Axial Resistance Gain of Piles Driven into Artesian Soils: Case Studies

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ABSTRACT
This paper presents two case studies that describe the results of static pile load tests at two bridge replacement sites for Ministry of Transportation Ontario (MTO) contracts in southern Ontario, Canada. Site #1 is located at Highway 401 and Fletcher's Creek in Mississauga, and Site #2 is located at Highway 400 and South Canal in the Regional Municipality of York. Both contracts involve the removal of existing structures and replacement with new bridges, under multi-year construction staging. The design of the new bridges included steel H-piles driven to refusal in cohesionless silty sand to sandy silt and sand and gravel till deposits under artesian groundwater conditions. Full-scale static pile load tests were conducted during construction, several months after the installation of the test pile at each of the sites. The purpose of the full-scale pile load tests was to compare the actual geotechnical resistance achieved in the full-scale tests to the estimated design geotechnical resistance value, as well as to resistance values measured on initial driving and retap using dynamic formula (Hiley) and/or pile dynamic analyzer (PDA) testing. The results of the full-scale static pile load tests are evaluated to assess set-up or strength gain over time, and examine whether higher geotechnical resistances can be achieved for piles driven to or into multi-layered soils under artesian conditions, for applicability at these and other highway bridge sites in similar conditions. Specifically, at the test sites, in view of the construction staging and the length of time between pile driving and bridge deck construction, the pile load test results have been used to optimize the design and construction of the next stage of pile foundations.

RÉSUMÉ
Cet article présente deux études de cas qui décrivent les résultats d'essais de charge statique de pieux sur deux sites de remplacement de ponts pour des contrats du ministère des Transports de l'Ontario (MTO) dans le sud de l'Ontario, au Canada. Le site no 1 est situé à l'autoroute 401 au Fletcher's Creek à Mississauga, et le site no 2 est situé à l'autoroute 400 au South Canal dans la municipalité régionale de York. Les deux contrats prévoient l'enlèvement des structures existantes et leur remplacement par de nouveaux ponts, dans le cadre de constructions s'étalant sur plusieurs années. La conception des nouveaux ponts comprenait des pieux en H en acier foncés dans des silts sableux non-cohérant et des dépôts de till et de sable et gravier dans des conditions artésiennes. Des essais de charge statique à pleine échelle ont été effectués pendant la construction, plusieurs mois après l'installation d'un pieu d'essai à chacun des sites. Le but des essais de charge à pleine échelle était de comparer les résistances géotechniques de conception prévues aux valeurs de résistance géotechniques mesurées lors du fonçage initial et du retapage des pieux au moyen de la formule dynamique (Hiley) et / ou d'analyseur dynamique de pieu (PDA), et les résistances géotechniques réelles obtenues dans les essais de charge statique plusieurs mois après l'installation des pieux, afin d'évaluer le gain de résistance avec le temps. Les résultats des essais de charge statique à pleine échelle sont examinés pour déterminer si des résistances géotechniques plus élevées peuvent être obtenues pour des pieux foncés dans des conditions artésiennes, pour appliquer ces résultats aux sites mêmes et à d'autres ponts dans des conditions similaires. Particulièrement aux sites d'essais, les résultats des essais de charge de pieux ont été utilisés pour optimiser la conception et la construction des phases suivantes d'installation de pieux, en tenant compte du temps écoulé entre la construction des pieux et la construction du tablier.

1 INTRODUCTION

Many water-crossing sites in Ontario are underlain by multi-layered soils under artesian conditions. Typically, these aquifer soils are comprised of silts, sands and gravels overlain by a thickness of a lower permeability or cohesive soil (i.e., an aquitard). Where the near-surface soils do not permit the use of shallow foundations due to inadequate bearing resistance, the preferred foundation option is frequently low-displacement driven piles such as steel H-piles. Other deep foundation options such as drilled shafts are considered to have higher risks around basal and sidewall instability in these conditions.

The determination of the axial resistance for piles driven to refusal within artesian zones is challenging due to the existing porewater pressure regime and the additional porewater pressures generated during driving. To account for the low level of understanding and the typical to high consequence for these highway bridges, empirical methods that are commonly used are considered to produce conservative axial pile resistances, and hence foundation designs that may be more expensive than required. Recognizing the value that full-scale pile load tests can provide in optimizing foundation design in these conditions, the Ministry of Transportation Ontario (MTO) conducted pile load tests at two artesian bridge sites in 2017.

The purpose of these tests was to compare the actual geotechnical resistance achieved in the full-scale tests to the design geotechnical resistance value estimated by geotechnical analyses, as well as to dynamic formula and/or dynamic analyses that were measured at the time of pile driving. This comparison was intended to verify and optimize correlations between theories, measured and calculated dynamic test results after initial driving, and actual measured longer-term geotechnical resistance of the pile in compression.

At both sites, the pile load testing occurred several months after pile installation (153 days at Site #1, and 258 days at Site #2). After these elapsed times, the full-scale pile load tests were expected to measure higher axial geotechnical resistances in comparison to the resistances calculated from dynamic testing performed on completion of initial driving of the test pile to the design tip elevation; however, this needed to be confirmed. This phenomenon of "set-up" or strength gain over time is attributed to an increase in porewater pressure within the soil matrix surrounding the pile during driving, followed by dissipation of porewater pressures over time leading to an increase in effective stress and associated gain in geotechnical resistance. Pile "set-up" has been studied analytically and proven in the field for more typical soil and groundwater conditions using full-scale pile load tests and dynamic testing methods in the past (McVay et al. 1999, Fellenius et al. 1989, Komurka et al. 2003); however, limited information is available to confirm strength gain for piles installed in artesian conditions.

The results of the full-scale pile load tests at Site#1 and Site#2 have been used to allow informed decisions to be made during the installation of the remaining stages of piling works. With the knowledge of strength gain with time for these sites, applying a strength gain correction can result in acceptance of "low" pile resistances at the time of initial driving. In addition, consideration can be given to using higher design geotechnical pile resistances at the site to optimize the next stage of pile installation.

The results of the full-scale pile load tests are also intended to be used for broader benefit for projects in similar artesian conditions. MTO is using the results to develop a better understanding of the potential increase in geotechnical resistance with time for piles under similar artesian pressures, and hence enable the use of higher axial geotechnical resistance values during design and construction of other highway structures in similar conditions.

2 SITE LOCATIONS

Site #1 – The Highway 401-Fletcher's Creek bridges are located approximately 1 km west of the Highway 401-Mavis Road interchange, and approximately 1 km east of the Credit River, in the City of Mississauga. Highway 401 in this area is currently a three-lane freeway in both eastbound and westbound directions. An existing double cell concrete box culvert is being replaced with two single-span bridges to accommodate the widening of Highway 401 over Fletcher's Creek. The foundations for the new bridges consist of driven steel HP 310x110 piles as part of an integral abutment design.

Site #2 – The Highway 400–South Canal bridges are located approximately 0.5 km north of Highway 9 in the Regional Municipality of York. Highway 400 is currently a three-lane freeway in both northbound and southbound directions, with existing multi-span bridges spanning over the South Canal waterway and Canal Bank Road. The replacement bridges are two-span structures with conventional abutments and centre piers supported on driven steel HP 310x110 piles.

For both sites, the construction is being performed in stages over multiple years to maintain the freeway traffic flow.

3 SUBSURFACE CONDITIONS

Site #1 – Fourteen boreholes were advanced as part of the foundation investigation at this site. The subsurface conditions generally consist of clayey silt fill to a depth of about 0.7 m to 1.5 m (outside of the existing highway embankment), underlain by firm to hard clayey silt with sand till to a depth of about 5.6 m. Below the cohesive till, the boreholes encountered a non-cohesive deposit of dense to very dense sand and silt to sand and gravel till (aquifer), containing cobbles and boulders. Artesian groundwater conditions were measured at the site during the foundation investigation, with levels as high as 1.2 m to 5 m above the ground surface at the borehole locations, corresponding to a piezometric level of about Elevation 165 m to 172 m.

Site #2 -Thirteen boreholes were advanced as part of the foundation investigation at this site. Based on the closest borehole to the test pile, the subsurface conditions generally consist of firm clayey silt containing organics / peat to a depth of about 2.6 m, underlain by firm to stiff clayey silt to silt containing silty sand interlayers to a depth of about 17.8 m, underlain by a compact to very dense sand and silt deposit (classified as a till in some adjacent boreholes) to a depth of 23.9 m, in turn underlain by interlayers of hard clayey silt and very dense sand to sand and gravel to the termination depth of 26.5 m. Flowing artesian groundwater conditions were encountered below a depth of 25.9 m, and the water level was measured to be 1.6 m above ground surface, corresponding to a piezometric level of about Elevation 222.6 m. Nearby boreholes were terminated as deep as 27.9 m and encountered similar conditions, with unstable "piping" conditions encountered in the boreholes at depths as shallow as 15.2 m below ground surface.

The subsurface conditions in the vicinity of the test piles are shown on Figures 1a and 1b for Site #1 and Site #2, respectively.

4 TEST PILE INSTALLATION

The test piles were to be installed consistent with the design specifications and installation method(s) that for the production piles at each site. The approximate location of the test piles are shown on Figures 2a and 2b.

Site #1 - Pile Driving Analyzer (PDA) testing and Hiley Dynamic Formula (Hiley) testing in accordance with MTO

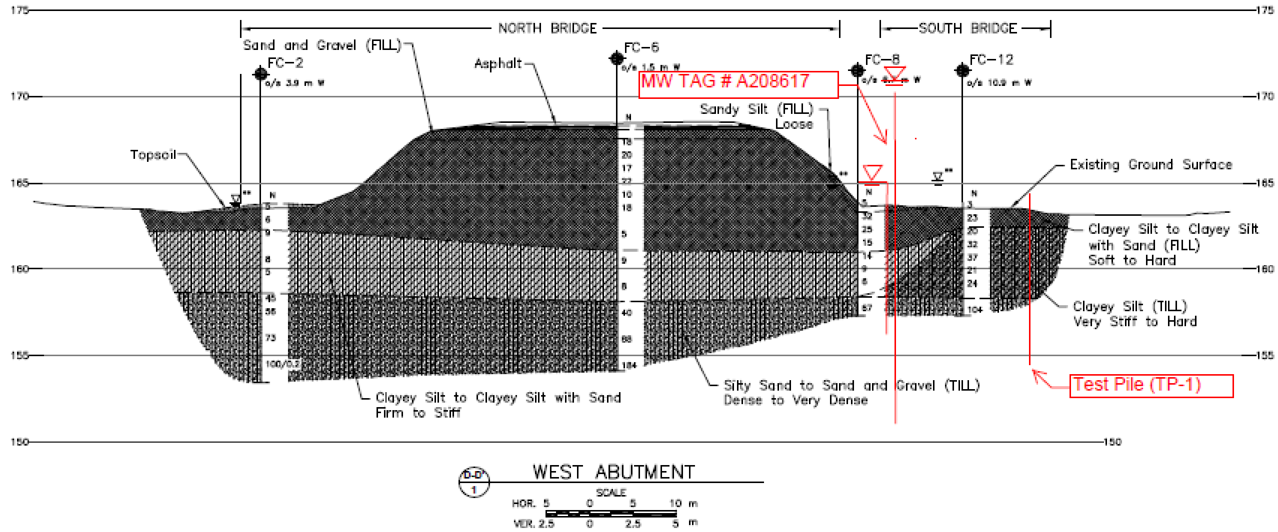

Figure 1a: Site #1 - Typical subsurface section at the area of the test pile

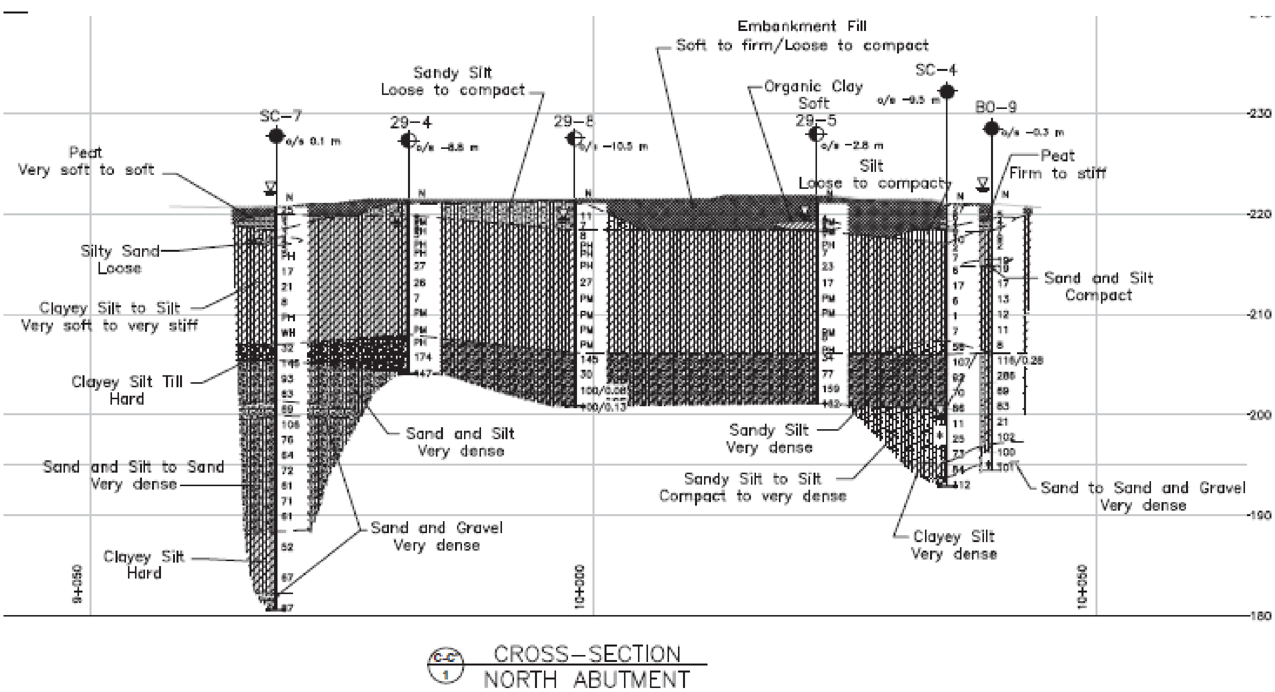

Figure 1b: Site #2 – Typical subsurface section at the area of the test pile

SS113-11 were performed during installation of the test pile. The steel HP310x110 test pile was driven using a Delmag D19-42 diesel hammer with a maximum rated energy of about 66 kJ. The average energy transferred to the top of the pile as measured by PDA testing was about 30 kJ with the hammer operating at a speed of about 38 blows per minute. The existing ground surface at the test pile location was at about Elevation 164.0 m, and the pile was driven to tip Elevation 154.4 m (i.e. final embedment depth of about 9.6 m below ground surface). The test pile was left in place over the winter and spring, and a pile load test (in general accordance with ASTM D1143) was performed in May 2017, approximately five months after pile installation.

Site #2 – PDA testing was performed during installation of the steel HP310X110 test pile, which was driven using a Liebherr H40/4 hydraulic hammer with a rated energy of about 30 kJ. The existing ground surface surrounding the test pile was at about Elevation 220.5 m, and the pile was driven to a final tip elevation of 204 m (i.e., final embedment depth of about 16.5 m below ground surface). The test pile was left in place over the winter and spring, and a pile load test (in general accordance with ASTM 1143) was performed in June 2017, approximately eight months after pile installation.

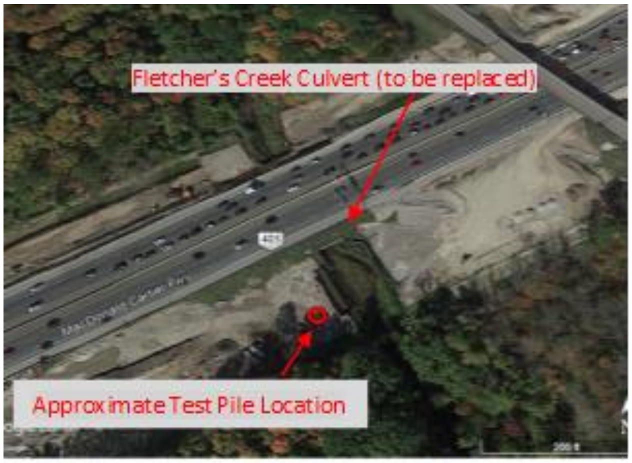

Figure 2a - Pile load test locations for Site #1

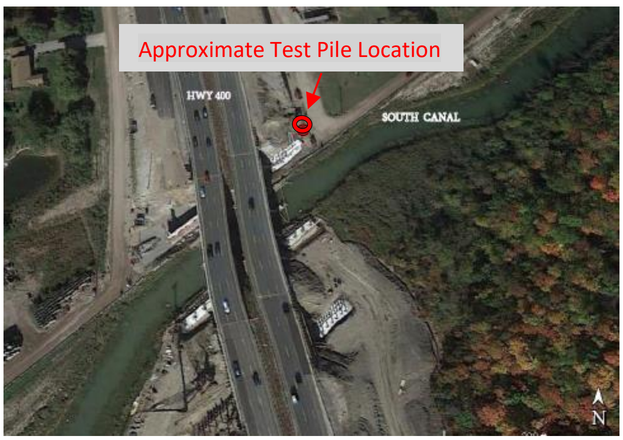

Figure 2b - Pile load test location for Site #2

5 PILE LOAD TEST SET-UP AND PROCEDURE

Site #1 -The reaction system required for the pile load test consisted of two W920x420 steel reaction beams supporting a dead weight (consisting of approximately 174 steel H-piles stacked in eight layers), temporarily supported on timber cribs. Each row of steel piles acted as beams to support the next layer which was shorter and placed perpendicular to the row beneath for increased stability and to resist the planned maximum test load of 2,600 kN.

Site #2 - The reaction system for the pile load test consisted of two W920x420 steel beams anchored at each end by two steel W610x155 beams, connected to reaction anchors consisting of two 760 mm diameter, 7 m deep caissons. A total of four reaction caissons were designed to resist the planned maximum test load of 2,550 kN

For both sites, a hydraulic cylinder jack was used to transfer the load between the top of the test pile and the reaction beam. Four dial gauges were set up radially on a reference frame to measure the vertical movements of the top of the pile as the test progressed. The dial gauge readings were used as the primary measurement system for pile axial movements, and a series of additional survey points at the top of the pile were measured periodically by a licensed surveyor as a secondary measurement system during the test. Figures 3a and 3b illustrate the pile load test set-up in progress for both sites.


Figure 3a - Site #1 Pile Load Test Set-Up


Figure 3b - Site #2 Pile Load Test Set-Up

The static load test at each site was carried out in general accordance with ASTM D1143-07 using a modified Procedure A – Quick Test method. For both test sites, loading was to be carried out in seven increments up to the maximum test load; however, at Site #2, the failure load was reached prior to achieving the maximum test load and only five loading increments were completed. All loading increments were held for a minimum 20 minutes, or until the rate of displacement was measured to be less than 0.25 mm/hour up to a maximum of 2 hours. The maximum load (if reached) was held for a total of 12 hours, and then the test pile was unloaded in four increments with a displacement reading taken prior to each unloading stage, and with a final displacement reading taken at least 6 hours after removal of the total load.

Load increments were applied by adjusting the hydraulic jack pressures and quality assurance was provided to check loading increments, calculate and determine hold times, and record the pile displacements from the dial gauges (i.e. primary measurement system). An independent surveyor recorded survey measurements of the pile displacement (secondary measurement system) and of several points on the reference beams and reaction frame to check that the set-up remained stable.

The pile load test at each site was completed within a period of about 24 hours. Construction operations at Site #1 were halted by the Contractor to minimize the impact of construction-induced vibrations on the test. Ongoing construction traffic adjacent to Site #2 led to noticeable “felt” vibrations at the area of the test; although the vibrations did not appear to affect dial gauge or survey reading accuracy, it is not known whether vibrations may have impacted the geotechnical resistance of the test pile and/or the performance of the reaction system caissons.

Reference beam deflections were reported to be less than 1 mm through the duration of the tests. For Site #1, the reaction frame (i.e. stacked H-piles) was observed to remain stable throughout the loading procedure. For Site #2, while attempting to increase the test load above 1,800 kN, the caisson reaction system started to heave / fail and the subsequent load increment of 2,200 kN could not be maintained before the hydraulic jack reached the maximum stroke. As a result, the Site #2 pile load test was interrupted and the pile completely unloaded to allow for the reaction frame/ beams to be lowered and re-set, after which time the load was re-applied in an attempt to reach the planned 2,200 kN load increment. However, the load could not be maintained on this second attempt and ultimate failure of the reaction caissons was reached.

6 PILE LOAD TEST RESULTS

A summary of the pile load test results for Site #1 and Site #2 is shown on Figures 4a and 4b, respectively. These figures include plots of: i) applied load vs. time; ii) pile movement vs. time; and iii) pile movement vs. applied load.

7 DISCUSSION

A comparison of the test pile ultimate and factored geotechnical resistance measured or calculated from the PDA test, Hiley test (where performed) and full-scale pile load test for both Sites is provided in Table 1. Further discussion on the original design resistance versus the tested pile resistance, and strength gain versus time, are provided below for each of the Sites.

7.1 Site #1

Due to the relatively high artesian conditions at this site, the foundation design recommended that steel HP 310x110 piles driven to found within the “100-blow” non-cohesive till be designed based on an ultimate geotechnical resistance of 1,800 kN and factored ultimate geotechnical resistance (f-ULS) of 900 kN; these values are lower than would typically be given for piles of similar length driven into very dense soils in non-artesian conditions. In addition to the artesian zones, the depth to “100-blow” soils was variable, and the presence of cobbles/boulders within the till created uncertainty in the design, with a potential requirement for variable pile lengths and tip depths.

At Site #1, the f-ULS values determined from the Hiley, PDA and pile load test are all greater than the design factored geotechnical resistance of 900 kN; in fact, during construction at this site, the majority of the production piles achieved the required ultimate geotechnical resistance upon initial driving.

The f-ULS resistance values from the Hiley and PDA test on initial driving of the test pile are about 5% to 22% higher than the design values; although the test pile was not re-tapped. Selected production piles at this site were re-tapped (typically within one day) and demonstrated some nominal increases and decreases from the Hiley or PDA test results on initial driving. By comparison, the pile load test result obtained 153 days after installation is more than 70% greater than the design value, although it is noted that the test pile did not reach failure at the maximum tested load of 2,600 kN in compression (see Figure 4a). The potential gain in geotechnical resistance over time increases by a greater proportion for factored resistances, considering the resistance factor for a full-scale pile load test is 0.6 compared to a factor of 0.4 and 0.5 for Hiley and PDA testing.

Comparing the results of the Hiley and PDA tests performed upon completion of initial driving and the full-scale pile load test performed 153 days after initial driving, there is greater than a 10% to 18% increase in the estimated ultimate geotechnical resistance (2,368 kN and 2,200 kN vs. 2,600 kN) over this approximately five-month timeframe. The actual increase in geotechnical resistance cannot be fully established from the results of the static pile load test, as the pile was not tested to ultimate failure; however, the results of the full-scale pile load test clearly suggest that set-up and long-term strength gain is achievable.

7.2 Site #2

The foundation design recommended that steel HP 310x110 piles driven into the “100-blow” soils would have a design ultimate geotechnical resistance of 2,550 kN and f-ULS of 1,275 kN; this was based on pile lengths of at least 16.5 m (pile tip levels at about Elevation 203 m to 204 m at the north abutment), accounting for some variability in the surface of the “100-blow” soil. While piles at the south abutment and centre pier achieved the required design resistances during initial driving or on retap, many production piles installed along the north abutment did not achieve the design geotechnical resistance, and PDA test results demonstrated ultimate (unfactored) geotechnical resistance values as low as 1,500 kN when tested within 24 hours after initial driving.

The lowest calculated PDA value of 1,100 kN was at the test pile location for the static pile load test, immediately upon completion of initial driving. Based on the PDA test results on initial driving, the estimated geotechnical resistance value in compression (ultimate geotechnical resistance of 1,100 kN, and f-ULS of 550 kN) was significantly lower than design requirements (f-ULS of 1,275 kN) at the design tip elevation. As per the test pile work plan, this test pile was intended to be driven deeper than Elevation 204 m if necessary, to attempt to achieve an ultimate geotechnical resistance value closer to the design value and more consistent with the PDA values measured on the production piles driven previously; however, the test pile was not driven deeper. The results of the PDA testing and reported “sets” on the test pile, compared with the results on the previous production piles, suggest that predominant end-bearing resistance within the “100-blow” soil was not effectively achieved.

Table 1 - Pile Geotechnical Resistance Comparison

Site	Pile Length (m)	Estimated Artesian Condition	Geotechnical Resistance (Analytical Results)			Geotechnical Resistance (Test Results)				
			Ultimate (kN)	Factor (CHBDC, 2014)	f-ULS (kN)	Test Method	Set-Up Period (Days)	Measured Ultimate (kN)	Factor (CHBDC, 2014)	Actual f-ULS (kN)
Site #1	9.6	+4 m above grade	1,800	0.4	900	Hiley	0	2368	0.4	947
						PDA	0	2200	0.5	1100
						PLT	153	>2,600	0.6	>1,560
Site #2	16.5	+1.6 m above grade	2,550	0.4	1,275	PDA	0	1,100	0.5	550*
						PLT	258	1,700	0.6	1,020*

*pile tip elevation was higher than desired

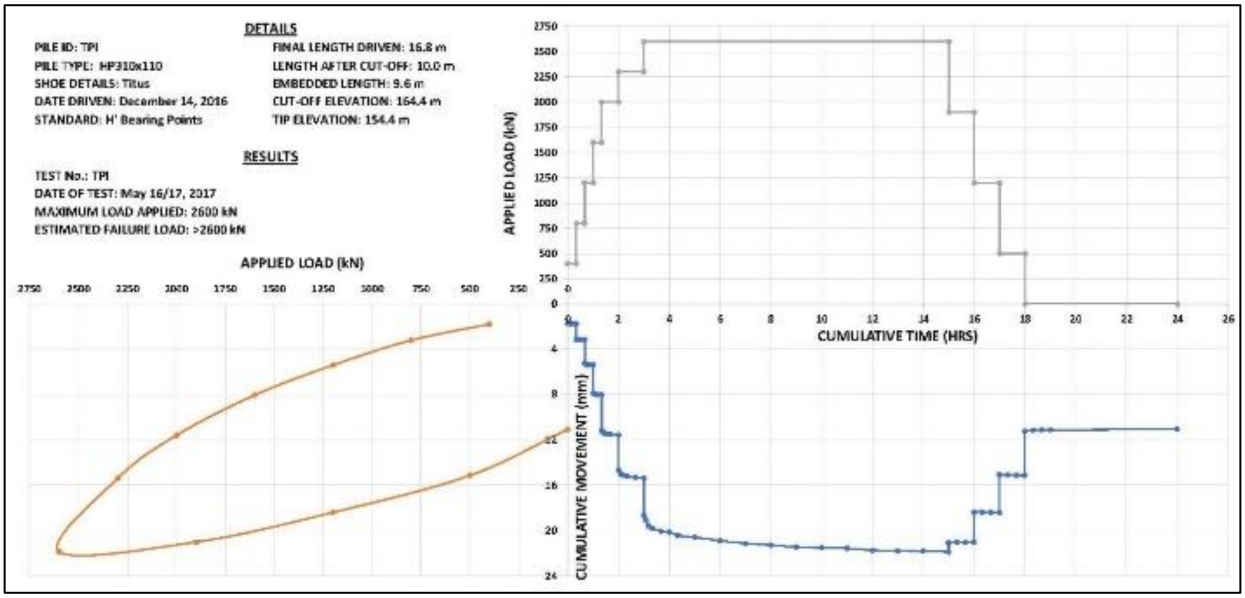

Figure 4a: Static pile load test results - Site #1

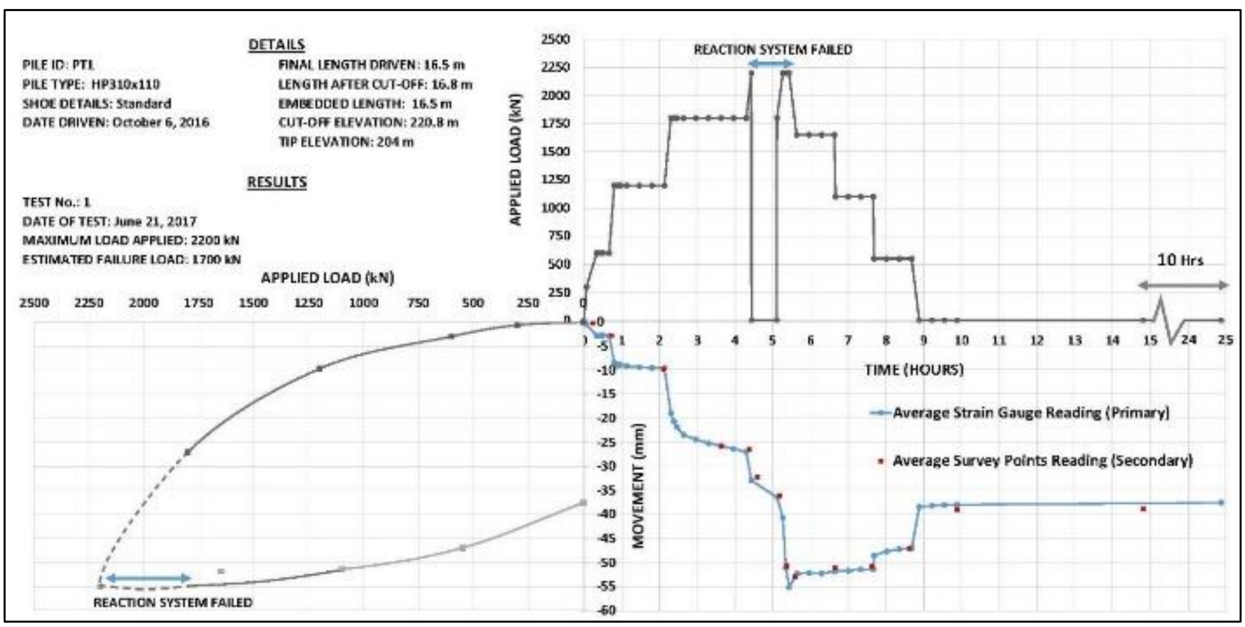

Figure 4b: Static pile load test results - Site #2

It is possible that the artesian conditions and/or silty soils near the tip of the pile (fitted with a driving shoe) may have influenced actual tip resistance during installation, and/or the pile simply may not have encountered or penetrated deep enough into the “100-blow” soils to achieve the anticipated design resistance.

For the pile load test at Site #2, taking into consideration the challenges and load fluctuations associated with the progressive failure of the reaction system, the test pile has been interpreted to have experienced a plunging failure at an ultimate load of approximately 1,700 kN in compression (see Figure 4b). As resistance within the “100-blow” soil was not effectively achieved for the test pile as was assumed in the foundation design, the results of the pile load test cannot be directly compared with the design values. However, notwithstanding the deficiency in the test pile, a 55% increase in ultimate geotechnical resistance occurred in the 258 days between the value measured by PDA testing on initial driving, and that measured in the full-scale static pile load test. The potential gain in geotechnical resistance over time increases by a greater proportion for factored resistances, considering the resistance factor for full-scale pile load testing is 0.6 compared to a factor of 0.5 for dynamic PDA testing.

The strength gain over time was also documented during production piling (for three piles) where penetration resistance increased and up to a 14% increase in ultimate geotechnical resistance was measured with PDA testing over an approximately five-day set-up period. Based on the test data, it is our opinion that the geotechnical resistance of the driven steel H-pile under compression loading at the test site experienced significant strength gain over time, in the 258 days that elapsed between the time of test pile installation to the static pile load test.

8 CONCLUSIONS

For both sites, it is anticipated that the geotechnical resistance of piles driven into similar multi-layer soils in artesian conditions will increase over time, compared to the Hiley and PDA test values measured upon completion of initial driving, or on retap a short time after initial driving.

The pile load test results have been used in the assessment of the remaining piling activities on the Sites, where production piles did not achieve the target geotechnical resistances on initial driving, given that the geotechnical resistance measured during initial driving has been shown to increase with time. This approach has allowed the design team to make informed decisions as to whether additional piles, or piles installed to greater depths, were warranted within specific foundation units/areas.

As a follow up to this study, the measured strength gain values can be compared against existing empirical formulas available in the literature that predict increase in pile resistance with time for piles installed in conventional clay or sand soils without artesian conditions (Yan and Yuen 2010, Svinkin and Skov 2000, Huang 1988, Svinkin, 1996, Skov and Denver 1988). Although the limited data provided in the current study is not sufficient to validate such prediction models, in the future and as more pile test results become available for similar artesian sites, it may become evident that existing prediction formulas can be used or new / modified empirical formulas should be developed to estimate pile set-up in similar multi-layered soils subjected to artesian groundwater conditions.

9 ACKNOWLEDGEMENTS

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10 REFERENCES

Canadian Standards Association (CSA), 2014. *Canadian Highway Bridge Design Code (CHBDC) and Commentary,* CSA Group, Mississauga, Ontario, Canada

Canadian Geotechnical Society (CGS). 2006. *Canadian Foundation Engineering Manual,* Fourth Edition, Canadian Geotechnical Society c/o BiTech Publisher Ltd., Richmond, British Columbia, Canada.

Fellenius, B.H., Riker, R.E., O'Brien, A.J., and Tracy, G.R. 1989. *Dynamic and static testing in a soil exhibiting setup.* American Society of Civil Engineers (ASCE), Journal of Geotechnical Engineering, Vol. 115, No. 7, pp. 984-1001.

Huang, S. 1988. *Application of Dynamic Measurement on Long H-Pile Driven into Soft Ground in Shanghai*, Proceedings 3rd International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, Ontario, Canada. pp. 635-643.

Komurka, V.E., Wagner, A.B., and Edil, T.B. 2003. *Estimating Soil / Pile Set-Up – Final Report*, sponsored by Wisconsin Department of Transportation – Research Coordination Section, Madison, WI, U.S.A.

McVay, M.C., Schmertmann, J., Townsend, F., and Bullock, P. 1999. *Pile freeze, a field and laboratory study.* Final Report, Florida Department of Transportation, Research Center.

Skov, R. and Denver, H. 1988. *Time-Dependence of Bearing Capacity of Piles*, Proc. 3rd International Conference on Application of Stress-Waves to Piles, pp. 1-10.

Svinkin, Mark R. 1996. *Setup and Relaxation in Glacial Sand – Discussion*, Journal of Geotechnical Engineering, Volume 122, No. 4, ASCE, pp. 319-321.

Svinkin, Mark R. and Skov, R. 2000. Set-up Effect of Cohesive Soils in Pile Capacity. Proceedings 6th International Conference on Application of Stress Wave to Piles, Sao Paulo, Brazil, Balkema, pp. 107-111.

Yan, W.M. and Yuen, K.V. 2010. *Prediction of pile set-up in clays and sands*, IOP Conference Series: Materials Science and Engineering.