Undrained Shear Strengths derived from Pile Load Tests for use in designing Helical Screw Piles and Stelcor Piles in Sensitive Clay Soils

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Abstract

To verify the undrained shear strengths as input data for use in the design of Helical Screw Piles and Stelcor Piles, a series of full compression and tension pile load tests were carried out in sensitive clay and silty clay deposits at the Barnston pump station site in Maple Ridge, BC. Both compression and tension pile load tests were performed in accordance with ASTM D1143, with load-settlement characteristics being recorded continuously, including ultimate load carrying capacity. The test piles were required to be loaded to 200% of the design load. However, both the Stelcor piles and helical screw piles failed before reaching the targeted design loads. Given the pile load tests results, it was important to derive the back-analysed values of undrained shear strength that were mobilized following pile installations. Back-analysis indicates that the undrained shear strength values (of c_u = 18-22 kPa) that were mobilized following pile installations were significantly lower (by about 2-2.5 times) than the values (of c_u = 40-50 kPa) that had been recommended for design. The subject screw pile load test results are compared with other full-scale load tests on screw piles in a) lightly over-consolidated sensitive clayey silts in Manitoba, and b) cohesive soils in Alberta and British Columbia. It is concluded that the undrained shear strength recommendations in use for the design of helical screw piles should take account of significant disturbance effects that impact soft, sensitive silty clay deposits during pile installation. Back-analysed values, and/or knowledge and experience of soil sensitivity, are the most appropriate parameters and approach for designing helical screw piles and Stelcor piles in sensitive clay and silty clay soils.

Site Location

The Barnston pump station is located at the end of 120A Avenue on the Pitt Meadows - Maple Ridge boundary in the conjoinment area of two branches of Fraser River; see Figs. 1 and 2. The elevation of the site at the time of our pile load tests ranged from +2.0 to +3.6 m. The Barnston Maple Ridge pump station is a key component of Metro Vancouver's Langley Water Supply program to meet increasing water demands in Maple Ridge, Surrey and Langley, and to replace the existing obsolete pump station.

Geology

Being located in the conjoinment of Fraser River and Pitt River in a very low altitude, almost sea level, geologically the area is covered with sedimentation of Fraser River and Pitt River, which comprise fine clay and silty clay deposits; see Fig. 2.

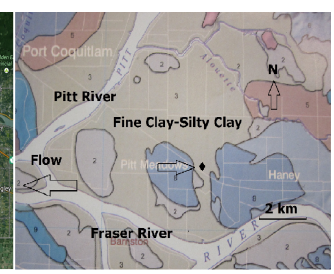


Figure 1: Site Location Plan Figure 2: Site Geology

Geotechnical Investigation

The geotechnical subsurface investigation had previously been performed (Golder, 2011). Four solid-stem auger holes and a seismic cone penetration test (SCPT) had been advanced. A hollow-stem auger hole had also been advanced to allow for undisturbed soil sampling and in-situ shear strength testing using the vane apparatus. The SCPT had been advanced to a depth about 30 m below the ground surface. Soil stratigraphy was generally consisting of fill overlying soft to stiff compressible soils. Groundwater had been encountered at geodetic elevation ranging from

-0.4 m to 0.8 m during drilling. Pore pressure dissipation test carried out at the hollow-stem auger hole indicated that the groundwater table at the time of the investigation was at approximately -0.3 m elevation. It was noted that the groundwater table observed during drilling might not have reached equilibrium. Groundwater table fluctuations were expected to vary seasonally as a function of precipitation, surface drainage effects, and water level fluctuations at the adjacent Fraser River and Pitt River.

Subsurface Conditions

Fill

Fill material varying in composition from organic silt to sand had been encountered in all boreholes. Clayey fill had also been encountered in some auger holes, while there was wood waste within the fill material in one auger hole. The thickness of fill ranged from 0.46 m to 1.52 m at the locations of the auger holes; see Fig. 3.

Silty Clay

The surficial fill layer was underlain by native deposits of clayey soil in all auger holes. A grey clayey soil layer ranging in composition from clayey silt to silty clay with trace sand was encountered from +1.2 to +2.4 m elevation. Furthermore, thin sand lenses or layers were encountered at elevations ranging from -3.1 m to -4.2m. In general, the strength of the clayey soil encountered above the groundwater table was identified to be stiff to very stiff. Based on the auger hole data, a stiff crust generally extended down to about zero (0m) elevation. Based on the observations made during drilling as well as the in-situ vane test results and data obtained from the SCPT, the clayey soil deposit below the groundwater table was identified as firm. The peak undrained shear strengths obtained from in-situ testing ranged from 51 to 55 kPa; see subsurface geotechnical model in Fig. 3. Geotechnical consolidation tests carried out on the relatively undisturbed piston-tube samples of the silty clay obtained at depths of between 5.2 and 5.8 m gave a compression index Cc of 0.27 and a rebound index Cr of 0.07. Based on review of the void ratio-stress plot, the silty clay is considered to be slightly over-consolidated. The SCPT sounding shows the native clay stratum had cone tip resistance values ranging from 8 to 13 bars, with sleeve friction resistance of less than 0.25 bars.

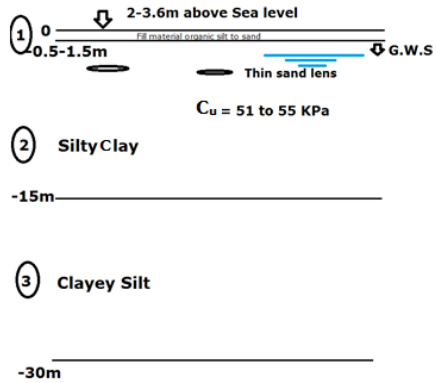


Figure 3: Subsurface Geotechnical Model

Clayey Silt

The SCPT sounding shows the native clay deposit was underlain by a silt layer at approximately -15.2 m elevation (at the hollow-stem auger hole location). The silt layer was described as sandy silt to clayey silt according to CPT Soil Behaviour Type Interpretation. The silt layer corresponded to cone tip resistance values ranging from 13 to 20 bars and sleeve friction resistance value of approximately 0.25 bars. At the west of the site the compressible clayey silts extended to at least 68 m below the ground surface.

Original Pile Design Recommendations

The installation of helical screw piles had been considered for uplift resistance at this site (Golder, 2011). Given the design foundation elevation of +0.6 m, where the subsurface conditions would consist primarily of firm to stiff clayey soils, the size and capacity of helical screw piles could be determined. The recommended ultimate unit skin friction and unit end bearing values for calculating axial compression and tensile capacity of helical piles were given as 40 kPa and 450 kPa respectively. A minimum centre-to-centre pile spacing of three helix diameters had been recommended for design purposes. In addition, where multiple helixes had been considered, the surface area of the cylinder between the uppermost and lowermost helix was to be multiplied by the undrained shear strength, recommended as 50 kPa, to determine the ultimate cylindrical shear capacity against uplift. For ultimate limit state (ULS) design it had been recommended to use a geotechnical resistance factor of 0.4 for determining factored compression resistance and a geotechnical resistance factor of 0.3 for determining factored tensile resistance.

Stelcor & Helical Screw Pile Tests

Pile Load Test Procedures and Pile Characteristics

GES (2012) was retained by Vickars Construction (now Terracana Foundation Solutions) to develop the screw pile design and pile installation configurations in a proposal for a design-build contract with Metro Vancouver. The scope of work also included observing pile load test installations and back-analysis of the results. Pile load tests were carried out in accordance with ASTM-D1143 with some modifications to the test procedures, as proposed by GES (2012) for the second series of the helical pile tests. After completion of these tests, the review of the load-settlement characteristics and determination of the ultimate load carrying capacity of both the Stelcor and the Helical Screw pile systems were carried out.

GES' design was based on the undrained shear strength values of 40-50 kPa recommended by Golder (2011). Pile load tests were carried out in compression and tension for both the Stelcor 500 pile and the Helical SS175 pile. The piles were installed in August 2012, with the pile load tests carried out about 14 and 28 days following pile installations. The Stelcor pile was fully grouted whereas the helical pile was grouted only above the highest helix and along the length of the pile shaft. The "effective" diameters of the piles were adopted as 10.5'' (268mm) for the Stelcor pile and 8'' (200mm) for the helical piles. The "effective" pile diameter, as defined by Mahmoud (1989), is the region of soil disturbance as a result of pile loading (rotation) through approximately one revolution, and is shown in Fig. 4. The effective diameter is determined from the following formula:

$$\text{Effective pile diameter } (D_e) = 2\,(0.15D) + D$$

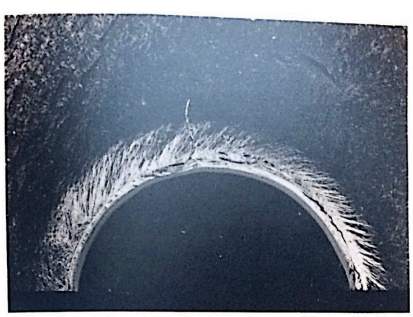


Figure 4: Region of Disturbed Soil Around a Pile

Fig. 5 shows the installation of a Stelcor pile; the latter were manufactured and supplied by Ideal Manufacturing Inc. of Webster, New York. As can be seen from Fig. 5, Stelcor piles essentially consist of a pile having helixes along its entire length.

Figure 5: Stelcor 500 Pile Installation

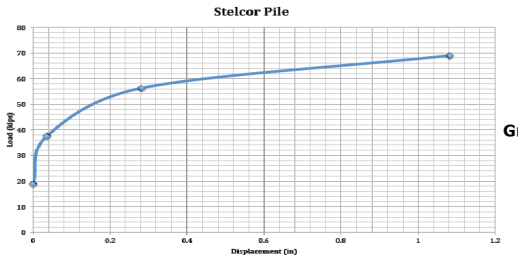
Pile load test specifications included applying vertical loads up to 200% of the design values. The first test was carried out on the Stelcor 500 pile. The Stelcor pile 'failed' at 92% of the design load under compression conditions; unload-reload loops were included in the testing procedure in order to establish the relative stiffness of the subsoils at different stages of the loading cycle (GES, 2012). The Helical screw pile 'failed' in compression at 95% of the design value. Under tensile conditions, the Stelcor pile 'failed' at 110% of the design load. Failure was defined as pile displacements corresponding to about 10-13% of the effective pile diameter. Characteristics of the pile load tests and the targeted design loads are shown in Table 1.

Table 1: Pile Characteristics and Targeted Design Loads

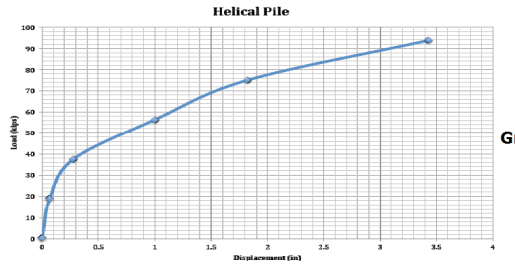
Pile Type	Effective Pile Diameter (in) m	Pile Length (ft.) m	Targeted Compression Design Load (kips) kN	Targeted Tensile Design Load (kips) kN
Stelcor 500	(10.5) 0.268	(38) 11.6	(75) 338	(35) 158
Helical SS175	(8) 0.200	(98) 30	(75) 338	(35) 158

GES' Test Results

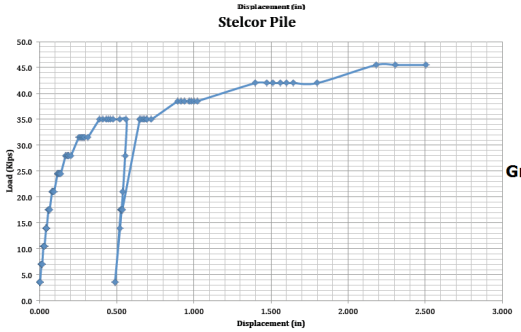
One of the objectives of the pile load testing program was to determine the "mobilized" undrained shear strength (c_u) of the firm silty clay zone through back-analysis. For the piles in compression, the measured ultimate load for the Stelcor pile was recorded as 310 kN (69 kips), corresponding to a back-analysed c_u value of 22 kPa; see Graph 1. The measured ultimate load for the helical pile was recorded as 320 kN (71 kips), corresponding to a back-analysed c_u value of 18 kPa; see Graph 2. Graph 3 shows the tensile load-displacement relationship corresponding to the maximum pile load that was recorded in tension (45 kips = 202 kN) for the Stelcor pile; giving the back-analysed c_u value of 19 kPa.



Graph.1



Graph.2



Graph.3

The back-analysed undrained shear strength values obtained from the pile load tests on the Stelcor and the helical piles were approximately 2.5 times smaller than the recommended design values.

Analysis of c_u Values

After observing the difference between the back-analysed values and the recommended values of undrained shear strength for use in the design of helical piles, it was important to determine what factors could have led to the difference of about 2.5 times. As previously stated, the shear strength values mobilized following pile installations were significantly lower than the measured in-situ strength values of 40-50 kPa. This was due to significant disturbance to the soft, sensitive silty clay deposits during pile installation. The installation disturbance caused by the Stelcor 500 pile was significantly higher than had been envisaged due to the large header (16'' header vs. 8'' stem) of the Stelcor system.

It is also noted that the mobilized strength measured from the Helical Screw piles corresponds to similar (lower) values of mobilized strength (18 kPa from the compression test). Thus, it is inferred that the significantly reduced mobilized shear strength is primarily attributable to the installation disturbance and the sensitivity of the subsoil deposits. According to findings by Mahmoud et al. (2000) in the area above the Kanaka Creek floodplain in Maple Ridge – in the general area of the Barnston site – the values of soil sensitivity, defined as the ratio of peak to remoulded vane strength, were approximately 3 in the weathered layer and between 4 and 6 in the un-weathered materials, with plasticity indices (PI) of 55% and 28% corresponding to the weathered and un-weathered layers, respectively. These values compare well with PI of 65-75% and sensitivity of 6-6.5 at the Barnston site. The back-analysis from the two compression tests on both the Stelcor 500 and the Helical Screw pile, as well as the tensile test on the Stelcor 500 pile, indicated "mobilized" undrained shear strength values in the range of 18-22 kPa, with the mobilized value obtained from the Helical Screw pile test being 18 kPa. Thus, the back-analysed ("mobilized") values for undrained shear strength (of 18 kPa) at the Barnston site are considered reasonable for use in the design of piles.

Pile Load Tests by C^3 Integrated Solutions

C3 Integrated Solutions (2013) also carried out pile load testing at the Barnston pump station site to determine which helical screw pile type was best suited for this project. Loads were applied in increments of approximately 30 kN. The restrictions on the C^3 Integrated Solutions tests were that a) the jack had a maximum stroke of 6'', b) gauges took readings up to 2'' and c) the jack had a capacity of 660 kN. The test was continued until one of the restrictions was reached or until the pile failed by continuous pull-out; see Fig. 6. Characteristics of the pile load tests carried out by C^3 Integrated Solutions are shown in Table 2.

Table 2: Pile Load Tests by C^3

Pile Type	Helical Plate Diameter (in) m	Installation Date	Test Date	Test Results
SS200	(24'') 0.610	April 2, 2013	April 11, 2013	Failed
SS225	(14'') 0.356	April 13, 2013	April 19, 2013	Failed
SS175	(14'') 0.356	June 4, 2013	June 11, 2013	Passed

Only one pile passed the load test and this pile was the same pile (Helical SS175) as the one used by GES-Vickars under their load testing program. However, the characteristics (diameter and length) of the SS175 pile from the C^3 testing program are somewhat different from that used by GES-Vickars. Nevertheless, the grouted pile diameter and pile length are almost the same as GES-Vickars'. The characteristics of the C^3 SS175 pile that was tested in compression and tension are given in Table 3.

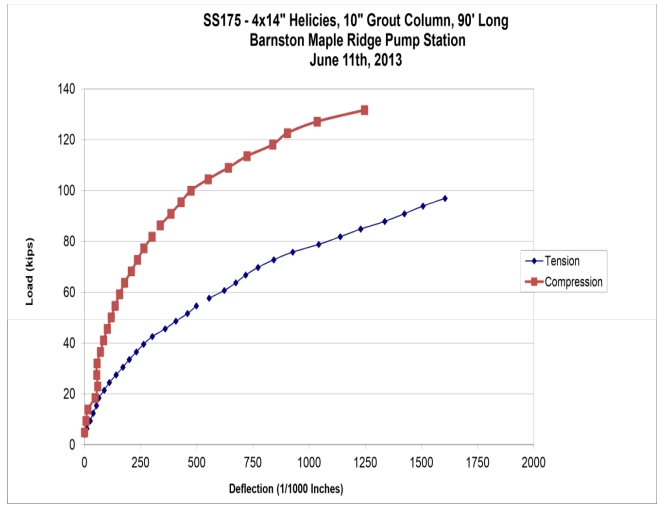
Table 3: Pile Characteristics and Targeted Loads - C^3

Pile Type	Pile Diameter (in) m	Pile Length (ft.) m	Targeted Load for Compression kN	Targeted Load for Tension kN
Helical SS175	(14) 0.356	(90) 27.5	405	265

Figure 6: Pile Load Test Setup by GES-Vickars

C^3 Tests Results

The objective of tests was to determine the undrained shear strength (c_u) for the SS175 and SS200 helical screw piles. For the piles in compression, the measured ultimate load for the SS175 helical pile was determined to be 585 kN (131.5 kips) corresponding to c_u of about 9 kPa. The measured ultimate load for the SS200 helical pile was determined to be 272 kN (61 kips) corresponding to c_u of about 20 kPa. Thus, the back-analysed values of undrained shear strength for the C^3 Helical piles were about 2.5-5.0 times smaller than the original design recommendations by Golder (2011). Graph 4 shows the load-deflection relationship for the compression and tensile pile load tests by C^3.



Graph 4: Load-Deflection Relationship for the C^3 Compression and Tensile Pile Load Tests

Table 4 summarizes the results of back-analysed undrained shear strength values from the C^3 and GES-Vickars pile load tests.

Table 4: Characteristics of Piles, Tested Capacities, and Back-Analysed Values of Undrained Shear Strength

Contractor	Pile Type	Pile Depth (in) m	Max Pile Load in Compression kN	Max. Pile Load in Tension kN	Back-Analysed Undrained Shear Strength in Compression kPa	Back-Analysed Undrained Shear Strength in Tension kPa
GES-Vickars	Stelcor 500	(38) 11.6	310	202.4	22	17.4
GES-Vickars	Helical SS175	(98) 30	320	NA	18	NA
C^3	Helical SS200	(30) 9.1	272	180	20	13
C^3	Helical SS175	(90) 27.5	585	431	9	2.8

Table 5 presents a comparison between the predicted and recorded loads from tests by GES-Vickars and C^3.

Table 5. Characteristics of Tested Piles and Comparison between Predicted and Recorded Loads (Results by GES-Vickars and C^3)

Contractor	Pile Type	Pile Diameter (in) m	Pile Depth (in) m	Predicted Max. Pile Load in Compression (kN)	Max. Recorded Pile Load in Compression (kN)	Ratio of Tested and Predicted Pile Capacities (%)	Back-Analysed Undrained Shear Strength in Compression (kPa)
GES-Vickars	Stelcor 500	(10.5) 0.268	(38) 11.6	750	310	40	22
GES-Vickars	Helical SS175	(8) 0.200	(98) 30	750	320	43	18
C^3	Helical SS200	(24) 0.610	(30) 9.1	NA	272	NA	20
C^3	Helical SS175	(14) 0.356	(90) 27.5	405	585	144	9

Comparison with Screw Pile Load Tests in Other Clayey Soils in Canada

To put in context the compression and tensile capacities of screw piles installed in clayey soils in Canada, the results of other pile load tests in Manitoba, Edmonton AB, and Fort St John BC are compared below.

Screw Pile Load Tests in Manitoba

The results of six screw pile load tests in Manitoba are included in Table 6 (see Sakr). The subsoils were described as glacial lacustrine silty clay deposits with undrained shear strength values ranging from 43-86 kPa, averaging 56 kPa, with liquid limit of 35% and PI=15% and a medium sensitivity of 3-4; groundwater was at ground surface. The shaft diameter was 0.2m with a spacing between helixes of 1.07m.

Table 6: Results of Screw Piles Load Tests in Manitoba

Load Test Type	Pile Length (m)	No. of Helixes	Inter Helixes Spacing Ratio S/D	Recorded Axial Capacity (kN)
C1	5	3	1.5	180
C2	3	3	1.5	160
C3	5	2	3	210
T1	5	3	1.5	210
T2	3	3	1.5	140
T3	5	2	3	210

C: Compression
T: Tension

Screw Pile Load Tests in Edmonton, AB

The results of six screw pile load tests in Edmonton, Alberta are included in Table 7 (see Tappeden and Sego, 2007). The subsoils were described as glacial lacustrine silty clay. The screw piles had helix diameters of 35.6 cm with shaft diameter being 21.9 cm.

Table 7: Results of Screw Pile Load Test in Edmonton, Alberta

Pile Test	Pile Length (m)	Calculated Capacity (kN)	Recorded Ultimate Axial Capacity (kN)	Ratio of Actual to Calculated Capacity
T2	8.53	331	161	0.49
T3	8.53	420	272	0.65
T5	9.75	457	176	0.39
T6	11.58	393	160	0.41
T7	10.36	400	200	0.5
T8	11.28	472	187	0.4
Av.	10	412	193	0.47

T: Tension

Screw Pile Load Tests in Fort St. John, BC

The results of two screw pile load tests in Fort St. John, BC are included in Table 8 (see Tappenden and Sego, 2007). The subsoils were described as stiff silty clay. The screw piles had a helix diameter of 45.7 cm with the shaft diameter being 11.4 cm.

Table 8: Results of Screw Pile Load Tests in Fort St. John BC

Load Test Type	Pile Length (m)	No of Helixes	Inter Helixes Spacing Ratio S/D	Installation Torque (kN-m)	Recorded Axial Capacity (kN)
C16	5	2	3.3	13.5	245
C17	4	1	-	8	169

C: Compression

Conclusions and Recommendations

The back-analysed undrained shear strengths obtained from the helical screw pile load tests by GES-Vickars and C^3 Integrated Solutions are significantly lower (by about 2.5-5 times) than the values recommended for design purposes. Significant disturbance to the soft, sensitive silty clay deposits during pile installation had resulted in significantly lower values of undrained shear strength being mobilized following pile installations. GES had anticipated such soil behaviour prior to pile installation and testing based on previous experience with these highly sensitive silty clay deposits at the Kanaka Creek floodplain area in Maple Ridge. The back-analysed undrained shear strength value of about 20 kPa, rather than 40-45 kPa, was deemed reasonable for use in the design of screw piles for the Barnston Maple Ridge pump station project. The result of the helical screw pile load tests carried out in silty clay soil in Manitoba also showed a reduction of about 2 times in compression capacity of the piles with respect to the predicted values. Similar variations in the ratio of recorded to predicted values were also seen in the results of screw pile load tests carried out in Edmonton, AB and Fort St. John BC. Therefore, the undrained shear strength recommendations for use in the design of helical screw piles should take account of significant disturbance effects that impact soft, sensitive silty clay deposits during pile installation. Back-analysed values, and/or knowledge and experience of soil sensitivity, are the most appropriate parameters and approach for designing helical screw piles and Stelcor piles in sensitive clay and silty clay soils.

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