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GEOTECHNICAL INVESTIGATION REPORT

Repair/Upgrade-Replacement of Hamlet Swing and Fixed Bridges, Hamlet, Ontario

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REPORT

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Parsons Inc. (Parsons) to provide geotechnical foundation engineering services in support of the design of the repair/upgrade or replacement of the swing and fixed portions of the Hamlet Bridge located at Canning Road, over the Trent Severn Waterway in Hamlet, Ontario as shown on Figure 1. Golder has also been retained to provide condition survey and concrete testing services, under separate cover.

This report provides the results of the geotechnical investigation and should be read in conjunction with the *"Important Information and Limitations of This Report"* (Appendix A). The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

Golder previously conducted a geotechnical investigation in 2011 for the fixed bridge, and specifically for the east abutment. The following documents were utilized in preparation of this report.

- Golder Associates Ltd. "Geotechnical Investigation, Hamlet Bridge (Fixed Span) over Trent Severn Waterway, Parks Canada Agency, Hamlet, Ontario", Report No. 11-1111-0118, dated March 7, 2012.
- Golder Associates Ltd. "Detailed Bridge Condition Survey, Hamlet Bridge, Trent Severn Waterway, Hamlet, Ontario", Report No. 11-1111-0118, dated March 9, 2012.
- Delcan Corporation "Hamlet Bridge (Bridge 57), Comprehensive Detailed Inspection and Structure Evaluation Report (PCA Project No. 2011-4650-20027340), dated March 2012.
- Original bridge drawings from 1915 and selected rehabilitation drawings from 1969.
- Drawing No. B-1 entitled "Hamlet Bridge (Bridge 57 & 58) Swing and Fixed Bridges, Proposed General Arrangement I" dated December 15, 2017.

2.0 SITE AND PROJECT DESCRIPTION

The existing bridge structure consists of a swing bridge on the west side and a fixed bridge on the east side. The 31 m long fixed span was originally built in 1905 for use at another location and was moved to its current location in 1915. The 60 m long equal arm swing bridge was constructed in 1922. The bridge is approximately 5.5 m wide and carries a single lane of traffic. The fixed bridge is supported by two through-trusses (Pratt trusses) and the swing bridge is supported by two through-trusses (Warren trusses) with a nail laminated timber deck and wearing surface. The bridge has four foundation elements: east abutment and east pier for the fixed bridge, and west abutment and west pivot pier for the swing bridge. The east pier supports both the swing bridge and the fixed bridge. The resting piers consist of a wooden cribbing structure topped with concrete blocks/cast concrete adjacent to the pivot pier.

Based on existing reports, the west abutment (of the swing bridge) was reconstructed sometime between 1985 and 1990. Further, the foundations of the fixed bridge have moved (i.e. settled and/or tilted) relative to the swing bridge based on reports of having to shorten the wooden bridge deck over time to accommodate this movement. Based on anecdotal information, voids/gaps have developed immediately behind the concrete abutment wall in recent years.

According to the information summarized in the Delcan report (2012), the east and west abutments are supported on spread footings within the overburden. The founding conditions of the two piers is not known with certainty.

While the original drawings indicate that the piers consist of concrete founded directly on the bedrock, it appears as though the original piers may be founded on timber cribbing which may have been "jacketed" in concrete at a later date over all or a portion of the distance from the top of the pier cap to the bedrock surface. A borehole drilled in the 1960's through the east pier indicates that the east pier consists of concrete over a thick sand layer over the bedrock, which contradicts the other available information about the footing. Therefore, it is not clear if the concrete in the piers is in intimate contact with the bedrock or to what extent.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on November 21 to 25, 2016, during which time six boreholes (16-1A/1B to 16-5) were advanced at the locations shown on the Borehole and Test Pit Location Plan, Figure 2. Boreholes 16-1, 16-4, and 16-5, were advanced from the road surface, and Boreholes 16-2 and 16-3 were advanced from a floating platform.

The field work for the previous Golder investigation at the site was carried out on September 28 and October 22, 2011, during which time three boreholes (BH1, BH2, and BH3), one test pit (TP1), and one horizontal concrete core hole (CH1) were advanced at the locations shown on the Borehole and Test Pit Location Plan, Figure 2.

The boreholes were drilled using a track-mounted (2016) or truck-mounted (2011) drill rig supplied and operated by a drilling specialist, under our supervision. Standard Penetration Testing (SPT) and sampling were carried out at regular intervals of depth in the boreholes using conventional 35 mm internal diameter split spoon sampling equipment advanced using an automatic hammer in accordance with ASTM D1586. Rock coring using NQ or HQ-sized rock coring equipment was carried out in Boreholes BH2, 16-1, 16-2 and 16-3. In addition, in situ vane shear tests were carried out in the relatively soft cohesive soils encountered in the boreholes. The results of the in situ tests (i.e., SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in the text below are uncorrected.

The shallow groundwater conditions were noted in the open boreholes/test pit during drilling/test pitting. A 64 mm diameter pipe for vertical shearwave profile (VSP) testing was installed in Borehole 16-1A. A 50 mm diameter monitoring well was installed in an adjacent unsampled Borehole16-1B to allow for further monitoring of groundwater levels. A 19 mm diameter piezometer was installed in Borehole BH2 to allow for further monitoring of the shallow groundwater levels. The remaining boreholes were backfilled in accordance with the current environmental regulations upon completion of drilling.

Test Pit TP1 was hand excavated by Golder staff at a location immediately south of the southeast wing wall. Horizontal Core Hole CH1 was advanced on the west face of the east abutment wall using a coring machine supplied and operated by a coring specialist, working under our supervision.

All soil samples obtained during this investigation were brought to our Whitby laboratory for further examination, natural water content testing and soil classification testing. The field work for these investigations was monitored by a member of our engineering staff who also determined the approximate borehole locations in the field, logged the boreholes and cared for the recovered samples.

The locations of the boreholes were referenced to the existing bridge features and the approximate ground surface elevations for Boreholes BH1 to BH3, 16-1, 16-4 and 16-5 were inferred from the topographic survey provided by Parsons in November, 2016. The surface water elevation at Boreholes 16-2 and 16-3 was also provided by Parsons. It is understood that the provided ground and water surface elevations are referenced to geodetic datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The bridge site is located in the area between physiographic regions known as the Georgian Bay Fringe and The Number 11 Strip, according to *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The general area is located at the southern fringe of the Canadian Shield and underlain by Precambrian rocks which can be classified as biotite, gneiss, schist and granite. The glaciated rock surface on land is quite undulating and features numerous ridges, knobs and erratic outcrops, which is consistent with the results of the investigation. Soil cover in the Georgian Bay Fringe is generally shallow or non-existent. Soil cover within the Number 11 Strip is typically characterized by stream deposited sand, silt and clay deposits within the hollows of the underlying rock.

4.2 Subsurface Conditions

The existing subgrade soils and shallow groundwater conditions encountered in the boreholes and test pits, as well as the results of the field and laboratory testing, are shown in detail on the Record of Borehole sheets. The 2016 boreholes are contained in Appendix B, the 2011 Boreholes are contained in Appendix C and the laboratory figures are contained in in Appendix D. Method of Soil Classification and Symbols and Terms Used on Records of Boreholes and Test Pits (2016 borehole logs), List of Abbreviations and Symbols (2011 borehole logs) and Lithological and Geotechnical Rock Description Terminology (rock logs) are provided in the respective appendices to assist with the interpretation of the borehole logs. It should be noted that the boundaries between the soil strata have been inferred from drilling observations and non-continuous sampling. They generally represent a transition from one soil type to another and should not be inferred to represent an exact plane of geological change. Further, conditions will vary between and beyond the boreholes.

The following is a summarized account of the subsurface conditions encountered in the boreholes drilled at the site, followed by more detailed descriptions of the existing fill and native soil strata, and shallow groundwater conditions.

The subsurface soil conditions generally consisted of shallow fill overlying very soft to soft silty clay underlain by very loose to dense sandy silt to silty sand underlain by granite/gneiss bedrock. Boreholes in the river channel typically encountered a thin layer of organic matter over the bedrock surface.

4.2.1 Pavement and Fill Materials

Pavement structure was encountered at the ground surface in Boreholes BH1, BH2, 16-1, 16-4, and 16-5. The pavement structure consisted of 20 mm to 85 mm of asphalt overlying about 100 mm 410 mm of granular base.

Fill materials were encountered in test pit TP1 and in all boreholes except Borehole BH3. In Boreholes BH1, BH2, 16-1, 16-4, and 16-5, the fill materials consist of non-cohesive sand and gravel, sand, silty sand, sandy silt, and organic silt. The fill materials extended to depths ranging from 2.1 m to 4.0 m below the existing ground surface. Boreholes 16-4 and 16-5 and Test Pit TP1 were terminated within the non-cohesive fill at depths of 0.9 m to 1.5 m below existing ground surface. Test Pit TP1 was terminated on a large piece of rock fill. The SPT ‘N’-values measured within the fill materials range from 0 blows (weight of hammer) to 5 blows per 0.3 m of penetration, indicating a very loose to loose state of compactness. The in-situ water content measured on samples of the fill range from about 2 percent to 44 percent in the cohesive fill. Grain size distribution curves for samples of the silty sand, sandy silt and organic silt fill are presented on Figures 3, 4, 5 and 13 in Appendix D. Based on the grain

size distribution, the sandy silt fill and organic silt fill are considered to be of highly frost susceptibility due to the high silt content of these materials.

Fill consisting of lake debris, shells, and leaves was encountered overlying the bedrock in Boreholes 16-2 and 16-3. The fill was 0.5 m and 0.2 m thick in Boreholes 16-2 and 16-3, respectively.

4.2.2 Concrete

Concrete was encountered in Boreholes BH2 and BH3. In Borehole BH2, the concrete extended from 2.4 m to 3.4 m below ground surface, and was likely constructed as a component of the existing abutment footing. In Borehole BH3, the concrete extended from ground surface to about 3.5 m (i.e. the abutment wall height at this location) below ground surface as shown in Photos 10 to 12 on Figures 13E to 13F in Appendix C. Two concrete core samples obtained from Borehole BH3 were tested for compressive strength as shown on Figure 9 in Appendix D. The measured compressive strengths were 18.2 MPa and 14.2 MPa.

Concrete was encountered in Core Hole CH1 carried out on the west face of the abutment wall as shown in the Photos 7, 13 to 15, on Figures 13D, 13G and 13H in Appendix C. Core Hole CH1 was located on the existing linear crack (i.e. construction joint). As noted in the photos, the linear construction joint is generally horizontal and runs through the entire corehole length. Based on the water stains observed from the construction joint surfaces, the concrete appears to be separating along most of the construction joint likely due to erosion along the joint. Some vertical cracks were observed in the retrieved cores. The length of the Core Hole CH1 is approximately 1.7 m (i.e. inclined backwall). The concrete wall is about 1.2 m wide at the ground surface.

4.2.3 Clayey Silt to Silty Clay

A deposit of brown to grey clayey silt to silty clay containing rootlets and organics was encountered in Boreholes BH1, BH2, and BH3 below the fill materials or concrete abutment and extended to depths ranging from 4.0 m to 5.3 m below the existing ground surface. Borehole BH3 was terminated in the silty clay deposit. The SPT 'N'-values measured within the clayey silt to silty clay range from 2 blows to 5 blows per 0.3 m of penetration. In-situ vane testing carried out within the clayey silt to silty clay gave undrained shear strength ranging from 20 kPa to 38 kPa, with calculated sensitivities ranging from approximately 3.5 to 4.0 with one outlier of 1.4. The overall results suggest a very soft to firm consistency. The natural water contents of the clayey silt to silty clay samples ranged from 23 percent to 47 percent.

Unconfined compression testing (ASTM D 2166-06) was carried out on a clayey silt sample recovered from the Borehole BH3. The measured unit weight of the clayey silt sample is 19.6 kN/m3. The undrained shear strength inferred from the compression test is 47 kPa, indicating a firm consistency.

The results of grain size distribution of samples of clayey silt are presented on Figure 6 in Appendix D. Based on the grain size distribution, the clayey silt samples are considered to be of high frost susceptibility.

The results of Atterberg limit tests performed on three samples of clayey silt are presented on Figure 8 in Appendix D and also summarized on the Record of Borehole sheets. The result of these laboratory tests on the samples of clayey silt indicated the liquid limits of 30, 35 and 41 percent, plastic limits of 21, 21 and 20 percent, and plasticity indices ranging from 9, 14 and 21, respectively.

4.2.4 Organic Silt

A deposit of dark grey organic silt was encountered in Borehole BH1 below the clayey silt to silty clay deposit and in Borehole 16-1 below the fill. The organic silt extended to depths ranging from 5.6 m to 8.2 m below the existing ground surface. The SPT 'N'-values measured within the organic silt range from 0 blows (weight of hammer) to 1 blow per 0.3 m of penetration, indicating a very loose relative density. The natural water contents of the organic silt samples ranges from 27 percent to 95 percent. The organic content measured on samples of the organic silt ranges from 3.4 percent to 14.1 percent as shown on Figure 12 in Appendix D. The results of Atterberg limit tests performed on two samples of the organic silt indicated that the material was non-plastic.

4.2.5 Fine Sandy Silt to Silty Fine Sand

A deposit of grey to dark grey fine sandy silt to silty fine sand was encountered in Boreholes BH1 and BH2 below the silty clay/clayey silt or organic silt and extended to depths of 7.1 m and 6.4 m below the existing ground surface. The SPT 'N'-values measured within the fine sandy silt to silty fine sand range from 1 blow and 4 blows per 0.3 m of penetration, indicating a very loose to loose relative density. The natural water contents measured on samples of the fine sandy silt to silty fine sand range from 23 percent to 32 percent.

4.2.6 Sand/Silty Sand and Gravel

A deposit of sand and silty sand and gravel was encountered in Boreholes BH1 and BH2 below the silty fine sand and extended to a depth of about 7.9 m below the existing ground surface. The sand/silty sand and gravel deposits were inferred to contain cobbles and boulders particularly above the bedrock surface. The SPT 'N'-values measured within the sand/silty sand and gravel range from 13 blows and greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense relative density. The natural water contents measured on two samples of the sand/silty sand and gravel are 10 percent and 12 percent. The result of a grain size distribution of a sample of sand is presented on Figure 7 in Appendix D.

4.2.7 Bedrock

Bedrock was encountered in Boreholes BH1, BH2 and 16-1 to 16-3 below the sand/silty sand and gravel (BH1 and BH2), below the silty clay (16-1) or below the lake debris (16-2 and 16-3) at the depths/elevations provided below The bedrock was cored in Boreholes BH2 and 16-1 to 16-3 for lengths of 1.5 m to 4.3 m. Based on the results of rock coring, the bedrock at the site generally consists of fresh to slightly weathered, dark grey to black and pink, fine to medium grained, biotite gneiss or granite bedrock. Photos of the bedrock core are in Appendix D.

Location	Borehole	Ground/Water* Surface Elevation	Depth to Bedrock Surface	Bedrock Elevation	Unconfined Compressive Strength
East Abutment	BH1	215.1 m	7.9 m	207.2 m	-
East Abutment	BH2	215.2 m	7.9 m	207.3 m	136 MPa
West Pivot Pier	16-2	212.6 m*	4.4 m	208.2 m	95 MPa
East Pier	16-3	212.6 m*	4.1 m	208.5 m	83 MPa
West Abutment	16-1A	215.1 m	8.2 m	206.9 m	70 MPa to 87 MPa
West Abutment	16-1B	215.1 m	9.2 m	205.9 m	-

The Total Core Recovery (TCR) of the core samples in ranges from 98 to 100 percent; the Solid Core Recovery (SCR) ranged from 84 to 100 percent; and the Rock Quality Designation (RQD) ranges from 84 to 100 percent. Based on these results and on our visual examination of the core samples, the rock quality of the biotite gneiss and granite generally indicating a rock mass of good to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006).

Laboratory UCS testing was carried out on four selected samples of the recovered bedrock core. The UCS values are presented on the Record of Drillhole sheets in Appendices B and C and are summarized above The UCS values indicate that the bedrock is strong to very strong (R4 to R5, 50 MPa < UCS < 250 MPa) in accordance with Table 3.5 of CFEM (2006).

4.3 Groundwater Conditions

Groundwater was encountered in Boreholes BH1 and BH2 upon completion of drilling at depths of about 4.1 m below the existing ground surface. Test Pit TP1 was dry upon completion. The groundwater levels measured in the piezometer/monitoring well in Boreholes BH2 and 16-1 are provided below.Details of our groundwater level observations are shown on the Record of Borehole sheets.

Location	Installation	Ground Surface Elevation	Groundwater Depth	Groundwater Elevation	Date
East Abutment	BH2	215.2 m	1.9 m	213.3 m	October 11, 2011
			2.5 m	212.7 m	December 13, 2016
West Abutment	16-1	215.1 m	1.9 m	213.2 m	December 13, 2016

The river channel water surface was measured in November 2016, by Parsons, at Elevation 212.6 m. The depth of the water column in Boreholes 16-2 and 16-3 was approximately 3.9 m. The water level in the river was about 2.9 m below the bridge deck when measured on September 28, 2011.

The groundwater levels at the site should be expected to fluctuate seasonally in response to changes in precipitation and snow melt, and should be expected to be higher during the spring season or during any period of heavy precipitation. The groundwater levels at the abutment area are likely influenced by the water level in the river.

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation recommendations for the proposed replacement or remediation of the existing bridge foundations. These recommendations are based on our interpretation of the factual data obtained from the boreholes and the test pit advanced during the subsurface investigations at the site. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives in support of the structural design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

5.1 Seismic Design

5.1.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations and Vertical Seismic Profiling (VSP) testing. The results of the VSP testing is contained in Golder's Technical Memorandum No. 1664670/1000 entitled "*Vertical Seismic Profiling Test Results, Hamlet Swing Bridge, Hamlet, Ontario*" and dated December 15, 2016, in Appendix E. The results of the VSP testing indicate that the average shear wave velocity (Vs30) from the proposed underside of pile cap (Elevations 213.0 m to 213.1 m) to 30 m below the underside of pile cap is 377 m/s. Based on the measured shear wave velocity, the site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC (360 < Vs30 < 760). Table 4.1 of the CHBDC indicates that Site Class A and B are not to be used if there is more than 3 metres of softer materials between the bedrock and the underside of the bridge foundations (i.e. footings or pile caps). Since more than 3 m of soft soils are present between the pile cap and bedrock, Site Classes A and B are not appropriate for this site.

5.1.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC, the peak ground acceleration (PGA) values and design spectral acceleration values for Site Class C are presented below.

Seismic Hazard Values for Reference Ground Condition Site Class C

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.028	0.042	0.064
Sa (0.2) (g)	0.051	0.074	0.110
Sa (0.5) (g)	0.038	0.054	0.080
Sa (1.0) (g)	0.022	0.033	0.049
Sa (2.0) (g)	0.011	0.017	0.026
Sa (≥10.0) (g)	0.0011	0.0017	0.0029

5.2 Foundations

It is our understanding that the existing fixed bridge existing swing bridge are to be replaced, with the east and west abutments and east pier to be replaced and the west pier to be rehabilitated. The resting pier, which is adjacent to the west pivot pier will be rehabilitated. It is noted that both replacement superstructures will be heavier than the existing superstructures. As discussed above, the existing east and west abutments are supported on spread footings within the overburden. The two existing piers may be comprised of concrete founded on bedrock although it is not clear if the concrete in the piers is in intimate contact with the bedrock.

5.2.1 Consequence and Site Understanding Classification

It is understood that the bridge widening is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC). In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC, 2014), the proposed bridge and its foundation system is considered to be classified as having a "typical consequence level" associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree

of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the CHBDC (2014) have been used for design.

5.2.2 Foundation Options

Discussion of the advantages and disadvantages associated with the possible rehabilitation or replacement options for each foundation element are provided below.

East Abutment:

Based on the level of deterioration and movement experienced at the east abutment (i.e. settlement and tilting, consistent with the soil conditions), we understand that the east abutment foundation is to be replaced. Technically feasible foundation options for the new abutment include: micropiles cased through the overburden and having the (uncased) bond zone socketed into the bedrock; or steel H-piles driven to the bedrock surface. Shallow spread footings are not recommended at this location given the very soft to very loose nature of the overburden soils, the corresponding low geotechnical resistance and probability of poor long-term performance. Spread footings founded on the bedrock surface, at about 8 m below the ground surface, would result in major excavation works which are not considered practical given the site constraints. Driven steel piles may not be feasible as the very soft/very loose soils at the east abutment may not support the heavy loads of large piling equipment and significant site preparation may be required to construct a working platform capable of supporting pile driving equipment. Further, vibrations from the pile driving could cause damage to the existing bridge and nearby buildings and would need to be carefully monitored and controlled. Seating of battered driven piles on the bedrock surface may also be problematic during construction. In comparison, the installation of micropiles would not create significant vibrations during installation and the installation equipment is relatively small and better suited to the limited working area at the site. We recommend that the preferred option be removal of the existing abutment and founding the replacement abutment on micropiles.

East Pier:

The founding condition(s) of the existing east pier are not clear in the available documentation. Although the original drawings indicate that the east pier consists of a concrete core founded directly on the bedrock, it appears as though the original pier may have been founded on timber cribbing which may have been "jacketed" in concrete at a later date over all or a portion of the distance between the top of the pier cap and the bedrock surface. A borehole drilled through the east pier in the 1960's indicates that the pier consists of concrete over a thick sand layer over the bedrock which contradicts the information from other drawings. Therefore, it is not clear if the concrete in the east pier forms a continuous monolithic block or if it is in intimate contact with the bedrock. It is noted that the borehole drilled as part of this investigation just to the north of the east pier encountered a thin amount of organic matter at the riverbed over a bedrock surface at Elevation 208.5 m.

The actual composition of the east pier and the nature of its founding condition at the bedrock surface needs to be confirmed if a rehabilitation or upgrade strategy is to be selected.

If it is confirmed that the pier is comprised of monolithic concrete in direct contact with the bedrock surface then it may be possible to upgrade the footing capacity by installing micropiles through the existing pier. If however, there is uncertainty about the existing footing (i.e. if it is comprised of a combination of concrete and timber cribbing and the concrete is not in direct contact with the bedrock), then advancing micropiles through the existing foundation

unit may be problematic as drilling through timber cribs can be very difficult. In this case, technically feasible foundation options would include: (i) installing micropiles around the outside of the existing pier with a new structurally reinforced pile cap tying into the micropiles and independent of the existing footing; or (ii) full replacement by constructing a new spread footing on bedrock. Spread footing are feasible but would require a cofferdam to be able to construct in the dry. Given the subsurface conditions at the pier (i.e. thin to non-existent overburden over strong to very strong bedrock), sheet piling for a cofferdam would have to incorporate toe-pins drilled and grouted into bedrock to provide fixity at the base and a tremie plug would be required to seal the base of the cofferdam.

It is understood that it may not be possible to utilize a micropile option that increases the size of the pier, for aesthetic reasons and that consideration is being given to removal of the existing footing and replacement at the same location. In this regard, as a cofferdam will be required for removal, we recommend that the replacement pier be founded on a spread footing directly over the bedrock and socketed into the bedrock as may be required for lateral resistance. A sheet piled cofferdam, constructed as described above, will be required to allow for construction in the dry. The same smaller, lightweight micropile equipment utilized for the abutments could be used for the rock sockets and as such, a smaller floating platform would be required for construction.

West (Pivot) Pier:

The composition and founding condition of the west pier is also not clear in the available documentation. The original drawings indicate that the west pier consist of an approximately 7.3 m (24 foot) essentially “square” footing founded directly on the bedrock with the adjacent resting piers (that abut the pivot pier) comprised of timber cribbing. Some documentation suggests the pier itself is founded on timber cribbing. It was not possible to advance a borehole through the pivot pier to check the composition and contact condition with the bedrock. The borehole drilled as part of this investigation, just to the north of the pier, encountered a thin amount of organic matter at the riverbed over bedrock surface at Elevation 208.2 m.

The existing available information suggests that this large pivot pier is constructed of concrete founded on bedrock. If this can be confirmed and if the concrete itself is of acceptable quality, the existing footing may provide sufficient geotechnical resistance for the new loading conditions. If there is insufficient geotechnical resistance available or if the structural condition of the existing footing is not acceptable, feasible replacement or rehabilitation foundation options include: (i) installing micropiles around (or possibly through) the existing pier with a structurally reinforced pile cap tying into the micropiles and independent of the existing footing; or (ii) full replacement by constructing a new spread footings on bedrock. The advantages and disadvantages are similar to those above for the east pier.

It is understood that the existing pier has sufficient resistance to support the proposed design. We recommend that the preferred option be the rehabilitation of the existing pier by resurfacing the concrete.

West Abutment:

It is our understanding that the west abutment was reconstructed in the late 1980’s and is assumed to comprise a spread footing founded within the overburden. Based on the borehole drilled for the west abutment, the overburden consists of very loose to loose sand fill over very loose organic silt. We understand that it may be possible to rehabilitate the footing depending on the concrete quality and if sufficient geotechnical resistance is available. If there is insufficient geotechnical resistance, feasible replacement or rehabilitation foundation options include micropiles for either underpinning the existing abutment foundation, or for support of a new abutment

foundation. While driven steel H-piles or the use of spread footings could also be considered, the disadvantages described above for the East Abutment would also apply at this location.

We recommend that the preferred option be the removal of the existing abutment and founding of the replacement abutment and wingwalls on micropiles.

5.3 Existing Foundations

5.3.1 Piers

The west pivot pier and east pier may consist of concrete founded directly on the bedrock surface. As per discussions above, it is not known if this in fact the case and this should be confirmed before comparing the new loadings relative to the geotechnical resistances.

A factored ultimate geotechnical resistance of 5 MPa (at ULS) may be used for the existing footings (if founded on bedrock), regardless of the footing width. This accounts for the unknown condition of the bedrock surface at the time the footings were constructed, however it is likely that in granite/gneiss bedrock, the surface would typically remain intact. Serviceability Limit States (SLS) conditions do not apply to footings placed on the granite/gneiss bedrock which is classified as non-yielding.

The geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.4 and Section C6.10.4 of the CHBDC 2014 and its Commentary. The factored geotechnical resistance for granite/gneiss quality bedrock given above assumes that the bedrock at and below the founding level has not been fractured, and that no adverse jointing is present below the footings.

The resistance to lateral forces/sliding resistance between cast-in-place concrete and the bedrock pier should be calculated in accordance with Section 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factors as noted in Section 5.2.1. A factored coefficient of friction, tan δ, of 0.34 may be used for the interface between the old cast-in-place concrete and bedrock.

5.3.2 Abutments and Wing Walls

The east and west abutments and wing walls consist of shallow spread footings founded within the overburden. The geotechnical resistance at the east abutment is very low which is consistent with the movement observed at this location. As discussed above, a spread footing replacement option within the overburden is not recommended and on the bedrock is not practical. The west abutment, since its reconstruction over 30 years ago, could be repaired or replaced, however the geotechnical resistances are low and this should be considered.

The estimated factored geotechnical resistance of the existing west abutment footing is about 100 kPa at ULS. The estimated factored geotechnical resistance at SLS, for 25 mm of settlement, is about 50 kPa. This is for the 3 m by 7 m west abutment footing founded at a depth of 3.2 m below the ground surface (based on available documentation) on the very loose to loose sand over very loose organic silt. A factored coefficient of friction, tan δ, of 0.34 may be used for the interface between cast-in-place concrete and sand fill material.

5.4 Shallow Foundations

To achieve an increased bearing resistance compared to the existing footings in the overburden, consideration should be given to replacing the footings for the east abutment and wing walls on engineered fill over the native sandy soils after removal of the upper very soft to firm silty clay to clayey sit and very loose organic silt. Due to the presence of organic silt below the west abutment, shallow foundations bearing on overburden are not considered feasible. For the east abutment, the existing fill, silty clay to clayey silt and organic silt soils should be removed and replaced with compacted engineered fill, such as OPSS.PROV 1010 Granular 'B' Type II material. The fill should be placed in loose lifts not greater than 0.3 m, in dry conditions, and compacted to 100 per cent of the Standard Proctor Maximum Dry Density in accordance with OPSS 501, although non-vibratory compaction and extreme care will need to be used to avoid disturbance to the underlying silty sand deposit.

Spread footings having a width of 3 m similar to the existing footing placed on at least 2 m of engineered fill over the native silty sand soils may be designed based on a factored geotechnical axial resistance at ULS of 125 kPa and a factored geotechnical resistance at SLS, for 25 mm of settlement, of 75 kPa. We should be contacted to review these recommendations if this option is being considered.

Alternatively, abutments could be replaced by spread footings on bedrock, however, extensive excavation and dewatering below the groundwater/river water level would be required and this is not considered practical.

Replacement footings for the piers should be founded directly on the on the granite/gneiss bedrock at Elevation 208.2 m and 208.5 m at the west and east piers, respectively. Since the bedrock surface elevation may be variable over the footing area, stepped footings on bedrock are permitted provided there is no more than 0.3 m elevation difference between the steps. Spread footings placed on the surface of the properly prepared and inspected bedrock surface may be designed based on a factored geotechnical axial resistance at ULS of 10 MPa. The geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

For spread footings founded in the overburden, a minimum of 1.8 m of soil cover should be provided for frost protection. Alternatively, rigid insulation could be used to offset soil cover as provided in Section 5.10. For spread footings placed on fresh granite/gneiss bedrock, frost protection cover is not required.

The geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.4 and Section C6.10.4 of the CHBDC 2014 and its Commentary.

The resistance to lateral forces/sliding resistance should be calculated in accordance with Section 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factors as noted in Section 5.2.1. A factored coefficient of friction, tan δ, of 0.34 may be used between cast-in-place concrete and engineered fill material for the west abutment/wing walls. An unfactored coefficient of friction, tan δ, of 0.56may be used for the interface between cast-in-place concrete and bedrock for the piers.

It is understood that the piers should be designed to resist a horizontal impact load of 7,600 KN parallel to the channel centreline and 3,800 KN perpendicular to the channel centreline. Should a shallow foundation on bedrock be considered for either pier, consideration should be given to socketing the spread footings into the bedrock to increase lateral resistance. The bedrock sockets could consist of micropiles as described in section 5.5.

5.5 Micropiles

As discussed in Section 5.2.2, given the limited working space at the abutments and piers, the soft/loose subsurface conditions at the abutments which may limit the ability to utilize large/heavy pile driving equipment, and the general requirement to minimize vibrations and disturbance to the existing bridge and nearby buildings, the use of drilled and grouted micropiles should be considered at this site for foundation replacement and/or underpinning.

The overburden conditions at the east abutment consist of loose fills, very soft clays and very loose organic silts and sands. At the west abutment, the overburden consists of loose fills overlying loose organic silt. At the piers, the overburden is generally thin to non-existent and contains organic matter. Given this, it is not practical to attempt to design micropiles to be bonded within the overburden and as such, all micropiles are recommended to be bonded into the strong to very strong gneissic bedrock.

5.5.1 Micropiles at Abutments

5.5.1.1 Micropile Design Assumptions

Design loads have not been provided for the abutment foundations, however, considering the potential for the loads to include combinations of axial and lateral load as well as bending moments and considering the availability of typical casing and bar sizes, the following two micropile sections are provided as options for the support of the abutments:

Option	Outer Steel Casing		Inner Steel Reinforcement	
	(metric)	*(imperial)*	*(metric)*	*(imperial)*
#1	HSS 273 x 13	10-3/4” x 1/2” wall	57 (bar)	#18 (bar)
#2	HSS 194 x 12	7-5/8” x 1/2” wall	43 (bar)	#14 (bar)

It should be noted that the effects of the smaller cross-section on the lateral stiffness of the micropile must be considered from a structural point of view.

The micropile design should be based on the premise that all foundation loads will have to be fully supported by the micropiles (i.e. no contribution from the abutment or pier footings). This recommendation is in part due to concerns over the soft/very loose nature of the founding soils and potential for ongoing settlement (at the abutments)).

The design is based on the approach outlined in the FHWA/NHI Micropile Design and Construction Reference Manual, Publication No. FHWA NHI-05-039 (FHWA/NHI 2005).

In all cases, the centre-to-centre spaces between individual micropiles should be at least 820 mm (32 inches) or three micropile diameters, whichever is greater. For micropiles with the bond zone in competent bedrock, it can be assumed that a group reduction factor for total axial capacity is not required. Group reduction factors for lateral loading may be required (as per Section 5.7) depending on the pile layout and spacing.

An ultimate geotechnical resistance factor (ϕ_{gu}) = 0.5 for the axial geotechnical capacity of the micropiles has been used based on the recommendations in CHBDC (2014) and in Section 5.9.2 (FHWA/NHI 2005), considering the

micropile bond zone will be formed in the competent strong to very strong bedrock and assuming that at least one verification test will be conducted prior to commencing production micropile installation.

In consideration of the potential for aggressive ground conditions at the site, it is recommended to have a minimum 1.6 mm section loss (all around) be included in the design of the outer casing of the micropiles.

5.5.1.2 Casing and Central Bar

As noted above, two micropile options are proposed for this site:

- outer HSS 273 x 13 (10-3/4" x 1/2" wall) casing with an inner 57 mm or #18 (2-1/4") central bar; and,
- outer HSS 194 x 12 (7-5/8" x 7/16" wall) casing with an inner 43 mm or # 14 (1-3/4) central bar.

The following sections describe the details of the micropile design.

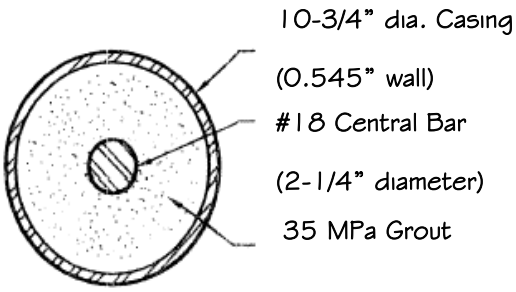
5.5.1.3 Micropile Design Details

Following the procedures outlined in the Micropile Design and Construction Reference Manual (FHWA/NHI 2005), it is recommended that the micropiles be comprised of the following components, steel grades and grout strength.

Option 1:

Steel Casing:

- API-N80 (threaded)
- 80 ksi, F_y = 552 MPa
- 10-3/4" (273 mm) outside diameter
- 0.545" (13.84 mm) wall thickness



Central Bar:

- Dywidag GEWI Threadbar (or equivalent)
- 75 ksi (Grade 500), Fy = 520 MPa
- #18 bar
- 2.25" (57 mm) diameter

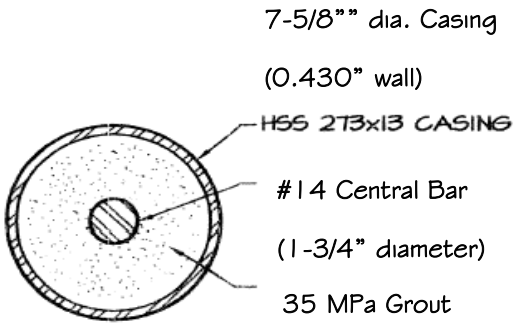
Grout:

- 35 MPa (minimum at 28 days)
- Water/Cement Ratio (by weight) < 0.45

Option 2:

Steel Casing:

- API-N80 (threaded)
- 80 ksi, F_y = 552 MPa
- 7-5/8" (194 mm) outside diameter
- 0.430" (12.70 mm) wall thickness



Central Bar:

- Dywidag GEWI Threadbar (or equivalent)
- 75 ksi (Grade 500), Fy = 520 MPa

- #14 bar
- 1.75” (43 mm) diameter

Grout:

- 35 MPa (minimum at 28 days)
- Water/Cement Ratio (by weight) < 0.45

In order to develop the axial geotechnical resistance, the micropiles will have to be socketed into the good to excellent quality, strong to very strong, gneissic bedrock. For design purposes, it is recommended that the outer steel casing extend at least 1.0 m below the top of bedrock (i.e. casing plunge length = 1.0 m). This plunge length will need to be confirmed by a lateral pile group analysis performed by the structural engineer based on the recommendations for the resistance to lateral loading provided in Section 5.7 of this report.

Verification pile load testing comprising at a minimum one axial load test (in compression) must be carried out prior to commencing production micropile installation. Depending on the structural design requirements (i.e. if axial tension loads and/or lateral loads will be acting on the foundation elements), additional pile load tests may also be required.

5.5.1.4 Axial Geotechnical Resistances

The axial geotechnical resistance of the micropiles will be primarily developed within the bond zone or the uncased lower section of the micropile socketed into the bedrock.

The grout-to-ground bond strength in the bedrock has been estimated based on the results of the tests performed on specimens of the bedrock core and from recommended values for bedrock found in Micropile Design and Construction Reference Manual (FHWA/NHI 2005) and in PTI (2014). Based on this information, a grout-to-ground/bedrock ultimate bond value (α_{bond}) of 2,500 kPa can be used for design. This value will have to be verified by the pre-production micropile load tests to be conducted at the site prior to the start of production piling.

Although the cased section of the pile will have a nominal diameter of 0.273 m (10-3/4”) or 0.194 m (7-5/8”), the uncased section of the pile in the bond zone within the bedrock will likely have a smaller diameter as a result of the method of installation and drilling that is likely to be adopted by the contractor. After advancing the casing to the required depth within the bedrock (i.e. at least 1.0 m as noted above), the contractor will seat the casing and then drill the bond zone below this depth, creating an open hole in the rock likely with a smaller diameter than the cased hole. For the purposes of design, it is assumed that the bond zone will have a minimum diameter of 0.229 m (9”) for the 0.273 m diameter pile option, and 0.15 m (6”) for the 0.194 m diameter pile option.

For the two micropile options being considered, the following factored axial geotechnical resistances at ULS may be used in design. The resistance of the plunge length (i.e. cased length below the bedrock surface, but above the bond zone) has been ignored in the design. Similarly, the resistance of the cased length above bedrock has also been ignored in the design due to the relatively weak overburden soils at the abutments).

Option	Micropile Details		Casing Plunge Length into Bedrock	Uncased Length, Bond Zone in Bedrock	Factored Axial Geotechnical Resistance at ULS (kN)
	Outer Casing	Inner Reinforcement			
1	10-3/4" x 1/2" (273 mm x 13.8 mm)	#18 (57 mm) solid bar	1.0 m	1.5 m	1,250
2	7-5/8" x 1/2" (194 mm x 12.7 mm)	#14 (43 mm) solid bar	1.0 m	1.5 m	750

The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, factored ULS conditions will govern the design for this foundation type.

5.5.1.5 Structural Design Considerations

The following structural design considerations will need to be evaluated as part of the overall design by the structural engineer.

Corrosion Potential and Protection

A steel section loss of a minimum of 1.6 mm of the wall thickness should be considered when evaluating the structural capacity of the micropiles. This section loss is the minimum recommended for sacrificial corrosion protection in the design of the micropiles outer casing (DFI 2004) considering the potential for aggressive ground conditions.

Corrosion protection of the central bar will be provided by specifying epoxy coating combined with grout cover.

Buckling Potential

At the east abutment, the overburden soils are comprised of very soft to firm clays and very loose silty/sandy soils, however their presence will provide some measure of lateral support to piles and as such, the potential for buckling is considered low at this location.

Given their relatively small diameter, the potential for buckling of the proposed micropile cross-section(s) should be checked by the structural engineer considering the combination of axial loads, lateral loads and bending moments that will be acting on the group and transferred to the critical pile(s).

Structural Capacity of Cased and Uncased Pile Sections

The axial loads along with any additional compressive stresses due to lateral loads and/or bending moments imposed on the pile group (and transferred to the critical pile(s)) should be considered in the evaluation of the structural capacity of the cased and uncased sections of the micropiles by the structural engineer.

Micropile Connection at Pile Cap

The connection between the top of the micropiles and the reinforced concrete pile cap should be designed to transfer both tension and compression loads. In addition, it is recommended that both the outer casing and inner central bar be extended into the pile cap to accommodate the load and moment transfer. The actual required

dimensions and thicknesses of the embedment, bearing plate and stiffener plates will have to be determined by the Structural Engineer.

At the east abutment, based on the concrete coring results, the existing concrete exhibited relatively poor quality, including uneven distribution of aggregate sizes and lack of aggregates in some sections of the cores. Some vertical cracks were also observed in the horizontal concrete cores. The compressive strengths of the concrete cores from the existing abutment ranged from about 14 MPa to 18 MPa, which should be considered in the structural design for underpinning as it may affect the underpinning strategy. Depending on the structural evaluation, full replacement of the east and west abutments, rather than underpinning, may be preferred.

5.5.1.6 Installation Considerations

The following construction and installation considerations will also need to be considered in the context of the overall project.

Drilling Requirements

The contractor must select a drilling method that will minimize the potential for ground loss and disturbance to the existing foundations during the advancement of the micropiles through the very soft clayey and very loose organic silt deposits, the very loose sandy/silty deposits (especially the through cobbles and boulders) and into the bedrock in order to minimize the risk of surface settlement and further movement of the existing abutment as well as disturbance to the piers and surrounding buildings.

In this regard, it is important that rotary duplex drilling techniques, with the cuttings returning up the inside of the casing, be utilized to advance the holes for the micropiles.

Micropile Grouting

Because the grout is such a vital component of the micropile, close attention must be paid to the control and quality of the product. A grout quality control plan, at a minimum including cube or cylinder compression testing and grout density (water/cement ratio) testing, should be included and reviewed by the engineer prior to the construction.

Type A micropile grout placement techniques (i.e. gravity fill placement techniques) by tremie methods are considered to be feasible for the cased micropile installation as discussed above. The grouting process should be inspected by the geotechnical engineer to ensure the quality.

Grouting of Annulus around Casing Plunge Length

Given the type of tooling and nature of the micropile casing drilling, a small annulus will be created between the outside of the micropile casing embedded in the bedrock (i.e. the plunge length) and the adjacent bedrock. At the abutments where the overburden is thick and will provide some amount of lateral resistance, this small annulus is expected to have negligible impact on the lateral performance of the micropiles. However, where the overburden is thin and very weak or non-existent, the presence of this annulus may affect the performance of the existing foundations depending on the design load combinations. In this regard, consideration may have to be given to specifying that this annular space be properly filled with grout as part of the construction.

Technical Specification

Once the overall foundation requirements for the project are known, if micropiles are adopted, it is recommended that a technical specification be developed to address the issues described above, to outline the minimum requirements for micropile load testing (i.e. both pre-production verification tests and production proof tests), and to specify the minimum requirements for the submissions, the quality control, and the materials, equipment and construction.

5.5.2 Micropiles at East Pier

It is understood the east pier will be removed and replaced with a spread footing founded on the bedrock surface. In addition, rock anchors will be required to resist concentrated impact forces from ship collisions.

5.5.2.1 Rock Anchor Design Criteria

The design criteria includes embedment lengths for a range of rock anchor and hole size options. It is our understanding that the rock anchor resistances are to be incorporated in the anchor design for the east pier to resist a Ship Concentrated Collision Impact Force of 7,600 kN (Ps) and that the Structural Engineer will use the information provided herein to design the anchors to resist loading scenarios as follows:

- 100% of the ship collision force, Ps, applied in a direction parallel to the alignment of the centreline of the navigable channel; and
- 50% of the ship collision force, Ps, applied normal to the direction of the centreline of the channel.

Rock anchor design criteria have been developed for untensioned 150 KSI (1,034 MPa ultimate tensile strength) fully threaded dowels such as the Williams All-Thread-Bar[1]. Based on the subsurface conditions encountered in nearby drillholes, it is assumed that the anchors will be installed in the fresh to slightly weathered granite and therefore the grout-to-rock bond strength for all anchors is assumed to be 2,500 kPa. A strength reduction factor of 0.4 was applied to the grout-to-anchor ultimate pull-out resistance based on the CHBDC. The actual bond strength for the rock-grout interface must be verified with rock anchor tension testing in the field. The following table provides the rock anchor design criteria and rock anchor type options.

Parameter	32 mm bar (1-1/4”)	46 mm (1-3/4”)	65 mm (2-1/2”)
Bar Minimum Yield Strength[1]	667 kN (150 kips)	1388 kN (312 kips)	2766 kN (622 kips)
Embedment / Bond Length	3.25 m	5.50 m	7.25 m
Hole Size Range	100 to 125 mm	125 to 150 mm	150 to 175 mm

5.5.2.2 Rock Anchor Installation Considerations

The commentary in this section relates to geotechnical engineering design aspects or the rock anchors for ship collision resistance at the east pier. The recommendations herein are based on our interpretation of the subsurface

[1] 150 KSI All-Thread-Bar properties (ASTM A722) have been taken from Williams Form accessed on their website January 16, 2018: http://www.williamsform.com/Ground_Anchors/Tieback_Tiedown_Anchors/tiedown_tieback_accessories.html

conditions and the project requirements as provided to Golder. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the rock anchors. The assumptions used as the basis of this memorandum regarding project layout, dimension, staging, and equipment, along with available geotechnical data should be reviewed by Parsons for consistency with the planned execution of the work.

Our professional services for this assignment address only the geotechnical (physical) design of rock anchors for the east pier. It is Golder's understanding that anchor arrangement and size selection will be determined by Parsons.

For rock anchor installation, it is assumed that all topsoil, overburden, loosened or moderately to completely weathered rock and deleterious material will be removed from the bedrock surface prior to anchor drilling and installation. Once the pier footing foundation has been cleaned, the rock conditions should be verified by a qualified geotechnical engineer. Based on available information, the bedrock is understood to be slightly weathered to fresh granite.

5.5.2.3 Rock Anchors

- Rock anchors shall be fully threaded Grade 150 All-Thread-Bar anchors or equivalent with minimum yield strengths as outlined in table above; size and arrangement of rock anchors shall be determined by others.
- Rock anchors shall be installed using a shrinkage compensating cement grout (Will-X or equivalent) to fully bond the anchor.
- Rock anchors should have a minimum stick-up length of 100 mm of coarse thread suitable for coupling, tensioning, and/ or proof testing;
- Rock anchors shall have a continuous thread for load resistance, coupling, and proof testing (if required) and;
- Couplings, if required, shall be compatible with the threaded bars; and
- Adequate corrosion protection for the anchors to suit the design life of the structure.

5.5.2.4 Grout

- Cement grout for anchors shall be pre-mixed, non-metallic shrinkage compensating grout placed according to the manufacturer's specifications (Will-X or equivalent);
- Grout shall have a minimum 28-day compressive strength of 35 MPa;
- Water for use in grout mixes shall be clean and free of deleterious substances and the water shall be filtered if necessary to reduce the suspended solids to less than 500 mg/litre;
- All anchors shall be fully cement-grouted using a water to cement ratio (by weight) of 0.35 to 0.45, or as per manufacturer's specifications;
- Cubes shall be cast to check the performance of the grout; and
- Bleed tests shall be performed to check the performance of the grout.

5.5.2.5 Rock Anchor Drillholes

- Drillholes for the rock anchors shall be drilled to depths as specified above based on the bar size;
- Diameter of drillholes in rock to host rock anchors shall be in accordance with the manufacturers recommendations for the bar size selected (including the required corrosion protection);
- Embedment length is dependent on rock anchor size and hole size, as outlined in table above;
- Bond length is presumed to be 100% of the embedment length; and
- All rock anchor holes shall be water tested and if necessary grouted and re-drilled prior to anchor installation unless the level of the grout can be verified in the field and additional grout added to the hole in the event of grout leakage (grout must be topped up if it leaks out of the hole prior to setting).

5.5.2.6 Rock Anchor Installation

The anchor criteria are based on embedment depths and fully bonded lengths as outlined in the above table. Prior to installation the bedrock conditions shall be assessed by a qualified geotechnical engineer.

- Rock anchor bond recommendations specified above represent the minimum requirements for adequate resistance to the design loads;
- All drillholes shall be flushed with water to remove all drill cuttings;
- Rock anchors shall be installed and cement-grouted at least 48 hours prior to rock excavation in any areas within 10 m of the anchor installation locations;
- Where rock anchors are required to be installed at angles less than 75 degrees to the rock face and/or if needed for testing concrete bearing pads shall be provided to ensure bearing surfaces are perpendicular to the axes of the rock anchors;
- Depths of each hole shall be no more than 100 mm deeper than the length of the rock anchor to be installed; and
- Plastic centralizers shall be positioned along the anchor bars every 1.5 m before installation and the bars should be maintained in a central position in the hole until the cement-grout has set. Note that a minimum of two centralizers per anchor shall be installed for maintaining central placement.

Because the ground-to-anchor bond stress is highly dependent on the construction/installation techniques, tension performance testing in accordance with FHWA-NHI-05-0390 (to be observed by Golder) will confirm anchor performance.

5.6 Driven Piles

As discussed above, the logistics of pile driving may preclude this from being a viable option. The potential for large vibrations during piling and the need for large piling equipment and limited space, along with low bearing capacity of the soils at the east abutment to support the equipment loading. However, this foundation option would eliminate the need for deep excavations and is an economical option. If considered, the abutments and piers could be founded on end bearing steel H-piles driven to practical refusal on the granite/gneiss bedrock.

The tip elevation of the piles at the foundation elements, based on the borehole information obtained, are as follows:

Location	Borehole	Ground/Water* Surface Elevation	Bedrock Elevation	Recommended Pile Tip Elevation	Anticipated Pile Length (below pile cap at 1.8 m depth)
East Abutment	BH1	215.1 m	207.2 m	207.2 m	6.1 m
	BH2	215.2 m	207.3 m		
West Pivot Pier	16-2	212.6 m*	208.2 m	208.2 m	4.4 m (below water)
East Pier	16-3	212.6 m*	208.5 m	208.5 m	4.1 m (below water)
West Abutment	16-1A	215.1 m	206.9 m	206.9 m	6.4 m
	16-1B	215.1 m	205.9 m		6.4 m

There should be a provision made in the contract for dealing with varying pile lengths due to the variability in the bedrock surface elevation as evidenced from the sloping bedrock behind the west abutment and the previous drawings documentation which shows a variable bedrock surface along the length of the bridge. Although not encountered in the boreholes at this site, obstructions such as cobbles and boulders may be present within the overburden and should be expected to be present above the bedrock. Further, obstructions such as old timbers, concrete or steel may also be present and should be identified in the contract documents.

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.6.1 Geotechnical Axial Resistance

For HP310X110 piles driven to the granite/gneiss bedrock assumed to have an average unconfined compressive strength of greater than 100 MPa, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. It should be noted that UCS testing on the bedrock samples is ongoing and this value may be revised pending these test results. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

We recommend that the final grade should not be increased and the average unit weight of the backfill materials adjacent to piles should not be more than the average unit weight of existing fills. Otherwise, additional loading may occur, that would cause additional stresses in the very soft silty clay soil deposits surrounding the piles. The additional stresses would cause settlement as well as downdrag (negative skin friction) on the piles, which should be considered in the structural designs (see Section 5.8).

5.6.2 Set Criteria

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) 903 (Deep Foundations). To minimize seating difficulties and to protect the pile tips from damage during driving through obstructions, the piles should be fitted with driving shoes to minimize damage to the pile tip during driving in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe).

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to be set to also avoid overdriving and possibly damaging the pile. The "set criteria" on each pile must be confirmed by a qualified, full-time, piling inspector. Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules, but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced to 75 per cent and the pile should be re-driven in 2 sets of 10 blows to improve the process of seating the pile on the sloping bedrock surface.
- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.
- The pile capacity should then be verified in the field by the use of Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity.

5.6.3 Monitoring

Conventional pile driving operations may cause vibrations that could cause some movement of soils in the surrounding area, which could potentially induce settlements of any nearby shallow foundations beneath adjacent structures (i.e., if they are not on piled foundations or shallow foundation on bedrock). Careful monitoring records of the piling operation and monitoring of the surrounding structures during piling are recommended. An evaluation of existing surrounding foundation types and a pre-construction condition survey should be carried out prior to pile driving operations.

The pile installation should be monitored full-time by the design engineers or their representatives. This is especially important with the founding conditions at this site, as the piles must be driven to develop their axial bearing resistances on bedrock, where bottom damage may occur. Consequently, Golder should be retained to monitor pile installation, as well as to review the design drawings and specifications prior to tendering.

Subsoil movement during pile driving should also be considered, especially for the pile driving adjacent to the existing sheet pile walls. Close control over final pile tip elevations must be maintained and re-tapping of selected piles may be necessary. Also, during winter pile driving operations, energy losses of 20% or more can be anticipated. These losses should be considered during selection of the pile driving equipment and during driving operations.

5.7 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. Alternatively, the resistance to lateral loading will have to be derived from the soil or bedrock in front of the piles. In the case of battered driven piles, precautions during driving are necessary in some situations (such as for specific soil/bedrock conditions/pile lengths and where the batter is shallower than about 1 horizontal to 6 vertical) to ensure that the piles do not deflect along the bedrock surface even with relatively horizontal bedrock. In this regard, it is recommended that the pile batter be restricted to 1H: 8V or steeper for driven piles. For drilled piles (i.e. micropiles), there is less of a concern and advancing casing into the strong to very strong bedrock, even when battered, should not be a problem so long as good drilling practices are followed.

The resistance to lateral loading in front of a single vertical pile may be estimated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m). However, the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum lateral pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material remains linear (CFEM, 2006). If one or more of these conditions are not satisfied, then it is recommended that the lateral pile resistance be evaluated using P-y curves as presented in Appendix F. P-y curves representing the non-linear response of the soils and linear response of the rock under lateral loading from the pile foundations have been generated using the commercially available program LPILE (version 2016) produced by ENSOFT Inc. The P-y curves have been generated considering the cyclic loading condition. The family of P-y curves are presented at 0.5 m depth increments in tabular format and shown graphically on Figures F1 and F2 for a single vertical 194 mm diameter micropile (Option 2) at east and west abutments, respectively.

The following equations as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (1992) may be used to calculate values of k_h.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction, as given below;

z is the depth (m); and,

B is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

The following values of n_h (Terzaghi, 1995 and Reese, 1975) and s_u may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses. The soil stratigraphy has been generalized and the values reflect the variability in the subsurface conditions at each foundation unit.

Location	Elevation (m)	Soil/Rock Type	n_h (MN/m^3)	s_u (kPa)
East Abutment (BH1, BH2)	211.8 – 209.9 209.9 - 207.2	Very soft to soft Silty Clay Very loose to compact Sandy Silt to Sand	- 10	20 to 40 -
	Below 207.2	Bedrock	(see below)	
Piers (16-2, 16-3)	Below 208.5 (E) Below 208.2 (W)	Bedrock	(see below)	
West Abutment (16-1)	213.3 – 211.1 211.1 – 206.9	Very loose to loose Sand Fill Very soft to soft Silty Clay Very Loose Organic Silt	5 - -	- 20 to 40 15 to 20
	Below 206.9	Bedrock	(see below)	

For piles socketed into the bedrock (i.e. the micropiles), it is anticipated that the rock will remain in the elastic range under the lateral loading. Based on this assumption, closed form solutions have been used for the estimation of the bedrock lateral spring constant, used in development of P-y curves.

Based on the assessed lateral rock mass elastic modulus of the gneissic bedrock, E_h = 34,000 MPa, and a Poisson's ratio of 0.2, the lateral rock mass spring constant is given by:

$$k_h = \frac{4\pi(1-\nu)E_h}{(3-4\nu)(1+\nu)}\frac{1}{\ln(r_o/r_i)} = \frac{4\pi(0.8)(34{,}000)}{(2.2)(1.2)}\frac{1}{\ln(10)} = 56{,}200 \ \text{MN/m/m}$$

Where:

r_i = radius of micropile

r_o = radius of 'zero' deformation; typically 10 to 15 pile diameters.

The passive resistance for the micropile bedrock sockets were analysed using the RMR assessment of the bedrock which utilizes the bedrock strength measurement obtained from the UCS testing on the recovered rock core. The ultimate lateral capacity of the bedrock is estimated to be 67 MPa. The capacity per metre length of micropile within the bedrock (in kN) can be determined by multiplying the Ultimate Lateral Capacity given above by the diameter of the micropile. A factor resistance of 0.5 should be applied to the ultimate capacity for design.

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (CHBDC Commentary 6.11.2.2).

The upper zone of the soil (down to a depth below the pile cap equal to about 1.5xB (after Broms, 1964, where B is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation. In addition, the upper 1.8 m zone of soil below the ground surface should be neglected as it is within the frost zone. The greater depth of these two cases should be used in assessing the depth of lateral resistance to be neglected in the upper portion of the pile.

Group action for lateral loading should be considered in soil, when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1986) as follows:

Pile Spacing in Direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre to centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

For rock sockets and when the centre to centre pile spacing is greater than 3 times pile diameter (D), subgrade reaction reduction factor is negligible.

5.8 Downdrag (Negative Skin Friction)

If any grade raise or widening of the approaches to the abutments is to be carried out that would result in the placement of new fill around or adjacent to the perimeter of the abutments and if the new or rehabilitated abutments are to be supported on piles, the additional loading to the very soft silty clay to clayey silt will cause downdrag and create drag loads on the piles. The estimated unfactored drag load acting on the steel H-piles at this location may be taken as 100 kN per pile. Downdrag loading on micropiles will require additional complex analysis. The structural capacity of the piles must be checked for the factored dead loads and downdrag loads in accordance with Section C6.8.4 of the CHBDC Commentary for ULS conditions.

5.9 Lateral Earth Pressures for Design of Abutments and Wing Walls

The lateral earth pressures acting on the abutment walls and any associated wing walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 *(Aggregates)* Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 *(Compacting)*. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (Case (a) on Figure C6.20 of the Commentary to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the Commentary to the CHBDC).

5.9.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_P	3.0

- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_P	3.7	3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

- Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
- Horizontal translation of 0.001 times the height of the wall; or,
- A combination of both.

- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

5.9.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6.5 of the *CHBDC*. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Seismic Active Pressure Coefficients, K_{AE}

	Design Earthquake	Site PGA	Granular 'A'	Granular 'B' Type II	SSM
Yielding Wall	**475-Yr**	0.028	0.28	0.28	0.34
	975-Yr	0.042	0.28	0.28	0.35
	2,475 Yr	0.064	0.29	0.29	0.36
Non-Yielding Wall	**475-Yr**	0.028	0.29	0.29	0.35
	975-Yr	0.042	0.30	0.30	0.37
	2,475 Yr	0.064	0.31	0.31	0.41

The K_{AE} value for a yielding wall is applicable provided that the wall can move up to $250k_h$ mm, where k_h is the site specific PGA as given in the table above. This corresponds to displacements of 7, 11, and 16 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

5.10 Scour Protection and Frost Protection

Based on the existing documentation, there is erosion at both the east and west abutments due to surface water run-off from the roadway. At the east abutment, sheet piles were installed in front of the east abutment for scour protection with large rip rap/stone. The sheet pile walls were installed many years after the bridge construction to prevent further erosion at the east abutment, however, it is noted that the large rip rap/stone has been mostly displaced. At the west abutment, the large rip rap/stone is largely intact with some displacement.

Abutment stems, pier caps, spread footings and for any associated concrete wing walls/retaining walls, should be founded at a minimum depth of 1.8 m below the lowest surrounding grade, to provide adequate protection against frost penetration. The bridge approach slopes and slopes at the abutments except for the section with existing sheet piles should be armoured with at least 1.0 m of stone riprap and rock blocks. It should be noted that the riprap or rock blocks should not be counted as earth cover for frost protection.

Rigid insulation may be used as an alternative to providing the standard depth of soil cover for frost protection of exterior footings. Assuming a minimum depth of soil cover of 300 mm adjacent to the exterior foundation wall, the insulation should consist of a 100 mm (4 inch) thick layer of Styrofoam HI-40 insulation (or equivalent), extending 1.8 m laterally (with a 2 percent downward slope) from the intersection of the exterior foundation wall and the top of footing. A 100 mm thick vertical section of Styrofoam should be placed against the upper foundation wall up to exterior grade level.

In addition, the bearing soil or bedrock and fresh concrete should be protected from freezing during cold weather construction.

5.11 Construction Considerations

5.11.1 Excavations and Groundwater Control

Depending on the foundation options choses for rehabilitation or replacement of the piers and abutments, excavations may extend to depths of up to 4 m or deeper below the ground surface through the existing fill and native soils. If space permits, open-cut excavations to the proposed depths should be carried out in accordance

with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fills are classified as Type 3 soil according to the OHSA; the existing very soft to soft silty clay, very loose organic silt and very loose to loose sandy silt to silty sand to sand or fill deposits below the groundwater table are classified as Type 4 soil according to OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical for Type 3 soil and 3 horizontal to 1 vertical for Type 4 soil. In addition, care must be taken during excavation to ensure that adequate support is provided for any existing structures and underground services located adjacent to the excavations.

If adjacent structures and/or utilities are susceptible to damage during construction, then engineered excavation support such as sheet-piles or braced soldier pile and lagging walls may be considered. However, the presence of the cobbles and boulders may make present difficulties for driving sheet-piling or soldier piles. Shoring should be designed by a Professional Engineer to meet the requirements of OPSS.PROV 539 including assessment of the potential for basal heave. Design of temporary works will be entirely the responsibility of the contractor.

Groundwater control in the form of positive dewatering (wells, well points, etc.) at the abutments and wing wall locations will be required to allow for construction of foundation elements in a dry conditions, depending on the foundation type chosen. Spread footings placed at the required depth at the west abutment will be at or below the groundwater level. Further, the proximity to the river may also allow for a direct groundwater connection. Where pile caps are required, depending on the final elevation, sump pumping may be considered but should be reviewed when final details are known. It should be noted that wet cohesionless sandy silt/silty sand and silt were encountered below the very soft to soft silty clay at the east abutment. Depending upon the actual thickness and extent of these wet sandy silt/silty sand/silt zones, some form of positive groundwater control may be required to maintain the stability of the base and side slopes of the excavations in these areas, in addition to pumping from sumps. Design of dewatering systems will be entirely the responsibility of the contractor.

A cofferdam or similar structure will be necessary to provide temporary excavation support and to facilitate groundwater control for the east pier replacement. A cofferdam may also be required at the east and west abutments, should excavations extend below the channel water level. Groundwater control measures or dewatering should be designed and implemented by a specialist contractor and carried out to a depth of at least 1 m below the excavation base level, or as necessary to ensure stable conditions during excavation. Where cofferdams extend to bedrock, toe-pins should be drilled and grouted into the bedrock. The base of the cofferdam should be sealed against the bedrock using a tremie plug on both sides of the wall. Since the depth to bedrock may be variable along the perimeter of the cofferdams, complete sealing of the sheet piles into the bedrock may not be possible.

The native soil deposits below the groundwater table at the abutments are very easily disturbed and may not be able to support heavy construction equipment. Concrete mud slabs or granular pads should be considered to provide stable working surfaces for the construction equipment.

It is recommended a “public digging” (i.e. test pitting) be carried out during the tender stage, to allow prospective bidders to assess the subsurface conditions and determine the type of groundwater control required, consistent with their equipment capabilities and the actual groundwater conditions at that time. The locations of the test pits should be determined in consultation with the geotechnical engineer.

An accurate prediction of the groundwater pumping volumes cannot be made at this time, as the flow rate would be dependent on construction methods adopted by the contractor. Even with the river water flow diverted and the

dewatering system installed to mitigate groundwater inflows, pumping volumes could exceed 50,000 L/day during initial drawdown stages and/or during periods of heavy precipitation. An application under the Environment Activity Section Registry (EASR) of the Ontario Ministry of Environment and Climate Change (MOECC) should be submitted in the event that the pumping volumes exceed 50,000 L/day. Under the EASR, a Permit to Take Water (PTTW) is not required for water taking for construction site dewatering for volumes less than 400,000 L/day. Additional hydrogeological study may be warranted in support of the PTTW depending on construction methods and equipment used. Pumping discharges should also conform to any requirements from the local municipalities, conservation agencies and Parks Canada.

5.11.2 Monitoring Wells

Monitoring wells were installed in selected boreholes to permit monitoring of the groundwater levels at the site. Ontario Regulation (O.Reg.) 903 amended by O.Reg. 128/03 of the Ontario Water Resources Act requires that monitoring wells/piezometers are properly abandoned / decommissioned by qualified personnel. We recommend that provision is included in the tender documents for the contractor to formally decommission the groundwater monitoring wells installed for this investigation after completion of dewatering activities. If requested, Golder could provide assistance to the property owner in arranging for the decommissioning of the monitoring wells by a licensed water well drilling contractor.

5.11.3 Obstructions

Although not encountered in the boreholes drilled for this site, cobbles and boulders may be encountered within the overburden and above the bedrock surface. Further, the presence of wood (old timbers), concrete and steel, particularly in unexpected locations, should be anticipated and accounted for in the contractor's equipment selection, especially in micro-pile installation or pile driving. It should be noted that the size, quantity and exact location of all obstructions cannot be definitely determined by single borehole locations and we recommended that the contract document include a warning to the potential for these obstructions.

5.12 Approach Embankments

The existing bridge approach embankments along this section of road are up to 3 m high relative to the river and also the surrounding topography. Based on visual observations, the existing approach embankment slopes appear to be stable, although some leaning sign posts were observed. It is understood that a grade raise of 0.6 m is required on the west approach and a grade raise of approximately 0.2 m is required at the east approach. Given the past disturbance/movement of the east abutment, grade raises will pose challenges with regards to settlement and potentially stability of the approach embankments and additional loading is not recommended.

5.12.1 Subgrade Preparation and Backfill Behind Abutments and Wing Walls

Based on the gradation analysis results, the backfill materials at the east abutment (i.e. sandy silt fill and organic fill) are considered to be highly frost susceptible. At the west abutment, the sand backfill material has low frost susceptibility. There is no evidence of drainage systems behind the abutment wall and wing walls. It is expected that the existing backfill will be removed for either the full replacement or rehabilitation options. The excavation should be backfilled to the underside of the pavement structure using Granular 'B', Type I material. The Granular 'B' backfill should be placed in accordance with OPSS.PROV 206. The final lift prior to placement of the granular subbase and base courses should be compacted to at least 100 percent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during

all engineered fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00. The drainage system should be installed above the local groundwater level.

The abutment walls and wing walls should be properly braced to provide sufficient temporary support during construction. The excavation and brace plans should be submitted and approved by the project engineer. The requirements for temporarily supporting the existing bridge during construction should be evaluated by the structural engineer.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding or pegged sod should be placed as soon as possible in accordance with OPSS 572. If this protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting, is recommended to reduce the potential for remedial works being required on the side slopes in the spring prior to topsoil and seeding.

5.12.2 Stability

The stability of the proposed abutments in this area is governed by the existing surface and subsurface conditions, proposed abutment and embankment geometry, and loading conditions. Site specific static slope stability analyses were undertaken to analyse the global factor of safety of the retaining wall using the commercially available program SLIDE (Version 7.0), produced by Rocscience Inc. employing the Morgenstern-Price Limit Equilibrium method of analysis.

The slope stability factor of safety is defined as the ratio of the forces tending to resist failure relative to the driving forces tending to cause failure. The Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.54 is adopted for the design of embankment slopes under static conditions at the end of construction as per the CHBDC (2014).

Slope stability analysis was carried out for the proposed geometry given in Drawing No. B1 and using an idealized soil profile based on the results of the field investigation. The soil parameters used for the stability assessment are based on the results of the borehole investigation and inspection. The groundwater level was assumed to be at Elevation 213.3 m based on groundwater measured in the wells and open boreholes installed as part of Golder's geotechnical investigation.

The results of the stability analysis indicate a factor of safety against global instability of the proposed abutments is about 1.5, and as such is considered to be acceptable.

Once detailed cross sections of the final geometry are available at the east and west abutments, Golder should be given the opportunity to confirm the stability results.

5.12.3 Settlement

Depending on the source and gradation of the Granular 'B', the unit weight of the replacement backfill may be greater than the unit weight of the original backfill. An increase in the unit weight of the backfill material would increase the loading on the native soils and could potentially result in consolidation of the very soft to soft silty clay and very loose organic silt deposits at the abutments. Further, the anticipated grade raises will also result in

additional loading on the subsoils and result in consolidation settlement. Due to the past problems associated with settlement/movement of the soft soils at the east abutment and possibly the west abutment, we recommend avoiding any additional loading at the abutments/approaches to minimize the potential for future settlement problems and avoid downdrag loads on the micropiles. The use of a lightweight fill may be considered for part of the backfill zone offset any additional backfill loading and avoid consolidation.

In order to offset the loading from the grade raise as well as from the different unit weight of backfill, consideration could be given to the use of lightweight expanded polystyrene (EPS) to replace soil backfill. An approximately 1 m thick zone of EPS would be required within the embankment mass to avoid inducing new consolidation settlements. At the west abutment, the 1 m thick zone of EPS would be required from the abutment wall to approximately 20 m behind the wall and then reduced to 0.5 m thick for another 10 m, for a total of 30 m distance from the abutment. At the east abutment, the 1 m thick zone of EPS would be required from the abutment wall to approximately 5 m behind the wall and then reduced to 0.5 m thick for another 5 m, for a total of 10 m distance from the abutment.

The EPS fill should be installed in accordance with the manufacturer's requirements. It is recommended that a minimum 300 mm thick levelling pad comprised of OPSS PROV. 1010 Granular 'A' be placed prior to the installation of the EPS. The EPS should be covered on top and sides by a 0.25 mm (10 mil) thick polyethylene sheet, followed by the placement of a protective cover/pavement structure over the EPS (for a minimum thickness of 1 m including compacted granular materials and asphalt). The EPS on the side slopes of the embankment should be covered with a minimum 1 m thick layer of conventional soil. A geomembrane should be placed between the EPS and the concrete abutment wall.

5.13 Pavement Design

It is understood that the Hamlet Bridge and the approaches to the bridge will be rehabilitated. The bridge is comprised of two spans, an east fixed span and a west swing span. The bridge is a single-lane crossing with an overall width of about 5.5 meters. The swing span features a nail laminated timber deck with planks laid on edge over a steel floor system with floor beams and stringers. The fixed span features a nail-laminated timber deck with planks laid on edge spanning transversely over a steel floor system composed of steel floor beams and stringers.

Based on the results of the 2011 and 2016 geotechnical investigation, the pavement structure encountered on the east approach to the bridge consists of approximately 80 mm of asphalt overlaying about 300 mm of granular material. On the west approach, the pavement structure consists of approximately 35 mm of asphalt overlaying about 80 mm of granular material.

The traffic information is not available at this time and the District of Muskoka "Road Needs Study, 2013 update" from January, 2014 classifies Muskoka Road 49 as a Rural/Semi-Urban Road with an AADT of 350. Based on the results of the geotechnical investigation and limited traffic information available at this time, the recommended pavement structure for the approaches to the bridge embankments is as follows:

- 40 mm HL3 Surface Course (or Superpave SP 12.5)
- 50 mm HL3 Binder Course (or Superpave SP 12.5)
- 150 mm New Granular A Base
- 400 mm Granular B, Type I Subbase

It is recommended that PG 58-34 asphalt cement be used for the HL3 surface and binder courses. The same mix type is recommended for both asphalt lifts due to the small quantities of hot mix asphalt so as to minimize the number of mix types. Alternatively, Superpave hot mix asphalt could be used in place of conventional Marshall Type mix. The traffic category A and PGAC 58-34 are recommended for the Superpave alternative for this project.

We understand that the existing backfill material behind the abutment walls and the wing walls will be removed and replaced with Granular 'B', Type I material and it is anticipated that this will require partial removal of the embankment fill at the approaches to the structure. The exposed embankment fill material should be proofrolled to delineate any loose/ soft areas. If such areas are identified, the fill material in those areas should be removed and replaced with compacted Granular 'B', Type I material. The embankment should be brought up to 240 mm below finished pavement grade using Granular 'B', Type I material (minimum of 400 mm).

The Granular 'B', Type I material should be placed in maximum 200 mm thick compacted lifts. The lower lifts should be uniformly compacted to 95 percent of the materials Standard Proctor Maximum Dry Density (SPMDD), with the upper two lifts (400 mm) compacted to 100 percent of the SPMDD.

The granular base material should be uniformly compacted to 100 percent of its SPMDD. If there is a delay in placing the HMA (more than 7 days), the granular base should be proof rolled prior to placement of the hot mix asphalt. The asphalt material should be compacted to 92 to 96.5 percent of its Maximum Relative Density (MRD).

Where new pavement abuts existing pavement (e.g. at the construction limits), proper transition should be constructed to key the new asphalt into the existing pavement. The existing asphalt edges should have a proper sawcut edge prior to keying in the new asphalt. It should be ensured that any undermined or broken edges resulting from the construction activities are removed by sawcutting.

6.0 CLOSURE

Prior to tendering, the geotechnical aspects of the final design drawings/specifications and proposed construction methodology should be reviewed by this office to confirm that the intent of this report has been met.

During construction, Golder personnel should confirm that the subsurface conditions encountered at the foundation locations are consistent with those in the boreholes. Sufficient site visits should also be carried out during bridge construction to monitor conformance with the pertinent project specifications.

We trust that this report provides sufficient geotechnical engineering information to complete the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Report Signature Page

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. E. M. POOT
90471921
1/19/18
PROVINCE OF ONTARIO

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EW/SEMP/mes

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LEGEND

- CONTOURS (M)
- RAILWAY
- ROAD
- WATERCOURSE
- APPROXIMATE SITE BOUNDARY
- BUILDING
- MUNICIPAL BOUNDARY
- WATERBODY
- WETLAND
- WOODED AREA

0 500 1,000

1:25,000 METRES

CLIENT
PARSONS CORPORATION

PROJECT
GEOTECHNICAL INVESTIGATION
HAMLET BRIDGE, HAMLET, ONTARIO

TITLE
KEY PLAN

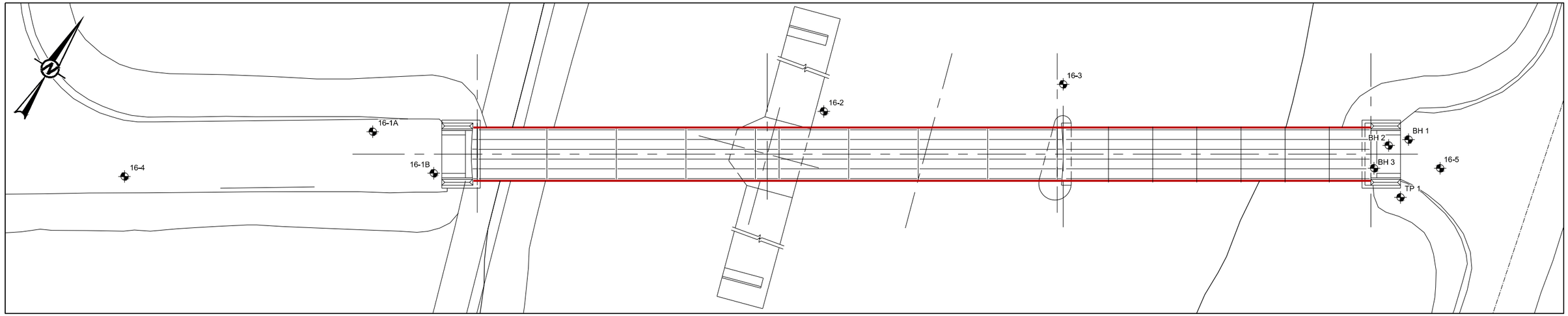
CONSULTANT

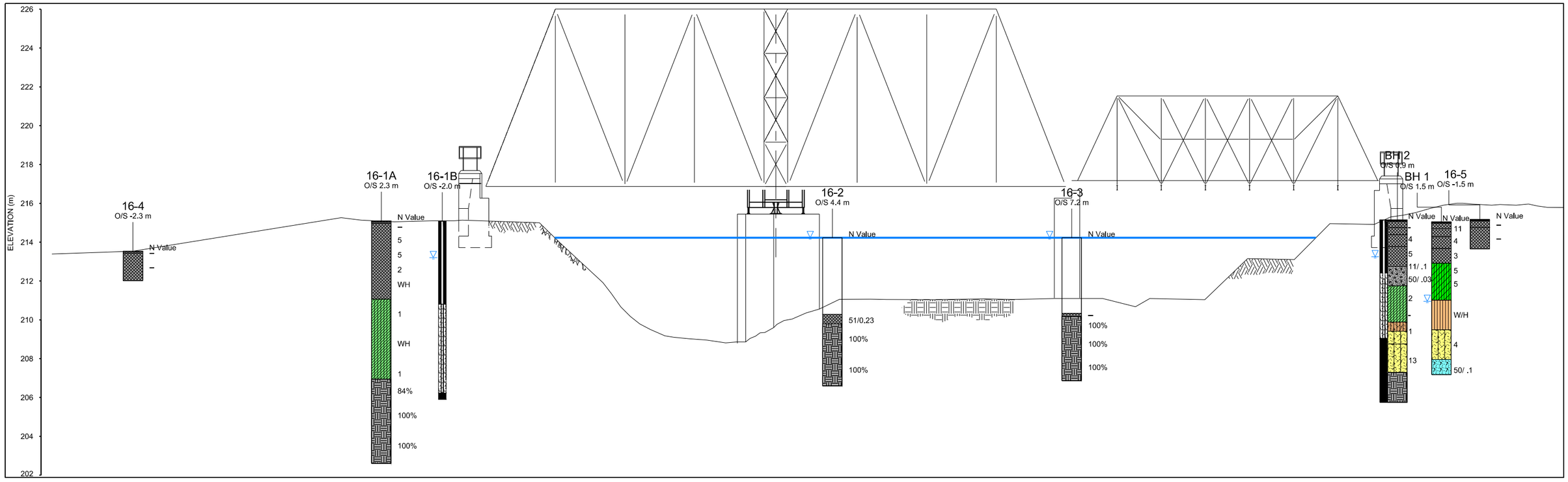
Golder Associates

YYYY-MM-DD	2016-12-07
DESIGNED	JT
PREPARED	JT
REVIEWED	EW
APPROVED	

PROJECT NO.	CONTROL	REV.	FIGURE
1659973			1

REFERENCE(S)
BASE DATA - MNR LIO, OBTAINED 2016
PRODUCED BY GOLDER ASSOCIATES LTD UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2016
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17N

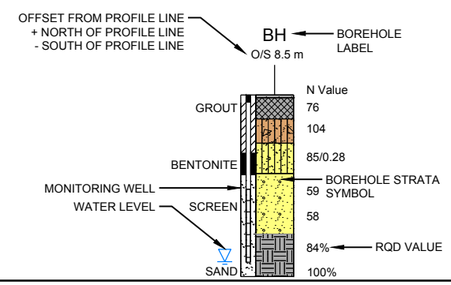




SOIL STRATIGRAPHY

- ASPHALT
- FILL
- CONRETE
- SANDY SILT
- ORGANIC SILT
- SILTY SAND
- SILTY CLAY
- CLAYEY SILT
- SAND AND GRAVEL
- BEDROCK

LEGEND



REFERENCE

BASE PLAN PROVIDED BY PARSONS CORPORATION, DRAWING FILE NAMED B-STR-MAJOR-GA_FOR_GOLDER.DWG, IN AN E-MAIL DATED DECEMBER 8, 2016.

CLIENT
PARSONS CORPORATION

CONSULTANT

Golder Associates

YYYY-MM-DD	2016-12-13
DESIGNED	
PREPARED	MK
REVIEWED	EW
APPROVED	

0 5 10
1:200 VERTICAL SCALE METRES

0 10 20
1:400 HORIZONTAL SCALE METRES

PROJECT
GEOTECHNICAL INVESTIGATION
HAMLET BRIDGE, HAMLET, ONTARIO

TITLE
BOREHOLE LOCATION PLAN AND PROFILE

PROJECT NO.	CONTROL	REV.	FIGURE
1659973		----	2

APPENDIX A

Important Information and Limitations of this Report

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use o f the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

APPENDIX B

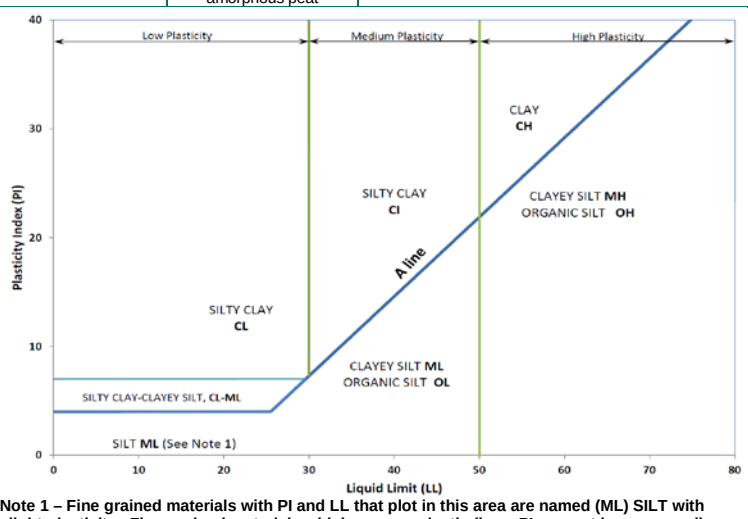
2016 Record of Borehole Sheets

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil		Laboratory Tests	Field Indicators: Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)		Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
					Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
					Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
				Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
					None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)		Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% (see Note 2)	CL	SILTY CLAY
				Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
				Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures								30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat								75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to ~~or~~ indicate~~s~~ a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-1A

BORING DATE: November 24, 2016

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80)	SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊢—○W—⊣ Wl) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		215.10								
0	Track Mounted Power Auger, 210 mm Hollow Stem Augers	ASPHALT		0.08								
		FILL - (SP) gravelly SAND; brown; non-cohesive, moist (GRANULAR BASE)		0.13	1	AS	--					70 mm VSP Casing
		FILL - (SP) SAND, medium, trace plastic fines; brown; non-cohesive, moist, very loose to loose										
1		- No recovery in Sample 2			2	SS	5					
					3	SS	5					
2					4	SS	2			○		
3		- No recovery in Sample 5			5	SS	WH					
4		(OL) ORGANIC SILT, trace sand; dark brown to grey, rootlets; non-cohesive, wet, very loose		211.06 / 4.04								
5					6	SS	1				94.6 OC= 14.1%	Grout
6					7	SS	WH				84.9 OC= 3.4%	
7					V1				>100			
8					8	SS	1			○		
	HQ Casing	- Spoon bouncing at 8.15 m depth		206.95 / 8.15								
		Granite (Bedrock)										
		See Record of Drillhole 16-1 for coring details.										
9		- Bedrock cored from 8.1 m to 12.5 m depth										
10												

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-1A

BORING DATE: November 24, 2016

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊖W WI) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Track Mounted Power Auger, HQ Casing	Granite (Bedrock) See Record of Drillhole 16-1 for coring details.									Grout
11											
12				202.60 12.50							
		END OF BOREHOLE NOTES: 1. Atterberg limit testing on samples 6 and 7 were non-plastic. 2. 50 mm Monitoring well installed in unsampled borehole located on 2.0 m South and 4.0 m East of this location.									
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

PROJECT: 1659973

RECORD OF DRILLHOLE: 16-1A

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Downing

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH	COLOUR % RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA: B Angle	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	PIEZOMETER OR STANDPIPE INSTALLATION
		Continued from Borehole Record 16-1		206.95																		
9	Track Mounted Power Auger, HQ Casing	GRANITE, fresh, strong, very thin to thinly bedded, non-porous, medium grained, crystaline, grey		8.15	1										JN,PL,RO JN,PL,RO JN,PL,RO							UCS=86.7 MPa
10					2																	Grout
11					3																	
12				202.60																		UCS=69.6 MPa
		END OF DRILLHOLE		12.50																		
13																						
14																						
15																						
16																						
17																						
18																						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED:

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-1B

BORING DATE: November 24, 2016

SHEET 1 OF 1

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○; 20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊖W WI; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		215.07							
0	Track Mounted Power Auger, NQ Casing	See Borehole 16-1A for soil stratigraphy.		0.00							50 mm PVC Monitoring Well
1											
2											Bentonite
3											
4											
5											
6											Sand
7											
8											
9				205.90							Bentonite
		END OF BOREHOLE		9.17							
10											

NOTES:

1. Groundwater measured in monitoring well at a depth of 1.86 m below ground surface, December 13, 2016.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-2

BORING DATE: November 23, 2016

SHEET 1 OF 1

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊖W Wl) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		212.58							▽
0	Track Mount Power Auger, NQ Casing	WATER		0.00							
				208.64							
4		FILL - lake debris, shell fragments, leaves - No recovery in Sample 1		3.94 208.14	1	SS	51/ 0.23				
		Granite (Bedrock) See Record of Drillhole 16-2 for coring details. - Bedrock cored from 4.1 m to 7.6 m depth		4.44 204.95							
		END OF BOREHOLE		7.63							

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

PROJECT: 1659973

RECORD OF DRILLHOLE: 16-2

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Downing

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH	COLOUR % RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA: B Angle	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	PIEZOMETER OR STANDPIPE INSTALLATION
		Continued from Borehole Record 16-2		208.14																		
5, 6, 7	Track Mount Power Auger; NQ Casing	GRANITE, fresh, strong, very thin to thinly bedded, non-porous, medium grained, crysaline, grey and pink		4.44	1																	
					2										JN,PL,RO; JN,PL,RO							UCS=95.4 MPa
				204.95																		
		END OF DRILLHOLE		7.63																		
8, 9, 10, 11, 12, 13, 14																						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED:

GTA-RCK 004 S:\CLIENTS\PUBLIC_WORKS_CANADA\HAMLET_BRIDGE\02_DATA\GINT\HAMLET_BRIDGE.GPJ GAL-MISS.GDT 1/17/18

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-3

BORING DATE: November 23, 2016

SHEET 1 OF 1

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊖W WI) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		212.58							▽
0	Track Mount Power Auger, NQ Casing	WATER		0.00							
1											
2											
3											
				208.69							
4		FILL - No recovery in Sample 1		3.89	1	AS	--				
		Granite (Bedrock) See Record of Drillhole 16-3 for coring details. - Bedrock cored from 4.1 m to 7.4 m depth		4.06							
5											
6											
7				205.22							
		END OF BOREHOLE		7.36							
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

PROJECT: 1659973

RECORD OF DRILLHOLE: 16-3

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Downing

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA B Angle	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	PIEZOMETER OR STANDPIPE INSTALLATION
		Continued from Borehole Record 16-3		208.52																		
5	Track Mount Power Auger, NQ Casing	GRANITE, fresh, strong, very thin to thinly bedded, non-porous, medium to large grained, crystaline, grey and pink		4.06	1																	
5					2										JN,PL,SM							UCS=82.6 MPa
6					3																	
7				205.22																		
		END OF DRILLHOLE		7.36																		
8																						
9																						
10																						
11																						
12																						
13																						
14																						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED:

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-4

BORING DATE: November 25, 2016

SHEET 1 OF 1

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊢—○W—⊣ Wl) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		213.13							
0	Track Mount Power Auger, 210 mm Hollow Stem	ASPHALT (20mm)		0.00	1	AS	--		○	MH	
		FILL - (GP-GM) GRAVEL and SAND, some silt; brown; non-cohesive, dry (GRANULAR BASE)		0.10							
1		FILL - (SP) SAND, medium, trace plastic fines; brown; non-cohesive, dry			2	AS	--		○		
				211.61							
		END OF BOREHOLE		1.52							
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

PROJECT: 1659973

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: 16-5

BORING DATE: November 25, 2016

SHEET 1 OF 1

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80)	SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3})	WATER CONTENT PERCENT (Wp ⊖W WI) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		215.17									
0	Track Mount Power Auger, 210 mm Hollow Stem	ASPHALT (80mm)		0.00									
		FILL - (SP) gravelly SAND; brown; non-cohesive, dry (GRANULAR BASE)		0.08 / 214.76	1	AS	--						
1		FILL - (ML) sandy SILT; brown, organic staining; non-cohesive, wet		0.41 / 213.65	2	AS	--						
		END OF BOREHOLE		1.52									
2													
3													
4													
5													
6													
7													
8													
9													
10													

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MB

CHECKED: SEMP

APPENDIX C

2011 Record of Borehole Sheets and Photographs

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P plasticity Index = $(w_l - w_p)$
w_S shrinkage limit
I_L liquidity index = $(w - w_p)/I_p$
I_C consistency index = $(w_l - w)/I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = $\tan\delta$
c' effective cohesion
c_u, S_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan\phi'$
2. Shear strength = (Compressive strength)/2

PROJECT: 11-1111-0118

RECORD OF BOREHOLE: BH 1

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE							
0	TRACK MOUNTED POWER AUGER AUTOMATIC HAMMER, 150 mm Diameter Solid Stem Augers	ASPHALT (80 mm)		0.00					
		GRANULAR BASE		0.08	1A	50 DO	11	MH	
		Compact dark brown silty sand, trace to some gravel (FILL)		0.35	1B				
1		Loose dark brown and brown mottled fine sandy silt, trace clay, trace fine sand, trace organic (FILL)		0.76	2	50 DO	4		
		Loose dark brown organic silt, some sand, trace clay (FILL)		1.37	3	50 DO	3		
2		Firm brown and grey mottled CLAYEY SILT to SILTY CLAY, some sand		2.13	4	50 DO	5		
3					5	50 DO	5		
4		Very loose dark grey ORGANIC SILT, trace to some clay, trace sand, trace rootlets		4.04					▽
5					6	50 DO	W/H		
		Very loose dark grey SILTY FINE SAND, trace clay		5.56					
6					7	50 DO	4		
7		Very dense grey SILTY SAND and GRAVEL, organic stains, containing cobbles and boulders		7.09					
					8	50 DO	50/.1		
8		END OF BOREHOLE DUE TO AUGER REFUSAL ON BIOTITE GNEISS BECROCK		7.87					Water encountered during drilling at a depth of 4.5 m below ground surface, Sept. 28/11 Water level in open borehole at a depth of 4.1 m below ground surface upon completion of drilling, Sept. 28, 2011
9									
10									

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ●; rem V. ⊕ U - ○; 20, 40, 60, 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 10, 20, 30, 40

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

GTA-BHS 001 11-1111-0118.GPJ GAL-MIS.GDT 11/9/11 MK Sept. 2011

PROJECT: 11-1111-0118

RECORD OF BOREHOLE: BH 2

SHEET 1 OF 2

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊢—○ W —⊣ Wl) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE									
0	TRUCK MOUNTED POWER AUGER, 150 mm Diameter Solid Stem Augers	ASPHALT (85 mm)		0.00							
		GRANULAR BASE (Slight hydrocarbon-like odour)		0.09							
		Brown silty sand and gravel (Slight hydrocarbon-like odour) (FILL)		0.40	1	AS	-		○		
1		Very loose brown fine sandy silt, trace gravel (FILL)		0.76	2	50 DO	4		○	MH	
		Loose dark brown organic silt, some sand, trace clay, trace roots (FILL)		1.37	3	50 DO	5		○	MH	Bentonite Seal
2					4A	50 DO	11/.1		○		
		Weathered concrete (PROBABLE ABUTMENT FOOTING)		2.40	4B						
3					5	50 DO	50/.03		○		
		Very soft to soft grey SILTY CLAY to CLAYEY SILT, trace sand, organic stains		3.40							
4					6	50 DO	2		○	MH	
								⊕ +			Silica Sand Filter
5					7	50 DO	-	⊕ +	○		
		Very loose dark grey FINE SANDY SILT, trace clay, organic stains, trace decayed root fragments		5.26	8A	50 DO	1		○		
		Very loose grey SILTY FINE SAND, organic stains		5.74	8B				○		
6											
		Compact grey SAND, some gravel, some silt, organic stains		6.40							
7					9	50 DO	13		○	MH	
											Bentonite Seal
8		Continue with coring Fresh to slightly weathered dark grey to black fine to medium grained BIOTITE GNEISS BEDROCK		7.87							
9											
		END OF BOREHOLE T.C.R. (Total Core Recovery) = 98% S.C.R. (Solid Core Recovery) = 97%		9.40							Water encountered during drilling at a depth of 4.5 m below ground surface, Sept. 28/11
10											

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

PROJECT: 11-1111-0118

RECORD OF BOREHOLE: BH 2

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊖W Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		R.Q.D. (Rock Quality Designation) = 93%									Water measured in piezometer at a depth of 1.9 mbelwo ground surface upon completion of drilling, Oct. 22/11
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

PROJECT: 11-1111-0118

RECORD OF BOREHOLE: BH 3

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT (Wp ⊖W Wl) (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE									
0	Coring	CONCRETE (NORTH ABUTMENT)		0.00							
1											
2											
3											
		Brown to grey CLAYEY SILT, some sand, organic stains, trace rootlets, zones of silty fine sand		3.48	1		-		⊖	MH	
					2		-		⊖	MH	
4		END OF BOREHOLE		4.04							
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

SITE PHOTOGRAPHS

Figure 13A

No.1: Overview of the Hamlet Bridge from north side of bridge, looking south; the east side (left on photo) is the fixed bridge; the west side (right on photo) is swing bridge.

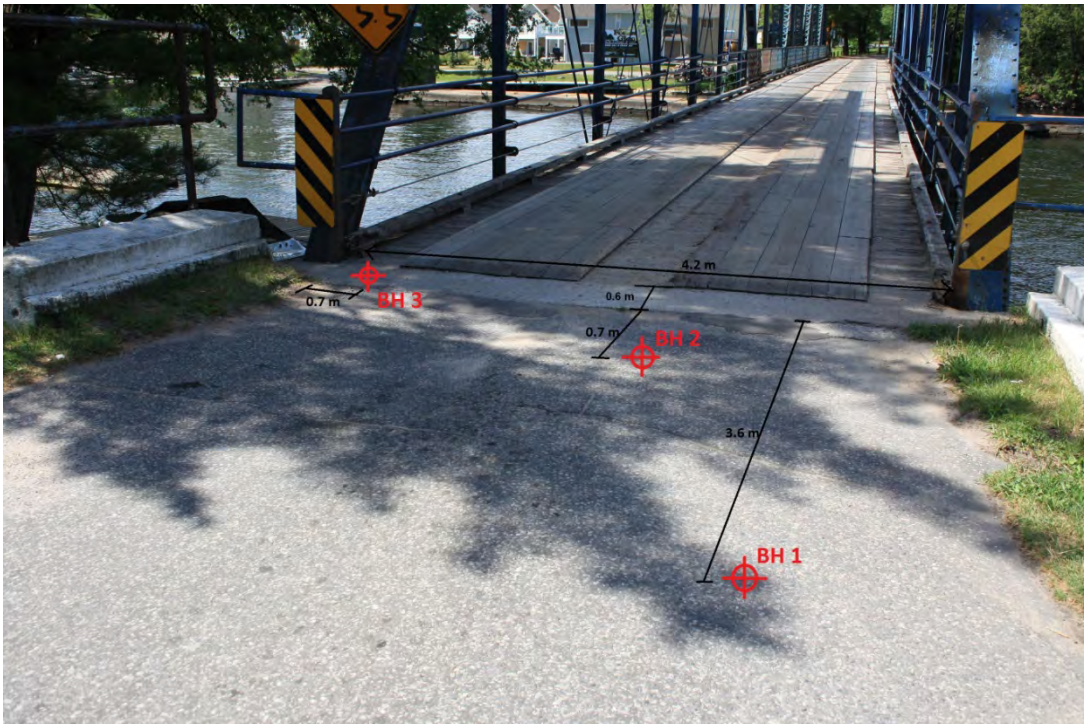
No.2: Overview of the Hamlet Bridge, looking west; the closest pier is the east pier which supports the west end of fixed bridge and east end of the swing bridge.

.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13B



No.3: Overview of borehole locations and approximate measurements at east abutment, looking west.

No.4: Test pit location at south side of south wing wall at east abutment and approximate measurements, looking north.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13C

No.5: North wing wall of east abutment, looking south; the linear cracks appear to be construction joints.

No.6: The north abutment concrete wall under the bridge; note the sheet pile wall installed for erosion protection; note the concrete platform behind the sheet pile walls.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13D

No.7: The north abutment concrete wall under the bridge; the linear crack appears to be the construction joint.

No.8: The concrete platform between the sheet piles and abutment wall; the core hole was terminated in the concrete due to refusal on steel rebar at a depth of about 0.5 m.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13E

No.9: Rock cores in Borehole 2

No.10: Concrete cores in vertical corehole at Borehole 3 location.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13F

No.11 Concrete cores in vertical corehole at Borehole 3 location.

No.12: Concrete cores in vertical corehole at Borehole 3 location.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13G

No.13: The cores of the horizontal corehole on the abutment wall.

No.14: The cores of the horizontal corehole on the abutment wall.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13H

No.15: Looking into the horizontal concrete corehole on the abutment wall.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

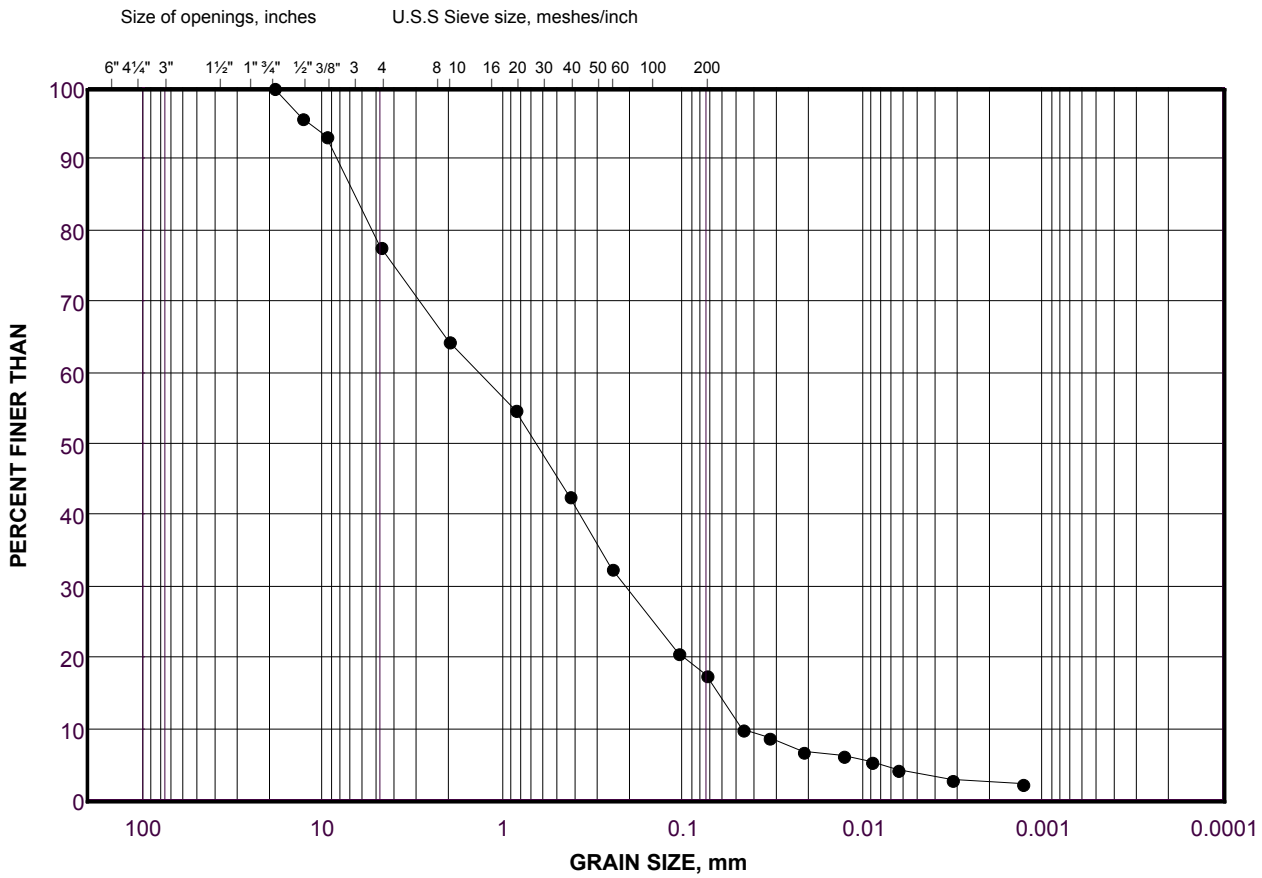
APPENDIX D

Laboratory Testing Results

GRAIN SIZE DISTRIBUTION

SILTY SAND (FILL)

FIGURE 3



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	1	1B	0.35 - 0.76

Project Number: 11-1111-0118

Checked By: DBL

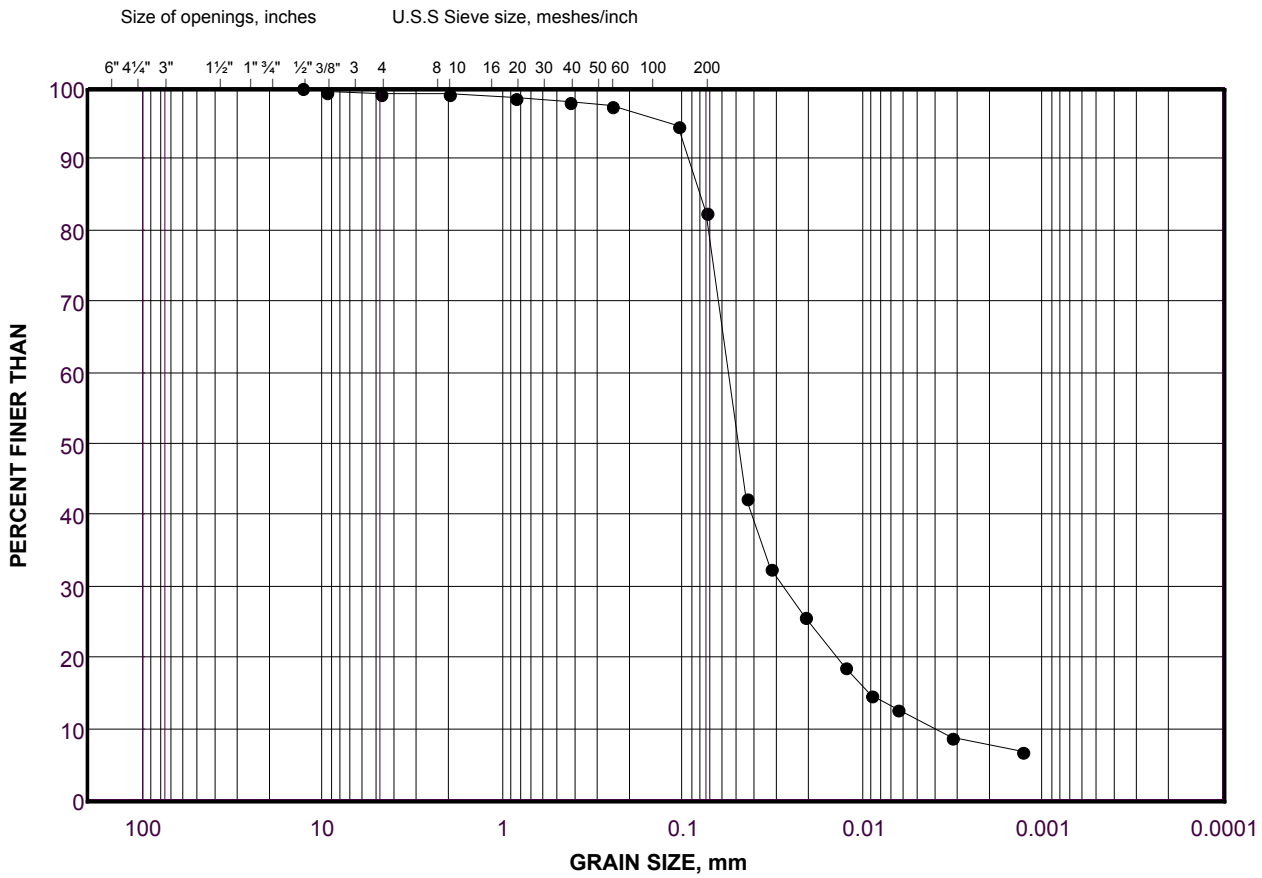
Golder Associates

Date: 28-Oct-11

GRAIN SIZE DISTRIBUTION

SANDY SILT (FILL)

FIGURE 4



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	2	0.76 - 1.22

Project Number: 11-1111-0118

Checked By: DBL

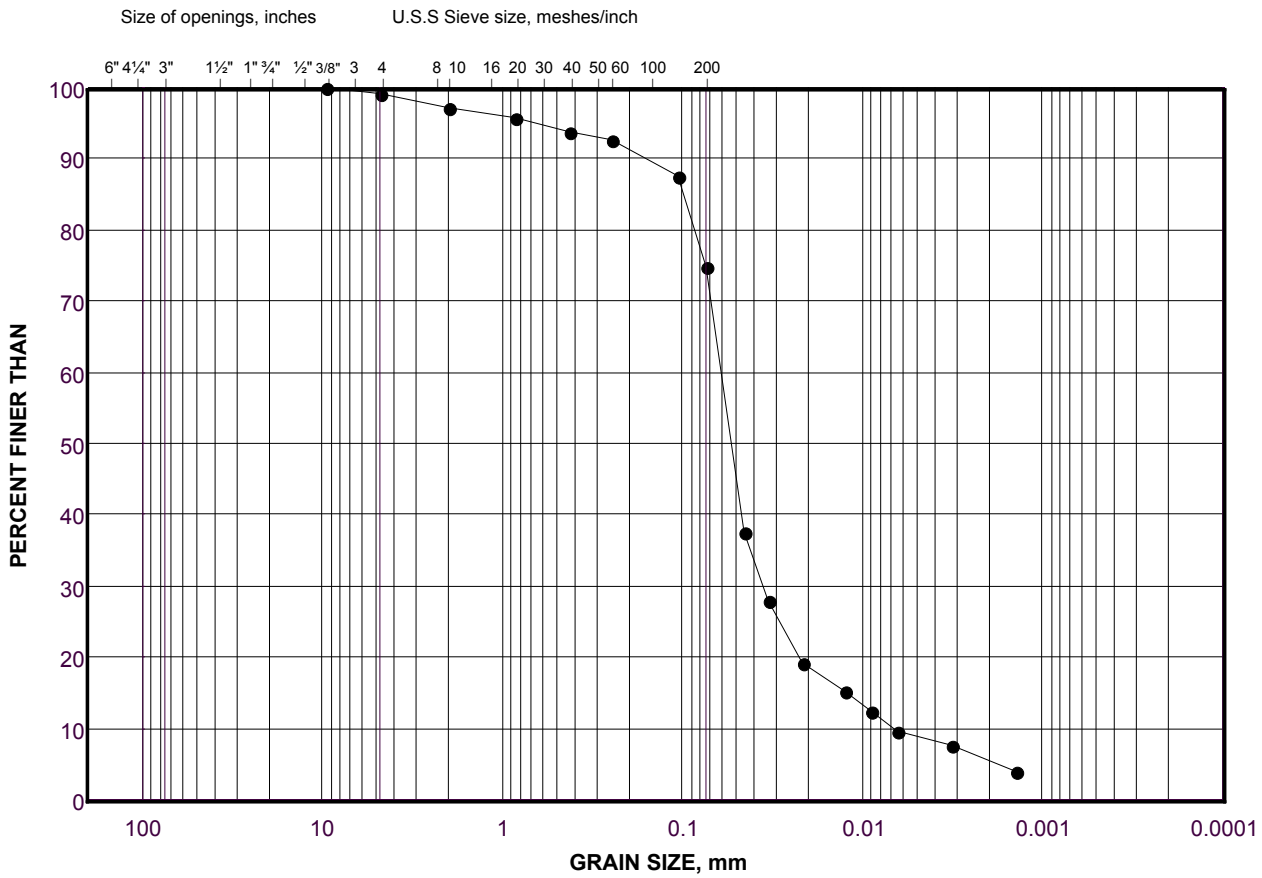
Golder Associates

Date: 28-Oct-11

GRAIN SIZE DISTRIBUTION

ORGANIC SILT (FILL)

FIGURE 5



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	3	1.52 - 1.98

Project Number: 11-1111-0118

Checked By: DBL

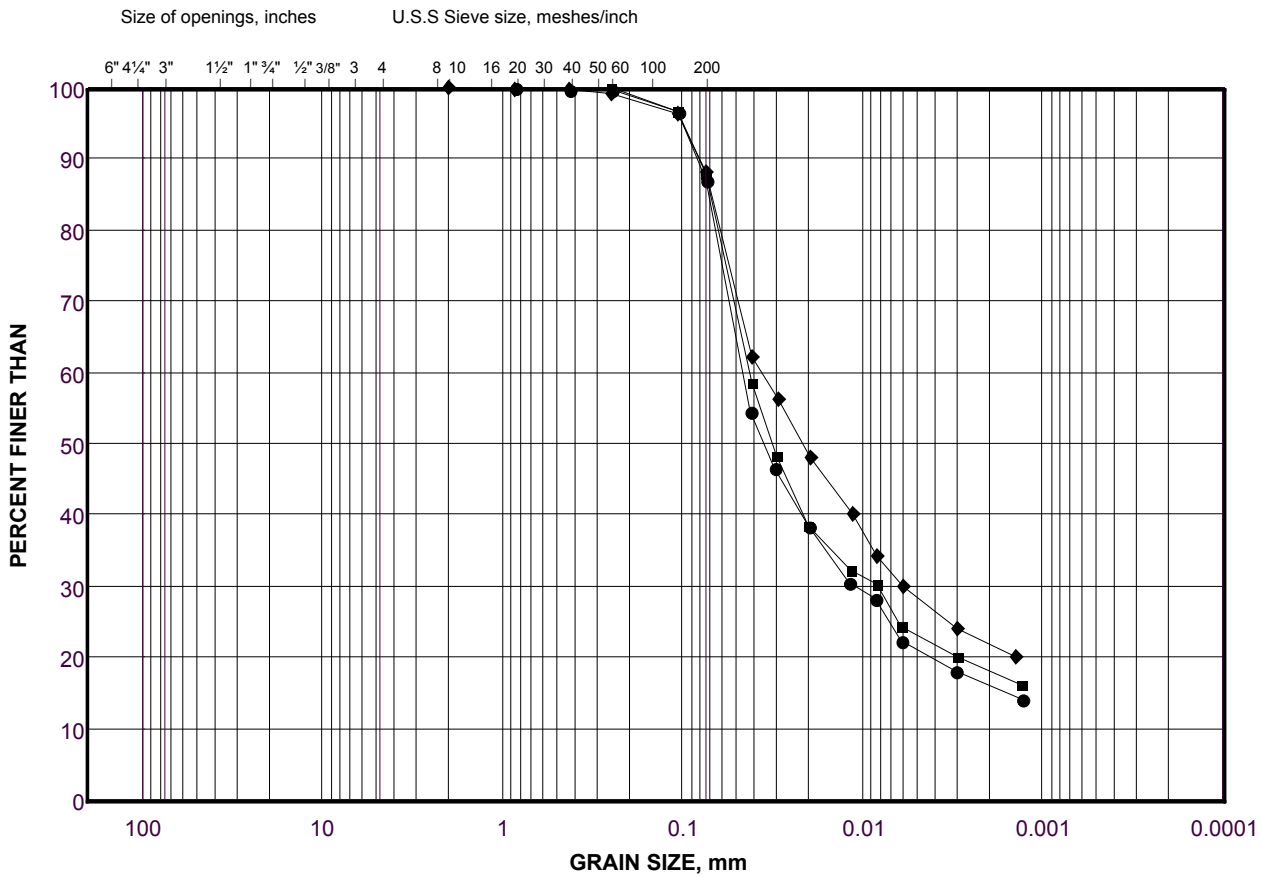
Golder Associates

Date: 07-Nov-11

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

FIGURE 6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	3	1	3.48 - 3.73
■	3	2	3.73 - 4.04
◆	2	6	4.57 - 5.03

Project Number: 11-1111-0118

Checked By: DBL

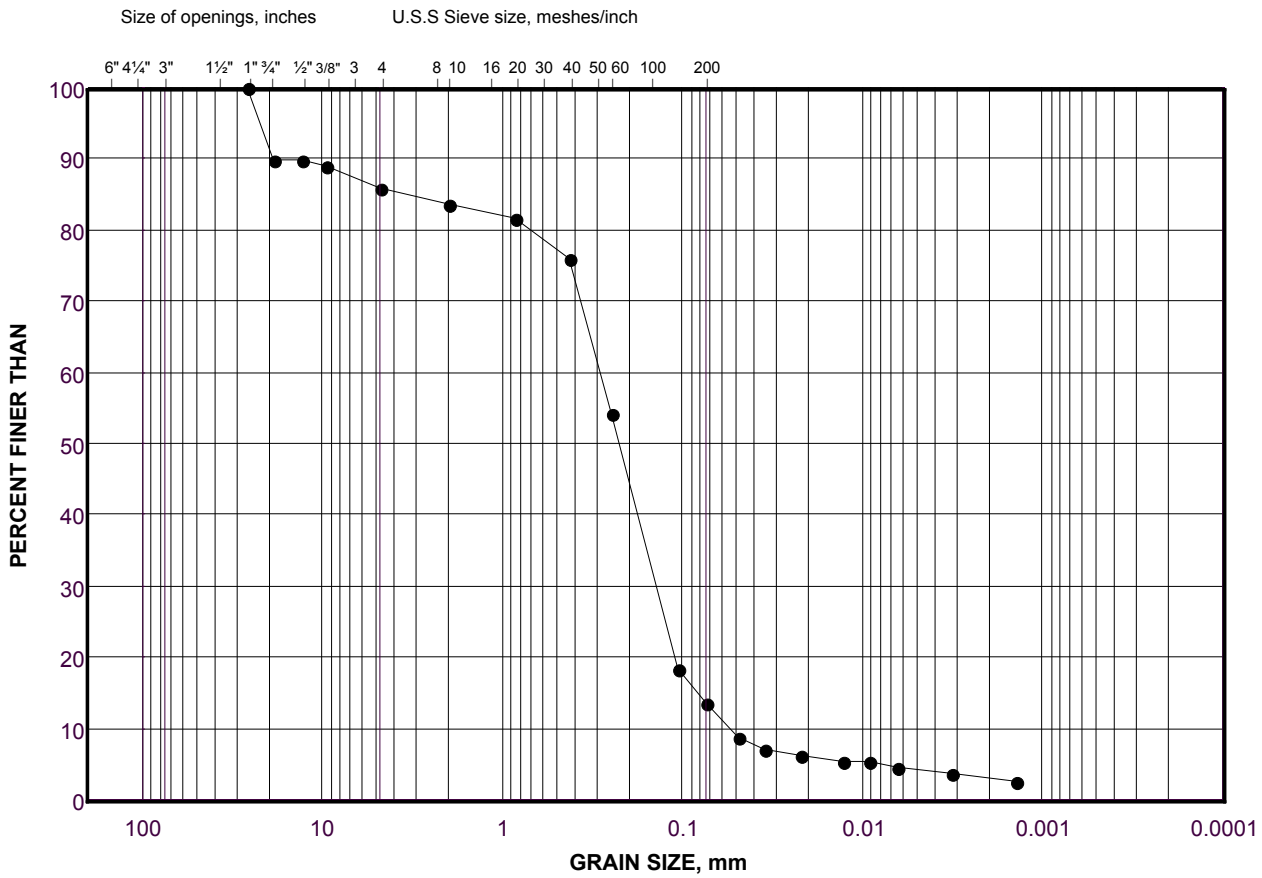
Golder Associates

Date: 07-Nov-11

GRAIN SIZE DISTRIBUTION

SAND

FIGURE 7



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

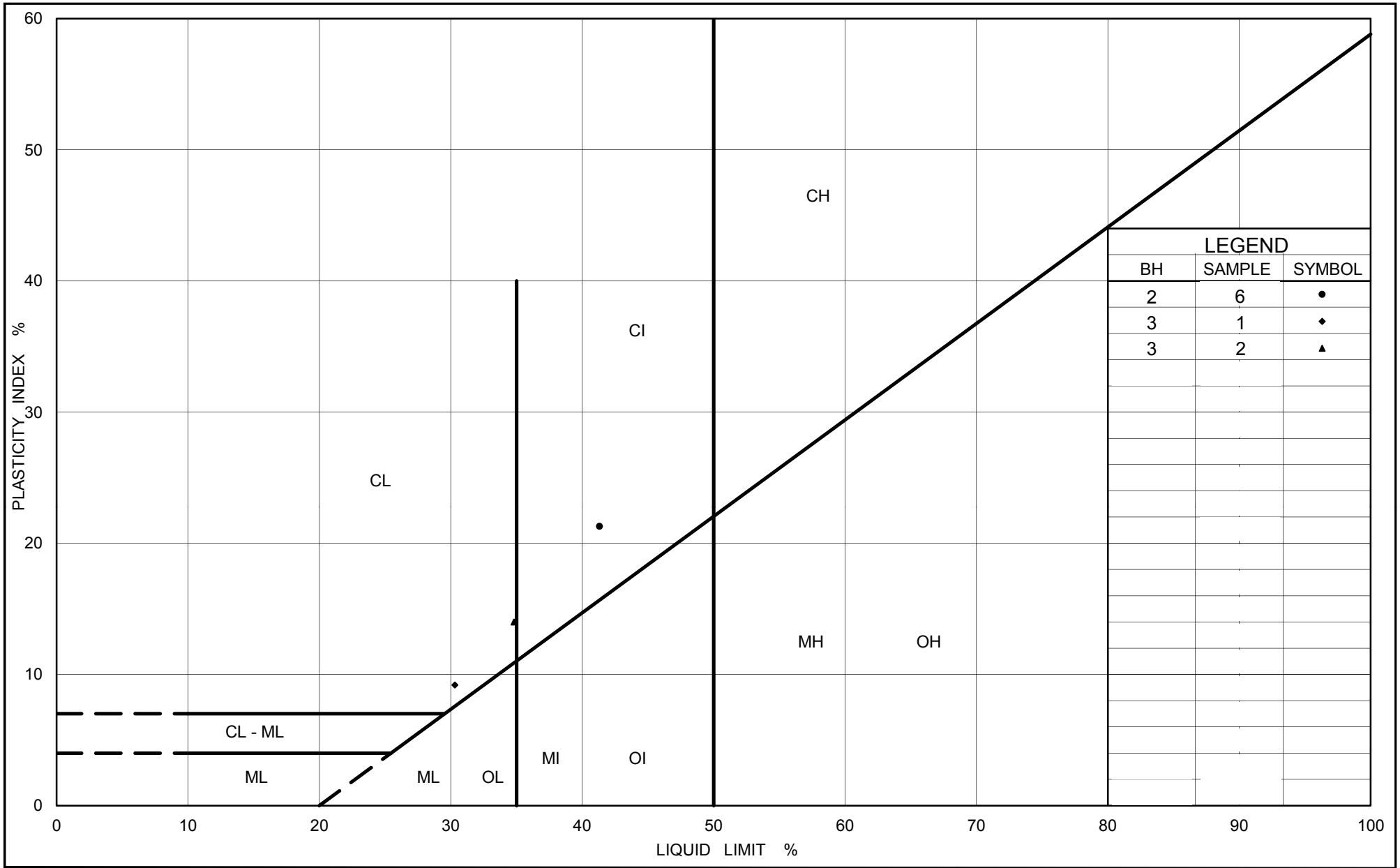
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	9	7.62 - 8.08

Project Number: 11-1111-0118

Checked By: DBL

Golder Associates

Date: 07-Nov-11



Golder Associates

PLASTICITY CHART

Figure No.	8
Project No.	11-1111-0118
Checked By:	DBL

OBTAINING AND TESTING DRILLED CORES FOR COMPRESSIVE STRENGTH TEST
(CSA A23.2-14C)

October 24, 2011

Golder Project Number: 11-1111-0118 **Figure 9**

Golder Associates Ltd.
100 Scotia Court
Whitby, ON L1N 8Y6

ATTENTION: Mr. David Lui

PROJECT: **Hamlet Bridge**

Date Received: October 13, 2011 **Date Tested:** October 18, 2011

Core Number:	**C2**	**C2**	**BH2 - Bedrock**
Depth:	1'4" – 3'5"	9'5" – 10'1"	26'4" – 27'4"
Golder Lab Number:	C-11-1355	C-11-1356	C-11-1357
Moisture Condition at Time of Test	Wet	Wet	Wet
Capping Material	Sulphur	Sulphur	Sulphur
Average Diameter, (mm)	94.1	94.2	93.0
Average Length (mm)	172.2	145.4	1.98
Density, (Mg/m^3)	2.255	2.165	2.434
Load, (kN)	130.24	102.47	237.07
Compressive Strength, (MPa)	18.7	14.7	136.7
Corrected Compressive Strength, (MPa)	**18.4**	**14.2**	**136.4**

Reviewed by: _____
Annmarie Jarvis, Laboratory Supervisor

Notice: The test data given herein pertain to the sample provided, and may not be applicable to material from other production zones/periods. This report constitutes a testing service only. Interpretation of the data given here may be provided upon request.

Figure 10A

UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0118	SAMPLE NUMBER	2
BOREHOLE NUMBER	3	SAMPLE DEPTH, m	3.73-4.04

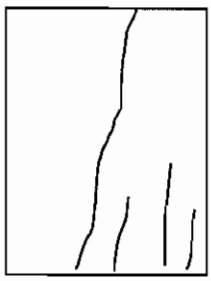
TEST CONDITIONS

MACHINE SPEED, mm/min	0.76	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	0.75	L/D	2.02

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.10	WATER CONTENT, (specimen) %	25.53
SAMPLE DIAMETER, cm	4.99	UNIT WEIGHT, kN/m^3	19.56
SAMPLE AREA, cm^2	19.58	DRY UNIT WT., kN/m^3	15.58
SAMPLE VOLUME, cm^3	197.76	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	394.58	VOID RATIO	0.70
DRY WEIGHT, g	314.33		

FAILURE SKETCH



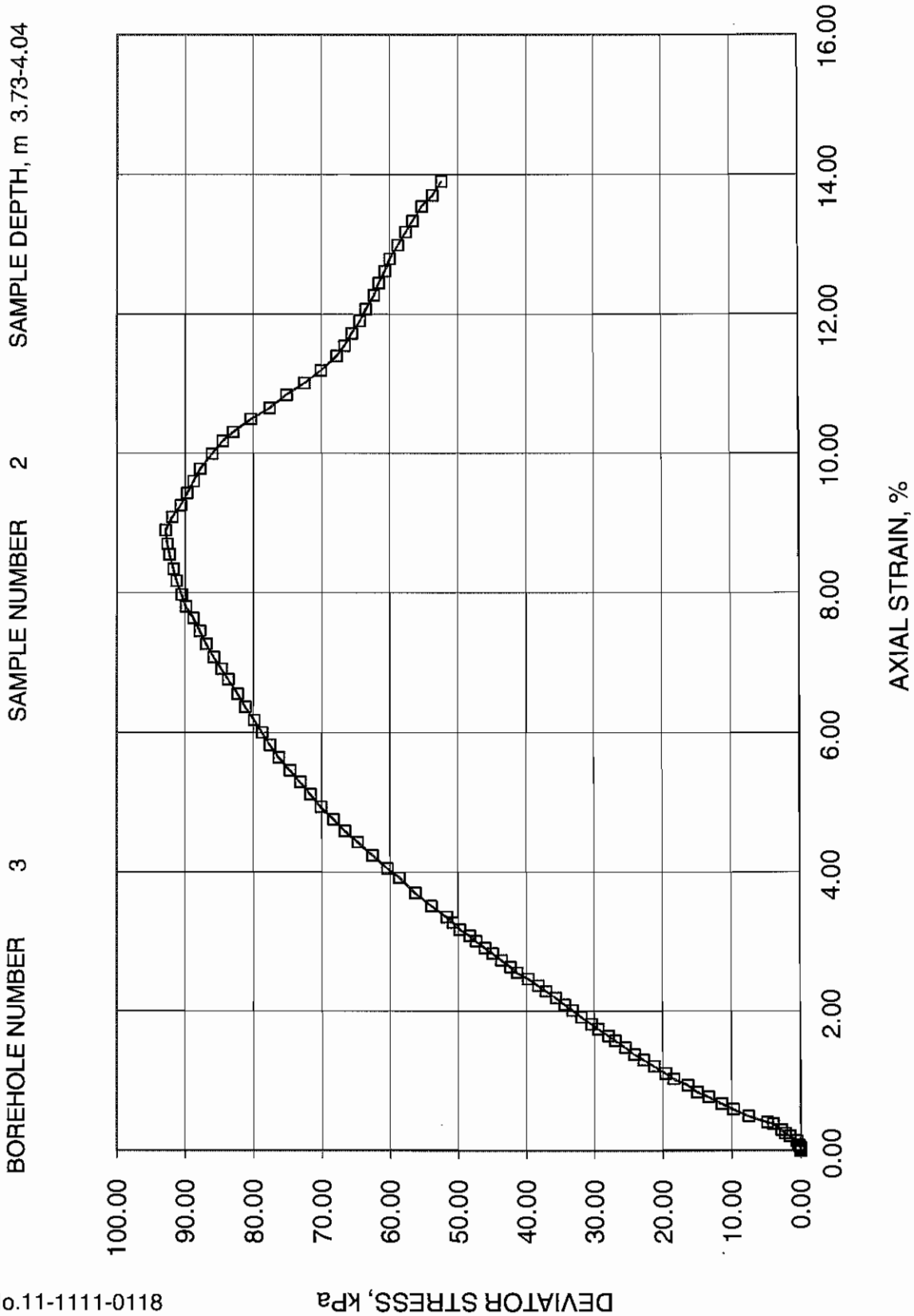
TEST RESULTS

STRAIN AT FAILURE, %	8.9	COMPRESSIVE STRESS, kPa	93

REMARKS:

DATE: 10/12/2011

UNCONFINED COMPRESSION TEST (UC)	FIGURE 10B



Project No.11-1111-0118

DENSITY AND POROSITY DETERMINATIONS OF IRREGULAR SHAPE SAMPLES

ASTM D 7263 - 09 Method A

Borehole Number	3 (A)	3 (B)
Sample Number	1	1
Depth, m	3.5-3.8	3.5-3.8
Wet Mass of Soil in Air, g	433.25	455.90
Wet Mass of Soil + Wax in Air, g	456.30	476.60
Wet Mass of Soil + Wax in Water, g	210.90	222.40
Weight of Wax, g	23.05	20.70
Displaced Volume, cm^3	245.40	254.20
Displaced Wax, cm^3	25.39	22.80
Volume of Soil, cm^3	220.01	231.40
Specific Gravity, assumed	2.70	2.70
Volume of Solids, cm^3	125.26	132.23
Volume of Voids, cm^3	94.75	99.18
Porosity	0.43	0.43
Water Content, %	28.10	27.70
Unit Weight, kN/m^3	19.31	19.32
Dry Unit Weight, kN/m^3	15.08	15.13

Project Number	11-1111-0118	Tested By	Larry
Date Tested	10/7/2011	Checked By	

Figure 11B

DENSITY (UNIT WEIGHT) OF SOIL SPECIMENS

ASTM D 7263 Method B

Borehole Number	2
Sample Number	7
Sample Depth, m	4.6-5.2
Weight of Soil + Tube, g	179.77
Weight of Tube, g	76.43
Weight of Soil, g	103.34
Diameter of Sample, cm	6.33
Length of Sample, cm	1.90
Volume of Sample, cc	59.71
Water Content, %	48.1
Wet Density, Mg/m^3	1.731
Dry Density, Mg/m^3	1.169
Unit Weight, kN/m^3	16.97
Borehole Number	
Sample Number	
Sample Depth, m	
Weight of Soil + Tube, g	
Weight of Tube, g	
Weight of Soil, g	
Diameter of Sample, cm	
Length of Sample, cm	
Volume of Sample, cc	
Water Content, %	
Wet Density, Mg/m^3	
Dry Density, Mg/m^3	
Unit Weight, kN/m^3	

Project Number	11-1111-0118	Tested By	Lina
Date Tested	10/25/2011	Checked By	MM

ORGANIC CONTENT (BURNING METHOD)

BOREHOLE NUMBER		2					
SAMPLE NUMBER		3					
CRUCIBLE NUMBER			8				
WEIGHT OF CRUCIBLE, g	W1	28.95	28.84				
WEIGHT OF CRUCIBLE & AIR DRY SAMPLE, g	W2	57.67	55.49				
WEIGHT OF AIR DRY SAMPLE (ORIGINAL), g	W2-W1	28.72	26.65				
WEIGHT AFTER BURNING SOIL & CRUCIBLE, g	W3	56.94	54.78				
WEIGHT OF ORGANICS, g	W2-W3	0.73	0.71				
PERCENT OF ORGANICS, % ((W2-W3)/(W2-W1))x100		2.54	2.66				
ORGANIC CONTENT, %		**2.6**					

PROJECT NUMBER	11-1111-0118	DATE OF TESTING	10/25/2011
TESTED BY	Renato / Lina	CHECKED BY	

Notes:

1. Samples dried at 110 degree centigrade prior to testing.

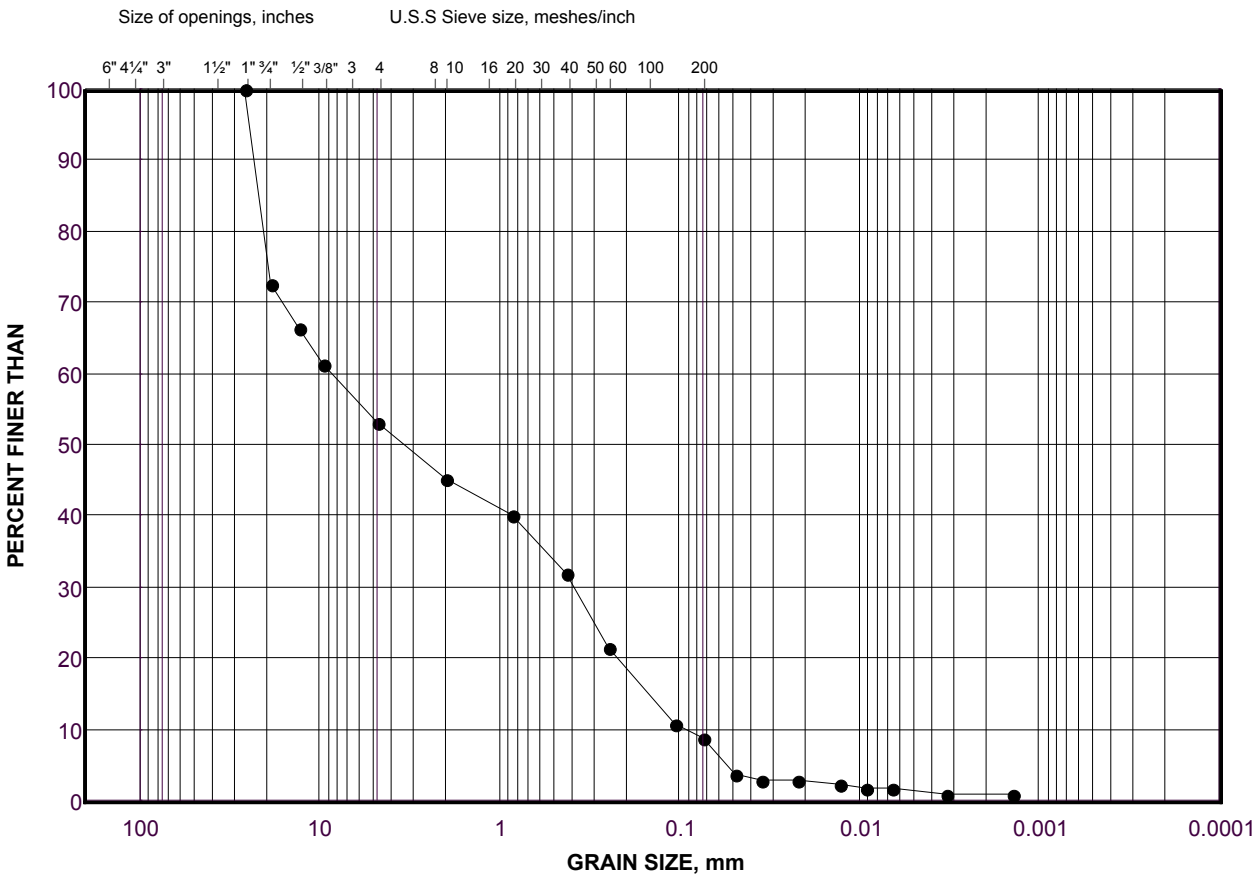
2. Test performed according to ASTM D2974 Standard, test method C.

3. Organic matter determined by burning the oven dried samples in a muffle furnace at 440 degree centigrade.

GRAIN SIZE DISTRIBUTION

(GP-GM) GRAVEL and SAND

FIGURE 13



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	16-4	1	0.0 - 0.10

Project Number: 1659973

Checked By: _____

Golder Associates

Date: 17-Jan-18

Borehole 16-1

Box 1: 8.2 m – 10.7 m

Borehole 16-1

Box 2: 10.7 m – 12.5 m

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT

Condition Survey and Geotechnical Investigation of the Hamlet Swing Bridge
Public Works and Government Services Canada

TITLE

Bedrock Core Photographs
Borehole 16-1

Golder Associates

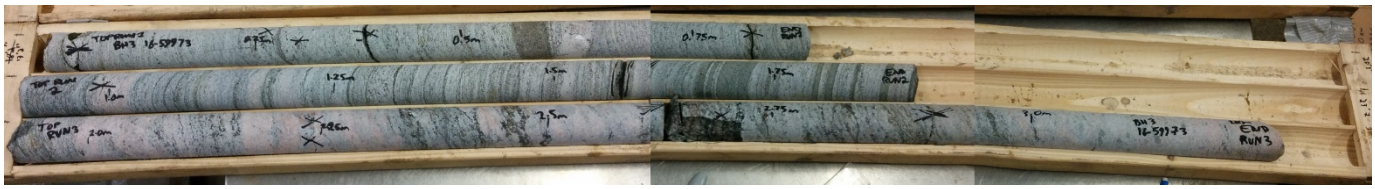
PROJECT No. 1659973			FILE No. ----		
DESIGN	EW	Dec 11	SCALE	NTS	REV.
CADD	-- --		FIGURE D1		
CHECK					
REVIEW	--				

REVISION DATE: October 14, 2016 BY: MT Project: 1531057

Borehole 16-2

Box 1: 4.4 m – 7.6 m

Borehole 16-3



Box 1: 4.1 – 7.4 m

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT

Condition Survey and Geotechnical Investigation of the Hamlet Swing Bridge
Public Works and Government Services Canada

TITLE

Bedrock Core Photographs
Borehole 16-2 and 16-3

Golder Associates

PROJECT No. 1659973			FILE No. ----	
DESIGN	EW	Dec 11	SCALE NTS	REV.
CADD	-- --		**FIGURE D2**	
CHECK				
REVIEW	--			

REVISION DATE: October 14, 2016 BY: MT Project: 1531057

APPENDIX E

Vertical Seismic Profiling Test Results

TECHNICAL MEMORANDUM

DATE December 15, 2016 **PROJECT No.** 1664670/1000

TO Michael Navarra, Sarah Poot
Golder Associates Ltd.

FROM Stephane Sol, P.Geo; Christopher Phillips, P.Geo **EMAIL** ssol@golder.com, cphillips@golder.com

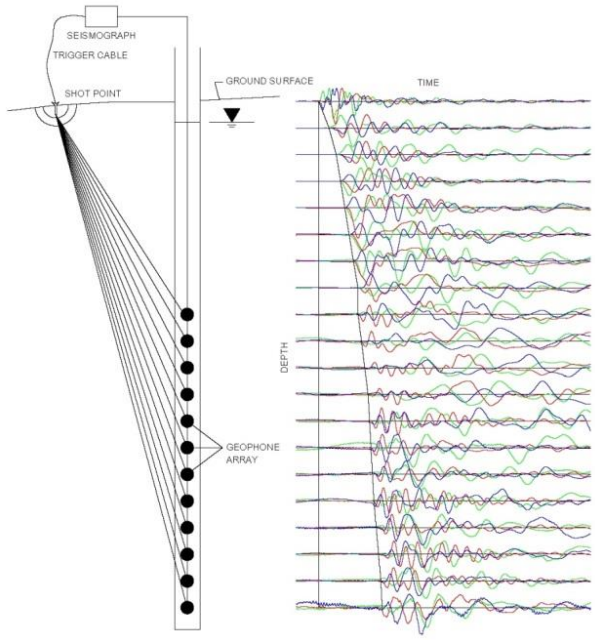
VERTICAL SEISMIC PROFILING TEST RESULTS
HAMLET SWING BRIDGE, HAMLET, ONTARIO

This memorandum presents the results of the Vertical Seismic Profiling (VSP) testing carried out at the Hamlet Swing Bridge in Hamlet, Ontario. VSP testing was completed in Borehole BH16-1A. VSP testing was carried out on December 13, 2016. BH16-1A was drilled to an approximate depth of 12.5 m below the existing ground surface and then cased with a 63.5 mm diameter PVC pipe grouted in place. The borehole consisted of 8.15 m of overburden overlaying granite bedrock. The overburden consisted of a small layer of asphalt overlaying approximately 4 m of non-cohesive moist sand fill and 4.1 m of very soft silty clay.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the 2015 National Building Code of Canada.



Example 1: Layout and resulting time traces from a VSP survey.

Fieldwork

The fieldwork was carried out on December 13, 2016, by personnel from the Golder Mississauga office.

Both compression and shear wave seismic sources were used and both were located 2 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kg sledge hammer vertically impacted on a metal plate. The seismic source for the shear wave test consisted of a metallic U-shaped object that was horizontally struck with a 9.9 kg sledge hammer on alternate ends to induce polarized shear waves. Test measurements started at 0.5 m from ground surface and were recorded in the borehole with a 3-component receiver spaced at 0.5 m intervals below the ground surface to a maximum depth (11.9 m).

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;

2) Low Pass Filtering of data to remove spurious high frequency noise;

3) First break picking of the compression and shear wave arrivals; and,

4) Calculation of the average compression and shear wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following four plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths for Borehole BH16-1A. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

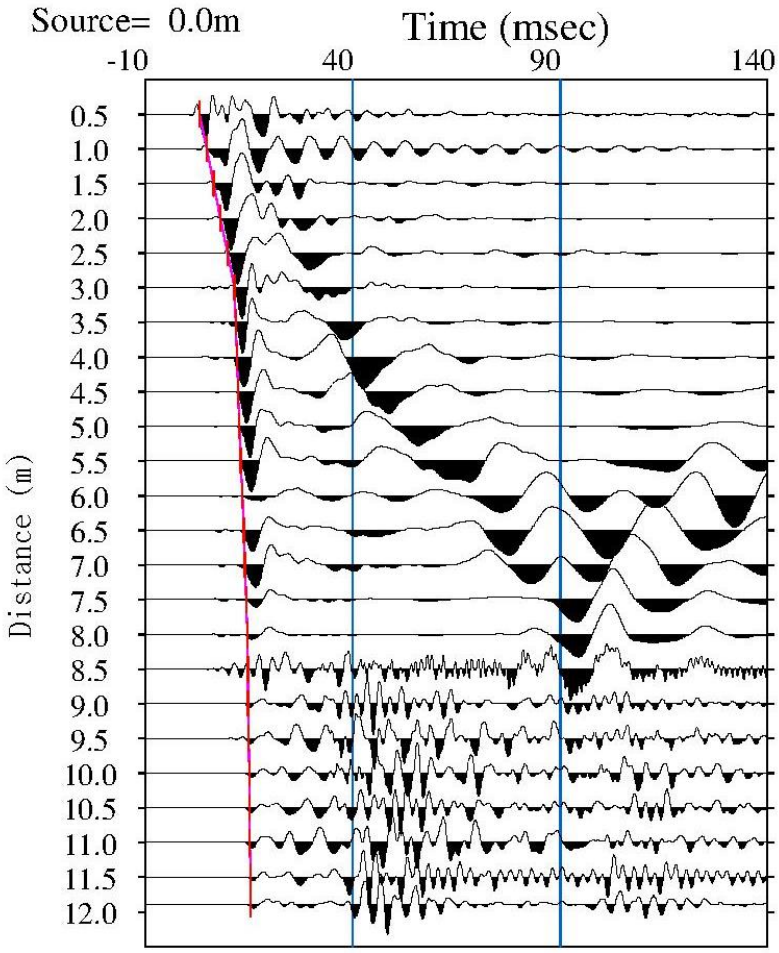


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-1A.

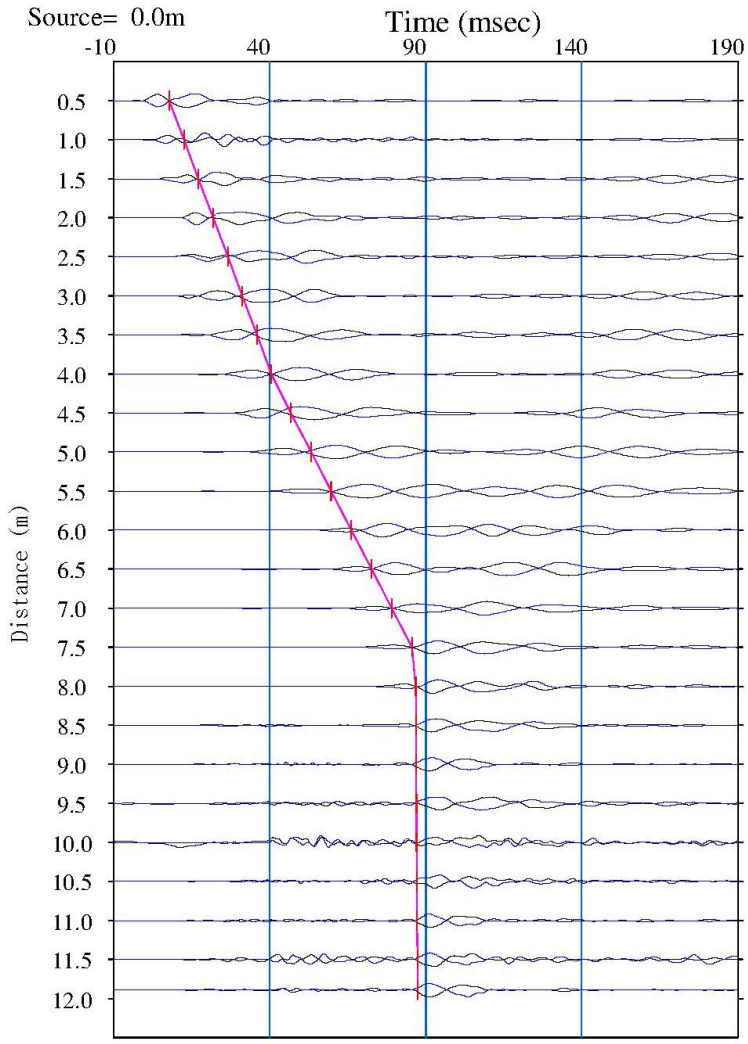


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-1A.

Results

The VSP results are summarized in Table 1. The compression and shear wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. For the top 8 m of overburden, the bulk density of 1,600 kg/m^3 was used. For the bedrock down to the bottom of the VSP test at 11.9 m, a bulk density of 2,600 kg per cubic metre was used.

The average shear wave velocity from ground surface to a depth of 30 m (Vs30) was measured to be 320 m/s. The average velocity was calculated assuming that the velocity from 11.9 m to a depth of 30 m was constant with an average shear wave velocity value of 2,400 m/s which is equal to the velocity of the

granite bedrock at the bottom of the borehole. If the foundations were located 2 m below the existing ground surface, Vs30 would be equal to 377 m/s. If the foundations were location 8.2 m below the existing ground surface (directly on bedrock), Vs30 would be equal to 2,300 m/s.

Limitations

This technical memorandum is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.

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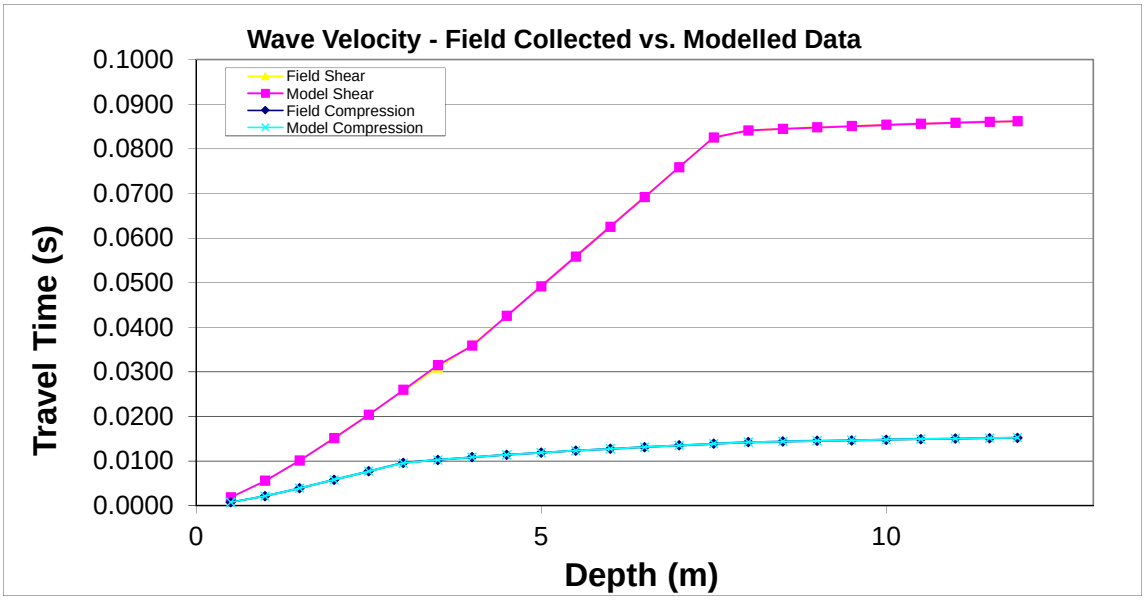
Attachment: Table 1 – Shear Wave Velocity Profile at BH-16-1A

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT 16-1A

Layer Depth (m)		Velocity (m/s)		Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	0.5	650	270	1600	0.40	117	326	520
0.5	1	360	135	1600	0.42	29	83	168
1.0	1.5	290	110	1600	0.42	19	55	109
1.5	2	265	100	1600	0.42	16	45	91
2.0	2.5	260	95	1600	0.42	14	41	89
2.5	3	270	90	1600	0.44	13	37	99
3.0	3.5	700	90	1600	0.49	13	39	767
3.5	4	840	115	1600	0.49	21	63	1101
4.0	4.5	950	75	1600	0.50	9	27	1432
4.5	5	1050	75	1600	0.50	9	27	1752
5.0	5.5	1100	75	1600	0.50	9	27	1924
5.5	6	1220	75	1600	0.50	9	27	2369
6.0	6.5	1250	75	1600	0.50	9	27	2488
6.5	7	1300	75	1600	0.50	9	27	2692
7.0	7.5	1350	75	1600	0.50	9	27	2904
7.5	8	1400	310	1600	0.47	154	453	2931
8.0	8.5	3600	1450	2600	0.40	5467	15341	26407
8.5	9	3600	1600	2600	0.38	6656	18330	24821
9.0	9.5	3600	1750	2600	0.35	7963	21424	23079
9.5	10	4100	1750	2600	0.39	7963	22114	33089
10.0	10.5	4100	2050	2600	0.33	10927	29137	29137
10.5	11	4100	2300	2600	0.27	13754	34946	25367
11.0	11.5	4100	2400	2600	0.24	14976	37122	23738
11.5	11.9	4100	2400	2600	0.24	14976	37122	23738



Notes
1. Depth presented relative to ground surface.
2. This table to be analyzed in conjunction with the accompanying report.

APPENDIX F

P-y Curves

Hamlet Swing and Fixed Bridges

1659973 - R1

P-y CURVES
194 mm Diameter Micropiles at East Abutment
Hamlet Swing and Fixed Bridge

Figure F1

SUMMARY OF P-y CURVES FOR A 194 mm diameter Micropile

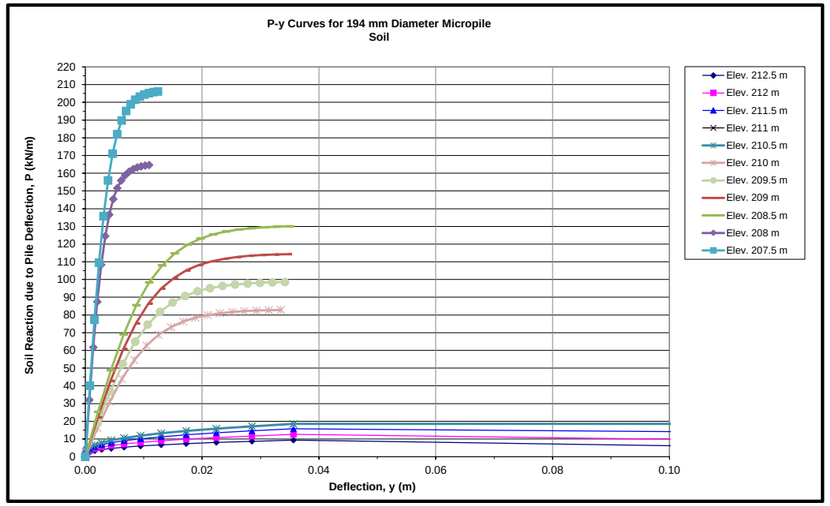
Description	**Very Soft to Soft Silty Clay**											
Depth (z) *	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m	
Elevation	Elev. 212.5 m		Elev. 212 m		Elev. 211.5 m		Elev. 211 m		Elev. 210.5 m		Elev. 210 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.669	0.000	0.897	0.000	1.126	0.000	1.321	0.000	1.321	0.002	16.126
	0.000	1.338	0.000	1.794	0.000	2.251	0.000	2.642	0.000	2.642	0.004	31.084
	0.000	2.006	0.000	2.692	0.000	3.377	0.000	3.964	0.000	3.964	0.006	44.021
	0.001	2.675	0.001	3.589	0.001	4.502	0.001	5.285	0.001	5.285	0.008	54.550
	0.002	3.344	0.002	4.486	0.002	5.628	0.002	6.606	0.002	6.606	0.010	62.704
	0.003	4.013	0.003	5.383	0.003	6.753	0.003	7.927	0.003	7.927	0.013	68.779
	0.004	4.682	0.004	6.280	0.004	7.879	0.004	9.248	0.004	9.248	0.015	73.175
	0.007	5.350	0.007	7.177	0.007	9.005	0.007	10.570	0.007	10.570	0.017	76.291
	0.009	6.019	0.009	8.075	0.009	10.130	0.009	11.891	0.009	11.891	0.019	78.465
	0.013	6.688	0.013	8.972	0.013	11.256	0.013	13.212	0.013	13.212	0.021	79.968
	0.017	7.357	0.017	9.869	0.017	12.381	0.017	14.533	0.017	14.533	0.023	80.998
	0.022	8.026	0.022	10.766	0.022	13.507	0.022	15.854	0.022	15.854	0.025	81.701
	0.029	8.694	0.029	11.663	0.029	14.632	0.029	17.176	0.029	17.176	0.027	82.179
	0.036	9.363	0.036	12.561	0.036	15.758	0.036	18.497	0.036	18.497	0.029	82.503
	0.178	2.428	0.178	6.514	0.178	12.258	0.178	18.497	0.178	18.497	0.031	82.723
	0.190	2.428	0.190	6.514	0.190	12.258	0.190	18.497	0.190	18.497	0.034	82.872

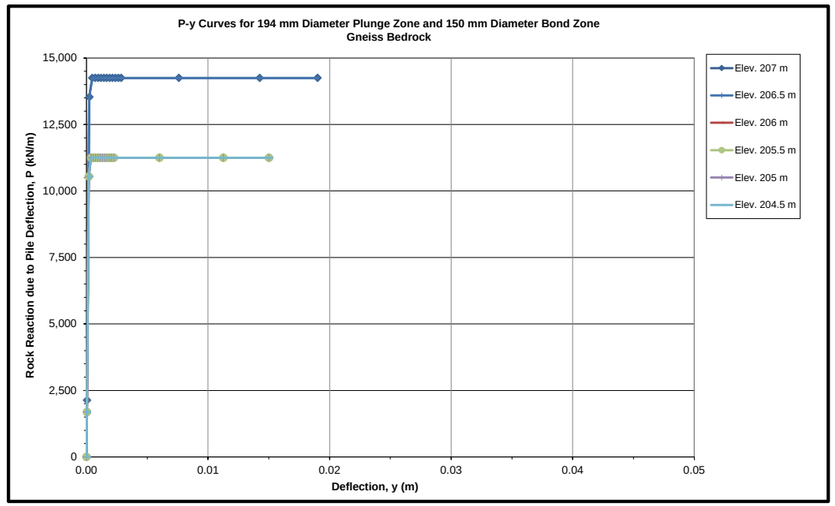
Description	**Very Loose to Compact Sand**									
Depth (z) *	z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m	
Elevation	Elev. 209.5 m		Elev. 209 m		Elev. 208.5 m		Elev. 208 m		Elev. 207.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	19.187	0.002	22.248	0.002	25.309	0.001	32.040	0.001	40.118
	0.004	36.984	0.004	42.884	0.004	48.784	0.001	61.759	0.002	77.330
	0.006	52.377	0.006	60.732	0.007	69.088	0.002	87.462	0.002	109.514
	0.009	64.905	0.009	75.259	0.009	85.614	0.003	108.383	0.003	135.709
	0.011	74.607	0.011	86.509	0.011	98.411	0.003	124.584	0.004	155.995
	0.013	81.834	0.013	94.890	0.013	107.945	0.004	136.653	0.005	171.108
	0.015	87.065	0.015	100.955	0.015	114.845	0.005	145.388	0.005	182.045
	0.017	90.772	0.017	105.253	0.018	119.734	0.005	151.577	0.006	189.795
	0.019	93.359	0.019	108.253	0.020	123.147	0.006	155.898	0.007	195.205
	0.021	95.147	0.022	110.326	0.022	125.505	0.007	158.883	0.008	198.942
	0.023	96.372	0.024	111.747	0.024	127.122	0.008	160.930	0.009	201.505
	0.026	97.209	0.026	112.717	0.026	128.225	0.008	162.327	0.009	203.254
	0.028	97.778	0.028	113.377	0.028	128.975	0.009	163.277	0.010	204.444
	0.030	98.164	0.030	113.824	0.031	129.485	0.010	163.921	0.011	205.251
	0.032	98.425	0.032	114.127	0.033	129.830	0.010	164.358	0.012	205.798
	0.034	98.602	0.035	114.333	0.035	130.063	0.011	164.653	0.012	206.168

Description	**Gneiss Bedrock (plunge length)**				**Gneiss Bedrock (bond zone)**							
Depth (z) *	z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m	
Elevation	Elev. 207 m		Elev. 206.5 m		Elev. 206 m		Elev. 205.5 m		Elev. 205 m		Elev. 204.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	2135.600	0.000	2135.600	0.000	1686.000	0.000	1686.000	0.000	1686.000	0.000	1686.000
	0.000	13530.833	0.000	13530.833	0.000	10546.250	0.000	10546.250	0.000	10546.250	0.000	10546.250
	0.000	14250.000	0.000	14250.000	0.000	11250.000	0.000	11250.000	0.000	11250.000	0.000	11250.000
	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.002	14250.000	0.002	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.002	14250.000	0.002	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.002	14250.000	0.002	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.002	14250.000	0.002	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.003	14250.000	0.003	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.003	14250.000	0.003	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.008	14250.000	0.008	14250.000	0.006	11250.000	0.006	11250.000	0.006	11250.000	0.006	11250.000
	0.014	14250.000	0.014	14250.000	0.011	11250.000	0.011	11250.000	0.011	11250.000	0.011	11250.000
	0.019	14250.000	0.019	14250.000	0.015	11250.000	0.015	11250.000	0.015	11250.000	0.015	11250.000

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap (Elevation 213 m).
The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical micropiles (i.e. no inclination).
2. Cylic loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).





Date: January 2018
Project No: 1659973-R1

Prepared By: ARV
Checked By: JC

Golder Associates

Hamlet Swing and Fixed Bridges
1659973 - R1

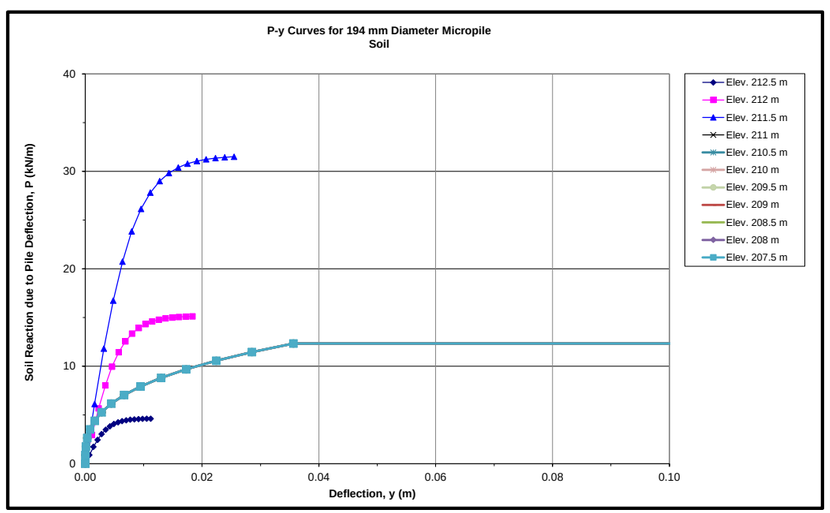
P-y CURVES
194 mm Micropiles at West Abutment
Hamlet Swing and Fixed Bridge

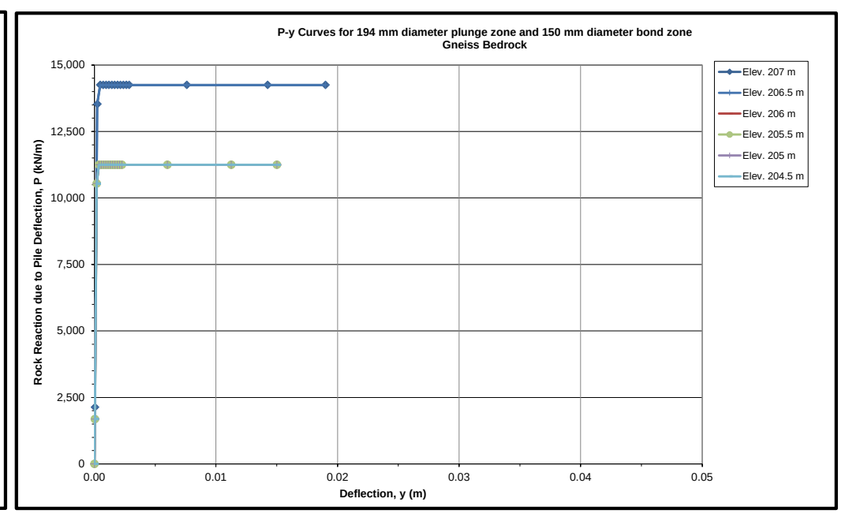
Figure F2

SUMMARY OF P-y CURVES FOR A 194 mm diameter Micropile

Description	Very Loose to loose Sand (Fill)						Very Soft Silty Clay																Gneiss Bedrock (plunge length)				Gneiss Bedrock (bond zone)							
*Depth (z) **	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m	
Elevation	Elev. 212.5 m		Elev. 212 m		Elev. 211.5 m		Elev. 211 m		Elev. 210.5 m		Elev. 210 m		Elev. 209.5 m		Elev. 209 m		Elev. 208.5 m		Elev. 208 m		Elev. 207.5 m		Elev. 207 m		Elev. 206.5 m		Elev. 206 m		Elev. 205.5 m		Elev. 205 m		Elev. 204.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.001	0.899	0.001	2.943	0.002	6.131	0.000	0.881	0.000	0.881	0.000	0.881	0.000	0.881	0.000	0.881	0.000	0.881	0.000	0.881	0.000	0.881	0.000	2135.600	0.000	2135.600	0.000	1686.000	0.000	1686.000	0.000	1686.000	0.000	1686.000
	0.001	1.734	0.002	5.673	0.003	11.818	0.000	1.762	0.000	1.762	0.000	1.762	0.000	1.762	0.000	1.762	0.000	1.762	0.000	1.762	0.000	1.762	0.000	13530.833	0.000	13530.833	0.000	10546.250	0.000	10546.250	0.000	10546.250	0.000	10546.250
	0.002	2.455	0.003	8.034	0.005	16.737	0.000	2.642	0.000	2.642	0.000	2.642	0.000	2.642	0.000	2.642	0.000	2.642	0.000	2.642	0.000	2.642	0.000	14250.000	0.000	14250.000	0.000	11250.000	0.000	11250.000	0.000	11250.000	0.000	11250.000
	0.003	3.043	0.005	9.956	0.006	20.740	0.001	3.523	0.001	3.523	0.001	3.523	0.001	3.523	0.001	3.523	0.001	3.523	0.001	3.523	0.001	3.523	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.004	3.498	0.006	11.444	0.008	23.840	0.002	4.404	0.002	4.404	0.002	4.404	0.002	4.404	0.002	4.404	0.002	4.404	0.002	4.404	0.002	4.404	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.004	3.836	0.007	12.553	0.010	26.150	0.003	5.285	0.003	5.285	0.003	5.285	0.003	5.285	0.003	5.285	0.003	5.285	0.003	5.285	0.003	5.285	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.005	4.082	0.008	13.355	0.011	27.821	0.004	6.166	0.004	6.166	0.004	6.166	0.004	6.166	0.004	6.166	0.004	6.166	0.004	6.166	0.004	6.166	0.001	14250.000	0.001	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.006	4.255	0.009	13.924	0.013	29.005	0.007	7.046	0.007	7.046	0.007	7.046	0.007	7.046	0.007	7.046	0.007	7.046	0.007	7.046	0.007	7.046	0.002	14250.000	0.002	14250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000	0.001	11250.000
	0.006	4.377	0.010	14.321	0.014	29.832	0.009	7.927	0.009	7.927	0.009	7.927	0.009	7.927	0.009	7.927	0.009	7.927	0.009	7.927	0.009	7.927	0.002	14250.000	0.002	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.007	4.461	0.011	14.595	0.016	30.403	0.013	8.808	0.013	8.808	0.013	8.808	0.013	8.808	0.013	8.808	0.013	8.808	0.013	8.808	0.013	8.808	0.002	14250.000	0.002	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.008	4.518	0.013	14.783	0.018	30.795	0.017	9.689	0.017	9.689	0.017	9.689	0.017	9.689	0.017	9.689	0.017	9.689	0.017	9.689	0.017	9.689	0.002	14250.000	0.002	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.008	4.557	0.014	14.911	0.019	31.062	0.022	10.570	0.022	10.570	0.022	10.570	0.022	10.570	0.022	10.570	0.022	10.570	0.022	10.570	0.022	10.570	0.003	14250.000	0.003	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.009	4.584	0.015	14.999	0.021	31.244	0.029	11.450	0.029	11.450	0.029	11.450	0.029	11.450	0.029	11.450	0.029	11.450	0.029	11.450	0.029	11.450	0.003	14250.000	0.003	14250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000	0.002	11250.000
	0.010	4.602	0.016	15.058	0.022	31.368	0.036	12.331	0.036	12.331	0.036	12.331	0.036	12.331	0.036	12.331	0.036	12.331	0.036	12.331	0.036	12.331	0.008	14250.000	0.008	14250.000	0.006	11250.000	0.006	11250.000	0.006	11250.000	0.006	11250.000
	0.011	4.614	0.017	15.098	0.024	31.451	0.178	12.331	0.178	12.331	0.178	12.331	0.178	12.331	0.178	12.331	0.178	12.331	0.178	12.331	0.178	12.331	0.014	14250.000	0.014	14250.000	0.011	11250.000	0.011	11250.000	0.011	11250.000	0.011	11250.000
	0.011	4.623	0.018	15.125	0.025	31.508	0.190	12.331	0.190	12.331	0.190	12.331	0.190	12.331	0.190	12.331	0.190	12.331	0.190	12.331	0.190	12.331	0.019	14250.000	0.019	14250.000	0.015	11250.000	0.015	11250.000	0.015	11250.000	0.015	11250.000

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap (Elevation 213 m).
The P-y curves have been generated based on the following assumptions:
1. P-y curves are generated for vertical micropiles (i.e. no inclination)
2. Cylic loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).





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