



**FINAL REPORT**

# **Preliminary Foundation Investigation and Design Report**

*Professor Day Drive Underpass*

*Highway 400 to Highway 404 Link (Bradford Bypass)*

*Simcoe County and York Region*

*MTO Assignment No. 2019-E-0048*

Submitted to:

**AECOM Canada**

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19136074-Rev0-PFIDR Professor Day Drive

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# PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
PROFESSOR DAY DRIVE UNDERPASS  
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)  
SIMCOE COUNTY AND YORK REGION  
MTO ASSIGNMENT NO. 2019-E-0048**

## 1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc. and hereafter referenced as WSP Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the proposed Bradford Bypass (BBP), a 16.3 kilometre (km) rural controlled access freeway connecting Highway 400 to Highway 404, in the County of Simcoe and Regional Municipality of York. This report addresses the proposed underpass to carry the future extension of Professor Day Drive over the proposed BBP at the approximate location shown on the Key Plan in Drawing 1.

The BBP was realigned around an archaeology site (west of the originally proposed bridge location) that was identified during preliminary design, and after completion of the foundation investigation along the original BBP alignment. Referring to Drawing 1, the proposed BBP alignment has been shifted some 300 m to the north of the original alignment and associated bridge crossing location. The foundation investigation summarized herein may be used for planning and preliminary assessment of the future Professor Day Drive underpass and additional foundation investigation will need to be undertaken for preliminary design.

## 2.0 SITE DESCRIPTION

As part of a municipal initiative, Professor Day Drive, currently a 2-lane road that runs in a north-south direction and terminates just north of 8<sup>th</sup> Line (Line 8), may be extended to the north at a future date, although the details are not currently known. As currently envisaged under the municipal initiative, the road extension will cross the BBP and bend sharply to the east (as shown at the top of Drawing 1) and is to connect with Yonge Street (County Road 4). The proposed BBP alignment will cross the future extension of Professor Day Drive about 900 m north of Line 8 (i.e., just south of the sharp bend). This section of the proposed BBP alignment is generally oriented in an east-west direction on a slight skew to the future extension of Professor Day Drive, which will generally run in a north-south direction at the future bridge location.

Land use surrounding the site is predominantly agricultural, apart from a residential development located about 500 m southeast of the site. Heavy tree cover is present immediately to the southwest of the bridge crossing (see Photograph 1) and about 300 m east of the site. A tree line borders the site to the west and extends north to Line 9. There are also multiple tree lines (separating properties) that run in an east-west direction and terminate at a private trail extending south from municipal address number 2673 on Line 9 towards the future extension of Professor Day Drive. The site is relatively flat (See Photograph 2) and the existing ground surface is at about Elevation 266 m.





**Photograph 1** – looking northwest from residential development towards proposed bridge crossing site (heavy tree cover present in the distance).



**Photograph 2** – looking south on Line 9 (bridge crossing site located at the far top left corner of the photograph) showing tree line bordering the site to the west.

### 3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between April 14 and April 21, 2021 (inclusive), during which time two boreholes (designated PDD-1 and PDD-2) were advanced at the locations shown on Drawing 1. As noted in Section 1.0, the original BBP alignment for this bridge crossing site was shifted approximately 300 m to the north, after the field investigation was completed, to avoid an archaeology site identified during preliminary design. Therefore, the boreholes advanced as part of the foundation investigation are located approximately 300 m south of the proposed bridge structure.

The boreholes were advanced using 210 mm outside diameter (O.D.) hollow stem augers to a depth of approximately 3.0 m below ground surface, followed by wash-rotary techniques (advancement of tricone with water/drilling mud) using a Diedrich D54 track-mounted drill rig. The drill rig was supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The wash-rotary technique was used to counter-balance hydrostatic forces and reduce disturbance at the sampling and testing interval. Water for drilling operations was imported to site using water totes supplied by Walker Drilling Inc.

Soil samples were generally obtained at 0.75 m, 1.5 m, and 3.0 m intervals of depth using a 50 mm O.D. split-spoon sampler driven with an automatic hammer in general accordance with Standard Penetration Test (SPT) procedure (ASTM D1586)<sup>1</sup>. The split-spoon samplers used in the investigation generally limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

The water level was measured within the hollow stem augers prior to the start of mud rotary operations and a standpipe piezometer was installed in both boreholes to allow monitoring of the groundwater level. The installed piezometers consist of a 50 mm diameter PVC pipe, with 3.0 m long slotted screens within a filter sand pack. The

<sup>1</sup> ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils



borehole and annulus surrounding the piezometer pipe above the filter sand pack was backfilled to near ground surface with bentonite pellets in both boreholes. The standpipe piezometers were left sticking up out of the ground and protected with a monument cover.

The field work was monitored on a full-time basis by a member of WSP Golder's engineering staff who located the boreholes in the field, directed the sampling and in-situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in individually labelled containers and transported to WSP Golder's laboratory in Mississauga for further visual review and geotechnical laboratory testing. Index and classification testing consisting of natural moisture content, Atterberg limits and grain size distribution were conducted on selected samples. The laboratory tests were carried out in general accordance with MTO and/or ASTM Standards, as applicable.

Two soil samples (one from Borehole PDD-1 and one from Borehole PDD-2) were submitted to Bureau Veritas Laboratories, a specialist analytical laboratory located in Mississauga, Ontario, under chain of custody procedures for testing of conductivity / resistivity, pH, and chemical analysis of soluble sulphate and chloride content, to assess the potential for the soil to cause deterioration to buried concrete and corrosion to steel.

The borehole locations were surveyed in the field by Golder personnel using a Trimble Geo 7X Global Positioning System (GPS) unit. The locations given on the Record of Borehole sheets and shown on Drawing 1 are positioned relative to NAD 83 MTM (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum (CGVD28 datum). The borehole locations, including the geographic coordinates, the ground surface elevations, and borehole depths, are summarized below.

Borehole No.	NAD83 MTM (Geographic) Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
PDD-1	4,887,622 (44.128559°)	298,073 (-79.584064°)	262.2	21.5
PDD-2	4,887,565 (44.128050°)	298,081 (-79.583960°)	262.1	21.8

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984), the general site lies near the border of three physiographic regions of Southern Ontario known as the Peterborough Drumlin field, the Schomberg Clay Plains, and Simcoe Lowlands.

The Peterborough Drumlin field region generally consists of calcareous till soils and is generally sandier (rather than stony) within Simcoe County. Many drumlins in this area are known to have shallow coverings of silt and fine sand which is probably wind-blown material. Deposits of clay typically lie between the drumlins in this area.

The Schomberg Clay Plain region consists of deep deposits of stratified clay and silt. In some areas, clay and silt varves (greater than 100 mm thick) are present with the clay layers typically containing up to 50% clay and 40% silt; however, the behaviour is described to be more like that of silt than clay. The Simcoe silty clay and silt loams are described to be poorly drained.

The Simcoe Lowlands physiographic region covers the central portion of the County of Simcoe. Following the retreat of the last glacial ice sheet, the lowland was flooded by the now extinct post-glacial Lake Algonquin. This past post-glacial lacustrine environment is marked by deep sand, silt and clay beds overlying glacial ground moraine material.

The overall topography of the area indicates the site is located near the edge of a rolling hill, possibly within a till plain and near the base of a drumlin which is consistent with the subsurface conditions encountered during the current investigation.

## 4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes from the investigation, including the piezometer installation details, water level readings, and the results of the in situ and laboratory tests, are provided on the Record of Borehole sheets in Appendix A. The results of the in situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in the section below are the values measured directly in the field and are uncorrected. The detailed results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix B. The results of the analytical testing are provided in Appendix C.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface soils encountered in the boreholes consist of a surficial layer (about 0.7 m thick) of cohesive fill underlain by a 2.3 m thick cohesive deposit of stiff to very stiff clayey silt. This deposit was underlain by a 2.6 m thick borderline cohesive deposit of stiff to very stiff clayey silt-silt to silt which was underlain by a 3.1 m to 7.6 m thick deposit of generally compact to dense silt and sand, in turn, underlain by till deposits of very dense or hard silty sand and clayey silt-silt to silt to the termination depth in both boreholes.

A more detailed description of the major stratigraphic units encountered in the boreholes is described in the sections below.

### 4.2.1 Clayey Silt Fill

A cohesive layer of clayey silt fill was encountered at ground surface in Boreholes PDD-1 and PDD-2. The layer was about 0.7 m thick, extending down to about Elevation 261.4 m.

The SPT ‘N’-values measured in the clayey silt fill range from 5 to 9 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

### 4.2.2 Clayey Silt

A cohesive deposit of clayey silt was encountered beneath the fill in Boreholes PDD-1 and PDD-2 at about Elevation 261.4 m. The deposit was about 2.3 m thick, extending down to about Elevation 259.1 m.

The SPT ‘N’-values measured in the clayey silt range from 10 to 19 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

Grain size distribution testing was carried out on a sample of the clayey silt deposit and the results are shown on Figure B1 in Appendix B.

Atterberg limits testing was carried out on two samples of the clayey silt and the samples had liquid limits ranging between 25% and 31%, plastic limits ranging between 16% and 18%, and plasticity indices ranging between 9% and 13%. These results, which are plotted on a plasticity chart on Figure B2, indicate that the deposit consists of clayey silt with low plasticity.

The natural water content measured on four samples of the clayey silt ranges between about 21% and 29%.

#### **4.2.3 Clayey Silt-Silt to Silt**

A borderline cohesive deposit of clayey silt-silt to silt was encountered beneath the clayey silt deposit in Boreholes PDD-1 and PDD-2 at about Elevation 259.1 m. The deposit was about 2.6 m thick, extending down to about Elevation 256.5 m.

The SPT 'N'-values measured in the clayey silt-silt to silt range from 12 to 16 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

Grain size distribution testing was carried out on two samples of the clayey silt-silt to silt deposit and the results are shown on Figure B3 in Appendix B.

Atterberg limits testing was carried out on two samples of the clayey silt-silt to silt and the samples had liquid limits ranging between 14% and 15%, plastic limits ranging between 10% and 11%, and a corresponding plasticity index of 4%. These results, which are plotted on a plasticity chart on Figure B4, indicate that the deposit consists of clayey silt-silt to silt with slight to low plasticity.

The natural water content measured on three samples of the clayey silt-silt to silt was about 10%.

#### **4.2.4 Silt and sand**

A non-cohesive deposit of silt and sand was encountered beneath the clayey silt-silt to silt deposit in Boreholes PDD-1 and PDD-2 at about Elevation 256.5 m. The deposit was about 3.1 m thick in Borehole PDD-1 and 7.6 m thick in Borehole PDD-2, extending down to about Elevations 253.4 m and 249.0 m, respectively.

The SPT 'N'-values measured in the silt and sand range between 9 and 38 blows per 0.3 m of penetration; the values typically measured 20 blows upwards to 38 blows per 0.3 m of penetration, indicating a compact to dense state of compactness, with one looser zone encountered in Borehole PDD-1 around Elevation 254 m. The SPT 'N'-value obtained in the looser zone yielded 9 blows per 0.3 m of penetration, indicating a loose state of compactness. Auger grinding was noted in Borehole PDD-1 at a depth of about 9.1 m (about Elevation 253.1 m), suggesting the presence of cobbles (and possibly boulders) within the silt and sand deposit.

Grain size distribution testing was carried out on three samples of the silt and sand deposit and the results are shown on Figure B5 in Appendix B.

Atterberg limits testing was carried out on the fines portion of three samples of the silt and sand and the samples had liquid limits ranging between 12% to 13%, a plastic limit of 10%, and corresponding plasticity indices ranging between 2% and 3%. These results, which are plotted on a plasticity chart on Figure B6, indicate that the fines portion of the deposit consists of silt with slight plasticity.

The natural water content measured on four samples of the silt and sand ranges between about 9% and 10%.

#### 4.2.5 Silty Sand Till

A non-cohesive deposit of silty sand till was encountered beneath the silt and sand in Boreholes PDD-1 and PDD-2 between Elevations 253.4 m and 249.0 m. The deposit was about 4.9 m to 8.4 m thick, extending down to about Elevation 244.9 m.

The SPT 'N'-values measured in the silty sand till in Borehole PDD-2 range between 57 and 100 blows per 0.3 m of penetration, indicating a very dense state of compactness. In Borehole PDD-1, the split spoon sampler met effective refusal (i.e., equivalent to 100 blows for less than 0.3 m of penetration) at each sampling interval and did not penetrate the entire SPT depth, indicating a very dense state of compactness. Auger grinding was noted in Borehole PDD-1 at a depth of about 15.2 m (about Elevation 247.0 m), suggesting the presence of cobbles (and possibly boulders) within the silty sand till deposit.

Grain size distribution testing was carried out on three samples of the silty sand till deposit and the results are shown on Figure B7 in Appendix B.

The natural water content measured on three samples of the silty sand till ranges between about 7% and 11%.

#### 4.2.6 Clayey Silt-Silt to Silt Till

A borderline cohesive deposit of clayey silt-silt to silt till was encountered beneath the silty sand till in Boreholes PDD-1 and PDD-2 at about Elevation 244.9 m. Both boreholes were terminated within the deposit at depths of about 21.5 m (about Elevation 240.7 m) and 21.8 m (about Elevation 240.3 m).

A single SPT 'N'-value measured in the clayey silt-silt to silt till yielded 100 blows per 0.3 m of penetration. In three other instances, the split spoon sampler met effective refusal (i.e., equivalent to 100 blows for less than 0.3 m of penetration) and did not penetrate the entire SPT depth. The measured SPT 'N'-values suggest the deposit has a hard consistency.

Atterberg limits testing was carried out on a sample of the clayey silt-silt to silt till and the sample had a liquid limit of 19%, a plastic limit of 15%, and a corresponding plasticity index of 4%. These results, which are plotted on a plasticity chart in Figure B8, indicate that the deposit consists of clayey silt-silt to silt with slight to low plasticity.

The natural water content measured on a sample of the clayey silt-silt to silt till was about 17%.

### 4.3 Groundwater Conditions

The groundwater levels measured in the open boreholes at the time of the investigation are not considered representative of the stabilized hydrostatic groundwater levels at the site. Where water levels taken during drilling operations are shown on the borehole records, they represent an unstabilized groundwater level recorded inside the hollow stem augers prior to introduction of drilling fluids/water.

A standpipe piezometer was installed in each borehole to allow monitoring of the stabilized groundwater level at this site. The groundwater levels recorded during drilling (i.e., the unstabilized groundwater levels) and in the piezometers (i.e., the stabilized groundwater levels) are shown on the borehole records in Appendix A and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth (Elevation) of Screen Interval / Sand Pack (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
PDD-1	262.2	4.6 to 7.6 (El. 257.6 to 254.6)	2.3	259.9	Apr. 19, 2021	Open borehole / inside hollow stem augers
			0.9	261.3	Apr. 21, 2021	Piezometer (upon completion of well install)
			0.2	262.0	Dec. 10, 2021	Piezometer
			0.4	261.8	Feb. 4, 2022	Piezometer
			-0.04	262.2	Feb. 28, 2023	Water inside piezometer was frozen above ground surface
PDD-2	262.1	10.7 to 13.7 (El. 251.4 to 248.4)	3.0	259.1	Apr. 14, 2021	Open borehole / inside hollow stem augers
			At ground surface	262.1	Dec. 11, 2021	Piezometer
			-0.1	262.2	Feb. 7, 2022	Water inside piezometer was frozen above ground surface
			-0.4	262.5	Feb. 28, 2023	

The groundwater levels measured in the piezometers indicate the groundwater is located near ground surface. Frozen water levels measured in the Winter suggest potential artesian conditions at this site, although the elevated readings may be attributable to accumulation and expansion of the ice within the piezometers.

The groundwater level and hydrostatic head at depth at this site will be subject to seasonal fluctuations and precipitation events; the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation and snow melt.

#### 4.4 Analytical Testing of Soil

Two soil samples (one from Borehole PDD-1 and one from Borehole PDD-2) were submitted for laboratory analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the test results are summarized below.

Borehole No., Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Chloride (µg/g)	Soluble Sulphate (µg/g)
PDD-1, SA 3	7.74	7,800	128	<20 <sup>1</sup>	<20 <sup>1</sup>
PDD-2, SA 2	7.63	7,600	131	<20 <sup>1</sup>	<20 <sup>1</sup>

Note 1: Less than reportable detection limit.

## 5.0 CLOSURE

This preliminary Foundation Investigation Report was prepared by Mr. Mark Henderson, P.Eng., a Geotechnical Engineer at WSP Golder. Mr. Kevin Bentley, P.Eng., a Geotechnical Engineer and MTO Foundations Designated Contact with WSP Golder, conducted a technical and quality review of the report.



## Signature Page

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# **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
PROFESSOR DAY DRIVE UNDERPASS  
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)  
SIMCOE COUNTY AND YORK REGION  
MTO ASSIGNMENT NO. 2019-E-0048**

## 6.0 DISCUSSION AND PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides preliminary foundation recommendations for planning and conceptual design of the Bradford Bypass and future Professor Day Drive underpass structure (a municipal initiative). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current subsurface exploration.

The Preliminary Foundation Design Report (Part B of this report), including the discussion and preliminary recommendations, are intended for the use of MTO and their designers for planning and conceptual design and shall not be used or relied upon for any other purpose or by any other parties, including the future construction contractor or design-build proponents. Contractors undertaking the work must make their own interpretation based on the factual data presented in the Preliminary Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the concept and preliminary design of the project and for which special provisions may be required in the future Contract Documents. Those requiring information on aspects of detail design and construction must make their own interpretation of the factual information provided and supplement as necessary, as such interpretation may affect preliminary and detail design, equipment selection, proposed construction methods, scheduling and the like.

### 6.2 Project Understanding

Based on the latest Bradford Bypass mainline alignment and profile drawings provided by AECOM (preliminary draft dated May 2023), the future Professor Day Drive extension and bridge crossing will be located at about Station 14+330 as shown on Drawing 1. This section of the BBP alignment is proposed to be constructed near the existing ground surface, with the future extension of Professor Day Drive and new underpass bridge expected to carry traffic over the Bradford Bypass. The structural classification of the bridge is defined as “major-route” by the structural designer and is to be confirmed by the owner as per Section 4.4.2 of the CHBDC (2019).

For planning purposes and based on the preliminary design of similar underpass structures for the project (e.g. 10<sup>th</sup> Sideroad), it is assumed that the future Professor Day Drive underpass will be a multi-span bridge (at least 2 spans) with approach embankments estimated to be about 10 m high.

No design details are available for the future Professor Day Drive bridge crossing, thus, the foundation information and recommendations provided herein are for general planning purposes only for conceptual design.

### 6.3 General Foundations Design Context

#### 6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code CAN/CSA S6-19* (CHBDC, 2019) and its *Commentary*, the bridge structure and its foundation system may be classified as having large traffic volumes and their performance as having potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design.

In addition, based on the preliminary level of foundation investigation completed to date at this location (see Part A of this report) in comparison to the degree of site understanding, the level of confidence for design of the Professor Day Drive bridge foundation elements and approach embankments has been assessed as a “low degree of site and prediction model understanding”. At the time of the borehole investigation the location of the

bridge and abutment foundations were not known, thus, the boreholes are not located near the actual foundation footprints. As such, the recommendations contained in the report are generalized for planning and ongoing preliminary design and further investigation will be required when actual locations of the abutments are known.

Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for planning purposes at this stage. During design, additional investigation and testing will need to be performed to increase the level of confidence and modify the geotechnical resistance factors as appropriate. In addition, reference is made to the MTO Material Engineering and Research Office (MERO) Memorandum #2020-01 (dated March 23, 2020) for future foundation, settlement and stability analyses during design, as applicable.

## 6.3.2 Seismic Design

### 6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance,  $\bar{N}_{60}$  and average undrained shear strength,  $s_u$  within the upper 22 m of the overburden (extrapolated to 30 m) below the founding level (assumed to be existing ground surface), the site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC (2019). The Site Class will need to be confirmed during design when deeper boreholes (up to 30 m deep) are advanced. Alternatively, consideration can be given to performing geophysical testing (e.g. Multi-Channel Analysis of Surface Waves (MASW) or Vertical Seismic Profiling (VSP)) to obtain shear wave velocities and confirm the Site Class designation.

The CHBDC (2019) states that the seismic hazard values associated with the design earthquakes should be those established from the *National Building Code of Canada* (NBCC) by the Geological Survey of Canada (GSC). The 2015 seismic hazard maps (referred to as the 5<sup>th</sup> generation seismic hazard maps) have been used for planning and preliminary design for this project, as referenced in the CHBDC (2019).

### 6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the 2019 CHBDC, the peak ground acceleration ( $PGA$ ), peak ground velocity ( $PGV$ ) and 5% damped spectral response acceleration ( $S_a(T)$ ) values for Site Class C were obtained for the bridge site using the NBCC website ([earthquakescanada.nrcan.gc.ca](http://earthquakescanada.nrcan.gc.ca)) and are summarized below.

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$PGA$ (g)	0.030	0.046	0.073
$PGV$ (m/s)	0.027	0.042	0.067
$S_a(0.2)$ (g)	0.052	0.078	0.121
$S_a(0.5)$ (g)	0.037	0.054	0.081
$S_a(1.0)$ (g)	0.021	0.032	0.048
$S_a(2.0)$ (g)	0.010	0.016	0.025
$S_a(5.0)$ (g)	0.002	0.004	0.006
$S_a(10.0)$ (g)	0.001	0.002	0.003

The values provided above are for the reference ground condition Site Class C and must be checked and modified (as appropriate) to the site-specific seismic site classification to be confirmed during preliminary or detail design to obtain applicable design spectral values. The design spectral values will need to be assessed along with the importance category (defined as “major-route” by the structural designer and to be confirmed by the owner as per Section 4.4.2 (CHBDC)) and actual structure periods to determine the Seismic Performance Category and level of seismic analysis required during design as per Table 4.10 of the CHBDC (2019).

### 6.3.2.3 Soil Liquefaction

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil which may lead to potentially large surface deformations, and under undrained conditions generate excess pore water pressures that can lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (analogous to slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of slopes often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

In general, the soils within a few hundred metres of the bridge site consist of stiff to very stiff clayey silt-silt to clayey silt soils underlain by generally compact to dense deposits of silt and sand, underlain by glacial till deposits of hard clayey silt-silt to silt and very dense silty sand. Considering the compactness, consistency and relatively low site-specific PGA, the site is estimated to have a low potential for liquefaction during a seismic event based on preliminary liquefaction assessment of the soil and preliminary analyses using the simplified stress-based method as per Section 6.14.8 of the CHBDC (2019). In addition to the simplified stress-based method outlined in the CHBDC, liquefaction potential of the cohesive deposits was checked using the empirically-derived method by Bray et. al (2004), as outlined in the Canadian Foundation Engineering Manual (CFEM), 2006. Based on this method and the range of natural water contents, liquid limits, and plasticity indices of the cohesive deposits encountered at the site, the cohesive deposits are not considered to be susceptible to liquefaction.

The potential for liquefaction will need to be reassessed when more site-specific foundation soil information and confirmed PGA is available during detail design.

## 6.4 Foundation Types

Based on the subsurface conditions encountered some 300 m from the bridge site, both shallow and deep foundation options have been considered for support of the new foundation elements. The preliminary recommendations provided herein will be subject to change when more detailed soil information and actual foundation locations are known. Foundation construction should be in general accordance with OPSS.PROV 902 (Excavating and Backfilling – Structures).

A summary of the general advantages and disadvantages associated with each option is provided below and a comparison of the foundation alternatives based on advantages, disadvantages, relative costs, and risks is provided in Table 1 following the text of this report.

Shallow foundations are considered marginally feasible if founded on competent stiff to very stiff clayey silt below any topsoil, fill materials, and peat/organic soil (not encountered during the current investigation but anticipated based on the forested areas near the footprint of the embankments). Factored geotechnical resistances at ultimate and serviceability limit state of 250 kPa and 150 kPa may be used for concept design of shallow foundations founded on the native stiff to very stiff clayey silt founded at a depth of 1.5 m below ground surface (i.e., frost depth). Higher geotechnical resistances could be achieved by perching footings within approach

embankments, above a minimum thickness of 3 m of engineered fill. While these shallow foundation options are feasible, consideration of settlements due to the anticipated approach embankment loading near the abutments will need to be considered and may eliminate shallow foundations as an option. Given that the BBP alignment has been shifted some 300 m north of the boreholes advanced as part of this investigation, there is a risk that founding soils may be less competent and shallow foundations may not be feasible for the anticipated high loads associated with the overpass structure. As a result, shallow foundations are not discussed further and will need to be reevaluated when foundation investigation is performed near the actual bridge site.

Deep foundations consisting of driven steel H or tube piles (with or without the pile cap perched within the approach embankments) are preferred from a conceptual design and constructability perspective, and this option will permit integral abutments. Caissons are also considered to be a feasible foundation option; however, although this option provides higher geotechnical resistances compared to shallow foundations or driven piles, it would be more costly and would not permit integral abutment design.

## 6.5 Deep Foundations

### 6.5.1 Steel H-Pile or Tube Foundations

Driven steel H-piles, such as conventional steel HP 310x110 piles, founded within the very dense / hard till deposit are considered feasible for support of the future foundation elements. Closed ended steel tube piles (324 mm outer diameter with a minimum wall thickness of 9.5 mm) are also considered a feasible deep foundation option; however, driven steel H-piles may be preferred over steel tube piles given that H-piles are most commonly used for integral abutment design and pose a lower risk of “hanging up” or being deflected from their vertical or battered orientation during installation if gravel pockets, cobbles or boulders are present due to their larger end area. As noted on the borehole records, auger grinding was observed in Borehole PDD-1 at about 9.1 m depth (Elevation 253.1 m), which is above the anticipated founding depth for driven piles.

The results of Boreholes PDD-1 and PDD-2 suggest a competent “end-bearing” till stratum (“100-blow” material) at a depth of about 13 m below ground surface. For piles founded about 2 m into the “100-blow” till deposit (i.e., pile lengths on the order of about 15 m below ground surface), a factored geotechnical resistance value at ultimate limit state of 1,300 kN can be used for preliminary design of conventional driven steel HP 310x110 piles. The factored geotechnical resistance value at serviceability limit state value is estimated to be equal or greater than the ultimate limit state value is not considered to govern the design. Consideration may also be given to using larger pile sizes (e.g. HP 360x108) in order to increase geotechnical resistance values during preliminary and detail design after site specific geotechnical information is available and after geotechnical capacities have been reviewed and revised.

For preliminary design, driven steel piles spaced at 3 pile diameters (centre-to-centre) can be assumed to act as single piles with no group interaction effects with regards to axial resistance. For piles spaced less than 3 diameters, the total pile axial resistance should be reduced by a group reduction factor ( $R_A$ ) (Reese, 2006) as follows:

Pile Spacing (d = Pile Diameter)	Pile Axial Resistance Group Reduction Factor ( $R_A$ )
3.0 d	1.0
1.5 d	0.7
1.25 d	0.55

Note: Reduction factors for other pile spacings may be interpolated from the values above.



Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS).PROV 903 (*Deep Foundations*) as amended by Special Provision 109F57. It is recommended that High-Strain Dynamic testing be specified on at least 10% of the piles or two piles at each foundation element (whichever is greater) in each stage of construction.

In order to optimize the design and reduce the risk of piles not achieving the design geotechnical resistance at the design tip elevation during construction, the design-builder or contractor can consider a combination of the following options:

- Advanced site-specific investigation during detail design to confirm or adjust axial geotechnical resistances for design based on a minimum typical degree of site understanding;
- High-strain dynamic testing (i.e. PDA) on all piles at end-of-initial drive (EOID) and at a specified number of piles on beginning-of-restrike (BOR) or retap;
- Advanced static pile load test as per ASTM D-1143, and
- Evaluation of strength gain with time to ascertain that geotechnical resistance will ultimately be achieved, either via a waiting period associated with static pile load testing prior to production piling, or a long restrike period demonstrated via PDA testing in advance of or during production piling.

The selected design and testing method(s) must consider logistical challenges and potential schedule impacts as part of the detailed design and planned construction, and optimized design and testing methods must be incorporated into SP109F57 and the future contract documents.

The subsequent pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven, to avoid possible damage to the piles, and to calibrate with the results of the high-strain dynamic testing or advanced static pile load testing.

For conventional integral abutment design, corrugated steel pipes (CSPs) backfilled with loose sand are recommended to be installed as part of the integral abutment design consistent with the MTO Structural Office Report SO-96-01 titled “Integral Abutment Bridges”.

### **6.5.2 Drilled Shafts (Caissons)**

Drilled shafts (caissons) are also considered feasible for supporting the future bridge structure foundations.

Auger grinding noticed in the generally compact to dense silt and sand deposit and within the till deposit in Borehole PDD-1, together with the frequent “100-blow” SPT and split spoon refusal measurements may be associated with the presence of gravel pockets, cobbles and/or boulders. Although not specifically encountered/recovered during the current investigation, consideration must be given to the potential presence of cobbles and boulders within the silt and sand and glacially derived till deposits during preliminary and detail design.

As discussed in the previous section, the results of Boreholes PDD-1 and PDD-2 suggest a competent “end-bearing” till stratum (“100-blow” material) at a depth of about 13 m below ground surface. For caissons founded about 2 m into the “100-blow” till deposit (i.e., minimum caisson lengths on the order of about 15 m below ground surface), a factored geotechnical resistance value at ultimate and serviceability limit state of 2,500 kN can be used for preliminary design of a 900 mm diameter caisson. Higher geotechnical resistance values in the order of

4,000 to 6,000 kN could likely be achieved for caissons at a similar depth but with larger diameters in the range of 1.2 m to 1.5 m, although these preliminary capacities will need to be confirmed when site specific geotechnical information is available.

For preliminary design, bored caisson piles spaced at 8 pile diameters (centre-to-centre) can be assumed for design purposes to act as single caisson piles, with no group interaction effects with regards to axial resistance. For caissons spaced less than 8 diameters, the total caisson axial resistance should be reduced by a group reduction factor ( $R_A$ ) (Reese, 2006) as follows:

Caisson Spacing ( $d$ = Pile Diameter)	Caisson Axial Resistance Group Reduction Factor ( $R_A$ )
9 $d$	1.0
6 $d$	0.9
4 $d$	0.75
3 $d$	0.7

Note: Reduction factors for other caisson pile spacings may be interpolated from the values above.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner may be required (based on the results of the current investigation) to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils. From an installation perspective, a permanent liner may be preferred over a temporary liner (particularly in the case of relatively deep shaft excavations) since there is no requirement to withdraw multiple casing strings and therefore allows for a faster installation time. Other drilled shaft construction methods such as polymer slurry drilling, which only requires a temporary “starter” casing to be withdrawn upon completion of concrete placement, could also be considered but would require a higher level of quality control / quality assurance and development of special provisions. From a design perspective, use of a permanent liner would decrease the available frictional resistance and corresponding design geotechnical resistance due to the difference in adhesion between the liner material and soil versus the adhesion between concrete and soil which would need to be considered during detail design.

Specialized construction techniques would be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (e.g. polymer slurry or other slurry mix) within the liner / open hole to prevent basal heave (especially if high groundwater or potential artesian conditions are encountered) and disturbance of water-bearing cohesionless layers along the shaft and at the base.

Given that the above drilled shaft capacities have both a shaft friction and end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the drilled shaft. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection device (SID) or a shaft quantitative inspection device (SQUID); because of the likelihood of water with entrained sediment or the presence of polymer slurry, it is recommended that SQUID testing be specified on some or all caissons to demonstrate and verify base cleaning. Should the inspection indicate that loosened or compressible material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

Alternatively, a design based solely on shaft friction may be considered provided the design geotechnical resistances are reduced accordingly and appropriate quality assurance procedures are adopted by the design-builder / contractor. The consistency and characteristics of the drilling slurry (particularly if a bentonite

slurry is used) will have an impact on the design geotechnical resistances and this will need to be considered during preliminary and detail design.

In order to optimize the design, the design-builder or contractor can consider a combination of the following options:

- Advanced site-specific investigation during preliminary and detail design to confirm or adjust axial geotechnical resistances for design based on a minimum typical degree of site understanding, and/or
- Advanced static pile load test as per ASTM D-1143, bi-directional static load ("Osterberg Cell") test (CFEM, 2006), or Statnamic Load Test (CFEM, 2006).

Caisson installation must be in accordance with OPSS.PROV 903 (Deep Foundations). MTO's recent special provision should be included in the Design-Build output specifications, as well as any Non-Standard Special Provisions (as applicable) to address the requirements for supply and installation of drilled shafts (caissons) including the use of temporary or permanent liners/casings and slurry, the placement of concrete by tremie methods, cleaning and inspection of the shafts and base of the drilled shafts as applicable, and quality control testing. Non-destructive post-construction testing in selected drilled shafts should also be included in the output specification and is recommended to verify the integrity of the concrete given the groundwater conditions, presence of saturated cohesionless soils, and specialized installation methods to counterbalance the hydrostatic pressures.

### 6.5.3 Resistance to Lateral Loads

The design of piles or caissons subjected to lateral loads should take into account such factors as the relative rigidity of the pile / caisson to the surrounding soil, the fixity condition at the head of the pile / caisson (i.e., at the pile / caisson cap level), the structural capacity of the pile / caisson to withstand bending moments and shear, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile / caisson and group effects. Lateral loading could be resisted fully or partially by the use of battered piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. For integral abutment design, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral pile / caisson analysis for detail design should be carried out using non-linear methods (such as p-y curves) when the pile / caisson group configuration is established as per the CHBDC (2019).

For preliminary design, the resistance to static lateral loading in front of a single pile or drilled shaft may be calculated using subgrade reaction theory (CFEM, 2002 as referenced in CHBDC, 2006); however, this method is only considered appropriate where the maximum pile deflections are small (less than 1% of the pile/caisson diameter), where the loading is static (no cycling) and where the pile/caisson material is linear. Estimated soil parameters for lateral loading analyses can be determined when more detailed soil information and actual foundation locations are known.

It is understood that an integral abutment design may be considered. Where the integral abutment design includes the installation of 3 m long Corrugated Steel Pipe (CSP) liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-pile will be generally free to flex and move laterally within the limits of the CSP.

Group action for lateral loading will also need to be considered for any pile groups as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2019).

#### 6.5.4 Downdrag Loads on Piles / Caissons

There are currently no preliminary design details available for the bridge structure approach embankments. The results of the boreholes advanced some 300 m from the bridge site indicate that cohesive deposits at least 5 m thick could be present at the bridge location and that very dense or hard “end-bearing” soils could be present at least 13 m below the ground surface. Accordingly, depending on the relative timing of embankment fill placement near the abutments and pile installation, and depending on the actual thickness / consistency of cohesive soils and estimated settlement magnitudes, the embankment fills could induce downdrag loads that will need to be accounted for in the assessment of the structural loading of the piles. Downdrag loads can likely be mitigated by designing piles / caissons to resist the additional load in the structural design and/or reducing downdrag forces by preloading the foundation soil to induce settlements prior to driving piles or installing caissons.

### 6.6 Frost Protection

The spread / strip footing(s) and pile / caisson caps should be founded at a minimum depth of 1.5 m below the lowest surrounding final grade, including any distance measured perpendicular to the sloping ground surface to provide adequate protection against frost penetration (as interpreted from OPSD 3090.101 – *Foundation Frost Penetration Depths for Southern Ontario*).

### 6.7 Approach Embankments

Conventional embankment construction is considered feasible at the site. Although it is expected that sufficient right-of-way is available, consideration could also be given to designing reinforced earth slopes or retaining walls where space limitations exist.

After supplemental foundation investigation is performed at the actual bridge site and site specific soil information is available, and design details for the embankments are known, global stability analyses should be carried out to establish the minimum Factor of Safety<sup>2</sup> for the embankment configuration. The minimum Factor of Safety should be compared to the minimum target Factors of Safety. The minimum target Factors of Safety may be taken as 1.4 for the temporary or “short-term” condition and 1.6 for the permanent or “long-term” condition, based on the limited geotechnical information at the site and using the geotechnical resistance factors from Table 6.2 of CHBDC (2019) and MERO (2020). When more detailed foundation investigation is completed at the site (typical or high level of understanding), the resistance factor can be increased and the target Factor of Safety for the temporary and permanent conditions can be decreased accordingly.

For embankment heights greater than 8 m, a 2 m wide mid-height bench should be incorporated into the embankment configuration as per OPS 202.010 (*Slope Flattening*). Prior to construction of the embankments, all topsoil, fill materials, and peat/organic soil (although not encountered during the current investigation but anticipated based on the forested areas near the footprint of the embankments) should be stripped from the footprint of the new embankments and replaced with suitable granular fill (see Section 6.9).

<sup>2</sup> The Factor of Safety (FoS) is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The Factor of Safety is equal to the inverse of the product of the consequence factor,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$  (i.e.  $FoS = 1/(\Psi \cdot \phi_{gu})$ ).

Based on the results of the current foundation investigation and assuming embankment heights up to 10 m above ground surface, an adequate Factor of Safety against global slope failure is anticipated for conceptual planning purposes. It is assumed that approach embankments will be designed with side slopes no steeper than 2H:1V and will be constructed with suitable granular embankment fill and founded below any surficial unsuitable soils (topsoil, fill, peat/organics, or any soft cohesive soils or very loose to loose non-cohesive soils).

### 6.7.1 Settlement

The target settlement performance criteria for design of approach embankments are outlined in MTO's "Embankment Settlement Criteria for Design", dated July 2, 2010. In general, new embankments approaching structural elements such as bridge abutments are to be designed such that total settlement and rate of differential settlement do not exceed 25 mm, over a 20-year period following completion of construction.

The sources of total settlement associated with approach embankments are considered to include the following:

- Immediate settlement of the granular / cohesionless deposits (short-term);
- Primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory – long-term); and,
- Secondary time dependent (creep) consolidation of the cohesive deposits (long term).

Based on the results of the current foundation investigation, subsurface soils are anticipated to be comprised of stiff to very stiff cohesive soils underlain by non-cohesive soils, in turn, underlain by very dense or hard glacial till deposits some 10 m to 15 m below ground surface. Given that the cohesive soils are expected to be overconsolidated, and that settlement associated with the non-cohesive soils is immediate and would occur rapidly during construction, post-construction settlements are expected to be within tolerable values and are not anticipated to be a concern at the approach embankments.

Additional foundation investigation at the actual bridge crossing site, particularly to determine the thickness of cohesive soils and their associated drainage / consolidation characteristics, will need to be carried out to confirm whether or not settlement mitigation options need to be implemented. Assuming approach embankments up to about 10 m high, total settlements are estimated to range between about 75 mm and 150 mm based on the available borehole information. For preliminary design, and if cohesive soils are softer or extend to greater depths than those encountered during the current foundation investigation, consideration should be given to mitigation measures such as preloading the embankment footprint (with or without surcharge) or ground improvement.

As mentioned in Section 6.4, the settlement of the foundation soils due to the approach embankment loading (and any other foundation locations where the grade is to be raised) will need to be considered for design of any shallow footings (excess settlement in addition to the f-SLS geotechnical reaction) and/or deep foundations (i.e., associated downdrag forces).

### 6.7.2 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wingwalls should be designed in accordance with Section 6 of the CHBDC (2019) and will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutment walls and wingwalls:

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in general accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall in accordance with Figure C6.31(a) of the *Commentary to the CHBDC* (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

## 6.8 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete or steel elements (e.g. reinforcing steel) of foundations or related structures buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure can be assessed based on indicators such as soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. The analytical results of these indicators for the soil samples submitted for testing (Borehole PDD-1 sample 3 and PDD-2 sample 2) are summarized in Section 4.4 and discussed below, and the analytical laboratory test report is included in Appendix C.

### 6.8.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 (*Additional requirements for concrete subjected to sulphate attack*) to assess potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the samples of soil tested, when the designer is selecting the exposure class for foundations or buried structures, the effects of sulphates may not need to be considered.

### 6.8.2 Potential for Corrosion

Borehole PDD-1 sample 3 and Borehole PDD-2 sample 2 had measured pH values of 7.7 and 7.6, respectively. According to the MTO Gravity Pipe Design Guidelines (2014), a pH less than 5.5 is considered strongly acidic while a pH greater than 8.5 is considered strongly alkaline; both of which are indicative of an increased potential for corrosion. Thus, the measured pH is not considered to be detrimental to concrete durability. It should be noted that the water levels in the area are subject to seasonal fluctuations and variations due to the precipitation events and the soil/water chemistry could also be variable.



Borehole PDD-1 sample 3 and Borehole PDD-2 sample 2 had resistivity values of 7,800 and 7,600 ohm-cm, respectively, indicating that the soil corrosiveness is very low ( $6,000 \text{ ohm-cm} < R < 10,000 \text{ ohm-cm}$ ), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014). Appropriate corrosion protection should be applied to the foundation element / materials and given that the foundations are located adjacent to the highway shoulder / ditches and will be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class for any concrete as defined by CSA A23.1 Table 1.

These recommendations are provided as guidance only; the designer or design-builder should take the results of the laboratory testing into consideration for selecting appropriate materials and corrosion susceptibility for design service of the structure foundations. Ultimately, it is the designer’s decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) and the CHBDC (2019) are satisfied. In addition, the samples analysed were recovered some 300 m south of the actual bridge crossing location and the soil/water chemistry may vary considerably.

## 6.9 Construction Considerations

### 6.9.1 Subgrade Preparation and Approach Embankment Construction

Prior to construction of the new approach embankments, it is recommended that all unsuitable soils such as topsoil or organics, and existing surficial fill materials or loosened/softened soils (e.g. re-worked soils from existing agricultural activities or softer cohesive soils than those encountered during the current foundation investigation) be stripped from the embankment footprint and replaced with OPSS Select Subgrade Material (SSM), Granular A or Granular B material. Although not encountered during the current foundation investigation, the presence of peat/organic soil should be anticipated based on the forested areas near the footprint of the embankments.

Engineered fill for construction of the new embankments should consist of OPSS.PROV 1010 (*Aggregates*) granular materials (i.e., SSM, Granular A or Granular B). Earth fill consisting of suitable borrow material from elsewhere on the project site may also be considered where sufficient volumes are available and provided stability and settlement targets are met. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). Permanent embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular fill. Where earth fill is used, slightly flatter side slopes on the order of 2.25H:1V or shallower may be necessary depending on the composition of the material to reduce the potential for shallow surficial failures.

In accordance with MTO’s standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (*Slope Flattening*).

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding or pegged sod as per OPSS.PROV 803 (*Vegetative Cover*) should be established as soon as possible after construction of the embankments.

### 6.9.2 Temporary Excavations

All temporary excavations (i.e., those that are open for a relatively short time period where personnel are required to enter) must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. For conventional shallow foundation construction or for pile / caisson caps below approach embankment footprints, temporary excavations are expected to extend to at least 1.5 m (frost depth). If pile and/or caisson caps are “perched” within the approach embankments,

temporary excavations could be reduced or eliminated. Deeper excavations may be necessary if near surface cohesive soils with softer consistencies than those encountered in Boreholes PDD-1 and PDD-2 are encountered at the actual bridge site location.

In accordance with OHSA, the soil type classification for temporary excavations will need to be determined when more boreholes / soil information is available at the actual bridge location. Temporary excavations need to be observed by qualified personnel to confirm the OHSA classification during construction.

Based on Boreholes PDD-1 and PDD-2, Type 2 and Type 3 soils should be expected for excavations above the groundwater table, whereas Type 4 soils should be expected for the encountered soils below the groundwater level or where softer / looser soils are present. Temporary excavations within Type 4 soils should be made with side slopes no steeper than 3H:1V, while those within Type 3 and Type 2 soils (sloped to within 1.2 m of the bottom of the excavation) should be made with side slopes no steeper than 1H:1V.

Where required, temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*) and Special Provision 105S09. The lateral movement of the temporary protection systems must meet the applicable Performance Level as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation.

### 6.9.3 Control of Groundwater / Surface Water

Based on the piezometers installed in Boreholes PDD-1 and PDD-2, the groundwater level was measured to be near the ground surface and will likely be near the ground surface at the actual bridge site location approximately 300 m north of the boreholes (given the relatively flat, low-lying topography in the area of the site). Additional investigation is required to determine if artesian conditions are present at the bridge site location.

For conventional shallow foundation construction or deep foundations with the underside of footing / pile cap elevation below the frost depth (1.2 m below ground surface), it is likely that groundwater could be controlled by trenching or diversion ditches with sufficient sumps and pumps provided the near-surface soils are similar to those encountered in Boreholes PDD-1 and PDD-2 (i.e., cohesive fill or native clayey silt to clayey silt-silt). If artesian conditions are present, depressurization of the underlying non-cohesive soil deposits would be required to maintain basal stability. The requirements to control groundwater or depressurize water-bearing silts and sands could be minimized or eliminated if pile and/or caisson caps are “perched” within the approach embankments. Excavations extending below the near-surface cohesive soils and into the water-bearing silts and sands would likely require advanced dewatering using well points prior to excavation or possibly a temporary pressure relief well system to maintain stability of the excavation base during construction.

Dewatering operations should be in accordance with OPSS.PROV 517 (*Dewatering*) as referenced in OPSS.PROV 902 (*Excavation and Backfilling – Structures*). Inclusion of a special provision for foundation dewatering will need to be considered in the future contract documents or output specifications to address potential instability / base heave of the foundation subgrade, temporary flow diversion and pre-construction survey requirements, as applicable.

Construction water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self registration on the MECP’s Environmental Activity and Sector Registry (EASR), requiring a “Water Taking Plan” and a “Discharge Plan” (to be developed by the Design-

Builder). A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. The contractor will be responsible for obtaining any required discharge approvals.

Surface water must be directed away from the excavations at all times and properly diverted / controlled such that the integrity of any foundation subgrade is maintained.

To reduce erosion of the permanent embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments as per OPSS.PROV 803 (Vegetative Cover). Temporary erosion protection on exposed cuts / fills must be in accordance with OPSS.PROV 804 (Temporary Erosion Control).

## 6.10 Recommendations for Additional Work

The preliminary foundation recommendations provided in this report are based on the limited available subsurface information from the two boreholes advanced nearly 300 m away from the proposed underpass structure. Additional foundation investigation and assessment is recommended to be carried out such that the level of confidence for design meets a minimum “typical degree of site and prediction model understanding” for the ultimate bridge configuration.

The additional investigation will need to explore the subsurface soil and groundwater conditions at the location of the bridge abutments and piers, approach embankments and any associated retaining walls. Consideration should be given to advancing Cone Penetration Tests (seismic CPTs with dissipation testing) within the upper cohesive and silt deposits to further characterize the near surface cohesive soils encountered that are challenging to sample and interpret with conventional push equipment, as well as pressuremeter tests within the granular and lower cohesive till deposits to refine settlement estimates. Consideration could also be given to using specialized piston samplers (as opposed to conventional thin-walled Shelby tube extraction methods) as an attempt to collect a sufficient number of less disturbed samples of the clayey deposits containing silt/sand seams and additional consolidation tests performed accordingly. In addition, the extent (bottom) and consolidation characteristics of the cohesive soils and depth to competent soil (100-blow or hard / very dense soils) should be confirmed across the bridge and approach embankment footprints.

After more detailed foundation investigation is complete, the global stability of the approach embankments and any retaining walls will need to be checked and the magnitude of foundation settlements and mitigation measures (including estimated preload times, if applicable) will need to be assessed. When more details are known on actual loading conditions, the foundation types, sizes and geotechnical resistances will need to be checked and revised as necessary. Construction staging associated with the future extension of Professor Day Drive as it may relate to the foundation engineering aspects of the bridge site (e.g. advanced preloading) will also need to be assessed.

The use of GSC 5<sup>th</sup> Generation or 6<sup>th</sup> Generation seismic hazard maps to define the Site Class should be confirmed for detail design. Geophysics testing, such as Multi-Channel Analysis of Surface Waves (MASW) or Vertical Seismic Profiling (VSP), may provide a more favourable Site Class designation at the actual bridge locations, and such testing can be considered during detail design. Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*.

Standpipe piezometers should be installed in the future boreholes and maintained operational to allow for continued monitoring of the groundwater level to assess dewatering requirements and if artesian conditions are

present. Prior to or during construction, the piezometers will need to be decommissioned in accordance with Ontario Regulation 903 (as amended).

## **7.0 CLOSURE**

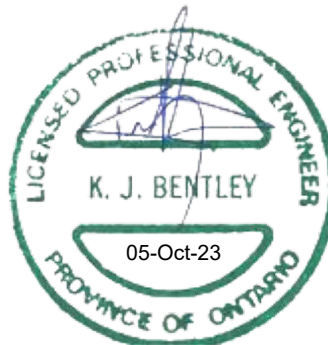
This Preliminary Foundation Design Report was prepared by Mr. Mark Henderson, P.Eng., a Geotechnical Engineer at WSP Golder. Mr. Kevin Bentley, P.Eng., a Geotechnical Engineer and MTO Foundations Designated Contact with WSP Golder, conducted a technical and quality review of the report.

## Signature Page

### WSP Golder



Mark Henderson, P.Eng.  
*Geotechnical Engineer*



Kevin J. Bentley, P.Eng.  
*MTO Foundations Designated Contact*

MH/KJB/al

[https://golderassociates.sharepoint.com/sites/120387/project files/6 deliverables/foundations/professor day drive/final/19136074-r-rev0-pfidr-pdd-2023'10'05.docx](https://golderassociates.sharepoint.com/sites/120387/project%20files/6%20deliverables/foundations/professor%20day%20drive/final/19136074-r-rev0-pfidr-pdd-2023'10'05.docx)

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- Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Standards Association, 2014. Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-14. CSA Group.
- Chapman, L.J. and Putnam, D. F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.
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- National Resources Canada, 2017. Earthquake Hazard. [http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index\\_2015-en.php](http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php).
- Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.
- Terzaghi, K.V., 1955. Evaluation of Coefficient of Subgrade Reaction. Getechnique, 5(4): 297-326.
- Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.
- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

### ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

### Canadian Standards Association (CSA):

CAN/CSA A23.114 Concrete Materials and Methods of Concrete Construction

### Commercial Software:

Settle3 (Version 5.015) by Rocscience Inc.

### Ontario Provisional Standard Drawing:

OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements
OPSD 3000.100	Steel H-Pile Driving Shoe
OPSD 3001.100	Steel Tube Driving Shoe

### Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting



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OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 803	Construction Specification for Vegetative Cover
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.PROV 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
Special Provision 109F57	Amendment to OPSS.PROV 903

**Ontario Regulations**

Ontario Regulation 903 Wells (as amended)

Ontario Regulation 213 Construction Projects (as amended)

**Ministry of Transportation, Ontario**

Gravity Pipe Design Guidelines, Circular Culverts and Storm Sewers, April 2014.

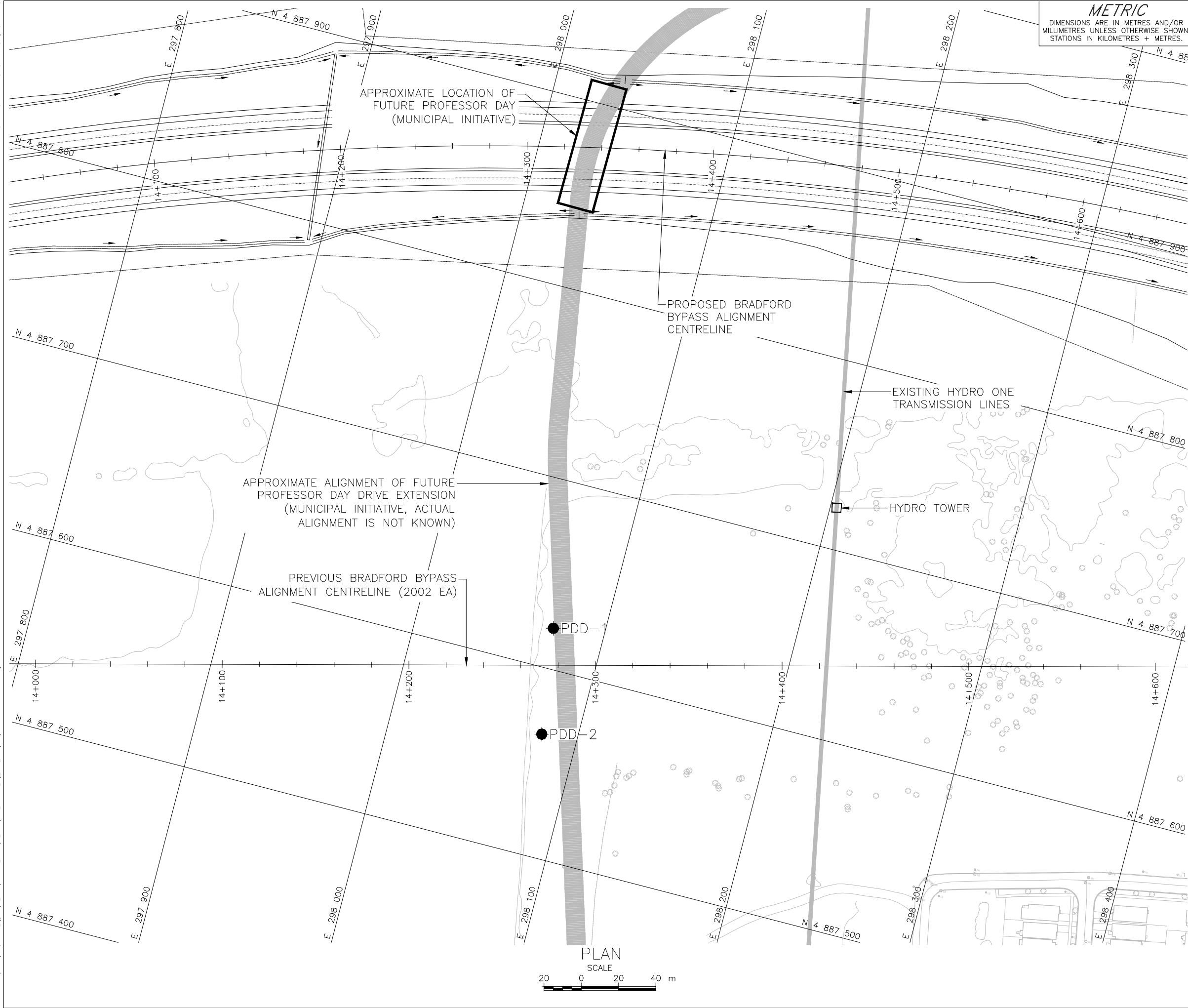
MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.

Provincial Engineering Memorandum #20201, Material Engineering and Research Office (MERO), March 23, 2020

Guideline for MTO Foundation Engineering Services, Version 3, dated April 2022

Table 1: Comparison of Foundation Alternatives – Future Professor Day Drive Underpass

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Strip / Spread footings founded on near surface cohesive soils	Marginally feasible at foundation elements	<ul style="list-style-type: none"><li>■ Conventional construction if no artesian conditions are present at actual bridge site location.</li></ul>	<ul style="list-style-type: none"><li>■ Depressurization of underlying silts and sands may be required if artesian conditions are present to maintain stable foundation subgrade.</li><li>■ Does not allow for conventional integral abutment design.</li><li>■ Preloading likely required to induce settlements from approach embankment loading in order to achieve design f-SLS value for shallow footings.</li><li>■ Relatively low f-ULS and f-SLS values in cohesive deposits.</li></ul>	<ul style="list-style-type: none"><li>■ Costs may be comparable or higher than deep foundation options when additional costs for dewatering systems and preloading are considered.</li></ul>	<ul style="list-style-type: none"><li>■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual foundation elements.</li><li>■ Risk of deeper excavation and increased dewatering (or depressurization if artesian conditions are encountered) efforts.</li><li>■ Risk of disturbance to founding subgrade if adequate dewatering or depressurization is not provided in saturated sands and silts.</li><li>■ Preloading likely required and risk of higher than anticipated settlements and/or preload times from approach embankment loading make this option less practical and not preferred.</li></ul>
“Perched” spread footings founded on a compacted granular pad within approach embankments	Not feasible for pier(s).  Marginally feasible at abutments	<ul style="list-style-type: none"><li>■ Conventional construction</li><li>■ Granular pad can be constructed within approach embankment for abutment locations to improve geotechnical resistance values, although f-SLS values may still be marginally adequate.</li><li>■ Founding level can easily be adjusted within approach embankment.</li><li>■ Depth of excavation, dewatering (or potentially depressurization if artesian conditions are encountered) effort, and height of abutment wall stems (i.e. volume of concrete / steel) can be reduced.</li></ul>	<ul style="list-style-type: none"><li>■ f-SLS values may still be marginally adequate and subexcavation of compressible near-surface soils or ground improvement may be required to reduce settlements.</li><li>■ Dewatering of saturated silts and sands may be required if subexcavation of compressible soils and replacement with granular soils is needed.</li><li>■ Does not allow for conventional integral abutment design.</li><li>■ Preloading likely required to induce settlements from approach embankment loading in order to achieve design f-SLS value for shallow footings.</li></ul>	<ul style="list-style-type: none"><li>■ Costs may be comparable to spread footings on native soil option given similar costs for dewatering (and potentially depressurization) and will depend on balance between increased volume of engineered fill versus reduced volume of concrete / steel and dewatering requirements.</li></ul>	<ul style="list-style-type: none"><li>■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual foundation element locations; although lower risk of geotechnical resistances being influenced as much as spread footings directly on native subgrade.</li><li>■ Lower risk to foundation performance if subgrade soils are wet / disturbed during construction as compacted granular pad will distribute and decrease loads on native soils.</li><li>■ Preloading required and risk of higher than anticipated settlements and/or preload times from approach embankment loading make this option less practical and not preferred.</li></ul>
Steel H-piles or tube piles driven into competent “end-bearing” stratum	Feasible at foundation elements	<ul style="list-style-type: none"><li>■ Conventional construction methods for driven steel pile foundations</li><li>■ Higher axial resistances available compared to shallow footings.</li><li>■ Pile caps perched within approach embankments can be considered to reduce or eliminate dewatering / subexcavation / depressurization requirements.</li><li>■ Allows for integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Noise and vibrations to adjacent properties, although limited residential properties are near the site (closest residential development is 500 m southeast of the site).</li><li>■ Dewatering / depressurization measures may be required at abutments for the construction of pile caps, unless perched in embankment fill at abutments.</li><li>■ Driving shoes and/or thicker pile section may be required to drive into the “100-blow” glacial till soils that may contain cobbles / boulders.</li></ul>	<ul style="list-style-type: none"><li>■ Lower relative cost than drilled shafts (caissons) and may be comparable to spread footings if dewatering/depressurization and subexcavation of unsuitable soils can be reduced by designing perched pile caps.</li></ul>	<ul style="list-style-type: none"><li>■ Reduced impact on design if variable near surface soils are encountered during detailed investigation.</li><li>■ Risk of piles “hanging up” or being deflected from alignment when driving through glacial till deposits or deposits that may contain pockets of gravel or cobbles and boulders.</li><li>■ Depending on thickness of cohesive soils at actual bridge location, settlement of approach embankments could cause potential downdrag loads on piles (reduced capacity) which will need to be considered in design and/or mitigated and monitored during construction.</li></ul>
Drilled Shafts (Caissons) installed into competent “end-bearing” stratum	Feasible at foundation elements	<ul style="list-style-type: none"><li>■ Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements.</li><li>■ May be designed to eliminate pile cap and associated temporary excavations / dewatering as the caissons could be cast continuously with structural columns to the underside of the superstructure.</li></ul>	<ul style="list-style-type: none"><li>■ Temporary or permanent liner or special measures such as polymer slurry will be required to reduce risk of loosening / softening of the sides of the drilled shaft and heave / blow-out at base of shaft during drilling and concrete placement (by tremie methods).</li><li>■ Generation and disposal of soil cuttings / slurry during drilled shaft advancement</li><li>■ Does not allow for conventional integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Higher relative cost than shallow foundations.</li><li>■ Higher cost than piles (per element) but reduced dewatering / subexcavation costs if caissons are cast continuously with structural columns to eliminate pile cap.</li></ul>	<ul style="list-style-type: none"><li>■ Reduced impact on design if variable near surface soils are encountered during detailed investigation.</li><li>■ In-situ testing to confirm design resistance is challenging and expensive (e.g. static pile load test) compared to PDA testing. Challenging to inspect the shaft and base of the drilled shafts due to the need for liners, slurry, and tremie concrete methods (especially if artesian conditions present).</li><li>■ Depending on thickness of cohesive soils at actual bridge location, settlement of approach embankments could cause potential downdrag loads on caissons, which will need to be considered in design and/or mitigated and monitored during construction.</li><li>■ Risk of difficulties penetrating through soil deposits that may contain pockets of gravel or cobbles and boulders (to be confirmed during detail design).</li></ul>



**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No.

BRADFORD BYPASS  
PROFESSOR DAY DRIVE UNDERPASS  
BOREHOLE LOCATIONS



LEGEND			
	Borehole	Current Investigation	

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
PDD-1	262.2	4887621.6	298072.8
PDD-2	262.1	4887565.1	298081.0



**NOTES**  
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

**REFERENCE**  
Base plans provided in digital format by Aecom, drawing file nos. X-Base\_Bradford Bypass.dwg and BRADFORD BY-PASS OG\_Combined.xml, received January 11, 2022.  
Previous horizontal alignment provided in digital format by Aecom, drawing file no. BBP-Hwy 400 IC Alignments.xml, received January 26, 2022.  
Horizontal alignments provided in digital format by Aecom, drawing file no. X-60636190-C-DES-All Alignments.xml, received May 12, 2023.  
Design plan provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Overall Plan.dwg, received May 12, 2023.

NO.	DATE	BY	REVISION
Geocres No. 31D00-830			
HWY. BRADFORD BYPASS	CHKD. MH	PROJECT NO. 19136074	DIST. .
SUBM'D. MA	CHKD. KJB	DATE: 10/04/2023	SITE: .
DRAWN: DD	CHKD. KJB	APPD. .	DWG. 1

**APPENDIX A**

# Borehole Records

# ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

## MINISTRY OF TRANSPORTATION, ONTARIO

### PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
FINES	Classified by plasticity	<0.075	< (200)

### MODIFIERS FOR SECONDARY COMPONENTS<sup>1,2</sup>

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component ( <i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some ( <i>i.e.</i> , some sand)
≤ 10	trace ( <i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

### PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

#### Cone Penetration Test (CPT)

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

#### Dynamic Cone Penetration Resistance (DCPT); $N_d$ :

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

### SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

### SOIL TESTS

w	water content
PL, $w_p$	plastic limit
LL, $w_L$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

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

### COARSE-GRAINED SOILS

#### Compactness<sup>1</sup>

Term	SPT 'N' (blows/0.3m) <sup>2</sup>
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

### FINE-GRAINED SOILS

#### Consistency

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

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

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

### Field Moisture Condition

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

# LIST OF SYMBOLS

## MINISTRY OF TRANSPORTATION, ONTARIO

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

### I. GENERAL

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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta\sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

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

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

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_{\alpha(e)}$	secondary compression index
$C_{\alpha}$	rate of secondary compression
$C_{\alpha(e)}$	modified secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

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

Notes: 1  
2

$\tau = c' + \sigma' \tan \phi'$   
shear strength = (compressive strength)/2



PROJECT 19136074

## RECORD OF BOREHOLE No. PDD-1

Sheet 1 of 3

METRIC

G.W.P. Assignment No 2019-E-0048

LOCATION N 4887621.6; E 298072.8 NAD83 / MTM Zone 10 (LAT. 44.128559; LONG. -79.584064)

ORIGINATED BY SS

DIST Central HWY BBP - Professor  
Day Drive

BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary

COMPILED BY MTI / MA

DATUM CGVD28 Surface Elevation:262.2 m

DATE Apr 19, 2021 - Apr 21, 2021

CHECKED BY MH


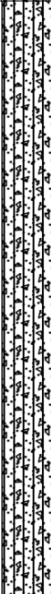

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
0.0	CLAYEY SILT (CL), trace gravel, trace organics (FILL) Stiff Dark brown Moist		1	SS	9		262														
261.5																					
0.7	CLAYEY SILT (CL), trace sand, becoming wet at about 2.3 m depth (Elev. 259.9 m) Stiff Brown mottled to grey, oxidation staining Moist		2	SS	11		261														
			3	SS	12																
							260														
	- 2.3 to 2.9 m: Spoon was wet when it came out of hole.		4	SS	10												0	7	63	30	
3.0																					
259.2	CLAYEY SILT-SILT (CL-ML) to SILT (ML) and sand, trace gravel Stiff Brown to grey Moist		5	SS	12		259														
			6	SS	13		258										3	36	48	13	
			7	SS	14		257														
256.6																					
5.6	SILT (ML) and sand, trace clay, trace to some gravel Loose to dense Grey Wet		8	SS	36		256														
							255														
			9	SS	9		254										5	40	45	10	
							253														
	- 9.1 m: Auger grinding at about 9.1 m depth (Elev. 253.1 m)		10	SS	34												13	36	42	9	

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>30</sup>% STRAIN AT FAILURE



PROJECT	19136074	LOCATION	N 4887621.6; E 298072.8 NAD83 / MTM Zone 10 (LAT. 44.128559; LONG. -79.584064)	RECORD OF BOREHOLE No. PDD-1	Sheet 2 of 3	METRIC
G.W.P.	Assignment No 2019-E-0048	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	ORIGINATED BY	SS	
DIST	Central HWY BBP - Professor Day Drive	DATE	Apr 19, 2021 - Apr 21, 2021	COMPILED BY	MTI / MA	
DATUM	CGVD28 Surface Elevation:262.2 m			CHECKED BY	MH	

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR	SA	SI	CL	REMARKS	
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	N* VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL							
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100	W <sub>p</sub>	W							W <sub>i</sub>
														NP Nonplastic								-----○-----
	SILT (ML) and sand, trace clay, trace to some gravel Loose to dense Grey Wet						252															
			11	SS	27																	
							251															
			12	SS	20		250							○								
13.3							249															
249.0	SILTY SAND (SM), trace clay, trace to some gravel, (TILL) Very Dense Grey Wet																					
			13	SS	73/0.12																	
							248															
	- 15.2 m: Auger grinding at about 15.2 m depth (Elev. 247.0 m)		14	SS	72/0.12		247							○			10	55	30	5		
							246															
244.9							245															
17.3	CLAYEY SILT-SILT (CL-ML) to SILT (ML), (TILL) Hard Grey Wet																					
			15	SS	65/0.10		244															
							243															

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>30</sup>% STRAIN AT FAILURE

PROJECT	19136074	<b>RECORD OF BOREHOLE No. PDD-1</b>			Sheet 3 of 3	<b>METRIC</b>	
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887621.6; E 298072.8 NAD83 / MTM Zone 10 (LAT. 44.128559; LONG. -79.584064)			ORIGINATED BY	SS
DIST	Central HWY BBP - Professor Day Drive	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary			COMPILED BY	MTI / MA
DATUM	CGVD28 Surface Elevation:262.2 m	DATE	Apr 19, 2021 - Apr 21, 2021			CHECKED BY	MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								NP Nonplastic													
									20	40	60	80	100	20	40	60					
	CLAYEY SILT-SILT (CL-ML) to SILT (ML), (TILL) Hard Grey Wet						242														
							241														
240.7			16	SS	41/0.05																
21.5	End of Borehole Notes: 1. Hollow Stem Augers to 3.05 m and then switched to mud rotary. 2. Groundwater was encountered at 2.3 m (Elev. 259.9 m) prior to initiation of mud rotary. 3. Water level in piezometer measured as follows: Depth(m) El. (m) Date 0.91 261.3 04/21/21 0.22 262.0 12/10/21 0.41 261.8 02/04/22 4. Water inside piezometer was frozen at 0.04 m above ground surface (Elev. 262.2 m) on Feb 28, 2023.						240														
							239														
							238														
							237														
							236														
							235														
							234														
							233														

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>3</sup>% STRAIN AT FAILURE

PROJECT 19136074

## RECORD OF BOREHOLE No. PDD-2

Sheet 1 of 3

METRIC

G.W.P. Assignment No 2019-E-0048

LOCATION N 4887565.1; E 298081 NAD83 / MTM Zone 10 (LAT. 44.12805; LONG. -79.58396)

ORIGINATED BY SS

DIST Central HWY BBP - Professor  
Day Drive

BOREHOLE TYPE 210 Hollow Stem Auger; Mud Rotary

COMPILED BY MTI/MA

DATUM CGVD28 Surface Elevation:262.1 m

DATE Apr 14, 2021 - Apr 19, 2021

CHECKED BY MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m <sup>3</sup>	GR	SA	SI	CL	REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100								
0.0	CLAYEY SILT (CL), trace sand, trace gravel, trace to some organic (FILL) Firm Black to brown Dry		1	SS	5		262														
261.4																					
0.7	CLAYEY SILT (CL), trace sand, trace gravel, oxidation staining Stiff to very stiff Brown Moist to wet		2	SS	10		261														
			3	SS	19		260														
			4	SS	16																
259.1																					
3.0	CLAYEY SILT-SILT (CL-ML) to SILT (ML) and sand, trace gravel Stiff to very stiff Grey Moist to wet		5	SS	14		259														
			6	SS	16		258										6	37	46	11	
			7	SS	13		257														
256.5																					
5.6	SILT (ML) and sand, trace clay, trace gravel Hard Grey Wet		8	SS	32		256										6	40	44	10	
			9	SS	38		255														
253.4																					
8.7	SILTY SAND (SM), trace clay, trace gravel to gravelly, (TILL) Very Dense Grey Moist Becoming wet at about 11.7 m depth (Elev. 250.4 m)		10	SS	71		253										25	31	38	6	

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>30</sup>% STRAIN AT FAILURE

PROJECT 19136074

## RECORD OF BOREHOLE No. PDD-2

Sheet 2 of 3

METRIC

G.W.P. Assignment No 2019-E-0048

LOCATION N 4887565.1; E 298081 NAD83 / MTM Zone 10 (LAT. 44.12805; LONG. -79.58396)

ORIGINATED BY SS

DIST Central HWY BBP - Professor  
Day Drive

BOREHOLE TYPE 210 Hollow Stem Auger; Mud Rotary

COMPILED BY MTI/MA

DATUM CGVD28 Surface Elevation: 262.1 m

DATE Apr 14, 2021 - Apr 19, 2021

CHECKED BY MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m <sup>3</sup>	GR SA SI CL				REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100					20 40 60								
	SILTY SAND (SM), trace clay, trace gravel to gravelly, (TILL) Very Dense Grey Moist Becoming wet at about 11.7 m depth (Elev. 250.4 m)		11	SS	86		252														
							251														
							250														
			12	SS	57		249														
							248														
			13	SS	100		247														
							246														
			14	SS	74		245														
							244														
			15	SS	100/0.28		243														
245.0 17.1	CLAYEY SILT-SILT (CL-ML) to SILT (ML) (TILL) Hard Grey Wet																				

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>3</sup>% STRAIN AT FAILURE

## METRIC

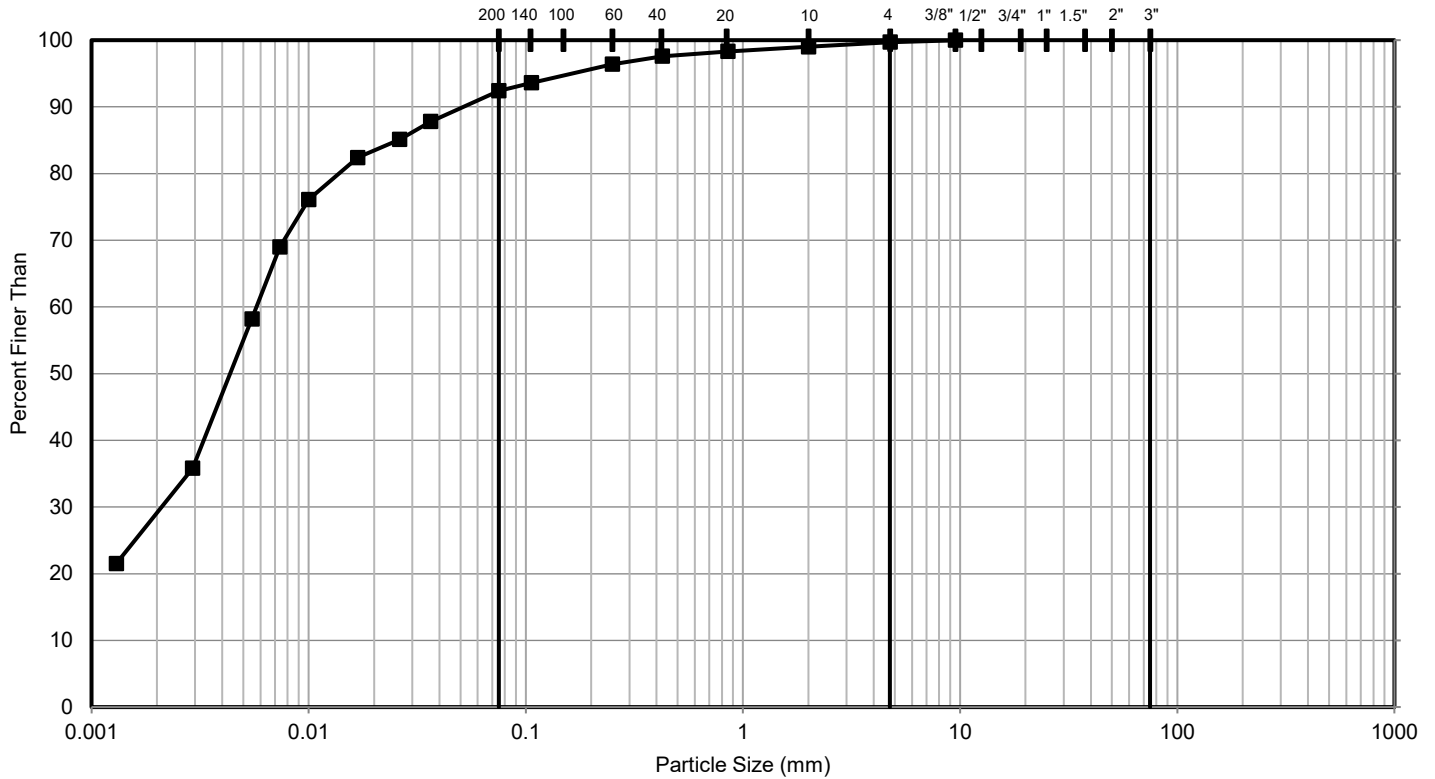
CHECKED BY                      MH

<sup>+</sup>, x<sup>3</sup> : Numbers refer to Sensitivity    o<sup>3</sup>% STRAIN AT FAILURE

**APPENDIX B**

# Geotechnical Laboratory Test Results

## Grain Size Distribution - Clayey Silt (CL)



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	PDD-1	4	2.3 - 2.9	259.9 to 259.3

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2023-07-07

DESIGNED MH

PREPARED MH

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Professor Day Drive

TITLE

Grain Size Distribution  
Clayey Silt (CL)

PROJECT NO.

19136074

CONTROL

1000

REV.

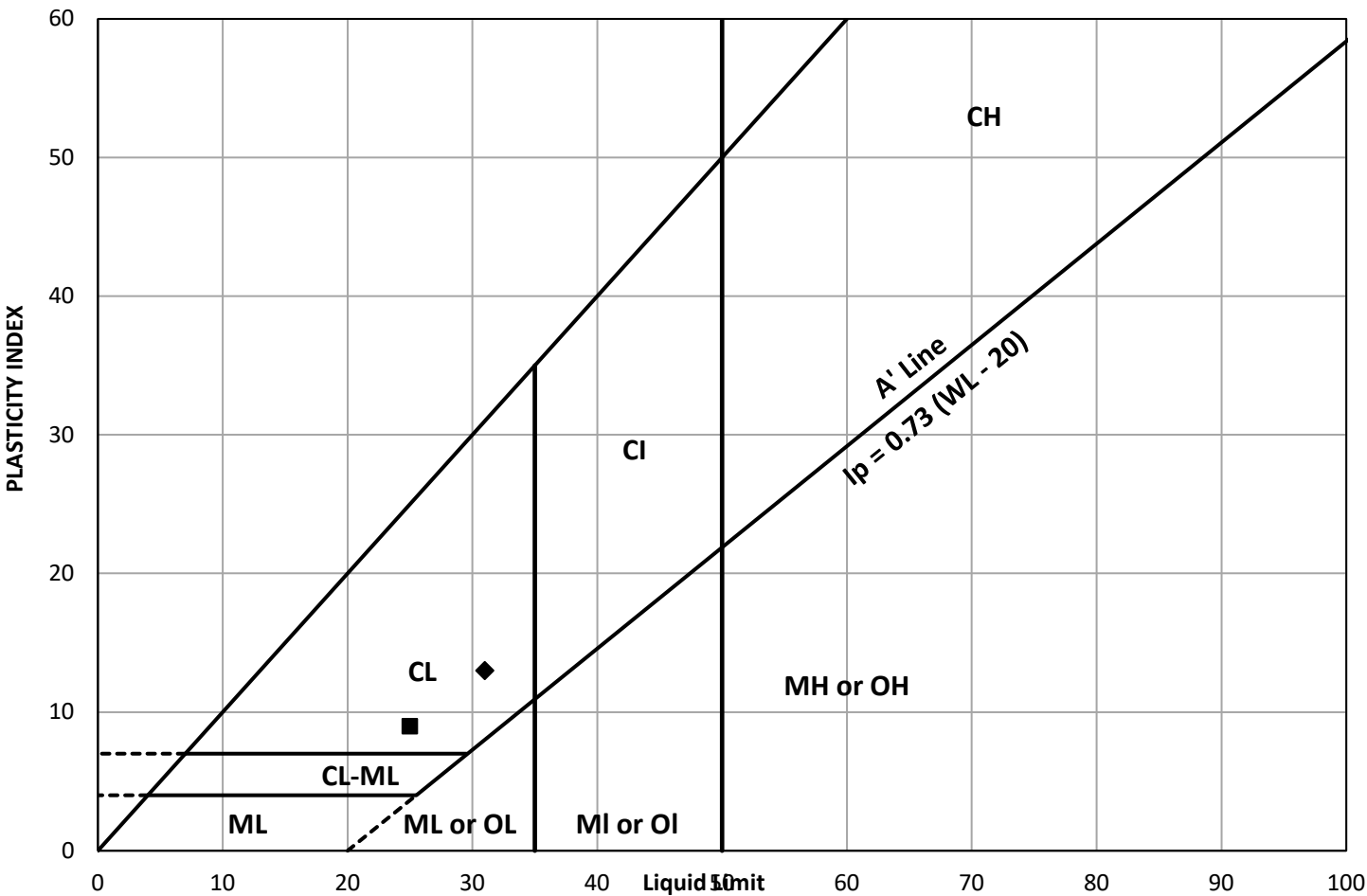
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FIGURE

B1



Plasticity Chart - Clayey Silt (CL)



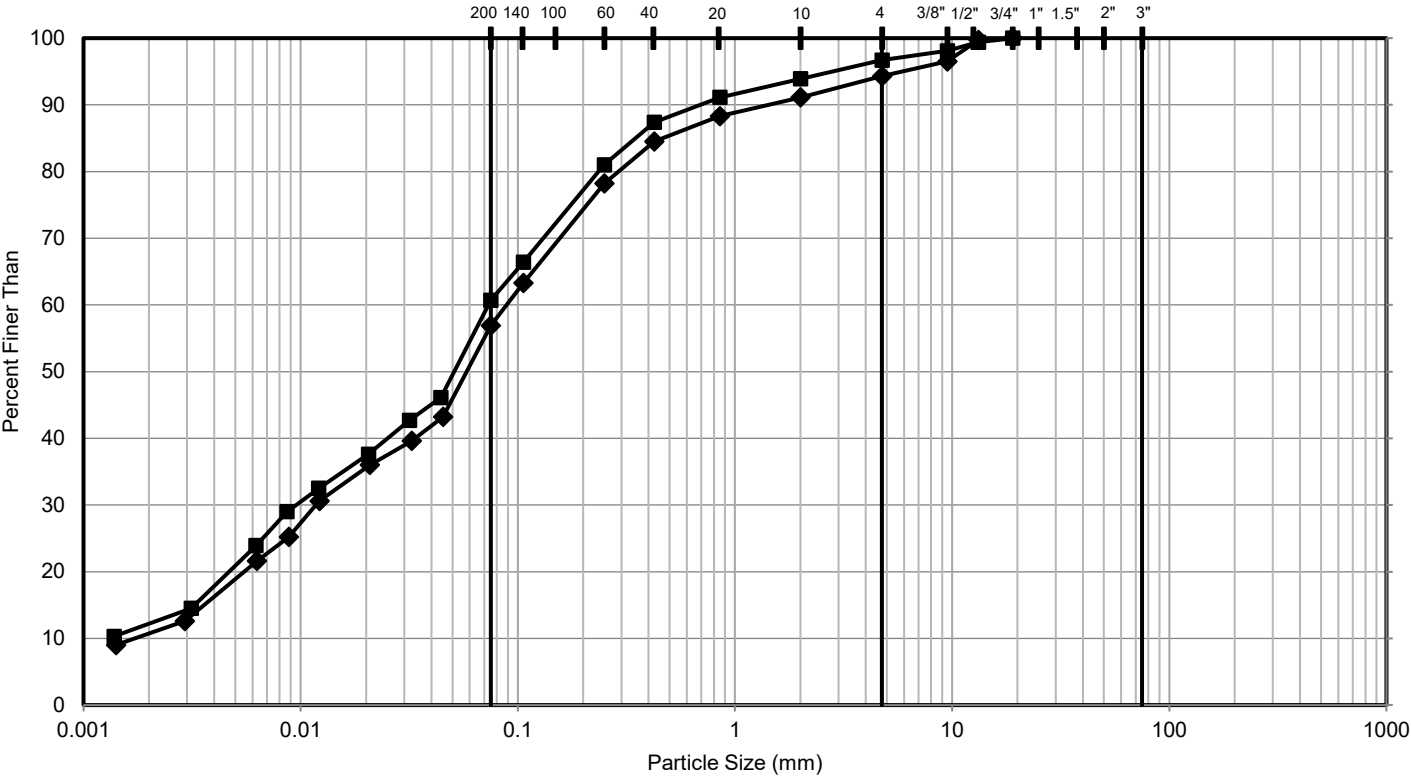
	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	PDD-1	4	2.3 - 2.9	21.9	25	16	9
◆	PDD-2	3	1.5 - 2.1	21.2	31	18	13

CLIENT			
AECOM / MTO			
	CONSULTANT	YYYY-MM-DD	2023-07-07
	DESIGNED		MH
	PREPARED		MH
	REVIEWED		KJB
	APPROVED		KJB

PROJECT		
Bradford Bypass - Professor Day Drive		
TITLE		
Plasticity Chart - Clayey Silt (CL)		
PROJECT NO.	CONTROL	FIGURE
19136074	1000	B2

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Professor Day Drive/DRAFT/Appendix B - Geotech Lab/Working Excel Files | FILE NAME: Laboratory Particle Size Distribution MTO.xlsm

Grain Size Distribution - Clayey Silt-Silt (CL-ML) to Silt (ML)



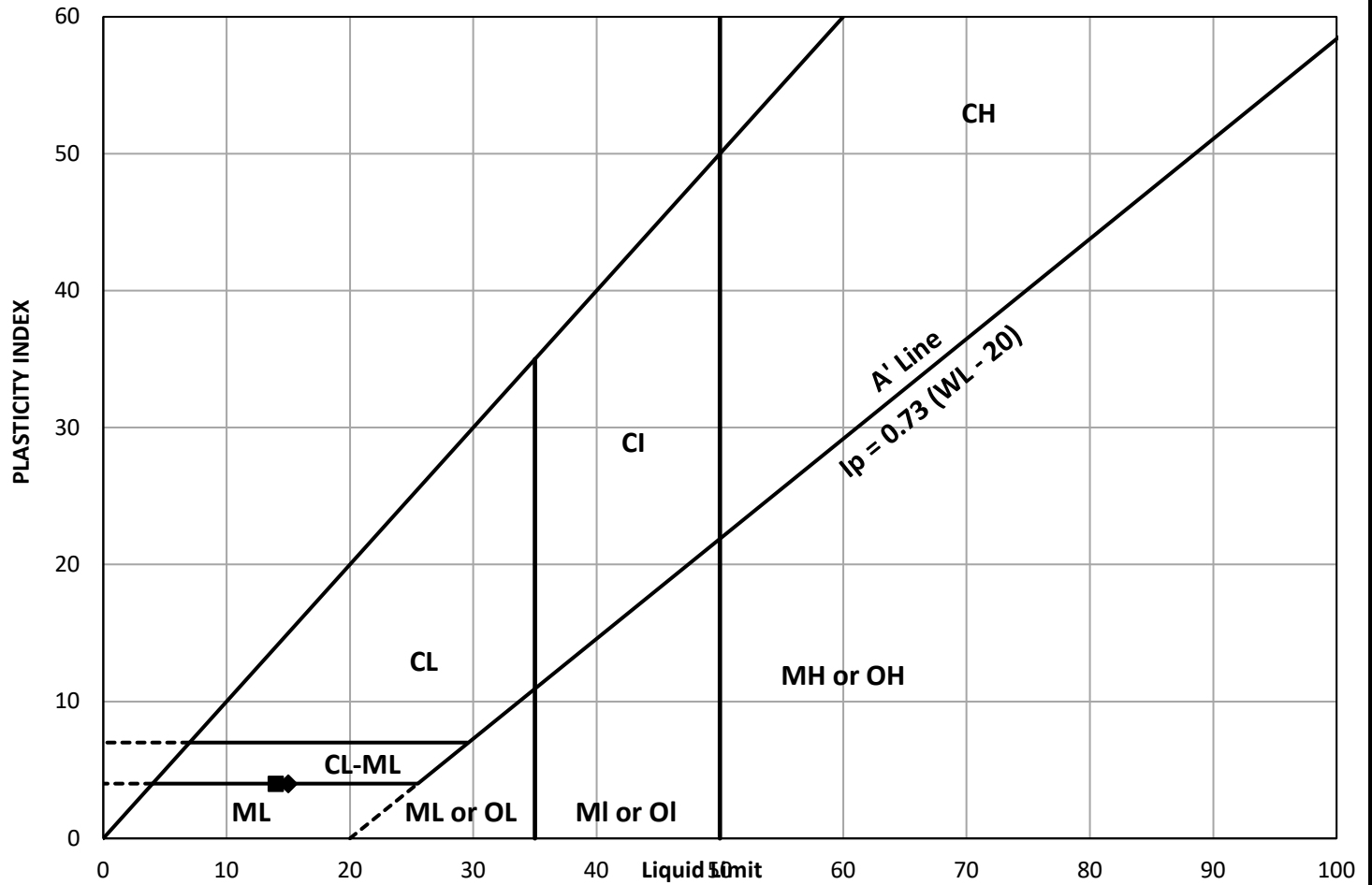
FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	PDD-1	6	3.8 - 4.4	258.4 to 257.8
◆	PDD-2	6	3.8 - 4.4	258.3 to 257.7

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Professor Day Drive	
CONSULTANT	YYYY-MM-DD	2023-07-07	
	DESIGNED	MH	
	PREPARED	MH	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		Grain Size Distribution	
		Clayey Silt-Silt (CL-ML) to Silt (ML)	
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B3



# Plasticity Chart - Clayey Silt-Silt (CL-ML) to Silt (ML)

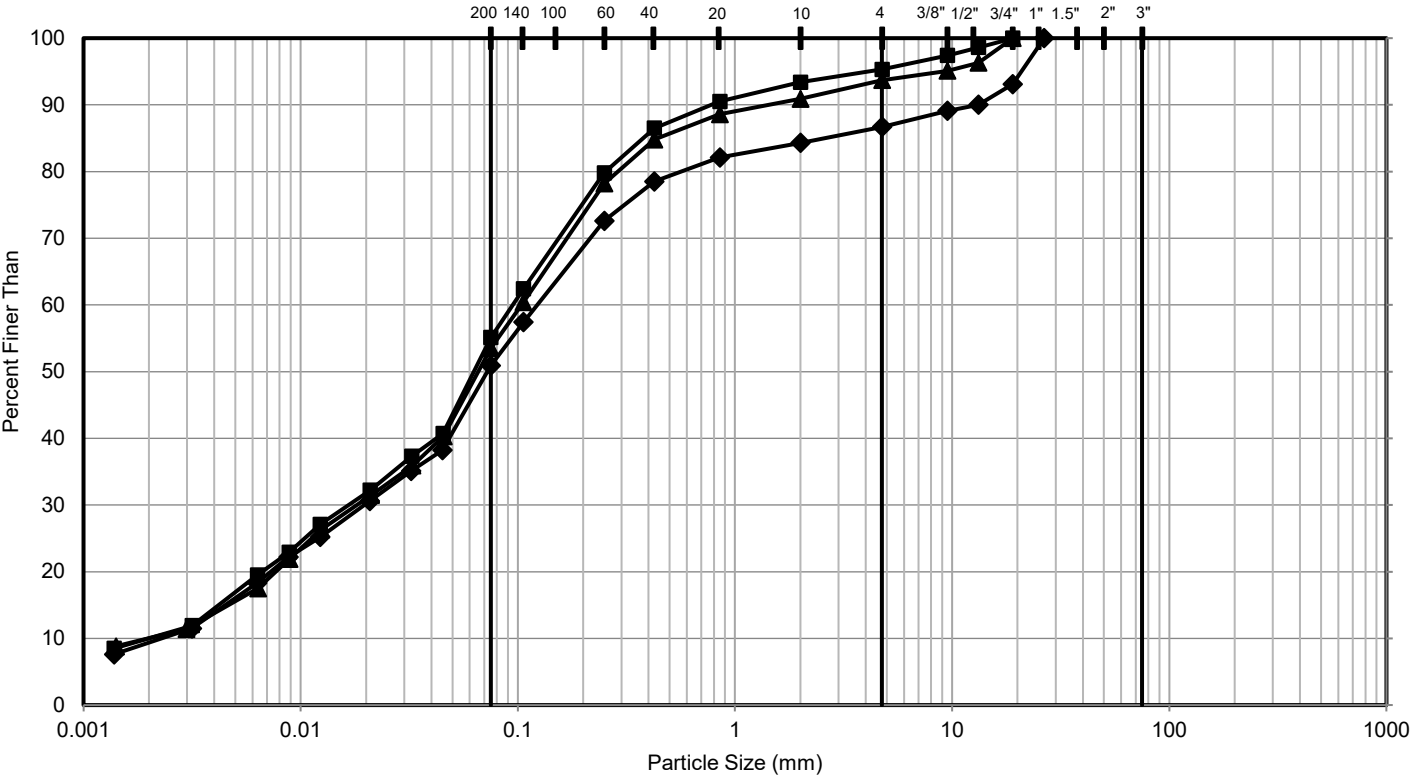


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	PDD-1	6	3.8 - 4.4	10.3	14	10	4
◆	PDD-2	6	3.8 - 4.4	9.7	15	11	4

CLIENT			
AECOM / MTO			
	CONSULTANT	YYYY-MM-DD	2023-07-07
	DESIGNED		MH
	PREPARED		MH
	REVIEWED		KJB
	APPROVED		KJB

PROJECT		
Bradford Bypass - Professor Day Drive		
TITLE		
Plasticity Chart - Clayey Silt-Silt (CL-ML) to Silt (ML)		
PROJECT NO.	CONTROL	FIGURE
19136074	1000	B4

Grain Size Distribution - Silt (ML) and sand



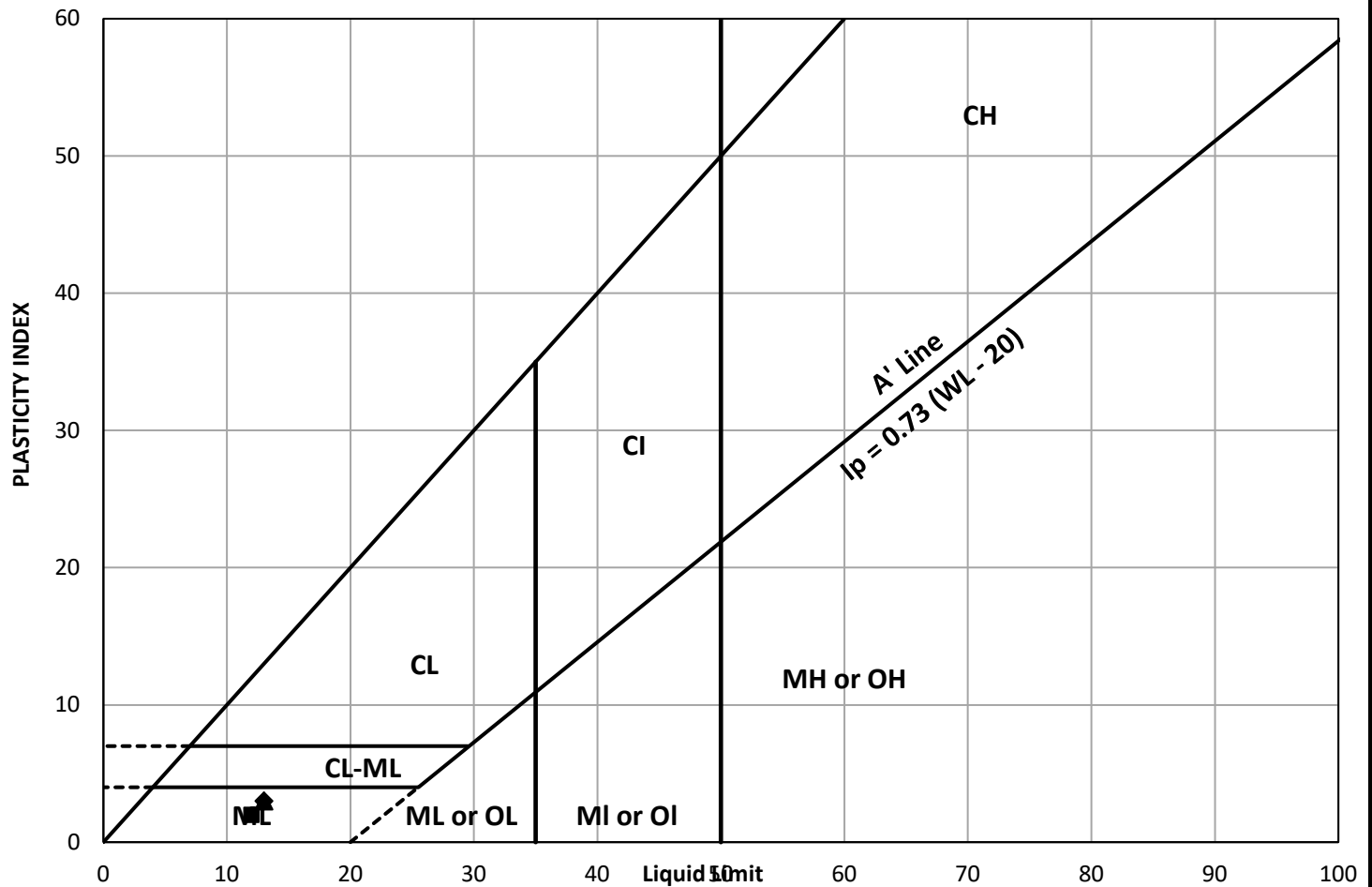
FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	PDD-1	9	7.6 - 8.2	254.6 to 254.0
◆	PDD-1	10	9.1 - 9.8	253.1 to 252.5
▲	PDD-2	8	6.1 - 6.7	256.0 to 255.4

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Professor Day Drive	
CONSULTANT	YYYY-MM-DD	2023-07-07	
	DESIGNED	MH	
	PREPARED	MH	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		Grain Size Distribution	
		Silt (ML) and sand	
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B5



## Plasticity Chart - Silt (ML) and sand



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	PDD-1	9	7.6 - 8.2	10.4	12	10	2
◆	PDD-1	10	9.1 - 9.8	8.1	13	10	3
▲	PDD-2	8	6.1 - 6.7	8.5	13	10	3

CLIENT

AECOM / MTO

CONSULTANT

**wsp GOLDER**

YYYY-MM-DD

2023-07-07

DESIGNED

MH

PREPARED

MH

REVIEWED

KJB

APPROVED

KJB

PROJECT

Bradford Bypass - Professor Day Drive

TITLE

Plasticity Chart - Silt (ML) and sand

PROJECT NO.

19136074

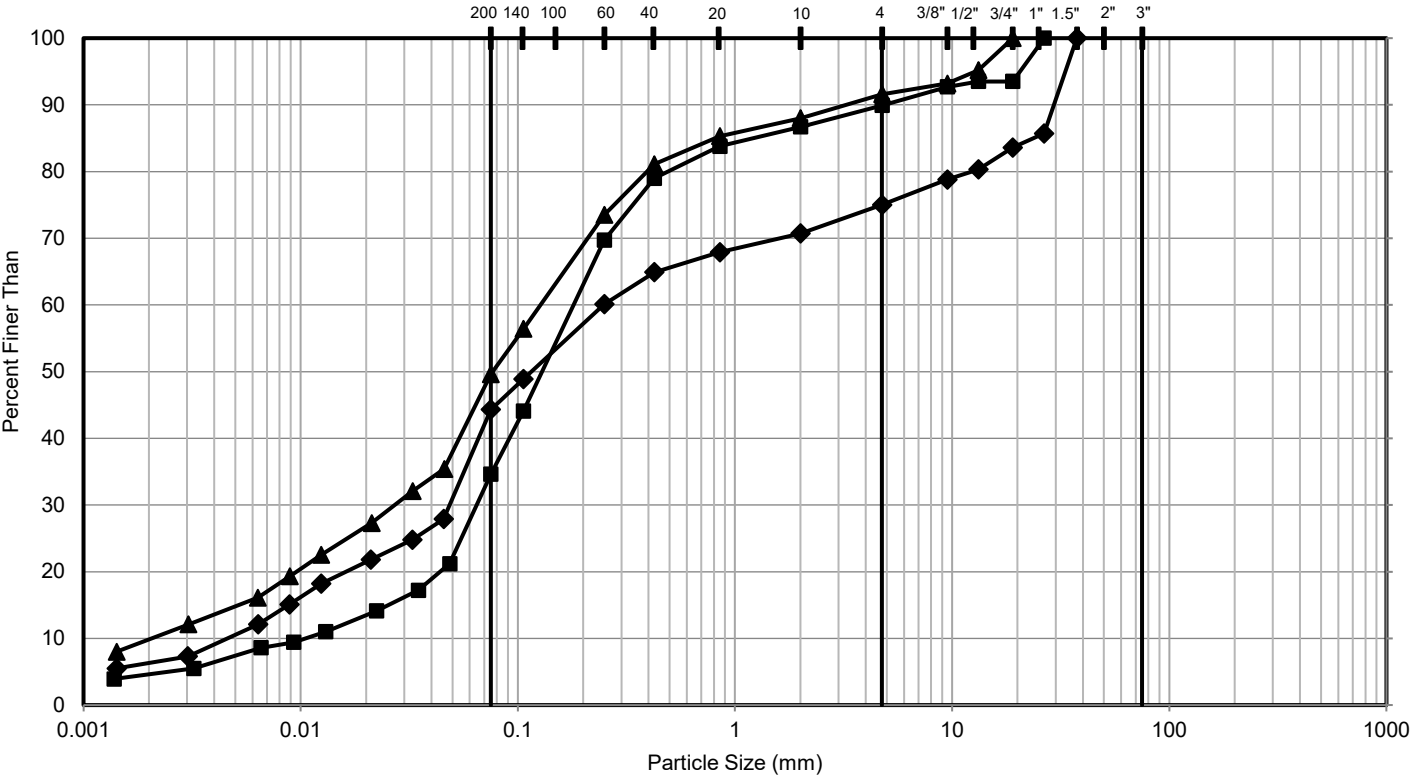
CONTROL

1000

FIGURE

B6

Grain Size Distribution - Silty Sand (SM) (TILL)



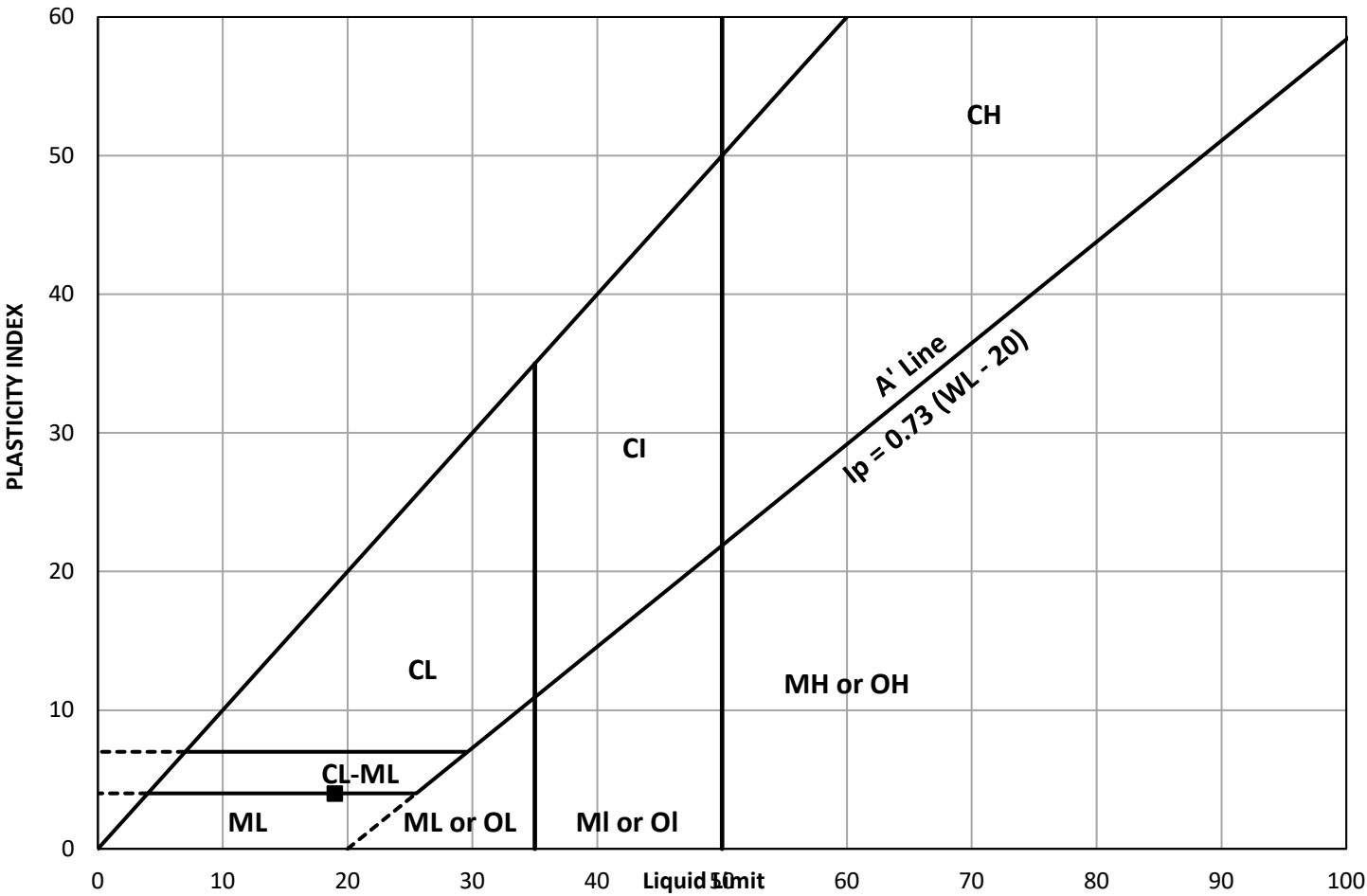
FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	PDD-1	14	15.2 - 15.5	247.0 to 246.7
◆	PDD-2	10	9.1 - 9.8	252.9 to 252.3
▲	PDD-2	12	12.2 - 12.8	249.9 to 249.3

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Professor Day Drive	
CONSULTANT	YYYY-MM-DD	2023-07-07	
	DESIGNED	MH	
	PREPARED	MH	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		Grain Size Distribution	
		Silty Sand (SM) (TILL)	
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B7



Plasticity Chart - Clayey Silt-Silt (CL-ML) to Silt (ML) (TILL)




	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	PDD-2	15	18.3 - 18.7	16.7	19	15	4

CLIENT

AECOM / MTO

CONSULTANT

 **GOLDER**

YYYY-MM-DD

2023-07-07

DESIGNED

MH

PREPARED

MH

REVIEWED

KJB

APPROVED

KJB

PROJECT

Bradford Bypass - Professor Day Drive

TITLE

Plasticity Chart - Clayey Silt-Silt (CL-ML) to Silt (ML) (TILL)

PROJECT NO.

19136074

CONTROL

1000

FIGURE

B8



**APPENDIX C**

# Analytical Laboratory Test Results



Your Project #: 19136074  
Site Location: BRADFORD BYPASS  
Your C.O.C. #: 827733-01-01

**Attention: Carter Comish**

WSP Canada Inc.  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2023/07/04**  
Report #: R7699610  
Version: 3 - Revision

**CERTIFICATE OF ANALYSIS – REVISED REPORT**

**BUREAU VERITAS JOB #: C1H2336**

**Received: 2021/06/22, 16:30**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2021/06/24	2021/06/24	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2021/06/25	2021/06/25	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl <sub>2</sub> EXTRACT	2	2021/06/24	2021/06/24	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2021/06/22	2021/06/25	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2021/06/24	2021/06/24	CAM SOP-00464	MOE E3013 m

**Remarks:**

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCCFP, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

\* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.



Your Project #: 19136074  
Site Location: BRADFORD BYPASS  
Your C.O.C. #: 827733-01-01

**Attention: Carter Comish**

WSP Canada Inc.  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2023/07/04**  
Report #: R7699610  
Version: 3 - Revision

**CERTIFICATE OF ANALYSIS – REVISED REPORT**

**BUREAU VERITAS JOB #: C1H2336**

**Received: 2021/06/22, 16:30**

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to:

Ankita Bhalla, Project Manager

Email: Ankita.Bhalla@bureauveritas.com

Phone# (905) 817-5700

=====

Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by Rodney Major, General Manager responsible for Ontario Environmental laboratory operations.



BUREAU  
VERITAS

Bureau Veritas Job #: C1H2336

Report Date: 2023/07/04

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

### RESULTS OF ANALYSES OF SOIL

<b>Bureau Veritas ID</b>		PXF840	PXF841		
<b>Sampling Date</b>		2021/04/19	2021/04/19		
<b>COC Number</b>		827733-01-01	827733-01-01		
	<b>UNITS</b>	<b>PDD-1 SS3</b>	<b>PDD-2 SS2</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>					
Resistivity	ohm-cm	7800	7600		7421780
<b>Inorganics</b>					
Soluble (20:1) Chloride (Cl-)	ug/g	<20	<20	20	7426573
Conductivity	umho/cm	128	131	2	7429034
Available (CaCl2) pH	pH	7.74	7.63		7426977
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	20	7426738
RDL = Reportable Detection Limit					
QC Batch = Quality Control Batch					



BUREAU  
VERITAS

Bureau Veritas Job #: C1H2336

Report Date: 2023/07/04

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

## TEST SUMMARY

**Bureau Veritas ID:** PXF840  
**Sample ID:** PDD-1 SS3  
**Matrix:** Soil

**Collected:** 2021/04/19  
**Shipped:**  
**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl <sub>2</sub> EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**Bureau Veritas ID:** PXF841  
**Sample ID:** PDD-2 SS2  
**Matrix:** Soil

**Collected:** 2021/04/19  
**Shipped:**  
**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl <sub>2</sub> EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan



### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	4.0°C
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Revised Report [2023/07/04]: Split report as per client request.

Revised Report [2023/01/03]: Split report as per client request.

**Results relate only to the items tested.**



**BUREAU  
VERITAS**

Bureau Veritas Job #: C1H2336

Report Date: 2023/07/04

## QUALITY ASSURANCE REPORT

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7426573	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2021/06/24	NC	70 - 130	105	70 - 130	<20	ug/g	3.5	35
7426738	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2021/06/24	112	70 - 130	102	70 - 130	<20	ug/g	NC	35
7426977	Available (CaCl <sub>2</sub> ) pH	2021/06/24			100	97 - 103			1.3	N/A
7429034	Conductivity	2021/06/25			100	90 - 110	<2	umho/cm	1.6	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference  $\leq 2 \times \text{RDL}$ ).





BUREAU  
VERITAS

Bureau Veritas Job #: C1H2336

Report Date: 2023/07/04

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

## VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

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Brad Newman, B.Sc., C.Chem., Scientific Service Specialist

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Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by {0}, {1} responsible for {2} {3} laboratory operations.



Bureau Veritas Laboratories  
6740 Campbell Road, Mississauga, Ontario Canada L5N 2L8 Tel: (905) 817-5700 Toll-free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

# CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:			REPORT TO:			PROJECT INFORMATION:			Laboratory Use Only:		
Company Name: #1326 Golder Associates Ltd			Company Name: <u>Golder</u>			Quotation #: B80683			BV Labs Job #:		
Attention: Accounts Payable			Attention: <u>Carter Comish</u>			P.O. #: <u>12855</u>			Bottle Order #:		
Address: 6925 Century Ave Suite 100			Address:			Project: <u>19131074</u>			COC #:		
Mississauga ON L5N 7K2			Tel: _____ Fax: _____			Project Name: <u>Bradford Bypass</u>			Project Manager:		
Tel: (905) 567-4444			Tel: _____ Fax: _____			Site #:			Ema Gitej		
Email: CanadaAccountsPayableInvoices@golder.com			Email: <u>Carter_Comish@golder.com</u>			Sampled By:			C8527733-01-01		
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY											
Regulation 153 (2011)				Other Regulations				Special Instructions			
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Medium/Fine				<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw							
<input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse				<input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw							
<input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC				<input type="checkbox"/> MISA Municipality _____							
<input type="checkbox"/> Table _____				<input type="checkbox"/> PWQO <input type="checkbox"/> Reg 406 Table _____							
<input type="checkbox"/> Other _____											
Include Criteria on Certificate of Analysis (Y/N)?											
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle):	Metals / Hg / Cr VI	Corrosivity pH short/CI, SO4, pH, EC/Resistivity	ANALYSIS REQUESTED (PLEASE BE SPECIFIC)			
1	B-2 SS3	21/05/12	AM	Soil			✓				
2	10-4 SS2	21/05/10					✓				
3	Y-1 SS3	21/05/12					✓				
4	10-2 SS3	21/04/13					✓				
5	Y-4 SS2	21/04/11					✓				
6	L-3 SS3	21/06/02					✓				
7	9-1 SS3	21/04/12					✓				
8	10-1 SS2	21/04/29					✓				
9	PDD-1 SS3	21/04/19					✓				
10	PDD-2 SS2	21/04/19					✓				
* RELINQUISHED BY: (Signature/Print)			Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)			Date: (YY/MM/DD)	Time	# jars used and not submitted	Laboratory Use Only
<u>Carter Comish</u>			21/06/22	4:00pm	<u>Carter Comish</u>			21/06/22	16:30		Time Sensitive
											Temperature (°C) on Recl:
											5/3/4
											Custody Seal
											Present
											Intact
											Yes
											No
* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.											
* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.											
** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.											
SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS											
White: BV Labs Yellow: Client											

Bureau Veritas Canada (2019) Inc.



## Page of

<b>INVOICE TO:</b>						<b>REPORT TO:</b>						<b>PROJECT INFORMATION:</b>						<b>Laboratory Use Only:</b>																	
Company Name: #1326 Golder Associates Ltd						Company Name: <u>Golder</u>						Quotation #: B80683						BV Labs Job #:						Bottle Order #:											
Attention: Accounts Payable						Attention: <u>Carter Comish</u>						P.O. #:												827733											
Address: 6925 Century Ave Suite 100						Address:						Project: <u>14136074</u>																							
Mississauga ON L5N 7K2												Project Name: <u>Broadford By-pass</u>												COC #:						Project Manager:					
Tel: (905) 567-4444 Fax: (905) 567-6561						Tel:						Site #:												Ema Gitej											
Email: CanadaAccountsPayableInvoices@golder.com						Email: <u>Carter_Comish@golder.com</u>						Sampled By:												C#827733-02-01											

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY										ANALYSIS REQUESTED (PLEASE BE SPECIFIC)										Turnaround Time (TAT) Required: Please provide advance notice for rush projects														
<b>Regulation 153 (2011)</b> <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Medium/Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC <input type="checkbox"/> Table _____					<b>Other Regulations</b> <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> MISA    Municipality _____ <input type="checkbox"/> PWQO <input type="checkbox"/> Reg 406 Table _____ <input type="checkbox"/> Other _____					<b>Special Instructions</b>  _____ _____ _____					Field Filtered (please circle): Metals / Hg / Cr-VI  Corrosivity plh short(Cl, SO4, pH, EC/Resistivity)										<b>Regular (Standard) TAT:</b> (will be applied if Rush TAT is not specified): Standard TAT ~ 5-7 Working days for most tests. Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.									
Include Criteria on Certificate of Analysis (Y/N)? _____										<b>Job Specific Rush TAT (if applies to entire submission)</b> Date Required: _____ Time Required: _____ Rush Confirmation Number: _____ (call lab for #)																								
	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix														# of Bottles	Comments														
1		10-3 SS3	21/04/20	AM	Soil																													
2		B-1 SS3	21/04/21																															
3																																		
4																																		
5																																		
6																																		
7																																		
8																																		
9																																		
10																																		

* RELINQUISHED BY: (Signature/Print) <u>[Signature]</u> Carter Comish	Date: (YY/MM/DD) 21/06/22	Time 4:00pm	RECEIVED BY: (Signature/Print) <u>[Signature]</u>	Date: (YY/MM/DD) 06/11/22	Time 16:00	# jars used and not submitted	Laboratory Use Only				
							Time Sensitive	Temperature (°C) on Recl 5/3/4	Custody Seal Present	Yes	No
								Intact			

\* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.  
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SAMPLES MUST BE KEPT COOL (< 10°C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS

White: BV Labs
Yellow: Client

