APPENDIX C - GEOTECHNICAL REPORT

GeoPro Consulting Limited

Geotechnical-Hydrogeology-Environmental-Materials-Inspection

Geotechnical Investigation

Proposed Replacement of St. Paul West CNR Bridge
St. Paul Street West over CNR Rail
City of St. Catharines, Niagara Region, Ontario

Prepared For:

Associated Engineering (Ontario) Limited

GeoPro Project No.: 18-2552G Revised

Report Date: August 2, 2019

Table of Contents

1 INTRODUCTION

GeoPro Consulting Limited (GeoPro) was retained by Associated Engineering (Ont.) Limited (the Client) to conduct a geotechnical investigation for the proposed replacement of St. Paul West CNR bridge located on St. Paul Street West over CNR Rail, City of St. Catharines, Niagara Region, Ontario.

The purpose of this geotechnical investigation was to obtain information on the existing subsurface conditions by means of a limited number of boreholes, in-situ tests and laboratory tests of soil samples to provide required geotechnical design information. Based on GeoPro's interpretation of the obtained data, geotechnical comments and recommendations related to the project designs are provided.

This report is prepared with the condition that the design will be in accordance with all applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practice. Furthermore, the recommendations and opinions in this report are applicable only to the proposed project as described above. On-going liaison and communication with GeoPro during the design stage and construction phase of the project is strongly recommended to confirm that the recommendations in this report are applicable and/or correctly interpreted and implemented. Also, any queries concerning the geotechnical aspects of the proposed project shall be directed to GeoPro for further elaboration and/or clarification.

This report is provided on the basis of the terms of reference presented in our approved proposal prepared based on our understanding of the project. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this report can be relied upon.

This report deals with geotechnical issues only. The geo-environmental (chemical) aspects of the subsurface conditions, including the consequences 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 were not investigated and were beyond the scope of this assignment. However, limited chemical testing was carried out on selected soil samples for disposal purposes.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. Laboratory testing follows ASTM or CSA Standards or modifications of these standards that have become standard practice in Ontario.

This report has been prepared for the Client only. Third party use of this report without GeoPro's consent is prohibited. The limitations to the report presented above form an integral part of the report and they must be considered in conjunction with this report.

2 PROJECT DESCRIPTION

The subject bridge is on St. Paul Street West between Great Western Street and Schickluna Street, in the City of St. Catharines. The existing St. Paul West CNR Bridge is a two lane three span slab on steel girder structure.

Based on the preliminary design information provided by the Client, a proposed replacement of St. Paul West CNR Bridge is considered. The proposed replacement consists of a single span bridge with a roadway cross section that includes two 3.5m vehicular lanes, two 1.8m wide paved shoulders/bicycle lanes and two 2.4 m wide sidewalks.

3 INVESTIGATION PROCEDURE

Field work for the geotechnical investigation was carried out on January 16 to 18 and February 13 to 15, 2019 during which time two (2) boreholes (Boreholes BH1 and BH2) were advanced to depths ranging from about 37.6 m to 57.1 m below the existing ground surface. The borehole locations are shown on attached Drawings.

A proposed borehole location plan prepared by GeoPro was provided to Client for review prior to the filed investigation work. The approved borehole locations were staked in the field by GeoPro; the borehole locations in the field were adjusted according to the drill rig accessibility and the underground utility conditions. It should be noted that Borehole BH1 was moved from the existing pavement to the boulevard due to the confliction to the existing utility/overhead cables. The field work for this investigation was monitored by a member of our engineering staff who logged the boreholes and cared for the recovered samples.

The boreholes were advanced in weathered shale below overburden. The weathered shale was recorded by sampling the soils at regular intervals of depth using a 50 mm O.D. split spoon sampler, in accordance with the Standard Penetration Test (ASTM D 1586) method. Upon encountering auger refusal in the weathered shale, the drill casing was sealed into bedrock and then the bedrock was sampled by diamond core drilling. The coring of rock was carried out with NQ size double tube wireline equipment, allowing recovery of 47 mm diameter rock cores. The monitoring technician/engineer recorded and visually described the rock samples. For the rock cores, the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) values and Fracture Indices (FI) were recorded in accordance with the conventions used by the International Society for Rock Mechanics (ISRM).

Groundwater condition observations were made in the boreholes during drilling and upon completion of drilling. A monitoring well (38 and 51 mm in diameter) was installed in each of Boreholes BH1 and BH2 to measure the groundwater table.

All soil samples obtained during this investigation were brought to our laboratory for further examination. These soil samples will be stored for a period of three (3) months after the day of issuing draft report, after which time they will be discarded unless we are advised otherwise in writing. Geotechnical classification testing (including water content, grain size distribution and Atterberg Limits, when applicable) was carried out on selected soil samples. The laboratory test results are attached to Figures.

The ground surface elevations at the as drilled borehole locations were not available at the time of preparing this report. Therefore, the stratigraphy at each borehole location has been referenced to the current grade level. Contractors performing the work should confirm the elevations prior to construction. The borehole locations plotted on Borehole Location Plan were based on the measurements of the site features and should be considered to be approximate.

4 SUBSURFACE CONDITIONS

4.1 Soil Conditions

Notes on sample descriptions are presented in Enclosure 1A. Explanations of terms used in the borehole logs are presented in Enclosure 1B. An explanation of terms used in the rock core logs is presented in Enclosure 1C. The subsurface conditions in the boreholes are presented in the individual borehole logs. Detailed descriptions of the major soil strata encountered in the boreholes drilled at the site are provided as follows.

Pavement Structure

A flexible pavement structure was observed in Borehole BH2. The composition thickness of pavement structure is summarized in the following table:

Pavement Structure (mm)		
Asphalt Concrete	**Granular Base**	**Granular Subbase**
180	220	220

Topsoil

Topsoil with a thickness of about 200 mm was encountered surficially in Borehole BH1. In general, the topsoil consists of high contents of organics with trace to some rootlets. It should be noted that the thickness of the topsoil explored at the borehole locations may not be representative for the site and should not be relied on to calculate the amount of topsoil at the site.

Fill and Probable Fill Materials

Fill and probable fill materials consisting of clayey silt, sandy silt and silty sand were encountered below the topsoil or granular subbase in Boreholes BH1 and BH2, and extended to depths ranging from about 1.4 m to 2.9 m below the existing ground surface. For the (probable) cohesive fill materials, an SPT N value of 9 blows per 300 mm penetration indicated a stiff consistency. For the cohesionless fill materials, SPT N values ranging from 6 to 15 blows per 300 mm penetration indicated a loose to compact compactness. The in-situ moisture content measured in the soil samples ranged from approximately 9% to 20%.

Upper Clayey Silt to Silty Clay

Upper clayey silt to silty clay deposits were encountered below the (probable) fill materials in Boreholes BH1 and BH2, and extended to depths ranging from about 32.7 m to 34.1 m below the existing ground surface. SPT N values ranging from 4 to 23 blows per 300 mm penetration indicated a soft to very stiff consistency. The natural moisture content measured in the soil samples ranged from approximately 20% to 40%.

Sandy Silt Till

Sandy silt till deposit was encountered below the upper clayey silt to silty clay deposits in Boreholes BH1 and BH2, and extended to depths ranging from about 36.0 m to 38.1 m below the existing ground surface. SPT N values ranging from 52 to 84 blows per 300 mm penetration indicated a very dense compactness. The natural moisture content measured in the soil samples was approximately 11%.

Upper Sandy Silt and Fine Sand and Silt to Silt

Upper sandy silt and fine sand and silt to silt deposits were encountered below the sandy silt till deposit in Boreholes BH1 and BH2, and extended to depths ranging from about 37.6 m to 44.5 m below the existing ground surface. Borehole BH2 was terminated in these deposits. SPT N values ranging from 38 to 68 blows per 300 mm penetration indicated a dense to very dense compactness. The natural moisture content measured in the soil samples ranged from approximately 14% to 15%.

Silty Sand and Gravel

Silty sand and gravel deposit was encountered below the upper fine sand and silt to silt deposit in Borehole BH1, and extended to a depth of about 45.3 m below the existing ground surface. An SPT N value of 66 blows per 300 mm penetration indicated a very dense compactness. The natural moisture content measured in the soil sample was approximately 10%.

Middle Clayey Silt

Middle clayey silt deposit was encountered below the silty sand and gravel deposit in Borehole BH1, and extended to a depth of about 48.8 m below the existing ground surface. An SPT N value of 66 blows per 300 mm penetration indicated a hard consistency. The natural moisture content measured in the soil sample was approximately 13%.

Lower Silt

Lower silt deposit was encountered below the middle clayey silt deposit in Borehole BH1, and extended to a depth of about 50.5 m below the existing ground surface. An SPT N value of 22 blows per 300 mm penetration indicated a compact compactness. The natural moisture content measured in the soil sample was approximately 15%.

Lower Silty Clay

Lower silty clay deposit was encountered below the lower silt deposit in Borehole BH1, and extended to a depth of about 51.2 m below the existing ground surface. An SPT N value of 22 blows per 300 mm penetration indicated a very stiff consistency. The natural moisture content measured in the soil sample was approximately 26%.

Queenston Formation Shale (Bedrock)

The shale bedrock was encountered in Borehole BH1 at an inferred depth of about 51.2 m below the existing ground surface. The shale bedrock was proven by rock core drilling in Borehole BH1 and inferred by sampling the weathered shale fragments at regular intervals of depth using a 50 mm O.D. split spoon sampler.

The shale bedrock of the Queenston Formation at the site primarily consists of typically highly weathered to fresh, reddish brown, fine to very fine grained, fissile, extremely weak to medium strong shale bedrock interbedded with fresh to slightly weathered, greenish grey, fine to very fine grained, fissile to massive, weak to strong siltstone. The inferred bedrock surface was at a depth of about 51.2 m below the existing ground surface. It is noted that variations in the bedrock surface should be expected.

Stress relief features such as folds and faults are common in the Queenston Formation. In these features the rock is heavily fractured and sheared, and contains layers of shale rubble and clay. Due to the fracturing, these features may also be groundwater conduits, which could result in excessive water flow into excavations.

Significant variation of bedrock depths should be anticipated beyond the boreholes. Photographs of the rock cores are presented in Appendix A. Detailed descriptions of the index properties and results of laboratory testing are presented in the following paragraphs.

Total Core Recovery (TCR)

The total core recovery indicates the total length of rock core recovered expressed as a percentage of the actual length of the core run (usually 1.5 m). The total core recovery ranged from 85% to 100% and appears to improve with increasing depth. Typically, TCR values greater than 90% are considered good. It should be noted that weathering of the rock near the rock surface, coring with smaller diameter NQ equipment, and mechanical breaks during the rock coring may all be contributing factors to lower TCR values.

Solid Core Recovery (SCR)

The solid core recovery is the total length of solid, cylindrical pieces of recovered rock core, expressed as a percentage of the actual length of core run (usually 1.5 m). The solid core recovery ranged from 79% to 98% in the investigation and also appeared to increase with depths. The SCR index was generally influenced by the orientations of the fractures; SCR was low when fractures oblique to the borehole axis were intercepted.

Rock Quality Designation (RQD)

The rock quality designation index is obtained by measuring the length of recovered rock core pieces which are longer than 100 mm and expressing their sum length as a percentage of the actual length of core run (usually 1.5 m). RQD is a function of the frequency of joints, bedding plane partings and fractures in rock cores. While the use of double tube core barrels provided reasonably good protection of the core during drilling and core retrieval, the fissile nature of the shale greatly influences RQD values of rock cores. Consequently, it is believed that the RQD values recorded generally underestimate the rock quality classification of the laminated shale. On the basis of the recorded RQD values which range from 67% to 92%, the rock quality (based on Deere's classification system) ranges from "fair" to "excellent", and the average value of approximately 80% suggests a rock of generally "good" quality.

Hard Layers

When recovering the core samples, the thickness of interbedded "hard" siltstone layers were measured and their aggregate expressed as a percentage of the actual length of core run (usually 1.5 m). "Hard layers" are defined herein as distinct stronger rock layers or lenses which have unconfined compressive strengths exceeding that of the bulk of rock mass. However, this is a subjective index based on visual examination and relatively basic index strength tests. The measured thicknesses of individual hard layers of the rock cores were typically less than 130 mm in the investigation. Percentage of hard layers ranged from 1% to 25% from the retrieved rock cores. The hard layers are mainly siltstone and may vary significantly in thickness over a short distance.

Fracture Index

The fracture index is a measure of the frequency of fracturing and bedding plane separations. It is expressed as the number of fractures per 0.3 m length of rock core run. Breaks which were obviously induced by drilling are excluded. A continuous vertical fracture, regardless of its length, is counted as one fracture. The recorded values ranged between 0 to greater than 25 in the investigation. It was observed that the planes of weaknesses along which the cores tended to break included planes of fissility and bedding, the contact surfaces between shale and siltstone bands and some oblique and subvertical joints. The joints along the planes of fissility and bedding surfaces were generally smooth and clean, while those along the bedding surfaces were generally more open and were occasionally infilled with clay. The occasional oblique and subvertical faults were often stepped to irregular and the joint surfaces were often rough to very rough.

Weathering

In general, weathering in the bedrock was limited to the surfaces of major discontinuities. Deeper penetrating weathering has occurred in the zones very close to the bedrock surface, where the degree of weathering is described predominantly as highly weathered to fresh. Below this, the degree of weathering ranged from slightly weathered to fresh, except along surfaces of major discontinuities, where the degree of weathering ranged from moderately weathered to highly weathered. The siltstone layers were generally fresh with only slight surficial weathering on joint surfaces in the zone close to bedrock surface.

Unconfined Compressive Strength (UCS)

Test results of unconfined compressive strengths of rock cores measured on seven (7) samples are presented in Appendix B. UCS test results ranged from 11.93 MPa to 37.74 MPa with average of 25.42 MPa. Based on these results, the rock is classified as a “weak to medium strong rock” according to ISRM.

Point Load Index Strength

Point load index strength tests were performed on forty-four (44) shale/siltstone samples. Tests were performed both in the axial and diametral orientation of the core samples. The test results are presented in rock core logs (Enclosure 2). Where the values were calculated by using the empirical relationship between unconfined compressive strength (UCS) and point load index strength as follows:

UCS [MPa] $\approx$ 24 $I_{S(50)}$

Where $I_{S(50)}$ is the point index strength in MPa for a 50 mm equivalent diameter core. This is a very approximate correlation after Franklin and Hoek. It should be noted that this empirical relationship may overestimate the UCS for shale.

The equivalent unconfined compressive strength of rock was inferred to range from 4.76 to 62.10 MPa with an average value of 24.26 MPa in the axial direction and from 0.63 to 18.32 MPa with an average value of 6.58 MPa in the diametral direction. Those values are indicative of “extremely weak” to “strong” rock under ISRM strength convention. The shale can often be broken by hand in the diametral direction, indicating considerable strength anisotropy along bedding planes, which is also proved by the much lower values obtained in the tests performed in the diametral direction.

Gas

The Queenston Formation is known to contain pockets of combustible gas. In some areas of the GTA, this gas has been found to migrate up into the overlying soils.

4.2 Groundwater Conditions

Groundwater condition observations made in the boreholes during and immediately upon completion of drilling are shown in the borehole logs and are also summarized in the following table.

Borehole No.	BH Depth (m)	Depth of Water Encountered during Drilling (mBGS)	Water Level upon Completion of Drilling (mBGS)	Cave-in Depth upon Completion of Drilling (mBGS)
BH1	57.1	7.6	-*	-
BH2	37.6	9.0	-*	-

Note: mBGS = meters below ground surface
* Water level was not measured upon completion of drilling due to use of drilling mud

Monitoring well (51 mm or 38 mm in diameter) was installed to monitor groundwater level. The monitoring well construction details and measured groundwater level are shown in the borehole logs and are also summarized in the following table.

Monitoring Well ID	Screen Interval (mBGS)	Water Level (mBGS)	
		February 26, 2019	March 5, 2019
BH1	12.2 – 15.2	3.23	4.85
BH2	34.2 – 37.2	18.30	-

Note: mBGS = meter below ground surface

It should be noted that groundwater levels can vary and are subject to seasonal fluctuations in response to weather events.

5 REVIEW OF SUBSURFACE CONDITIONS AND RECOMMENDATIONS OF SOIL PARAMETERS

The main findings of the soil strata are summarized as follows:

- A pavement structure was intercepted in Borehole BH2;
- Fill materials with varying thicknesses were encountered at both borehole locations up to depths varying from 1.4 m (BH1, i.e. west of the existing bridge) to 2.9 m (BH2, i.e. east of the existing bridge);
- Underlying the fill materials, native cohesive clayey deposits, cohesionless sandy/silty/gravelly deposits and glacial tills were encountered at both boreholes;
- Shale bedrock was encountered in Borehole BH1;

- Groundwater table measured in the boreholes ranged from 3.23 m to 18.30 m below the ground surface.

The recommended soil parameters are summarized in the following Table 1:

Table 1: Recommended Unfactored Soil Parameters

Fill Materials	18	8	28°	1.3	0.36	0.53	2.8	-
Soft to Firm Clayey Silt to Silty Clay	17	7	26°	-	0.38	0.56	2.6	12
Stiff to Very Stiff Clayey Silt and Silty Clay	18	8	28°	-	0.36	0.53	2.8	50
Hard Clayey Silt	19	9	29°	-	0.35	0.52	2.9	200
Compact Silt	20	10	28°	4.4	0.36	0.53	2.8	-
Dense to Very Dense Sandy Silt and Fine Sand and Silt to Silt	21	11	31°	11	0.32	0.49	3.1	-
Very Dense Sandy Silt Till	22	12	31°	11	0.32	0.49	3.1	-
Very Dense Silty Sand and Gravel	23	13	33°	11	0.30	0.46	3.4	-

Notes: c_u = Undrained shear strength of soil (kPa);
ϕ' = Effective angle of friction of soil (degrees);
K_a = Coefficient of active earth pressure;
K_o = Coefficient of earth pressure at rest;
K_p = Passive earth pressure coefficient;
γ = Bulk unit weight of soil (kN/m^3);
γ' = Effective unit weight of soil below the groundwater level (kN/m^3);
n_h = Parameter for Horizontal Subgrade Reaction (MN/m^3)

6 DISCUSSION AND RECOMMENDATIONS

This report contains the geotechnical engineering recommendations and comments. These recommendations and comments are based on factual information and are intended only for use by the design engineers. The number of boreholes and test pits may not be sufficient to determine all the factors that may affect construction methods and costs. Subsurface conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. Construction methods discussed, however, express GeoPro's opinion only and are not intended to direct the contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may have an effect on the construction.

The design drawings of the project were not available when this report was prepared. Once the design drawings and detailed site plan are available, this report will be reviewed by GeoPro, and further recommendations will be provided as needed.

6.1 Bridge Foundation Design Considerations

Based on the borehole information, shallow foundations are not considered to be a desirable option since the soil strengths in the vicinity of Boreholes BH1 and BH2 are not considered to be sufficient to support the proposed bridge on a spread footing. Augered caissons (bored piles) are also considered to be not feasible due to the high groundwater tables and the presence of the cohesionless soils at the site. Therefore, consideration could be given to supporting the bridge abutments on driven steel H piles founded in the competent very dense glacial tills or shale bedrock. The driven steel H-pile foundation would also permit integral abutment design.

6.1.1 Driven H Piles

The vertical axial geotechnical resistance of an HP pile driven to an adequate set in the sound shale bedrock are shown in the following table.

Pile Section	Factored Axial Resistance at Ultimate Limit States (ULS) (kN)
HP 310x110	1800*
HP 310x132	2100*
HP 310 x 152**	2400*

*in the sound shale bedrock
**Due to the proposed driving depth, HP310x152 is recommended

The Serviceability Limit States (SLS) condition will not govern for piles founded on bedrock. The shale bedrock depth in the vicinity of Borehole BH2 may vary from the bedrock depth in Borehole BH1, which shall be considered in the contract. The contractor shall be warned about the bedrock variation which may cause additional length of piles and additional splicing.

Alternatively, the vertical axial geotechnical resistance of an HP 310x110 pile driven to an adequate set in the very dense tills are shown in the following table.

BHs	Factored Axial Resistance at ULS (kN)	Axial Resistance at SLS (kN)	Pile Tip Depth below Ground Surface (m)	Anticipated Founding Soil
BH1	960	800	35.5	Very Dense Sandy Silt Till
BH2	960	800	33.0	Very Dense Sandy Silt Till

For the foundations designed to the specified bearing resistance values at the serviceability limit states (SLS), the anticipated maximum total and differential settlements of the foundations are expected to be less than 25 mm and 20 mm, respectively.

The horizontal spacing of the piles should be at least 30 inches or 3.0 times the pile size. For end bearing driven piles, the vertical resistances will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to applicable Canadian Highway Bridge Design Code (CHBDC).

The structural resistance of the pile should be checked by the structural designer. At any time, the pile stresses should not exceed 85% of the pile steel yield stress or follow the requirement in Canadian Highway Bridge Design Code (CHBDC). Plumbness and location of the driven pile should follow the requirements of the design specification provided by the structural engineer. Any misaligned or damaged piles should be replaced. The possibility of the piles encountering potential obstructions, such as cobbles and boulders and other debris in fill should be anticipated.

It should be noted that the recommended foundation type, founding depths, and bearing resistances were based on the borehole information only. The geotechnical recommendations and comments are necessarily on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to the subsurface conditions between and beyond the boreholes when foundation construction is underway. The interpretation between and beyond the boreholes and the recommendations of this report **must** therefore be checked through field inspections provided by a qualified geotechnical engineer from GeoPro to validate the information for use during the construction stage. Due to the anticipated variation of the subsurface conditions at this specific site, the geotechnical engineer who carried out the geotechnical investigation shall be retained during the construction stage to avoid the potential misinterpretation of the soil information presented in the report.

6.1.2 Downdrag (Negative Skin Friction)

Downdrag or negative skin friction of piles may occur wherever piles are on or adjacent to, recently placed fills or existing fill which undergoes ongoing settlement. This can occur at the abutment locations should any grade raise be considered. Lightweight foam concrete fill (LFCF) is to be considered for the proposed grade raise of up to 2.5 m as discussed below. It is understood that a proposed grade raise of LFCF with a unit weight of 20% of the granular materials (i.e. 4 to 6 kN/m^3) is to be considered at the bridge site. The downdrag may not be considered in the design due to the up to 2.5 m of LFCF (i.e. unit weight of 4 to 6 kN/m^3) used at the site.

6.1.3 Lateral Resistances

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soils, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. The lateral resistance of the piles can be supplemented, if desired, by horizontal components of battered components of battered piles.

Ultimate Lateral Earth Resistance

The equations presented in the following may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the pile. For preliminary assessment purposes, the assessed horizontal passive resistance and geotechnical reaction at SLS in accordance with CHBDC shall be referenced.

For cohesive soils, the passive earth pressure on the pile at a depth Z can be determined from the following expression:

$$p_{ult} = 6C_u$$

For cohesionless soils, the p_{ult} value can be calculated using the following equation:

$$p_{ult} = 3\gamma Z K_p$$

The ultimate lateral earth resistance (force) on a short pile section of length l_z at depth Z can be expressed as

$$\Delta R_u = l_z B p_{ult}$$

Where

p_{ult} = the passive earth pressure on the pile at a depth Z, in kPa.

ΔR_u = ultimate lateral earth resistance on a pile section of length l_z and at depth Z, in kN.

Z = depth below final grade, in metre.

L = length of pile, in metre. Should be limited within six times the pile diameter/size

B = size (diameter) of pile, in metre

γ = unit weight of soil, in kN/m^3

K_p = passive earth pressure coefficient

C_u = undrained shear strength of clayey silt and bedrock, in kPa.

The passive lateral resistance of the soils within the frost depth should be ignored.

The direction of the lateral earth resistance (ΔR_u) is opposite to the direction of the lateral movement of the pile at depth *Z*.

The lateral capacity of the pile itself depends on the lateral earth resistance (ΔR_u) along the pile, and on the constraint conditions at the top of the pile. For analyses of the proposed piles founded in the very dense glacial tills, shale or long piles, it can be assumed that the base (bottom) of the piles will not move in both the vertical and horizontal directions.

For a short pile section of length l_z at depth Z, the factored lateral geotechnical resistance ( ) at the ultimate limit states (ULS) can be determined from the following expression:

$$\Delta R_{ULS} = \Phi_h \Delta R_u$$

where Φ_h is the lateral earth resistance factor. According to the Canadian Foundation Engineering Manual, 4th Edition (CFEM, 2006), the lateral earth resistance factor can be taken as Φ_h = 0.5.

The lateral capacity of piles at SLS should be determined according to the lateral deflection of the piles calculated using the modulus of horizontal subgrade reaction of the soil (k_h) described in the following sections.

Modulus of Horizontal Subgrade Reaction (k_h)

The modulus of horizontal subgrade reaction of the soil (k_h) can be used to evaluate the lateral deflection and bending of the proposed piles, where k_h is determined as given in the section below.

In the model of pile-soil interaction, the lateral earth resistance of soil can be simulated by a series of linear springs, and the stiffness coefficient of the springs or spring constant (K_{spr}) can be obtained from the calculated values of the modulus of horizontal subgrade reaction (k_h). For a pile with a diameter of *B* and a distance of *t* between two adjacent springs, the value of K_{spr} can be calculated using

$$K_{spr} = B \cdot t \cdot k_h.$$

ΔR_{ULS}

The unit of K_{spr} is kN/m, and the unit of t is metre (m).

Cohesive Clayey Soils:

The modulus of horizontal subgrade reaction (k_h) of the cohesive soils can be calculated using the following equation:

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

where B represents the diameter of the pile and C_u is the undrained shear strength of the cohesive soils as given in **Table 1** in Section 5.

Non-cohesive Silty/Sandy/Gravelly Soils:

For the non-cohesive silty/sandy/gravelly soils, the value of the modulus of horizontal subgrade reaction k_h can be estimated using

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

Where Z is the depth, B is the diameter of pile, and n_h is a coefficient related to soil density, as listed on **Table 1** in Section 5.

It should be noted that the lateral resistance of soil is limited and the linear springs used in the analysis should not be loaded beyond the allowable passive lateral resistance of the corresponding soil.

The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments, piers or retaining walls. The SLS resistance will normally be greater than the ULS resistance for pile embedded in very stiff or dense soils. This 10 mm limitation of horizontal movement does not apply where an analysis of the structure including the foundation indicates that a horizontal movement of more than 10 mm can be accommodated by both the foundation and the structure at SLS. In many cases, integral abutment bridges may have total lengths that result in SLS horizontal deflection that are greater than 10 mm.

Geotechnical Parameters for Lateral Resistance Design

For the lateral resistance design, pile-soil interaction analysis may be carried out using the geotechnical parameters provided in the following **Table 1** in Section

Group Effect

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_h) may have to be reduced based on pile spacing.

Where a pile group is oriented **parallel** to the direction of loading, the group action may be considered by reducing the values of coefficient of horizontal subgrade reaction (k_h) by a reduction factor, R as follows:

Pile Spacing in direction of Loading (B = Pile Diameter)	Subgrade Reaction Reduction Factor (R)*
8B	1.0
6B	0.7
4B	0.4
3B	0.25

*Intermediate values may be obtained by interpolation.

Where a pile group is oriented **perpendicular** to the direction of loading, the group action may be considered by reducing the values of the coefficient of horizontal subgrade reaction (k_h) by a reduction factor (R), which can be expressed as

$$R = 0.5\left(1 + \frac{x}{3B}\right)$$

In the above equation, *x* represents the centre-to-centre distance between adjacent piles, and B is the diameter of the pile. If the centre-to-centre distance between the adjacent piles is equal to or greater than 3 times its diameter (3B), the group action effect can be ignored.

6.1.4 Pile Driving

Pile installation should be in accordance with the OPSS 903, April 2016. The contractor who is performing the pile driving should retain a qualified geotechnical engineer as the QVE engineer.

As noted above, cobbles and boulders should be anticipated in the subsoils and as such, prospective foundation contractors should be alerted to this in the tender documents. If required by the engineer, the use of driving shoes or other means of stiffening of the tips of the piles in accordance with OPSD 3000 should be considered to minimize potential damage to the pile toes when driving into the dense to very dense and hard native strata, which should be included in the tender documents. For piles socketed in shale bedrock, rock point (such as Hard Bite HP7780-B or the equivalent) shall be considered. All points are to be installed in accordance with OPSS 903.

Note No.2 from Article 3.3.3 Pile Driving Notes in the MTO Structural Manual should be used on the Foundation Design Drawing, i.e. “Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of two times of the factored Axial Resistance at ULS presented in Section 6.1.1 per pile but must be driven until dense deposits or sound bedrock in accordance with OPSS 903, April 2016”.

The piling contractor should ensure that the pile-driving hammer is powerful enough to achieve the required bearing resistances and required pile driven depths, but will not cause damage of the piles during the pile driving. Care must be taken to avoid overdriving and damaging the pile. The pile driving should be observed, on a full time basis, by an experienced geotechnical technician, who will record penetration resistance and pile tip elevation, etc. The technician must be supervised by a professional engineer experienced in this type of work.

As a preliminary guideline, pile should be driven with a suitable hammer capable of delivering at least 30 kilojoules/blow but less than 100 kilojoules/below (i.e. subject to designed pile length and founding soils in dense deposit or sound shale bedrock) with an energy transfer (efficiency) in the order of 50%. Effective refusal can generally be assumed to have been obtained when at least 10 blows have been recorded for 1 inch of pile penetration using a suitable sized hammer. The contractor should retain a geotechnical consultant for a driveability analysis to determine the proper rated energy of the hammer to be used for the driving to achieve the effective refusal. The type of the hammer selected by the Contractor should be approved by the engineers prior to construction.

Due to the variation of the soil/bedrock conditions encountered in the boreholes at the designed founding depths, the actual pile penetration depths to achieve the above provided bearing resistances may be greatly variable. As noted above, the selection of a suitable hammer is critical for the successful pile driving. The contractor should allow for some variation in pile length and this aspect should be taken into consideration when ordering the piles. The unit price of driving extra length of piles including additional pile splicing should be included in the contract.

During the driving process, piles that have already been driven will need to be monitored in order to determine if heaving occurred due to the effect of the driving of adjacent piles. If pile heaving occurs, the affected piles will need to be re-driven. Re-tapping to check that relaxation has not occurred will also be necessary. Furthermore, it may be necessary to stagger the driving of the piles.

In consideration of the anticipated presence of the cohesionless silty/sandy/gravelly deposits at the proposed founding depths, waiting time for the pore pressures to dissipate and repeating pile driving test should be considered in the contract. For project schedule purpose, a waiting period of about two weeks may be considered for the pile set up and re-tap.

Conventional pile driving operations may cause vibrations that could affect nearby structures. An evaluation of existing surrounding foundation types and a pre-construction condition survey should be carried out, if applicable, prior to pile driving operations.

6.1.5 Integral Abutment

Should an integral abutment structure be designed for the proposed bridge, the piles should be end bearing on the sound shale bedrock or competent soils, using the geotechnical bearing resistance discussed above in this report. HP 310x110 or other sizes depending on the design axial loads may be considered for the integral abutments. Piles should be installed with their weak axis perpendicular to the center line of the beams. Piles may be fitted with driving points to protect the toes and improve penetration as discussed before. The designer shall design the H-piles considering the structural resistance of the piles and the geotechnical resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group in consideration of the scour.

To accommodate movement of the integral abutment system, two concentric Corrugated Steel Pipes (CSPs) that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with granular material meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements shown in the table below may be used. Refer to MTO Report SO 96-01 for further details.

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 μm	#30	80 – 100
425 μm	#40	40 – 80
250 μm	#60	5 – 25
150 μm	#100	0 – 6

The CSP or auger hole constructed as part of the integral abutment will be carried out in various soils below the groundwater tables. The groundwater table should be lowered to about 1 m below the base of the CSP and the excavation may have to be supported by a steel liner to prevent caving of the excavation side walls.

Integral bridge abutments experience stresses due to the cyclic thermal expansion and contraction of the bridge deck, pushing the abutments into and out of the bridge embankment. The abutment stem wall and diaphragm should be designed to withstand a passive earth pressure state, which may not be fully mobilized in consideration of the length of the bridge. The cyclic expansion can develop into large lateral earth pressures on the abutments as the expansion of the superstructure occurs. The lateral earth pressures increase over time as the soil behind the abutment fills the void and becomes increasingly wedged in. Granular materials such as OPSS Granular A and Granular B Type II should be used as backfill materials behind the abutment wall to at least 1 m beyond the edge of the approach slabs.

Use of an approach slab is required. The existing soils are considered to be frost susceptible and should be removed and replaced with OPSS Granular A or Granular B Type II as noted above. The granular base should also extend to at least 1 m beyond the edge of the approach slabs and sufficient drainage should be provided for the granular base. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads on abutments is permitted. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

6.2 Earth Pressures and Retaining Structures

Backfilling behind bridge abutments and any retaining (wing) walls should consist of granular materials in accordance with the applicable Standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic pressure build-up.

Computation of earth pressures acting against bridge abutments, retaining walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code (CHBDC). For design purposes, the following properties can be assumed for level and upward sloping backfill.

Compacted Granular 'A' or Granular 'B' Type II

Angle of Internal Friction ϕ=35° (unfactored)

Unit weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
K_a=0.27	K_a=0.34	K_a=0.40
K_b=0.35	K_b=0.44	K_b=0.50
K_o=0.43	K_o=0.56	K_o=0.62
K*=0.45	K*=0.60	K*=0.66

Compacted Granular 'B' Type I

Angle of Internal Friction ϕ=32° (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
K_a=0.31	K_a=0.39	K_a=0.47
K_b=0.39	K_b=0.49	K_b=0.57
K_o=0.47	K_o=0.62	K_o=0.69
K*=0.54	K*=0.68	K*=0.78

Note: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts

K_o is the coefficient of earth pressure at rest

K* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

These values are based on the assumption that the backfill behind the retaining structures is free-draining granular material and adequate drainage is provided.

The earth pressure coefficient to be adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop.

The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients. The use of vibratory compaction equipment behind the abutments and the retaining walls should be restricted in size.

As an alternative to conventional retaining walls, consideration could be given to Retained Soil System in which case the designer will have to include the geometric, performance and appearance requirements. The Retained Soil System must be designed and constructed by a specialized contractor.

6.3 Seismic Considerations

6.3.1 Seismic Design Parameters

Based on the results of site investigation and in accordance with Section the CHBDC (2006), the following seismic parameters may be considered for the designs:

- Velocity Related Seismic Zone: 0
- Zonal Velocity Ratio: 0.05
- Acceleration Related Seismic Zone: 1
- Zonal Acceleration Ratio: 0.05
- Peak Horizontal Acceleration: 0.08

The soil profile type at this site may be classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (Ground Motion Amplification Factor) of 1.5 may be considered in the seismic design.

6.3.2 Retaining Wall Seismic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. For the design of retaining walls, the coefficients of horizontal earth pressure in the following table may be used.

Compacted Granular ‘A’ or Granular ‘B’ Type II

Angle of Internal Friction ϕ=35° (unfactored) Unit weight = 22 kN/m^3 Wall friction angle $\delta = 1/2\,\phi = 17.5°$

Wall Condition	**Non-yield**	**Yield**
Active (K_{AE})*	0.29	0.26
Passive (K_{PE})*	6.90	7.19
At Rest (K_{OE})**	0.58	0.48

Compacted Granular 'B' Type I

Angle of Internal Friction ϕ=32° (unfactored) Unit weight = 21 kN/m^3 Wall friction angle $\delta = 1/2\,\phi = 16°$

Wall Condition	**Non-yield**	**Yield**
Active (K_{AE})*	0.33	0.30
Passive (K_{PE})*	5.41	5.64
At Rest (K_{OE})**	0.62	0.52

*After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall; weight of sloping backfill above top of wall shall be treated as a surcharge
**After Wood

6.4 Frost Protection

All foundations exposed to seasonal freezing conditions must have at least 1.2 m of soil cover or its thermal equivalent for frost protection. Abutment stems, pier caps, and any associated concrete wing walls/retaining wall footings, should be founded at a minimum depth of 1.2 m below the lowest surrounding grade, to provide adequate protection against frost penetration. It should be noted that the scour protection, such as rip rap and rock blocks should not be considered as earth cover for frost protection purposes.

6.5 Excavation and Groundwater Control

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill materials, native soft to firm clayey soils and cohesionless silty/sandy/gravelly soils can be classified as Type 3 Soil above groundwater table and as Type 4 Soil below the water table. The stiff to hard clayey soil and glacial till deposits can be classified as Type 2 Soil above groundwater table and as Type 3 soil below the water table.

Cobbles/Boulders are anticipated in the native soils. Provisions must be made in the excavation contract for the removal of possible cobbles and boulders in the native soil or potential obstructions in the fill materials.

The excavations for proposed abutments and removal of unsuitable soil at the approach embankment areas may extend to a maximum depth of about 4 to 5 m below the existing ground surface through the existing fill materials and native clayey silt to silty clay soils below the groundwater tables. If space permits, open-cut excavations to the proposed depths may be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. However, the high groundwater table may make the excavation very difficult. The presence of the cobbles and boulders in the native soils may make the sheet pile driving difficult.

Groundwater control during excavation within the fill materials, native stiff to very stiff clayey soils above the groundwater table at the site can be handled, as required, by pumping from properly constructed and filtered sumps located within the excavations. However, more significant seepage should be expected once the excavations extend below the prevailing groundwater tables in the fill materials, native soft to firm clayey soils and any wet silty/sandy layers/zones within the native clayey soils. Depending upon the actual thickness and extent of these soils, the prevailing groundwater level at the time of construction, some form of positive groundwater control, in addition to pumping from sump, may be required to maintain the stability of the base and side slopes of the excavations in these areas. In order to maintain a dry work space, the prevailing groundwater table should be drawn down to at least 1 m below the bottom elevation of the excavation. It should be noted that any construction dewatering or water taking in Ontario is governed by Ontario Regulation 387/04 - Water Taking and Transfer, made under the Ontario Water Resources Act (OWRA), and/or Ontario Regulation 63/16 – Registrations under Part II.2 of the Act – Water Taking, made under Environmental Protection Act. Based on these regulations, water taking of more than 400,000 L/day is subject to a Permit to Take Water (PTTW), while water taking of 50,000 L/day to 400,000 L/day is to be registered through the Environmental Activity and Sector Registry (EASR)

Groundwater control at these locations would be required to allow for construction of foundation elements in a dry condition. Groundwater control measures or dewatering should be carried out by a specialist contractor to draw down the groundwater level to at least 1.0 m below the base level of the excavation to ensure stable conditions during excavation.

The existing fill materials and native clayey soils are extremely easy to be disturbed and may not be able to provide a sufficient support for construction equipment. A sufficient thickness of mud slab consisting of lean concrete will have to be considered to provide a stable work plat form.

6.6 Approach Embankment Design

The final design elevations of the proposed bridge approach embankments are unknown. However, settlement may occur should any grade raise be considered for the existing embankment due to the presence of the soft to firm clayey soils.

6.6.1 Slope Geometry – Fill Less Than 4.5 m

Embankment slopes less than 4.5 m in height and constructed using local or imported fills are expected to have a sufficient factor of safety provided the slope angle is not steeper than 2H:1V and all soft/loose materials and any other deleterious materials are completely removed from the embankment areas. Earth grading should be carried out in accordance with the OPSD 200 series of specifications. To ensure adequate and uniform support throughout the pavement structure, the placement of borrow material should be carefully controlled. Mixing of materials from different sources, which could result in differential settlement, frost heave, or drainage problems, should be avoided. Vegetation should be established as early as possible to control surface erosion.

6.6.2 Slope Geometry – Fill Greater than 4.5 m or Slope Steeper than 2H:1V

Additional geotechnical evaluation of embankment stability should be carried out for fill embankments greater than 4.5 m in height or for slopes that are steeper than 2H:1V.

6.6.3 Subgrade Preparation and Embankment Construction

6.6.3.1 Removal of Organic and Other Deleterious Materials

Prior to the placement of any engineered fill for the new approach embankment construction, all topsoil, organic and other deleterious materials should be stripped from below the proposed approach embankment areas in accordance with SP206.

6.6.3.2 Embankment Settlement

Engineered fill compacted to 100% of SPMDD will settle under its own weight approximately 0.25% to 0.75% of the fill thickness. The larger settlements are expected for clayey fill materials. The designer and the structural engineer must be aware of this settlement. For example, where the engineered fill is 5 m in thickness, the settlement of fill under its own weight is expected to be in the range of 25 mm on a non-yielding subgrade. The settlement of the engineered fill will occur over a longer period of time. For engineered fill consisting of sandy silt to silty sand material, about 75% of the settlement is expected to occur within 3 months after the placement of the fill. Should clayey soils be used for the embankment, the time for the settlement to occur would be longer.

Select Subgrade Material (SSM) should be considered for the approach embankment placement in order to minimize the time for the settlement to occur. For the backfill immediately behind the integral abutments, granular back fill, such as Granular A or Granular B Type II, should be considered.

It is understood that no grade raise is anticipated for the road grade. Should this be the case, no deep seated settlement is anticipated from the underlying soft clayey soils.

It should also be noted that the settlement of the embankment at the abutment locations will cause the downdrag (negative skin friction) of the piles. In consideration of the proposed long piles, the negative friction may become significant compared with the geotechnical resistances of the piles. Therefore, the grade raise in the abutment locations should be carried out in advance and the piles shall be installed after all the settlement has ceased in order to avoid the negative skin friction of the piles.

6.6.3.3 Embankment Fill Placement

The exposed subgrade soils should be inspected by a geotechnical engineer from GeoPro prior to placement of embankment fill, proofrolled to identify soft / loosened areas, and any poorly performing areas should be subexcavated and replaced with suitable backfill.

Construction of the embankment or backfilling in subexcavated areas should be carried out using Select Subgrade Material (SSM) meeting the specifications of OPSS 1010.

Embankment fill should be placed in accordance with Special Provision SP206S03. 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.

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.

6.7 Retained Soil System (RSS) Wall and Light Weight Fill

Based on the preliminary design information provided by the Client, light weight fill, such as lightweight foam concrete fill (LFCF) is to be considered for the grade raise (up to 2.5 m) to minimize the load increase in conjunction with a Retained Soil System (RSS) wall. The RSS wall shall be designed by the specialist engineer who will have to include the geometric, performance and appearance requirements. The Retained Soil System is usually designed and installed by the engineers of a specialist contractor. The selected LFCF light weight fill shall be reviewed by the RSS wall design engineer. The final RSS wall should be reviewed by the geotechnical engineer.

The type of the retaining wall was not available at the time of preparing the report. Sliding resistance between the proposed retaining wall base and subgrade should be calculated in accordance with applicable manual(s) or guidelines. For preliminary design purposes, the coefficients of friction summarized in the following table may be assumed between the retaining wall base and the subgrade soils. This value is unfactored and a sufficient factor of safety should be applied in calculating the horizontal resistance.

Coefficient of friction between pour-in-place concrete footings and existing subgrade soils	μ= 0.30 (unfactored)
Coefficient of friction between precast concrete footings and existing subgrade soils	μ= 0.25 (unfactored)

The external modes of failure, such as overturning and global instability (circular failures), should be checked in conjunction with sliding and bearing capacity modes of failures by the engineer/designer. Global stability analysis must be carried out by the geotechnical engineer once the detailed retaining wall design is available. The required minimum factor of safety for sliding and overturning should be 1.5 and 2.0 respectively. The required minimum factor of safety for global stability should be 1.5.

Due to the low unit weight of the LFCF material which is about 20% of a regular granular fill material, friction forces underneath of the LFCF block are proportionally reduced therefore the Safety Factor against Sliding usually controls design and it is mandatory to check it by the design specialist engineer

Special attention should be paid to any high water elevations. LFCF often weights less than water and thus the entire structure will float if significantly submerged. Also any water accumulation in the backfill behind the Reinforced Earth (RE) mass is undesirable therefore a good drainage system is required.

The difference in stiffness between the LFCF and standard granular backfill materials makes necessary checking the bending capacity of panels in the interface between those materials.

The construction process of walls with LFCF differs in some aspects from the standard procedures such as formwork, leaking protection and placing seats for strips to avoid cold joints.

The design and installation of formworks shall be done to withhold the LFCF in wet state and lining with poly sheeting impermeable membrane may be required to prevent leakage. The flowable fill is commonly placed in a 600 mm to 750 mm lifts, and it should always encompass the reinforcing strips by at least 150 mm to avoid cold joints. There should be a minimum of 24 hours between lifts in order to make certain that the flowable fill has cured and gained sufficient strength.

Prior to the placement of any light weight fill, all topsoil, organic soils and other deleterious materials should be stripped from the proposed grade raise areas in accordance with SP206.

The removal of organic soils and other deleterious materials shall be monitored by a geotechnical engineer from GeoPro on a full-time basis.

The exposed subgrade soils shall be inspected by a geotechnical engineer from GeoPro prior to placement of light weight fill, proofrolled to identify soft/loosened areas, and any poorly performing areas should be subexcavated and replaced with suitable backfill.

6.8 Pavement Design

The traffic data, including the percentage of the commercial traffic, is not available. The pavement structure provided in the following table is based on the existing pavement structure and subgrade conditions encountered in the boreholes. The recommended pavement structures should be considered for preliminary design purposes only. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements. The pavement structure should also conform to the requirements of the local municipality.

Material		**Thickness of Pavement (mm)**
Hot-Mix Asphalt (OPSS 1150/ OPSS 1151)	HL 3 HS/SP12.5 FC2 Surface Course	50
	HL 8 HS/SP19.0 Binder Course	130 (2 Lifts)
Granular Material (OPSS 1010)	Granular A Base (19 mm Crusher Run Limestone)	450
Total Pavement Thickness (mm)		630
Prepared and Approved Subgrade		

The construction procedure may be considered as follows:

- Completely remove the existing topsoil, organic matter and any other obviously deleterious materials to the depth required to accommodate the new pavement structure (about 630 mm below the proposed pavement surface);
- The exposed subgrade surface should be graded and compacted to 98 percent of Standard Proctor Maximum Dry Density (SPMDD);
- The prepared subgrade should be carefully proofrolled using a heavily loaded truck in conjunction with the inspection by the geotechnical engineer from GeoPro; any soft/loose or wet areas or other obviously deleterious materials must be excavated and properly replaced with material similar to the existing subgrade soils or other granular soils approved by the geotechnical engineer;
- All backfill materials should be placed in uniform loose lifts not exceeding 200 mm thickness and compacted to at least 98 percent of SPMDD. The finished subgrade should be provided with a grade of 3 percent towards the positive drainages;
- Place a minimum 450 mm of Granular A or 19 mm Crusher Run Limestone base course in loose lifts not exceeding 200 mm thickness, compact to 100 percent of SPMDD; and
- Place 180 mm of hot-mix asphalt (130 mm of OPSS 1150 HL 8 HS binder course in two lifts and one lift of 50 mm OPSS 1150 HL 3 HS surface course; or 130 mm of OPSS 1151 SP19.0 binder course in two lifts and one lift of 50 mm OPSS SP12.5 FC2 surface course), produced and placed in accordance with OPSS 310. The surface of the completed pavement should be provided with a grade of 2 percent.

The constructed pavement Structural Number is 136, which is greater than the Design Structural Numbers (110). As such, the pavement is structurally adequate for the expected traffic loads over the 20-year design period with a regular maintenance.

6.8.1 Drainage Improvements

The provision of adequate subsurface and surface drainage is critical to the structural performance of a pavement. Drainage improvements can significantly reduce the overall structural improvements required in future. The use of properly constructed side ditch leading to a positive outlet should be considered for the section of roadway. As the existing side ditches were relatively shallow or non-existent at some locations, these side ditches should be reconstructed, with ditches cleaned of any vegetation and deepened as necessary and restored to a free-flowing condition. In this regard, proper drainage consists of well defined (and maintained) ditching to the required depth below the top of subgrade leading to a positive outlet in accordance with municipal or OPSS specifications. Pavement should be provided with a continuous centre-to-edge cross-fall of 2%.

6.8.2 General Pavement Recommendations

6.8.2.1 Pavement Materials

The following hot-mix asphalt mix types should be selected:

HL 3 HS & SP12.5FC2	Surface Course; and
HL 8 HS & SP19.0	Binder Course

These hot mix asphalt mixes should be designed and produced in conformance with OPSS 1150 requirements.

Granular A material should be used as base/subbase course and the Granular A material should meet OPSS 1010 specifications.

6.8.2.2 Asphalt Cement Grade

Performance graded asphalt cement PGAC 58-28 or 64-28 conforming to OPSS 1101 requirements is recommended for the HMA binder and surface courses.

6.8.2.3 Tack Coat

A tack coat (SS1) should be applied to all construction joints prior to placing hot mix asphalt to create an adhesive bond. Prior to placing hot mix asphalt, SS1 tack coat must also be applied to all existing surfaces and between all new lifts in accordance with OPSS 308 requirements.

6.8.2.4 Compaction

All granular base and subbase materials should be placed in uniform lifts not exceeding 200 mm loose thickness and compacted to 100 percent of the material’s SPMDD at ±2 percent of the materials Optimum Moisture Content (OMC). Hot mix asphalt should be placed and compacted in accordance with OPSS 310 specifications.

6.8.2.5 Pavement Tapers

At the limits of construction, appropriate tapering of the pavement thickness to match the existing pavement structure should be implemented in accordance with OPSS and the applicable local municipality specifications.

6.8.2.6 Subgrade Preparation

All topsoil, and any organic or other unsuitable soils should be stripped from the subgrade area. Following stripping, the site should be graded to the subgrade level and approved. The subgrade should then be proofrolled by a heavily loaded truck, in the presence of the geotechnical engineer from GeoPro. Any soft spots exposed during the proofroll should be completely removed and replaced by selected fill materials, similar to the existing subgrade soils and approved by the geotechnical engineer from GeoPro. The subgrade should then be re-compacted from the surface to at least 98% of its SPMDD. If the moisture contents of the local soil materials cannot be maintained at ±2% of the OMC, imported select materials may need to be used.

The final subgrade should be shaped properly to facilitate rapid drainage and to prevent the formation of local depressions in which water could accumulate. Proper shaping which allows the water to escape towards the sides (where it can be removed by means of subdrains or ditches) should be considered for the project. Otherwise, any water trapped in the granular base material may cause problems due to softened subgrade, and differential frost heave, etc.

Any fill materials required for re-grading the site or backfill should be free of topsoil, organic or any other unsuitable matter and must be approved by the geotechnical engineer from GeoPro. The approved fill materials should be placed in thin layers not exceeding 300 mm (uncompacted loose lift thickness) and compacted to at least 98% of its SPMDD or as per local municipal standards. The placing, spreading and rolling of the subgrade should be in accordance with OPSS or local municipal standards.

Frequent field density tests or full-time inspection should be carried out by the geotechnical engineer from GeoPro based on the project specifications or follow OPSS or local municipal standards.

6.8.2.7 Maintenance

Systematic routine preventative maintenance is strongly recommended for all newly constructed pavements. Crack routing and sealing will generally be required within 2 to 3 years after pavement construction. As the pavement ages, it will also be necessary to patch areas of medium to high severity distresses, such as potholes and ravelling. Routine maintenance should also be considered to extend the life of the pavement.

7 ENVIRONMENTAL SOIL ANALYTICAL RESULTS

7.1 Soil Sample Submission

In order to provide information on the chemical quality of the subsurface soils, selected soil samples were submitted to Paracel Environmental Laboratories ("Paracel") in Ottawa, Ontario for chemical analyses. Descriptions of the selected soil samples and analytical parameters are presented in the following table:

Sample ID	Soil Depth (mBGS)	Primary Soil	Analytical Parameters
BH1 SS1+SS2A	0.2 – 0.9	Fill: Clayey Silt and Sandy Silt to Silty Sand	M&I; PAHs; PHCs; VOCs
BH1 SS3	1.5 – 2.0	Clayey Silt to Silty Clay	M&I
BH2 SS2+SS3	0.8 – 2.0	Fill: Clayey Silt and Silty Sand	M&I; PAHs; PHCs; VOCs
BH2 SS5	3.0 – 3.5	Clayey Silt to Silty Clay	M&I

Note: M&I = Metals and Inorganics
PAHs = Polycyclic aromatic hydrocarbons
PHCs = Petroleum Hydrocarbons Fractions F1 to F4
VOCs = Volatile Organic Compounds

It should be noted that at the time of the sampling, no obvious visual or olfactory evidence of environmental impact (i.e. staining or odours) was observed at the sampling locations.

7.2 Soil Analysis Results

Selected soil samples were analysed for the parameters of M&I, PAHs, VOCs and PHCs, under Ontario Regulation 153/04 ("O. Reg. 153/04") as amended. A copy of the soil analytical results is provided in the Laboratory Certificates of Analyse, attached to Appendix C.

The soil analytical results were compared with the Ministry of the Environment, Conservation and Parks ("MECP") "Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act", April 2011, Table 1: Full Depth Background Site Condition Standards for Residential/Parkland/Institutional/Industrial/Commercial/Community Property Uses (2011 MECP Table 1 Standards); Table 2: Full Depth Generic Site Condition Standards in a Potable Ground Water Condition (2011 MECP Table 2 Standards), and Table 3: Full Depth Generic Site Condition Standards in a non-potable Ground Water Condition (2011 MECP Table 3 Standards).

Based on the comparison, exceedances of MECP Table 1, Table 2 and Table 3 standards were noted for Electrical Conductivity (EC) and Sodium Adsorption Ratio (SAR) in the tested soil samples in Borehole BH2. The exceedance values detected in the soil samples are summarized in the following table.

Soil Sample ID	Parameter	Detected Value / Unit	MECP Table 1 Standards Guideline Value	MECP Table 2 and 3 Standards (R/P/I) Guideline Value	MECP Table 2 and 3 Standards (I/C/C) Guideline Value
BH2 SS2+SS3	SAR	23.3	**2.4**	**5**	**12**
	EC	2.35	**0.57** mS/cm	**0.7** mS/cm	**1.4** mS/cm
BH2 SS5	SAR	12.5	**2.4**	**5**	**12**
	EC	2.40	**0.57** mS/cm	**0.7** mS/cm	**1.4** mS/cm

Note: R/P/I = Residential, Parkland and Institutional Property Use
I/C/C = Industrial, Commercial and Community Property Use
0.57 = standard value exceeded by the analytical result

7.3 Discussion of Analytical Results

Based on the analytical results, exceedances of MECP Table 1, Table 2 and Table 3 Standards were noted for EC and SAR in the tested soil samples. It should be noted that the samples with exceedances of EC and SAR values were taken from the borehole located on the roadway. The elevated EC and SAR values in the tested soil samples may likely be attributed to the application of de-icing salt on the road.

Based on the results of soil sample analysis, GeoPro would recommend the following disposal options:

1) The soils generated near Borehole BH1 at the tested depths with no indicated exceedances can be re-used at the Site or a receiving site would accept the soils as per the test results; and
2) The soils generated at the same tested sample depth from Borehole BH2 may be disposed at facilities, which are suitable to accept salt-impacted excess soil (i.e., certain former aggregate sites, mines, etc.) or at a licensed landfill site. However, additional chemical testing may be required by these facilities.

It should be noted that the analytical results of the chemical test refer only to the soil samples tested, which were obtained from specific sampling locations and sampling depths, and that the soil chemistry may vary between and beyond the location and depth of the samples taken. Therefore, soil materials to be used on site or transported to other sites must be inspected during excavation for indication of variance in composition or any chemical/environmental constraints. If conditions indicate significant variations, further chemical testing should be carried out.

Please note that the level of testing outlined herein is meant to provide a broad indication of soil quality based on the limited soil samples tested. The analytical results contained in this report should not be considered a warranty with respect to the soil quality or the use of the soil for any specific purpose. Furthermore, it must be noted that our scope of work was only limited to the review of the analytical results of the limited number of samples. The scope of work did not include any environmental evaluation or assessment of the subject site (such as a Phase One or Phase Two Environmental Site Assessment).

Sites accepting fill may have requirements relating to its aesthetic or engineering properties in addition to its chemical quality. Some receiving sites may have specific chemical testing protocols, which may require additional tests to meet the requirements. The requirements for accepting the fill at an off-site location must be confirmed in advance. GeoPro would be pleased to assist once the receiving sites are determined and the requirements of the receiving sites are available.

8 MONITORING AND TESTING

The geotechnical aspects of the final design drawings and specifications should be reviewed by GeoPro prior to tendering and construction, to confirm that the intent of this report has been met. During construction, full-time engineered fill monitoring and sufficient foundation inspections, subgrade inspections, in-situ density tests and materials testing should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications.

9 CLOSURE

We appreciate the opportunity to be of service to you and trust that this report provides sufficient geotechnical engineering information to facilitate the detailed design of this project. We look forward to providing you with continuing service during the construction stage. Please do not hesitate to contact our office should you wish to discuss, in further detail, any aspects of this project.

GEOPRO CONSULTING LIMITED

Dylan Q. Xiao, M.A.Sc., P.Eng.
Geotechnical Group

David B. Liu

David B. Liu, P.Eng., Principal

LICENSED PROFESSIONAL ENGINEER
D. B. LIU
100107874
20190802
PROVINCE OF ONTARIO

DRAWINGS

Legend:

⯐ **Borehole Location**

Client:	**Associated Engineering (Ontario) Limited**	Project No.: **18-2552G**	Drawing No.: **1**
Drawn: **HA**	Approved: **DL**	Title:	**Borehole Location Plan**
Date: **March 2019**	Scale: **N.T.S**	Project:	**Geotechnical Investigation** **Proposed Replacement of Saint Paul West CNR Bridge** **St. Paul West over CNR Rail, City of St. Catharines,** **Niagara Region, Ontario**
Original Size: **Letter**	Rev: **YZ**		***GeoPro* Consulting Limited**

ENCLOSURES

Enclosure 1A: Notes on Sample Descriptions

1. Each soil stratum is described according to the *Modified Unified Soil Classification System*. The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined according to Canadian Foundation Engineering Manual, 4th Edition. Different soil classification systems may be used by others. Please note that a description of the soil stratums is based on visual and tactile examination of the samples augmented with field and laboratory test results, such as a grain size analysis and/or Atterberg Limits testing. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Enclosure 1B: Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
NR	No recovery
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

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).

PM – Samples advanced by manual pressure
WR – Samples advanced by weight of sampler and rod
WH – Samples advanced by static weight of hammer

Dynamic Cone 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).

Piezo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60 degree conical tip and a projected end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurement of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

Textural Classification of Soils (ASTM D2487)

Classification	Particle Size
Boulders	> 300 mm
Cobbles	75 mm - 300 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm(*)

(*) Canadian Foundation Engineering Manual (4th Edition)

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Compactness Condition (Formerly Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
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
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

Enclosure 1C: Explanation of Terms Used in the Rock Core Logs

Strength (ISRM)

Term	Grade	Description	Uniaxial Compressive Strength (MPa)	(psi)
Extremely weak rock	RO	Indented by thumbnail	0.25-1.0	36-145
Very weak	R1	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0	145-725
Weak rock	R2	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25	725-3625
Medium Strong	R3	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50	3625-7250
Strong rock	R4	Specimen require more than one blow of geological hammer to fracture it	50-100	7250-14500
Very strong rock	R5	Specimen requires many blows of geological hammer to fracture it	100-250	14500-36250
Extremely strong rock	R6	Specimen can only be chipped with geological hammer	>250	>36250

Bedding

Term	Bed Thickness	
Very thickly bedded	>2 m	>6.5 ft
Thickly bedded	600 mm-2 m	2.00-6.50 ft
Medium bedded	200 mm-600 mm	0.65-2.00 ft
Thinly bedded	60 mm-200 mm	0.20-0.65 ft
Very thinly bedded	20 mm-60 mm	0.06-0.20 ft
Laminated	6 mm-20 mm	0.02-0.06 ft
Thinly laminated	<6 mm	<0.02 ft

TCR (Total Core Recovery)

Sum of lengths of rock core recovered from a core run, divided by the length of the core run and expressed as a percentage.

SCR (Solid Core Rocovery)

Sum length of solid, full diameter drill core recovered expressed as a percentage of the total length of the core run.

RQD (Rock Quality Designation, after Deere, 1968)

Sum of lengths of pieces of rock core measured along centreline of core equal to or greater than 100 mm from a core run, divided by the length of the core run and expressed as a percentage. Core fractured by drilling is considered intact. RQD normally quoted for N-size or H-size core.

RQD(%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very poor

Weathering (ISRM)

Term	Grade	Description
Fresh	W1	No visible sign of rock material weathering
Slightly weathered	W2	Discolouration indicates weathering of rock material and discontinuity surface. All the rock material may be discoloured by weathering and may be somewhat weaker than in its fresh condition
Moderately weathered	W3	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a either as a continuous framework or as corestones
Highly weathered	W4	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones
Completely weathered	W5	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact
Residual soil	W6	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported

(FI) Fracture Index

Expressed as the number of discontinuities per 300mm (1 ft). Excludes drill-induced fractures and fragmented zones. Reported as “>25" if frequency exceeds 25 fractures/0.3m.

Broken Zone

Zone of full diameter core of very low RQD which may include some drill-induced fractures.

Fragmented Zone

Zone where core is less than full diameter and RQD = 0.

Discontinuity Spacing (ISRM)

Term	Average Spacing	
Extremely widely spaced	>6 m	>20.00 ft
Very widely spaced	2 m-6 m	6.50-20.00 ft
Widely spaced	600 mm-2 m	2.00-6.50 ft
Moderately spaced	200 mm-600 mm	0.65-2.00 ft
Closely spaced	60 mm-200 mm	0.20-0.65 ft
Very closely spaced	20 mm-60 mm	0.06-0.20 ft
Extremely closely spaced	<20 mm	>0.06 ft

Note: Excludes drill-induced fractures and fragmented rock.

Discontinuity Orientation

Discontinuity, fracture and bedding plane orientations are cited as the acute angle measured with respect to the core axis. Fractures perpendicular to the core axis are at 90° and those parallel to the core axis are at 0°.

LOG OF BOREHOLE BH1

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge
CLIENT: Associated Engineering (Ont.) Ltd.
PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA

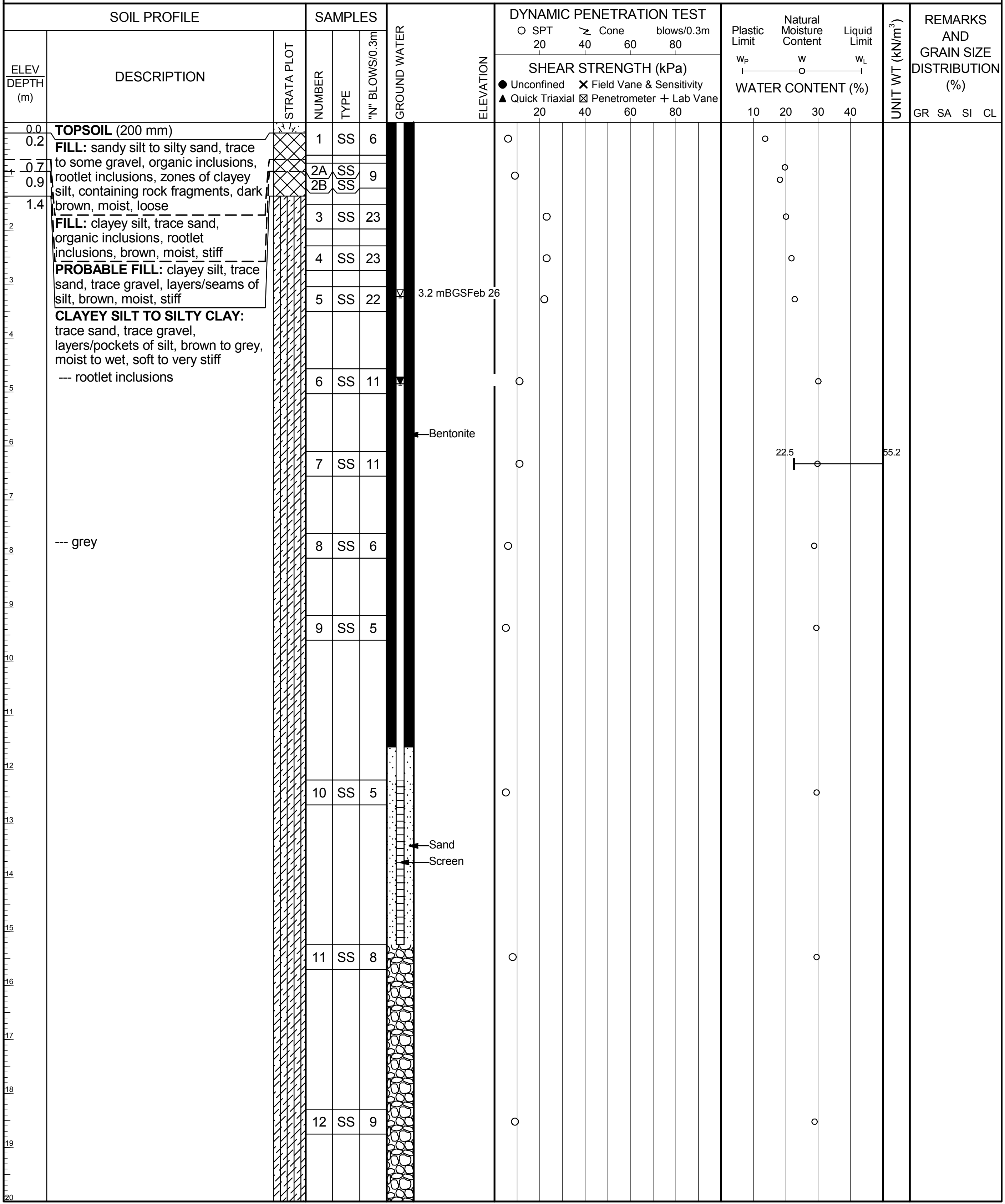
METHOD: Continuous Flight Auger - Auto Hammer
FIELD ENGINEER: HR
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 205 mm
DATE: 2019-02-13 to 15
REF. NO.: 18-2552G
ENCL. NO.: 2

SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION
0.0	**TOPSOIL** (200 mm)
0.2	**FILL:** sandy silt to silty sand, trace to some gravel, organic inclusions, rootlet inclusions, zones of clayey silt, containing rock fragments, dark brown, moist, loose
0.7	**FILL:** clayey silt, trace sand, organic inclusions, rootlet inclusions, brown, moist, stiff
0.9	**PROBABLE FILL:** clayey silt, trace sand, trace gravel, layers/seams of silt, brown, moist, stiff
1.4	**CLAYEY SILT TO SILTY CLAY:** trace sand, trace gravel, layers/pockets of silt, brown to grey, moist to wet, soft to very stiff --- rootlet inclusions --- grey

SAMPLES

NUMBER	TYPE	"N" BLOWS/0.3m
1	SS	6
2A	SS	9
2B	SS	
3	SS	23
4	SS	23
5	SS	22
6	SS	11
7	SS	11
8	SS	6
9	SS	5
10	SS	5
11	SS	8
12	SS	9



Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3, \times^3$: Numbers refer to Sensitivity
▲ ε=3% Strain at Failure

LOG OF BOREHOLE BH1

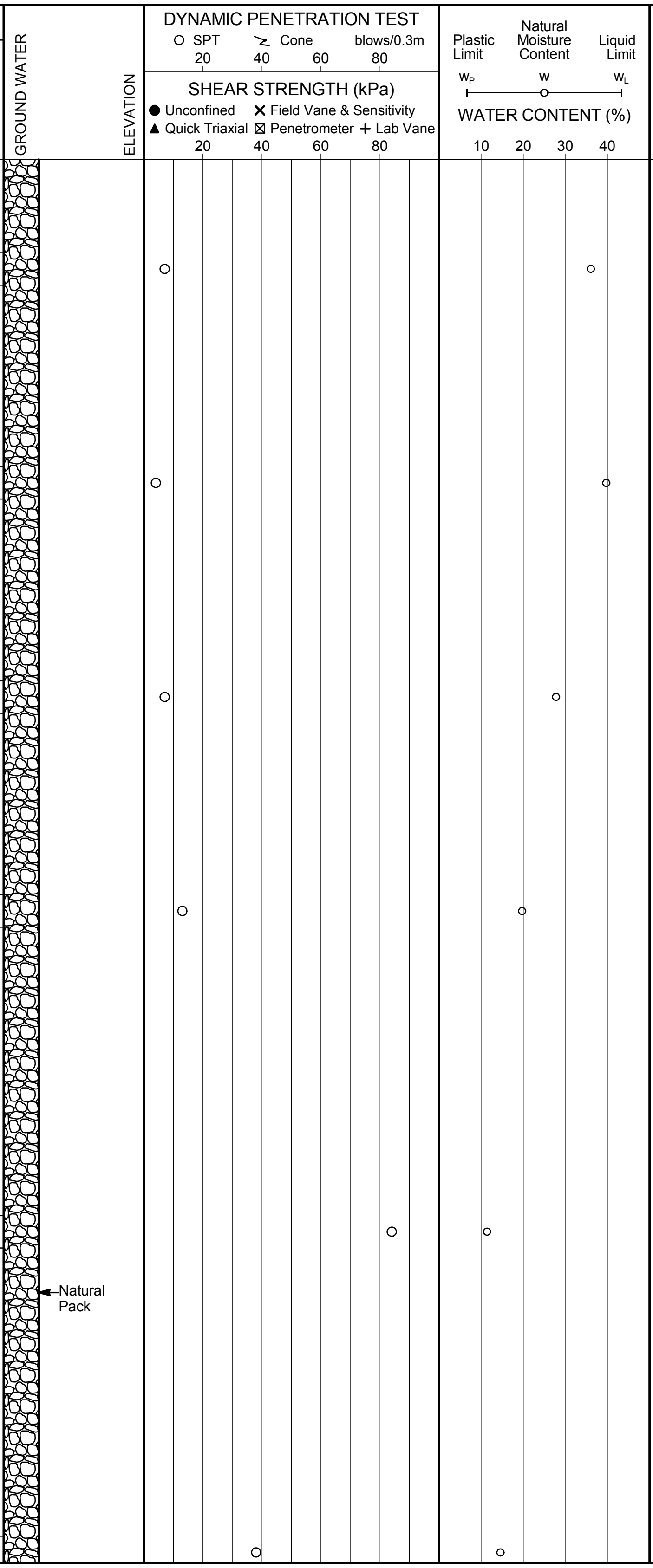
PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge

DRILLING DATA

CLIENT: Associated Engineering (Ont.) Ltd.
METHOD: Continuous Flight Auger - Auto Hammer
DIAMETER: 205 mm
PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario
FIELD ENGINEER: HR
DATE: 2019-02-13 to 15
DATUM: N/A
SAMPLE REVIEW: DX
REF. NO.: 18-2552G
BH LOCATION: See Borehole Location Plan
CHECKED: DL
ENCL. NO.: 2

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3m
	CLAYEY SILT TO SILTY CLAY: trace sand, trace gravel, layers/pockets of silt, brown to grey, moist to wet, soft to very stiff(Continued)			
		13	SS	7
		14	SS	4
	--- zones of silt	15	SS	7
		16	SS	13
34.1	**SANDY SILT TILL:** trace to some clay, trace gravel, layers of sandy silt, layers of silty sand, containing cobbles and boulders, reddish grey, moist, very dense			
		17	SS	84
38.1	**FINE SAND AND SILT TO SILT:** trace clay, layers/zones of clayey silt, grey, wet, dense			
		18	SS	38

Depth scale (m): 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40



UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ▲ ε=3% Strain at Failure

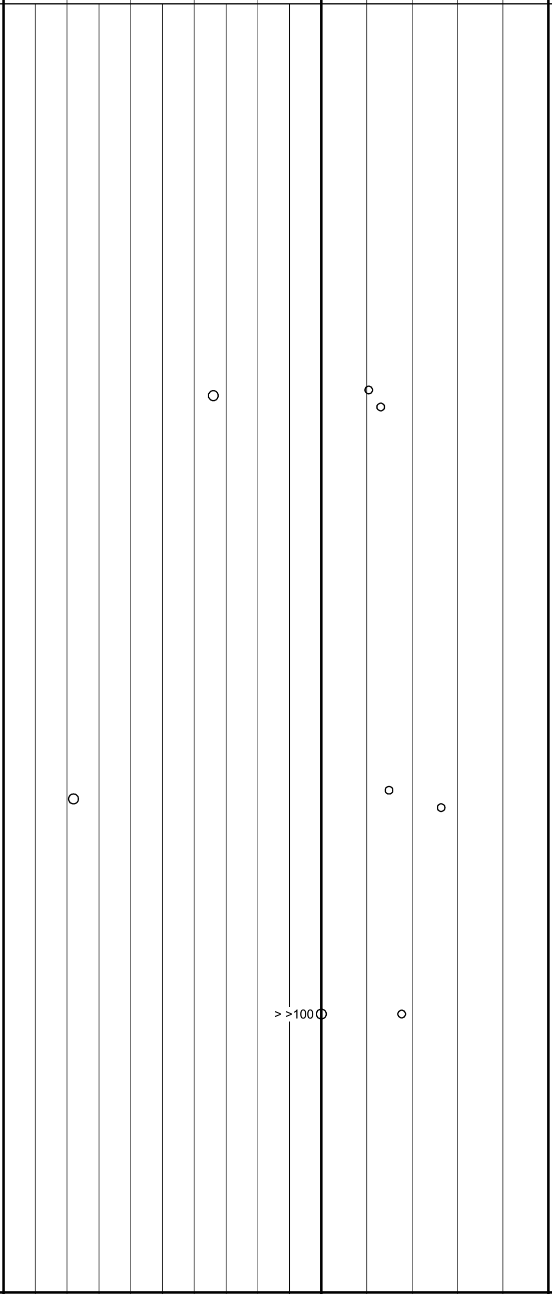
LOG OF BOREHOLE BH1

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge
CLIENT: Associated Engineering (Ont.) Ltd.
PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger - Auto Hammer
FIELD ENGINEER: HR
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 205 mm
DATE: 2019-02-13 to 15
REF. NO.: 18-2552G
ENCL. NO.: 2

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3m
	FINE SAND AND SILT TO SILT: trace clay, layers/zones of clayey silt, grey, wet, dense(Continued)			
44.5	**SILTY SAND AND GRAVEL:** trace clay, containing shale fragments, cobbles and boulders, reddish brown, wet, very dense	19A 19B	SS SS	66
45.3	**CLAYEY SILT:** some sand, trace gravel, layers/zones of silt, layers of sand and gravel, containing shale fragments, reddish grey, moist to wet, hard			
48.8	**SILT:** trace to some clay, trace sand, seams of clayey silt, reddish brown, wet, compact	20A 20B	SS SS	22
50.5	**SILTY CLAY:** trace sand, layers of silt, grey, moist, very stiff			
51.2	**QUEENSTON FORMATION:** interbedded with siltstone, reddish brown (see the following rock core log)	21	SS	100/ 50 mm
57.1	**END OF BOREHOLE**			

DYNAMIC PENETRATION TEST: ○ SPT, Cone, blows/0.3m (20, 40, 60, 80)
SHEAR STRENGTH (kPa): ● Unconfined, ✕ Field Vane & Sensitivity, ▲ Quick Triaxial, ⊠ Penetrometer, + Lab Vane (20, 40, 60, 80)
Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L; WATER CONTENT (%) (10, 20, 30, 40)
UNIT WT (kN/m³)
REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL



Notes:
1) Water encountered at a depth of 7.6 m below ground surface (mBGS) during drilling.
2) Water level was not measured on completion of drilling due to use of mud.
3) 51 mm dia. monitoring well was installed in borehole upon

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: +³, ✕³: Numbers refer to Sensitivity; ▲ ε=3% Strain at Failure

LOG OF BOREHOLE BH1

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge
CLIENT: Associated Engineering (Ont.) Ltd.
PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger - Auto Hammer
FIELD ENGINEER: HR
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 205 mm
DATE: 2019-02-13 to 15
REF. NO.: 18-2552G
ENCL. NO.: 2

SOIL PROFILE			SAMPLES					DYNAMIC PENETRATION TEST ○ SPT ≯ Cone blows/0.3m 20 40 60 80 SHEAR STRENGTH (kPa) ● Unconfined ✕ Field Vane & Sensitivity ▲ Quick Triaxial ⊠ Penetrometer + Lab Vane 20 40 60 80	Plastic Limit w_P Natural Moisture Content w Liquid Limit w_L WATER CONTENT (%) 10 20 30 40		REMARKS AND GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3m	GROUND WATER	ELEVATION			UNIT WT (kN/m^3)	GR SA SI CL
	completion of drilling. Water Level Readings Date W.L. Depth (mBGS) Feb. 26, 2019 3.23 Mar. 5, 2019 4.85										

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity
▲ ε=3% Strain at Failure

LOG OF ROCK CORE BH1

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge

CLIENT: Associated Engineering (Ont.) Ltd.

LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario

DATUM: N/A

BH LOCATION: See Borehole Location Plan

DRILLING DATA

Method: Continuous Flight Auger - Auto Hammer

Diameter: 205 mm

Date: Feb/13 to 15/2019

REF. NO.: 18-2552G

ENCL NO.: 2

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE	TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)*	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm³) E (GPa)
51.2	Rock Surface															
53.3	**QUEENSTON FORMATION:** highly weathered to fresh, laminated to thickly bedded, reddish brown, extremely weak to medium strong, horizontally **SHALE** (75% to 99%), interbedded with laminated to thinly bedded, greenish grey, weak to strong **SILTSTONE** (1% to 25%). Siltstone (hard) layers generally less than 50 mm thick except at the following depths: Depth(m) Thickness(mm) 53.81 130 56.04 130		1	NQ	85	79	25	67	>25 1	highly weathered to fresh - highly weathered layer at 53.34 m - 53.45 m (110 mm)			27.59 34.65 22.57 22.91 26.05	1.26 7.69 9.48	24.91	2.66
54.0			2	NQ	100	86	1	72	5 5 2 0 2	slightly weathered to fresh - containing clast of calcite - gypsum vein at 54.64 m and 55.19 m - highly weathered layers at 54.25 m - 54.27 m (20 mm), 54.27 m - 54.28 m (10 mm), 54.30 m - 54.31 m (10 mm) and 54.41 m - 54.42 m (10 mm) - fragmented zone at 54.46 m - 54.48 m (20 mm)			22.03 14.44 17.03 7.91 6.60 21.57 4.76 16.00 20.04 23.61 17.15	5.13 1.37 0.63 6.53 10.53	18.51 27.46 11.93 27.23	2.66 2.64 2.60 2.64
55.5			3	NQ	100	98	13	92	5 0 0 0 0	slightly weathered to fresh - containing clast of calcite - gypsum vein at 55.83 m and 56.97 m - fragmented zone at 55.59 m - 55.60 m (10 mm)			6.76 20.73 25.94 21.03 27.98 25.80 43.37 62.10 21.07 30.16 36.85	6.95 18.32 7.06 4.00	37.74 30.17	2.67 2.64
57.1													33.16 38.37 31.48 17.13 29.52			

Depth scale: 52, 53, 54, 55, 56, 57

Weathering Index: W1-Fresh, W2-Slightly weathered, W3-Moderately weathered, W4-Highly weathered, W5-Completely weathered

θ = angle to the core axis

E = Modulus of Elasticity

*: UCS [Mpa] ≈ 24 $I_{S(50)}$

LOG OF BOREHOLE BH2

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge

DRILLING DATA

CLIENT: Associated Engineering (Ont.) Ltd.

METHOD: Continuous Flight Auger - Auto Hammer

DIAMETER: 205 mm

PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario

FIELD ENGINEER: HR

DATE: 2019-01-16 to 18 & 2-13

DATUM: N/A

SAMPLE REVIEW: DX

REF. NO.: 18-2552G

BH LOCATION: See Borehole Location Plan

CHECKED: DL

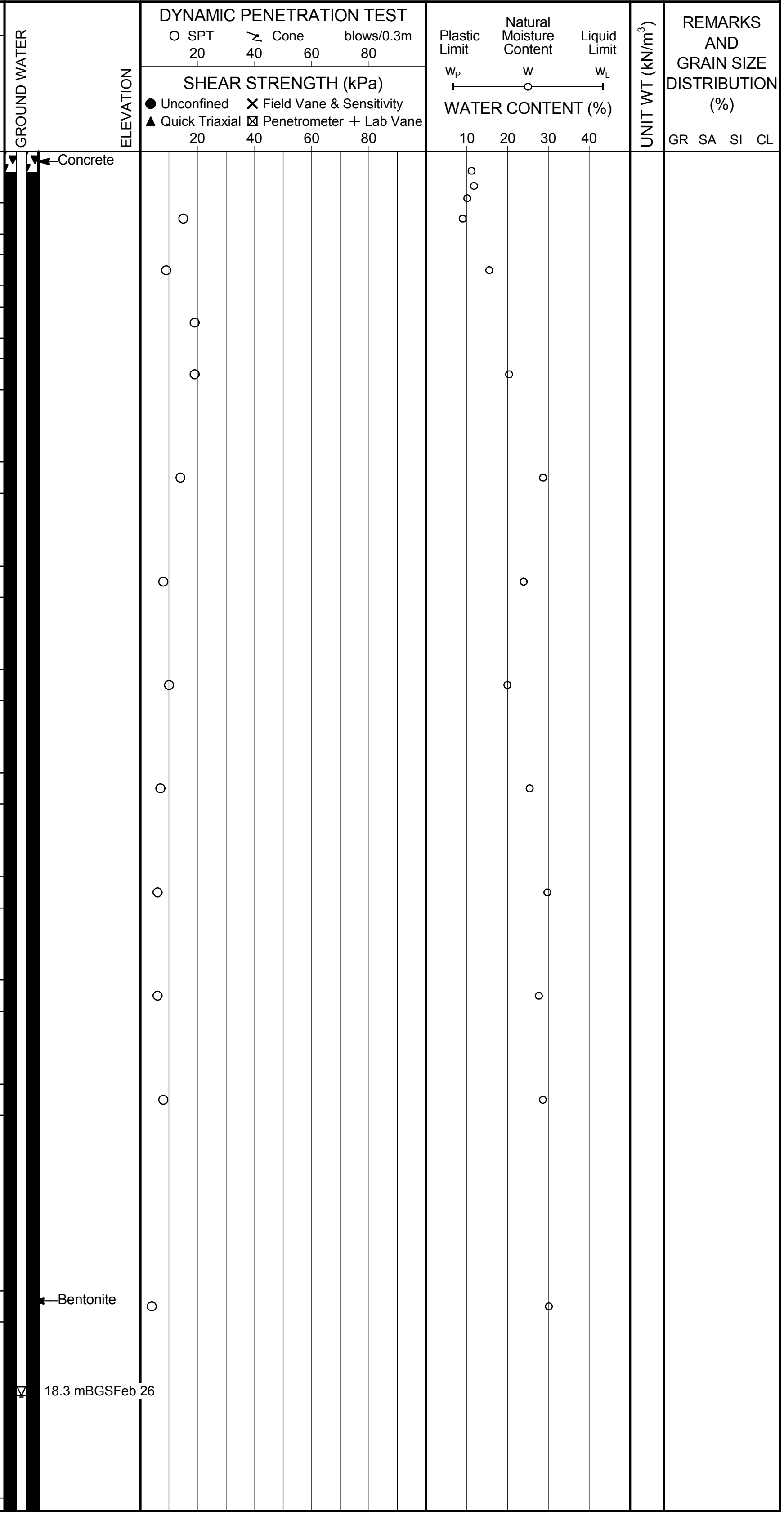
ENCL. NO.: 3

SOIL PROFILE

ELEV / DEPTH (m)	DESCRIPTION
0.0	**ASPHALT:** (180 mm)
0.2	**GRANULAR BASE:** (220 mm)
0.4	**GRANULAR SUBBASE:** (220 mm)
0.6 0.8	**FILL:** clayey silt, trace sand, organic inclusions, brown, moist
1.4	**FILL:** silty sand, trace gravel, layers of sand and silt, brown, moist, compact
2.1	**FILL:** clayey silt, trace sand, layers/zones of sandy silt, pockets of sand, brown, moist, stiff **NO SAMPLE RECOVERY DUE TO PROBABLE COBBLES AND BOULDERS:** likely fill clayey silt
2.9	**CLAYEY SILT TO SILTY CLAY:** trace sand, trace gravel, layers of silt, seams of sand, containing shale fragments, brown to grey, moist to wet, soft to very stiff --- rootlet inclusions --- rootlet inclusions --- grey

SAMPLES

NUMBER	TYPE	"N" BLOWS/0.3m
1A	AS	
1B	AS	
1C	AS	
2	SS	15
3	SS	9
4	NR	19
5	SS	19
6	SS	14
7	SS	8
8	SS	10
9	SS	7
10	SS	6
11	SS	6
12	SS	8
13	SS	4



Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity

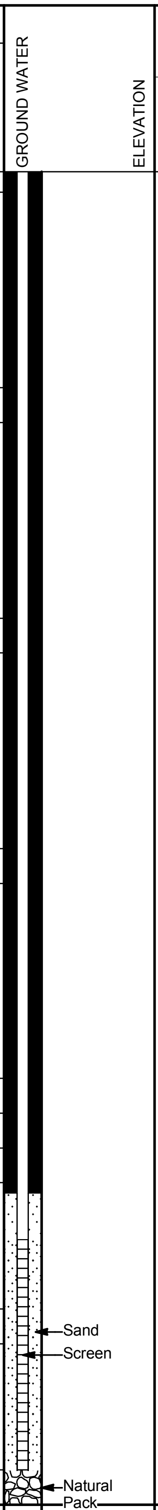
▲ ε=3% Strain at Failure

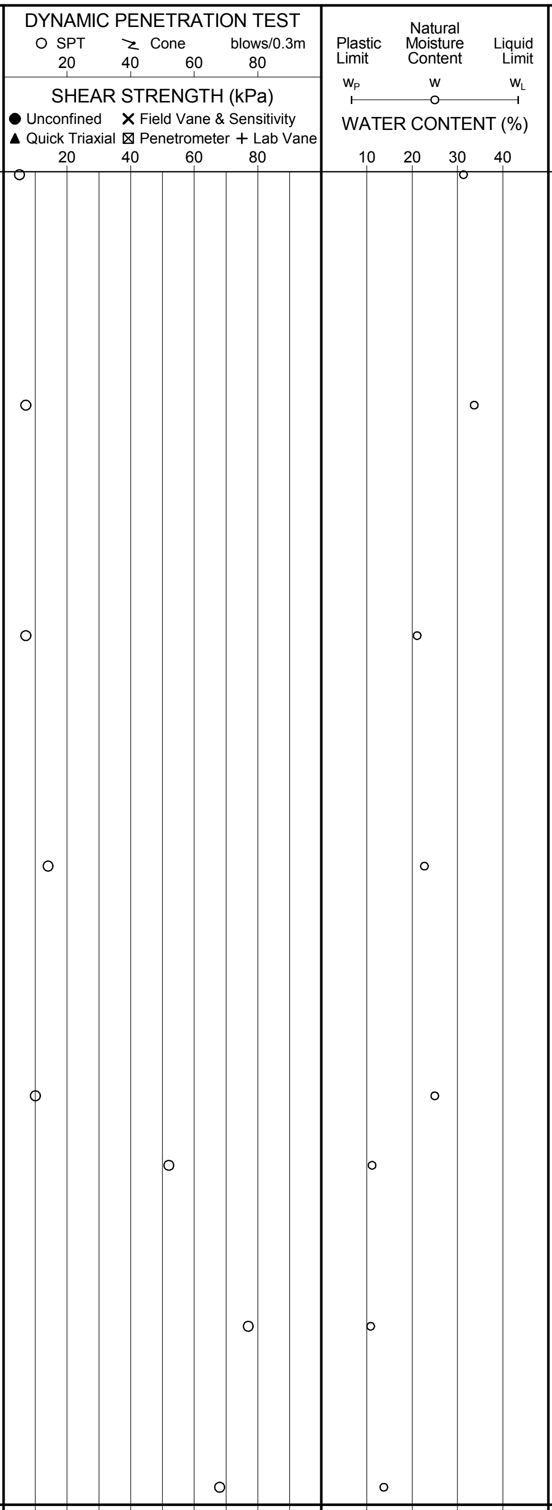
LOG OF BOREHOLE BH2

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge
CLIENT: Associated Engineering (Ont.) Ltd.
PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger - Auto Hammer
FIELD ENGINEER: HR
SAMPLE REVIEW: DX
CHECKED: DL
DIAMETER: 205 mm
DATE: 2019-01-16 to 18 & 2-13
REF. NO.: 18-2552G
ENCL. NO.: 3

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT TO SILTY CLAY: trace sand, trace gravel, layers of silt, seams of sand, containing shale fragments, brown to grey, moist to wet, soft to very stiff(Continued)	14	SS	5	
		15	SS	7	
	--- zones of silt	16	SS	7	
		17	SS	14	
		18	SS	10	
32.7	**SANDY SILT TILL:** trace to some clay, trace gravel, seams/layers of clayey silt, layers of sandy silt, layers of silty sand, containing shale fragments, cobbles and boulders, reddish grey, moist, very dense	19	SS	52	8 20 57 15
	--- zones of silty sand	20	SS	77	
36.0	**SANDY SILT:** some clay, trace gravel, seams/layers of clayey silt, layers of silty sand, layers of gravelly sand, reddish grey to grey, moist to wet, very dense	21	SS	68	6 27 52 15
37.6	**END OF BOREHOLE**				





Notes:
1) Water encountered at a depth of 9.0 m below ground surface (mBGS) during drilling.
2) Water level was not measured on completion of drilling due to use

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity
▲ ε=3% Strain at Failure

LOG OF BOREHOLE BH2

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge
CLIENT: Associated Engineering (Ont.) Ltd.
PROJECT LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario
DATUM: N/A
BH LOCATION: See Borehole Location Plan

DRILLING DATA
METHOD: Continuous Flight Auger - Auto Hammer
FIELD ENGINEER: HR
SAMPLE REVIEW: DX
CHECKED: DL

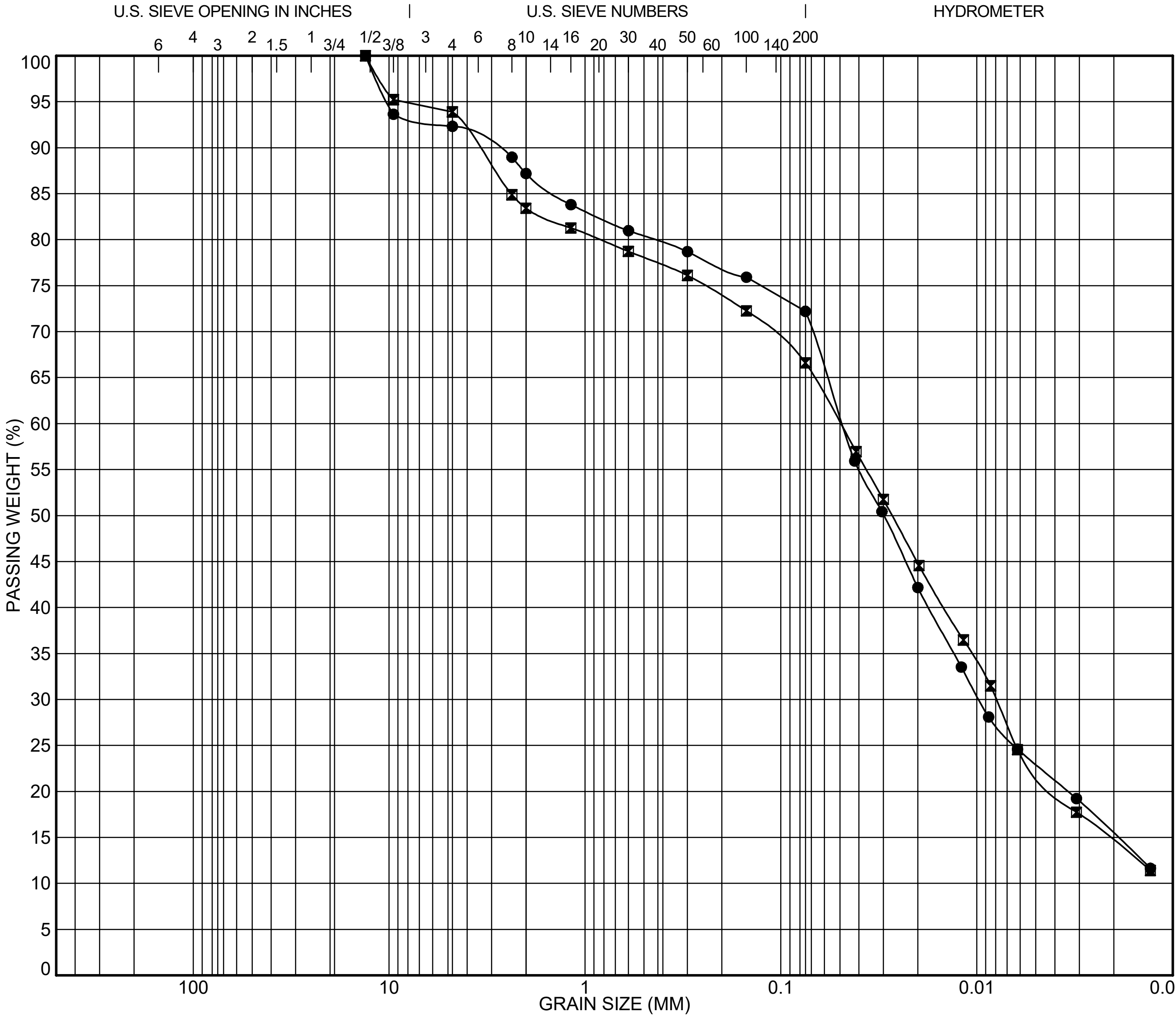
DIAMETER: 205 mm
DATE: 2019-01-16 to 18 & 2-13
REF. NO.: 18-2552G
ENCL. NO.: 3

SOIL PROFILE			SAMPLES			GROUND WATER	ELEVATION	DYNAMIC PENETRATION TEST ○ SPT ⩘ Cone blows/0.3m 20 40 60 80 SHEAR STRENGTH (kPa) ● Unconfined ✕ Field Vane & Sensitivity ▲ Quick Triaxial ⊠ Penetrometer + Lab Vane 20 40 60 80	Plastic Limit w_P / Natural Moisture Content w / Liquid Limit w_L WATER CONTENT (%) 10 20 30 40	UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3m						
	of mud. 3) 38 mm dia. monitoring well was installed in borehole upon completion of drilling. Water Level Reading Date W.L. Depth (mBGS) Feb. 26, 2019 18.3										

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
▲ ε=3% Strain at Failure

FIGURES



COBBLES	GRAVEL		SAND			SILT	CLAY
	coarse	fine	coarse	medium	fine		

	Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**BH2**	**SS19**	**32.92**						
☒	**BH2**	**SS21**	**37.18**						

	Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH2**	**SS19**	**32.92**	**13.2**	**0.049**	**0.01**		**7.7**	**20.1**	**56.8**	**15.4**
☒	**BH2**	**SS21**	**37.18**	**13.2**	**0.05**	**0.008**		**6.1**	**27.3**	**52.1**	**14.5**

GeoPro
CONSULTING LIMITED

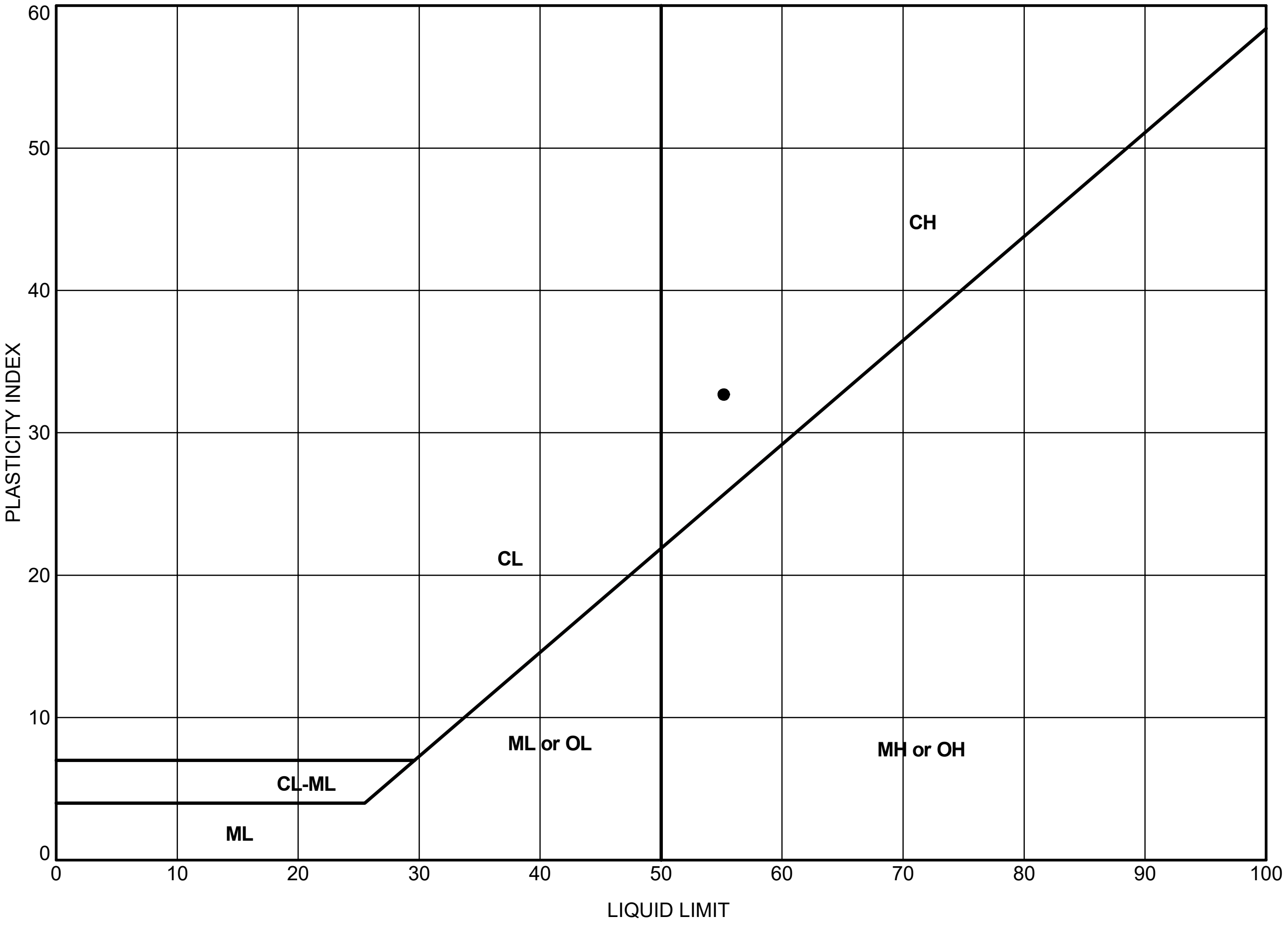
Unit 57, 40 Vogell Road, Richmond Hill, Ontario L4B 3N6
Tel: 905-237-8336 Fax: 905-248-3699
office@geoproconsulting.ca www.geoproconsulting.ca

GRAIN SIZE DISTRIBUTION

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge

LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario

PROJECT NO.: 18-2552G	SAMPLED ON: 2019-01-16
FIGURE NO.: 1	TESTED ON: 2019-03-05



	Specimen Identification			LL	PL	PI	Fines	Classification
●	**BH1**	**SS7**	**6.10**	**55**	**22**	**33**		

GeoPro
CONSULTING LIMITED

Unit 57, 40 Vogell Road, Richmond Hill, Ontario L4B 3N6
Tel: 905-237-8336 Fax: 905-248-3699
office@geoproconsulting.ca www.geoproconsulting.ca

ATTERBERG LIMITS' RESULTS

PROJECT: Geotechnical Investigation for Proposed Replacement of St. Paul West CNR Bridge

LOCATION: St. Paul West, St. Catharines, Niagara Region, Ontario

PROJECT NO.: 18-2552G

SAMPLED ON: 2019-02-13

FIGURE NO.: 2

TESTED ON: 2019-03-05

APPENDIX A

Photo 1: Borehole BH1 Rock Core
Run 1: 175’0” – 177’2” (53.34 m – 54.00 m)
Run 2: 177’2” – 182’0” (54.00 m – 55.47 m)

Photo 2: Borehole BH1 Rock Core
Run 3: 182’0” – 187’2” (55.47 m – 57.05 m)

APPENDIX B

Summary of Unconfined Compressive Strength of Rock
ASTM D 7012

Project No. 18-2552G

Project: Proposed Replacement of Saint Paul West CNR Bridge

BH No	Sample Depth (m)	Bulk Density (g/cm^3)	UCS (MPa)	Rock Type
BH1	53.66 – 53.89	2.66	24.91	Queenston Formation
	54.00 – 54.12	2.66	18.51	Queenston Formation
	54.70 – 54.83	2.64	27.46	Queenston Formation
	55.05 – 55.20	2.60	11.93	Queenston Formation
	55.24 – 55.41	2.64	27.23	Queenston Formation
	55.92 – 56.11	2.64	37.74	Queenston Formation
	56.20 – 56.62	2.67	30.17	Queenston Formation

APPENDIX C

PARACEL LABORATORIES LTD. | TRUSTED. RESPONSIVE. RELIABLE.

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

GeoPro Consulting Limited

40 Vogell Road, Unit 57
Richmond Hill, ON L4B 3N6
Attn: Sarena Medina

Client PO:
Project: 18-2552G
Custody:

Report Date: 1-Mar-2019
Order Date: 25-Feb-2019

Order #: 1909070

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1909070-01	BH1 SS1+SS2A
1909070-02	BH2 SS2+SS3
1909070-03	BH1 SS3
1909070-04	BH2 SS5

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Boron, available	MOE (HWE), EPA 200.7 - ICP-OES	27-Feb-19	27-Feb-19
Chromium, hexavalent - soil	MOE E3056 - Extraction, colourimetric	26-Feb-19	27-Feb-19
Conductivity	MOE E3138 - probe @25 °C, water ext	28-Feb-19	28-Feb-19
Cyanide, free	MOE E3015 - Auto Colour, water extraction	25-Feb-19	27-Feb-19
Mercury by CVAA	EPA 7471B - CVAA, digestion	27-Feb-19	28-Feb-19
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	28-Feb-19	28-Feb-19
PHC F1	CWS Tier 1 - P&T GC-FID	27-Feb-19	28-Feb-19
PHCs F2 to F4	CWS Tier 1 - GC-FID, extraction	25-Feb-19	27-Feb-19
REG 153: Metals by ICP/MS, soil	EPA 6020 - Digestion - ICP-MS	27-Feb-19	27-Feb-19
REG 153: PAHs by GC-MS	EPA 8270 - GC-MS, extraction	26-Feb-19	28-Feb-19
REG 153: VOCs by P&T GC/MS	EPA 8260 - P&T GC-MS	27-Feb-19	28-Feb-19
SAR	Calculated	27-Feb-19	28-Feb-19
Solids, %	Gravimetric, calculation	28-Feb-19	28-Feb-19

Certificate of Analysis
Client: GeoPro Consulting Limited
Client PO:

Report Date: 01-Mar-2019
Order Date: 25-Feb-2019
Project Description: 18-2552G

Client ID: **Sample Date:** **Sample ID:**		BH1 SS1+SS2A 02/14/2019 09:00 1909070-01	BH2 SS2+SS3 02/13/2019 09:00 1909070-02	BH1 SS3 02/14/2019 09:00 1909070-03	BH2 SS5 02/13/2019 09:00 1909070-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	81.9	86.0	83.1	82.9
General Inorganics					
SAR	0.01 N/A	0.31	23.3	0.19	12.5
Conductivity	5 uS/cm	245	2350	192	2400
Cyanide, free	0.03 ug/g dry	<0.03	<0.03	<0.03	<0.03
pH	0.05 pH Units	7.77	7.90	7.86	7.92
Metals					
Antimony	1.0 ug/g dry	<1.0	<1.0	<1.0	<1.0
Arsenic	1.0 ug/g dry	6.5	5.2	5.3	4.8
Barium	1.0 ug/g dry	120	109	146	154
Beryllium	0.5 ug/g dry	1.2	1.1	1.1	1.2
Boron	5.0 ug/g dry	11.9	8.3	13.6	15.1
Boron, available	0.5 ug/g dry	<0.5	<0.5	<0.5	<0.5
Cadmium	0.5 ug/g dry	<0.5	<0.5	<0.5	<0.5
Chromium	5.0 ug/g dry	31.4	24.6	33.2	33.5
Chromium (VI)	0.2 ug/g dry	<0.2	<0.2	<0.2	<0.2
Cobalt	1.0 ug/g dry	13.7	11.5	15.4	15.1
Copper	5.0 ug/g dry	31.4	31.1	30.4	26.9
Lead	1.0 ug/g dry	18.2	10.8	10.5	10.2
Mercury	0.1 ug/g dry	<0.1	<0.1	<0.1	<0.1
Molybdenum	1.0 ug/g dry	<1.0	<1.0	<1.0	<1.0
Nickel	5.0 ug/g dry	32.2	24.2	34.1	33.7
Selenium	1.0 ug/g dry	<1.0	<1.0	<1.0	<1.0
Silver	0.3 ug/g dry	<0.3	<0.3	<0.3	<0.3
Thallium	1.0 ug/g dry	<1.0	<1.0	<1.0	<1.0
Uranium	1.0 ug/g dry	<1.0	<1.0	<1.0	<1.0
Vanadium	10.0 ug/g dry	42.3	35.5	44.1	44.1
Zinc	20.0 ug/g dry	77.4	58.6	70.1	67.7
Volatiles					
Acetone	0.50 ug/g dry	<0.50	<0.50	-	-
Benzene	0.02 ug/g dry	<0.02	<0.02	-	-
Bromodichloromethane	0.05 ug/g dry	<0.05	<0.05	-	-
Bromoform	0.05 ug/g dry	<0.05	<0.05	-	-
Bromomethane	0.05 ug/g dry	<0.05	<0.05	-	-
Carbon Tetrachloride	0.05 ug/g dry	<0.05	<0.05	-	-

Certificate of Analysis

Client: GeoPro Consulting Limited

Client PO:

Report Date: 01-Mar-2019

Order Date: 25-Feb-2019

Project Description: 18-2552G

Client ID: **Sample Date:** **Sample ID:**	**MDL/Units**	BH1 SS1+SS2A 02/14/2019 09:00 1909070-01 Soil	BH2 SS2+SS3 02/13/2019 09:00 1909070-02 Soil	BH1 SS3 02/14/2019 09:00 1909070-03 Soil	BH2 SS5 02/13/2019 09:00 1909070-04 Soil
Chlorobenzene	0.05 ug/g dry	<0.05	<0.05	-	-
Chloroform	0.05 ug/g dry	<0.05	<0.05	-	-
Dibromochloromethane	0.05 ug/g dry	<0.05	<0.05	-	-
Dichlorodifluoromethane	0.05 ug/g dry	<0.05	<0.05	-	-
1,2-Dichlorobenzene	0.05 ug/g dry	<0.05	<0.05	-	-
1,3-Dichlorobenzene	0.05 ug/g dry	<0.05	<0.05	-	-
1,4-Dichlorobenzene	0.05 ug/g dry	<0.05	<0.05	-	-
1,1-Dichloroethane	0.05 ug/g dry	<0.05	<0.05	-	-
1,2-Dichloroethane	0.05 ug/g dry	<0.05	<0.05	-	-
1,1-Dichloroethylene	0.05 ug/g dry	<0.05	<0.05	-	-
cis-1,2-Dichloroethylene	0.05 ug/g dry	<0.05	<0.05	-	-
trans-1,2-Dichloroethylene	0.05 ug/g dry	<0.05	<0.05	-	-
1,2-Dichloropropane	0.05 ug/g dry	<0.05	<0.05	-	-
cis-1,3-Dichloropropylene	0.05 ug/g dry	<0.05	<0.05	-	-
trans-1,3-Dichloropropylene	0.05 ug/g dry	<0.05	<0.05	-	-
1,3-Dichloropropene, total	0.05 ug/g dry	<0.05	<0.05	-	-
Ethylbenzene	0.05 ug/g dry	<0.05	<0.05	-	-
Ethylene dibromide (dibromoethan	0.05 ug/g dry	<0.05	<0.05	-	-
Hexane	0.05 ug/g dry	<0.05	<0.05	-	-
Methyl Ethyl Ketone (2-Butanone)	0.50 ug/g dry	<0.50	<0.50	-	-
Methyl Isobutyl Ketone	0.50 ug/g dry	<0.50	<0.50	-	-
Methyl tert-butyl ether	0.05 ug/g dry	<0.05	<0.05	-	-
Methylene Chloride	0.05 ug/g dry	<0.05	<0.05	-	-
Styrene	0.05 ug/g dry	<0.05	<0.05	-	-
1,1,1,2-Tetrachloroethane	0.05 ug/g dry	<0.05	<0.05	-	-
1,1,2,2-Tetrachloroethane	0.05 ug/g dry	<0.05	<0.05	-	-
Tetrachloroethylene	0.05 ug/g dry	<0.05	<0.05	-	-
Toluene	0.05 ug/g dry	<0.05	<0.05	-	-
1,1,1-Trichloroethane	0.05 ug/g dry	<0.05	<0.05	-	-
1,1,2-Trichloroethane	0.05 ug/g dry	<0.05	<0.05	-	-
Trichloroethylene	0.05 ug/g dry	<0.05	<0.05	-	-
Trichlorofluoromethane	0.05 ug/g dry	<0.05	<0.05	-	-
Vinyl chloride	0.02 ug/g dry	<0.02	<0.02	-	-
m,p-Xylenes	0.05 ug/g dry	<0.05	<0.05	-	-
o-Xylene	0.05 ug/g dry	<0.05	<0.05	-	-
Xylenes, total	0.05 ug/g dry	<0.05	<0.05	-	-

Certificate of Analysis

Client: GeoPro Consulting Limited

Client PO:

Report Date: 01-Mar-2019

Order Date: 25-Feb-2019

Project Description: 18-2552G

Client ID:		BH1 SS1+SS2A	BH2 SS2+SS3	BH1 SS3	BH2 SS5
Sample Date:		02/14/2019 09:00	02/13/2019 09:00	02/14/2019 09:00	02/13/2019 09:00
Sample ID:		1909070-01	1909070-02	1909070-03	1909070-04
	MDL/Units	Soil	Soil	Soil	Soil
4-Bromofluorobenzene	Surrogate	117%	117%	-	-
Dibromofluoromethane	Surrogate	97.9%	97.9%	-	-
Toluene-d8	Surrogate	111%	109%	-	-
Hydrocarbons					
F1 PHCs (C6-C10)	7 ug/g dry	<7	<7	-	-
F2 PHCs (C10-C16)	4 ug/g dry	<4	<4	-	-
F3 PHCs (C16-C34)	8 ug/g dry	<8	<8	-	-
F4 PHCs (C34-C50)	6 ug/g dry	<6	<6	-	-
Semi-Volatiles					
Acenaphthene	0.02 ug/g dry	<0.02	<0.02	-	-
Acenaphthylene	0.02 ug/g dry	<0.02	0.02	-	-
Anthracene	0.02 ug/g dry	0.02	0.04	-	-
Benzo [a] anthracene	0.02 ug/g dry	0.07	0.08	-	-
Benzo [a] pyrene	0.02 ug/g dry	0.06	0.06	-	-
Benzo [b] fluoranthene	0.02 ug/g dry	0.10	0.08	-	-
Benzo [g,h,i] perylene	0.02 ug/g dry	0.05	0.04	-	-
Benzo [k] fluoranthene	0.02 ug/g dry	0.05	0.04	-	-
Chrysene	0.02 ug/g dry	0.09	0.07	-	-
Dibenzo [a,h] anthracene	0.02 ug/g dry	<0.02	<0.02	-	-
Fluoranthene	0.02 ug/g dry	0.15	0.13	-	-
Fluorene	0.02 ug/g dry	<0.02	<0.02	-	-
Indeno [1,2,3-cd] pyrene	0.02 ug/g dry	0.04	0.03	-	-
1-Methylnaphthalene	0.02 ug/g dry	<0.02	<0.02	-	-
2-Methylnaphthalene	0.02 ug/g dry	<0.02	<0.02	-	-
Methylnaphthalene (1&2)	0.04 ug/g dry	<0.04	<0.04	-	-
Naphthalene	0.01 ug/g dry	<0.01	<0.01	-	-
Phenanthrene	0.02 ug/g dry	0.08	0.07	-	-
Pyrene	0.02 ug/g dry	0.13	0.12	-	-
2-Fluorobiphenyl	Surrogate	79.8%	80.8%	-	-
Terphenyl-d14	Surrogate	88.5%	94.8%	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
General Inorganics									
Conductivity	ND	5	uS/cm						
Cyanide, free	ND	0.03	ug/g						
Hydrocarbons									
F1 PHCs (C6-C10)	ND	7	ug/g						
F2 PHCs (C10-C16)	ND	4	ug/g						
F3 PHCs (C16-C34)	ND	8	ug/g						
F4 PHCs (C34-C50)	ND	6	ug/g						
Metals									
Antimony	ND	1.0	ug/g						
Arsenic	ND	1.0	ug/g						
Barium	ND	1.0	ug/g						
Beryllium	ND	0.5	ug/g						
Boron, available	ND	0.5	ug/g						
Boron	ND	5.0	ug/g						
Cadmium	ND	0.5	ug/g						
Chromium (VI)	ND	0.2	ug/g						
Chromium	ND	5.0	ug/g						
Cobalt	ND	1.0	ug/g						
Copper	ND	5.0	ug/g						
Lead	ND	1.0	ug/g						
Mercury	ND	0.1	ug/g						
Molybdenum	ND	1.0	ug/g						
Nickel	ND	5.0	ug/g						
Selenium	ND	1.0	ug/g						
Silver	ND	0.3	ug/g						
Thallium	ND	1.0	ug/g						
Uranium	ND	1.0	ug/g						
Vanadium	ND	10.0	ug/g						
Zinc	ND	20.0	ug/g						
Semi-Volatiles									
Acenaphthene	ND	0.02	ug/g						
Acenaphthylene	ND	0.02	ug/g						
Anthracene	ND	0.02	ug/g						
Benzo [a] anthracene	ND	0.02	ug/g						
Benzo [a] pyrene	ND	0.02	ug/g						
Benzo [b] fluoranthene	ND	0.02	ug/g						
Benzo [g,h,i] perylene	ND	0.02	ug/g						
Benzo [k] fluoranthene	ND	0.02	ug/g						
Chrysene	ND	0.02	ug/g						
Dibenzo [a,h] anthracene	ND	0.02	ug/g						
Fluoranthene	ND	0.02	ug/g						
Fluorene	ND	0.02	ug/g						
Indeno [1,2,3-cd] pyrene	ND	0.02	ug/g						
1-Methylnaphthalene	ND	0.02	ug/g						
2-Methylnaphthalene	ND	0.02	ug/g						
Methylnaphthalene (1&2)	ND	0.04	ug/g						
Naphthalene	ND	0.01	ug/g						
Phenanthrene	ND	0.02	ug/g						
Pyrene	ND	0.02	ug/g						
Surrogate: 2-Fluorobiphenyl	*1.39*		*ug/g*		*105*	*50-140*			
Surrogate: Terphenyl-d14	*1.48*		*ug/g*		*111*	*50-140*			
Volatiles									
Acetone	ND	0.50	ug/g						
Benzene	ND	0.02	ug/g						
Bromodichloromethane	ND	0.05	ug/g						
Bromoform	ND	0.05	ug/g						
Bromomethane	ND	0.05	ug/g						

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Carbon Tetrachloride	ND	0.05	ug/g						
Chlorobenzene	ND	0.05	ug/g						
Chloroform	ND	0.05	ug/g						
Dibromochloromethane	ND	0.05	ug/g						
Dichlorodifluoromethane	ND	0.05	ug/g						
1,2-Dichlorobenzene	ND	0.05	ug/g						
1,3-Dichlorobenzene	ND	0.05	ug/g						
1,4-Dichlorobenzene	ND	0.05	ug/g						
1,1-Dichloroethane	ND	0.05	ug/g						
1,2-Dichloroethane	ND	0.05	ug/g						
1,1-Dichloroethylene	ND	0.05	ug/g						
cis-1,2-Dichloroethylene	ND	0.05	ug/g						
trans-1,2-Dichloroethylene	ND	0.05	ug/g						
1,2-Dichloropropane	ND	0.05	ug/g						
cis-1,3-Dichloropropylene	ND	0.05	ug/g						
trans-1,3-Dichloropropylene	ND	0.05	ug/g						
1,3-Dichloropropene, total	ND	0.05	ug/g						
Ethylbenzene	ND	0.05	ug/g						
Ethylene dibromide (dibromoethane	ND	0.05	ug/g						
Hexane	ND	0.05	ug/g						
Methyl Ethyl Ketone (2-Butanone)	ND	0.50	ug/g						
Methyl Isobutyl Ketone	ND	0.50	ug/g						
Methyl tert-butyl ether	ND	0.05	ug/g						
Methylene Chloride	ND	0.05	ug/g						
Styrene	ND	0.05	ug/g						
1,1,1,2-Tetrachloroethane	ND	0.05	ug/g						
1,1,2,2-Tetrachloroethane	ND	0.05	ug/g						
Tetrachloroethylene	ND	0.05	ug/g						
Toluene	ND	0.05	ug/g						
1,1,1-Trichloroethane	ND	0.05	ug/g						
1,1,2-Trichloroethane	ND	0.05	ug/g						
Trichloroethylene	ND	0.05	ug/g						
Trichlorofluoromethane	ND	0.05	ug/g						
Vinyl chloride	ND	0.02	ug/g						
m,p-Xylenes	ND	0.05	ug/g						
o-Xylene	ND	0.05	ug/g						
Xylenes, total	ND	0.05	ug/g						
Surrogate: 4-Bromofluorobenzene	*9.18*		*ug/g*		*115*	*50-140*			
Surrogate: Dibromofluoromethane	*8.24*		*ug/g*		*103*	*50-140*			
Surrogate: Toluene-d8	*8.22*		*ug/g*		*103*	*50-140*			

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
General Inorganics									
SAR	0.25	0.01	N/A	0.26			3.9	200	
Conductivity	382	5	uS/cm	374			2.2	6.2	
Cyanide, free	ND	0.03	ug/g dry	ND				35	
pH	7.90	0.05	pH Units	7.77			1.7	10	
Hydrocarbons									
F1 PHCs (C6-C10)	ND	7	ug/g dry	ND				40	
F2 PHCs (C10-C16)	ND	4	ug/g dry	ND				30	
F3 PHCs (C16-C34)	ND	8	ug/g dry	ND				30	
F4 PHCs (C34-C50)	ND	6	ug/g dry	ND				30	
Metals									
Antimony	1.4	1.0	ug/g dry	ND			0.0	30	
Arsenic	12.0	1.0	ug/g dry	10.2			16.1	30	
Barium	27.3	1.0	ug/g dry	23.2			16.3	30	
Beryllium	0.8	0.5	ug/g dry	ND			0.0	30	
Boron, available	0.50	0.5	ug/g dry	0.52			3.4	35	
Boron	9.0	5.0	ug/g dry	7.4			19.1	30	
Cadmium	0.6	0.5	ug/g dry	ND			0.0	30	
Chromium (VI)	ND	0.2	ug/g dry	ND				35	
Chromium	13.9	5.0	ug/g dry	12.6			10.0	30	
Cobalt	6.0	1.0	ug/g dry	5.0			19.0	30	
Copper	24.1	5.0	ug/g dry	20.8			14.7	30	
Lead	11.4	1.0	ug/g dry	9.7			16.5	30	
Mercury	ND	0.1	ug/g dry	ND			0.0	30	
Molybdenum	2.1	1.0	ug/g dry	1.5			32.8	30	QR-01
Nickel	13.6	5.0	ug/g dry	12.1			11.3	30	
Selenium	ND	1.0	ug/g dry	ND			0.0	30	
Silver	0.4	0.3	ug/g dry	ND			0.0	30	
Thallium	ND	1.0	ug/g dry	ND			0.0	30	
Uranium	1.3	1.0	ug/g dry	ND			0.0	30	
Vanadium	27.3	10.0	ug/g dry	23.8			13.5	30	
Zinc	82.0	20.0	ug/g dry	65.5			22.3	30	
Physical Characteristics									
% Solids	88.6	0.1	% by Wt.	87.6			1.2	25	
Semi-Volatiles									
Acenaphthene	ND	0.02	ug/g dry	ND				40	
Acenaphthylene	ND	0.02	ug/g dry	ND				40	
Anthracene	ND	0.02	ug/g dry	ND				40	
Benzo [a] anthracene	ND	0.02	ug/g dry	ND				40	
Benzo [a] pyrene	ND	0.02	ug/g dry	ND				40	
Benzo [b] fluoranthene	ND	0.02	ug/g dry	ND				40	
Benzo [g,h,i] perylene	ND	0.02	ug/g dry	ND				40	
Benzo [k] fluoranthene	ND	0.02	ug/g dry	ND				40	
Chrysene	ND	0.02	ug/g dry	ND				40	
Dibenzo [a,h] anthracene	ND	0.02	ug/g dry	ND				40	
Fluoranthene	ND	0.02	ug/g dry	ND				40	
Fluorene	ND	0.02	ug/g dry	ND				40	
Indeno [1,2,3-cd] pyrene	ND	0.02	ug/g dry	ND				40	
1-Methylnaphthalene	ND	0.02	ug/g dry	ND				40	
2-Methylnaphthalene	ND	0.02	ug/g dry	ND				40	
Naphthalene	ND	0.01	ug/g dry	ND				40	
Phenanthrene	ND	0.02	ug/g dry	ND				40	
Pyrene	ND	0.02	ug/g dry	ND				40	
Surrogate: 2-Fluorobiphenyl	*1.49*		*ug/g dry*		*97.9*	*50-140*			
Surrogate: Terphenyl-d14	*1.61*		*ug/g dry*		*106*	*50-140*			
Volatiles									
Acetone	ND	0.50	ug/g dry	ND				50	

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Benzene	ND	0.02	ug/g dry	ND				50	
Bromodichloromethane	ND	0.05	ug/g dry	ND				50	
Bromoform	ND	0.05	ug/g dry	ND				50	
Bromomethane	ND	0.05	ug/g dry	ND				50	
Carbon Tetrachloride	ND	0.05	ug/g dry	ND				50	
Chlorobenzene	ND	0.05	ug/g dry	ND				50	
Chloroform	ND	0.05	ug/g dry	ND				50	
Dibromochloromethane	ND	0.05	ug/g dry	ND				50	
Dichlorodifluoromethane	ND	0.05	ug/g dry	ND				50	
1,2-Dichlorobenzene	ND	0.05	ug/g dry	ND				50	
1,3-Dichlorobenzene	ND	0.05	ug/g dry	ND				50	
1,4-Dichlorobenzene	ND	0.05	ug/g dry	ND				50	
1,1-Dichloroethane	ND	0.05	ug/g dry	ND				50	
1,2-Dichloroethane	ND	0.05	ug/g dry	ND				50	
1,1-Dichloroethylene	ND	0.05	ug/g dry	ND				50	
cis-1,2-Dichloroethylene	ND	0.05	ug/g dry	ND				50	
trans-1,2-Dichloroethylene	ND	0.05	ug/g dry	ND				50	
1,2-Dichloropropane	ND	0.05	ug/g dry	ND				50	
cis-1,3-Dichloropropylene	ND	0.05	ug/g dry	ND				50	
trans-1,3-Dichloropropylene	ND	0.05	ug/g dry	ND				50	
Ethylbenzene	ND	0.05	ug/g dry	ND				50	
Ethylene dibromide (dibromoethane	ND	0.05	ug/g dry	ND				50	
Hexane	ND	0.05	ug/g dry	ND				50	
Methyl Ethyl Ketone (2-Butanone)	ND	0.50	ug/g dry	ND				50	
Methyl Isobutyl Ketone	ND	0.50	ug/g dry	ND				50	
Methyl tert-butyl ether	ND	0.05	ug/g dry	ND				50	
Methylene Chloride	ND	0.05	ug/g dry	ND				50	
Styrene	ND	0.05	ug/g dry	ND				50	
1,1,1,2-Tetrachloroethane	ND	0.05	ug/g dry	ND				50	
1,1,2,2-Tetrachloroethane	ND	0.05	ug/g dry	ND				50	
Tetrachloroethylene	ND	0.05	ug/g dry	ND				50	
Toluene	ND	0.05	ug/g dry	ND				50	
1,1,1-Trichloroethane	ND	0.05	ug/g dry	ND				50	
1,1,2-Trichloroethane	ND	0.05	ug/g dry	ND				50	
Trichloroethylene	ND	0.05	ug/g dry	ND				50	
Trichlorofluoromethane	ND	0.05	ug/g dry	ND				50	
Vinyl chloride	ND	0.02	ug/g dry	ND				50	
m,p-Xylenes	ND	0.05	ug/g dry	ND				50	
o-Xylene	ND	0.05	ug/g dry	ND				50	
Surrogate: 4-Bromofluorobenzene	*11.6*		*ug/g dry*		*117*	*50-140*			
Surrogate: Dibromofluoromethane	*9.82*		*ug/g dry*		*99.2*	*50-140*			
Surrogate: Toluene-d8	*11.0*		*ug/g dry*		*111*	*50-140*			

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
General Inorganics									
Cyanide, free	0.313	0.03	ug/g		104	70-130			
Hydrocarbons									
F1 PHCs (C6-C10)	189	7	ug/g		94.7	80-120			
F2 PHCs (C10-C16)	129	4	ug/g	ND	129	60-140			
F3 PHCs (C16-C34)	338	8	ug/g	ND	138	60-140			
F4 PHCs (C34-C50)	207	6	ug/g	ND	133	60-140			
Metals									
Antimony	118	1.0	ug/g	ND	94.8	70-130			
Arsenic	130	1.0	ug/g	10.2	95.8	70-130			
Barium	130	1.0	ug/g	23.2	85.3	70-130			
Beryllium	135	0.5	ug/g	ND	108	70-130			
Boron, available	4.36	0.5	ug/g	0.52	76.8	70-122			
Boron	116	5.0	ug/g	7.4	87.2	70-130			
Cadmium	135	0.5	ug/g	ND	108	70-130			
Chromium (VI)	0.2		mg/L	ND	75.5	70-130			
Chromium	136	5.0	ug/g	12.6	98.8	70-130			
Cobalt	124	1.0	ug/g	5.0	95.3	70-130			
Copper	131	5.0	ug/g	20.8	88.3	70-130			
Lead	123	1.0	ug/g	9.7	90.4	70-130			
Mercury	1.40	0.1	ug/g	ND	93.2	70-130			
Molybdenum	123	1.0	ug/g	1.5	97.4	70-130			
Nickel	134	5.0	ug/g	12.1	97.7	70-130			
Selenium	132	1.0	ug/g	ND	105	70-130			
Silver	122	0.3	ug/g	ND	97.2	70-130			
Thallium	125	1.0	ug/g	ND	99.7	70-130			
Uranium	125	1.0	ug/g	ND	99.9	70-130			
Vanadium	138	10.0	ug/g	23.8	91.0	70-130			
Zinc	131	20.0	ug/g	65.5	52.5	70-130			QM-07
Semi-Volatiles									
Acenaphthene	0.181	0.02	ug/g	ND	95.1	50-140			
Acenaphthylene	0.164	0.02	ug/g	ND	86.3	50-140			
Anthracene	0.180	0.02	ug/g	ND	94.7	50-140			
Benzo [a] anthracene	0.163	0.02	ug/g	ND	85.9	50-140			
Benzo [a] pyrene	0.140	0.02	ug/g	ND	73.7	50-140			
Benzo [b] fluoranthene	0.218	0.02	ug/g	ND	115	50-140			
Benzo [g,h,i] perylene	0.152	0.02	ug/g	ND	80.1	50-140			
Benzo [k] fluoranthene	0.181	0.02	ug/g	ND	95.3	50-140			
Chrysene	0.189	0.02	ug/g	ND	99.4	50-140			
Dibenzo [a,h] anthracene	0.128	0.02	ug/g	ND	67.5	50-140			
Fluoranthene	0.156	0.02	ug/g	ND	82.0	50-140			
Fluorene	0.148	0.02	ug/g	ND	77.8	50-140			
Indeno [1,2,3-cd] pyrene	0.130	0.02	ug/g	ND	68.7	50-140			
1-Methylnaphthalene	0.143	0.02	ug/g	ND	75.2	50-140			
2-Methylnaphthalene	0.159	0.02	ug/g	ND	83.5	50-140			
Naphthalene	0.179	0.01	ug/g	ND	94.3	50-140			
Phenanthrene	0.166	0.02	ug/g	ND	87.7	50-140			
Pyrene	0.160	0.02	ug/g	ND	84.3	50-140			
Surrogate: 2-Fluorobiphenyl	*1.22*		*ug/g*		*80.3*	*50-140*			
Volatiles									
Acetone	6.52	0.50	ug/g		65.2	50-140			

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Benzene	3.56	0.02	ug/g		89.1	60-130			
Bromodichloromethane	3.45	0.05	ug/g		86.2	60-130			
Bromoform	3.95	0.05	ug/g		98.8	60-130			
Bromomethane	2.81	0.05	ug/g		70.2	50-140			
Carbon Tetrachloride	3.50	0.05	ug/g		87.4	60-130			
Chlorobenzene	3.35	0.05	ug/g		83.9	60-130			
Chloroform	3.69	0.05	ug/g		92.1	60-130			
Dibromochloromethane	3.74	0.05	ug/g		93.4	60-130			
Dichlorodifluoromethane	2.47	0.05	ug/g		61.8	50-140			
1,2-Dichlorobenzene	3.98	0.05	ug/g		99.5	60-130			
1,3-Dichlorobenzene	3.82	0.05	ug/g		95.4	60-130			
1,4-Dichlorobenzene	3.70	0.05	ug/g		92.4	60-130			
1,1-Dichloroethane	3.00	0.05	ug/g		74.9	60-130			
1,2-Dichloroethane	3.54	0.05	ug/g		88.6	60-130			
1,1-Dichloroethylene	3.14	0.05	ug/g		78.6	60-130			
cis-1,2-Dichloroethylene	3.28	0.05	ug/g		82.1	60-130			
trans-1,2-Dichloroethylene	3.06	0.05	ug/g		76.5	60-130			
1,2-Dichloropropane	2.92	0.05	ug/g		73.0	60-130			
cis-1,3-Dichloropropylene	3.37	0.05	ug/g		84.1	60-130			
trans-1,3-Dichloropropylene	2.84	0.05	ug/g		71.0	60-130			
Ethylbenzene	3.43	0.05	ug/g		85.8	60-130			
Ethylene dibromide (dibromoethane	3.92	0.05	ug/g		97.9	60-130			
Hexane	3.25	0.05	ug/g		81.2	60-130			
Methyl Ethyl Ketone (2-Butanone)	6.25	0.50	ug/g		62.5	50-140			
Methyl Isobutyl Ketone	6.07	0.50	ug/g		60.7	50-140			
Methyl tert-butyl ether	6.68	0.05	ug/g		66.8	50-140			
Methylene Chloride	3.31	0.05	ug/g		82.7	60-130			
Styrene	3.52	0.05	ug/g		88.1	60-130			
1,1,1,2-Tetrachloroethane	3.55	0.05	ug/g		88.7	60-130			
1,1,2,2-Tetrachloroethane	3.75	0.05	ug/g		93.6	60-130			
Tetrachloroethylene	3.93	0.05	ug/g		98.2	60-130			
Toluene	3.72	0.05	ug/g		93.1	60-130			
1,1,1-Trichloroethane	3.70	0.05	ug/g		92.4	60-130			
1,1,2-Trichloroethane	3.59	0.05	ug/g		89.7	60-130			
Trichloroethylene	2.98	0.05	ug/g		74.6	60-130			
Trichlorofluoromethane	2.69	0.05	ug/g		67.3	50-140			
Vinyl chloride	2.78	0.02	ug/g		69.4	50-140			
m,p-Xylenes	6.81	0.05	ug/g		85.1	60-130			
o-Xylene	3.43	0.05	ug/g		85.7	60-130			

Qualifier Notes:

QC Qualifiers :

QM-07 : The spike recovery was outside acceptance limits for the MS and/or MSD. The batch was accepted based on other acceptable QC.

QR-01 : Duplicate RPD is high, however, the sample result is less than 10x the MDL.

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

CCME PHC additional information:

- The method for the analysis of PHCs complies with the Reference Method for the CWS PHC and is validated for use in the laboratory. All prescribed quality criteria identified in the method has been met.
- F1 range corrected for BTEX.
- F2 to F3 ranges corrected for appropriate PAHs where available.
- The gravimetric heavy hydrocarbons (F4G) are not to be added to C6 to C50 hydrocarbons.
- In the case where F4 and F4G are both reported, the greater of the two results is to be used for comparison to CWS PHC criteria.

Paracel ID: 1909070

PARACEL
LABORATORIES LTD.

TRUSTE
RESPON
RELIABLE.

rent Blvd.
K1G 4J8
17
cellabs.com

Chain of Custody
(Lab Use Only)

Page 1 of 1

Client Name: GeoPro Consulting Limited

Contact Name: Sarena Medina

Address: 40 Vogell Road, Richmond Hill, ON

Telephone: 905-237-8336

Project Reference: 18-2552G

Quote #

PO # dylanx@geoproconsulting.ca

Email Address: sarenam@geoproconsulting.ca
office@geoproconsulting.ca
mohamedyanis.bournas@geoproconsulting.ca

Turnaround Time:

☐ 1 Day ☐ 3 Day
☐ 2 Day ☑ Regular

Date Required: ______

Criteria: ☑ O. Reg. 153/04 (As Amended) Table __ ☐ RSC Filing ☐ O. Reg. 558/00 ☐ PWQO ☐ CCME ☐ SUB (Storm) ☐ SUB (Sanitary) Municipality: ______ ☐ Other: ______

Matrix Type: S (Soil/Sed.) **GW** (Ground Water) **SW** (Surface Water) **SS** (Storm/Sanitary Sewer) **P** (Paint) **A** (Air) **O** (Other)

Paracel Order Number: 1909070

Required Analyses

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	M&I	PHCs	VOCs	PAHs							
1	BH1 SS1+SS2A	soil		4	Feb 14 2019		☑	☑	☑	☑	☐	☐	3x	120	☐ +	Vial	☐
2	BH2 SS2+SS3	〃		4	Feb 13 2019		☑	☑	☑	☑	☐	☐	☐	☐	☐	☐	☐
3	BH1 SS3	〃	#	2	Feb 14 2019		☑	☐	☐	☐	☐	☐	☐	2x	120	☐	☐
4	BH2 SS5	〃	#	2	Feb 13 2019		☑	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
5							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
6							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
7							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
8							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
9							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
10							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:

Method of Delivery: DASCO

Relinquished By (Sign): Shwela /SPV

Relinquished By (Print): Shwela

Date/Time: Feb 25 2019

Received by Driver/Depot: Dewal

Date/Time: 25-FEB-19 13:49

Temperature: 9.8 °C

Received at Lab:

Date/Time: 02/26/2019 10:44a

Temperature: 7.6 °C

Verified By:

Date/Time: Feb 25-19

pH Verified [ ] By: 14:16

Chain of Custody (Blank) - Rev 0.4 Feb 2016

LIMITATIONS TO THE REPORT

This report is intended solely for the Client named. The report is prepared based on the work has been undertaken in accordance with normally accepted geotechnical engineering practices in Ontario.

The comments and recommendations given in this report are based on information determined at the limited number of the test hole and test pit locations. The boundaries between the various strata as shown on the borehole logs are based on non-continuous sampling and represent an inferred transition between the various strata and their lateral continuation rather than a precise plane of geological change. Subsurface and groundwater conditions between and beyond the test holes and test pits may differ significantly from those encountered at the test hole and test pit locations. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole and test pit locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

It should be noted that the results of the designated substance and chemical analysis refer only to the sample analyzed which was obtained from specific sampling location and sampling depth, and the presence of designated substance and soil chemistry may vary between and beyond the location and depth of the sample taken. Please note that the level of chemical testing outlined herein is meant to provide a broad indication of soil quality based on the limited soil samples tested. The analytical results contained in this report should not be considered a warranty with respect to the soil quality or the use of the soil for any specific purpose or the acceptability of the soils for any excess soil receiving sites.

The report reflects our best judgment based on the information available to GeoPro Consulting Limited at the time of preparation. Unless otherwise agreed in writing by GeoPro Consulting Limited, it shall not be used to express or imply warranty as to any other purposes. No portion of this report shall be used as a separate entity, it is written to be read in its entirety. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated.

The design recommendations given in this report are applicable only to the project designed and constructed completely in accordance with the details stated in this report. Otherwise, our responsibility is limited to interpreting the subsurface information at the borehole or test pit locations.

Should any comments and recommendations provided in this report be made on any construction related issues, they are intended only for the guidance of the designers. The number of test holes and test pits may not be sufficient to determine all the factors that may affect construction activities, methods and costs. Such as, the thickness of surficial topsoil or fill layers may vary significantly and unpredictably; the amount of the cobbles and boulders may vary significantly than what described in the report; unexpected water bearing zones/layers with various thickness and extent may be encountered in the fill and native soils. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and make their own conclusions as to how the subsurface conditions may affect their work and determine the proper construction methods.

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

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.