 15 m pile lengths, and the material parameters outlined above, the following factored ultimate (f-ULS) and unfactored serviceability (SLS) geotechnical resistances were estimated:

– Axial Capacity: 14,800 kN (f-ULS) / 37,500 kN (SLS)

– Lateral Capacity: 1,750 kN to 2,100 kN (f-ULS) / 950 kN to 1,260 kN (SLS)

Geotechnical resistance factors (Φ_{gu}) of 0.4 and 0.5 were applied to the axial f-ULS and lateral f-ULS, respectively. Axial and lateral tolerances of 25 mm and 10 mm of deflection, respectively, were assumed in determination of SLS. The preliminary lateral resistance assumes an axial load of about 10,000 kN is applied on the caissons. A range of lateral resistance was provided based on varying finished floor elevations (FFE), caisson lengths and overburden conditions.

4 Pile Load Tests

4.1 Background

As shown in Fig. 2, an axial PLT using an O-Cell assembly installed in a test shaft, and a lateral PLT using the same test shaft and a reaction shaft were carried out to inform drilled shaft design. Preliminary estimates of lateral deflection due to the proposed loading on individual caissons were greater than the project's tolerance of 10 mm. The purpose of the PLT was to prove the shaft and base capacity of the rock socket, better characterize soil stiffness, and ultimately, increase efficiency in the caisson foundation design.

4.2 Test Methods and Procedures

In 2021, Loadtest (a division of Fugro USA Land Inc.) performed a bi-directional axial load test using the O-Cell assembly installed within a full-scale, nominal 1676 diameter rock socket with a nominal 1829 mm diameter shaft (LP-1). Three days later, Loadtest performed a lateral load test using the same test shaft (LP-1) as well as a reaction shaft (RXN-1). Representatives from WSP Golder were on site to observe both tests.

The test caissons were both advanced through the overburden through open hole augering. A liner was placed after the caisson had been advanced to bedrock. Each caisson was outfitted with the following instrumentation to monitor strain and movement, and for quality control of the concrete: (i) Crosshole sonic logging (CSL) casings (6 per caisson), (ii) Slope inclinometer (SI) casing (2 per caisson), (iii) Telltales (LP-1 only), and (iv) Strain Gauges (6 per caisson).

4.3 Pile Load Test Results

Axial. The maximum sustained bi-directional load applied to the test shaft (LP-1) was 14.77 MN, with displacements above and below the O-Cell assembly of 46.47 mm (upwards on the rock socket shaft) and 17.61 mm (downwards on the rock socket base), respectively. The maximum applied unit end bearing stress was calculated by Loadtest to be 18,075 kPa on the 1020 mm diameter projected base area from the 940 mm diameter O-cell assembly bottom plate (Chicago method) resting on an 80 mm thick grout levelling layer (see Fig. 2).

WSP Golder carried out an independent interpretation of the axial O-Cell load test data and found a similar maximum applied unit end bearing capacity of about 18,000 kPa. Plots of the test shaft's rock socket side/wall and base responses were developed by WSP

Golder and are shown on Figs. 4 and 5, respectively. Review of the load deformation curve (i.e., mobilized base strength vs. displacement) for the test shaft socket base indicates that the ultimate capacity of the base was not reached before the test was terminated due to premature failure of the rock socket side walls; in short, the ultimate unit shaft resistance of the rock socket walls was lower than anticipated at the preliminary design stage (and the value of unit shaft resistance used to design the rock socket dimensions for the O-cell test). However, the stiffness of the side wall shear behaviour and base response is higher than anticipated at the preliminary design stage. Both results provide valuable insight into the behaviour of the shale bedrock at the site and were incorporated into the updated analysis for the assessment of the geotechnical resistances of the production caissons.

Lateral. The maximum sustained lateral load applied to the piles was 3.04 MN, and at the maximum load, the lateral head deflections of the test pile and reaction pile were 52.70 mm and 37.60 mm, respectively. The lateral load testing revealed that the lateral

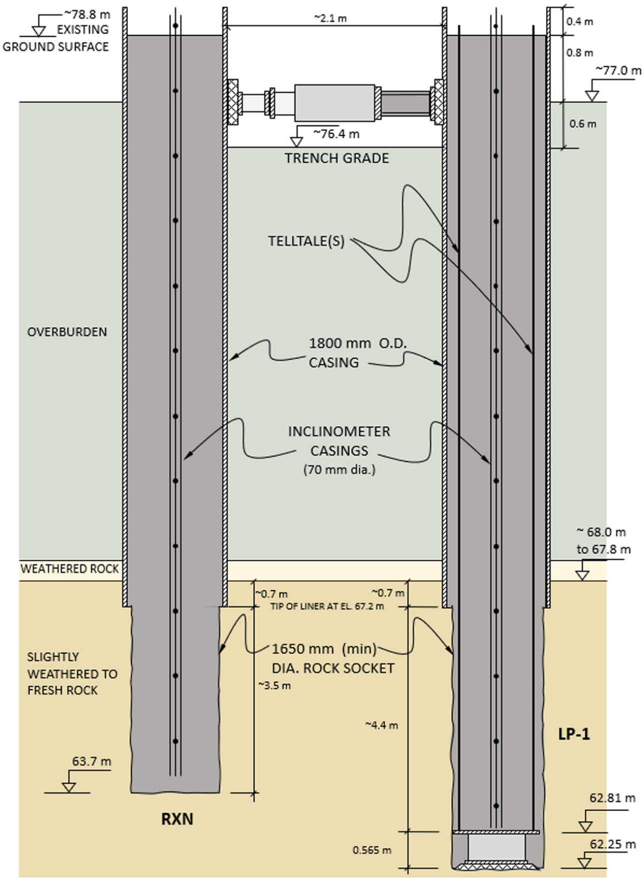


Fig. 2. Pile Load Test Schematic

response of the overburden at the test shaft(s) was stiffer than assumed in preliminary (prediction) modelling.

5 Analysis

5.1 Revised Parameters

Overburden Parameters. The factual lateral test data provided by Loadtest in the Lateral Load Testing report was used to carry out a back-analysis to refine the ε_{50} input parameter for the overburden soils used in the lateral capacity analysis. The back-analysis was carried out in RSPile using a pushover-type analysis. The lateral load testing revealed that the lateral response of the overburden at the test shaft(s) was stiffer than assumed in preliminary (prediction) modelling. As shown in Fig. 3, in the back-analysis, a "curve-fitting" procedure was used to revise the ε_{50} values in the lateral pile analysis until the soil response in the modelling more closely reflected that measured during the full-scale lateral load test. The "curve-fitting" analysis found that reducing the lower-bound ε_{50} used in the preliminary analysis to about one-fifth (1/5), or the equivalent of applying a factor of about 0.2, most accurately modelled the overburden soil stiffness interpreted from the lateral load test.

The contribution of the lateral deflection in the bedrock to the total deflection at the pile head is minimal which is reasonable considering the stiffness and thickness of the overburden soils. In addition, the magnitude of lateral deflection at the top of the

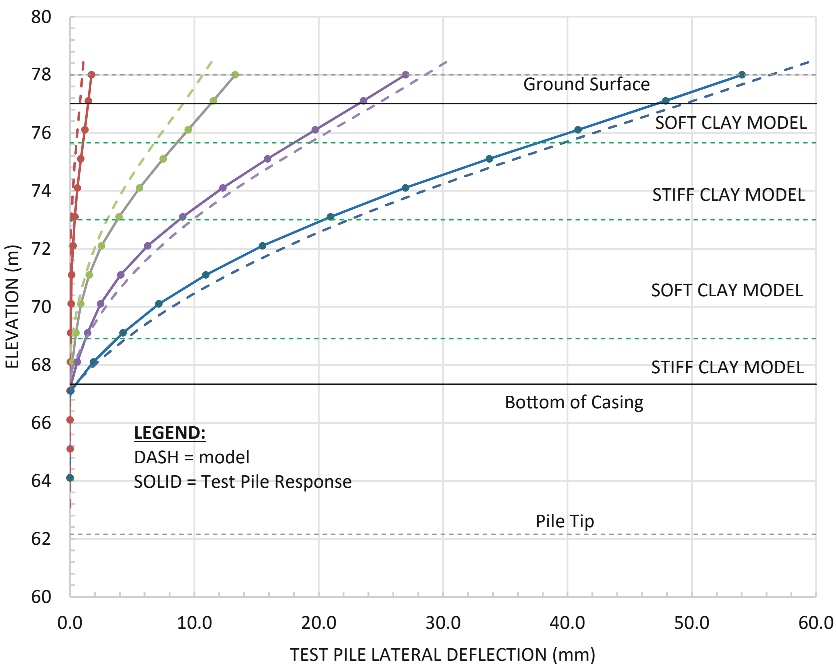


Fig. 3. Curve-fitting Pushover Analysis – Elevation vs. Lateral Deflection

Table 3. Strain Ratios Employed in Lateral Geotechnical Resistance Analysis.

Soil Deposit	Strain Ratio (e_{50})		
	Average	Lower-bound	Post-PLT
Firm Silty Clay	0.012 to 0.008	0.009 to 0.007	0.002 to 0.001
Stiff to Very Stiff Silty Clay	0.008 to 0.005	0.007 to 0.0045	0.001 to 0.0009
Firm to Stiff Silty Clay to Silty Clay Till	0.009 to 0.006	0.007 to 0.005	0.0014 to 0.001
Hard Silty Clay Till	0.0045	0.004	0.0008
Hard Residual Soil	0.0045	0.004	0.0008

bedrock socket measured during the load test (about 2 mm) is similar to the predicted lateral deflection in the model. As such, a back-analysis to refine the parameters used to characterize this response in the rock socket (i.e., lateral spring constant and subgrade reaction modulus) was not deemed necessary, and therefore was not carried out. No revisions were made to the bedrock parameters used to characterize the lateral response (i.e., lateral spring constant and subgrade reaction modulus) of the rock socket and the original parameters used in the preliminary (prediction) modelling, as shown in Table 2.

The revised ε_{50} input parameters for the overburden soils used to characterize the lateral response are shown in Table 3.

Bedrock Parameters. The preliminary and revised ultimate unit shaft friction and end bearing stiffness based on the results of previous load testing (used for the Preliminary prediction modelling) and the full-scale Pile Load Test (Post-PLT) at the site, respectively, are shown in Table 4. It is important to note that although the ultimate unit shaft friction and end bearing stiffnesses used for the design of the production caissons after back-analysis of the PLT are lower than the corresponding preliminary values, these parameters are modelled non-linearly in the analysis. In this regard, a higher initial stiffness for the rock socket was employed in the updated analysis based on the results of the full-scale PLT resulting in a stiffer calculated response for the production caissons. As shown in Figs. 4 and 5, the socket side wall is stiffer at displacements less than about 7 mm, and more significantly the base is stiffer at displacements less than about 55 mm.

The revised bedrock parameters for the axial response of the rock socket (i.e., t-z and Q-z curves) are shown on Figs. 4 and 5 for the socket wall and base, respectively, and the initial end bearing stiffness, ultimate unit end bearing stiffness and ultimate unit shaft friction are summarized in Table 4.

5.2 Revised Geotechnical Capacity Estimates

For load and resistance factored design (LRFD) (i.e., limit states design), the following f-ULS and SLS geotechnical resistances were recommended assuming 1.8 m diameter caissons through the overburden with 1.65 m diameter rock sockets extending minimum 3.5 m below the bedrock surface. In accordance with the National Building Code (NBC) 2015, and for the purposes of a direct comparison the f-ULS values estimated at the preliminary (prediction) modelling stage, geotechnical resistance factors (Φ_{gu}) of 0.4

Table 4. Preliminary and Post-PLT Stiffness, Ultimate Unit Shaft Friction and End Bearing Stiffness.

Bedrock Weathering	Initial Shaft Stiffness (kPa/m)		Ultimate Unit Shaft Friction (kPa)		Initial End Bearing Stiffness (kPa/m)[2]		Ultimate Unit End Bearing Stiffness (kPa) [3]	
	Prelim.	Post-PLT	Prelim.	Post-PLT	Prelim.	Post-PLT	Prelim.	Post-PLT
Highly Weathered	55,000	See Fig. 4 [1]	1100	300	--	--	25,000	--
Moderately Weathered to Fresh	55,000	See Fig. 4 [1]	1100	650	335,000	750,000	25,000	> 18,000 [3]

Note(s):
1. Shaft stiffness modelled as non-linear response (i.e. t-z curve). See Fig. 4.
2. End bearing stiffness modelled as a non-linear response (i.e. Q-z curve). See Fig. 5.
3. Ultimate end bearing stiffness not reached in axial load test; test ended at 18 MPa with 18 mm of settlement.

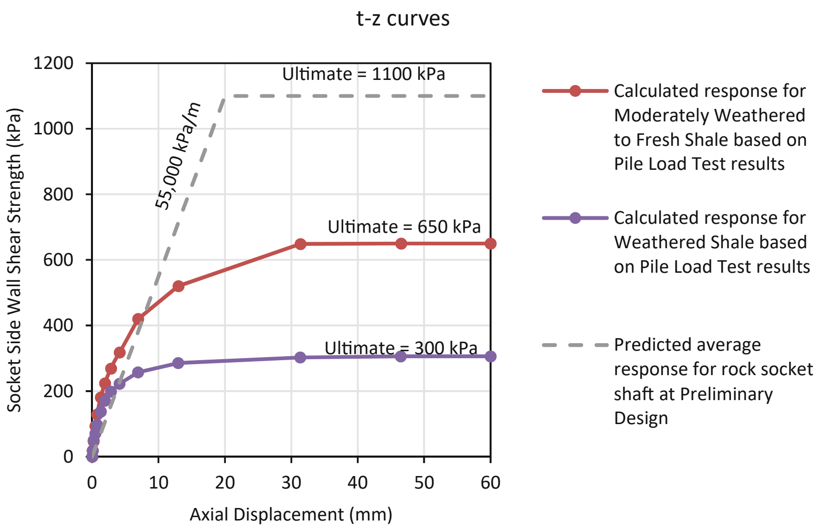
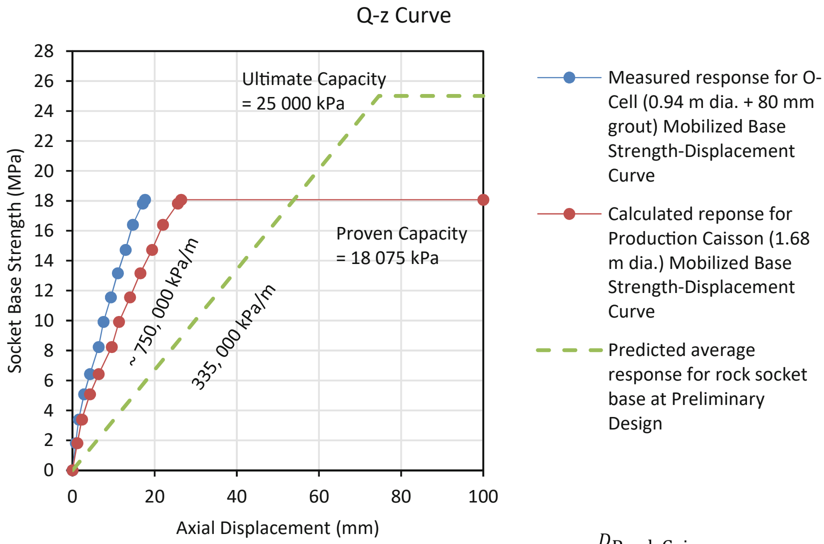


Fig. 4. Test Caisson (LP-1) Axial Rock Socket Shaft Response – Socket Wall Shear Strength vs. Axial Displacement

and 0.5 were applied to the axial f-ULS and lateral f-ULS, respectively. No geotechnical resistance factors (Φ_{gs}) were applied to the axial serviceability geotechnical resistances. The preliminary and post- PLT recommendations, considering the revised ε_{50} parameter and updated t-z / Q-z curves, as well as the new pile loading conditions, are shown in Table 5.



$$\Delta z_{\text{Prod. Caisson}} = \frac{D_{\text{Prod. Caisson}}}{D_{\text{O-cell}}} \Delta z_{\text{O-Cell}}$$

Fig. 5. Test Caisson (LP-1) Axial Rock Socket Base Response – Socket Base Strength vs. Axial Displacement

Table 5. Geotechnical Resistances based on Load & Resistance Factored (Limit States Design)

Direction	Caisson Diameter (m) Soil / Rock	Factored Ultimate Geotechnical Resistance, f-ULS [1]		Serviceability Geotechnical Resistance, f-ULS [1]	
		Preliminary	Post-PLT	Preliminary	Post-PLT
Axial	1.8 m / 1.65 m	14,800 kN (6,900 kPa) [3]	21,400 kN (10,000 kPa) [3]	37,500 kN (17,500 kPa) [3]	44,000 kN (20,600 kPa) [3]
Lateral	1.8 m / 1.65 m	1,750 kN to 2,100 kN [4]	1,300 kN to 2050 kN [4]	950 kN to 1,260 kN [4]	1090 kN to 1250 kN [4]

Note(s):

1. Geotechnical resistance factors (Φ_{gu}) of 0.4 and 0.5 were applied to the axial f-ULS and lateral f-ULS, respectively for both Preliminary and Post-PLT analyses for direct comparison.
2. Axial and lateral tolerances of 25 mm and 10 mm of deflection, respectively, were assumed in determination of SLS.
3. The f-ULS and SLS provided in units of stress are approximate only and based on geometry of the rock socket base (i.e., 1.65 m diameter). If rock socket dimensions different than those indicated above are adopted, the average f-ULS and SLS unit resistance (in kPa) may change.
4. A range of lateral resistance is provided based on varying finished floor elevations (FFE), caisson lengths and overburden conditions.

It is noted that a higher Φ_{gu} (up to 0.6) could be applied to the assessment of the f-ULS values for the production caissons (as discussed in Sect. 6) based on the results of the site specific, full-scale, and equivalent geometry PLT.

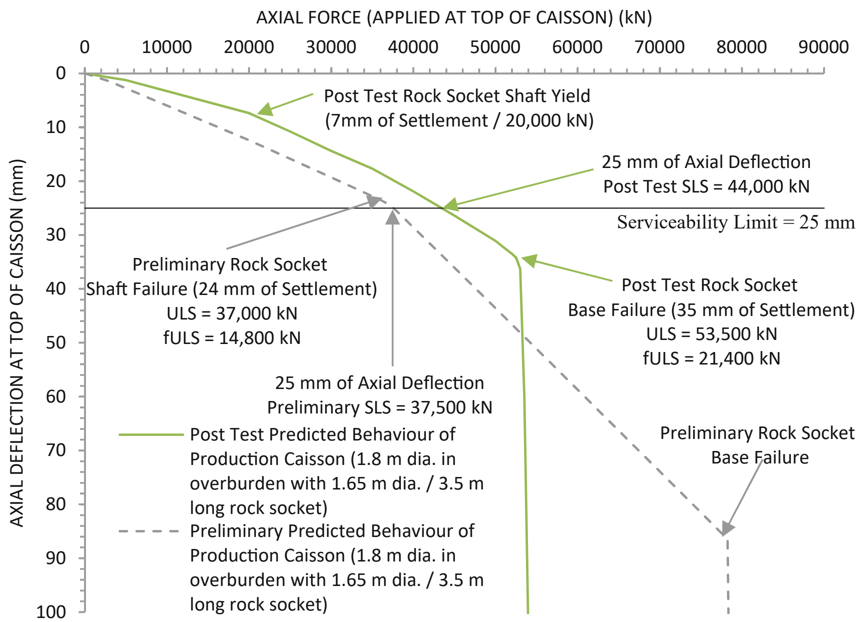


Fig. 6. Estimated Geotechnical Axial Capacity for Production Caissons – Axial Deflection vs. Axial Force

The calculated plots of axial force vs. axial deflection at the top of the caisson from both the preliminary (prediction) modelling and the post-PLT that were used to assess the f-ULS and SLS values are shown in Fig. 6. These plots were generated in RSPile by applying varying axial load and computing the axial deflection at the top of the caisson ("a push-down type analysis"). The authors note that it would be convenient to add a "push-down" auto compute feature to RSPile, similar to the lateral push-over analysis function.

6 Discussion

Evaluation of the axial PLT results from the O-cell testing demonstrated an ultimate (unfactored) unit end bearing resistance, q(t), of at least 18,000 kPa, and ultimate (unfactored) unit side shear, f(s), of about 300 kPa and 650 kPa in the upper, weathered and lower, less weathered portions of the shale bedrock, respectively. Review of the load-deformation curve (i.e., mobilized base resistance vs. displacement) for the test shaft socket base indicated the ultimate capacity of the base was not reached before the test was terminated due to premature failure of the socket side walls. However, the stiffness of the side wall shear behaviour and base response was higher than estimated from typical correlations. The resistance and sidewall stiffness were used in RSPile to confirm the response of the rock socket and to carry the design forward to detail design for the production caissons.

The results from similar bi-directional axial load (O-Cell) testing carried out in three other drilled shafts with rock sockets in the Georgian Bay shale in southern Toronto, where 5Mpa < UCS < 10Mpa, found maximum applied q(t) ranging from > 14,000 kPa to 34,000 kPa and f(s) ranging from 300 kPa to 1410 kPa. The range of f(s) values reflects the variability in shale bedrock in Toronto, and the lower end values reflect the weaker rock conditions in particular in areas close to Lake Ontario.

The axial load testing results indicate q(t) = [1.8 to 3.4] × UCS, f(s) = [0.09 to 0.16] × $\sqrt{}$UCS for the upper weathered zone and f(s) = [0.21 to 0.45] × $\sqrt{}$UCS for the lower, less weathered zone for the geotechnical resistance of rock sockets in the weak shale bedrock in Southern Toronto. These back-calculated values from the PLT compare reasonably well with the typical relationship of ultimate q(t) = [2 to 3] × UCS recommended in literature, and the relationship of ultimate f(s) = [0.20 to 0.30] × $\sqrt{}$UCS in literature , as well as for piles in weak bedrock (UCS < 6Mpa) as summarized in Latapie (2019). It is noted however, that the for the weak shale bedrock near Lake Ontario, the back-calculated values of f(s) appear to be more consistent with the lower bound of the equation range reported in literature.

The lateral load testing results indicate that the lateral response of the silty clay/clay till overburden at the test shaft(s) was stiffer than anticipated. In the back-analysis, a "curve-fitting" procedure was used in the RSPile push-over analysis to revise the ε_{50} values in the analysis until the soil response in the modelling more closely reflected that measured during the test. The "curve-fitting" analysis found that reducing the strain ratios picked from literature (Matlock, 1970) to about one-fifth (1/5), most accurately modelled the overburden soil stiffness interpreted from the lateral load test, suggesting that the actual firm to stiff silty clay overburden is up to five times stiffer than the conventional lateral soil response models would suggest.

Consideration could be given to applying a Φ_{gu} factor of 0.6 (instead of 0.4) to the axial f-ULS in accordance with NBC (2015) as the axial resistance of the piles has now been confirmed by full-scale static PLTs carried out at the site (see *Table K-1 – Resistance Factors for Shallow and Deep Foundations of Structural Commentaries, K Foundations*). A Φ_{gu} factor of 0.6 would increase the axial f-ULS from 21,400 kN (10,000 kPa) to 32,100 kN (15,000 kPa); an increase of 50%. The decision to adopt a higher Φgu for the pile design at this site should take into consideration the potential for variability in the bedrock conditions between the test pile location and the actual production caisson locations as well as the expected performance with a higher Φ_{gu}.

7 Conclusions

The results of these full-scale PLTs in the weak shale bedrock and firm to stiff silty clay till in the GTA are valuable to the design engineer in that they avoid some of the preconceived notions and so-called 'typical' or 'rule-of-thumb' values that are conservative and have been used for the design of drilled shafts/rock sockets in southern Ontario for many years. The willingness of geotechnical engineers to educate owners and clients on the value of incorporating innovative full-scale testing into the design process has resulted in less conservative and overall, less costly design on recent projects in the GTA. However, the author's note that given the contribution of construction methodology to the performance of caissons, care is required during construction along with proper inspection, to achieve consistency in the performance of the production caissons.

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