Instrumentation Monitoring Program to Measure the Magnitude, Distribution and Time Dependency of Drag Load on Abutment Piles: A Case Study

Osama Drbe, Abouzar Sadrekarimi & Hesham EL Naggar,
Western University, London, Ontario, Canada
Tony Sangiuliano
Ministry of Transportation Ontario, Toronto, Ontario, Canada
Robert Moen
GKM Consultants Inc., Sainte Julie, Quebec, Canada
James Ng
Blackbird Constructors 407 General Partnership., Mississauga, Ontario, Canada

ABSTRACT
Negative skin friction or drag load are important considerations in assessing the geotechnical and structural performance of deep foundation units. The presence of drag load increases the load applied on abutment piles and needs to be considered in the foundation design. To develop a reliable and economical bridge foundation design, an accurate determination of the magnitude of drag load and a better understanding of the load transfer mechanism is needed. Comprehensive instrumentation and monitoring program were employed to measure the magnitude, distribution, and time dependency of drag load on driven steel H-piles for a three span integral bridge structure. This paper describes the installation of the different instruments and data acquisition system, including the protection requirements, challenges, and responses of piles' instrumentation to the structural loads.

RÉSUMÉ
Le frottement latéral négatif est un aspect important de l'évaluation de la performance structurale et géotechnique de fondations. La force de friction appliquée sur les pieux du pilier doit être prise en compte dans la conception de toute fondation. L'élaboration et la conception de fondations de pont fiables et économiques requièrent une évaluation précise de la force de friction et une compréhension améliorée du mécanisme de transfert de la charge. Un réseau exhaustif d'instruments et un programme de surveillance ont été utilisés pour mesurer l'amplitude, la distribution, et la réponse temporelle de la force sur des pieux en H en acier devant supporter la structure d'un pont à trois portées. Ce travail décrit l'installation des différents instruments et l'acquisition de données, avec une attention particulière portée au choix des protections des instruments, aux défis affrontés, et à la réponse des instruments aux charges structurales.

1 INTRODUCTION

There are numerous locations across the province of Ontario where bridges are designed and constructed at sites where the subsoils are comprised of compressible native soil deposits. At these sites, foundations for bridges are typically deep foundation units comprised of driven piles or drilled shafts that derive their axial resistance from shaft and/or tip resistance. Embankment loadings from the approaches induce immediate and long term settlements of the native compressible soils. The relative movement between the soil surrounding the deep foundation units and the piles themselves produce additional frictional loads known as drag loads.

The prediction of the magnitude, distribution and time dependency of these drag loads can be very challenging. These predictive models have resulted in conservative predictions resulting in more expensive foundation designs.

The MTO is challenged to determine the most cost effective method of sustaining its infrastructure. At bridges sites with compressible soils, a determination of the drag loads needs to be carried out. This evaluation has significant implications as to whether a bridge is a candidate for rehabilitation or replacement.

In order to improve the understanding of drag loads, the Ministry of Transportation - Ontario (MTO) is investing in research to explore actual magnitudes, distribution and time dependency of drag loads on driven steel H-piles for a three-span integral bridge structure at Nash Rd/Black Creek and Hwy 418 (East Durham Link). The bridge is part of the Hwy 407 East Phase 2 Project that is being administered by the Ministry of Transportation and Infrastructure Ontario. This investigation includes the design and installation of comprehensive instrumentation monitoring program that will enable near real time data to be gathered and analyzed. This information will be used to determine improved models and methods of prediction of drag loads.

This paper describes the instrumentation monitoring program including installation, protection, and installation challenges along with initial response of piles' instrumentation to the structural loading during construction.

2 PROJECT BACKGROUND

The Hwy 407 East Project (Figure 1) is located in Southern Ontario, Canada and includes three highways: Highway 407, Highway 412 and Highway 418. The Highway 407 East Project is the first publicly-owned and publicly-controlled set of tolled highways in Ontario. It is the extension of the existing Highway 407, from its present east end (at about 10 km east of the City of Toronto), to about 50 km east. Phase 1 of the project from Brock Road in Pickering to Harmony Road in Oshawa was opened to the public in June 2016. Phase 2 of the project, in which the instrumentation program is conducted, consists of about 35 km of 4-lane divided highway with 7 full interchanges and 1 partial interchange. The initial portion of Phase 2 from Harmony Rd to Taunton Rd/Hwy 418 is scheduled to be opened in late 2017. The remainder of Phase 2 from Taunton Rd/Hwy 418 to Hwy 35/115 is scheduled to be completed by 2020.

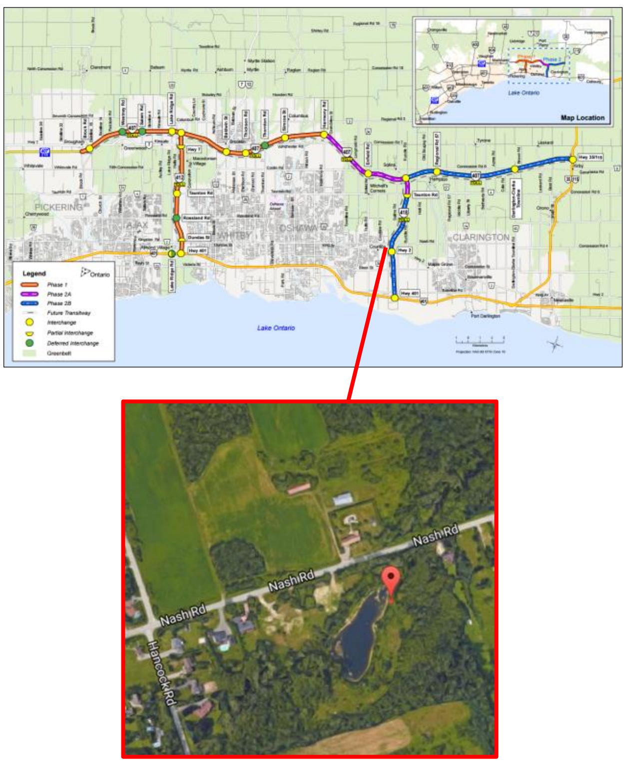


Figure 1. Project Location (Highway 407 Project, n.d.; Google Maps, n.d.)

The instrumentation site is located along the proposed Hwy 418 alignment within a rural area in the municipality of Clarington, Ontario. Two separate bridges are proposed to carry Hwy 418 over the existing Nash Road and Black Creek. The site area generally slopes from the northeast to the southwest and is located approximately 60 m south of Nash Road. A site location plan is provided in Figure 1 above. The exact location of the study area is at the south abutment of northbound lane of E-21 structure (bridge).

3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

3.1 Site Geology

Geologically the project site is situated within the Iroquois Plain region. The Physiography of Southern Ontario (L. J. Chapman and D. F. Putnam), which is a drumlinized till plain consisting of Quaternary glacial sediments overlying Ordovician bedrock. The Oak Ridges Moraine forms the northern boundary of this geophysical region. This moraine consists of predominantly sand and gravel deposits and is a major regional aquifer.

3.2 Subsurface Conditions

Based on the general geotechnical report for the area, the general subsurface stratigraphy at the site as determined by the advancement of conventional boreholes and piezocone tests consists of a surficial stratum silty sand/sandy silt up to 2.3 m in thickness underlain by a clayey silt/silty clay up to 8.1 m in thickness which in turn is underlain by a silty sand to sandy silt till.

The stratum of most interest as it relates to the particular study of drag loads is the clayey silt to silty clay deposit encountered at the south abutment of the northbound lane of Hwy 418. This layer was encountered at a depth of 0.2 m to 2.3 m and extended to a depth of 3.8 m (Elev 136.9 m) to 9.2 m (Elev 130.1 m) below ground surface. The explored thickness of this deposit was between 3.1 m and 8.1 m.

Index tests showed that the water content of the Silty clay to clayey silt layer varies from 13 % to 39 %, liquid and plastic limits ranging from 28 % to 33 % and 14 % to 18 %, respectively. The compression index of the soft clay is about 0.45 based on the geotechnical design report for structure E-21 (exp, 2016).

The results of the two piezocone penetration tests exhibit an undrained shear strength ranging from 18 - 227 kPa, cone tip resistance ranging from 0.4 - 3.2 MPa, and the over consolidation ration is about 2.

Three additional boreholes were drilled before installing the instrumentation at the south abutment area (two adjacent to the abutment, and one under the abutment foot print) in order to collect undisturbed samples for further lab tests.

4 FOUNDATION DESIGN

4.1 Current Design Methods for Predicting Drag Load

The current design methods of pile foundation consider three major aspects; geotechnical capacity, structural capacity and settlement. Fellenius (1984, 1988, and 2004) proposed the “Unified Design Method” for the design of individual piles or a pile group considering negative skin friction. The method is based on lessons learned from several case studies. It includes the evaluation of the structural strength of a pile, pile settlement, and its geotechnical capacity. The first step of design is to locate the neutral plane by satisfying the force equilibrium between the applied forces (dead load and drag load) and resisting forces (positive shaft friction and tip resistance). To illustrate further, two load distribution curves are drawn; one represents applied forces and the other represents the resisting loads. The location where the two curves intersect is considered as the neutral plane. Fellenius (1984) recommended the use of β method over other total stress methods in obtaining the skin friction.

Fellenius (1984, 1988, and 2004) suggested considering the structural capacity of the pile at the neutral plane and the pile cap. In the case of considering the pile capacity at the neutral plane, the dead load plus drag load should be considered only. In the case of considering the pile capacity at pile cap, the dead load plus and live load should be considered only. Regarding settlement, the settlement of the pile is equal to the settlement of soil at the neutral plane plus the elastic deformation due to the dead load and drag load. Fellenius (1984) emphasized that the drag load must not be included in calculating the geotechnical capacity of the pile or pile group. The Canadian Engineering Foundation Manual (CEFM, 2006), and Canadian Highway Bridge Design Code (CAN/CSA-S6-06) adopt the unified design method in calculating drag load.

ASSHTO (2014) considers negative skin friction if the soil settles 10 mm or more relative to the pile shaft. In ASSHTO (2014), the drag load is calculated considering the negative skin friction for all soil layers contributing to drag load using α, λ or β method. The neutral plane method as presented in NCHRP 393 (Briaud and Tucker, 1997) can also be used in determining the drag load. Briaud and Tucker (1997) proposed that the location of the neutral plane is the location where the pile settlement envelope and soil settlement profile intersect. In this method, the pile settlement envelope is constructed by calculating the settlement at multiple points along the pile length assuming several locations for the neutral plane. The settlement at each location is the elastic settlement in the pile below a point plus the settlement of the pile tip under dead load only. Once the neutral plane is determined, the negative skin friction and thus the drag load can be calculated. ASSHTO (2014) recognizes drag load as an additional load that should be resisted by the tip resistance and positive skin friction below the lowest layer that contributes to the drag load. Structurally, the pile foundation should be designed to resist the structural load plus the drag load. Regarding settlement aspect, the structure should be designed to tolerate the settlement resulting from applied loads including drag load.

In order to monitor drag loads in the field and evaluate the methods discussed above, instruments to measure load transferred to the piles from the abutment, soil settlement, change in porewater pressure, and settlement of the abutment shall be used.

4.2 Project Specifics

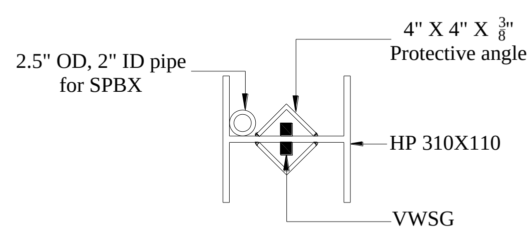
Each structure is a three-span structure with integral abutments and two piers. The superstructure consists of a concrete deck supported by prestressed concrete girders. All structural foundations consist of 310 x 110 steel H piles driven to refusal criteria in the lower very dense silty sand till. At the south abutment of the northbound lane of Hwy 418, there are a total of fourteen (14) vertical piles. The piles are designed for a factored ULS axial geotechnical resistance of 1,000 kN.

Approach embankments up to 9 metres in height will be constructed at the south abutment of the northbound lane after the bridge construction. It is estimated that under the embankment loading the native clayey silt to silty clay soils will experience a primary consolidation settlement of 150 mm. The relative displacement of the native soils and the piles driven to the glacial till deposit will result in drag loads on the piles. A drag load of 300 kN (unfactored ULS) has been included in the foundation design. The adequacy of this value is to be determined through the monitoring of these piles.

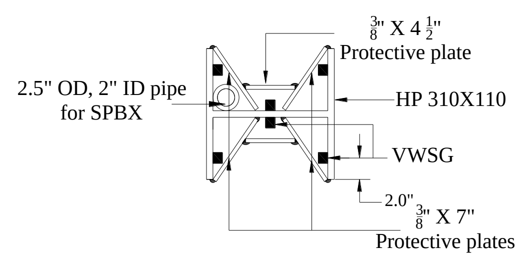
5 INSTRUMENTATION PROGRAM

An instrumentation program is used to monitor the following four (4) parameters: total deformation of the pile, localized strain on the pile, settlement of the surrounding soils (excluding the engineered fill) and porewater pressure at the vicinity of the piles. All instrumentation was installed and monitoring prior to significant construction activities on the bridge abutment, apart from the driving of the piles.

The total deformation of the pile is to be measured with a single anchor extensometer (SPBX) installed within a 2" mm (50 mm) ID steel pipe welded to the pile (Figure 2). One extensometer was installed on each of the three (3) instrumented piles (denoted as Pile 1, Pile 7, and Pile 8).



a) Piles 1, and 8



b) Pile 7

Figure 2. Schematics of pile instrumentation

The extensometer anchors were set in place using expandable hydraulic anchors after the completion of driving. The deformation of the pile is measured using a vibrating wire displacement transducer installed in the head of the extensometer at the top of the pile as shown in Figure 3. The transducer has a range of measurement of 100 mm.

The baseline measurement was established at the time of installation.

Figure 3. Vibrating wire displacement transducer

Localized strain, and thus the forces on the pile are to be measured using individual strain gauges. The strain gauges were installed in one of two cross-sectional arrangements (Figure 2). The arrangement with only two strain gauges per cross-section is most suitable for determining the average strain due to compression or tension within the cross-section, whereas the arrangement with strain gauges in the flange as well as the web is suitable for identifying those strains induced by bending of the pile. A total of 68 strain gauges were installed at different locations along the pile length including two piles (denoted as Pile 1 and Pile 8) with 16 strain gauges each, and one pile (denoted as Pile 7) with 36 strain gauges. 62 strain gauges out of 68 were installed before driving as following 14, 14, and 34 for piles 1, 8, and 7, respectively. It is worth mentioning that 2 gauges at pile 1 were in the cut-off elevation, so they were cut later. After driving, two strain gauges for each pile were installed at the piles' heads (which they will be embedded in the concrete). Figure 4 shows a vibrating wire strain gauge (VWSG) welded on a steel H-pile.

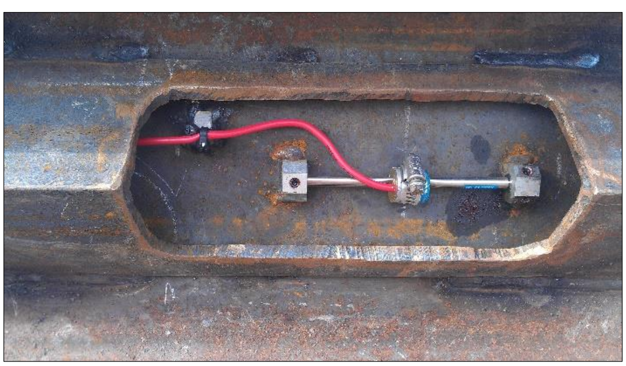


Figure 4. Vibrating wire strain gauge

The strain gauges were installed prior to driving with the cabling run within the protection (see Section 6 for description of protection) and out the top of the pile. The strain gauges had a total range of about 4000 με (micro strain) and were all installed slightly in tension as the pile driving was expected to induce some permanent deformation due to compression.

Settlement of the soils beneath the engineered fill is to be measured using a multi-point borehole extensometer (MPBX). Two (2) extensometers were installed adjacent to Piles 9 and 8 (at South and North sides of the northbound lane of the south abutment). Each extensometer had five (5) anchor points that act independently of each other at the installed depth. Each anchor (Figure 5) was grouted in place using a grout mix that is compatible with the local soil conditions.

Figure 5. Groutable Anchor

A fibreglass rod with a sheathing to isolate it from the surrounding soils is used to connect the anchors to the extensometer's head. A vibrating wire transducer with a measurement range of 200 mm was connected to the rod at the head of the extensometer for each of the five anchors as shown in Figure 6. Displacements between the location of the head and the individual anchors are measured. The extensometers were installed after pile driving was complete. It should be noted that a traditional survey of the completed structure has been undertaken by the MTO's geomatics department to verify the overall settlement of the structure, inclusive of the piles and engineered fill.

Figure 6. Head of multi-point borehole extensometer

Porewater pressure in the soils adjacent to the instrumented piles are to be measured by vibrating wire piezometers (Figure 7). Ten (10) piezometers (VWPs) were installed between four (4) locations with three of the locations consisting of clusters of three (3) instruments distributed vertically, and one reference piezometer located outside of the expected zone of influence of construction. The chosen piezometers had a pressure range of 350 kPa. All piezometers were installed prior to pile driving.

Figure 7. Vibrating Wire piezometer

An automated data acquisition system (ADAS) with remote data management system (RMDS) capabilities was chosen for this project. The ADAS (Figure 8) consists of seven (7) multiplexers (MUX), a datalogger, a solar charging unit, and a battery sufficient to run the system for greater than two (2) week without charging. A grounding rod and surge protection module were installed for lightning surge protection. In addition, a cellular modem is used to transmit the data such that additional processing at an offsite server may be conducted and to allow access to the processed data by all invested parties.

Figure 8. Automated data acquisition system (ADAS)

6 PROTECTION MEASURES

Protection of the instruments and cables on a project such as this is always paramount in order to preserve the integrity of the data throughout the entire study period. The main concerns with regards to this project were the loss of instruments during driving of the pile, the damage to cables due to welding and cutting activities and cable stretching due to the anticipated settlement of native soils.

The act of driving a pile into the ground is an inherently violent process, and thus the fear of losing instruments from frictional forces and/or dynamic forces generated during pile driving needed to be addressed. It was determined that the most appropriate manner to protect the strain gauges and their cables was with either thick angle steel or individual pieces of steel plate, the former being used with the two (2) gauge cross section and the latter with the six (6) gauge cross section. Steel angles and plates were welded onto the pile with alternating 6'' (150 mm) welds with about 10'' - 11'' (250 - 300 mm) spacing in order to reduce the reinforcing action of this additional steel. The protective measures from frictional tear off appeared to have been successful.

As can be expected the welding of this steel in close proximity to the installed instruments and the even more susceptible cables presented a risk of fire or at least melting. To prevent the fire at the expected locations of concern a fibreglass welding divider sheet was cut to size and permanently installed along the instruments and cables within the steel protection areas. In the expected areas of exposure no cables or strain gauges were lost during the welding or cut-offs. Pinch and rubbing risks at the exit point of each pile was addressed using padding and/or rubber sheathing to isolate the cable from the metal edges. There was a cable damage in an area that will be discussed in Section 9, which was damaged during pile cut-off. Figure 9 shows the protection of the VW strain gauges of Piles 1 and 8 from heat and frictional forces

a) Covering the cables, and sensors by a fire resistance fabric

b) Adding extra piece of protecting angle

Figure 9. Protection measures for Piles 1 and 8

It is worth mentioning that no specific protection measures were adopted during the installation of vibrating wire piezometers, single-point extensometer, and multi-point extensometer.

The estimated settlement at this site is expected to be in the range of 150 mm, as such, high levels of strain in the cables, which can lead to jacket failure or in extreme cases cable breakage, had to be addressed. Cables at this site were laid out in a serpentine fashion (Figure 10) with about 200 mm of clean sand sized material separating different layers of cabling. It remains to be seen if there are any failures due to cable strain. This will not be fully determined until the settlement or study period is complete.

Figure 10. Serpentine layout of cable bundles

7 SURVIVEABILITY AND DURABILITY

One of the aspects of selecting instruments is based on their survivability during installation and the need for long-term monitoring (Bica et al. 2013). Therefore, VW gauges were used in this project because of their known long-term stability under static loading. Instruments that depend on liquid fluids for operation were excluded because of their reduced sensitivity as a result of aeration of their liquid-filled tubes over time.

Since the initial construction of the bridge abutment nearly 5 months ago, we have (and will continue) collecting data from the VW piezometers, single-point extensometers, and multi-point extensometers. Some of the VW strain gauges were affected by pile driving during the installation of the first pile (Pile 1). It is worth mentioning that pile 1 was driven over the termination criterion of 7 blows per inch for three consecutive depth increments (10 inches ≈ 0.25 m). Later, the termination criterion was amended to 5 blows per inch for three consecutive depth increments (10 inches ≈ 0.25 m). A summary of the survivability of the instruments is shown in Table 1.

Table 1. Number of Functional Instruments over Time

Time	VWSG (P1)	VWSG (P7)	VWSG (P8)	MPBX/ SPBX	VWP
Initially Installed	14*	34*	14*	5	10
Immediately after Driving	12	34	12	5	10
Prior To Bridge Construction	10	34	12	5	10
Five Months After Installation	10	34	12	5	10
Survivability Rate (%)	71	100	85	100	100

*Number of VW strain gauges installed prior to pile driving

8 PILES' RESPONSE TO CONSTRUCTION

There are three reasons for negative skin friction in driven piles. Initially, negative skin friction develops between the soil and driven piles as a "Locked-in load" or residual load during the driving of a pile. This phenomenon develops in a short time after pile installation. Then, increasing the load on the ground surface due to surcharge loads, site grading, and dewatering cause dissipation of induced porewater pressure and leads to settlements and drag loads. Finally, the subsequent primary and/or secondary settlement (long-term response) also causes negative skin friction and simultaneous drag load.

Vibrating wire strain gauges were relied upon to measure both short, and long term behaviours of the piles. For this reason, several baselines were measured which included measurements prior to and after the driving of the piles as following: 1) when the pile is horizontal, laying on its stands; 2) when the pile is vertical with its toe on the ground; 3) when the pile is vertical, hung in the air by the crane; 4) immediately after driving of the piles.

The attempt to measure residual stresses by comparing the readings before and after installation was failed as it was indicated that drift of strain gauges readings occurred due to driving. For this case, the most suitable baseline was that measured post driving after the driving induced strains had stabilized.

Figure 11 shows the response of vibrating wire strain gauges at depth 3 of Pile 8 due to the piers loads and girders (Figure 12). The negative sign (-ve) in Figure 11 indicates that the gauges are in compression due to the external loads. Figure 13 shows the location of vibrating wire strain gauges at depth 3 of Pile 8.

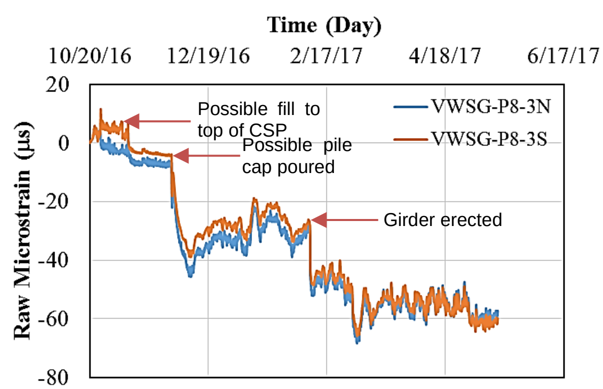


Figure 11. Raw Microstrain vs. Time (mm/dd/year format)

Figure 12. Northbound lane of the south abutment

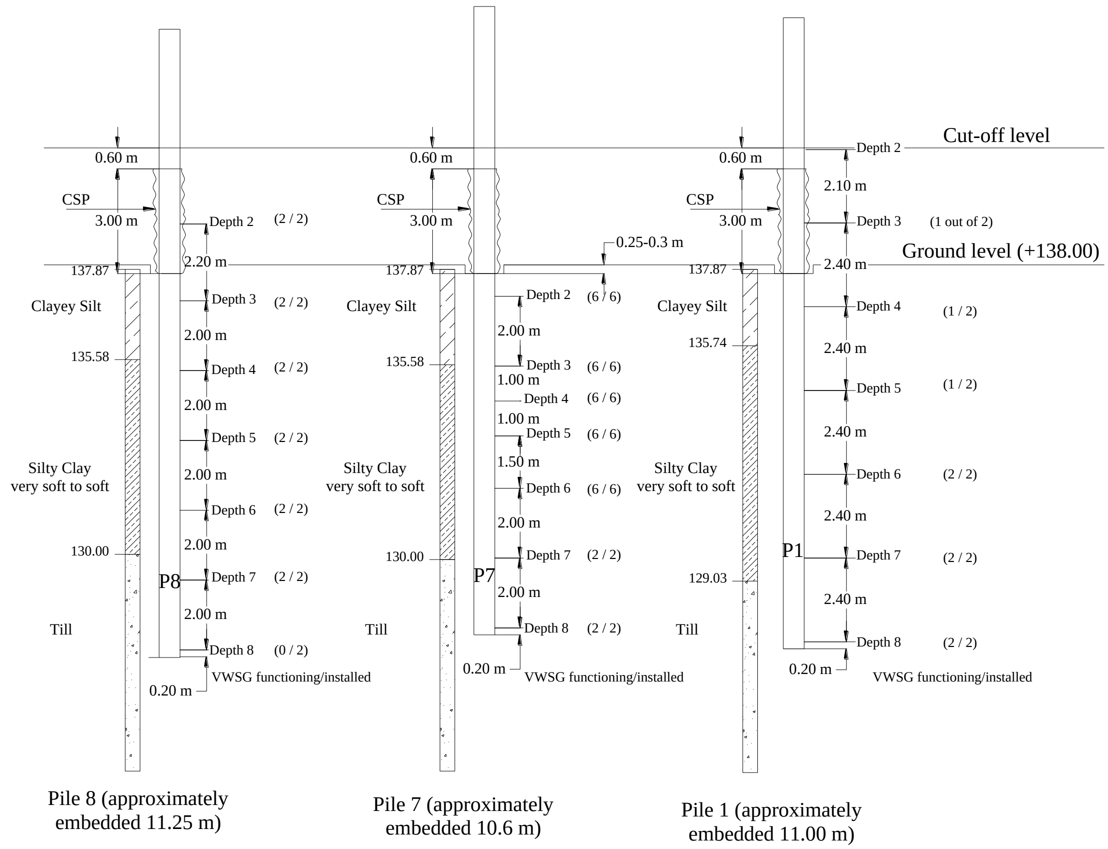


Figure 13. Locations of vibrating wire strain gauges.

9 LESSONS LEARNED

In a project with the dichotomy of the forces induced in driving the methods used in construction of the foundation elements and the delicacy and minute levels of measurement, conflicts in priorities were expected. The following are those lessons learned through the unexpected and conflicting actions of multiple parties on site.

9.1 Damaged Cables

It was originally understood that the placement of sand within the corrugated steel pipes (CSP) surrounding the protruding piles (Figure 14) was to occur prior to the cut-off to the excess pile lengths. Unfortunately, this did not occur in one location, and due to the absence of this or an alternate protective measure, debris and heat from an oxyacetylene torch created a fire which in a short period of time damaged many cables on a single pile. Splicing each of the damaged cables, while time consuming, did allow for the salvaging of all the damaged instruments.

When working with multiple parties, consistent clear communication is required and letting all parties know why certain objectives are important to you is imperative to planning and operating efficiently.

Figure 14. Installation of corrugated steel pipes

9.2 Settlement Concerns

Generally, an MPBX should be installed several days prior to any anticipated ground movement to allow the MPBX to stabilize. After installation, slight movements may occur but generally cease after three (3) days. Initial readings shall be taken several times to ensure a good baseline reading. After installing the south and north MPBXs (on November 4th, 2016), settlement values of 0.4 mm, and 6.7 mm were respectively measured in this project. A day after (on November 5th, 2016) and during some earthwork construction activities, a sharp increase to about 14 mm and 35 mm was observed in the settlement readings of the south and north MPBXs respectively as shown in Figure 15.

The initial interpretation for the possible causes of these higher values of settlement is that since the MPBX anchors were installed in an unstable layer and/or the bentonite grout had not hardened enough, the heads of MPBXs were somewhat displaced by earthwork activities. The north MPBX was installed 2 days after the installation of the south MPBX, which could partly explain the different amounts of settlement measured by these MPBXs.

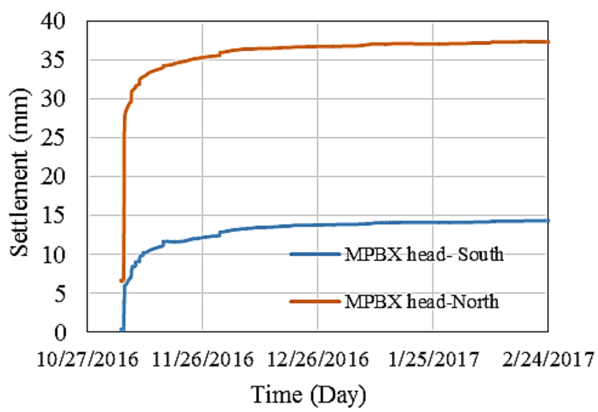


Figure 15. Settlements measured during constriction (time is in mm/dd/year format)

9.3 Effect of Driving Forces on Strain Gauges

The optimum method to monitor piles during and after driving for short and long terms is that using a combination of both electrical resistance strain gauges and vibrating wire strain gauges. The purpose of electrical resistance strain gauges is to monitor the strains and stresses during driving of the piles and hence the residual stresses while the purpose of the vibrating wire strain gauges is to monitor the elastic settlement of the piles and hence the vertical strain and stresses on the piles for a long term. Unfortunately, the electrical resistance strain gauges were not used in this study as they do not normally survive in the long term because they are more sensitive to humidity and other weather-related effects (Bica et al. 2013).

ACKNOWLEDGEMENTS

The authors would like to acknowledge the contribution of Exp Services Inc. (exp) to the project.

REFFERENCES

American Association of State Highway and Transportation Officials (AASHTO). 2014. AASHTO LRFD Bridge Design Specifications, US Customary Units, Seventh Edition, with 2015 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, D.C., 1960 p.

Bica, A. V.D., Prezzi, M., Seo, H., Salgado, E., and Kim, D. 2013. Instrumentation and Axial Load Testing of Displacement Piles. *Geotechnical Engineering.*

Briaud, J.L., and Tucker, L. 1997. Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles, *Transportation Research Board*, Washington, NCHRP Report 393.

Canadian Foundation Engineering Manual (CFEM). 2006. 4th edition. *The Canadian Geotechnical Society*. Barbara Goulet, Calgary.

Canadian Standards Association (CSA) International. 2006. Canadian Highway Bridge Design Code. Toronto: CSA International,

Exp Services Inc. (exp). 2016. Geotechnical Design Report for Structure E-21. Prepared to Blackbird Constructors 407GP.

Fellenius, B. H. 1984. Negative skin friction and settlement of piles. Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, November 28 - 30, 12 p.

Fellenius, B.H. 1988. Unified Design of Piles and Pile Groups. *Transportation Research Board*, Washington, TRB Record 1169, pp. 75-82.

Fellenius, B.H., 2004. Unified Design of Piled Foundations with Emphasis on Settlement Analysis. Current Practice and Future Trends in Deep Foundations GeoInstitute Geo-TRANS Conference, Los Angeles, July 27-30, 2004, Edited by J.A. DiMaggio and M.H. Hussein. *ASCE Geotechnical Special Publication*, GSP 125, pp. 253-275.

Google. (n.d.). Google Maps for Nash Rd/Black Creek, Courtice, On. Retrieved May 15, 2016, from https://www.google.ca/maps/place/Nash+Rd,+Courtice,+ON/@43.9187063,-78.7658792,419m/data=!3m1!1e3!4m5!3m4!1s0x89d503f646424205:0x17add1423df23a99!8m2!3d43.9133658!4d-78.7925801

Highway 407 Project - Ontario's Publicly Owned Toll Highways. (n.d.). Retrieved May 15, 2017, from http://highway407east.com