Highway 400 and Highway 89 Underpass Bridge Replacement – Evaluation of Pile Load Tests: A Case Study

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Reflection on Resources

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ABSTRACT
As part of the widening of Highway 400 through Innisfil, Ontario, the existing bridge carrying Highway 89 over Highway 400 was replaced with a new, longer bridge under Ministry of Transportation, Ontario (MTO) contract 2018-2024. The abutments and pier were designed to be supported by steel HP 310x110 piles driven to a competent stratum. Unlike the new bridge, the existing bridge was founded on shallow foundations and did not offer any insight on the geotechnical resistance or performance of deep foundations at this site. Given the lack of site-specific information on deep foundations in this area and the predicted relatively low axial geotechnical resistance for driven steel piles based on empirical methods, a full-scale static pile load test was carried out on a pre-production test pile~~,~~ Dynamic testing using the Pile Driving Analyzer (PDA) was carried out during the pile installation. The pile load test revealed that the test pile driven to the original design pile tip elevation did not achieve the design pile capacity. The test pile was therefore driven deeper into “100 blow” glacial till deposit and re-tested.

This paper provides a summary of the static pile load test, together with the results from the Pile Driving Analyzer (PDA) carried out during the test pile installation, and a comparison to the geotechnical resistances estimated using several design methods. The paper also includes a discussion on additional subsurface investigation and testing undertaken following completion of the static pile load test to gain a better understanding of the subsurface conditions at the test pile location and correlate these data with the results of the static pile load test. This additional investigation included an SPT-sampled borehole, sonic drilling, and piezocone (CPT) testing. Excess porewater pressure measurements (including peak values and rates of dissipation) were also measured adjacent to the test pile.

RÉSUMÉ
Dans le cadre de l'agrandissement de l'autoroute 400 à Innisfil, en Ontario, le pont existant reliant l'autoroute 89 au-dessus l'autoroute 400 a été remplacé par un nouveau pont plus long, en vertu d'un contrat du ministère des Transports de l'Ontario (MTO). Les culées et les piliers du pont ont été conçus pour être supportés par des pieux en acier HP 310x110, battus jusqu’à un substratum compétent. Contrairement au nouveau pont, le pont existant repose sur des fondations superficielles, et ne donne pas d’appréciation de la résistance ou de la performance géotechnique des fondations profondes à ce site. Compte tenu du manque d'informations sur la performance des fondations profondes spécifiques au site, et d’une résistance géotechnique axiale relativement faible selon les prédictions avec méthode empiriques pour les pieux en acier battus, un essai de chargement statique à pleine échelle a été effectué sur un pieu d'essai préproduction qui avait été assujetti à un essai dynamique lors de l’installation. L'essai de chargement statique a révélé que le pieu d'essai battu jusqu’à l'élévation de la pointe prévue lors de la conception initiale n'avait pas atteint la capacité de conception, même après la « mise en place » du pieu après plusieurs jours. Ainsi, le pieu d'essai a été battu plus profondément dans le dépôt de till dit des «100 coups » et testé de nouveau, afin d’appuyer une conception révisée pour les pieux de production.

Cet article présente un résumé de l'essai de chargement statique et dynamique (PDA) des pieux et de l’utilisation de la formule de battage des pieux de Hiley effectués lors de l'installation des pieux, ainsi qu'une comparaison entre les résistances géotechniques estimées à l'aide de différentes méthodes de conception. L'article comprend également une discussion sur les investigations et les tests supplémentaires entrepris après l’essai de chargement statique. Ces tests supplémentaires ont été effectués afin d’avoir une meilleure compréhension des conditions souterraines à l'emplacement du pieu d'essai, et corréler les données récoltées avec les résultats du chargement statique. Cette investigation supplémentaire comprenait des essais de pénétration standard (SPT), du forage sonique et des essais de pénétration au piézocône (CPT). Les mesures d’excès de pression interstitielle, incluant les valeurs maximales et les taux de dissipation, enregistrées à l’aide des piézomètres à corde vibrante installée à côté du pieu d'essai seront présenté et feront aussi parti des discussions de cet article.

1 INTRODUCTION

As part of the widening of Highway 400 through Innisfil, Ontario, the existing bridge carrying Highway 89 over Highway 400 was replaced with a new, longer bridge as part of Ministry of Transportation, Ontario (MTO) Contract 2018-2024. The abutments and pier were designed to be supported by steel HP 310x110 piles driven to a founding stratum within the overburden.

A static pile load test and associated Pile Dynamic Analyzer (PDA) testing was completed in 2019 adjacent to the proposed Highway 400/89 underpass. The expected ultimate geotechnical resistance was not achieved, as measured by both PDA testing and the static pile load test. Additional geotechnical investigations were then completed at the pile load test site to assess the difference between the expected and achieved ultimate geotechnical resistances.

This paper presents the results of the geotechnical investigation and analyses carried out at the pile load test site to compare the predicted versus actual pile capacity.

2 SITE DESCRIPTION

The Highway 400 and Highway 89 interchange is located about 20 km south of Barrie in the Town of Innisfil, Ontario. Before the widening, Highway 400 consisted of three lanes of traffic in each of the northbound and southbound direction and Highway 89, oriented in an east-west direction, consisted of one lane of traffic in each direction.

The new underpass was built along a new alignment about 30 m north of the existing Highway 89. The new underpass is a two-span integral abutment structure with a total span length of 78 m and a width of approximately 33.5 m.

3 DESIGN SITE INVESTIGATION AND SUBSURFACE CONDITIONS

3.1 Investigation Methods

The initial field investigation was carried out in 2017 with nine boreholes advanced near the foundation elements. In situ Standard Penetration Testing (SPT) and field vane shear testing, using MTO standard “N”-sized vanes, was carried out. Samples of the cohesive soils were obtained at selected locations using 76 mm outer diameter thin-walled Shelby tubes to obtain relatively undisturbed samples. Standpipe piezometers were installed in two 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, three one-dimensional consolidation (oedometer) tests were carried out on selected samples of the upper clayey silt to silty clay deposit.

3.2 Subsurface Conditions

In general, as shown in Figure 1, the subsurface conditions consist of a layer of topsoil or pavement structure and fill, underlain by a sequence of sands, varved clayey silt, silt to silty sand, clayey silt and glacial till. In general, the groundwater levels ranged from 0.7 m to 2.7 m below ground surface. The groundwater level measured in the standpipe piezometer installed in the lower granular deposit was above the top of the lower granular deposit

4 PILE DESIGN

The design factored Ultimate Limit State (fULS) axial geotechnical resistance for two pile length alternatives are shown in Table 1.

The factored Serviceability Limit State (fSLS) geotechnical resistance to mobilize 25 mm of settlement was estimated to exceed the fULS and is not discussed further in this paper.

Table 1. Design fULS axial geotechnical resistances

Element	Approx. Pile Length / Tip Elev. (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)
West Abutment	39.5 / 192.5	Very dense silt and sand to silt	1,250
	51.0 / 181.0	“100-blow” clayey silt/silt and sand till	1,600
Centre Pier	35.0 / 192.5	Dense to very dense silt	1,250
	45.5 / 182.0	Hard clayey silt/very dense silty sand till	1,600
East Abutment	39.5 / 192.5	Very stiff to hard sandy clayey silt	1,250
	51.0 / 181.0	“100-blow” clayey silt/silt and sand till	1,600

The typical approach for predicting pile capacity on MTO projects is to use static formula and whenever possible in situ pile load tests. The design fULS geotechnical resistances outlined in Table 1 were assessed using static methods of analyses based on fundamental soil shear strength parameters. The following five effective stress (σ'_v) and SPT methods for assessing axial pile resistance were considered in the original analysis:

(1) σ'_v - Method using combined shaft resistance coefficient, β (as outlined in *Canadian Foundation Engineering Manual*, CFEM)
2) σ'_v - Method using coefficient of lateral earth pressure, K_s (as partially outlined in CFEM)
3) σ'_v - Method by Poulos and Davis (1980)
4) SPT - Method by Meyerhof (1976)
5) SPT - Method by Decourt (1995)

Limiting values of ultimate shaft and base resistance within the sand deposits were applied to the results of the static analysis methods according to the American Petroleum Institute (API), 1984. Engineering judgment based on previous experience in similar conditions was also applied to the range of results to select an appropriate factored ultimate resistance value for design.

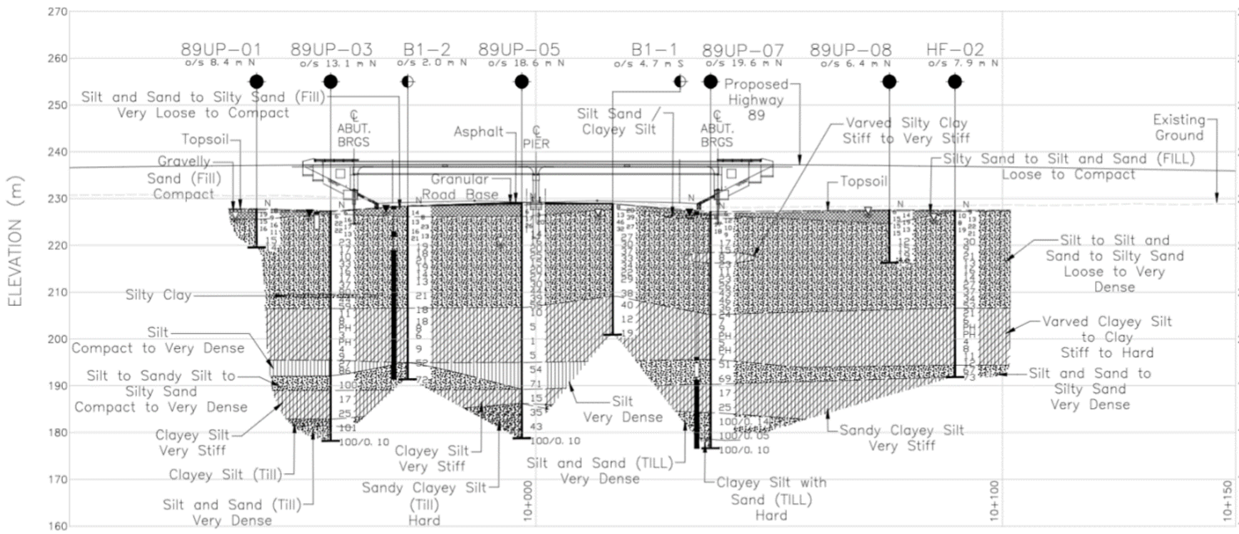

Figure 1: Site Stratigraphy

Table 2: Soil strata thicknesses and material parameters applied within pile design analyses

Strata	Deposit Elevation		SPT N-Values (blows per 0.3 m)	Shear Strength, S_u (kPa)	Unit Weight, γ (kN/m3)	Friction Angle, ϕ' (°)	Coefficients[1]		
	Surface (m)	Bottom (m)					α	β	N_t, N_q or N
Very Loose to Very Dense Non-cohesive Fill	234.6 to 225.7	226.7 to 224.6	3 to 56	-	19	30	-	0.3	-
Loose to Very Dense Upper Granular	227.3 to 224.6	211	6 to 72	-	19	32	-	0.6	-
	211	209.1 to 205.3	30 to 80	-	19	36	-	0.8	-
Firm to Very Stiff Upper Cohesive	209.1 to 205.3	195.7 to 194.0	Weight of Hammer to 40	60	18.5	-	0.63	-	-
Compact to Very Dense Lower Granular	195.7 to 194.0	190.9 to 189.0	27 to 100	-	19	36	-	0.8	110 / 110
Very Stiff to Hard Lower Cohesive	190.9 to 189.0	186.3 to 182.9	15 to 77	100	18.5	-	0.48	-	-
Glacial Till	186.2 to 182.9	178.8 to 176.6[2]	35 to 101	-	21	38	-	0.8	110 / 145

[1]where α is the adhesion coefficient, β is the combined shaft resistance coefficient and N_t, N_q and N are bearing capacity factors.
[2]Boreholes terminated within the glacial till deposit; as such the actual bottom of the deposit is unknown

The soil strata thicknesses and material parameters applied within these analyses were selected based on the design investigation results at the proposed east and west abutments and piers as summarized in Table 2

Using the methods and model parameters described above, fULS resistances of 1,000 kN to 1,260 kN were estimated for piles with tip elevations at 192.5 m (i.e., “shorter" piles of approximately 35 m length) and 1,450 kN to 1,920 kN for piles with tip elevations at 181.0 m (i.e., “longer” piles of approximately 46 m length). All methods estimated a base fULS of 460 kN and the estimated shaft fULS ranged from 540 kN to 960 kN for the shorter piles and 990 kN to 1,460 kN for the longer piles. These results are summarized in Table 8 in Section 7.3.

The parameters used in the above analyses were generally taken as average values, consistent with the 2014 (applicable to design of project) and 2019 (current) *Canadian Highway Bridge Design Code (CHBDC)*, in conjunction with application of the resistance factors for “typical degree of understanding” from CHBDC. As all methods are considered reasonably applicable for the subsurface conditions at this site, the average total (shaft plus base) fULS value was selected for design of “shorter” piles, while a design value between the minimum and average (i.e., between 1,450 kN and 1,740 kN) was selected for the "longer" piles. As previously discussed and shown in Table 3, design fULS values of 1,250 KN and 1,600 kN were selected for the shorter and longer piles, respectively.

5 PILE LOAD TEST

5.1 Background

During detailed design, the merits of conducting a pile load test were reviewed and discussed. In view of the embedment lengths and the number of piles at the three structure foundation locations, the quantity of pile lengths exceeded 5 km. In order to effect cost savings, it was decided to conduct the pile load test in anticipation that the capacity would be greater than the capacity predicted by the abovementioned static formula.

In addition, MTO projects have encountered challenges in achieving the pile capacity at the end of installation and shortly thereafter when restriking the piles. This has necessitated engineering intervention which poses the risk of contractual delay and also the possibility of additional and/or deeper piles – both expensive consequences. To mitigate these risks, investment was made in a static pile load test for this site.

5.2 Test Methods and Procedures

Given the relatively low axial geotechnical resistances estimated, a full-scale static pile load test was undertaken under the direction of Urkkada. The test pile was installed approximately 43 m north of the proposed west abutment (due to staging constraints) to Elevation 192.5 m, approximately 36 m below existing ground surface, and PDA testing and static pile load testing was completed.

Three vibrating wire piezometers (VWP), designated as VWP07, VWP08 and VWP09, were installed 0.5 m from the test pile and three VWPs (designated as VWP10, VWP11 and VWP12) were installed 1.2 m from the test pile monitored before, during and after the test pile installation.

The geotechnical resistance from the static pile load testing was assessed using the Davisson Offset Load Limit Criterion. A CHBDC resistance factor of 0.6 (typical degree of site understanding for a static test) was used to factor the ultimate pile resistances.

Two pile load tests, Load Test #1 and Load Test #2, were carried out in October 2019 on the test pile with the tip at Elevation 192.5 m (within the till), with an interval of two days between tests. The results of the initial static pile load tests indicated that the design ultimate geotechnical resistance for the shorter piles of 2,500 kN (i.e., factored ultimate resistance of 1,250 kN) was not achieved.

In reviewing the PLT results, various alternatives (i.e., changes to the number of piles, pile length and tip elevation, and geotechnical resistance) were discussed with MTO and the designer, and it was decided to drive the test pile to the deeper "100 blow" till deposit. The test pile was then driven to a tip Elevation of 177.75 m, a founding elevation based on the increase in the number of blows per 25 mm of pile driving. Based on boreholes advanced during the design investigation, it was expected that the pile tip was driven into the clayey silt till deposit.

Two additional static pile load tests were subsequently carried out (Load Test #3 on October 28 and 29, 2019 and Load Test #4 on November 12, 2019) on the test pile with the tip at Elevation 177.75 m.

5.3 PDA Test Results

PDA testing was carried out on the test pile with the tip at Elevation 192.5 m, at the end of initial drive (EOID) and during the restriking three days later. PDA testing was also carried out at the EOID after driving the pile to the deeper tip elevation of 177.65 m. The results of the PDA testing and CAPWAP analysis are summarized in Table 3.

Table 3. Results of the PDA testing and CAPWAP analysis

Test Pile Tip Elev. (m)	Testing Condition	Unfactored Resistance (kN)			Factored Total Resistance[1] (kN)
		Shaft	Base	Total	
192.5	EOID	650	200	850	425
	Restrike	875	275	1,150	575
177.75	EOID	975	525	1,500	750

[1]0.5 resistance factor per CHBDC for typical degree of site understanding for a dynamic test.

The results of the PDA testing were similar to the static load test results as discussed in Section 5.4 (recognizing there is a different resistance factor for each test in accordance with CHBDC). The results of the PDA testing revealed that the actual capacity was significantly lower than the predicted design capacity.

5.4 Static Pile Load Test Results

The results of Load Test #1 and Load Test #2, carried out on the test pile with the tip at Elevation 192.5 m, and Load Test #3 and Load Test #4, with the tip at Elevation 177.75 m, are shown in Table 4.

Table 4. Results of Static Pile Load Testing

Test No.	Tip Elev. (m)	ULS[1] (kN)	fULS[2] (kN)	Failure Type
1	192.5	780	468	-
2	192.5	860	516	Plunging at 1,300 kN
3	177.75	1630	978	-
4	177.75	1830	1,098	Plunging at 2,100 kN

[1]Assessed using the Davisson Offset Load Limit Criterion
[2]Factored using CHBDC resistance factor of 0.6 (typical degree of site understanding for a static test)

A plunging failure was observed at a load of 1,300 kN during Load Test #2 carried out on the test pile with tip Elevation 192.5 m. On November 12, 2019 (four weeks after the test pile was driven to Elevation 177.75 m, and two weeks after Load Test #3), the test pile was loaded to plunging failure at 2,100 kN during Load Test #4.

An ultimate (unfactored) geotechnical resistance of 1830 kN was assessed using the Davisson Offset method for the pile driven to tip elevation 177.75 and tested to plunging failure. The factored geotechnical resistance was therefore 1,098 kN (using a 0.6 resistance factor per CHBDC for typical degree of site understanding for a static test), which indicated that the design fULS of 1,600 kN for the longer piles was not achieved.

6 ADDITIONAL POST-TEST INVESTIGATIONS

6.1 Investigation Procedures

Upon completion of the full-scale static and PDA testing, which indicated that the design geotechnical resistances were not achieved for either the shorter or longer pile, additional investigations were completed to assess the subsurface conditions at the test pile location, reassess the methods of analysis for estimating the geotechnical resistance, and consider the potential reasons for the differences between the estimated and test capacities.

The additional geotechnical investigation was carried out at the pile load test site in January and February 2020. Three test holes (Boreholes TP-20, SCPT20-TP20 and Borehole TP-20S) were advanced adjacent to the test pile.

Borehole TP-20 was advanced to a depth of 55.1 m (Elevation 173.4 m), about 4.3 m below the final test pile tip elevation. Soil sampling, SPT and field vane shear testing were carried out as described in Section 3.1.

SCPT20-TP20, an SCPTu test with pore pressure dissipation (PPD) and shear wave velocity measurements, was carried out to refusal three times at depths of 16.1 m (Elevation 212.2 m), 32.6 m (Elevation 195.7 m) and a final depth of 47.0 m (Elevation 181.3 m). Upon refusal at Elevation 212.3 m and 195.7 m, sonic coring methods were used to obtain a continuous soil core from the top of the cone push to a depth where SCPTu testing could be resumed to the next refusal depth. The final depth of cone refusal, 47.0 m below ground surface, was approximately 3.5 m above the test pile tip.

Borehole TP-20S was advanced concurrent with and over top of the SCPTu, with a continuous soil core sample obtained from ground surface to a depth of 54.7 m (Elevation 173.6 m), about 4.2 m below the final tip elevation of the test pile.

6.2 Investigation Results

Based on the additional subsurface investigation completed, the stratigraphy at the test pile site consists of a 0.6 m thick sand and gravel fill layer, underlain by a 21.2 m to 21.8 m thick upper granular deposit consisting of compact to very dense silt to sand, which is underlain by a 9.7 m to 10.8 m thick upper cohesive deposit consisting of firm to stiff clayey silt. The upper deposits are underlain by a 7.7 m to 6.0 m thick lower granular deposit consisting of dense to very dense silt to silty sand, which is underlain by a 4.5 m to 7.0 m thick lower cohesive deposit consisting of very stiff to hard clayey silt.

These deposits are underlain by a glacial till deposit consisting of hard clayey silt and very dense sandy silt to silt and sand, which was not fully penetrated but is at least 11.4 m thick. A layer of relatively loose silt and sand was encountered within this till, approximately 1.95 m below the final test pile tip elevation.

The assumed phreatic surface based on pore pressure dissipation tests completed as part of the SCPTu at Borehole TP-20S is at a depth of approximately 1.3 m (Elevation 227.0 m).

7 ANALYSIS

7.1 Subsurface Conditions Comparison

The elevation of the interface of each deposit, as encountered in Boreholes TP-20 and TP-20S, is generally within the ranges of the interface elevations encountered during the original investigation for design. The results of the geotechnical index testing were also generally consistent between the two investigations. As shown in Figure 2, the SPT N-values measured in Borehole TP-20 and the N-values interpreted from SCPTu20-TP20 are within the range of SPT N-values collected within each of the native deposits encountered during the design investigation.

As shown in Figure 3, the shear strength estimated using the results of SCPTu20-TP20 is generally higher than the design line values for the upper stiff to very stiff cohesive deposit, except between about Elevations 201 m to 202 m, where the design line is consistent with the average strength from the CPT results.

Although the surface of the till deposit in Borehole TP-20 is generally consistent with the elevation at which the till was encountered in the original design investigation boreholes, the zone in which SPT “N” values are greater than 100 blows per 0.3 m of penetration (“100-blow soil”) is at least 0.7 m deeper in Borehole TP-20 adjacent to the test pile. In addition, a layer of loose silt and sand was encountered within the till deposit approximately 1.95 m below the final tip elevation of the test pile.

7.2 Porewater Pressures

The porewater pressures recorded during the CPTu advance are shown in Figure 4 relative to elevation. The hydrostatic pressure (based on a water level 1.3 m below ground surface) and the peak water pressures recorded in the VWPs 0.5 m from the test pile during piling are also shown, along with the strata at the VWP tip elevation in Figure 4. The peak water pressures recorded by the VWPs dissipated within a few minutes in the sand and silt deposits and within a few hours in the clayey silt.

Based on the monitoring data from the VWPs 0.5 m from the test pile, the excess pressure dissipated prior to the completion of the PDA test at EOID and prior to the completion of subsequent PDA tests that indicated a strength gain over time.

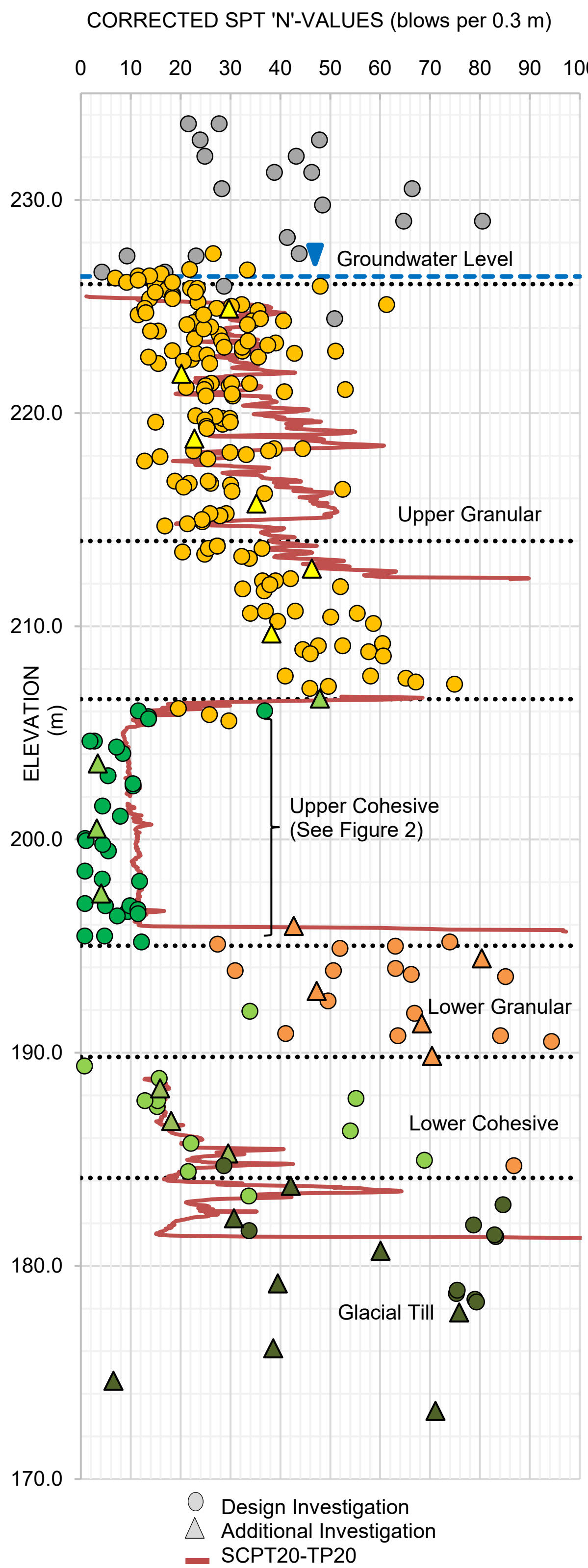

Figure 2. Corrected SPT N-values vs. Elevation.

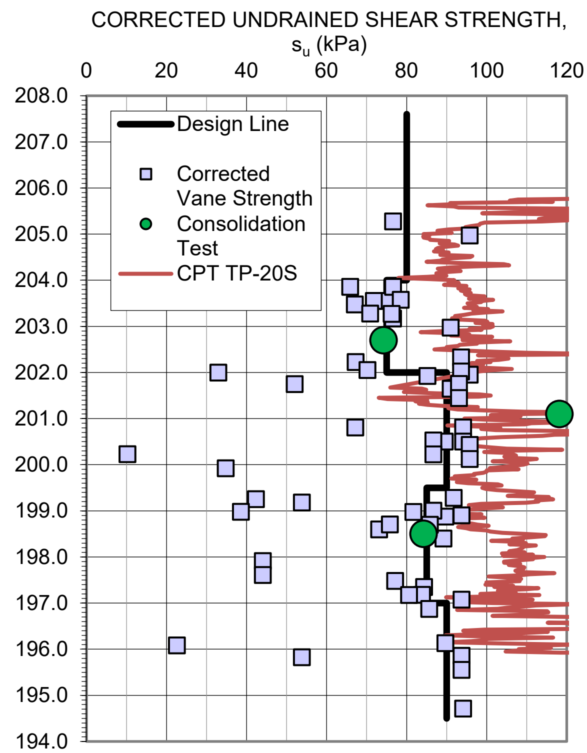

Figure 3. Undrained Shear Strength in Upper Stiff to Very Stiff Cohesive Deposit.

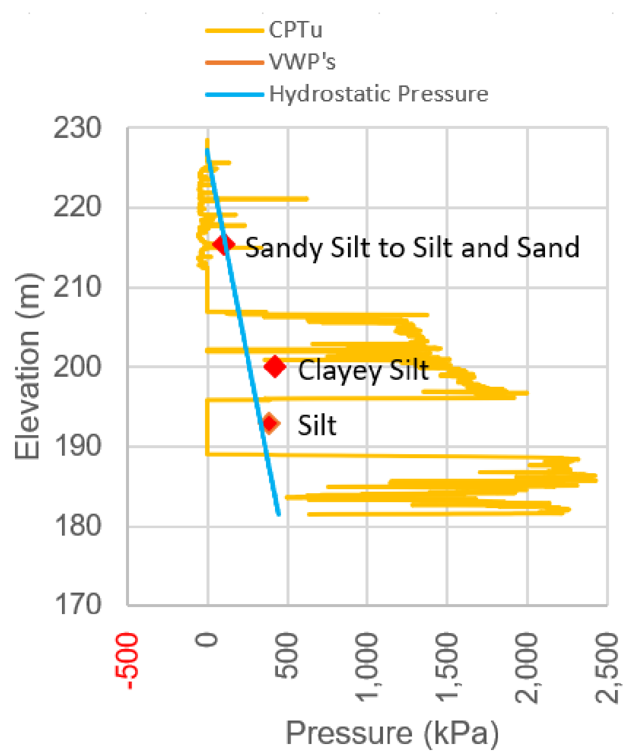

Figure 4. Porewater Pressure vs. Elevation

7.3 Analysis Methods

Analyses for the test pile geotechnical resistance were carried out in accordance with the five effective stress and SPT methods described in Section 4, based on the soil stratigraphy encountered in the test holes (i.e., Borehole TP-20, SCPT20-TP20 and Borehole TP-20S) at the test pile location.

In addition, the test pile geotechnical resistances were estimated based on the SCPTu data (i.e. Borehole No. SCPTu20-TP20). The following two direct cone penetration testing (CPT) methods for determining axial pile capacity were utilized to assess the axial pile capacity for comparison with the design and achieved values:

1) The French Method or Laboratoire Central des Ponts et Chauses (LCPC) Method (Bustamante and Gianeselli, 1982), as described in CFEM
2) CPTu Method (Eslami and Fellenius, 1995, 1996; Eslami, 1996)

The direct CPT methods equate the measured cone resistance to the pile unit toe resistance and, as the sleeve friction is a more variable measurement, proportion the shaft resistance to the measured cone resistance.

The CPT methods were used to assess solely the portion of the design axial pile capacity provided by shaft resistance. These methods could not be used to assess the test pile's base resistance in this case as the cone penetration test, SCPTu20 TP20, was terminated 3.5 m above the test pile tip, and could not be pushed through the soil deposits of the founding stratum.

It should be noted that only one CPT was available for the analyses. Ideally more CPTs with greater site coverage would be used for these analyses to reduce uncertainty.

Table 5 provides a summary of the unfactored shaft ultimate geotechnical resistances for a driven HP 310x110 pile based on the five effective stress and SPT methods and the two CPT methods.

7.4 Design and Post-Test Resistance Comparison

Table 5 summarizes the original geotechnical resistances for the piles estimated during the design and the post-test geotechnical resistances estimated based on the investigations at the test pile. The comparison in Table 5 indicates the differences between the original and post-test estimates are relatively small, in the order of 100 to 200 kN, and are likely due to the small changes in strata boundaries. Also as shown in Table 5, the base resistance is typically estimated to be higher than the API limits and the base resistance values are therefore governed by the API limits.

8 DISCUSSION

The results of the static pile load testing, PDA testing and CAPWAP analysis are less than predicted by all design methods and are less than the API limit (i.e., the base resistances from PDA testing are all less than API limiting values). This suggests that all of the design methods over-estimate the pile capacities for this particular site and subsurface conditions, with the resistance over-estimated in all analyses by factors of about 2 to 3.

For the shorter piles, the analysis estimates of base resistance are about five times the resistance indicated by the PDA testing. For the deeper piles, the ratio of the estimated base resistance to the test resistance is less than the ratio for shaft resistance. This should be expected since the base resistance at both pile tip elevations is the same, due to the application of the API limiting values, and the ratio will be less as the shaft length and therefore the shaft resistance rises. This suggests that it may be prudent to further discount the base resistance when there is a risk that the pile toes will not be founded within hard/very dense soils that may contain thin loosened zones (as at this site), to less than the API limits for design.

Table 5 also indicates that the accuracy of the shaft and total resistance values estimated from analysis (in comparison to the test results) does not increase with depth and is, in fact, slightly worse at the deeper pile tip elevation.

The static methods of analysis for estimating pile resistance require engineering judgement, which introduces uncertainty, when choosing material parameters to represent shaft resistance. The choice of material parameters is often based on correlations that may not be applicable to the site-specific materials and conditions because the research used to develop these methods is often specific to particular materials (e.g., clean sands) or to specific research sites and conditions and the difference between a particular design site and the site or materials used for development of the empirical formulae may be subtle or not fully understood.

The geotechnical resistance analyses using dynamic cone penetration testing results could be considered less sensitive to engineering judgement, but those methods use CPT tip resistance as a proxy when calculating shaft resistance, which is again not a direct measurement for the relevant material parameters for use in design. It is also worth noting that one of the CPT design methods over-predicts the resistance achieved at this site by a factor of about 5.

In addition to the inherent uncertainty in friction pile design, the inconsistencies in depth to and thickness of the "100-blow" glacial till material and the presence of a looser silt and sand zone at the test pile location, as described in Section 7.1, could have contributed to some uncertainty in the estimation of toe resistance.

The results also indicate that there is a time-dependent gain in resistance. Static Load test #4, carried out four weeks after the initial static load test with the pile tip at the final elevation of 177.75 m, showed a 12% gain in fULS in comparison to Static Load Test #3. There was a similar gain of about 10% in the two-day interval between Static Load Test #1 and Static Load Test #2. This is consistent with the results of the VWP measurements which indicate a significant increase in excess pore water pressure of up to 195 kPa during and immediately after driving the pile. This excess pore water pressure dissipates quickly thereafter.

Table 5. Summary of Estimated Geotechnical Resistance based on Detail Design and Test Pile Analyses

Investigation	σ'v,SPT or CPT Method	Pile Tip at Elev. 192.5 m Shaft fULS (kN)	Base fULS (kN)	Total fULS (kN)	Pile Tip at Elev. 181.0 m Shaft fULS (kN)	Base fULS (kN)	Total fULS (kN)
Detailed Design (2017)	σ'v - Method 1	960	460	1420	1460	460	1920
	σ'v - Method 2	540	460	1000	990	460	1450
	σ'v - Method 3	810	460	1270	1230	460	1680
	SPT - Method 4	820	460	1280	1320	460	1770
	SPT - Method 5	900	460	1350	1400	460	1860
Post-Test Pile Analysis (2020)	σ'v - Method 1	1120	460	1580	1600	460	2060
	σ'v - Method 2	650	460	1110	1000	460	1460
	σ'v - Method 3	1030	460	1490	1430	460	1890
	SPT - Method 4	1030	460	1490	1580	460	2040
	SPT - Method 5	1190	460	1640	1740	460	2200
	CPT - Method 1	650	N/A	N/A	860	N/A	N/A
	CPT - Method 2	1640	N/A	N/A	1930	N/A	N/A
Detailed Design (2017)	Min. – Max. (average)	540 – 960 (800)	460	1000 – 1420 (1260)	990 – 1460 (1280)	460	1450 – 1920 (1740)
Post Test Pile Analysis (2020)[1]	Min. – Max. (average)	650 – 1190 (1000)	460	1110 – 1640 (1460)	1000 – 1740 (1470)	460	1460 – 2200 (1930)

[1]Minimum, maximum and average fULS values of the post-test pile analysis do not include the shaft fULS estimated using CPT methods

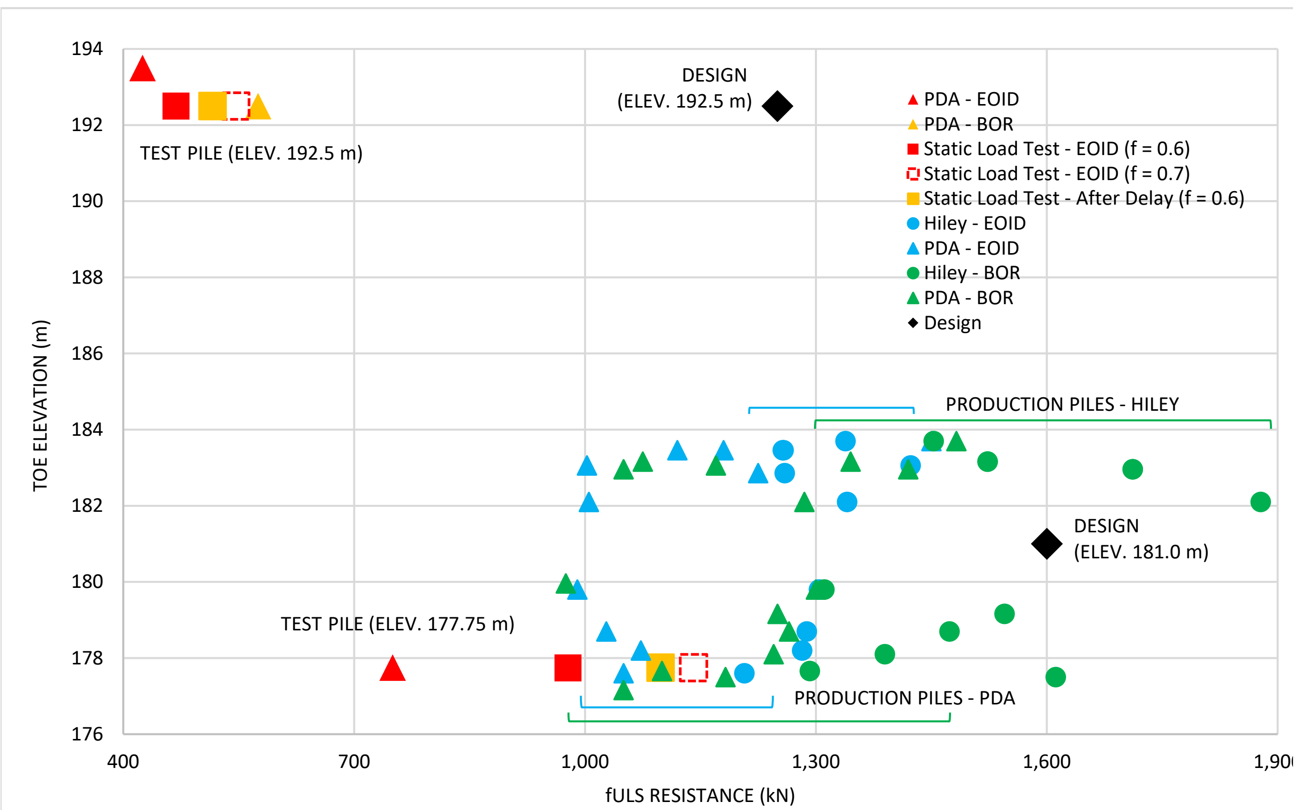

Figure 5: Factored Ultimate Limit State Resistance vs. Toe Elevation

As can be seen in Figure 5, the fULS estimated from the pile load test at Elevation 177.75 m (i.e., 978 kN at EOID) is at the low end of the range of results from PDA testing carried out on the production piles at the abutments and pier of the new bridge, which range from 990 kN to 1450 kN for EOID, and 975 kN to 1483 kN for BOR. It is worth noting that if a high degree of site understanding had been assumed for the static load test (which may be justified based on the level of investigation), the obtained fULS would have been 1,270 kN (i.e., within the mid-range of the values obtained from the PDA tests on production piles).

9 CONCLUSIONS

The results of the investigations and testing outlined in this paper do not conclusively demonstrate why the achieved pile capacity varied significantly from the design value.

Piles have been designed using these methods in similar soils within the Greater Toronto Area in Ontario, Canada, and have been successfully constructed with ultimate geotechnical resistances verified by PDA testing, and therefore the typically accepted empirical methods do have applicability for design. However, the analysis for this site implies the design methods may not account for the potential variability in subsurface conditions and may expose the owner to more construction risk than may be understood, since these methods have resulted in prior successes. Infrastructure owners can accept that risk or choose to implement design or construction measures to further reduce that risk, understanding that there are costs associated with both approaches.

Accepting the risk will result in additional costs on some projects when testing indicates the available pile resistance will be less than needed and re-design (e.g., more or deeper piles) will need to be undertaken, with a corresponding, unplanned increase in construction cost as a result of a claim by the contractor. To manage this risk, it is suggested that contracts include unit rate provisions for reasonably longer piles than shown in the contract documents.

The owner can alternatively choose to be more conservative in design (which implies additional cost for structures since more or deeper piles will be employed for design) using the typical methodologies outlined herein. That could be accomplished by neglecting base resistance, limiting the shaft resistances, using lower resistance factors (e.g., low degree of site understanding), or some combination of the foregoing.

The results also indicate that some delay, of even 2 to 3 days, between driving the pile and PDA testing will result in an increase in the tested resistance, which may be helpful for piles with marginal results.

Additional analysis of these results and from other sites may help to more fully understand the behaviour and risks of friction piles installed within these types of soils. Further work is planned to look at mechanisms, such as ‘frictional fatigue’ as described by Randolph et al. 1994, that may have affected the realized capacity of the piles.

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