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Sustained Capacity of Friction Piles in a Clay Deposit Treated by Electro-Osmosis: Observations over Five Decades

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ABSTRACT
The 178 m long, three-span Pic River Bridge near Marathon, Ontario is founded on relatively short friction piles driven into an 18 m deep soft to firm varved silty clay layer underlain by over 70 m of stratified silt and silty fine sand deposits under a maximum 6 m of artesian head. Due to the artesian pressure at depth, the capacity of long friction piles driven into the silt and sand deposits was determined by load testing to be significantly less than that of short friction piles installed within the clay deposit. The original foundation design in 1959 was therefore based on 16.5 m long friction piles installed within the clay deposit. The clay properties were improved by applying electro-osmotic treatment at the two piers and east abutment. The electro-osmotic treatment doubled the ultimate pile capacity from 300 to 600 kN per pile. Subsequent load tests on selected piles conducted from 1961 to 1992 indicated that the increased pile capacities were being sustained.

Rehabilitation of the bridge involving a superstructure replacement was carried out in 2015 and 2016 along with settlement monitoring of the existing foundations. Static pile load tests were conducted on selected piles in 2013 to confirm that the pile capacities have not diminished with time. The results indicate that the pile capacity improvements achieved by the electro-osmotic treatment of the clay have been sustained over a 54-year period. Static cone penetration tests and shear vane tests were also undertaken near the test piles to assess the improvement of clay properties due to the electro-osmotic treatment. Pre-rehabilitation settlement analyses predicted negligible immediate settlement and 10 to 20 mm of long-term settlement in 25 years. The monitoring data collected between 2015 and 2017 indicated generally less than 5 mm of settlement at abutments and piers.

RÉSUMÉ
Le pont de la rivière Pic à trois travées de 178 m de long, près de Marathon, en Ontario, est fondé sur des pieux de friction relativement courts enfoncés dans une couche d'argile molle à ferme de 18 m de profondeur reposant sur plus de 70 m de dépôts de limon stratifié et de sable fin limoneux sous un maximum de 6 m de tête artésienne. En raison de la pression artésienne en profondeur, la capacité des longs pieux à friction enfoncés dans les dépôts de limon et de sable a été déterminée par les tests de charge comme étant nettement inférieure à celle des pieux à friction courts installés dans le dépôt d'argile. La conception originale de la fondation en 1959 était donc basée sur des pieux de friction de 16,5 m de long installés dans le gisement d'argile. Les propriétés de l'argile ont été améliorées en appliquant un traitement électroosmotique aux deux piliers et au pilier est. Le traitement électroosmotique a doublé la capacité ultime de la pile de 300 à 600 kN par pile. Des tests de charge ultérieurs sur des pieux sélectionnés effectués de 1961 à 1992 ont indiqué que l'augmentation des capacités des pieux se maintenait.

La réhabilitation du pont impliquant un remplacement de la superstructure a été réalisée en 2015 et 2016 ainsi que le suivi de la colonisation des fondations existantes. Des tests statiques de charge de pieux ont été effectués sur des pieux sélectionnés en 2013 pour confirmer que les capacités des pieux n'ont pas diminué avec le temps. Les résultats indiquent que les améliorations de la capacité du pieu obtenues par le traitement électroosmotique de l'argile se sont maintenues sur une période de 54 ans. Des tests statiques de pénétration au cône et des tests avec des aubes de cisaillement ont également été effectués près des pieux pour évaluer l'amélioration des propriétés de l'argile due au traitement électroosmotique. Les analyses de peuplement avant la réhabilitation ont prédit un peuplement immédiat négligeable et 10 à 20 mm de peuplement à long terme en 25 ans. Les données de surveillance collectées entre 2015 et 2017 indiquent généralement moins de 5 mm de tassement au niveau des culées et des piles.

1 INTRODUCTION

The Pic River Bridge on Highway 17 near Marathon, Ontario, constructed in 1959, is founded on relatively short friction piles driven into an 18 m deep soft to firm varved silty clay layer which was treated by electro-osmosis to increase the carrying capacity of the piles. Bridge rehabilitation involving a superstructure replacement was completed in 2017. The superstructure replacement involved a 15% increase in pile load at the piers. In order to confirm that the pile capacities have not diminished with time and that the pier piles can carry the additional load, static pile load tests were conducted on selected piles in 2013. The load tests supplemented periodic load testing carried out on the piles since construction to establish a period of 54 years of monitoring pile capacities. Static cone penetration tests and shear vane tests were also undertaken in the vicinity of the test piles to assess the improvement of clay properties due to electro-osmotic treatment.

The results of these load tests over 5 decades are summarized in this paper to demonstrate that the pile capacities have not diminished with time and that the pier piles can accommodate the load increase imposed by superstructure replacement.

2 SITE DESCRIPTION AND GEOLOGY

The Pic River Bridge is located on Highway 17 approximately 8 km east of Marathon, Ontario. The highway crosses the Pic River over a three-span steel truss structure supported on two piers and two abutments. The bridge is about 178 m long and 11. 7 m wide. The approach fill height ranges up to 3.5 m at the west abutment and 2.5 m at the east abutment. A snowmobile trail bridge exists to the north of the highway bridge.

The river channel at the bridge is approximately 67 m wide and 7.5 m deep. The river flows to the south and is relatively fast flowing at this location. Rock fill erosion protection is visible above the river level in the lower parts of the approach embankment and valley slope. A photograph of the site is shown in Figure 1.

The river valley is underlain by approximately 18 m thick deposit of soft to firm varved clay, grading to stratified silt and then to silty fine sand at 70 to 80 m depth. The depth to bedrock is greater than 80 m and was not determined at the site. Artesian water pressure was encountered in the silt strata. The maximum artesian head was 6 m above ground surface at a depth of 80 m as reported in the original 1958 investigation.

3 FOUNDATION DESIGN BACKGROUND

The piers and abutments of the Pic River Bridge are supported on steel H Piles (12BP53/HP310x79) driven *i*nto the clay deposit at the piers and east abutment and into the underlying silt strata at the west abutment.

Figure 1. Looking east from west bank of Pic river (2011)

During the original bridge design in 1959, the planned design load for the piles was set at 350 kN per pile. Initially the piles were driven to lengths ranging from 16.5 to 50.5 m using a 2-ton drop hammer falling 2.5 m. Load tests on these initial piles indicated ultimate pile capacities of 135 to 355 kN which did not meet the planned design load of 350 kN. The load tests also showed that the pile capacity decreased with an increase in pile embedment depth due to artesian pressures at depth. Piles tested up to 400 days after initial driving showed no significant increase in capacity when compared to the capacity measured 5 days after initial driving (Milligan, 1994).

In response to the low pile capacities, and in order to increase pile capacity, Ministry of Transportation Ontario (MTO) carried out electro-osmotic treatment of the foundation clay at selected foundation elements. The purpose of the electro-osmotic treatment was not only to increase the pile capacity but also to reduce the potential for foundation settlement and to avoid redesign of the foundation system.

For an initial test run of the electro-osmotic treatment, an electric arc welder with a maximum capacity of 375 Amperes at 115 Volts was used. The anode of the welder was hooked up to the test piles and the cathode was connected to the head frame connecting the two anchor piles. At the west pier, the energy source was connected to a 50 m long test pile group and the current maintained for 2.5 hours. At the east pier, electro-osmosis was similarly applied for 3 hours to a 20 m long test pile. At the end of this time period of application of electro-osmosis, the anodic test pile capacities increased from 150 kN to 350 kN for an untreated pile at the west pier and from 350 kN to more than 500 kN at the east pier (Geocon 1959). MTO consulted with late Professor Leo

Casagrande on the design of the electro-osmotic treatment.

Based on these favourable load test results of the treated test piles, MTO decided to apply electro-osmotic treatment to the foundation clay at the east abutment, east pier and west pier. The test pile at the west abutment was first tested after 8 days following initial driving and the load carrying characteristics of this pile had markedly improved when retested 54 days after driving. Therefore, the piles at the west abutment did not warrant electro-osmotic treatment. At the final design stage, a pile length of 16.5 m was selected to prevent penetration into the underlying silt deposit which was under artesian pressure. The foundation design resulted in using 95 friction piles at the two piers, and 33 and 22 friction piles at east and west abutments, respectively. The pile spacing ranged from 1.0 to 1.8 m. A pile capacity of 135 kN per pile was adopted for final design, even after improvement of foundation soils by electro-osmotic treatment. The design capacity was likely selected based on application of a safety factor to the measured post-treatment capacity and considerations for limiting settlement of the pile group. Details and description of the pile load tests during original bridge design described in the 1959 report by Geocon.

As part of the original foundation design in 1959, MTO installed a number of additional piles at each pier which were isolated from the load bearing pile group by boxing out access portals in the pile cap to permit load testing during and after completion of electro-osmotic treatment of the foundation clay. A reaction beam was cast into the pile cap above each test pile to enable application of static load on the piles. Subsequent to electro-osmotic treatment, static load testing of selected piles was carried out by MTO in 1961, 1968, 1971, and 1992 to confirm that the load capacity was sustained. The results of the load tests are available in MTO files and summarized in a paper by Milligan (1994).

Rehabilitation of the bridge in 2015/2016 included replacement of the bridge deck and modifications to the abutments and piers. The new deck will generally increase the loads on the piles from an original design load of 135 kN to the following loads:

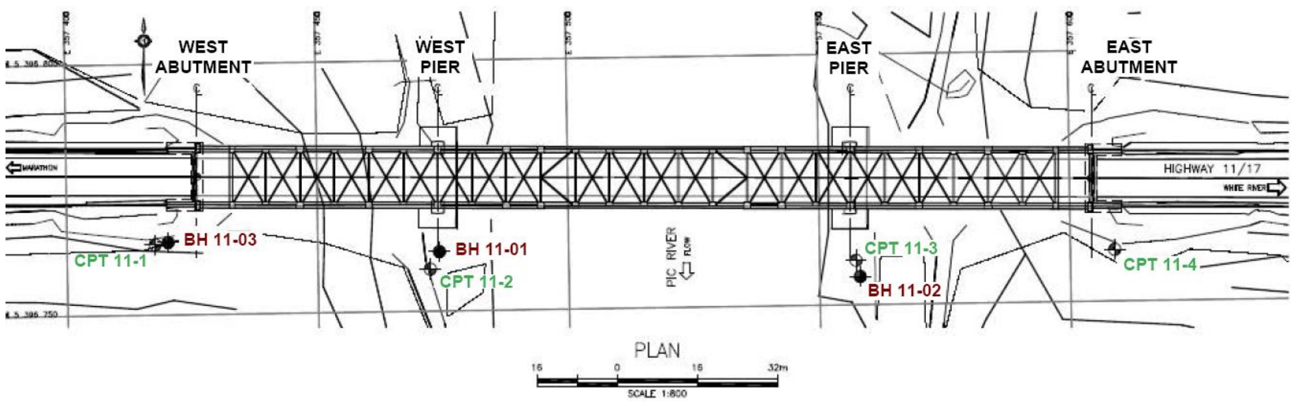

Figure 2. Locations of boreholes and piezocones (2011)

Table 1. Design loads of the replacement bridge

Foundation Element	ULS (kN)	SLS (kN)
West Abutment	215	143
West and East Piers	209	156
East Abutment	143	95

Assuming that the SLS loads are similar to working stress design load in the original design, this implies a load increase of 15% on the piles at the piers and a load decrease of 30% at the east abutment. The new SLS load at the west abutment is essentially same as the original design load. In light of this requirement MTO initiated a program in 2013 to carry out static pile load tests on three selected piles at the piers to confirm that the ultimate pile capacities are being maintained and to assess the current pile capacity and evaluate the load displacement behavior of the test piles some 50 years after construction. An in-situ program of piezocone and vane shear testing was also undertaken to evaluate the condition of the foundation clay that was treated by electro-osmosis.

4 INVESTIGATION PROGRAM

The investigation program consisted of the following components:

- Borehole Drilling Program
- Static Pile Load Tests
- Piezocone and Field Vane Testing

4.1 Borehole Drilling Program

A foundation investigation, carried out in April 2011, consisted of drilling three boreholes to depths of 26.5 to 46.3 m near the west pier, the east pier and the west abutment. Standard Penetration tests and shear vane tests were carried out at selected intervals in each borehole. Undisturbed Shelby tube samples of the foundation clay were collected from the boreholes. Piezocone testing was conducted near each foundation element to complement borehole information. Figure 2 shows the approximate locations of the boreholes and piezocones.

The samples collected from the boreholes were subjected to water content and index tests consisting of gradation and Atterberg Limit tests.

4.2 Static Pile Load Tests

Static Pile Load tests were conducted on the following piles:

- East Pier: Pile Nos. G-5 and E-16
- West Pier: Pile No. E-2

The location of the test piles at each pier is shown on Figure 3.

Each pile was loaded to a maximum load ranging from 700 kN to greater than 900 kN and settlement of each pile was recorded at prescribed intervals under each load increment. A photograph of the load test set up is presented in Figure 4.

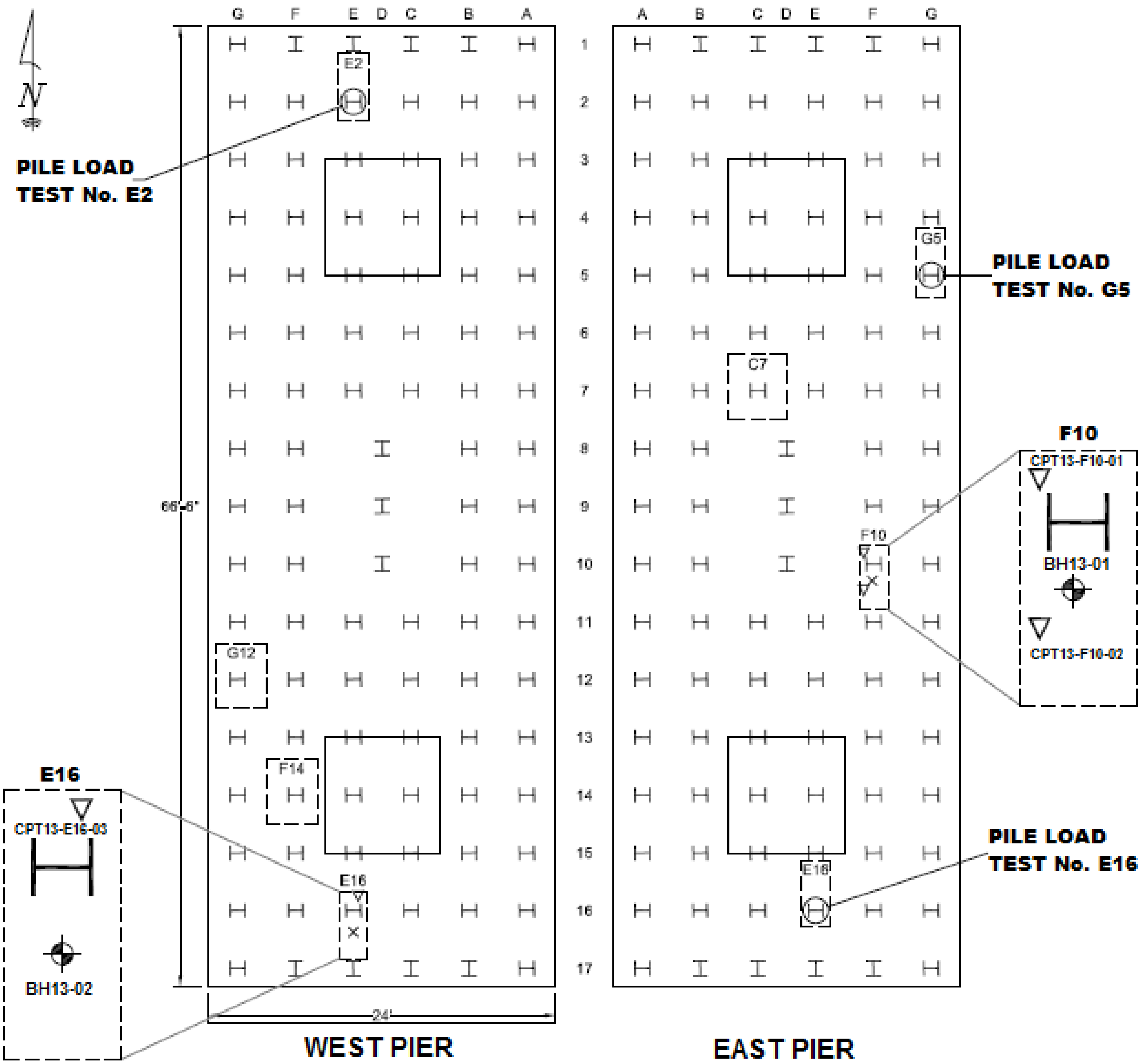

Figure 3. Locations of test piles, boreholes and piezocones (2013)

4.3 Piezocone and Field Vane Testing

In addition to the pile load testing program, static piezocone tests (CPT) were conducted at two locations within the east pier (CPT 13-F, 10-01 and 10-02) and one location within the west pier (CPT 13-E16-03). The CPT test locations are shown on Figure 3.

The piezocones were pushed to a depth of 11.8 m. One of the CPT tests was pushed 0.23 m from the centre of a pile while the second test was pushed 0.56 m away from the pile to compare soil properties within and outside the zone of electro-osmotic treatment. Pore pressure dissipation tests were conducted at selected depths within the foundation clay.

Field vane shear tests were conducted at 0.75 m depth intervals to 12 m depth in pile access portal F10.

Figure 4. Pile load test set-up at Pile E-16, East Pier

5 INVESTIGATION RESULTS

5.1 Stratigraphy

Figure 5 presents a stratigraphic profile at the bridge site. The stratigraphy encountered in the boreholes consisted of 1 to 2 m of rockfill overlying 2 to 4 m of loose sand and silt which was overlying the varved silty clay.

The silty clay deposit, about 13 to 25 m thick, is soft to stiff. The deposit contains 5 to 10 mm thick varves of silt. The clay deposit transitions to a compact silt layer with clay bands below a depth of 18 to 27 m.

The varved clay is of intermediate to high plasticity. The approximately 25 mm thick dark grey silty clay bands exhibited moisture contents of 35 to 65%, a clay content of 70%, and a silt content of about 30%, while the typically 12 mm thick light grey clayey silt bands exhibited moisture contents of 20 to 30%, a clay content of 30% and a silt content of 70%. The undrained shear strength of the clay ranged from about 16 to 40 kPa in the upper 4.5 m of the deposit and increased to 32 to 76 kPa below this level.

The underlying silt layer is loose to dense with moisture content of 20 to 40%. The silt gradation includes 83 to 96% silt sized particles with 4 to 17% clay size fraction.

The piezometer installed in the silt deposit in the borehole indicated a piezometric level of 1.6 m below ground surface to a small artesian load of 1 m above the ground surface. It should be noted that the maximum artesian head was 6 m above ground surface in the silt strata at a depth of 80 m during the 1958 investigation.

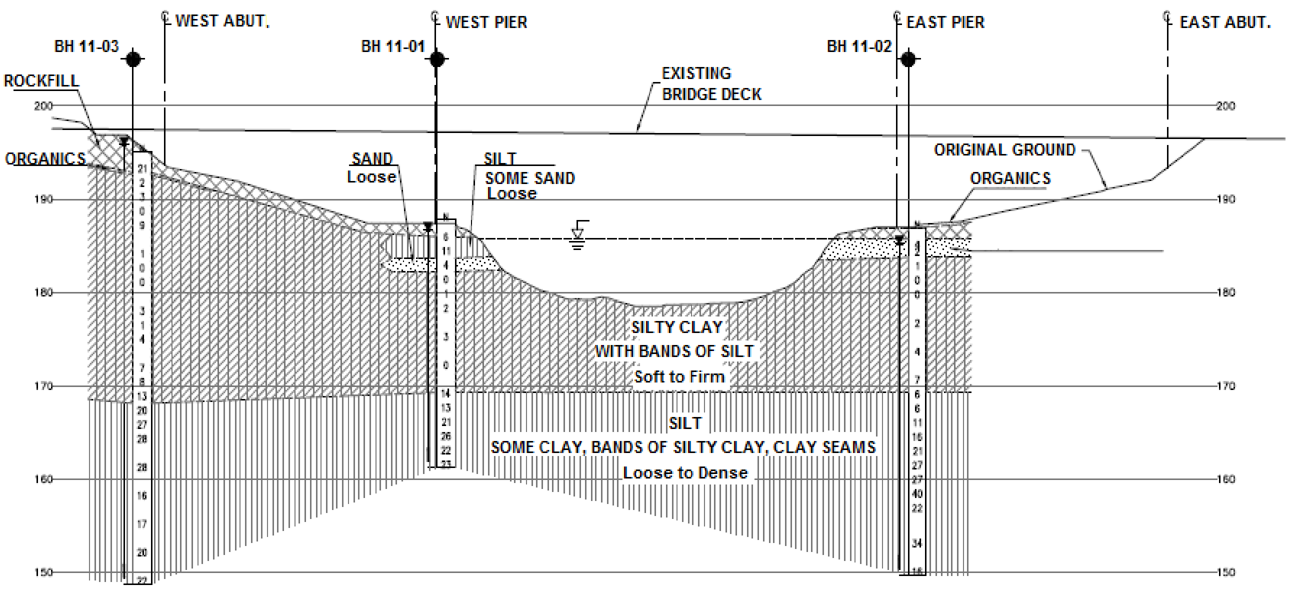

Figure 5. Stratigraphic profile at the Pic River site

5.2 Static Pile Load Tests

The results of the static pile load tests on the three piles are plotted on the load-settlement curves in Figures 6, 7 and 8 along with the historical load test curves. A review of the pile load test data indicated the following:

1) The interpreted ultimate capacity of Piles G-5 and E-16 at the east pier ranges between 550 and 600 kN.

2) The load/deformation behavior of the test piles at the east pier is essentially elastic below 600 kN and the pile settlements are less than 5 mm. The settlement increased more rapidly with each load increment over 600 kN, reaching a maximum of 19 to 20 mm (indicating pile failure) at applied loads of 690 and 750 kN before the load test was discontinued.

3) The load test for Pile E-2 at the west pier indicated that the pile did not reach failure at a loading of 900 kN and the pile settlement at this loading was in the order of 8 mm.

4) Each pile was unloaded and reloaded at one point during the load tests and the resulting load settlement behavior remained essentially elastic.

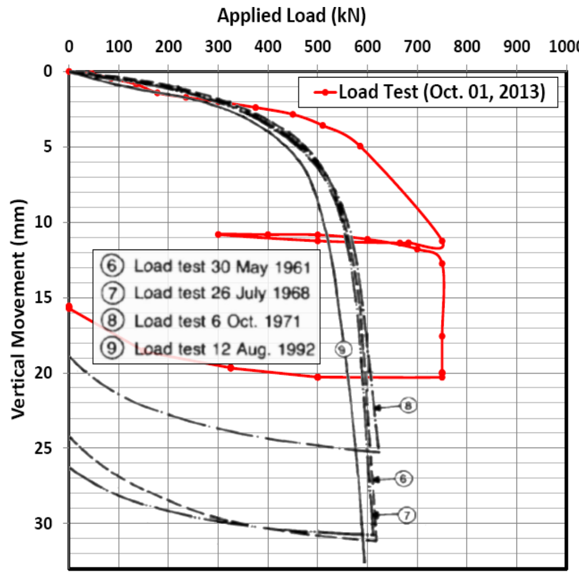

Figure 6. Load deformation curves of East Pier G-5

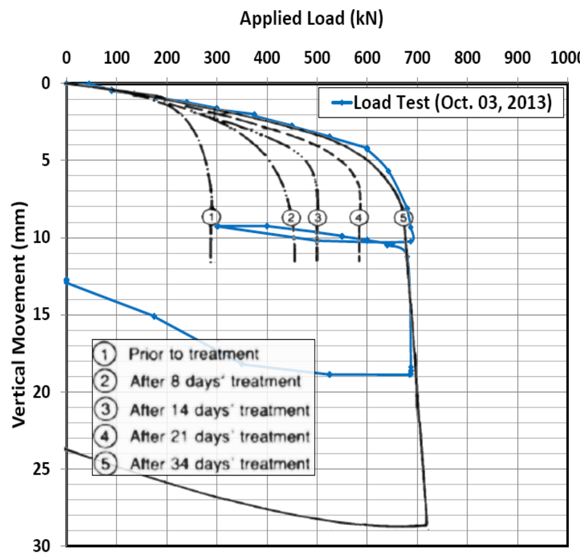

Figure 7. Load deformation curves of East Pier E-16

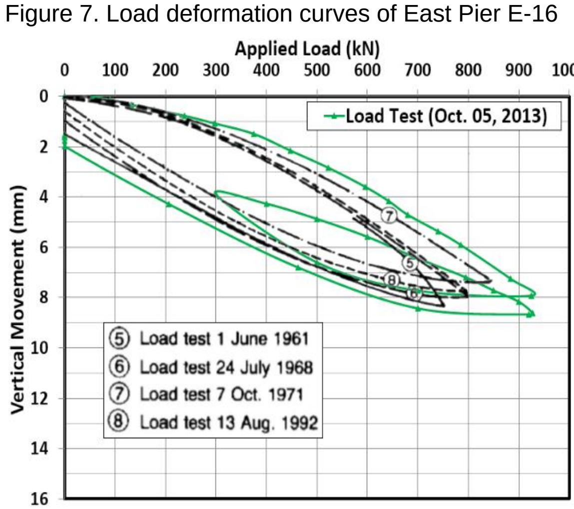

Figure 8. Load deformation curves of West Pier E-2

5.3 Piezometer and Vane Tests

The results of piezocone tests are presented in Figures 9 and 10. The results of field vane tests indicate undrained shear strengths of the treated clay from 48 to 80 kPa at the east pier and 53 to 75 kPa at the west pier.

Figures 9 and 10 present a comparison of undrained shear strength (Su) of the untreated clay (outside pile group) versus the undrained shear strength of the treated clay around the piles (within pile group) of the east and west piers, respectively. The average Su profile from the original 1958 investigation appears very similar to the Su profile of the untreated clay outside the pile group tested in 2011. At both pier locations, Su for the untreated clay ranges from 20 to 58 kPa, while the Su values of the treated clay ranged from 50 to 70 kPa. This data tends to indicate an increase in the clay strength by about 50% after electro-osmotic treatment.

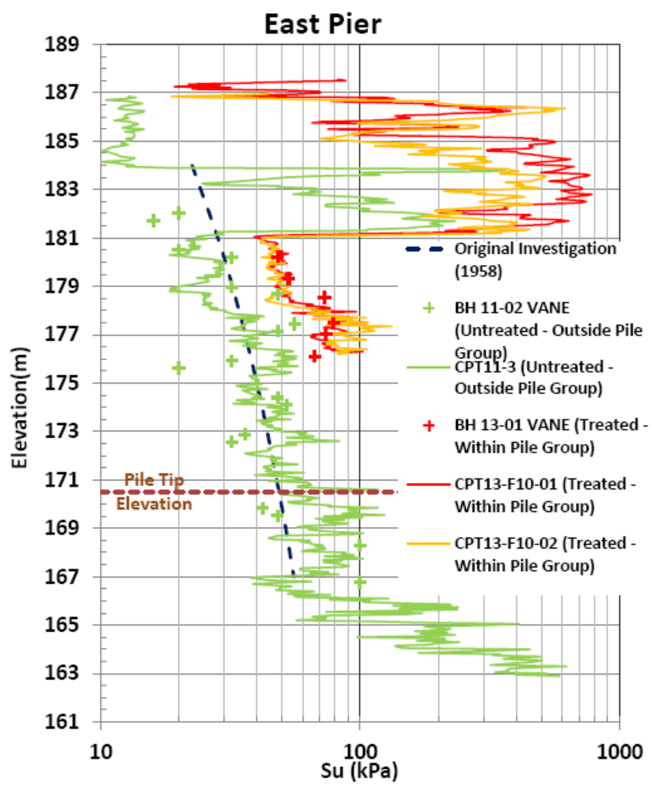

Figure 9. Comparison of undrained shear strengths of untreated clay and treated clay at the East Pier

6 GEOTECHNICAL ASSESSMENT

6.1 Pile Load Tests

The load settlement curves from the current load tests in Figures 6, 7 and 8 are plotted along with the load settlement curves previously conducted in 1961, 1968, 1971, and 1992. The results indicate that the ultimate pile capacities of all three piles have been maintained and appear to have increased for piles G-5 and E-2.

Based on the results of the pile load tests, an ultimate capacity of 600 kN per pile may be assumed for a single pile within the east and west pier pile groups. The factored geotechnical resistance at Ultimate Limit State (ULS) per pile is therefore 360 kN (600 kN x resistance factor of 0.6). Based on the load-settlement curves, the immediate (elastic) settlement of a single pile subjected the increased design load of 156 kN per pile will be 2 mm or less.

The load test results indicate that a single pile at the piers can accommodate an increase in design load from 135 kN to 156 kN per pile.

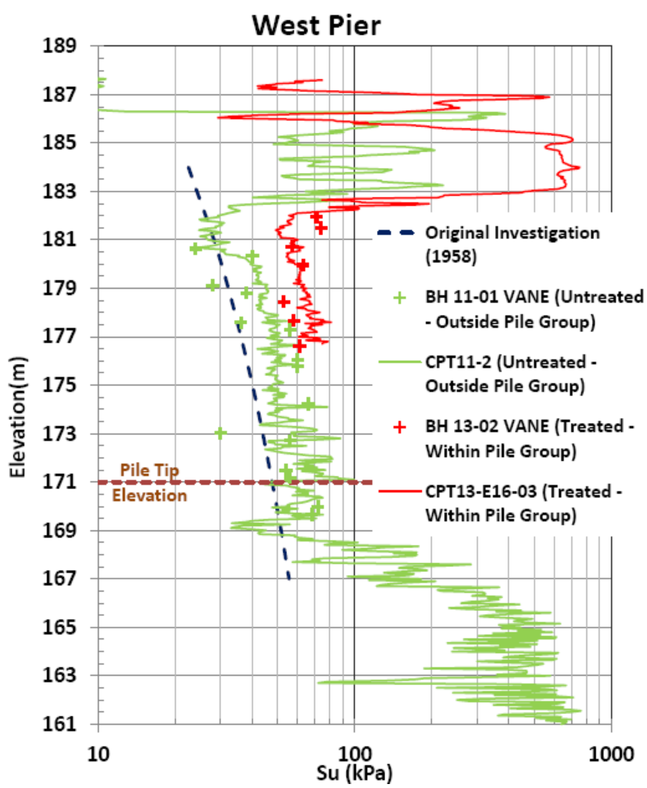

Figure 10. Comparison of undrained shear strengths of untreated clay and treated clay at the West Pier

The assessment of carrying capacities was followed by an estimation of settlement of the pile groups under the new bridge deck load resulting from the rehabilitation of the bridge. The total settlement will include both immediate (elastic) settlement of the pile groups as well as post-construction consolidation and creep settlement in the clay. Geotechnical programs including GROUP (developed by Ensoft) and Settle^{3D} (developed by Rocscience) were used to carry out the settlement analysis. The settlements tabulated below are estimated for a design service load of 175 kN per pile:

The above settlement estimates are within the settlement tolerance that can be accommodated by the rehabilitated bridge.

Table 2. Estimated settlements of the pile groups

Foundation Element	Estimated Settlement (mm)			
	Immediate (elastic)	Post-construction		
		25-year	50-year	75-year
West Abutment	1	10	15	19
West Pier	2	15	20	25
East Pier	3	20	25	30
East Abutment	2	16	23	28

6.2 Monitoring During Bridge Rehabilitation

The performance of the foundations of the existing bridge were monitored during and subsequent to the rehabilitation of the bridge between approximately May 2015 and July 2017. This included settlement monitoring of the abutment and pier caps by surveying of settlement monitoring points. In addition, the pore pressure response of the foundation clay near the foundation elements was monitored using vibrating wire piezometers.

Settlement monitoring points were installed near the centre and at the corners of each concrete pile cap. Accuracy of the settlement survey was maintained at ±2 mm. The results of the settlement monitoring collected over the 2-year period indicated less than 5 mm of settlement at each foundation element. This observation agreed well with the estimated foundation settlements presented in Table 2.

A total of eight vibrating wire piezometers were installed in the clay deposit with two near each foundation element. The piezometer tips were located at depths ranging from 7 to 14 m below ground surface. Each pair of piezometers were typically spaced at 3 m in depth between the two. The measured pore pressures in the clay during the bridge rehabilitation generally fluctuated with the hydrostatic pressure in the ground.

7 CONCLUSIONS

The existing Pic River Bridge is supported on pile groups consisting of steel H-piles driven into soft to stiff varved clay. During initial construction, electro-osmotic treatment was applied to the foundation clay to improve pile capacity and settlement characteristics of the foundation clay.

Proposed rehabilitation of the bridge would include deck replacement and an increased load on the existing pier piles by up to 15%. Confirmation was required that the existing pile foundations could accommodate the increased loading at the piers and had maintained the improved capacity realized by electro-osmotic treatment.

The results of a foundation investigation program and static load tests on selected test piles in 2013 led to the following conclusions:

1. The pile capacity improvements realized by electro-osmotic treatment have been maintained since original construction in 1959. Test pile G-5 at the east pier showed about 25% higher capacity than the previous load test results. The mechanism behind the

post electro-osmosis capacity increase is not well understood but it is postulated that this may be attributed to shaft shear setup associated with aging effect of driven piles in clay. However, the capacity increase was not observed for the test pile E-16 within the same pile group.

2. The piezocone and vane shear strength testing indicate that the strength of the clay within the entire block of the pile group has been significantly improved by the electro-osmotic treatment.

3. The pile load tests indicate that the existing pier piles will be able to carry the 15% load increase.

4. Based on the results of the pile load tests and computation of pile group settlements, the settlement estimates due to increased pile loads are within the settlement tolerance of the rehabilitated bridge.

5. Settlement and pore pressure response were monitored during the rehabilitation of the bridge. Minimal settlements of the foundation elements were noted under the increased load from the new bridge deck. No discernable increase in pore pressure was noted in the foundation clay during the bridge rehabilitation work.

It should be noted that the success of electro-osmotic treatment in improving driven pile capacity in varved clay deposit at the Pic River Bridge is a very site-specific case study and may not be replicated at other sites with differing soil conditions. An extensive field and laboratory testing program will be necessary to prove the applicability of the electro-osmotic treatment for a particular site. In addition, post-treatment monitoring should be implemented to confirm retention of the improvements in soil properties and foundation capacity in the long term.

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The pile load tests, CPT testing and installation of instruments were supervised by Mark Farrant of Thurber Engineering. Monitoring of geotechnical instruments were supervised by Luke Gilarski, formerly with Thurber Engineering.

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