Reuse of Existing Steel Pile Foundations – Greenock Creek Bridge, Walkerton, Ontario, Canada

Tony Sangiuliano – Ministry of Transportation of Ontario, Canada

David Staseff – Ministry of Transportation of Ontario, Canada

PK Chatterji – Thurber Engineering Ltd, Oakville, Ontario, Canada

Keli Shi – Thurber Engineering Ltd, Oakville, Ontario, Canada

Rodolfo de Castro Jr. – Thurber Engineering Ltd, Oakville, Ontario, Canada

Abstract

Following a preliminary assessment of bridge replacement/rehabilitation options for the existing Greenock Creek Bridge located near Walkerton in Southwestern Ontario, Canada, the removal of the existing superstructure and the reuse of the existing foundations was selected as the preferred option. The preferred alternative was based on a condition survey of the deck and superstructure, and traffic mobility, economic, sustainability and environmental advantages. The decision is demonstrative of the Ministry of Transportation's (MTO) commitment to assess new technologies and applications that support its initiatives and operational needs and specifically to encourage the reuse of existing foundations.

The existing structure, built in 1971, is a single span structure that carries Hwy 9 over Greenock Creek. The existing abutments are founded on H-Pile foundations that, according to the original construction contract drawings, are approximately 8 m in length and installed at 1:4 and 1:8 batters.

The subsurface conditions at the site consist of clayey silt to silt underlain by silty sand with gravel. The groundwater table is at or near the ground surface. The relative density of the subsoils is loose to dense in the surficial 3 to 5 m and becoming very dense below these depths.

This paper describes the process of evaluation of the capacity, integrity and durability of the existing steel HP310x79 H-piles. A comprehensive programme was planned, organized and implemented that included exploratory boreholes to verify the subsurface conditions, retrieval of soil samples to determine the corrosivity of the soil and groundwater around the piles in order to assess the degree of corrosivity and geophysical testing including parallel seismic, and borehole magnetometer testing to estimate the embedment lengths of the existing steel piles.

The results of this investigation were used to assess if the existing steel pile foundations can carry the new superstructure loads for another 75 years.

Introduction

Following a preliminary assessment of bridge replacement/rehabilitation options for the existing single span Greenock Creek Bridge on Highway 9 near Walkerton in Southwestern Ontario, the removal and replacement of the existing superstructure and the reuse of the existing pile foundations was selected as the preferred option. The preferred alternative was based on a condition survey of the superstructure and traffic mobility, cost effectiveness, sustainability and environmental advantages. The new Ministry of Transportation Guideline for Reuse of Existing Foundations has been applied for this project revolutionizing the approach to assessing the integrity, durability and carrying capacity of the existing piles.

According to these guidelines, major items that require assessment for reusing bridge foundations include:

1. Understanding of the current condition and integrity of the foundation
2. Determining the load-carrying capacity of the foundation
3. Estimating the remaining service life of the foundation, and
4. Evaluating the reused foundation using the guidelines and codes developed for new foundations.

A thorough investigation of the subsurface and foundation is needed to complete the assessment.

The existing bridge is a 17.7 m long single span structure with the abutments supported on eleven HP310x79 piles battered at 1H:4V to 1H:8V. The piles were designed to be 26 feet (7.9 m) long and were driven into very dense silty sand with gravel till. Design load per pile was reported to be 70 tons (630 kN). No pile driving records were available for the existing piles to indicate the as-built length of the piles and corresponding pile capacities.

The bridge was constructed in 1971 and the bridge is planned for rehabilitation or replacement. An assessment was required to determine whether the existing piles had suffered section loss due to corrosion resulting from road salting operation and to assess the load carrying capacity of the existing piles over an additional service life of 75 years. As discussed in the paper, it was not practical nor cost-effective to expose the abutment piles. Hence an indirect approach was chosen for evaluation of the condition of the existing piles, which included:

- Drilling a number of boreholes near the existing foundations to collect soil and groundwater samples. Select boreholes were also used to conduct geophysical testing in an attempt to estimate the vertical length of the piles.
- Conducting resistivity, chloride and pH testing on the samples to assess degree of corrosivity of the soils around the piles.
- Correlating the degree of corrosivity measured in the soil and groundwater to potential loss of pile section reported in the literature.
- Determining the structural and geotechnical load carrying capacities of the existing piles by analyses.
- Estimating the remaining service life of the existing piles

The results of this testing programme and assessment are presented in this paper, and indicate that the existing piles may be reused to support the replacement superstructure.

Site Description and Geology

Greenock Creek Bridge is located on Highway 9 in the Town of Walkerton in Southwestern Ontario. The existing bridge, constructed in 1971, is a single span structure with a span of 17.7 m. The approach embankments to the bridge are 3 to 4 m high.

At the bridge site, the Greenock Creek valley is about 18 m wide and the creek is about 1.5 m deep. The existing piles are submerged below the creek water level.

The geology of the site indicates that the site is characterized by presence of modern alluvial deposit underlain by glaciofluvial and glaciolacustrine deposits containing silt and clay underlain by sand and gravel and glacial till. Limestone and dolostone bedrock underlies the overburden soils at depth.

A photograph of the bridge site is presented in Figure 1.

Figure 1 – Photograph of Greenock Creek Bridge

Site Investigation and Testing

The in situ and laboratory investigation phase consisted of the following components:

- Drilling three sampled boreholes at each abutment for a total of six boreholes to define stratigraphy and to collect soil and groundwater samples for testing of corrosivity parameters.
- Testing of corrosivity parameters (resistivity, chloride and pH) on a number of soil and groundwater samples to determine degree of corrosivity of the soils and groundwater surrounding the piles.
- A set of downhole geophysical tests to estimate the length of the installed piles. Geophysical testing is required since no pile driving records were available for the

existing piles and the length of the installed piles were required to estimate their capacity.

The depth of the boreholes drilled ranged from 10.4 to 15.5 m. Four of the boreholes were drilled approximately 1 m offset from the existing piles to facilitate geophysical testing. A 50 mm diameter PVC pipe was installed and grouted in four boreholes close to the piles for conducting geophysical tests. Approximate borehole locations and existing pile locations are presented in Figure 2.

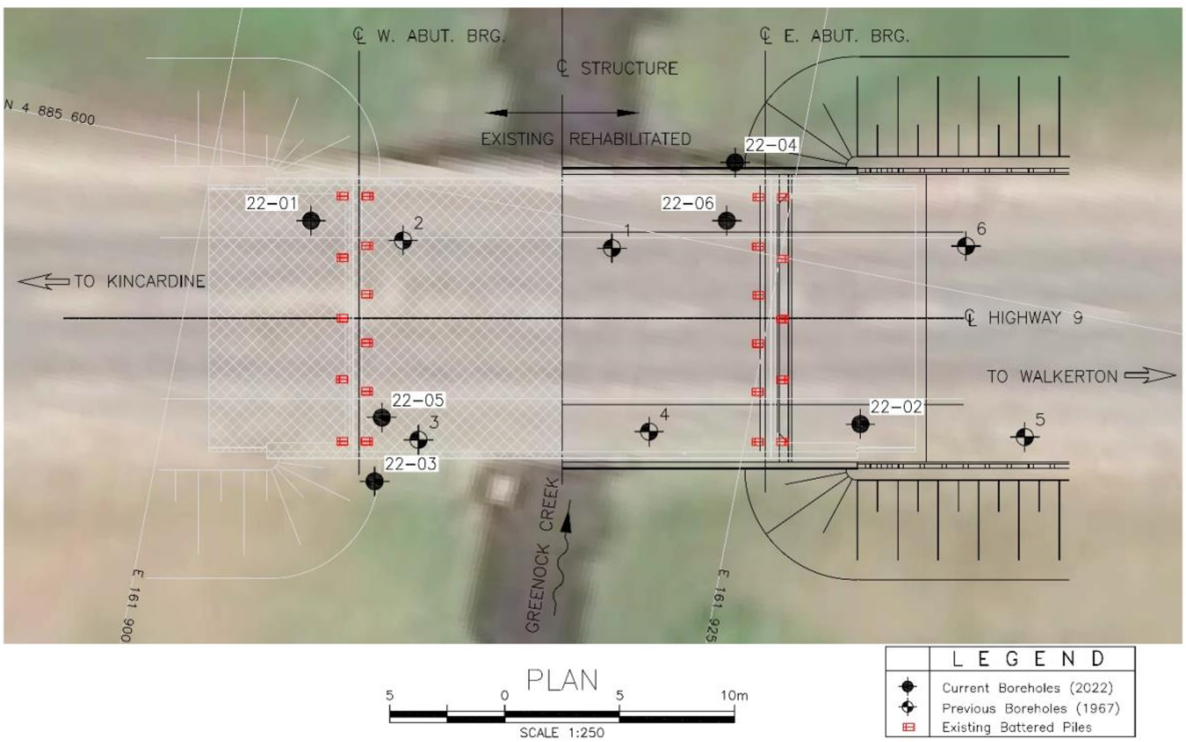

Figure 2 – Borehole Location Plan

Selected samples collected from the drilling programme were subjected to moisture content, gradation and Atterberg Limit tests. In addition, 38 soil samples and groundwater and creek water samples were subjected to corrosivity parameter testing including resistivity, electrical conductivity, chloride, sulphate, pH, redox potential and sodium absorption ratio (SAR). Samples taken at close intervals from the top of the existing piles to a depth of 8 m were subjected to corrosivity testing.

The downhole geophysical tests conducted at the abutments consisted of the following tests to estimate the pile lengths:

- Parallel Seismic Test as per ASTM D8381/D8381M-21
- Borehole Magnetometer Test as per ASTM D6726-01
- Electromagnetic Test as per ASTM D6726-01
- Natural Gamma Test as per ASTM D6274-18

A photograph of the geophysical test set up is shown in Figure 3.

Figure 3 – Geophysical Testing in progress

Soil Stratigraphy

Figure 4 presents a soil profile at the bridge site. The subsurface stratigraphy consisted of asphalt pavement underlain by 3 to 4 m of sand fill approach embankment. The fill is underlain by about 1 m of silty clay to clayey silt which overlies a 3 to 4 m thick layer of compact to very dense sand and gravel. These units are underlain by a very dense layer of sandy silt glacial till. The existing piles were driven into this dense till layer. The groundwater was noted to be at 2.5 to 3.5 m depth coincident with the creek water level.

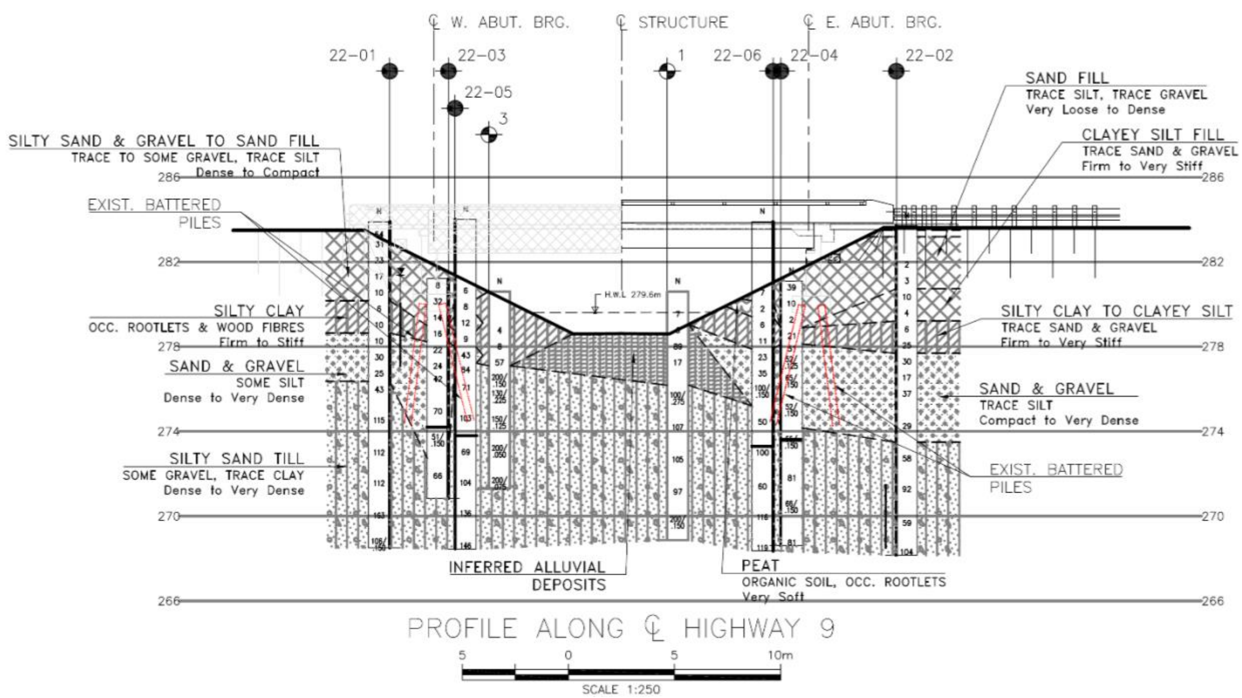

Figure 4 – Soil Stratigraphy

Laboratory Test Results

The results of the geotechnical laboratory results at each soil stratum are summarized in Table 1.

Table 1. Geotechnical Laboratory Results

<table>
<tr><th rowspan="2">Soil Stratum</th><th rowspan="2">Moisture Content (%)</th><th colspan="4">Grain Size Distribution (%)</th><th colspan="3">Atterberg Limits (%)</th></tr>
<tr><th>Gravel</th><th>Sand</th><th>Silt</th><th>Clay</th><th>PL</th><th>LL</th><th>PI</th></tr>
<tr><td>Sand & Gravel Fill / Sand Fill</td><td>5 to 21</td><td>6 to 53</td><td>39 to 50</td><td>40</td><td>4</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>Silty Clay to Clayey Silt</td><td>12 to 53</td><td>0</td><td>2 to 3</td><td>77 to 82</td><td>16 to 21</td><td>21 to 23</td><td>14 to 16</td><td>6 to 7</td></tr>
<tr><td>Sand & Gravel</td><td>3 to 25</td><td>27 to 81</td><td>15 to 63</td><td colspan="2">3 to 21</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>Silty Sand Till</td><td>6 to 11</td><td>9 to 20</td><td>39 to 43</td><td>37 to 43</td><td>6 to 8</td><td>-</td><td>-</td><td>-</td></tr>
</table>

The results of the corrosivity tests are summarized on Figure 5, which also schematically shows the relative location of the existing piles. The results were compared to the following guideline, Tables 2a, 2b and 2c, proposed by European Committee for Standardization EN1993-5, Eurocode 3 – Design of Steel Structures- Part 5, Piling.

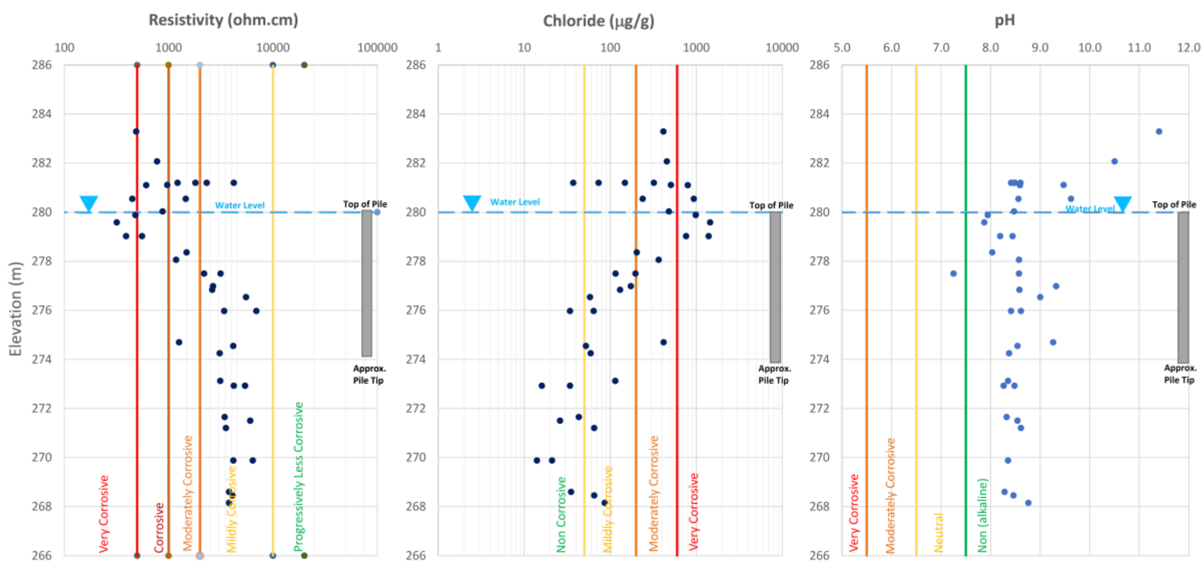

Figure 5 – Corrosivity Test Result Summary

Table 2a. Effect of Chlorides on Corrosion Potential

Inorganic Chloride Concentration (ug/g)	Degree of Corrosivity
Above 600	Very Corrosive
200 to 600	Moderately Corrosive
50 to 200	Mildly Corrosive
Below 50	Non-Corrosive

Table 2b. Effect of Resistivity on Corrosion Potential

Soil Resistivity (Ohm.com)	Degree of Corrosivity
Below 500	Very Corrosive
500 to 1,000	Corrosive
1,000 to 1,500	Mildly Corrosive
1,500 to 10,000	Progressively Less Corrosive

Table 2c. Effect of pH on Corrosion Potential

pH Value	Degree of Corrosivity
Below 5.5	Very Corrosive
5.5 to 6.5	Moderately Corrosive
6.5 to 7.5	Neutral
Above 7.5	None (Alkaline)

The following conclusion may be drawn from Figure 5:

- The upper 3 to 4 m thick approach embankment fills are generally corrosive to steel piles mainly due to low resistivity and high chloride content. This is largely the result of road salting operations over the years.

- The soils surrounding the upper 1 m of the piles are corrosive in nature due to low resistivity and high chloride content.
- Below 1 m from the top of the piles, the resistivity and chloride content indicate a mildly corrosive soil environment.
- The groundwater tested indicates a moderately corrosive environment.
- pH content of the soils/groundwater tested indicates a non-corrosive (alkaline) and neutral environment.
- The top of the piles is at the water level. The piles are in submerged condition.

Geophysical Testing

Since no pile driving records were available for this site, four different down-hole geophysical testing methodologies were employed in an attempt to estimate the vertical length of the battered piles nearest to Boreholes 22-03, 22-04, 22-05 and 22-06. The following geophysical tests were conducted:

- Borehole Magnetometer – involves measuring changes in the magnetic field down the borehole. Magnetic field readings were time sampled and referenced to cable-timed cable markers at 0.5 m intervals.
- Parallel Seismic – relies on the accurate measurement of the transit time for a generated wave to travel from a shot-point on the surface to a receiver (geophone) at sequential depths within the borehole. The velocities at which the waves propagate are then determined from arrival times of the impulse signals.

- Electromagnetic Measurements (EM) – provide a measurement of the electrical conductivity/resistivity of the surrounding borehole using inductive electromagnetic technique. It also provides an estimate of the magnetic susceptibility.
- Natural Gamma Ray Logging – this method was used in conjunction with the EM method to distinguish metallic conductors (high conductivity, low gamma) from conductive clay layers in soils.

An example of downhole geophysical test results are presented in Figure 6 at the east abutment of the bridge. The figure shows the estimated base of pile depths based on the results of the Seismic First Arrival Time, First Derivative of Vertical Magnetic Field, Horizontal Magnetic Field, Apparent Conductivity, Natural Gamma and Magnetic Susceptibility. The magnetic field values (both horizontal and the gradient of the vertical components) for the boreholes show two anomalous zones, one extending from the surface to about 3 m depth, and another centred about 7 m depth. The lower of these two anomalies likely represent the approximate bases of the piles. The gradient of the vertical component showed much clearer peaks than the horizontal component, suggesting that the vertical derivative may be able to delineate the pile extent with greater precision.

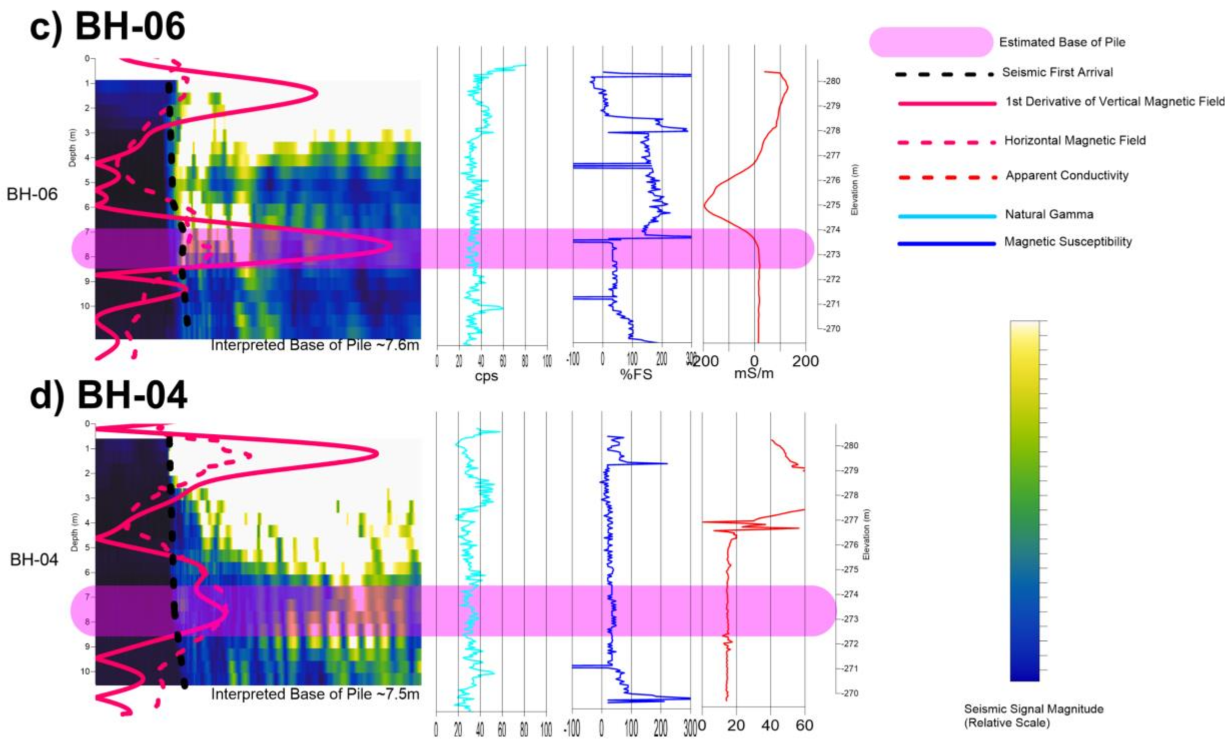

Figure 6 – Geophysical Test Results Indicating Estimated Pile Depth at East Abutment

The estimated pile lengths based on the geophysical testing is listed below:

West Abutment estimated battered pile length: 6.3 to 6.5 m

East Abutment estimated battered pile length: 7.1 to 7.2 m

The design pile length was 7.9 m. Out of the four geophysical methods attempted, the Parallel Seismic and Borehole Magnetometer methods appear to give the clearest estimation of pile depth for this specific application. The parallel seismic test responds to vibration induced into the pile while the borehole magnetometer test measures perturbation in the earth's magnetic field due to the presence of ferrous metal (steel pile). The natural gamma and electromagnetic conductivity tests are influenced by variations of clay content, moisture content and metal. The primary reason why natural gamma and electrical conductivity methods did not respond as well as the other two methods is suspected to be the borehole offset of approximately 1 m from the nearest pile. The borehole conductivity tool measures the bulk conductivity of the material within a distance of 10 to 100 cm with the highest sensitivity within 30 cm. Additionally the contribution of the steel pile to the bulk conductivity can be masked by the variations in ground conductivity.

Capacity Assessment of Existing Piles

Pile Corrosion Assessment

The corrosivity test results reported in Figure 5 indicated that the upper 1 m of the piles are located in a corrosive soil and groundwater environment. The remaining pile is embedded in mildly corrosive environment. Corrosion of steel piles is more likely to occur when they are exposed to corrosive soil and groundwater as well as the presence of moisture and oxygen. Deeper soils have no access to oxygen, resulting in low corrosion potential.

The consensus between CALTRANS, NACE, Eurocode and British Standard BS8002: 2015 indicate corrosion rates are low to negligible when the piles are submerged. The piles at the Greenock Bridge site are submerged and hence the corrosion potential may be described as low to negligible.

However, as indicated above, the corrosivity test results indicate that the upper 1 m of the existing piles appear to be embedded in corrosive soils. Accordingly, a section loss should be applied to the existing piles for assessment of the available pile capacity.

The following pile section losses summarized in Table 2 are suggested in various guidelines for potentially corrosive soil and groundwater environment:

Table 2: Pile Section Loss in Various Guidelines

Reference	Section Loss per Side of Pile
CALTRANS	0.025 mm/year
Ottawa LRT Specifications	0.015 mm/year
Eurocode and BS8002-2015	0.015 mm/year

In light of the fact that only the upper 1 m of the pile may be in corrosive environment and that the Greenock Creek Bridge piles are submerged, a corrosion rate of 0.015 mm/year was selected for estimating section loss of the existing piles over the past and future services lives of the rehabilitated structure.

Pile Capacity Assessment

The bridge designer estimated that the proposed rehabilitation consisting of superstructure replacement will result in factored axial reaction of 630 kN per pile at ULS and 480 kN per pile at SLS occurring in the front battered piles. The pile capacity assessment consisted of comparing the demand on the piles with the structural and geotechnical capacity of the piles.

Factored Structural Resistance

The designer also estimated the following factored structural resistance (Table 3) of the HP310x79 piles assuming steel strength of 230 MPa and applying the resistance factors recommended in Canadian Highway Bridge Design Code:

Table 3: Pile Factored Structural Resistances

Level of Corrosion	**Pile Factored Structural Resistance (HP310x79)**
No pile corrosion or section loss	1215 kN/pile
35% section loss due to pile corrosion ~65% of pile area	790 kN/pile
65% section loss due to pile corrosion ~35% of pile area	425 kN/pile

Using a rate of section loss of 0.015 mm/year/face, and a total assumed past and future service life of 130 years for the piles, the section loss is estimated to be 0.015 mm/year x 130 years x 2 faces or 3.9 mm of loss due to corrosion. This amounts to 35% corrosion loss on an 11 mm thick pile section and the available corresponding factored pile structural resistance as listed above is 790 kN per pile. This factored pile structural resistance is higher than the load demand on the existing piles of 630 kN ULS and 480 kN SLS due to deck replacement work. Accordingly, the remaining factored structural resistance of the existing piles even after 35% section loss is adequate to carry the future bridge loads.

Geotechnical Capacity

An assessment of the geotechnical capacity of the existing HP310x79 piles, 6.3 to 7.2 m long as estimated from the geophysical testing, was also carried out. The ultimate geotechnical resistances of the piles in resisting axial compressive load were estimated by adding the ultimate shaft resistance and the ultimate tip resistance as per the Canadian Highway Bridge Design Code. A shaft resistance coefficient (β) ranging from 0.6 to 0.8 was used for the compact to dense sand and gravel, and 1.3 to 1.4 for the underlying very dense silty sand till. A method proposed by O'Neil and Reese (1999) was employed to estimate the beta values in the very dense cohesionless till. The unit shaft friction resistances adopted in the calculation were capped at the limiting shaft friction values recommended by the American Petroleum Institute (API) to account for the potential effect of shaft fatigue for piles driven into sands and silts. A toe resistance factor (Nt) of 200 for the very dense till was used to estimate the end bearing resistance of the piles. The estimated unit toe resistance in the very dense till was comparable to the observed toe resistance values obtained from a High-Strain Dynamic Testing using a Pile Driving Analyzer (PDA) on H-piles driven into similar cohesionless till. The ultimate shaft capacity of the existing piles was estimated to be about 400 kN while the ultimate end bearing was estimated to be 1600 kN.

The assessment indicated that the geotechnical capacity of these short piles driven to refusal in the till is 800 kN at factored ULS and 650 to 700 kN at factored SLS. These geotechnical resistances are also higher than the load demand on the existing piles resulting from the deck replacement work.

Based on the above assessment it was concluded that the existing pile foundations and the abutments may be reused for the next life cycle of 75 years with superstructure replacement proposed by the bridge designer.

Discussion

While degree of corrosion of piles is best assessed by exposing the piles for visual examination, this was not practical nor cost-effective at the Greenock Bridge site. Accordingly, an indirect approach of measuring the corrosivity parameters in the soils and groundwater surrounding the existing piles and correlating these parameters to existing guidelines for assessment of corrosion of piles was adopted. The corrosivity data indicated that the upper 1 m of the 6 to 7 m long piles is likely surrounded by corrosive environment. The corrosive environment is largely the result of road salting operations.

Based on available guidelines for pile section loss in corrosive soils, a section loss of 0.015 mm/year per side of pile was selected. The factored structural resistance of each existing pile assuming the above section loss was estimated to be 790 kN which is higher than the maximum axial load demand of 630 kN at ULS on the existing piles due to proposed superstructure replacement. The factored geotechnical resistances of each pile were estimated to be 800 kN at ULS and 650 to 700 kN at SLS and these resistances are higher than the maximum axial load at ULS of 630 kN per pile. Based on this assessment, the Ministry of Transportation of Ontario (MTO) and the designers concluded that the existing abutment pile foundations may be reused for the next life cycle of 75 years with proposed rehabilitation consisting of superstructure replacement. This study also indicated that downhole geophysical test methods consisting of Parallel Seismic Testing and Borehole Magnetometer Testing were deemed the most reliable based on published guidance for the geophysical test used in estimating the length of the existing piles where no pile driving records are available.

The reuse of foundations at the Greenock Creek Bridge is expected to result in considerable savings in both construction cost and schedule. The cost savings will be derived from design efficiencies and construction activities related to the construction of new foundations including but not limited to elimination of new piles, wingwall and abutment removal, wingwall and abutment formwork and concreting, and a reduction in shoring and traffic protection. It is estimated that for staged construction, the construction time savings is expected to be 10 to 13 weeks and associated cost saving will be in the order of $500,000.

Conclusions

This paper presents a methodology for assessment of impact of corrosion of the capacity of the steel piles. This methodology allows an assessment of whether the existing foundations can be reused and whether the service life of the existing foundations can be extended.

The reuse of bridge foundations is a viable option that can present a significant cost savings in bridge replacement and rehabilitation efforts for aging infrastructure. The potential time savings associated with reuse of foundation can, in turn, reduce mobility impacts, increase the economic viability and sustainability of a project, and reduce environmental impact. The MTO has developed a guideline for reuse of foundations and has developed a decision-making model based on balanced risk management. This guideline has been applied to the rehabilitation of Greenock Creek Bridge near Walkerton, Ontario. The results of this study indicate that the existing pile foundations may be reused at this site for rehabilitation of the bridge.

This was a pioneering study of its kind by MTO to assess reuse of foundations for a bridge rehabilitation project. It is anticipated that additional bridge sites where reuse of foundations is practical and cost effective will be undertaken to provide additional confidence in assessment of foundation reuse. MTO is also working on pile load testing and extraction of old steel piles at several sites where a bridge has been demolished to assess corrosion loss and remaining carrying capacity of old piles. The results of these investigations will be reported when completed. These projects are demonstrative of MTO's commitment to assess technologies and applications that support its initiative and operational needs and specifically to encourage the reuse of existing foundations.

Acknowledgements

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