Frontop Engineering Limited

Geotechnical Investigation
Transit Project Assessment Process (TPAP) for Lakeshore Road – Parts C – Cycling Path/Car Parking (Railway Crossing at Credit River)

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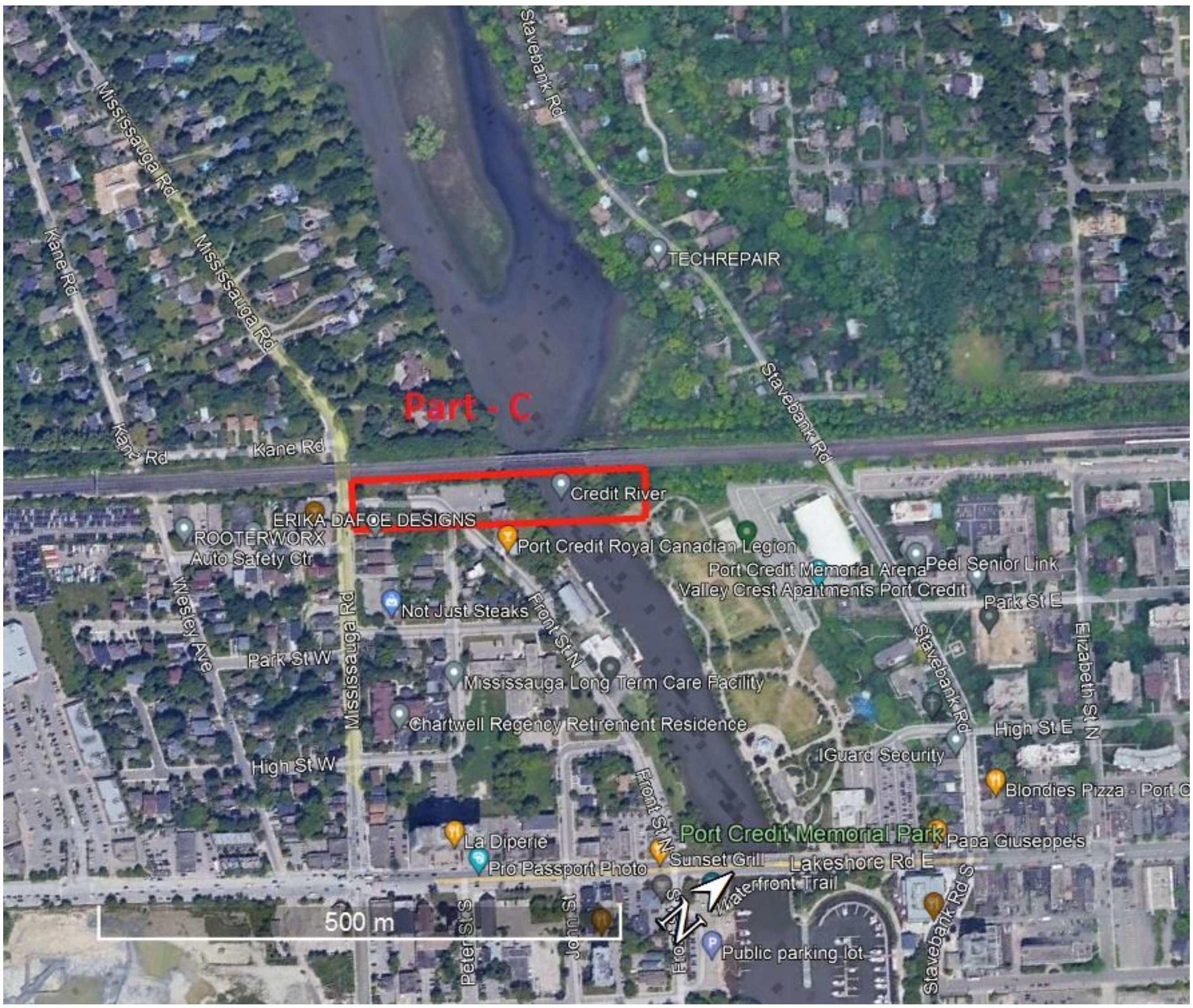

Prepared for: HDR Corporation

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1 INTRODUCTION

Frontop Engineering Ltd. (Frontop) was retained by HDR Corporation behalf of City of Mississauga to undertake a Geotechnical Investigation - Transit Project Assessment Process (TPAP) – Parts A to C, Lakeshore Road for the proposed rehabilitation and widening of pavement as well as structural foundations for critical design areas of Lakeshore Road from Etobicoke Creek in the east to Oakville Border in the west approximately thirteen (13) km.

Based on the proposal, the proposed Transit Project Assessment Process (TPAP) – Lakeshore Road and Royal Windsor Drive includes the following three parts:

1. Part A: Lakeshore Road, Mississauga, from Etobicoke Creek to East Avenue (approximately 2.3 km long);
2. Part B: Lakeshore Road and Royal Windsor Drive, Mississauga, from East Avenue to Winston Churchill Boulevard (approximately 10.7 km. long);
3. Part C: Located at Railway Crossing at Credit River, Mississauga.

The purpose of the geotechnical investigation was to determine the subsurface conditions at borehole locations and from the findings in the boreholes make engineering recommendations for the bridges, abutment, piers, embankments, pavements and underground utilities.

Frontop was required to provide the following six reports as the final results of the geotechnical and geo-environmental investigations:

1. Geotechnical report for Part A - structure;
2. Geotechnical report for Part A - pavement;
3. Geotechnical report for Part B - structure;
4. Geotechnical report for Part B - pavement;
5. Geotechnical report for Part C - cycling path/car parking;
6. Geo-environmental report for Part A, Part B and Part C.

This report is focus Part C – cycling path/car parking and is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with the applicable codes and standards. 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 office can be relied upon.

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.

This report has been prepared for HDR Corporation and City of Mississauga and its designers. Third party use of this report without Frontop consent is prohibited. The limitation conditions presented in this report form an integral part of the report and they must be considered in conjunction with this report.

2 FIELD AND LABORATORY WORK

Field work for the investigations breaks into the following steps and components:

1. Borehole location layout;
2. Locating for underground utilities;
3. Traffic Control, drilling, in-situ testing and soil sampling, monitoring well installation and site restoration;
4. Groundwater level monitoring and sampling.

Traffic control plans were made in accordance with the Ontario Traffic Manual (OTM) Book 7, and traffic control was implemented as required by the permit and following the guidelines of the Ontario Traffic Manual (OTM) Book 7 during drilling.

Boreholes were marked at sites according to the borehole locations prescribed by the Client and considering the accessibility, overhead space and site utilities conditions. The marked borehole locations were used for permit application and utility locates.

Permits for road occupancy were acquired from the City of Mississauga, and utility locates were acquired from public locating agencies on a single borehole basis.

The borehole drilling was supervised by the field geotechnical engineers, who kept track of the permit and locates documents, coordinated all field activities, recorded happenings during drilling, collected soil samples, directed the installation of monitoring wells, and completed site restoration. Boreholes that were not installed with monitoring wells were backfilled in general accordance with Regulation 903. The execution of the drilling program was supervised by senior geotechnical engineer of Frontop.

A total of ten (10) boreholes were drilled for the current investigation both side of Railway Crossing at Credit River, Mississauga. The boreholes are subdivided into two sections, as follows:

- Six (6) boreholes (No. BH-05 to BH-10) for the cycling path/car parking.
- Credit River Railway Crossing Bridge: Four (4) boreholes (No. BH-01 to BH-04) for the bridge.

The boreholes were carried out with drilling rigs CME-75 (truck or track mounted) or GT-8(track mounted) and using solid / hollow stem continuous flight auger equipment by Davis Drilling Limited and James Drilling under the direction and supervision of Frontop Engineering Ltd. (Frontop) personnel. The type of drilling method used to advance the boreholes is identified in the respective borehole logs.

Samples were retrieved at regular intervals with a 50 mm O.D. split-barrel sampler driven with a hammer weighing 624 N and dropping 760 mm in accordance with the Standard Penetration Test (ASTM D 1586) method. The samples were visually classified and logged in the field and returned to the Frontop Engineering Ltd. (Frontop) laboratory for detailed examination by the project engineer and for laboratory testing.

The bedrock was cored in four boreholes (BH-01 to BH-04); with HQ double tube wireline equipment providing 63mm dia. rock core samples. The coring was carried out under the full time supervision of a representative from Frontop who identified and described the rock samples, noting and recording the percentages of total and solid rock core recovery, RQD values, fracture index and the percentage and thicknesses of hard layers.

Water level observations were made during drilling and in the open boreholes at the completion of the drilling operations. Monitoring wells were installed in Boreholes BH-01 and BH-03 for long-term (stabilized) groundwater levels monitoring.

The surveying of the borehole locations was undertaken by Frontop Engineering Ltd. (Frontop) personnel. The ground surface geodetic elevations at the locations of the boreholes are presented on the borehole log sheets in **Appendix B** and are also summarized **Appendix E**.

The lab testing included natural moisture content determination, grain size analysis, hydrometer analysis, atterberg limit test and rock point load test. Natural moisture content determination was conducted for all soil samples according to the standards of LS-701 and ASTM D2216, and the results were shown inside the borehole logs, **Appendix B.** Grain size analysis test was conducted for selected soil samples in order to find and calibrate soil classifications. The grain size analysis test was conducted in accordance with standards of LS-702, ASTM D421 and ASTM D422, and the results are attached as **Appendix C.**

3 SITE AND SUBSURFACE CONDITIONS

As described in Section 2, four (4) deep boreholes were drilled at the Credit River Railway Crossing Bridge area, and the remaining six (6) boreholes were drilled for the cycling path/car parking. The BH location plan for the subject site within project limits is provided on **Appendix A, Drawing 1.** The soil and groundwater conditions are summarized in following sections.

3.1 Boreholes for Cycling Path/Car Parking

Six (6) boreholes (BH-05 to BH-10) were drilled in the area. The borehole locations are shown on **Appendix A, Drawings 1**. Detailed subsurface conditions are presented on the Borehole Logs. The soil and groundwater conditions encountered at these borehole locations are summarized as follows.

Existing Pavement Structure:

Four (4) boreholes were drilled on the road and car parking surface. **Table 3.1** summarize the asphalt and granular thicknesses at the borehole locations.

Table 3.1 Thicknesses of Asphalt and Granular Base/Subbase at Borehole Locations

Borehole No.	Asphalt (mm)	Granular Base/Subbase (mm)	Note
BH-05	80	500	Augered
BH-06	70	450	Augered
BH-08	180	250	Augered
BH-09	280	330	Augered

Topsoil: Two (2) boreholes (BH-07 and BH-10) were drilled on the grass area and encountered a 0 to 100 mm thick topsoil layer at the surface. The thickness of the topsoil in each borehole was shown in the borehole log. 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 Material: Fill material was encountered below the pavement structure or topsoil in majority of the boreholes, (except BH-08 and BH-09) to depths varying from 0.6 to 4.3 m. The fill material was heterogeneous and consisted of clayey silt, sandy silt to silty sand and gravelly sand and was generally present in a compact state / soft to stiff consistency, with occasional very stiff layers. Trace to some inclusions of topsoil / organics, wood/glass chips were noted in fill material.

Grain size analysis of one sample (BH-05/SS2) was conducted and the results are presented in **Appendix C**, with the following fractions:

- Gravel: 4%
- Sand: 25%
- Silt: 55%
- Clay: 16%

Silty Sand, Sandy Silt to Silt: Underneath the fill material in the boreholes (BH-05 to BH-09), was encountered to depths varying from 0.4 to 1.8 m. The silty sand, sandy silt to silt soil deposits were present in a loose to dense state with occasional dense layers, with measured SPT 'N' values ranging from 5 to 41 blows per 300 mm penetration.

Grain size analysis of two samples (BH-06/SS3 and BH-07/SS2) were conducted and the results are presented in **Appendix C**, with the following fractions:

- Gravel: 0% - 7%
- Sand: 7% - 14%
- Silt: 77% - 90%
- Clay: 2% - 3%

Groundwater Conditions:

The groundwater was observed in all boreholes while drilling operation and shown on the borehole logs, attached in **Appendix B** and also summarized in the following **Table 3.2**.

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

Table 3.2 Groundwater Levels Condition

BH No.	Data of Observing	Depth of Groundwater (m.b.g.s)	Elevation of Groundwater (m.a.s.l)
BH-05	July 29, 2022	2.3	81.1
BH-06	July 29, 2022	No water	No water
BH-07	Aug 26, 2022	2.4	81
BH-08	Oct 04, 2022	2.3	79.7
BH-09	Oct 04, 2022	No water	No water
BH-10	July 29, 2022	3.1	74.2

3.2 Railway Crossing at Credit River

Four boreholes (BH-01 to BH-04) were carried out in the both side of Railway Crossing at Credit River, Mississauga. The borehole locations are shown on **Appendix A**. Detailed subsurface conditions are presented on the Borehole Logs. The soil and groundwater conditions are summarized as follows.

Existing Pavement Structure:

Two (2) boreholes (BH-03 and BH-04) were drilled on the parking surface at west side of Railway Crossing at Credit River. **Table 3.3** summarize the asphalt and granular thicknesses at the borehole locations.

Table 3.3 Thicknesses of Asphalt and Granular Base/Subbase at Borehole Locations

Borehole No.	Asphalt (mm)	Granular Base/Subbase (mm)	Note
BH-03	80	350	Augered
BH-04	80	350	Augered

Topsoil: Two boreholes were drilled on the grass area at east side of Railway Crossing at Credit River and encountered a 150 mm thick topsoil layer at the surface. The thickness of the topsoil in each borehole was shown in the borehole log. 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 Material: Fill material was encountered below the topsoil or pavement in the boreholes to depths varying from 4.1 to 6.4 m. The fill material consists of sandy silt to silty sand and was generally present in a very loose to compact state. Trace to some inclusions of topsoil / organics, asphalt/wood chips were noted in fill material.

Grain size analysis of two samples (BH-03/SS6 and BH-04/SS3) were conducted and the results are presented in **Appendix C,** with the following fractions:

- Gravel: 4% - 6%
- Sand: 14% - 37%
- Silt: 52% - 78%
- Clay: 4% - 5%

Soil with Peat: Clayey silt with peat, silty sand with peat were encountered at two boreholes (BH-01 and BH-02) at east side of Railway Crossing. This unit is below the fill material, extending to the depth varied from 11.7 to 11.9 m below ground surface. It presents in a very soft to soft consistency or very loose relative density, with measured SPT 'N' value from 0 to 2 blows per 300 mm penetration.

Grain size analysis of one sample (BH-02/SS7) was conducted and the results are presented in **Appendix C**, with the following fractions:

- Gravel: 1%
- Sand: 15%
- Silt: 75%
- Clay: 9%

Sandy Material: Gravelly sand were encountered at two boreholes (BH-01 and BH-02) at east side of Railway Crossing deposit underneath the peat material, extending to depths varied from 13.1 to 14.2 m below existing grades and overlying shale bedrock at BH-02. This sandy material was present in a compact to dense relative density, with the measured SPT 'N' values ranging from 22 to 46 blows per 300 mm penetration. Occasional auger is grinding during drilling.

Grain size analysis of one sample (BH-01/SS11) was conducted and the results are presented in **Appendix C**, with the following fractions:

- Gravel: 20%
- Sand: 62%
- Silt: 17%
- Clay: 1%

Silt: This unit was encountered at BH-03 at west side of Railway Crossing deposit underneath the fill material, extending to depth of 7.2 m below existing grade. This unit was present in a compact relative density, with the measured SPT 'N' value of 19 blows per 300 mm penetration.

Grain size analysis of one sample (BH-03/SS8) was conducted and the results are presented in **Appendix C**, with the following fractions:

- Gravel: 0%
- Sand: 2%
- Silt: 89%
- Clay: 9%

Silty Clay Till: Silty clay till deposit was found in all boreholes except BH-02, extending to depths varying from 7.6 to 13.9 m below existing grades and overlying shale bedrock. This till was present in a very stiff to hard consistency with the measured SPT 'N' values ranging from 20 to over 50 blows per 300 mm penetration. Occasional sand seams, cobble and boulder were encountered in the till deposits during drilling.

Shale Bed Rock: The grey shale bedrock encountered in all boreholes belongs to Georgian Bay Formation. The assumed shale bedrock surface was found at depths varying from 7.6 to 14.2 m below the existing grade. The approximate depth and elevation of the shale bedrock surface encountered in the boreholes are presented on **Table 3.4** below.

Table 3.4: Approximate Depth and Elevation of Shale Bedrock Surface

Borehole No.	Depth of Shale Bedrock Surface below Existing Ground (m)	Approximate Elevation of Shale Bedrock Surface (m.a.s.l)	Notes
BH-01	13.9	63.5	Bedrock cored
BH-02	14.2	63	Bedrock cored
BH-03	8.4	74.5	Bedrock cored
BH-04	7.6	75	Bedrock cored

Commonly the till overlying the shale contains slabs of limestone which would give a false indication of the bedrock level. Similarly the depth of weathering cannot be determined accurately due to the presence of limestone layers.

Shale bedrock was cored up to a depth of 18.9 m in BH-01, 19 m in BH-02, 13.4 m in BH-03 and 13.7 m in BH-04, with detailed coring information shown on the rock core logs. Photographs of rock cores are presented in **Appendix D** of this report. The rock was visually identified as belonging to the Georgian Bay Formation, consisting of grey shale ('mudstone') making up about 78 to 99 percent of the rock profile, interbedded with thin greyish siltstone and limestone layers forming the remaining 1 to 22 percent. Top 1.5 m of the shale bedrock was generally highly weathered. Total Core Recovery (TCR) achieved with the HQ double tube size core bit in the boreholes are 100 percent with Solid Core Recovery (SCR) varying from 57 to 98 percent. Generally, less core recovery was experienced only near the surface of the rock, where the formation is highly weathered and generally increased with depth. The RQD value for other cored runs varied from 32 to 98 percent, indicating fair quality of bedrock.

As mentioned before, the shale bedrock generally contains layers of sandstone, limestone and dolostone. At this site, the hard layers comprised about 1 to 22 percent of the unit. However, higher concentrations of hard layers can be present. The hard layers are usually less than 150 mm thick. The thicker layers have been observed to be as much as 750 to 900 mm at other sites. The layers are actually lenses and they can vary significantly in thickness over short distance.

Twenty four (24) point load index strength tests were performed witch include fourteen (14) shale/limy shale samples and ten (10) of either siltstone/limestone or siltstone/shale samples. The test results are presented on the borehole log sheets in **Appendix B** and are also summarized **Appendix C**. We have utilized the empirical approximate relationship between unconfined compressive strength (UCS) and point load index strength as follows:

$$\textbf{UCS [MPa]} \approx \textbf{24 I}_{S(50)}$$

where $I_{S(50)}$ is the point index strength in MPa for a 50mm equivalent diameter core. This is an approximate correlation after Franklin and Hoek, which may overestimate the UCS for shale (soft) rock.

The approximate unconfined compressive strength of the limestone/siltstone samples ranged from 43 to 121 MPa in the axial direction. Those values are indicative of "medium strong" to "very strong" rock under ISRM strength convention. The approximate UCS of the shale was lower than that of limestone/siltstone, ranging from 25 to 64 MPa in the axial direction and ranging from 1 to 20 MPa in the diametral direction. These values indicate a "very 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. In light of the fissility of the shale, the diametral point load test results should be considered with caution. Also, it should be noted that in general, rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.

Groundwater Conditions: The groundwater was observed in all boreholes while drilling operation and shown on the borehole logs, attached in **Appendix B** and also summarized in the following **Table 3.5**.

Table 3.5: Groundwater Levels Condition

BH No.	Data of Observing	Depth of Groundwater (m.b.g.s)	Elevation of Groundwater (m.a.s.l)
BH-01	July 27, 2022	1.5	75.8
BH-02	July 28, 2022	4.6	72.6
BH-03	Aug 25, 2022	No water	No
BH-04	Sep 13, 2022	4.6	78.1

Two (2) monitoring wells were installed at all boreholes for the long-term monitoring of groundwater level. Monitoring wells were installed within bedrock. The water levels were measured at November 22. 2022 and the observed groundwater level is 8.8 m (Elev. 74.1 m) at BH-02 location. Groundwater level in these monitoring wells is shown on the borehole logs, attached in **Appendix B** and also summarized in the following **Table 3.6**. BH-01 could not access to get water level measurement due to the monitoring well area was occupied by a lot of park chairs.

Table 3.6: Groundwater Levels Observed in Monitoring Well

BH No.	Date of Drilling	Date of Observation	Depth of Groundwater (m.b.g.s)	Elevation of Groundwater (m.a.s.l)
BH-01	July 27/22	Nov 22/22	Can't access	
BH-03	Aug 25/22	Nov 22/22	8.8	74.1

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

4 FOUNDATIONS

Based on the borehole information, geotechnical discussion and recommendations for the design of the Credit River Crossing Bridge is presented as follows.

4.1 Recommended Soil Parameters

In simplified terms, the subsurface soils explored in the boreholes below the fill and peat materials generally consist of silty clay (till) deposits. Shale bedrock was found in the bridge area at about 14 m at east side and 8 - 9 m at west side of the Railway Crossing below the existing grade.

The proposed soil parameters (it is base on local experience, SPT N-values, and published papers i.e Bowles, Foundation Analysis and Design, 1996, Cao, et. al. 2015) for the design of foundations and ground support systems are summarized in **Table 4.1**.

Table 4.1: Recommended Soil Parameters

Soil Type	Cohesionless Soils: Sand, Silt, Sandy Silt to Silty Sand, Sand and Gravel (Till)						Cohesive Soils - Silty Clay to Clayey Silt Deposits (Till)				
SPT 'N'	**4 - 10**	**11 - 19**	**20 - 29**	**30 - 39**	**40 – 50**	**> 50**	**4 – 8**	**8 - 15**	**15 - 30**	**30 - 50**	**>50**
Unit weight (kN/m^3)	19	20	21	21.5	22	22.5	19	20	21	21.5	22.5
Effective angle of internal friction (^{o}), ϕ'	26	28	30	32	34	37	26	28	30	32	34

Soil Type	**Cohesionless Soils: Sand, Silt, Sandy Silt to Silty Sand, Sand and Gravel (Till)**						**Cohesive Soils - Silty Clay to Clayey Silt Deposits (Till)**				
SPT 'N'	**4 - 10**	**11 - 19**	**20 - 29**	**30 - 39**	**40 – 50**	**> 50**	**4 – 8**	**8 - 15**	**15 - 30**	**30 - 50**	**>50**
Undrained shear strength, C_u (kPa)	-	-	-	-	-	-	25 - 50	50 - 100	100 - 200	200 - 300	300
Coefficient of earth pressure: Active, K_a At rest, K_o Passive, K_p	0.39 0.56 2.56	0.36 0.53 2.77	0.33 0.50 3.0	0.31 0.47 3.25	0.28 0.44 3.54	0.25 0.40 4.03	0.39 0.56 2.56	0.36 0.53 2.77	0.33 0.50 3.0	0.31 0.47 3.25	0.28 0.44 3.54
Elastic modulus (MPa)	5	10	25	35	45	50	5	10	25	40	50
Poisson's ratio	0.35	0.33	0.31	0.3	0.29	0.28	0.35	0.33	0.31	0.3	0.28
Parameter for horizontal subgrade reaction, n_h (MN/m^3)	2	4	7	10	12	15	-	-	-	-	-

Some soil parameters may be adjusted according to additional field test results, such as field vane shear testing.

4.2 Discussion – Foundation Options

For suitability comparison of foundation options, the following types of foundations are listed for discussion purpose:

- Footings
- Drilled caissons
- Driven piles

Two boreholes (BH-01 and BH-02) were carried out in the east of Railway Crossing area. In the boreholes, water bearing sandy soils were encountered below soft peat material, extending to depths varying from 13.1 to 14.2 m, overlying silty clay till or shale bedrock. Shale bedrock was encountered below silty clay till at depths ranging from 13.9 to 14.2 m.

Two boreholes (BH-03 and BH-04) were carried out in the west side of Railway Crossing area. In the boreholes, silty clay till was encountered below silt or fill material at depths varied from 6.4 to 7.2 m and overlying the shale bedrock. Shale bedrock was encountered below silty clay till at depths ranging from 7.6 to 8.4 m.

The upper water bearing sandy soils are considered not suitable to support the footings. If footings are considered as an option to support the bridge structure, they must be founded on the hard silty clay till/bedrock encountered below the sandy soils. Considering the silty clay till is very close to the bedrock surface, we recommend the footing directly found on the bedrock surface. In that case, positive dewatering will be required prior to excavation in water bearing sandy soils. Water must be lowered to at least 1 m below the lowest excavation level. Deep foundations such as driven piles or drilled caissons founded on the sound shale bedrock can be used to support the proposed bridge structure.

4.2.1 Footings

The structure can be supported by spread and strip footings founded on shale bedrock. The bearing values and the corresponding founding elevations at the borehole locations are summarized on **Table 4.2.**

Table 4.2: Bearing Values and Founding Levels of Spread Footings

BH No.	Material	Bearing Resistance at SLS*	Factored Geotechnical Resistance at ULS**	Minimum Depth Below Existing Ground (m)	Founding Level At or Below Elevation (m.a.s.l)	Note
BH-01	Shale	1000	1500	13.9	63.5	Water at Elev.75.8
BH-02	Shale	1000	1500	14.2	63	Water at Elev.72.6
BH-03	Shale	1000	1500	8.4	74.5	Water at Elev.75.8
BH-04	Shale	1000	1500	7.6	75	Water at Elev.78.1

***SLS: for spread footing – bearing resistance at Serviceability Limit States (SLS) in kPa; for H pile – axial resistance at SLS in kN;**
****ULS: for spread footing – factored bearing resistance at Ultimate Limit States (ULS) in kPa; for H pile – factored axial Resistance at ULS in kN.**

For the recommended geotechnical bearing pressure, the footing size for buildings is assumed as 4 m by 4 m in dimension or less for spread footing and 1.8 m in width or less for strip footing; and the footing size for culverts, bridges and retaining walls is assumed as 4 m in width. The impact of lateral earth pressure has been considered in the recommended bearing pressure for the footings of culvert, bridge and retaining wall. The recommended geotechnical bearing resistance should be updated when the final structure assessment report becomes available.

All footing bases must be inspected by P. Eng prior to placing concrete to confirm the founding soil conditions and the bearing capacity.

Footings designed to the specified bearing capacity at the serviceability limit states (SLS) are expected to settle less than 25 mm total and 19 mm differential.

Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

All footings exposed to seasonal freezing conditions must have at least 1.2 metres of soil cover for frost protection.

4.2.2 Driven Piles

Based on the borehole information, the proposed bridge foundations can be supported on driven piles. The piles can consist of steel H-piles such as HP310x110, to be driven minimum 2 m into the bedrock.

For preliminary design purpose, the ultimate axial bearing capacity of the piles driven into the bedrock can be taken as:

HP 310x110 piles:

Factored geotechnical resistance at ULS = 1500 kN/pile

The H pile axial resistance at SLS can be taken as the same of the factored ULS resistance. Downdrag or negative skin friction of piles may occur where piles are installed in a compressible clay deposit that is subject to consolidation. Considering the piles will be installed mainly in the sandy/silty deposits or very stiff to hard clayey till, the downdrag or negative skin friction of piles can be ignored.

The recommended axial resistance values are based on assumption of a single pile or pile group with a minimum center-to-center pile spacing greater than 3D, where D is the pile diameter. Group capacity of pile foundations should be evaluated using a suitable method if the minimum pile spacing is less than 3D in any case. However, considering the characteristics of soil the H piles penetrate at this site the pile group efficiency of one might be considered.

The recommended factored geotechnical resistance at ULS should be confirmed by dynamic testing procedures, ASTM D4945, using the Pile Driving Analyzer (PDA). A resistance factor of 0.5 should be adopted to derive the factored geotechnical resistance of pile at ULS from the unfactored ultimate bearing capacity of pile measured from the PDA test. The PDA test should be carried out on 10% of the production H piles and minimum 1 PDA test should be performed at each location of abutments.

It should be noted that 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 pile-driving hammer must be capable of driving the piles to the required capacity without damaging it. The piling hammer should be sized to be able to deliver at least 70 KJ energy per blow. The cap-block may be modified to minimize over stressing of the pile. Pile driving should be observed, on a full-time basis, by an experienced soil technician, who will record penetration resistance, pile toe elevation, etc. The technician must be supervised by a professional engineer experienced in this type of work.

If the piles encounter refusal before sufficiently penetrating into the recommended bearing zone, then pile capacities may need to be revisited and alternative measures sought. Therefore, pile driving records should be kept particularly, if refusal is met above the recommended bearing zone.

It should be noted that the pile tip elevation provided previously is for initial guidance and estimating purposes only. Due to potentially variable soil conditions, the actual pile tip elevation will vary. The contract should allow for some variation in pile lengths and this aspect should be taken into consideration when ordering the piles. The possibility of piles encountering potential cobbles and boulders or any other obstruction during angering or driving should be anticipated. In view of this, the tips of the piles should be stiffened to minimize damage to the piles while penetrating in recommended bearing zone. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded). Stiffening of the tops of the piles may also be required.

During the driving process, piles that have already been driven will need to be monitored to assess if heaving occurred due to the effects of driving of adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Re-tapping, to check that relaxation has not occurred, will be necessary. Furthermore, it may be necessary to stagger the driving of the piles. The piles should be provided with reinforced tips, as per OPSD 3000.100.

Driving H piles may cause vibration of nearby structures and settlement of soil, and produce high level noise, and hence may have adverse impact to nearby structures, underground utilities and the safety and wellness of people living nearby or passing by. Comprehensive vibration monitoring, settlement monitoring and neighbor noticing plans are recommended. Pre-construction survey should be conducted for all structures, utilities and houses that are anticipated to be affected by pile driving.

During the installation of the piles, some voids will be created around the pile shaft below the ground surface. In order to achieve the lateral resistance of the upper portion of the piles, all voids created during the construction around the piles must be backfilled with unshrinkable fill (U-fill, 0.4MPa) or with Granular 'A' material compacted to 100% SPMDD.

4.2.3 Drilled Caissons

Based on the borehole information, the proposed drilled piers / caissons founded in sound shale bedrock can be designed for axial bearing capacity value of 5.0 MPa at SLS and for a factored geotechnical resistance of 7.5 MPa at ULS. The drilled piers / caissons must be founded at least 1.0 m into the sound bedrock (i.e. 2.5 m below the surface of shale bedrock), or socketed minimum 2 times caisson diameter below the bedrock surface, whichever is greater / deeper.

All caisson bases must be inspected by Frontop on full time basis to ensure that the caisson bases consist of undisturbed sound shale, free from loose/disturbed materials.

The presence of hard layers (such as limestone) in the shale bedrock may require coring of the bedrock to reach the design founding level of the caissons.

The presence of the sandy soils above the shale bedrock will interfere the construction of the caissons. An oversize liner will be required and must be sealed in the underlying silty clay till. Prior to the placement of concrete, any seepage water at the caisson bases must be removed.

Where it is necessary to place caissons at different levels in bedrock, the upper footing must be founded below an imaginary 1 horizontal to 1 vertical (1H:1V) line drawn up from the base of the lower caisson. The lower caisson must be installed first to help minimize the risk of undermining the upper caisson

4.4 Lateral Resistances of Piles and Caissons

Based on the borehole information, the native soils in the boreholes generally consisted of cohesive deposits (i.e. silty clay till), cohesionless (silty sand, gravelly sand and silt) soils and shale bedrock.

The lateral resistance of the piles can be supplemented, if desired, by horizontal components of batter piles.

4.4.1 Ultimate Lateral Earth Resistance

For cohesive soils (clayey silt to silty clay) and bedrock, the passive earth pressure on the pile/caisson at a depth Z can be determined from the following expression:

$$p_{ult} = 6C_u$$

For cohesionless (sandy) soils (sand, silty sand, silt and sandy silt and sand and gravel), 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/caisson at a depth Z, in kPa.

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

Z = depth below final grade, in metre.

L = length of pile/caisson, in metre.

B = size (diameter) of pile/caisson, in metre

γ = unit weight of soil, in kN/m^3
$\gamma = 21\ kN/m^3$ for soils above the groundwater table
$\gamma = 11\ kN/m^3$ for soils below the groundwater table (submerged unit weight)

K_p = passive earth pressure coefficient, generally $K_p = 3.0$ for the cohesionless soils (sand, silty sand, silt and sandy silt and gravel) or as listed in **Table 4.1.**

C_u = undrained shear strength of cohesive soils (clayey silt to silty clay) and bedrock, in kPa.

The passive lateral resistance of the soils within a depth of 1.2 m (frost depth) should be ignored.

The suggested C_u values of the clayey silt to silty clay deposits are given on **Table 4.1**. For the calculation of the lateral resistance, the C_u value of the shale bedrock can be assumed to be 2000 kPa.

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

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

For a short pile/caisson section of length l_z at depth Z, the factored lateral geotechnical resistance (ΔR_{ULS}) 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 $\Phi_h = 0.5$.

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

4.4.2 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/caissons, where k_h is determined as given in Sections (1) and (2) below.

In the model of pile-soil and caisson-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/caisson with a diameter of *B* and a distance of *t* between two adjacent springs, the value of K_{spr} can be calculated using

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

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

(1) Clayey Soils and Bedrock:

The modulus of horizontal subgrade reaction (k_h) of the clayey soils (i.e. the clayey silt and the silty clay deposits) and shale bedrock can be calculated using the following equation:

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

where *B* represents the diameter of the pile/caisson and C_u is the undrained shear strength of the clayey silt as given on **Table 4.1**. For the calculation of k_h, the C_u value of the shale bedrock can be assumed to be 2000 kPa.

(2) Cohesionless/Sandy Deposits:

For the cohesionless/Sandy deposits (sand, silty sand, silt and sandy silt and gravel), 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/caisson, and n_h is a coefficient related to soil density, as listed on **Table 4.1**.

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.

4.4.3 Group Effect

For closely spaced piles/caissons (x < 3B), the reduction factor (β_{grp}) for the lateral earth resistance of the pile/caisson can be expressed as

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

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

4.5 Other Comments on Pile Foundations

The tills are known to contain large cobbles and boulders. This may cause problems for the construction of foundations, especially for driven piles.

Group effect on the bearing capacity of piles should be considered if the horizontal centre-to-centre spacing between the adjacent piles is less than 3 times the pile diameter.

The bearing capacity and the required depth of the piles and the driving criteria for practical refusal must be determined by field pile diving analyzer (PDA) tests. The depth of the piles will be economized from the results of this initial stage PDA testing. PDA testing is also required at re-tapping at about 1 to 2 weeks after the initial driving, in order to examine the set-up effect on the decrease or increase of pile capacity with time.

Pile PDA testing will also be required for 10% to 20% of the production piles.

The piling contractor should ensure that the pile-driving hammer is powerful enough to achieve the required bearing capacity and depth of the piles, but will not cause damage of the piles during the pile driving. It is recommended that the pile tip be reinforced with driving shoe as per MTO Standards. Care must be taken to avoid overdriving and damaging the pile tip, i.e. the structural capacity of the piles should not be exceeded. The possibility of the piles encountering potential obstructions in fill and native soil should be anticipated. Stiffening of the tops of the piles may also be required.

The pile driving should be observed, on a full time basis, by an experienced soil technician, who will record penetration resistance, pile tip elevation etc. The technician must be supervised by a professional geotechnical engineer experienced in this type of work.

During the driving process, piles that have already been driven will need to be monitored to determine if heaving occurred due to the effect of driving of the adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Re-tapping to check that relaxation has not occurred will be necessary. Furthermore, it may be necessary to stagger the driving of the piles. The piles should be provided with reinforced tips.

It should be noted that the recommended foundation type and bearing capacities based on the borehole information are for preliminary design stage only. The investigation 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 conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by Frontop to validate the information for use during the construction stage.

4.6 Erosion/Scour Protection

Erosion and scour protection should be provided for the foundations, piers and abutments of the bridge. Proper erosion and scour protection should also be provided along the sides of the watercourse near the bridge structure.

The erosion and scour protection should be designed by a specialist river engineer/scientist who is familiar with the findings of this investigation.

4.7 Frost Protection

All footings and pile caps exposed to seasonal freezing conditions must have at least 1.2 metres of soil cover or its thermal equivalent for frost protection.

4.8 Seismic Consideration

Based on the existing geotechnical information in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC 2020) and considering the shale is soft rock and the shear wave velocity is not measured, the site is considered as Class 'C' for seismic site response for the culvert/bridge/retaining wall foundations founded on native generally very dense/hard soil or bedrock.

The PGA (peak ground acceleration in unit of 9.81 m/s^2), PGV (peak ground velocity in unit of m/s), and spectral accelerations Sa (T) (in unit of 9.81 m/s^2; T is the period in unit of s for 1:2475 years (2%-in-50-year) are summarized in **Table 4.3** using the Government of Canada Natural Resources 2020 National Building Code of Canada Seismic Hazard Tool (http://earthquakescanada.nrcan.gc.ca) for Class 'C' site.

Table 4.3: Summary of PGA, PGV, and Spectral Accelerations for 1:2475 years – Class 'C' Site

Location	PGA	PGV	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)
Railway Crossing at Credit River	0.181	0.129	0.329	0.198	0.103	0.0472	0.0122	0.00410

Section 4.6.5 of CHBDC requires that seismically induced lateral soil pressures on the back of abutment shall be included in design, where appropriate. These pressures may be calculated with the Mononobe-Okabe method. For the design of abutments and the retaining wall, the coefficients of horizontal earth pressure for assumed backfill are recommended as in the following table.

Compacted Granular A or Granular B Type II (Angle of Internal Friction ϕ=35° (unfactored) Unit weight = 22 kN/m^3 Wall friction angle $\delta = 0.5\phi = 17.5^o$	
Active (K_{AE})*	0.37
Active (K_{AE}) (3H:1V)	0.55
Passive (K_{PE})*	6.29
Compacted Granular B Type I Angle of Internal Friction ϕ=32° (unfactored) Unit weight = 21 kN/m^3 Wall friction angle $\delta=0.5\phi=16^o$	
Wall Condition	Non-yield
Active (K_{AE})*	0.41
Active (K_{AE}) (3H:1V)	0.66

Passive (K_{PE})*	4.90

*After Mononobe and Okabe with PGA = 0.184g in horizontal and 0 in vertical, 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

The coefficients of horizontal earth pressure for common subsoil within the project area are recommended as in following table.

Parameter	Loose fill/sand	Compact fill/sand	Clayey till	Silty or sandy till
Friction angle (°)	28	30	32	34
Wall friction angle (°)	14	15	16	17
Active (K_{AE})*	0.47	0.44	0.41	0.38
Active (K_{AE}) (3H:1V)	-	0.79	0.58	0.59
Passive (K_{PE})*	3.63	4.20	4.90	5.77

*After Mononobe and Okabe with PGA = 0.184g in horizontal and 0 in vertical, 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

Liquefaction:

Liquefaction takes place when loosely packed, saturated sediments at or near the ground surface lose their strength in response to strong ground shaking. Based on subsoil and groundwater condition at the sites of bridge and the retaining walls, liquefaction is not anticipated.

5 APPROACH EMBANKMENTS

Based on the borehole information, our comments and recommendations on the embankments are presented as follows.

5.1 Embankments at Railway Crossing Over Credit River

The soil conditions below the approach embankments generally consisted of loose to compact surficial sandy silt and very soft peat material overlying stiff silty clay till deposits over shale bedrock. The boreholes indicate that the soil conditions below the approach embankments are considered normal and relatively competent in terms of slope stability and settlements.

All organic and otherwise unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1H:1V from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected and approved by Frontop. It should then be compacted, where feasible, from the surface using a suitable compactor. With this procedure, conventional 2H:1V side slopes of embankments should not cause foundation instability of the embankments. The settlement of the foundation soils due to the embankment loading is expected to be within 25 mm.

Proper benching of the existing embankment slope should be implemented if and where abutting into the existing embankments. This can be constructed in accordance with OPSD 208.01 – Benching of Earth Slope.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill, i.e. select subgrade materials (SSM) or Granular 'B' – OPSS 1010. The embankment fill should be placed on the approved and properly rolled subgrade in lifts not exceeding 200 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The degree of compaction should be increased to 98% of SPMDD for the upper 1.0 m of subgrade.

In addition, the settlement of the new embankment fills under their own weight can be expected to occur. However, if SSM or granular soils are used, about half of this settlement should be completed within two months and the remaining half substantially completed within one year.

5.2 Reinforced Earth Slopes

It is understood that due to the restricted site conditions, some embankment slopes need to be constructed to 1H:1V in steepness. In order to avoid retaining wall systems, it is recommended that the steep slopes consist of reinforced earth slopes (such as Tensar Sierra Slope Retention System). The slope retention system is reinforced with geogrids. It can create slopes up to 70° and blends naturally with the surrounding environment with vegetation on the slope surface. The reinforced earth slope can tolerate large differential settlement. The reinforced earth slopes must be designed and constructed by a specialized contractor.

6 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.

6.1 Earth Pressures and Design Parameters

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) S6-06. For design purposes, the following properties can be assumed for 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.

A lateral earth pressure coefficient of minimum 0.4 should be adopted for both the bridge structures including the compaction effect, as the backfill behind the abutment walls will be progressive.

6.2 Retained Soil System (RSS) Walls

It is understood that retained soil system (RSS) walls will be adopted for the wing walls. The RSS walls must be designed and constructed by a specialty contractor. The designer of the RSS walls should evaluate the stability and safety of the walls in terms of bearing capacity, global stability, overturning and horizontal sliding. The general soil parameters are presented in Section 4.1 and **Table 4.1** of this report.

Based on the borehole information (BH-01 and BH-02), the native soils suitable for supporting the RSS walls at east side of Railway Crossing were at or below Elevation 65.4 m and based on the borehole information (BH-03 and BH-04), the native soils suitable for supporting the RSS walls at west side of Railway Crossing were at or below Elevation 76.2 to 77 m at the east abutment

If the design founding elevations of the RSS walls are higher than suitable native soils indicated above, then engineered fill can be used to raise the grade and to support the RSS wall footings.

Prior to the construction of the RSS walls, all existing fill and other unsuitable materials below the wall base levels must be removed and replaced with engineered fill. The engineered fill should consist of approved, acceptable earth fill, i.e. select subgrade materials (SSM) or Granular 'B' (OPSS 1010) compacted to at least 98% of the material's Standard Proctor Maximum Dry Density (SPMDD).

Underneath the footing base, a granular pad founded on engineered fill or competent native soil will be required to support the footings. The granular pad (Granular A pad) should consist of Granular 'A' material compacted to 100% SPMDD. The compacted Granular A pads must extend minimum 0.5 m beyond the footing edge at the footing base level and then slopes down at 1H:1V or flatter. The minimum thickness of the Granular A pad is 300 mm. The thickness of the Granular A pad can be increased where higher bearing capacity is required. The bearing capacity values for the RSS wall footings are as follows:

- Provide minimum 300 mm of Granular A pad below RSS footing level for bearing capacity of 150 kPa at SLS and 225 kPa at ULS at the top of the Granular A pad;
- Provide minimum 600 mm of Granular A pad below RSS footing level for bearing capacity of 200 kPa at SLS and 300 kPa at ULS at the top of the Granular A pad;
- Provide minimum 1200 mm of Granular A pad below RSS footing level for bearing capacity of 300 kPa at SLS and 450 kPa at ULS at the top of the Granular A pad.

The specialty designer must ensure that the RSS walls are safe in terms of horizontal sliding and global stability. The coefficient of friction (μ) between the compacted granular fill of the RSS walls and the subgrade (engineered fill or competent native soils) may be taken as 0.55.

The ratio of minimum reinforced mass width of the RSS walls to reinforced wall height is typically about 0.7 for RSS walls. In this case, the RSS walls are generally considered stable in terms of global stability. The specialty designer must evaluate of the global stability of RSS walls based on the details of the walls (i.e. height, width and elevation) and the soil conditions at the site (i.e. material type of engineered fill and native soil). The minimum required factor of safety is 1.5 for global stability of the RSS walls.

7 EXCAVATIONS AND DEWATERING

Excavations can be carried out with heavy hydraulic backhoe. Excavation of the shale (if any) can be carried out using heaviest available single tooth ripper equipment. It may be necessary at some locations to utilize jackhammer type equipment to "open" the limestone layers for the ripper.

At the east side of the bridge structure over Credit River, the top of footing is at about Elev. 63 m, which is well below the groundwater table (at about Elev. 72.6 to 75.8 m). At west side of the bridge structure over Credit River, the top of footing is at about Elev. 76 m, which is below the groundwater table (at about Elev. 78 m). Positive dewatering will be required prior to any excavations in sandy fill or native sandy soils below groundwater table; otherwise, it will result in an unstable base and flowing sides. A contractor specializing in dewatering should be retained to design the dewatering systems. Groundwater table must be lowered to at least 1.0 m below the lowest excavation level / trench base.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill material can be classified as Type 3 Soil above groundwater table and as Type 4 soil below the water table. The very soft/loose peat material can be classified as Type 4 soil. The very stiff to hard silty clay till can be classified as Type 2 Soil above groundwater table and as Type 3 soil below the groundwater table. Sandy soils below groundwater table can be classified as Type 4 Soil.

It should be noted that the till is a non-sorted sediment and therefore may contain boulders. Possible large obstructions such as buried concrete pieces are also anticipated in the fill material. Provisions must be made in the excavation contract for the removal of possible boulders in the till or obstructions in the fill material.

8 CYCLING PATH/CAR PARKING

Based on the site plan provided by the Client, the driveway and parking area are anticipated to be paved with asphalt concrete. The following provides a preliminary discussion about pavement.

8.1 Subgrade Preparation

The topsoil and loose foreign materials should be completely stripped. The underlying native soil should be stripped as much as required for grade and inspected for soft spots. Soft spots should be subexcavated.

Low area should be brought to grade by backfilling with granular materials or free draining clean fill materials approved by the geotechnical staff from Frontop. Fill material should be applied in a lift of not more than 200 mm, and be compacted to 98 percent of Standard Proctor Maximum Dry Density (SPMDD) throughout.

The completed subgrade should be inspected for signs of rutting or displacement. Areas showing signs of rutting or displacement should be recompacted and retested, or the material should be subexcavated and replaced with free draining clean fill materials approved by the geotechnical engineer from Frontop. The fill materials may consist of either granular material or local inorganic soils provided that its moisture content is within ±2 percent of Optimum Moisture Content (OMC). Fill should be placed and compacted in accordance with OPSS MUNI 501 and the final 300 mm of the subgrade should be compacted to 98 percent of SPMDD.

The final subgrade should be cambered or otherwise shaped properly to facilitate rapid drainage and to prevent the formation of local depressions in which water could accumulate.

8.2 Pavement Structure

It is understood that both driveway and parking lots will be used by light vehicles. Based on provincial practices and Frontop's experience, the pavement structure for the driveway and parking lots is recommended in following **Table 8.1**.

Table 8.1: Pavement Structure

Material		**Thickness of Pavement (mm)**	
		Driveway	**Parking Lot**
Hot-Mix Asphalt (OPSS 1150)	HL3 Surface Course	40	40
	HL8 Binder Course	50	50
Granular Materials (OPSS 1010)	Granular A Base (19 mm Crusher Run Limestone)	150	150
	Granular B Type II Subbase	300	300
Prepared and Approved Subgrade			

8.3 Frost Penetration Depth

Frost penetration depth in the project area is 1.2m.

9 GEOTECHNICAL CONSIDERATION

9.1 Shoring and Trench Box

As presented above, the excavation has to be supported if the excavation walls are not flatted as required by the Regulation 213/91. Full-scale shoring system could be considered, however the supporting members in such limited space will interfere with pipe laying severely.

Based on the nature of the development, soil conditions and excavation, consideration can be given to trench boxes. It should be noted that a trench box provides protection for construction personnel but will not stop the finite movement of the adjacent soil and cause loss of ground, especially when working close to granular base courses below existing pavements or along existing utility trenches backfilled with granular materials. Trench boxes also reduce the contractor's ability to compact backfill materials placed between the trench wall and the outer trench box shell, and increase the likelihood of post-construction settlements along the trench walls. Therefore, the tolerance against disturbance of any structure located above a 1 horizontal to 1 vertical line projected up from the base of the excavation should be assessed prior to construction

When trench boxes are used along existing roadways, settlements may occur along the trench wall and manifest itself months after completion of backfilling. In such cases, following the backfilling of the trench, road reparation, should include a provision for saw-cutting the asphalt at least 1 m back from the trench edges, then recompacting the upper trench backfill, and then repaving.

During excavation using trench boxes, the excavation pit should be left open for as short a period of time as possible and completely backfilled at the end of each working day. Care must be taken during the excavation near important underground structures (i.e. culvert, gas utilities, etc.) and aboveground structures located within or adjacent to the excavation. The owner of the utility/service should also be contacted prior to excavating near their easement to confirm that the proposed excavation meets their requirements. Settlement monitoring for both underground and aboveground structures might have to be considered.

Lateral Earth Pressure

The lateral earth pressure for the design of retaining walls, shoring, or trench boxes can be estimated from the following expression; the expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall.

$p = K (\gamma h + q)$

p = Lateral earth pressure in kPa acting at depth h

K = Earth pressure coefficient, assuming vertical walls and horizontal backfill for permanent construction)

γ = Unit weight of backfill

h = Depth to point of interest in meters

q = Equivalent value of surcharge on the ground surface in kPa

The suggested soil parameters (unfactored) for the retaining wall design and/or ground support systems are summarized below.

Soil Type	**Unit Weight γ (kN/m^3)**	**Effective angle of internal friction (Φ')**	**Coefficient of Earth Pressure**		
			Active K_a	**At rest K_o**	**Passive K_p**
Granular A	22	35	0.27	0.43	3.69
Granular B	21	32	0.31	0.47	3.25
Sand Fill	19	28	0.36	0.53	2.77
Non-Cohesive Deposits	19	30	0.33	0.50	3.00
Till Deposits	21	32	0.31	0.47	3.25
Cohesive Deposits	18	28	0.36	0.53	2.77

9.2 Pipe Bedding and Support

The soils above the groundwater level, or properly dewatered if encountered below the groundwater level, will provide adequate support for the sewer pipes and allow the use of normal Class B type bedding. The recommended minimum thickness of granular bedding below the invert of the pipes is 150 mm. The thickness of the bedding may, however, must be increased depending on the pipe diameter or in accordance with local standards or if wet or weak subgrade conditions are encountered, especially when the soil at the trench base level consists of wet, dilatant silt.

The bedding material should consist of well graded granular material such as Granular 'A' or equivalent. The bedding material should be compacted to at least 95 percent of its SPMDD. After installing the pipe on the bedding, a granular surround of approved bedding material, which extends at least 300 mm above the obvert of the pipe, or as set out by the local authority or municipality, should be placed. It is recommended that Frontop be on site during excavations to assess the suitability of the subgrade materials to support the pipes.

If localized wet trench conditions are encountered, a uniformly graded clear stone may be used provided a suitable, approved filter fabric (geotextile) is placed in conjunction with the clear stone. The geotextile must extend underneath the clear stone, along the sides of the trench, and wrapped on top of the clear stone such that **the clear stone is fully wrapped by the geotextile**. A minimum geotextile overlap of 1 m is required; alternatively stitching of the geotextile could be considered. **Frontop should be on site on a full-time basis if this method is being considered.**

Localized, wet and unstable soils encountered within generally stable soil zones can be generally stabilized by 'punching' a 50 mm well graded crusher run limestone pad into the soft subgrade prior to bedding placement. The thickness of the 'pad' will depend on field conditions and should be examined by Frontop personnel during the construction operations.

9.3 Trench Backfill

The excavated soils can be used as construction backfill provided their moisture content at the time of placement is within 2% of the optimum moisture content and that the soils do not contain organic/peat content. Boulders or cobbles greater than 200 mm in size should be removed from the trench backfill. Frontop should be on site during all trench backfilling operations to confirm the suitability of the material being used.

If granular soils are encountered, smooth drum type vibratory rollers are recommended. Cohesive soils, if encountered, should be compacted with sheepsfoot type vibratory compactors. The trench backfill should be placed in maximum 0.3 m lift thickness and compacted to at least 98 percent of its SPMDD. Trench backfilling operations should be avoided during freezing weather.

It is preferable that the native soils be re-used from approximately the position at which they are excavated so that frost response characteristics of the soils after construction remain essentially similar. If required, consideration may also be given to backfilling trenches with a well graded, compacted granular soil such as Granular 'B' material.

It should be noted that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for the compaction requirements noted above. Stockpiles should therefore be covered with tarpaulins to help minimize moisture increases.

9.4 Geo-Environmental Consideration

Geo-environmental consideration and excess soil management are presented in a separately report.

10 DESIGN REVIEW, MONITORING AND INSPECTION

Designs of different stages and design changes during construction should be reviewed by the geotechnical engineer of Frontop to confirm that the geotechnical recommendations and comments have been properly interpreted and implemented, and that the intention of the report has been meet, and to provide geotechnical input as required.

During construction, full-time engineered fill monitoring, sufficient foundation inspections, slope inspection, subgrade inspections, in-situ density testing, and materials sampling and testing should be carried out by Frontop to confirm that the conditions exposed and encountered are consistent with those encountered in the boreholes and assumed in the report, and to monitor conformance to the pertinent project specifications.

11 LIMITATIONS

Frontop Engineering Limited should be retained for a general review of designs and for required monitoring and inspection. If not accorded the privilege of making this review, Frontop will assume no responsibility for the interpretation of the recommendations in this geotechnical report.
The comments given in this report are intended only for the guidance of design engineers.

It should be noted that the recommended pavement revitalization and reconstruction were based on the borehole information only. Since the boreholes only determine the localized underground conditions at the boreholes, the interpretation of borehole information must, therefore, be validated during excavation operations. Whenever excavation exposes conditions that have not been observed during this investigation, Frontop should be contacted to assess the situation and additional testing and study may be required.

This report was prepared by Frontop for the account of the Client. 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. Frontop accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

12 CLOSURE

We trust this report is satisfactory for your purposes. Should you have any questions or comments, please do not hesitate to contact our office.

Yours truly,
Frontop Engineering Limited

Hadi Shahrokhifard B. Eng. Field Engineer	Justin Zhou, M. Eng., P. Eng. Geotechnical Engineer
Hossein Behnamfard, P. Eng Project Engineer	Kambiz Mosaddegh, P. Eng Senior Project Manager
Frank C. Liu, P. Eng Senior Hydrogeo/Environmental Engineer	Frank Feng, P. Eng Geo-Division Manager

REFERENCE:

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition.

Guide for Design of Pavement Structures, 1993, AASHTO.

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Bowles, Foundation Analysis and Design, 1996, Cao, et. al. 2015.

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Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils, D1586, ASTM

Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass, D2216, ASTM

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils, D4318, ASTM

Standard Test Methods for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications, D5731, ASTM

Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, D6913, ASTM

Terms Used for Overburden Borehole and Sample Log

Sample method:

SS split spoon
ST Shelby tube
AS auger sample
WS wash sample
RC rock core
WH weight of hammer
PH pressure, hydraulic

Penetration Resistance:

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.)

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.).

Soil Description

Cohesionless Soil:

Relative Density	'N' value
very loose	< 4
loose	4-10
compact	10-30
dense	30 - 50
very dense	> 50

Cohesive Soil:

Consistency	Undrained Shear Strength, kPa	'N' value
very soft	< 12	< 2
soft	12-25	2-4
firm	25-50	4-8
stiff	50-100	8-15
very stiff	100-200	16-30
hard	> 200	> 30

Soil Composition:

	% by weight
'trace' (e.g. trace silt)	< 10
'some' (e.g. some gravel)	10 – 20
adjective (e.g. sandy)	20 – 35
'and' (e.g. sand and gravel)	35 – 50

Soil Test:

MH mechanical sieve and hydrometer analysis
W water content
Wl liquid limit
Wp plastic limit
Ip plasticity index
k coefficient of permeability
γ soil unit weight, bulk
Φ' angle of internal friction
c' cohesion shear strength
Cc compression index

Grain Size Example:

Gravel (<5mm) – grain size of gravel is less than 5 mm in diameter.

Bedding Thickness Example:

Interlayer or interbed (20 mm) – thickness of interlayer or inter bed is 20 mm.

Terms Used for Rock Core Log

Weathering (ISRM)

Term	Grade	Description
Fresh	W1	No sign of weathering of discontinuity surface
Slightly	W2	Discolouration or iron stained of discontinuity surface.
Moderately	W3	Less than half of the rock material is decomposed or disintegrated to a soil. Original fabric still intact. Fresh or discoloured rock is present either as a continuous framework or as corestones.
Highly	W4	More than half of the rock material is decomposed or disintegrated to a soil. Original fabric still intact. Fresh or discoloured rock is present either as a continuous framework or as corestones
Completely	W5	All rock material is decomposed and/or disintegrated to a soil. Original fabric still intact.
Residual	W6	All rock material is converted to soil. Original fabrics are destroyed. There is a large change in volume, but the soil has not been significantly transported

Bedding

Term	Bed Thickness
Very thickly bedded	>2 m
Thickly bedded	600 mm-2 m
Medium bedded	200 mm-600 mm
Thinly bedded	60 mm-200 mm
Very thinly bedded	20 mm-60 mm
Laminated	6 mm-20 mm
Thinly laminated	<6 mm

Total Core Recovery (T-R)

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

Solid Core Recovery (S-R)

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

Rock Quality Designation (RQD)

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

Fracture Index (FI)

Expressed as the number of fractures per 300mm, excluding 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
Very widely spaced	2 m-6 m
Widely spaced	600 mm-2 m
Moderately spaced	200 mm-600 mm
Closely spaced	60 mm-200 mm
Very closely spaced	20 mm-60 mm
Extremely closely spaced	<20 mm

(Excluding drill-induced fractures and fragmented rock)

Strength (ISRM)

Term	Grade	Description	Uniaxial Compressive Strength (MPa)
Extremely weak rock	RO	Indented by thumbnail	0.25-1.0
Very weak	R1	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0
Weak	R2	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25
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
Strong	R4	Specimen require more than one blow of geological hammer to fracture it	50-100
Very strong	R5	Specimen requires many blows of geological hammer to fracture it	100-250
Extremely strong	R6	Specimen can only be chipped with geological hammer	>250

Appendix A – Borehole Location Plan

FRONTOP

ENGINEERING LTD

101 Amber Street, Units 1&2
Markham ON L3R 3B2
Tel: 905.947.0900; Fax: .905.305.9370
Email:info@frontop.ca;
Web: www.frontop.ca

LEGEND:

Borehole

Monitoring Well

NOTES:

The boundaries and soil types have been established only at borehole locations. Between boreholes, they are assumed and may be subject to considerable error.

Rev.	Date	Mark
0	November 2022	
Scale		NTS

Geotechnical Investigation for Transit Project Assessment Process (TPAP) for Lakeshore Road – Part C

Ref. No.GEO22-04-20A

Drawing No 1
Borehole Location Plan

Appendix B – Borehole Logs

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-01

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 01
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/27/2022
COMPILED BY JZ
BH LOCATION: N 4823346.093 E 297442.593

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT / REMARKS
77.33	Ground Surface				Flush Mount Cover	
77.18 / 0.15	**TOPSOIL:** (150 mm)					
	Brown, moist, SANDY SILT, trace clay, trace gravel, trace rootlets; FILL	1	SS	14		
	silty clay layer at 0.76m --	2	SS	7		
	wet spoon at 1.52m --	3	SS	15	W. L. 75.81 m July 27. 2022	
	organic at 2.29m --	4	SS	7		
	trace asphalt chips at 3.05m --	5	SS	8		
73.22 / 4.11	Dark gray, wet, very soft to soft, CLAYEY SILT, with peat, trace gravel; (CL-ML, G5)	6	SS	2		77.4
		7	SS	2		57.5
		8	SS	0	Holeplug	42.9
68.64 / 8.69	Gray, wet, very loose, SILTY SAND, with peat, trace gravel; (SM, G4)	9	SS	1		45.2
	clayey peat layer at 10.67m --	10	SS	2		
65.44						

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ^{8}=3% Strain at Failure

FRONTOP-SOIL-ROCK-MARCH-04-2019.GLB 0-FRONTOP SOIL LOG 2019 GEO22-04-20A_C.GPJ 1/27/23

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 01
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/27/2022
COMPILED BY JZ
BH LOCATION: N 4823346.093 E 297442.593

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION (Continued)	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	NATURAL MOISTURE CONTENT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
11.89	Gray, wet, dense, GRAVELLY SAND, silty; (SP. G1)(Continued) saturated at 12.9m --	11	SS	46		65	o	20 62 17 1
64.22 / 13.11	Gray, moist, hard, SILTY CLAY, trace sand, trace gravel, trace weathered shale; TILL (CL, G6)	12	SS	50 / 125mm		64	o	
63.46 / 13.87	**SHALE BEDROCK** grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Rock coring started from 14.49m Refer to rock core log	1	CORE			63, 62		
		2	CORE		Sand	61		
		3	CORE		Screen	60, 59		
58.48 / 18.85	**END OF BOREHOLE**							

Notes:
1) Water was encountered at 1.52m below ground surface during drilling operation;
2) 50mm dia. monitoring well was installed upon completion.

Water Level Readings:

Date	W.L.Depth (m)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m^3)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: +3, ×3: Numbers refer to Sensitivity; O 8=3% Strain at Failure

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 01
LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/27/2022
COMPILED BY JZ
BH LOCATION: N 4823346.093 E 297442.593

(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)
63.5	Rock Surface															
13.9	**SHALE BEDROCK** moderately weathered to fresh, laminated to thinly bedded, grey, very weak to medium strong, **SHALE** and **LIMEY SHALE** (87% to 99%), interbedded with thinly laminated to medium bedded with slightly weathered to fresh, light grey, medium strong to very strong **SILTSTONE** and **LIMESTONE** / **SHALEY LIMESTONE** (1% to 13%) (Georgian Bay Formation). Siltstone and limestone (hard layer) thickness generally less than 50mm, except below depths: Depth(m) Thickness(mm) 14.48 160; 15.80 70															
62.9 / 14.5			1	HQ	100	81	13	81	1, 1, 25, 1	Fragmented zone: 15.09-15.16m; Fracture at: 15.57-15.61m, θ=0°; Soft layer 15.16m ~ 15.33m (W5)	W3/W2		113	42		
61.5 / 15.8			2	HQ	100	94	9	89	2, 1, 4, 1, 0	Fracture at: 16.09-16.12m, θ=0°	W2/W1		33	4		
59.9 / 17.4			3	HQ	100	98	1	98	0, 3, 0, 3, 1	Fracture at: 17.95-17.96m, θ=0°, 2sets; 18.45-18.46m, θ=0°, 2sets	W2/W1		64	13		
58.5 / 18.9	**END OF BOREHOLE**															

FRONTOP-SOIL-ROCK-MARCH-04-2019.GLB 0-FRONTOP ROCK CORE-2016 GEO22-04-20A_C.GPJ 1/27/23

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 $I_{S(50)}$

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C | REF. NO.: GEO22-04-20A

CLIENT: City of Mississauga | Method: Hollow Stem Auger/HQ Coring | ENCL NO.: C - 02

PROJECT LOCATION: Railway Crossing at Credit River, Mississauga | Diameter: 203mm/63mm | ORIGINATED BY EY

DATUM: MTM, ZONE 10 | Date: Jul/28/2022 | COMPILED BY JZ

BH LOCATION: N 4823324.213 E 297441.205

ELEV (m) / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
77.18	Ground Surface						
77.03 / 0.00	**TOPSOIL:** (150 mm)						
0.15	Brown, damp to moist, SANDY SILT to SILTY SAND, trace gravel, trace clay, trace rootlets, trace organic; FILL	1	SS	9			
		2	SS	13			
		3	SS	7			
	dark gray, wet, trace wood chips, gas smell at 2.29m --	4	SS	32			
	wood chips at 3.05m	5	SS	2		67.2	
	dark gray, wet, clayey peat, trace wood chips below 4.57m	6	SS	2	W. L. 72.61 m July 28. 2022	104.5	
71.08 / 6.10	Dark gray, wet, very soft, CLAYEY SILT, with peat, sandy, trace organic,trace gravel; (ML, G5)	7	SS	1		81	1 15 75 9
		8	SS	0		45.6	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (kN/m^3)

Elevation: 77, 76, 75, 74, 73, 72, 71, 70

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ^{8}=3% Strain at Failure

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 02
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/28/2022
COMPILED BY JZ
BH LOCATION: N 4823324.213 E 297441.205

ELEV (m) / DEPTH	DESCRIPTION (SOIL PROFILE, Continued)	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION	Water content / remarks
	Dark gray, wet, very soft, CLAYEY SILT, with peat, sandy, trace organic,trace gravel; (ML, G5)(Continued)				69	
		9	SS	0	68	57.7
					67	
		10	SS	0	66	
65.45 / 11.73	Gray, saturated, compact, GRAVELLY SAND, trace silt, trace clay, trace cobbles; (SP. G1)				65	
		11	SS	22		
	auger grinding at 13.11m --				64	
63.03 / 14.15	**SHALE BEDROCK** grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Coring started from 14.15m. Refer to rock core log	1	CORE		63	
		2	CORE		62	

Depth scale (m): 9, 10, 11, 12, 13, 14, 15, 16

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
POCKET PEN. (Cu) (kPa) | NATURAL 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

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-02

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 02
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/28/2022
COMPILED BY JZ
BH LOCATION: N 4823324.213 E 297441.205

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION (Continued)	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L (WATER CONTENT %)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SHALE BEDROCK grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Coring started from 14.15m Refer to rock core log(Continued)		3	CORE			61					
17							60					
18			4	CORE			59					
58.18 19.00	**END OF BOREHOLE** Notes: 1). Borehole was caved to a depth of 12.5m below ground surface upon completion of soil sampling; 2). Water was measured at a depth of 4.57m below ground surface upon completion of soil sampling.											

Dynamic cone penetration scale: 20 40 60 80 100. Shear strength legend: ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; scale 20 40 60 80 100. Water content scale: 10 20 30.

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

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

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A

CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 02

LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY

DATUM: MTM, ZONE 10
Date: Jul/28/2022
COMPILED BY JZ

BH LOCATION: N 4823324.213 E 297441.205

(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)
63.0	Rock Surface															
14.2 – 62.6 / 14.6	**SHALE BEDROCK** highly weathered to fresh, laminated to thinly bedded, grey, very weak to medium strong, **SHALE** and **LIMEY SHALE** (89% to 99%), interbedded with thinly laminated to medium bedded with slightly weathered to fresh, light grey, medium strong to strong **SILTSTONE** and **LIMESTONE** / **SHALEY LIMESTONE** (1% to 11%) (Georgian Bay Formation). Siltstone and limestone (hard layer) thickness generally less than 50mm, except below depths: Depth(m) Thickness(mm) 15.90 70		1	HQ	100	44	5	0	11	Fracture at: 14.22-14.25m, θ=0° 14.15m ~ 14.2m (W5) 14.4m ~ 14.55m (W5)	W4/W3		81	45		
62.6 / 14.6 – 61.2 / 16.0			2	HQ	100	88	8	73	23 0 0 2 4	Fracture at: 15.62-15.63 m, θ=45° 15.90-15.98m, θ=0°, 2sets 14.55m ~ 14.66m (W4) 14.76m ~ 14.86m (W5)	W4/W3		41			
61.2 / 16.0 – 59.8 / 17.4			3	HQ	100	96	11	91	0 0 4 1 1	Fracture at: 16.84-16.88m, θ=0°, 2sets	W2/W1		51	1		
59.8 / 17.4 – 58.2 / 19.0			4	HQ	100	94	1	91	10 0 0 2 0	Fracture at: 18.45-18.48m, θ=45° Soft layer 17.4m ~ 17.5m (W5)	W2/W1		25	3		
19.0	**END OF BOREHOLE**															

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 BH-03

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A

CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 03

PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY

DATUM: MTM, ZONE 10
Date: Aug/25/2022
COMPILED BY JZ

BH LOCATION: N 4823254.463 E 297341.31

SOIL PROFILE

ELEV (m) / DEPTH	DESCRIPTION
82.84	Ground Surface
82.76 / 0.08	**ASPHALT:** (80 mm)
	BASE GRANULAR: (350 mm) Brown, moist, GRAVELLY SAND, trace silt;
82.41 / 0.43	Brown, moist, SANDY SILT to SILTY SAND, trace gravel; FILL trace asphalt chips at 0.76m -- trace coal and peat at 1.52m -- trace peat at 2.3m -- trace flyash at 3.8m -- trace organic at 4.6m --
77.05 / 5.79	Brown to gray, moist, compact, SILT, trace sand, some clay; (ML, G5)
75.68 / 7.16	Gray, moist, very stiff, SLTY CLAY, some sand, trace gravel, trace weathered shale; TILL(CL, G3C)
74.46 / 8.38	**SHALE BEDROCK** grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Rock coring started from 8.53m Refer to rock core log

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	37	
2	SS	50/ 125mm	
3	SS	10	
4	SS	9	
5	SS	3	
6	SS	4	6 37 52 5
7	SS	10	
8	SS	19	0 2 89 9
9	SS	20	
10	SS	50/ 25mm	
1	CORE		

GROUND WATER CONDITIONS: Flush Mount Cover; Holeplug; W. L. 74.09 m Nov 22. 2022

ELEVATION: 82, 81, 80, 79, 78, 77, 76, 75, 73

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED; + FIELD VANE & Sensitivity; ● QUICK TRIAXIAL; × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m^3)

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 BH-03

FRONTOP ENGINEERING LTD

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 03
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Aug/25/2022
COMPILED BY JZ
BH LOCATION: N 4823254.463 E 297341.31

(m) ELEV DEPTH	SOIL PROFILE – DESCRIPTION (Continued)	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SHALE BEDROCK grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Rock coring started from 8.53m Refer to rock core log(Continued)	2	CORE		Sand	72	
11		3	CORE		Screen		
12		4	CORE			70	
13							
69.48 13.36	**END OF BOREHOLE** Notes: 1) BH was open and no water upon completion of soil sampling; 2) 50mm dia. monitoring well was installed upon completion. Water Level Readings: Date W.L.Depth (m) Nov 22. 2022 8.75						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (kN/m^3)

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

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 8=3% Strain at Failure

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A

CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 03

LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY EY

DATUM: MTM, ZONE 10
Date: Aug/25/2022
COMPILED BY JZ

BH LOCATION: N 4823254.463 E 297341.31

74.5 Rock Surface

8.4 **SHALE BEDROCK** moderately weathered to fresh, laminated to thinly bedded, grey, very weak to medium strong, **SHALE** and **LIMEY SHALE** (81% to 95%), interbedded with thinly laminated to thinly bedded with slightly weathered to fresh, light grey, medium strong to strong **SILTSTONE** and **LIMESTONE** / **SHALEY LIMESTONE** (5% to 19%) (Georgian Bay Formation).

Siltstone and limestone (hard layer) thickness generally less than 50mm, except below depths:

Depth(m)	Thickness(mm)
8.53	50
11.26	60
12.32	70
13.14	100

Ground water: W. L. 74.1 m Nov 22. 2022

ELEV (m) / DEPTH	Core sample NUMBER	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)
73.5 / 8.5 – 73.1 / 9.8	1	HQ	100	65	9	48	10, 7, 19, 1	Fragmented zone: 8.70-8.75m, 9.11-9.32m. Fracture at: 8.53-8.59m, θ=0°; 8.81-9.02m, θ=30°	W3/W2					
73.1 / 9.8 – 71.8 / 11.1	2	HQ	100	88	5	59	4, 6, 5, 6	Fragmented zone: 10.67-10.72m. Fracture at: 9.78-9.83m, θ=0°; 9.96-9.97m, θ=0°; 10.13-10.20m, θ=45°; 10.35-10.39m, θ=45°; 10.85-10.87m, θ=60°	W2/W1		72 51	20 1		
71.8 / 11.1 – 71.0 / 11.8	3	HQ	100	83	12	63	9, 7	Fragmented zone: 11.33-11.42m. Fracture at: 11.20-11.23m, θ=0°; 11.26-11.29m, θ=0° 2sets; 11.68-11.72m, θ=0°	W3/W2		43	10		
71.0 / 11.8 – 69.5 / 13.4	4	HQ	100	90	19	62	2, 7, 1, 9, 4	Fracture at: 12.32-12.40m, θ=0°; 13.00-13.06m, θ=30°; 13.06-13.12m, θ=45°; 13.23-13.26m, θ=0°. Soft layer 12.85m ~ 12.89m (W5)	W3/W2					

69.5
13.4 **END OF BOREHOLE**

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 BH-04

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 03
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY HS
DATUM: MTM, ZONE 10
Date: Sep/13/2022
COMPILED BY JZ
BH LOCATION: N 4823252.966 E 297344.45

SOIL PROFILE

ELEV (m) / DEPTH	DESCRIPTION
82.65	Ground Surface
82.57 / 0.08	**ASPHALT:** (80 mm)
82.22 / 0.43	**BASE GRANULAR:** (350 mm) Brown, moist, GRAVELLY SAND, trace silt;
	Brown, moist, SILTY SAND to SANDY SILT, trace clay, trace gravel; FILL trace coal at 0.9m -- wet spoon at 4.57m -- brown silty clay layers below 5.6m --
76.25 / 6.40	Brown, moist, hard, SLTY CLAY, some sand, trace gravel; TILL(CL, G3C)
75.03 / 7.62	**SHALE BEDROCK** grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Rock coring started from 9.27m Refer to rock core log

SAMPLES

NUMBER	TYPE	"N" BLOWS/0.3 m	Approx. Depth (m)	Water Content (%)	Remarks and Grain Size Distribution (%) GR SA SI CL
1	SS	29	0.4	~7	
2	SS	12	1.0	~12	
3	SS	4	1.8	~14	4 14 78 4
4	SS	8	2.5	~13	
5	SS	3	3.3	~15	
6	SS	4	4.8	~25	
7	SS	7	6.3	~25	
8	SS	85	7.8	~6	
9	SS	50/100mm	9.1	~3	

GROUND WATER CONDITIONS: W. L. 78.08 m Sep 13. 2022

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

POCKET PEN. (Cu) (kPa) — NATURAL UNIT WT (kN/m^3)

ELEVATION: 82, 81, 80, 79, 78, 77, 76, 75, 74, 73

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ^{8}=3% Strain at Failure

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-04

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 03
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY HS
DATUM: MTM, ZONE 10
Date: Sep/13/2022
COMPILED BY JZ
BH LOCATION: N 4823252.966 E 297344.45

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued											
	SHALE BEDROCK grey shale interbedded with siltstone and limestone (Georgian Bay Formation). Rock coring started from 9.27m Refer to rock core log(Continued)		1	CORE			72					
11			2	CORE			71					
12							70					
13			3	CORE			69					
68.96 13.69	**END OF BOREHOLE** Notes: 1) BH was open upon completion of soil sampling; 2) Water encountered at 4.57m below gound surface during drilling operation.											

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: +3, ×3: Numbers refer to Sensitivity; O 8=3% Strain at Failure

LOG OF ROCK CORE BH-04

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A

CLIENT: City of Mississauga
Method: Hollow Stem Auger/HQ Coring
ENCL NO.: C - 03

LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 203mm/63mm
ORIGINATED BY HS

DATUM: MTM, ZONE 10
Date: Sep/13/2022
COMPILED BY JZ

BH LOCATION: N 4823252.966 E 297344.45

(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)
75.0	Rock Surface															
7.6 73.4	**SHALE BEDROCK** highly weathered to slightly weathered, laminated to thinly bedded, grey, very weak to medium strong, **SHALE** and **LIMEY SHALE** (78% to 96%), interbedded with thinly laminated to thinly bedded with slightly weathered to fresh, light grey, medium strong to very strong **SILTSTONE** and **LIMESTONE** / **SHALEY LIMESTONE** (4% to 22%) (Georgian Bay Formation).															
9.3 71.9	Siltstone and limestone (hard layer) thickness generally less than 50mm, except below depths: Depth(m) Thickness(mm) 11.01 60 12.83 80 13.51 160		1	HQ	100	57	4	32	16 14 3 15 0	Fragmented zone: 9.27-9.37m 9.77-9.86m 10.35-10.49m Fracture at: 9.42-9.45m, θ=0° 9.54-9.65m, θ=5° 9.65-9.77m, θ=30° 10.31-10.35m, θ=0°	W4/W3		40	1		
10.8 70.3			2	HQ	100	90	10	72	14 3 2 0 7	Fragmented zone: 10.92-11.01m Fracture at: 10.80-10.85m, θ=0° 11.01-11.07m, θ=5° 12.04-12.07m, θ=0° 12.19-12.24m, θ=10° 12.27-12.34m, θ=5° Soft layer 12.04m ~ 12.05m (W5)	W3/W2					
12.3 69.0			3	HQ	100	72	22	47	16 7 16 2 0	Fragmented zone: 12.55-12.70m 13.00-13.16m Fracture at: 12.34-12.55m, θ=20° 12.70-12.81m, θ=0° 13.27-13.32m, θ=0°	W3/W2		121 34	50		
13.7	**END OF BOREHOLE**															

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 BH-05

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Solid Stem Auger
ENCL NO.: C - 05
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 150mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/29/2022
COMPILED BY JZ
BH LOCATION: N 4823237.435 E 297327.111

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
83.36	Ground Surface							
83.28 / 0.00	**ASPHALT:** (80 mm)							
0.08	**BASE GRANULAR:** (500 mm) Brown, moist, GRAVELLY SAND, trace silt;	1	SS	35		83	~5	
82.78 / 0.58	Brown, moist, SANDY SILT to SILTY SAND, trace gravel, trace organic, trace silty clay pockets; FILL	2	SS	10		82	~10	4 25 55 16
81.53 / 1.83	Brown to gray, moist to wet, dense, SANDY SILT to SILTY SAND, dilantancy; (SM, G4)	3	SS	13			~13	
					∇ W. L. 81.07 m July 29. 2022			
		4	SS	41			~20	
		5	SS	32		80	~19	
79.70 / 3.66	**END OF BOREHOLE**							

Notes:
1). Borehole was open upon completion of drilling;
2) Water was encountered at a depth of 2.29m below ground surface during the drilling operation.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (kN/m^3)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-06

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Solid Stem Auger
ENCL NO.: C - 06
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 150mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/29/2022
COMPILED BY JZ
BH LOCATION: N 4823213.114 E 297307.104

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	WATER CONTENT (%)	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
83.77	Ground Surface					
83.70 / 0.00	**ASPHALT:** (70 mm)					
0.07	**BASE GRANULAR:** (450 mm) Brown, moist, GRAVELLY SAND, trace silt;	1	SS	24	~2	
83.25 / 0.52	Brown, damp, SANDY SILT, trace gravel; FILL	2	SS	12	~7	
82.55 / 1.22	Brown to gray, moist to wet, compact to dense, SILT, some sand, trace clay; (SM, G4)	3	SS	16	~15	0 7 90 3
		4	SS	27	~17	
	gray color and dilatance below 2.9m --	5	SS	34	~14	
80.11 / 3.66	**END OF BOREHOLE** Notes: 1). Borehole was open upon completion of drilling.					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION (83, 82, 81); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 10 20 30; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m³); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

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

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-07

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Solid Stem Auger
ENCL NO.: C - 07
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 150mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Aug/26/2022
COMPILED BY JZ
BH LOCATION: N 4823190.035 E 297295.684

ELEV (m) / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	GR	SA	SI	CL
83.44	Ground Surface											
0.00	Brown, damp, GRAVELLY SAND, trace clay; FILL		1	SS	28		83					
82.83 / 0.61	Brown, moist, compact, SANDY SILT, trace clay; (ML, G5)		2	SS	18		82		7	14	77	2
			3	SS	27							
	wet and color change to gray below 2.44m --		4	SS	20	W. L. 81.00 m Aug 26. 2022	81					
	sasurated at 3m --		5	SS	14		80					
79.78 / 3.66	**END OF BOREHOLE** Notes: 1). Borehole was open upon completion of drilling; 2) Water was encountered at a depth of 2.44m below ground surface during the drilling operation.											

Column headers also include: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%).

FRONTOP-SOIL-ROCK-MARCH-04-2019.GLB 0-FRONTOP SOIL LOG 2019 GEO22-04-20A_C.GPJ 1/27/23

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity; ○ 8=3% Strain at Failure

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-08

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Solid Stem Auger
ENCL NO.: C - 08
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 150mm
ORIGINATED BY HS
DATUM: MTM, ZONE 10
Date: Oct/04/2022
COMPILED BY JZ
BH LOCATION: N 4823157.582 E 297276.18

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	NATURAL MOISTURE CONTENT (approx. %)
82.00	Ground Surface							
0.00 / 81.82	**ASPHALT:** (180 mm)							
0.18 / 81.57	**BASE GRANULAR:** (250 mm) Brown, moist, GRAVELLY SAND, trace silt;		1	SS	28			~0
0.43	Brown, moist to wet, compact, SILTY SAND to SANDY SILT, trace clay; (SM, G4)		2	SS	18		81	~8
			3	SS	27			~16
	gray color and wet below 2.1m --					∇ W. L. 79.70 m Oct 04. 2022	80	
			4	SS	23			~18
			5	SS	17		79	~16
78.49 / 3.51	**END OF BOREHOLE** Notes: 1). Borehole was open upon completion of drilling; 2) Water was encountered at 2.3m below ground surface during drilling operatiom.							

Column headings also printed on the log: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) (10, 20, 30); POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL.

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

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

LOG OF BOREHOLE BH-09

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Solid Stem Auger
ENCL NO.: C - 09
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 150mm
ORIGINATED BY HS
DATUM: MTM, ZONE 10
Date: Oct/04/2022
COMPILED BY JZ
BH LOCATION: N 4823139.221 E 297262.384

ELEV (m) / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	WATER CONTENT (%)
80.77	Ground Surface				
0.00	**ASPHALT:** (280 mm)				
80.49 / 0.28	**BASE GRANULAR:** (330 mm) Brown, moist, GRAVELLY SAND, trace silt;	1	SS	32	
80.16 / 0.61	Brown, moist, loose to compact, SILTY SAND to SANDY SILT, trace clay; (SM, G4)	2	SS	11	
		3	SS	5	
		4	SS	10	
		5	SS	15	
77.26 / 3.51	**END OF BOREHOLE** Notes: 1). Borehole was open and no water upon completion of drilling.				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION (80, 79, 78); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

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

FRONTOP ENGINEERING LTD

LOG OF BOREHOLE BH-10

PROJECT: Geotechnical and Environmental Investigation -Transit Project Assessment Process (TPAP) - Part C
REF. NO.: GEO22-04-20A
CLIENT: City of Mississauga
Method: Solid Stem Auger
ENCL NO.: C - 10
PROJECT LOCATION: Railway Crossing at Credit River, Mississauga
Diameter: 150 mm
ORIGINATED BY EY
DATUM: MTM, ZONE 10
Date: Jul/29/2022
COMPILED BY JZ
BH LOCATION: N 4823341.935 E 297453.896

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS
77.22	Ground Surface				
77.12 / 0.00	**TOPSOIL:** (100 mm)				
0.10	Brown to gray, damp to moist, CLAYEY SILT, some sand to sandy, trace gravel, trace rootlet, FILL	1	SS	18	
	trace organic, glass chips at 0.76m --	2	SS	2	
	glass chips at 1.52m --	3	SS	10	
	wet spoon, wood and glass chips at 2.29m -- saturated below 2.29m --	4	SS	6	
	sandy, wood chips at 3.05m --	5	SS	4	W. L. 74.17 m July 29. 2022
	peat, glass and wood chips at 3.81m --	6	SS	3	
72.95 / 4.27	**END OF BOREHOLE**				

Notes:
1). Borehole was open upon completion of drilling;
2) Water was measured at a depth of 3.05m below ground surface upon completion of drilling.

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION (77, 76, 75, 74, 73); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

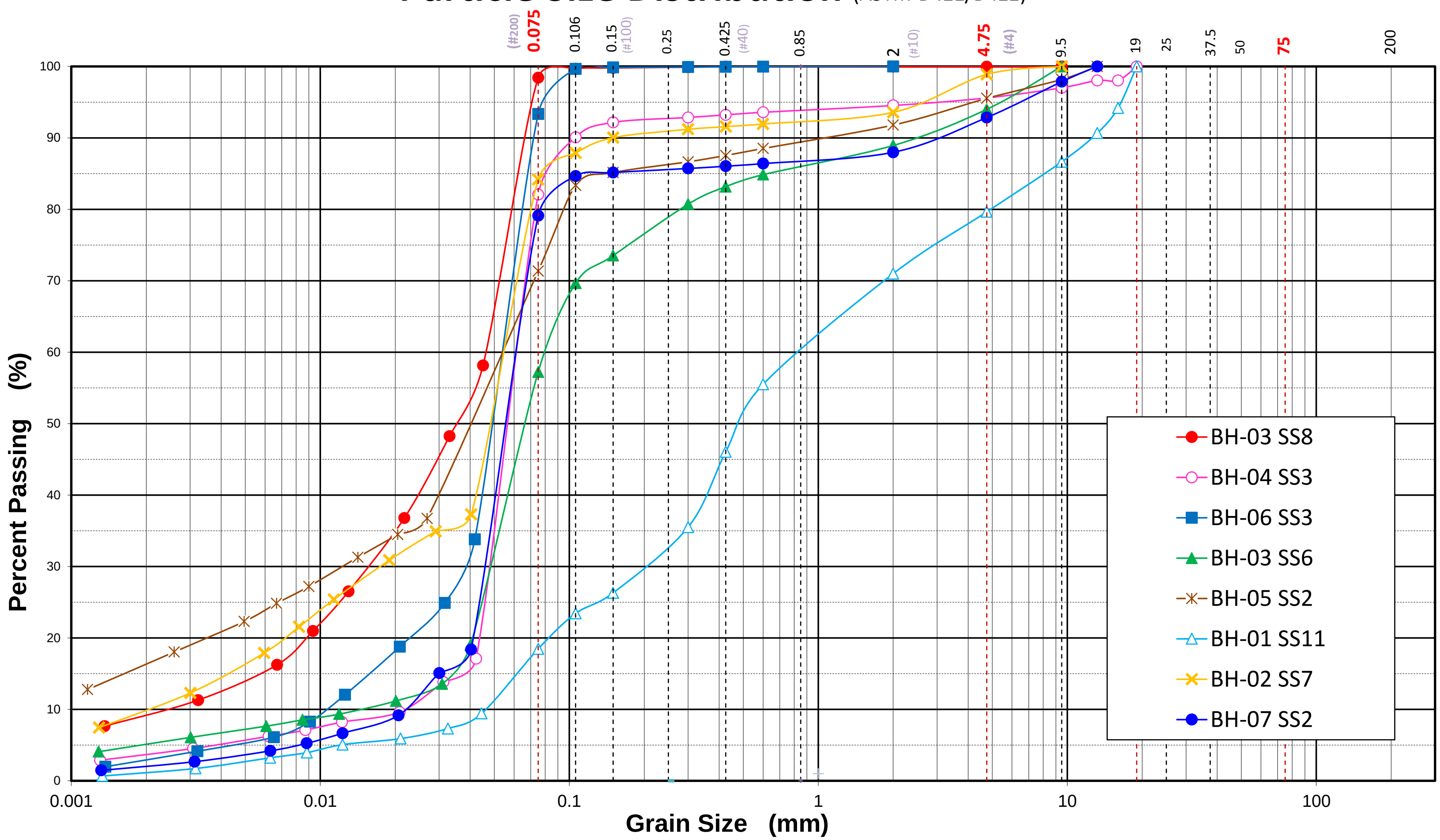
GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ^{8}=3% Strain at Failure

FRONTOP-SOIL-ROCK-MARCH-04-2019.GLB 0-FRONTOP SOIL LOG 2019 GEO22-04-20A_C.GPJ 1/27/23

Appendix C: Lab Test Results

Particle Size Distribution (ASTM-D421/D422)



Silt and Clay		Sand			Gravel		Cobble +
Clay	Silt	Fine	Medium	Coarse	Fine	Coarse	

Frontop Engineering Limited 101 Amber Street, Markham ON L3R 3B2	GRAIN SIZE DISTRIBUTION	Figure No	B2
		Project No	GEO22-04-20A
		Date	Jan. 23/2023

Results of Point Load Index Strength Tests (Part-C)

BH No.	Run No.	Depth(m)	Rock Type	Point Load Inedex $I_{s(50)}$ (MPa)		Approximate Uniaxial Compressive Strength (MPa)*
				Diametral	Axial	
BH-01	1	14.58	Limestone	1.7		42
BH-01	1	14.58	Limestone		4.7	113
BH-01	2	16.51	Shale	0.1		4
BH-01	2	16.51	Shale		1.4	33
BH-01	3	18.11	Limey shale	0.6		13
BH-01	3	18.11	Limey shale		2.7	64
BH-02	1	14.40	Limestone	1.9		45
BH-02	1	14.40	Limestone		3.4	81
BH-02	2	15.77	Shale		1.7	41
BH-02	3	17.12	Shale	<0.05		1
BH-02	3	17.12	Shale with limestone seams		2.1	51
BH-02	4	18.26	Shale	0.1		3
BH-02	4	18.26	Shale		1.0	25
BH-03	2	9.91	Shale	0.8		20
BH-03	2	9.91	Shaly limestone		3.0	72
BH-03	2	10.77	Shale	0.1		1
BH-03	2	10.77	Shaly limestone		2.1	51
BH-03	3	11.61	Shale	0.4		10
BH-03	3	11.61	Shaly limestone		1.8	43
BH-04	1	10.74	Shale	<0.05		1
BH-04	1	10.74	Shale with limestone seams		1.7	40
BH-04	3	12.85	Siltstone	2.1		50
BH-04	3	12.85	Siltstone		5.0	121
BH-04	3	13.41	Limey shale with shale		1.4	34

Appendix D: Rock Core Photos

Rock Core Photo BH-01(Part-C) Run 1

Run 1: 47’6” - 51’10” (14.48 – 15.80 m)

Rock Core Photo BH-01(Part-C) Run 2

Run 2: 51’10” - 57’1” (15.80 – 17.40 m)

Rock Core Photo BH-01(Part-C) Run 3

Run 3: 57’1” - 61’10” (17.40 – 18.85 m)

Rock Core Photo BH-02(Part-C) Run 1

Run 1: 46’5” - 47’9” (14.15 – 14.55 m)

Rock Core Photo BH-02(Part-C) Run 2

Run 2: 47’9” - 52’5” (14.55 – 15.98 m)

Rock Core Photo BH-02(Part-C) Run 3

Run 3: 52’5” - 56’11” (15.98 – 17.35 m)

Rock Core Photo BH-02(Part-C) Run 4

Run 4: 56’11” - 62’4” (17.35 – 19.00 m)

Rock Core Photo BH-03(Part-C) Run 1

Run 1: 28’0” - 32’0” (8.53 – 9.75 m)

Rock Core Photo BH-03(Part-C) Run 2

Run 2: 32’0” - 36’3” (9.75 – 11.05 m)

Rock Core Photo BH-03(Part-C) Run 3

Run 3: 36’3” - 38’9” (11.05 – 11.81 m)

Rock Core Photo BH-03(Part-C) Run 4

Run 4: 38’9” - 43’10” (11.81 – 13.36 m)

Rock Core Photo BH-04(Part-C) Run 1

Run 1: 30’5” - 35’5” (9.27 – 10.79 m)

Rock Core Photo BH-04(Part-C) Run 2

Run 2: 35’5” - 40’6” (10.79 – 12.34 m)

Rock Core Photo BH-04(Part-C) Run 3

Run 3: 40’6” - 44’11” (12.34 – 13.69 m)

Appendix E: Borehole Coordination and Elevation

Summary of BH Coordination and Elevation (MTM, Zone 10)– Part C

BH No.	North (m)	East (m)	Elevation (m.a.s.l)
BH-01	4823346.093	297442.593	77.33
BH-02	4823324.213	297441.205	77.184
BH-03	4823254.463	297341.31	82.842
BH-04	4823252.966	297344.45	82.646
BH-05	4823237.435	297327.111	83.36
BH-06	4823213.114	297307.104	83.768
BH-07	4823190.035	297295.684	83.441
BH-08	4823157.582	297276.18	82.001
BH-09	4823139.221	297262.384	80.769
BH-10	4823341.935	297453.896	77.221