FINAL REPORT

Preliminary Foundation Investigation and Design Report

Bathurst Street Overpass
Highway 400 to Highway 404 Link (Bradford Bypass)
Simcoe County and York Region
MTO Assignment No. 2019-E-0048

Submitted to:

AECOM Canada Ltd.

300 Water Street
Whitby, ON L1N 9J2

Submitted by:

WSP Golder

6925 Century Avenue, Suite 600, Mississauga, Ontario, L5N 7K2

+1 905 567 4444

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HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
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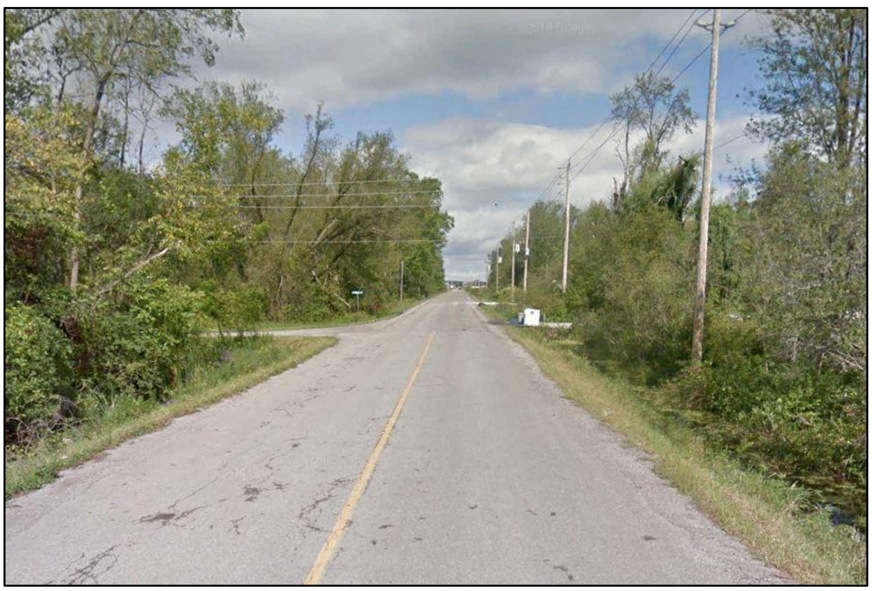
1.0 INTRODUCTION

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 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 overpass (twin single-span structures) to carry the proposed new highway westbound lanes (WBL) and eastbound lanes (EBL) over Bathurst Street at the location shown on the Key Plan in Drawing 1.

2.0 SITE DESCRIPTION

The proposed single-span twin bridges will cross Bathurst Street just south of Hochreiter Road, which is located within the Region of York, between the west and east branches of Holland River. The site is generally bounded by a mix of forested and agricultural areas. There is a private marina located east of the site (on the west side of Holland River) and just north of the proposed new highway alignment. Heavy tree cover is present within the footprint of the overpass structures on both sides of Bathurst Street (see Photograph 1) and standing water was observed within and beyond the existing road ditches. The existing Bathurst Street is an undivided arterial road with two lanes of traffic (one lane in each the north and south direction) having an existing road surface elevation of about 220 m.



Photograph 1 – Looking north on Bathurst Street, towards the intersection of Bathurst Street and Hochreiter Road

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between April 21 and April 27, 2021, and between May 26 and June 2, 2021, during which time two boreholes (designated B-1 and B-2) were advanced at the locations shown on Drawing 1. Borehole B-1 was advanced off the roadway platform on the east side of Bathurst Street near the footprint of the proposed westbound bridge, whereas Borehole B-2 was advanced through the existing roadway platform of Bathurst Street near the footprint of the eastbound bridge.

The boreholes were advanced using 210 millimetre (mm) outside diameter (O.D.) hollow-stem augers generally set 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 D50 track-mounted drill 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 used for the drilling operation was brought to site in totes (portable plastic tanks) by the drilling subcontractor.

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)[1]. 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. In situ field vane shear tests were carried out in Borehole B-1 using an MTO 'N'-vane in the cohesive soils, where feasible, to assess peak and remoulded undrained shear strengths in general accordance with ASTM D2573[2].

Where encountered, 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 Borehole B-2 to allow monitoring of the groundwater level. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 3.0 m long slotted screen within a filter sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with bentonite pellets. The monitoring well was capped with a casing installed flush with the road surface.

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 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. All laboratory tests were carried out in general accordance with MTO and / or ASTM Standards, as applicable.

Two soil samples, one from each borehole, were submitted to a specialist analytical laboratory (Bureau Veritas Laboratories of 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 MTM NAD 83 (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.

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[1] ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

[2] ASTM D2573 – Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

Borehole No.	MTM NAD83 (Geographic) Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
B-1	4,888,314 (44.134818)	302,597 (-79.527534)	218.9	50.9
B-2	4,888,256 (44.134295)	302,587 (-79.527657)	219.5	46.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of the Bradford Bypass is located in an area defined as the Simcoe Lowlands physiographic region, as delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984).

The Simcoe Lowlands physiographic region covers the central portion of the County of Simcoe and northern portion of York Region. 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.

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 and 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 and shear strengths from the field vanes) as presented on the borehole records and in Section 4 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 and on the stratigraphic profile on Drawing 1 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 advanced near the proposed Bathurst Street overpass consist of surficial layers of pavement structure and fill materials underlain by an upper cohesive deposit of clayey silt-silt to sandy clayey silt-silt with a variable consistency ranging from firm to hard. The clayey silt-silt to sandy clayey silt-silt is interlayered with non-cohesive deposits of generally compact silty sand to sandy silt, in turn, underlain by an extensive lower cohesive deposit of generally very stiff clayey silt to silty clay.

Detailed descriptions of the major layers encountered in the boreholes are provided in the following sections.

4.2.1 Asphalt

A 150 mm thick layer of asphalt was present at the road surface of Bathurst Street in Borehole B-2.

4.2.2 Gravelly Sand Fill

A 0.6 m thick layer of gravelly sand fill was encountered below the asphalt in Borehole B-2. The base of the fill layer extended to Elevation 218.7 m.

A single SPT 'N'-value obtained in the gravelly sand fill yielded 65 blows per 0.3 m of penetration, indicating a very dense state of compactness.

4.2.3 Silty Sand Fill

A 0.7 m thick layer of silty sand fill containing trace organics was encountered in Borehole B-1 at ground surface. This borehole was drilled outside of the existing Bathurst Street pavement structure. The base of the fill layer extended to Elevation 218.2 m.

A single SPT 'N'-value obtained in the silty sand fill yielded 3 blows per 0.3 m of penetration, indicating a very loose state of compactness.

4.2.4 Clayey Silt-Silt to Sandy Clayey Silt-Silt (Upper Cohesive Deposit)

An upper cohesive deposit of clayey silt-silt to sandy clayey silt-silt was encountered underlying the silty sand fill in Borehole B-1 and underlying the gravelly sand fill in Borehole B-2. The deposit was encountered at depths of 0.7 m to 0.8 m below ground surface (Elevations 218.7 m to 218.2 m) and was approximately 22.4 m to 22.5 m thick, extending to a depth of 23.2 m (Elevations 196.3 m to 195.7 m) in Boreholes B-1 and B-2. The deposit contained non-cohesive interlayers comprised of silty sand to sandy silt, which are described in subsection 4.2.5.

The SPT 'N'-values measured in the clayey silt-silt to sandy clayey silt-silt range from 6 to 52 blows per 0.3 m of penetration; the values typically measured about 10 blows up to approximately 23 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency. A firm zone, with SPT "N"-values of 6 and 7 blows per 0.3 m of penetration, was encountered within the upper 3 m of the deposit in both boreholes, and a harder zone, with SPT "N"-values of 42 and 52 blows per 0.3 m of penetration, was encountered between depths of 18.3 m and 22.0 m (Elevations 201.2 m and 197.5 m) in Borehole B-2.

Grain size distribution testing was carried out on six samples of the clayey silt-silt to sandy clayey silt-silt and the results are shown on Figure B1 in Appendix B.

Atterberg limits testing was carried out on seven samples of the clayey silt-silt to sandy clayey silt-silt and the samples had liquid limits ranging between 16% and 20%, plastic limits ranging between 11% and 15%, and plasticity indices ranging between 4% and 7%. These results, which are plotted on a plasticity chart on Figure B2, indicate that the deposit consists of clayey silt-silt of low plasticity.

The natural water content measured on selected samples of the clayey silt-silt to sandy clayey silt-silt ranges between about 15% and 23% and generally around 20% on average, near or slightly above the liquid limit for the material.

4.2.5 Silty Sand to Sandy Silt Interlayers

The upper cohesive deposit of clayey silt-silt to sandy clayey silt-silt deposit contained non-cohesive interlayers comprised of silty sand to sandy silt. Two major layers were encountered at a depth of about 2.1 to 2.3 m and 11.7 to 13.3 m in both boreholes. The layers were approximately 3.0 m to 5.7 m thick, extending to depths of 16.3 m to 17.1 m below ground surface (Elevations 203.2 m to 201.8 m) in Boreholes B-1 and B-2.

The SPT 'N'-values measured in the silty sand to sandy silt interlayers range from 4 to 38 blows per 0.3 m of penetration; the values typically measured about 13 blows up to 30 blows per 0.3 m of penetration, indicating a compact state of compactness. A loose zone, with SPT "N"-values of 4 to 8 blows per 0.3 m of penetration, was encountered within the upper 4 m below ground surface.

Grain size distribution testing was carried out on seven samples of the silty sand to sandy silt interlayers and the results are shown on Figure B3 in Appendix B.

Atterberg limits testing was carried out on the fines portion of three samples of the silty sand to sandy silt interlayers and one sample indicates that the material is non-plastic, and the other two samples had liquid limits of 14% and 17%, plastic limits of 11% and 16%, and corresponding plasticity indices of 1% and 3%. These results, which are plotted on a plasticity chart on Figure B4, indicate that the fines portion of the silty sand to sandy silt interlayers are non-plastic to slightly plastic.

The natural water content measured on selected samples of the silty sand to sandy silt ranges between about 16% and 20%.

4.2.6 Clayey Silt to Silty Clay (Lower Cohesive Deposit)

An extensive lower cohesive deposit of clayey silt to silty clay was encountered underlying the upper cohesive deposit of clayey silt-silt to sandy clayey silt-silt in Boreholes B-1 and B-2. The deposit was encountered at a depth of 23.2 m (Elevations 195.7 m and 196.3 m) in Boreholes B-1 and B-2 which were terminated within the deposit at depths of 50.9 m and 46.3 m (Elevations 168.0 m and 173.1 m) respectively.

The SPT 'N'-values measured in the clayey silt to silty clay range from 6 to 17 blows per 0.3 m of penetration. Six in situ field vane shear tests were carried out within the clayey silt to silty clay in Borehole B-1 between depths of 25.6 m and 40.8 m (Elevations 192.7 m to 178.1 m). Two tests yielded intact undrained shear strengths of 72 kPa and 86 kPa and remoulded undrained shear strengths of 24 kPa and 48 kPa and four tests yielded intact undrained shear strengths in excess of 96 kPa; remoulded tests were not conducted where the intact undrained shear strength exceeded the capacity of the testing apparatus (i.e. 96 kPa). The results of the in situ field vane shear tests carried out in Borehole B-1 indicate a stiff consistency, and the results of the SPT 'N'-values carried out in both boreholes suggest a firm to very stiff consistency. Based on the ratio of the intact to remoulded undrained shear strengths (1.7 to 3.0), the clayey silt to silty clay deposit is classified as having low to medium sensitivity (CFEM, 2006).

Grain size distribution testing was carried out on two samples of the clayey silt to silty clay and the results are shown on Figure B5 in Appendix B.

Atterberg limits testing was carried out on five samples of the clayey silt to silty clay and the samples had liquid limits ranging between 24% and 41%, plastic limits ranging between 14% and 17%, and plasticity indices ranging between 10% and 24%. These results, which are plotted on a plasticity chart on Figure B6, indicate that the deposit consists of clayey silt to silty clay of low to intermediate plasticity.

The natural water content measured on selected samples of the clayey silt to silty clay ranges between about 23% and 29%, generally near or slightly below the liquid limit for the material.

4.3 Groundwater Conditions

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

A standpipe piezometer was installed in Borehole B-2 to allow monitoring of the groundwater level at this site. The groundwater levels recorded during drilling and in the piezometer are shown on the borehole records in Appendix A and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
B-1	218.9	3.1	215.8	April 21, 2021	Open borehole / inside hollow stem auger
B-2	219.5	1.4	218.1	February 15, 2023	Piezometer

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. Observations of open water within the ditches and surrounding low lying areas during the investigation suggests the groundwater level is near the native ground surface.

4.4 Analytical Testing of Soil

Two soil samples (one from each borehole) were submitted for 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)
B-1, SA 3	7.88	8700	115	<20[1]	<20[1]
B-2, SA 3	7.66	1500	651	270	<20[1]

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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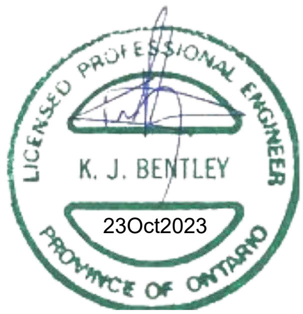


Mark Henderson, P.Eng.
Geotechnical Engineer III

Kevin Bentley, M.E.Sc., P.Eng.
Designated MTO Foundations Contact

MH/KJB/al

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
BATHURST STREET OVERPASS
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
SIMCOE COUNTY AND YORK REGION
MTO ASSIGNMENT NO. 2019-E-0048

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for planning and preliminary design of the Bradford Bypass and Bathurst Street overpass structures. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation.

The Preliminary Foundation Design Report (Part B of this report) including the discussion and recommendations are intended for the use of MTO and their designers for planning and preliminary design and shall not be used or relied upon for any other purpose or by any other parties, including the 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 construction must make their own interpretation of the factual information provided (and supplement as necessary for detail design) as such interpretation may affect detail design, equipment selection, proposed construction methods, scheduling and the like.

6.2 Project Understanding

Based on the General Arrangement (GA) drawing provided by AECOM (preliminary draft dated March 2023), twin bridge structures are proposed to carry the Bradford Bypass eastbound and westbound lanes over Bathurst Street. Each single span bridge structure will accommodate two lanes of traffic in the eastbound and westbound direction (four lanes total) for the interim configuration, with an ultimate configuration to accommodate four lanes in each direction (eight lanes total) requiring future bridge widenings. The structural classification of the bridge(s) is defined as “major-route” by the structural designer at this preliminary design stage.

Based on the GA drawing, the existing road surface of Bathurst Street between the proposed westbound and eastbound bridges is at about Elevation 220 m, and the existing ground surface adjacent to Bathurst Street along the proposed highway centreline ranges from about Elevation 218.5 m to 219.5 m. The proposed Bradford Bypass highway grade is shown to be at about Elevation 229 m (i.e., approach embankment heights on the order of about 10 m above the existing ground surface). The interim design configuration will consist of an approximately 14 m wide bridge (two 3.8 m wide lanes, 2.5 m and 3.0 m wide shoulders, and two 0.5 m wide concrete barriers) for both the eastbound and westbound directions, and the ultimate configuration will consist of future widening of the bridges toward the highway centreline (essentially join the bridges together less a 1 m gap) with an additional width of about 10 m being added to each bridge (to accommodate two 3.8 m wide lanes, a 3.6 m shoulder, a 0.5 m wide concrete barrier, and HOV lane buffer zone).

6.3 General Foundation 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 Bathurst

Street 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 locations of the abutment foundations were not confirmed and based on this together with access considerations, the boreholes are not located directly within the 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, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of CHBDC (2019) have been used for at this stage of preliminary design. During detail design, additional investigation and testing may be performed to increase the level of confidence and potentially 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 detail 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, $\overline{N}_{60}$ and average undrained shear strength, s_u within the upper 30 m of the overburden below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019), in the absence of any geophysical testing. Geophysics testing, such as Multi-Channel Analysis of Surface Waves (MASW) or Vertical Seismic Profiling (VSP), may provide a more favourable Site Class designation, and such testing can be considered during detail design.

The CHBDC (2019) states that the seismic hazard values associated with the design earthquakes should be those established for the *National Building Code of Canada* (NBCC) by the Geological Survey of Canada (GSC). The 2015 seismic hazard maps (referred to as the 5th generation seismic hazard maps) have been used for 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 D were obtained for the bridge site using the NBCC website (*earthquakescanada.nrcan.gc.ca*) and are summarized below.

Seismic Hazard Values for Site Class D	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475year return period)
PGA (g)	0.039	0.059	0.095
PGV (m/s)	0.040	0.062	0.098
$S_a(0.2)$ (g)	0.064	0.098	0.151
$S_a(0.5)$ (g)	0.054	0.079	0.119
$S_a(1.0)$ (g)	0.033	0.050	0.074
$S_a(2.0)$ (g)	0.016	0.025	0.039
$S_a(5.0)$ (g)	0.003	0.006	0.009
$S_a(10.0)$ (g)	0.001	0.003	0.004

The values provided above are for the reference ground condition Site Class D and must be checked and modified (as appropriate) to the site-specific seismic site classification to be confirmed during 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 detail 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 slide often accompany liquefaction along rivers and other shorelines.

In general, the soils at these bridge sites consist of interlayered stiff to very stiff clayey silt to clayey silt-silt soils and generally compact silty sand to sandy silt soils. Considering the compactness, consistency and plasticity index of the soils and the 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). The potential for liquefaction (especially for the looser near surface cohesionless soils encountered in the boreholes) will need to be reassessed when more site-specific foundation soil information is available during detail design.

6.4 Foundation Types

Based on the proposed single-span twin structure configuration and subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments. 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 "perched" on a compacted granular pad founded on the compact silty sand to sandy silt layer (at a depth of about 4 m below ground surface and below the existing fill, firm clayey soils and loose sandy silt to silty sand zones) are feasible for the abutment foundations; however, some time dependent settlement of the underlying cohesive soil will occur under the approximately 10 m high approach embankment loading, and hence this option is not considered the preferred alternative from a geotechnical/foundations perspective. Deep foundations consisting of driven steel H- or tube piles with the pile cap perched within the approach embankments is preferred, 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.

6.4.1 Shallow Foundations

Strip or spread footings founded on the native compact silty sand to sandy silt (at or below the approximate elevations identified below) are considered feasible for support of the structure abutments. The feasibility of using shallow foundations will need to be reassessed when actual structure loads, footing sizes and actual groundwater conditions are known.

Based on the boreholes, subexcavation up to about 4 m below existing ground surface (and 3 m below anticipated groundwater level) to remove loose soils and reach the competent founding strata are anticipated to be required. Consideration could be given to subexcavating the unsuitable soils and placing engineered fill such that spread footings could be "perched" within approach embankments to increase geotechnical resistance values.

The following geotechnical resistances may be used for preliminary design, assuming a 3 m to 5 m wide footing:

Anticipated Founding Stratum at Bridge Locations	Founding Elevation[1]	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance[2]
Compact silty sand to sandy silt	215 m	3 m	500 kPa	175 kPa
		5 m	600 kPa	125 kPa
Granular pad on compact silty sand to sandy silt	Min. 3 m of granular fill above El. 215 m	3 m	600 kPa	225 kPa
	Min. 5 m of granular fill above El. 215 m	5 m	750 kPa	175 kPa

Notes:
1. Subexcavation to about 4 m below ground surface (and below groundwater) is required to remove unsuitable soils to a competent founding stratum.
2. For 25 mm of settlement independent of any settlements induced by surrounding grade changes / embankment loading. Higher settlements may occur at abutment areas associated with the embankment loading.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width, founding elevation, and thickness of granular pad (as applicable) and as such, the geotechnical resistances must be reviewed and revised if the footing width varies from that specified above or if the founding soils differ from that given in the previous section.

Resistance to lateral loads / sliding resistance between the new concrete footing and the subgrade should be calculated in accordance with Section 6.10.4 of *CHBDC* (2019), applying the appropriate consequence and degree of site understanding factors as applicable during detailed design. For preliminary design, the effective angle of friction and unfactored coefficient of friction, tan δ, between the cast-in-place concrete footings and the compact silty sand to sandy silt may be taken as 32° (with an effective cohesion of zero) and 0.62, respectively. The effective angle of friction and unfactored coefficient of friction, tan δ, between the cast-in-place concrete footings and a Granular 'A' pad may be taken as 33° and 0.65, respectively.

6.5 Deep Foundations

6.5.1 Steel H-Pile or Tube Foundations

Driven steel H-piles founded within the lower clayey silt to silty clay deposit are considered feasible for the support of the new abutments. Closed ended steel tube piles 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.

Consideration should be given to "perched" pile caps within the embankment fill to reduce subexcavation and dewatering requirements, although settlement due to the embankment will need to be assessed and mitigated during detail design. The factored ultimate and serviceability geotechnical axial resistances for a range of driven steel H- and tube piles for two different pile lengths (with corresponding pile tip elevations) for the bridges is provided below for preliminary design purposes.

The following geotechnical resistances may be used for preliminary design:

Foundation Element	Pile Type	Approximate Pile Length[1]	Estimated Pile Tip Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
Eastbound and Westbound Bridge Abutments	HP 310x110	30 m	187.5 m	800	>800
		40 m	177.5 m	1,100	>1,100
	HP 360x108	30 m	187.5 m	925	>925
		40 m	177.5 m	1,250	>1250
	324 mm dia. tube pile (min. 9.5 mm thick)	30 m	187.5 m	700	>700
		40 m	177.5 m	950	>950
	406 mm dia. tube pile (min. 9.5 mm thick)	30 m	187.5 m	900	>900
		40 m	177.5 m	1,200	>1,200

Notes:
1. Measured from estimated underside of pile cap at about El. 217.5 m (i.e. approximately frost depth below existing ground surface).
2. Resistance values assume single pile and do not take into account pile group efficiency.
3. Consideration should be given to using a heavier H-pile section (310x132 or 360x132) or thicker tube pile (13 mm thickness) if piles are to be driven longer than 30 m.

The estimated factored ultimate geotechnical resistance is calculated on both shaft and tip resistances, but predominantly shaft and assume piles have had sufficient time to "set-up" and allow pore pressures to dissipate after initial driving in order to achieve the design geotechnical resistances. It is noted that some “relaxation” may also occur in the compact to dense silts and sands. The time required for piles to “set up” or “relax” depends on many factors and is difficult to predict. As per Section 18.2.7.5 of CFEM (2006), it is advisable to delay testing for at least two weeks after driving.

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, and re-tapping of piles performed no sooner than 2 weeks after initial driving.

In order to optimize the design, schedule 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;
- 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”.

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.

6.5.2 Drilled Shafts (Caissons)

Caissons are considered marginally feasible for supporting the bridge structure abutments. Long friction caissons (>30 m) are likely required as no competent end-bearing strata and no “100-blow” soil was encountered within a 50 m depth.

The following axial geotechnical resistances may be used for preliminary design of the caissons:

Caisson Diameter	Approximate Caisson Length[1]	Estimated Caisson Base Elevation [1]	Factored Ultimate Geotechnical Resistance[2]	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement[2]
0.9 m	30 m	187.5 m	2,000 kN	>2,000 kN
1.5 m	30 m	187.5 m	3,500 kN	>3,500 kN

Notes:

1. Measured from estimated underside of pile cap at about El. 217.5 m (i.e. approximately frost depth below existing ground surface).
2. Resistance values assume single caisson and do not take into account caisson group efficiency.

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 is expected to be required (at least in the upper zone) to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils and cohesive soils containing silt and sand interlayers. 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, but higher material cost. 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 and preparation of the future contact documents.

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 open hole / liner to prevent basal heave and disturbance of water-bearing cohesionless layers/interlayers 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 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 allowed to be used) will have an impact on the design geotechnical resistances and this will need to be considered during detail design and future contract documents.

In order to optimize the design, 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 the degree of 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 and modified to address the requirements for supply and installation of drilled shafts (caissons) including the use of temporary or permanent liners/casings and slurry type, 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 non-standard 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 where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 2002 as referenced in CHBDC, 2006).

For non-cohesive soils:

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

Where n_h is the constant of subgrade reaction (kPa/m);
z is the depth (m); and
B is the pile / caisson diameter or width (m).

For cohesive soils:

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

Where s_u is the undrained shear strength of the soil (kPa); and
B is the pile / caisson diameter or width (m).

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.

The following values of n_h and S_u may be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) for the structural analysis of the piles or drilled shafts at this site, as summarized below using the interpreted stratigraphic conditions from the boreholes. The range in the values reflect the variability of the subsurface conditions, the soil properties and groundwater level, and the approximate nature of the linear-elastic subgrade reaction analysis. In developing these recommendations, the design groundwater level has been taken at approximately Elevation 217 m.

Foundation Element	Soil Unit	Above or Below GWL	Elevation	n_h (kPa/m)	S_u (kPa)
Eastbound Bridge Abutments (Borehole B-1)	Existing Fill / New Granular Fill (New Granular 'A' or 'B' Type II)	Above	Above 218.2	5,000 – 7,000 / 40,000 – 50,000	-
	Loose sand within CSP (if applicable)		Above GWL	1,500 – 2,500	-
	Sandy Clayey Silt-Silt (firm to very stiff)		218.2 to 216.6	-	50 - 100
	Silty Sand (loose to compact)	Below	216.6 to 211.7	7,000 – 15,000	-
	Clayey Silt-Silt to Silt (stiff to very stiff)		211.7 to 207.1	-	75 - 100
	Silty Sand (compact to dense)		207.1 to 201.8	15,000 – 25,000	-
	Clayey Silt-Silt (very stiff)		201.8 to 195.7	-	100 - 150
	Clayey Silt to Silty Clay (firm to very stiff)		195.7 to 168.0	-	60 - 100
Westbound Bridge Abutments (Borehole B-2)	Existing Fill / New Granular Fill (New Granular 'A' or 'B" Type II)	Above	Above 218.7	7,000 – 40,000 / 40,000 – 50,000	-
	Loose sand within CSP (if applicable)		Above GWL	1,500 – 2,500	-
	Sandy Clayey Silt-Silt (firm to stiff)		218.7 to 217.2	-	50 - 100
	Sandy Silt (loose to dense)	Below	217.2 to 211.5	7,000 – 15,000	-
	Clayey Silt-Silt (stiff to very stiff)	Below	211.5 to 206.2	-	75 - 100
	Sandy Silt (compact)		206.2 to 203.2	10,000 – 20,000	-
	Clayey Silt-Silt (very stiff to hard)		203.2 to 196.3	-	100 - 200
	Clayey Silt (stiff to very stiff)		196.3 to 173.1	-	60 - 100

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile / caisson efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2019).

6.5.4 Downdrag Loads on Piles / Caissons

Based on the preliminary design, the east and west approach embankments for both bridges are to be 10 m high with total settlements in the foundation soils estimated to be greater than 200 mm due to the embankment loading (see Section 6.7.2). Accordingly, depending on the relative timing of embankment fill placement at and near the abutments, and pile installation, the embankment fills could induce significant downdrag loads that will need to be accounted for in the assessment of the structural loading of the piles.

The magnitude of the downdrag loads is a function of the size of the loaded area which includes relative downward movement of the soil mass around the piles and the amount of settlement remaining after the piles are installed. The depth of influence, or depth over which negative skin friction develops, is dependent on both the size of the loaded area and weight of the applied load. If piles can be installed after the majority of settlement has occurred, downdrag loads will be reduced. Downdrag loads can also be mitigated by:

- preloading prior to pile installation;
- use of lightweight fill in the abutment area that could consist of expanded polystyrene (EPS blocks), tire derived aggregate (TDA), cellular concrete, water-cooled blast furnace slag or a combination of these;
- use of friction reducers such as bitumen coating or installation of isolators which prevent direct contact between the pile and soil such as pile sleeves or bentonite slurry; and/or
- use of heavier pile sections.

The downdrag loads must be assessed during detail design and mitigated accordingly.

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

For preliminary design, it is assumed that the approach embankment side slopes will be inclined at 2 horizontal to 1 vertical (2H:1V) with an overall height of about 10 m of new fill over the existing ground surface. A 2 m wide mid-height bench was modelled along the embankment slopes as required for embankment heights greater than 8 m in height as per OPSD 202.010 (*Slope Flattening*).

For preliminary design, it is assumed that prior to construction of the new approach embankments, all topsoil and peat/organic soil (although not encountered during the current investigation but anticipated based on the forested area within the footprint of the embankments) and existing fill materials be stripped from the footprint of the new embankments and replaced with suitable granular fill (see Section 6.10.1).

Global stability and settlement analyses were carried out for the proposed preliminary east and west approach embankment configurations using the current borehole information at the site, supplemented and tempered by engineering judgement based on the information obtained from the explorations (which included boreholes and seismic cone penetration tests) at the adjacent Holland River (West) and Holland River East sites which bound the Bathurst Street site on to the west and east side respectively.

6.7.1 Global Stability

Limit equilibrium global stability analyses were carried out for the proposed approach embankments using the commercially available program Slide2 (version 9.017), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety of numerous potential circular failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety 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, Ψ, and the geotechnical resistance factor, ϕ_{gu} (i.e. $FoS = 1/(\Psi \cdot \phi_{gu})$.

Accordingly, given the limited geotechnical information at the site, minimum target Factors of Safety of 1.4 and 1.6 have been used for the preliminary design of the approach embankment slopes for the temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2019) and MERO (2020).

6.7.1.1 Parameter Selection

For the non-cohesive native soils encountered at the site, effective stress parameters were employed in the analysis assuming drained conditions, and the strength parameters were estimated from empirical correlations based on the in situ SPT "N"-values and tempered by engineering judgment based on the experience gained from the adjacent sites. For the cohesive native soils encountered at the site, total stress parameters were employed for the short-term, undrained conditions. The total stress parameters (i.e., undrained shear strength, s_u) were estimated from correlations with the SPT 'N'-values and results of the field vane tests. Effective friction angles have also been estimated for these deposits for analysis of the Factor of Safety in the long-term, drained condition.

Summarized below are the simplified stratigraphy and the associated soil parameters employed for the stability analyses. A bulk unit weight of 21 kN/m^3 and effective friction angle of 36º was assumed for the new granular embankment fill.

Location	Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (º)	Undrained Shear Strength (kPa)
East Approach Embankment (Borehole B-1)	Sandy Clayey Silt-Silt (Firm to Very Stiff)	19	30	50 to 100
	Silty Sand (Loose to Compact)	20	33	--
	Clayey Silt-Silt (Stiff to Very Stiff)	19	31	75 to 100
	Silty Sand (Compact to Dense)	20	34	--
	Clayey Silt-Silt (Very Stiff)	19	30	100 to 150
	Clayey Silt to Silty Clay (Firm to Very Stiff)	19	28	60 to 100
West Approach Embankment (Borehole B-2)	Sandy Clayey Silt-Silt (Firm to Stiff)	19	30	50 to 100
	Sandy Silt (Loose to Dense)	20	33	--
	Clayey Silt-Silt (Stiff to Very Stiff)	19	31	75 to 100
	Sandy Silt (Compact)	20	34	--
	Clayey Silt-Silt (Very Stiff to Hard)	19	30	100 to 200
	Clayey Silt (Stiff to Very Stiff)	19	28	60 to 100

6.7.1.2 Results of Analysis

The stability analyses indicate that for the short-term (undrained) condition, the approach embankments at the abutments will have a global Factor of Safety of greater than 1.4, and for the long-term (permanent) conditions, the approach embankments at the abutments will have a global Factor of Safety of greater than 1.6. The results of the stability analyses are shown on Figures 1 and 2 following the text of this report.

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.

6.7.2 Settlement

To estimate the magnitude of settlement as a result of the proposed embankments, analyses were carried out near the abutment locations using the commercially available computer program Settle 3 (Version 5.012) from Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on Westergaard's (1938) solution. Based on the anticipated interim and future bridge configurations, the following settlement models were employed for planning and preliminary design:

- Single Embankment Model for Interim Configuration: assumes multistage construction with separate approach embankment (14 m wide consisting of 2H:1V side slopes on both sides) built on the east and west side of both bridges for the interim configuration. In the future, the embankments would be widened to the inside (which is not modelled and which would induce further settlement) consistent with the proposed bridge widening for the ultimate configuration; and
- Continuous Embankment Model Spanning Width of Ultimate Configuration: assumes single stage approach embankment construction with one continuous embankment (39 m wide and 2H:1V side slopes on exterior slopes only) extending the entire width of the ultimate configuration. In the future, the bridges would be widened to the inside where the approach embankment has already been constructed and foundation soils will essentially be preloaded to accommodate ultimate configuration.

The settlement analyses assume that all existing fill material, organics and any loosened/softened soils within the footprint of the embankments will be subexcavated and replaced with new granular fill prior to placement of the embankment material.

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.

6.7.2.1 Parameter Selection

The sources of total settlement within the depth of influence of the embankment are considered to include the following:

- Immediate settlement of the granular soils (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).

The immediate compression of the non-cohesive soil deposits were modelled based on the established correlations based on SPT "N"-values, as presented in Bowles (1984) and by Kulhawy and Mayne (1990), as well engineering judgement from experience with similar soils in this region of Ontario.

The consolidation settlement of the cohesive deposits was assessed using the results of the in situ field vane tests and correlations based on SPT "N"-values to estimate the stress history for the cohesive deposits. The preconsolidation pressure was estimated using the correlation proposed by Mesri (1975). The results of the laboratory index tests were used to assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Rendon-Herrero (1980), Bowles (1984), Sowers (1970), Wood and Wroth (1978), and Terzaghi and Peck (1967) in collaboration with the consolidation tests performed at the adjacent Holland River sites.

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was estimated using the results of the laboratory consolidation tests and the results of dissipation tests from seismic Cone Penetration Tests (sCPTs) from the nearby Holland River sites.

6.7.2.2 Results of Analyses

A summary of the estimated magnitude of total settlement for the approach embankments is presented in the table below, assuming the use of conventional granular fill for construction. The estimated settlements do not account for settlement of the embankment fill itself, which would need to be assessed during detail design.

Considering the full vertical extent (thickness) of the lower cohesive deposit was not confirmed from the exploration boreholes (that extended to greater than 50 m depth) and the depth of influence of the embankments extend well below the termination depth of the boreholes, the actual magnitude of settlement may vary considerably. Based on borehole information from the nearby Holland River sites, the lower cohesive deposit of clayey silt to silty clay is anticipated to be 30 m thick; however, a query of nearby well records from the MECP database suggest that the cohesive deposit could be 50 m thick. As a result, a range of estimated settlements is provided below for the two embankment geometry models and estimated range in thickness of the lower cohesive deposit. The actual thickness of the lower cohesive deposit should be confirmed during detail design.

Embankment Geometry Model	Estimated Thickness of Lower Cohesive Deposit (m)	Estimated Total Settlement over a 20-Year Period[1]	Estimated Time-Dependent Consolidation Settlement (mm) over a 20-Year Period
Single Embankment for Interim Configuration	30	150 – 170	120
	40	200 – 220	160
	50	230 – 250	190
Continuous Embankment Spanning Width of Ultimate Configuration	30	200 – 220	160
	40	250 – 280	220
	50	290 – 320	260

Notes:

1. The total settlement is defined as the sum of the immediate settlement due to elastic compression of the non-cohesive deposits as well as primary and secondary settlements due to time dependent consolidation of the cohesive deposits. The total elastic compression was estimated to be range from about 30 to 60 mm, and secondary consolidation settlements were estimated to be less than 5 mm to 10 mm for both embankment geometries.

Based on the estimated magnitude of settlement above for both embankment geometry models and the anticipated range in thickness of the lower cohesive deposit, settlement mitigation options will be required to meet the settlement performance criterion.

6.7.2.3 Mitigation Options

Several settlement mitigation options have been considered to meet the settlement performance criterion and a brief discussion on these alternatives is provided below. Other ground improvement measures such as full subexcavation and replacement, rammed aggregate piers, deep soil mixing, and dynamic compaction are not considered suitable or cost effective due to the composition, thickness and depth of the compressible deposits and such options are not discussed further for preliminary assessment.

- **Preloading (with or without Surcharge):** In cases where the subsurface conditions are sufficiently permeable, such as the upper cohesive deposit containing silt and sand interlayers at this site, preloading to allow excess pore pressures to dissipate to induce settlement in a reasonable period of time (less than 6 months) is ideal. Given the extensive thickness of the less permeable lower cohesive deposit, preloading is anticipated to take a minimum of 3 to 6 years (depending on the depth to, and thickness of, the lower cohesive deposit) to reduce settlements to tolerable levels. As a result, surcharging is anticipated to reduce preload times to 6 months to 2.5 years (depending on the actual thickness of the lower cohesive deposit, embankment geometry and thickness of the surcharge) in advance of bridge construction, which is considered feasible.

- **Preloading (with or without Surcharge) with Wick Drains**: Prefabricated vertical drains could be installed prior to construction of the embankments to relieve pore pressures and accelerate settlements. Prefabricated vertical drains would typically be installed on a 2 m triangular grid pattern and penetrate to the bottom of the cohesive deposits. Following construction of the embankments, a surcharge could be placed to further accelerate consolidation. Given that the top of the lower cohesive deposit was encountered at a depth of 25 m, and is likely 30 m to 50 m thick, conventional wick drain installation and effectiveness may not be practical at this site but should be considered during detail design to accelerate the construction schedule.

- **Lightweight Slag or Cellular Concrete**: Various lightweight fill materials are available, from lightweight slag with a unit weight of approximately 14 kN/m^3, to cellular concrete with a unit weight between 4 and 7 kN/m^3. However, for the volume of fill required for the new embankments, a similar preloading period to using conventional fill materials may still be required to achieve the settlement performance criterion. Floatation concerns within the floodplain will also need to be considered.

- **Lightweight Expanded Polystyrene:** The use of expanded polystyrene (EPS) is another alternative that can be considered to significantly reduce the magnitude of consolidation settlement. Where required, EPS can be used to achieve the settlement performance criterion without preloading and therefore, will reduce the length of time for construction. However, the disadvantage of using EPS is the high cost relative to conventional fill or other lightweight fill options. Floatation concerns within the floodplain will also need to be considered.

Based on the above considerations, preloading with surcharging is considered the technically preferred alternative to mitigate long-term post-construction settlement at this site. Consideration should be given to an advanced contract option at this site to allow sufficient preload time with settlement monitoring in advance of bridge construction.

6.7.2.4 Preloading with Surcharge

Based on the estimated coefficient of consolidation, c_v = 1.34 x 10^{-2}(cm^2/s), for the majority of the compressible cohesive deposits, it is estimated that the following preload periods will be required for the approach embankments, assuming a 5 m surcharge (i.e. 15 m high embankment) and two-way drainage of the lower cohesive deposit.

Embankment Geometry	Thickness of Lower Cohesive Deposit (m)[1]	Estimated Preload Period	
		days	months
Single Embankment for Interim Configuration	30	200 - 300	6 - 10
	40	500 - 600	16 - 20
	50	850 - 950	28 - 31
Continuous Embankment Spanning Width of Ultimate Configuration	30	250 – 350	8 - 12
	40	800 – 900	26 - 30
	50	1,400 – 1,600	46 - 54

Notes: [1] To be confirmed during detail design.

Since the full vertical extents of the lower cohesive deposit has not been confirmed (i.e., the thickness of the drainage path is unknown), and the c_v estimate is based on limited data, the actual time rates of consolidation may vary considerably from the estimates provided above. Additional investigation to delineate the vertical extents and characteristics of the lower cohesive deposit to refine the time-rates of consolidation estimates provided above will need to be carried out during detail design to ascertain whether or not preload and surcharging (possibly with wick drains), are feasible settlement mitigation options.

The design-builder / contractor will need to monitor actual settlements upon completion of the preload period so that the embankment is constructed to the design geometric requirements. Considering the size of the embankment (to accommodate twin structures) and length of the preload period, if this alternative is to be adopted, the magnitude and time-rate of settlement during and after construction of the preload embankment should be assessed by a monitoring program consisting of settlement plates (SPs) and vibrating wire piezometers (VWPs) to confirm the end of the preload period.

Based on the estimates of settlement and preload (with surcharge) times discussed above for the different embankment geometries considered for the initial stage of construction, it is recommended that the approach embankment geometry for the ultimate bridge(s) configuration be constructed at the initial stage of construction to induce the majority of all (interim and future configuration) anticipated settlement at the approach embankments. The advantages of constructing the ultimate configuration of the approach embankments as early as possible includes reduced future construction staging and more importantly reduced impacts of differential settlement on the interim approach embankment configuration (including downdrag on piles and differential movement of the highway and any settlement sensitive structures) as a result of the future adjacent embankment fill loading. The disadvantages are increased initial preload times and higher initial costs. As mentioned previously, 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 spread footings (excess settlement in addition to the f-SLS geotechnical resistance) and/or deep foundations (i.e. associated downdrag forces).

6.7.3 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 are summarized in Section 4.4 and discussed below, and the analytical laboratory test reports are 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

Based on the soil analytical test results, the measured pH ranges between 7.7 and 7.9. 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. 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.

The resistivity measured in the tested soil samples (values of 1,500 and 8,700 ohm-cm) indicates that the soil corrosiveness is variable, ranging from severe (R < 2,000 ohm-cm) to very low (6000 ohm-cm < R < 10,000 ohm-cm) as per Table 3.2 (MTO, 2014). Further, given that the foundations are located adjacent to and below the future highway and will be exposed to de-icing salt, consideration should be given to corrosion protection and 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 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) are satisfied.

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, peat/organic soil, and existing surficial fill materials or loosened/softened soils be stripped from the embankment footprint and be replaced with OPSS Select Subgrade Material (SSM), Granular A or Granular B. If stripping extends below the groundwater, Granular A or Granular B Type II is preferred to reduce or eliminate dewatering efforts provided the temporary excavation remains stable. Based on the boreholes, up to about 1 m below ground surface is anticipated provided settlement mitigation options are adopted as discussed in Section 6.7.2.

Engineered fill for construction of the new approach embankments should consist of OPSS.PROV 1010 (*Aggregates*) granular materials (i.e. SSM, Granular A or Granular B). 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 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, vegetative cover should be established as per OPSS.PROV 803. Depending on the time of year, temporary erosion control measures such as mulch, bonded fibre matrix (BFM), fiber reinforced matrix (FRM), or erosion control blankets (ECB), should be applied as per OPSS.PROV 804 (*Temporary Erosion Control*) as soon as possible after construction of the embankments.

6.9.2 Temporary Excavations

In general, temporary excavations extending up to about 4 m below ground surface are required for shallow foundations (including subexcavation and replacement with a granular pad for a “perched” spread footing option). Temporary excavations can be reduced to about 1.5 m deep below ground surface (i.e. frost depth) or eliminated for pile and caisson caps “perched” within the approach embankments, as applicable.

All temporary excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. As per OHSA, the existing fill materials, loose to compact silty sand to sandy silt deposits, and the firm to stiff clayey silt-silt deposits above the groundwater level are classified as Type 3 soils. Below the groundwater level, the silty sand to sandy silt soils and sandy clayey silt-silt deposit should be classified as Type 4 soils.

As such, temporary excavations (i.e., those that are open for a relatively short time period where personnel are required to enter) within Type 4 soils should be made with side slopes no steeper than 3H:1V, while those within Type 3 soils 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 Performance Level 2 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

The groundwater level measured in the piezometer installed in Borehole B-2 was at about Elevation 218 m (about 1.4 m below ground surface) when checked in February 2023. The groundwater level is anticipated to fluctuate seasonally and will likely be higher in the spring and fall months.

The excavations for shallow foundations (if applicable) are anticipated to extend about 4 m below existing ground surface (about Elevation 215 m) and will be about 3 m below the measured groundwater level. As such, it is expected that advanced dewatering using well points prior to excavation will be required for the foundation excavations. For stripping operations, the groundwater level is expected to be near or slightly above the depth of subexcavation and can likely be controlled by ditching and pumping from sumps.

It is recommended that the groundwater level be lowered to at least 1 m below the base of the subexcavation level, resulting in temporary groundwater lowering of up to 5 m below the original ground surface (i.e., to about Elevation 214 m) for shallow foundation options. 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. Temporary erosion protection on exposed cuts / fills must be in accordance with OPSS.PROV 804 (Temporary Erosion Control).

6.9.4 Obstructions during Pile Driving / Caisson Installation

During pile installation through the existing fill soils, there is a risk of encountering pockets of gravel or cobbles and boulders. It is recommended that steel H-piles or tube piles be reinforced and protected from damage with appropriate driving shoes as per OPSD 3000.100 (Steel H-Pile Driving Shoe) or 3001.100 (Steel Tube Driving

Shoe) or equivalent. Caisson installation equipment must be capable of penetrating and/or removing obstructions as required.

6.10 Recommendations for Additional Work

The preliminary foundation recommendations provided in this report are based on the limited available subsurface information in the two boreholes advanced near the proposed overpass 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, approach embankments and any associated retaining walls. Consideration should be given to advancing seismic Cone Penetration Tests (with dissipation testing) and pressuremeter tests to refine the settlement estimates and further characterize the firm to very stiff clayey silt-silt to silty clay deposits encountered that are challenging to sample and interpret with conventional push equipment. Consideration could be given to using specialized piston samplers or longer tube samplers (as opposed to conventional thin-walled Shelby tube extraction methods) or mini-block samplers (similar to Sherbrooke sampler but smaller diameter) as an attempt to collect less disturbed samples of the clayey deposits containing silt/sand seams and additional consolidation tests (including larger diameter consolidation tests if larger diameter samples are collected) performed accordingly. In addition, the extent (bottom) and characteristics of the lower cohesive deposit and depth to competent soil (100-blow or competent granular soil) should be confirmed across the bridge and approach embankment footprints. Consideration could also be given to constructing a test fill pad with settlement monitoring at / near the site during design to improve prediction of settlement magnitude and rates.

After more detailed foundation investigation is complete, the global stability of the approach embankments and retaining walls will need to be checked and the magnitude of foundation settlements and any mitigation measures (including estimated preload times) will need to be reassessed, especially if the ultimate configuration of the bridge approach embankments is to be constructed at the interim stage. 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. The use of GSC 5th Generation or 6th Generation seismic hazard maps to define the Site Class should be confirmed for detail design, and geophysical testing (MASW or VSP) be performed if the difference between a Site Class C or D will impact the seismic performance category classification.

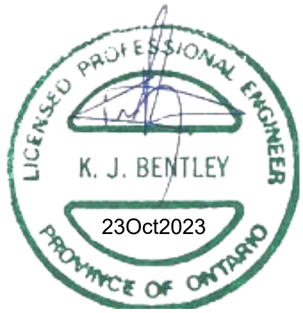
Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*. The existing standpipe piezometer (installed in Borehole B-2) should be maintained operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometer will need to be decommissioned in accordance with Ontario Regulation 903 (as amended). Additional piezometers should be installed near the proposed foundation elements to provide the necessary information to assess dewatering requirements.

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 III

Kevin Bentley, M.E.Sc., P.Eng.
Designated MTO Foundations Contact

MH/KJB/al

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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.

Slide2 (Version 9.017) 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

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 501 Construction Specification for Compacting

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)

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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 - Bathurst Street Overpass

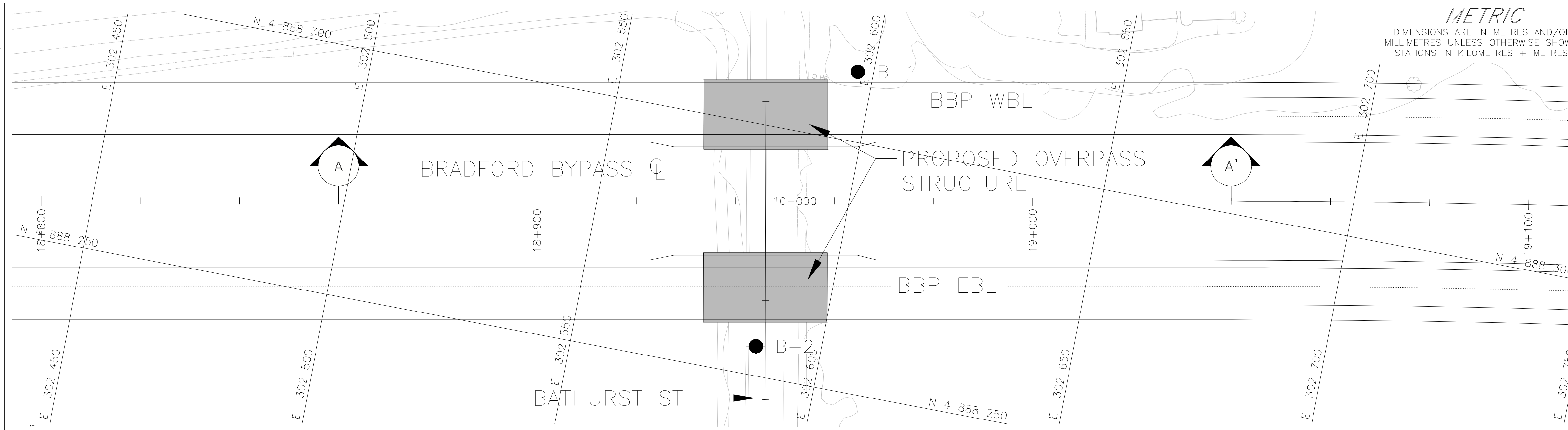
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Strip / Spread footings founded on native compact silty sand to sandy silt	Feasible at both abutment locations	■ Conventional construction ■ Relatively competent soils at shallow depth (below surficial loose/firm soil layers) will provide adequate geotechnical resistance.	■ Lower geotechnical resistance compared to deep foundations. ■ Excavation of unsuitable soils to about 4 m depth is required to reach competent founding stratum. ■ Dewatering in saturated silts and sands will be required to allow for excavation and construction of footings in dry conditions and maintain stable foundation subgrade. ■ Temporary protection systems may be needed if Bathurst St. is to remain open during construction; alternatively, closure or temporary realignment of Bathurst St. may also be required during construction. ■ Does not allow for conventional integral abutment design.	■ Lower cost than deep foundations although additional costs for dewatering and temporary protection systems will need to be considered.	■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual abutment locations. ■ Risk of deeper excavation and increased dewatering and/or temporary shoring efforts. ■ Risk of disturbance to founding subgrade if adequate dewatering is not provided in saturated sands and silts
"Perched" abutment spread footings founded on a compacted granular pad within approach embankments	Feasible at both abutment locations	■ Conventional construction ■ Granular pad can be constructed within approach embankment for abutment locations. ■ Founding level can easily be adjusted within approach embankment. ■ Depth of excavation, dewatering effort, and height of abutment wall stems can be reduced. ■ Increased geotechnical resistance compared to shallow foundation on native deposits.	■ Lower geotechnical resistance compared to deep foundations. ■ Subexcavation and replacement of unsuitable soils to about 4 m depth is required within foundation zone of influence to mitigate settlement under embankment loading, or other settlement mitigation (such as preloading and/or surcharging or ground improvement) required to be developed during detail design. ■ Dewatering of surficial silty sand may be required to allow for subexcavation and placement and compaction of granular pad in dry conditions and maintain stable subgrade. ■ Temporary protection systems may be needed if Bathurst St. is to remain open during construction; alternatively, closure or temporary realignment of Bathurst St. may also be required during construction. ■ Does not allow for conventional integral abutment design.	■ Lower cost than deep foundations ■ Similar costs for spread footings founded on native soil due to subexcavation and dewatering to construct granular pad.	■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual abutment locations. ■ Risk of deeper excavation and increased dewatering and/or temporary shoring efforts. ■ 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.
Steel H-piles or tube piles driven to 30 m or 40 m depth	Feasible at both abutment locations	■ Conventional construction methods for driven steel pile foundations. ■ Higher axial resistances available compared to shallow footings. ■ Pile caps perched within approach embankments can be considered to reduce or eliminate dewatering / subexcavation requirements. ■ Allows for integral abutment design.	■ Noise and vibrations to adjacent properties, although limited residential and industry near site. ■ Dewatering measures may be required at abutments for the construction of pile caps, unless perched in embankment fill at abutments. ■ Driving shoes and/or thicker pile section may be required to drive greater than 30 m length.	■ Lower relative cost than drilled shafts (caissons) ■ Comparable cost to spread footings if dewatering and subexcavation of unsuitable soils can be reduced by designed perched pile caps.	■ Reduced impact on design if variable near surface soils are encountered during detailed investigation. ■ Risk of piles "hanging up" or being deflected from alignment when driving through soils that may contain pockets of gravel, cobbles and boulders (to be confirmed during detail design).
Drilled Shafts (Caissons) installed to 30 m or 40 m depth	Feasible at both abutment locations	■ Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. ■ 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.	■ 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). ■ Generation and disposal of soils cuttings / slurry during drilled shaft advancement ■ Does not allow for conventional integral abutment design.	■ Higher relative cost than shallow foundations. ■ Higher cost than piles but reduced dewatering / subexcavation costs if pier caissons are cast continuously with structural columns to eliminate pile cap.	■ Reduced impact on design if variable near surface soils are encountered during detailed investigation. ■ Risk of difficulties penetrating through soil deposits that may contain pockets of gravel or cobbles and boulders (to be confirmed during detail design). ■ Challenging to inspect the shaft and base of the drilled shafts due to the need for liners, slurry, and tremie concrete methods.

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

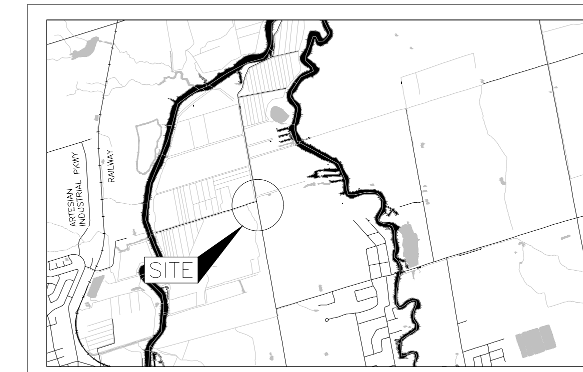
CONT No.
WP No.

BRADFORD BYPASS
BATHURST STREET OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



PLAN
SCALE
10 0 10 20 m



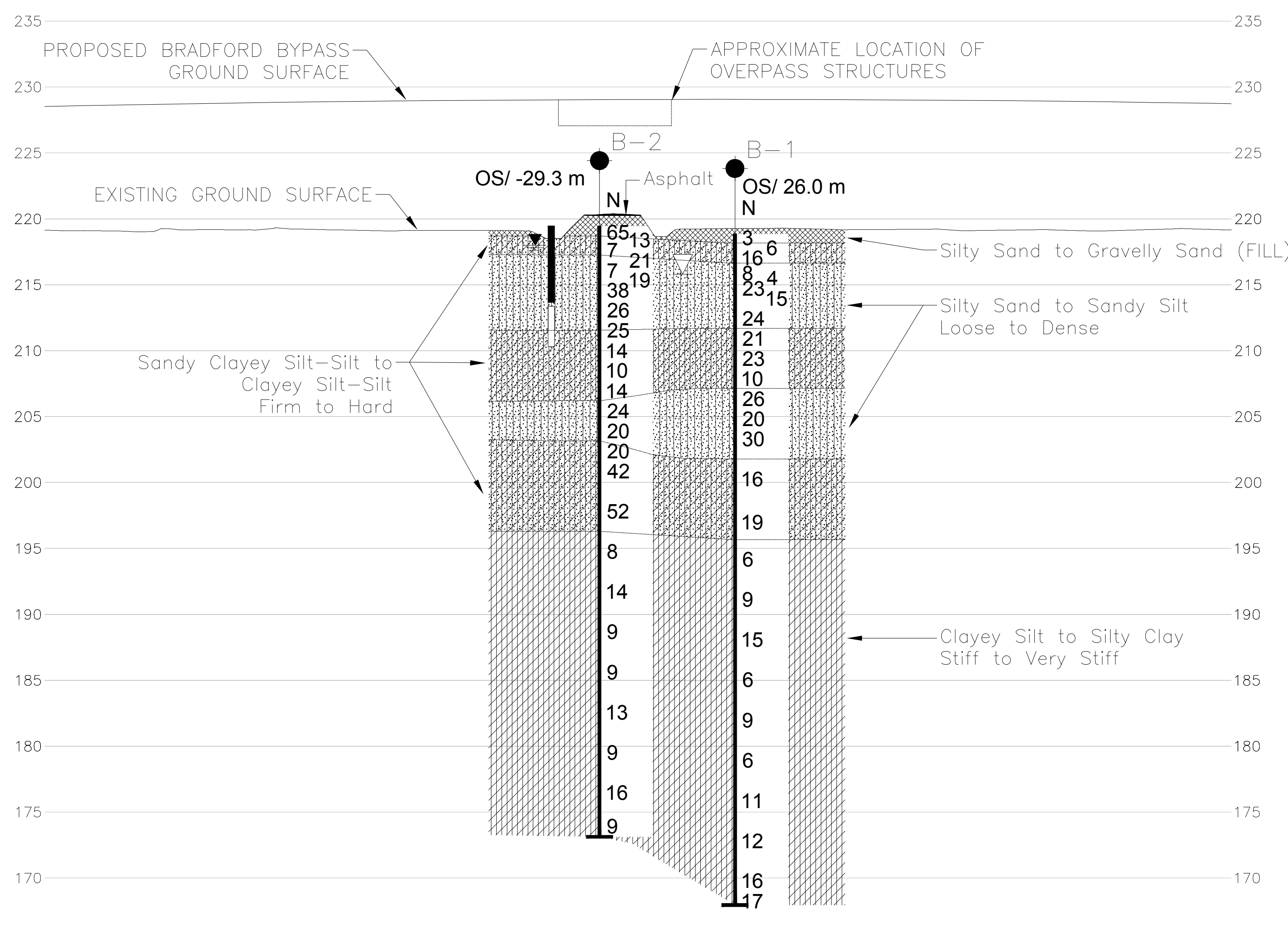
KEY PLAN
SCALE
1 0 1 2 km

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on February 15, 2023
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B–1	218.9	4888314.0	302596.8
B–2	219.5	4888255.9	302587.0



A–A' PROFILE BRADFORD BYPASS ℄

HORIZONTAL SCALE
10 0 10 20 m

VERTICAL SCALE
5 0 5 10 m

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.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Aecom, drawing file no. X-Base_Bradford Bypass.dwg and BRADFORD BY-PASS OG_Combined.xml, received January 11, 2022.
Vertical alignment provided in digital format by Aecom, drawing file no. ACAD-X-60636190-C-DES-BBP Mainline Align and Profile.dwg, received September 9, 2022.
Horizontal alignment provided in digital format by Aecom, drawing file no. X-60636190-C-DES-All Alignments.xml, received September 14, 2022.
Design plan provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Overall Plan.dwg, received September 14, 2022.

LICENSED PROFESSIONAL ENGINEER K. J. BENTLEY Oct. 20, 2023 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER M. S. HENDERSON 100523882 Oct. 20, 2023 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D00–826

HWY.		PROJECT NO. 19136074	DIST.
SUBM'D. KJB	CHKD. MH	DATE: 10/17/2023	SITE:
DRAWN: DD/SA	CHKD. MH	APPD. KJB	DWG. ----

MINISTRY OF TRANSPORTATION, ONTARIO

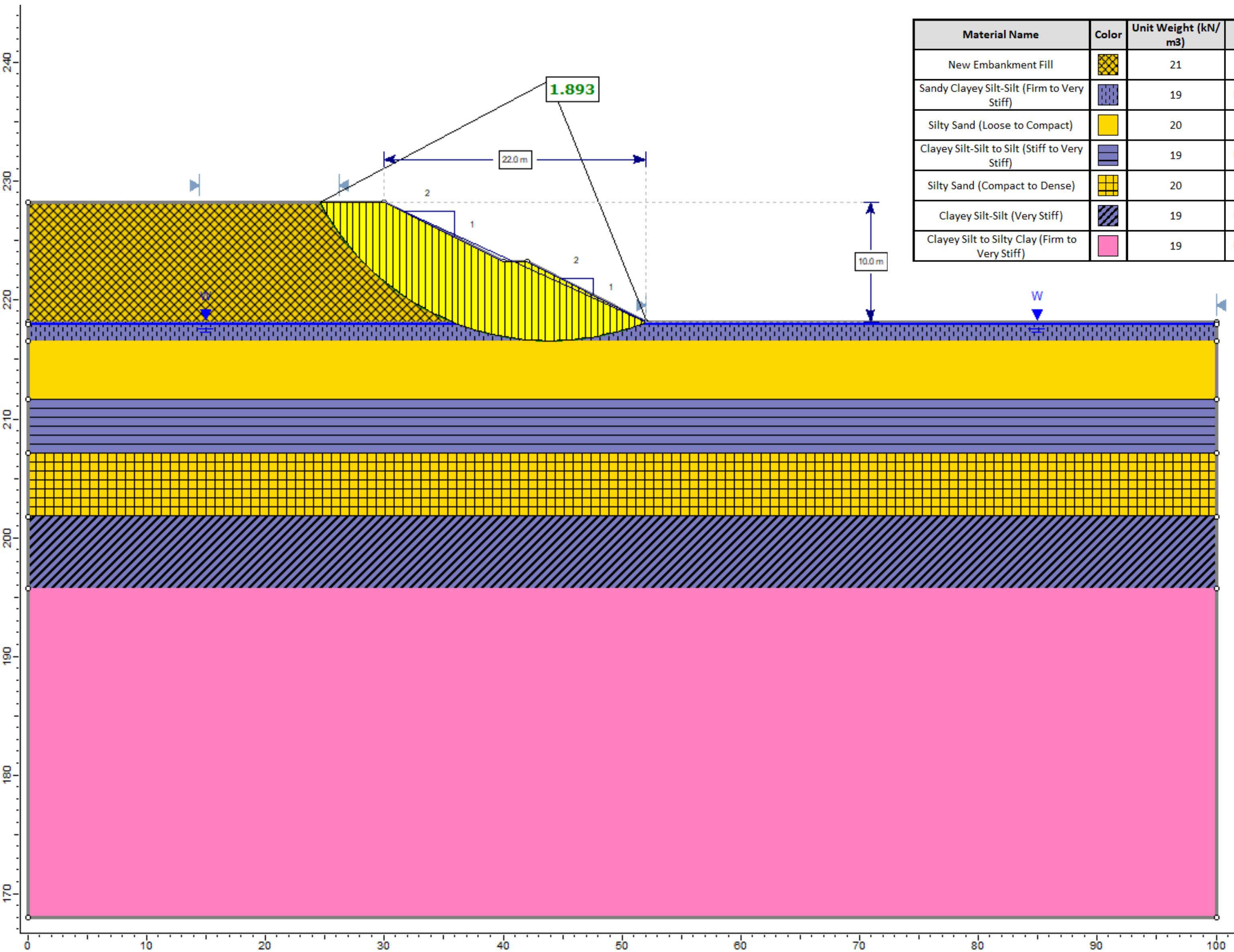
PLOT DATE: October 19, 2023
FILENAME: S:\Clients\MTO\Bradford_Bypass\99_PROJ\19136074_Aecom\40_PROD\0013_Bathurst_Street_Overpass\19136074-0013-BC-0001.dwg

Global Stability Analysis Results (Undrained Condition)

Critical Case (Eastbound Bridge, Borehole B-1)

Figure 1

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
New Embankment Fill		21	Mohr-Coulomb	0	36		Water Surface	Custom	1
Sandy Clayey Silt-Silt (Firm to Very Stiff)		19	Undrained	50		Constant	Water Surface	Custom	1
Silty Sand (Loose to Compact)		20	Mohr-Coulomb	0	33		Water Surface	Custom	1
Clayey Silt-Silt to Silt (Stiff to Very Stiff)		19	Undrained	75		Constant	Water Surface	Custom	1
Silty Sand (Compact to Dense)		20	Mohr-Coulomb	0	34		Water Surface	Custom	1
Clayey Silt-Silt (Very Stiff)		19	Undrained	100		Constant	Water Surface	Custom	1
Clayey Silt to Silty Clay (Firm to Very Stiff)		19	Undrained	60		Constant	Water Surface	Custom	1



Date: September 2023

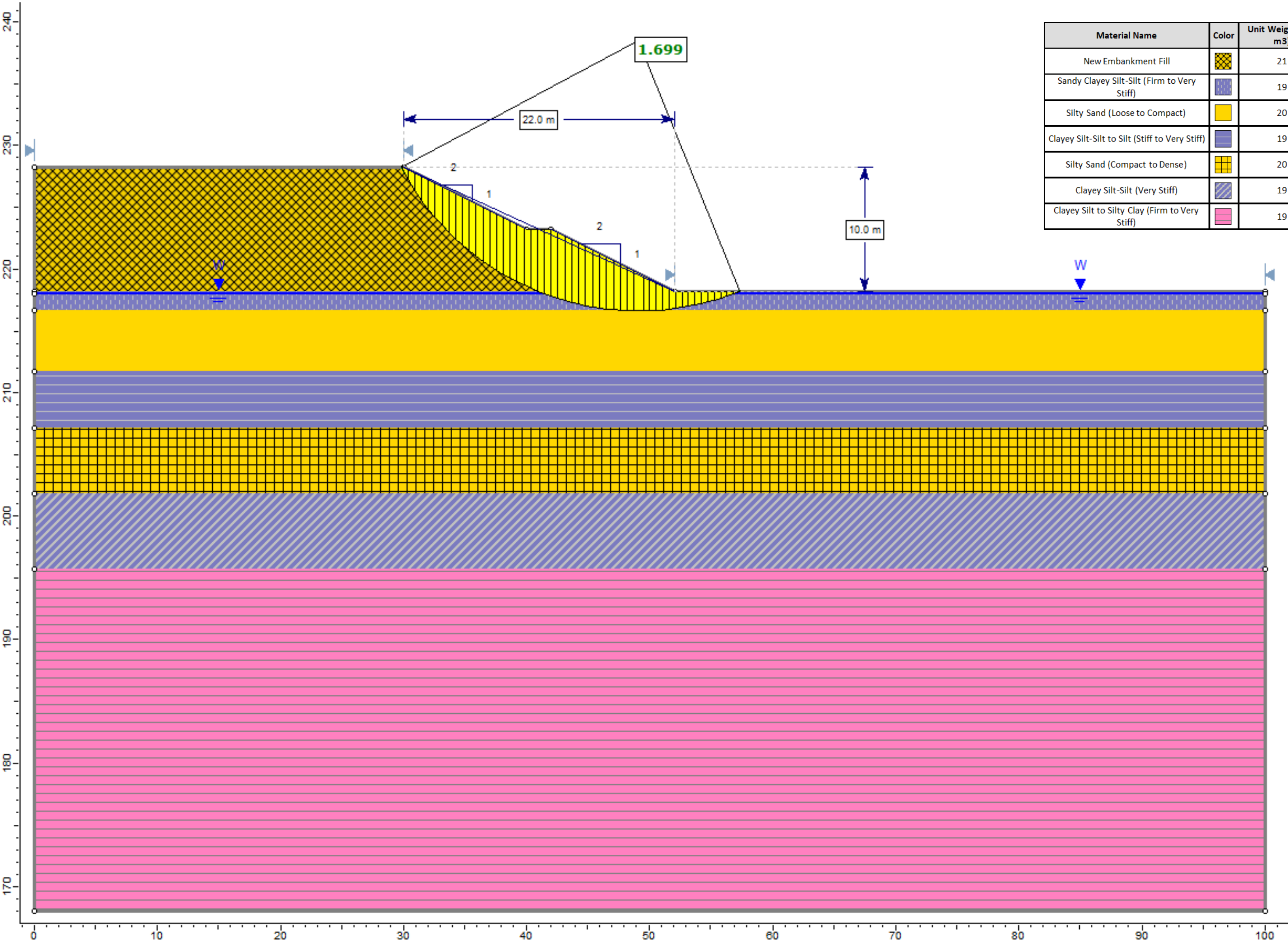
Project No: 19136074

Analysis By: AM **Reviewed By:** KJB

Global Stability Analysis Results (Drained Condition)

Critical Case (Eastbound Bridge, Borehole B-1)

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
New Embankment Fill		21	Mohr-Coulomb	0	36	Water Surface	Custom	1
Sandy Clayey Silt-Silt (Firm to Very Stiff)		19	Mohr-Coulomb	0	30	Water Surface	Custom	1
Silty Sand (Loose to Compact)		20	Mohr-Coulomb	0	33	Water Surface	Custom	1
Clayey Silt-Silt to Silt (Stiff to Very Stiff)		19	Mohr-Coulomb	0	31	Water Surface	Custom	1
Silty Sand (Compact to Dense)		20	Mohr-Coulomb	0	34	Water Surface	Custom	1
Clayey Silt-Silt (Very Stiff)		19	Mohr-Coulomb	0	30	Water Surface	Custom	1
Clayey Silt to Silty Clay (Firm to Very Stiff)		19	Mohr-Coulomb	0	28	Water Surface	Custom	1

Date: September 2023

Project No: 19136074

Analysis By: AM **Reviewed By:** KJB

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 Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (*i.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* 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^2 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[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

Term	SPT 'N' (blows/0.3m)[2]
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 grainsize. 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'[1,2] (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

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ. Unit weight symbol is γ. where $\gamma = \rho \cdot g$ (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_α	rate of secondary compression
$C_{\alpha(\varepsilon)}$	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
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 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

Ontario

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PROJECT	19136074	RECORD OF BOREHOLE	No. B-1	Sheet 1 of 6	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888314; E 302596.8 NAD83 / MTM Zone 10 (LAT. 44.134818; LONG. -79.527534)	ORIGINATED BY	SS
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MH
DATUM	CGVD28 Surface Elevation:218.9 m	DATE	Apr 21, 2021 - Apr 27, 2021	CHECKED BY	KJB

SOIL PROFILE | SAMPLES | GROUNDWATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): X Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined | WATER CONTENT (%) PL W_p NMC W LL W_l (20 40 60), NP Nonplastic | UNIT WEIGHT γ kN/m³ | GR SA SI CL | REMARKS

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
0.0	SILTY SAND (SM), (FILL), trace gravel, trace organics Very loose Black to brown Moist	1	SS	3				
218.2 / 0.7	Sandy CLAYEY SILT-SILT (CL-ML) Firm to very stiff Brown Moist to wet	2	SS	6	0	26	63	11
	- 1.5 to 2.1 m: Sand seams encountered within sample.	3	SS	16	0	30	59	11
216.6 / 2.2	SILTY SAND (SM), trace clay, Loose to Compact Brown to grey Wet	4	SS	8	0	74	20	6
		5	SS	4	0	51	48	1
		6	SS	23	0	82	16	2
		7	SS	15	0	53	43	4
		8	SS	24				
211.7 / 7.2	CLAYEY SILT-SILT (CL-ML) to SILT (ML), trace sand Very stiff Grey Wet	9	SS	21				
		10	SS	23	0	2	91	7

Elevation scale: 218, 217, 216, 215, 214, 213, 212, 211, 210, 209

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity   o³% STRAIN AT FAILURE

Ontario

PROJECT 19136074	RECORD OF BOREHOLE No. B-1	Sheet 2 of 6	METRIC
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888314; E 302596.8 NAD83 / MTM Zone 10 (LAT. 44.134818; LONG. -79.527534)	ORIGINATED BY SS	
DIST Central HWY BBP - Bathurst St	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY MH	
DATUM CGVD28 Surface Elevation:218.9 m	DATE Apr 21, 2021 - Apr 27, 2021	CHECKED BY KJB	

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	CLAYEY SILT-SILT (CL-ML) to SILT (ML), trace sand Very stiff Grey Wet								
		11	SS	10	208				
207.1 / 11.7	SILTY SAND (SM), trace clay Compact to dense Grey Moist to Wet				207				
		12	SS	26	206				
		13	SS	20	205				
					204				
		14	SS	30	203	0	67	32	1
201.8 / 17.1	CLAYEY SILT-SILT (CL-ML) Very Stiff Grey Wet				202				
					201				
		15	SS	16	200				
					199				

Column headings: SOIL PROFILE; SAMPLES; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane 20 40 60 80 100; WATER CONTENT (%) PL NMC LL W_p W W_l NP Nonplastic 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

PROJECT	19136074		**RECORD OF BOREHOLE**	**No. B-1**	Sheet 3 of 6		**METRIC**
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888314; E 302596.8 NAD83 / MTM Zone 10 (LAT. 44.134818; LONG. -79.527534)			ORIGINATED BY	SS
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary			COMPILED BY	MH
DATUM	CGVD28 Surface Elevation:218.9 m	DATE	Apr 21, 2021 - Apr 27, 2021			CHECKED BY	KJB

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	REMARKS
	CLAYEY SILT-SILT (CL-ML) Very Stiff Grey Wet				198			
		16	SS	19	197			
					196			
195.7 23.2	CLAYEY SILT (CL) to SILTY CLAY (CI) Firm to very stiff Grey Wet				195			
		17	SS	6	194			
					193			
						>96 kPa		
					192			
		18	SS	9	191			
						>96 kPa		
					190			
					189			

Legend: Field Vane (X), Remoulded (⊕), Pocket Pen (■), Quick Triaxial (●), Unconfined (○). Dynamic cone penetration resistance plot scale: 20 40 60 80 100. Water content scale: 20 40 60 (PL W_p, NMC W, LL W_l; NP Nonplastic). Unit weight γ kN/m³. GR SA SI CL.

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

PROJECT	19136074	**RECORD OF BOREHOLE No. B-1**	Sheet 4 of 6	**METRIC**	
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888314; E 302596.8 NAD83 / MTM Zone 10 (LAT. 44.134818; LONG. -79.527534)	ORIGINATED BY	SS
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MH
DATUM	CGVD28 Surface Elevation:218.9 m	DATE	Apr 21, 2021 - Apr 27, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	REMARKS
	CLAYEY SILT (CL) to SILTY CLAY (CI) Firm to very stiff Grey Wet - 30.9 to 31.0 m: 100 mm thick sandy silt pocket encountered						
		19	SS	15	188		
					187		
					186		
		20	SS	6	185		
					184	× >96 kPa	
					183		
		21	SS	9	182		
					181	× >96 kPa	
					180		
		22	SS	6	179		

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): × Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined; WATER CONTENT (%) PL NMC LL, W_p W W_l, NP Nonplastic (20 40 60); UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

PROJECT 19136074	RECORD OF BOREHOLE No. B-1	Sheet 5 of 6	METRIC
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888314; E 302596.8 NAD83 / MTM Zone 10 (LAT. 44.134818; LONG. -79.527534)		ORIGINATED BY SS
DIST Central HWY BBP - Bathurst St	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY MH
DATUM CGVD28 Surface Elevation:218.9 m	DATE Apr 21, 2021 - Apr 27, 2021		CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL) to SILTY CLAY (CI) Firm to very stiff Grey Wet										
						178	⊕ ×				
						177					
		23	SS	11		176		├─o─┤			
						175					
						174					
		24	SS	12		173					
						172					
						171					
		25	SS	16		170					
						169					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): × Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined

WATER CONTENT (%): PL (W_p), NMC (W), LL (W_l); NP Nonplastic; 20 40 60

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

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PROJECT 19136074 | **RECORD OF BOREHOLE No. B-1** | | **METRIC**

G.W.P. Assignment No 2019-E-0048 | LOCATION N 4888314; E 302596.8 NAD83 / MTM Zone 10 (LAT. 44.134818; LONG. -79.527534) | ORIGINATED BY SS

DIST Central | HWY BBP - Bathurst St | BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary | COMPILED BY MH

DATUM CGVD28 Surface Elevation:218.9 m | DATE Apr 21, 2021 - Apr 27, 2021 | CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT	GR SA SI CL	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane; 20 40 60 80 100	PL NMC LL; W_p W W_l; NP Nonplastic; 20 40 60	γ kN/m³		
	CLAYEY SILT (CL) to SILTY CLAY (CI) Firm to very stiff Grey Wet		26	SS	17							
168.0 50.9	End of Borehole Note: 1. Hollow Stem Augers to 3.05 m depth and then switched to mud rotary. 2. Water level measured at a depth of 3.05 m prior to mud rotary.						168					
							167					
							166					
							165					
							164					
							163					
							162					
							161					
							160					
							159					

$+^3$, $\times^3$: Numbers refer to Sensitivity ○^{3}% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No. B-2

METRIC

PROJECT	19136074		
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888255.9; E 302587 NAD83 / MTM Zone 10 (LAT. 44.134295; LONG. -79.527657)
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	May 26, 2021 - Jun 02, 2021
ORIGINATED BY	SS		
COMPILED BY	MH		
CHECKED BY	KJB		

ELEV. / DEPTH	DESCRIPTION
0.0	ASPHALT (150 mm thick)
219.3 / 0.2	Gravelly SAND (SW) (FILL) Very dense Brown Moist
218.7 / 0.8	Sandy CLAYEY SILT-SILT (CL-ML) Firm to stiff Brown Moist
217.2 / 2.2	Sandy SILT (ML), trace clay Loose to dense Brown and black Moist to wet
211.5 / 7.9	CLAYEY SILT-SILT (CL-ML), trace sand Stiff to very stiff Grey Moist to wet

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL	REMARKS
1	SS	65					
2	SS	13					
3	SS	7	0	22	69	9	
4	SS	21					
5	SS	7					
6	SS	19					
7	SS	38	0	23	67	10	
8	SS	26					NP
9A / 9B	SS	25					
10	SS	14					

Elevation scale: 219, 218, 217, 216, 215, 214, 213, 212, 211, 210

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): X Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined
WATER CONTENT (%): PL W_p, NMC W, LL W_l; NP Nonplastic; 20 40 60
UNIT WEIGHT γ kN/m³

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity   o³% STRAIN AT FAILURE

Ontario

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PROJECT 19136074 | **RECORD OF BOREHOLE No. B-2** | Sheet 2 of 5 | **METRIC**

G.W.P. Assignment No 2019-E-0048 | LOCATION N 4888255.9; E 302587 NAD83 / MTM Zone 10 (LAT. 44.134295; LONG. -79.527657) | ORIGINATED BY SS

DIST Central | HWY BBP - Bathurst St | BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary | COMPILED BY MH

DATUM CGVD28 Surface Elevation:219.5 m | DATE May 26, 2021 - Jun 02, 2021 | CHECKED BY KJB

ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
	CLAYEY SILT-SILT (CL-ML), trace sand Stiff to very stiff Grey Moist to wet				209						
		11	SS	10							
					208						
		12	SS	14	207						
206.2 / 13.3	Sandy SILT (ML), contains clay pockets Compact to very dense Grey Moist to wet				206						
		13	SS	24		o	0	26	64	10	
					205						
		14	SS	20	204						
203.2 / 16.3	Sandy CLAYEY SILT-SILT (CL-ML) to CLAYEY SILT-SILT (CL-ML), trace sand Very stiff to hard Grey Moist				203						
		15	SS	20		H-o	0	21	64	15	
					202						
		16	SS	42	201						
					200						

Column headers also include: STRATA PLOT; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Field Vane, X Remoulded; WATER CONTENT (%) PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³.

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity   o³% STRAIN AT FAILURE

Ontario

PROJECT	19136074				
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888255.9; E 302587 NAD83 / MTM Zone 10 (LAT. 44.134295; LONG. -79.527657)	ORIGINATED BY	SS
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MH
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	May 26, 2021 - Jun 02, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
	Sandy CLAYEY SILT-SILT (CL-ML) to CLAYEY SILT-SILT (CL-ML), trace sand Very stiff to hard Grey Moist				199						
		17	SS	52	198	o	0	7	87	6	
					197						
196.3 / 23.2	CLAYEY SILT (CL), trace sand Stiff to very stiff Grey Moist				196						
		18	SS	8	195	o	0	0	70	30	
					194						
					193						
		19	SS	14	192						
					191						
					190						

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane; WATER CONTENT (%) PL W_p NMC W LL W_l NP Nonplastic 20 40 60; UNIT WEIGHT γ kN/m³

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

wsp

PROJECT	19136074	RECORD OF BOREHOLE No. B-2	Sheet 4 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION N 4888255.9; E 302587 NAD83 / MTM Zone 10 (LAT. 44.134295; LONG. -79.527657)	ORIGINATED BY	SS
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MH
DATUM	CGVD28 Surface Elevation:219.5 m	DATE May 26, 2021 - Jun 02, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL), trace sand Stiff to very stiff Grey Moist						189					
			20	SS	9							
							188					
							187					
							186					
			21	SS	9							
							185					
							184					
							183					
			22	SS	13							
							182					
							181					
							180					
			23	SS	9						0 0 61 39	

Legend: ✕ Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined. DCPT scale: 20 40 60 80 100. Water content: PL (W_p), NMC (W), LL (W_l); NP Nonplastic; scale 20 40 60.

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

PROJECT	19136074	**RECORD OF BOREHOLE**	**No. B-2**	Sheet 5 of 5	**METRIC**
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888255.9; E 302587 NAD83 / MTM Zone 10 (LAT. 44.134295; LONG. -79.527657)	ORIGINATED BY	SS
DIST	Central HWY BBP - Bathurst St	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MH
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	May 26, 2021 - Jun 02, 2021	CHECKED BY	KJB

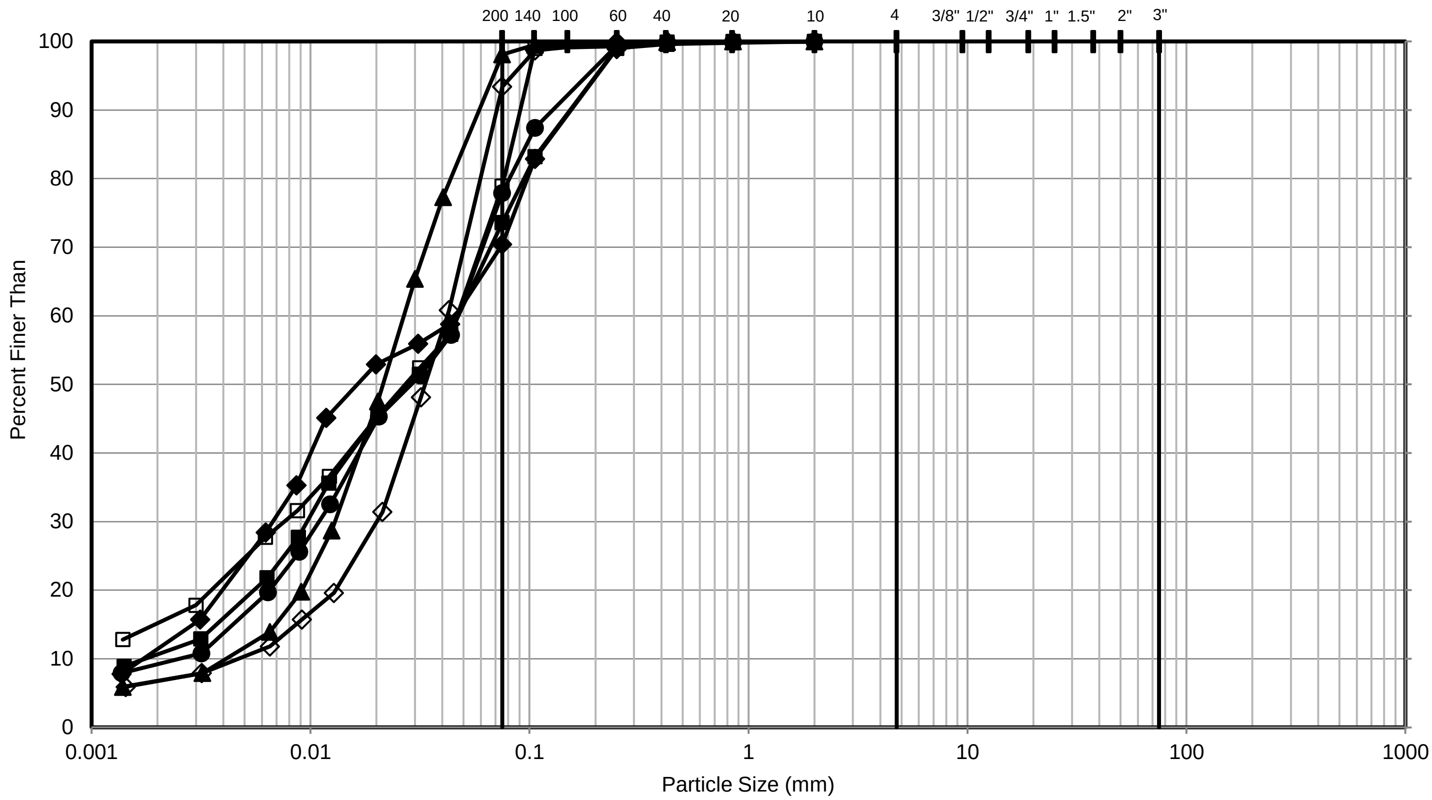
SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane	WATER CONTENT (%) PL W_p, NMC W, LL W_l; NP Nonplastic; 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES							
	CLAYEY SILT (CL), trace sand Stiff to very stiff Grey Moist						179					
							178					
							177					
			24	SS	16							
							176					
							175					
							174					
			25	SS	9							
173.1 46.3	End of Borehole Note: 1. Hollow Stem Augers to 3.05 m depth and then switched to mud rotary. 2. Attempted to check water level in piezometer on February 1, 2023, however flush mount casing was frozen in place. 3. Water level in piezometer measured at a depth of 1.4 m (Elev. 218.0 m) on February 15, 2023.						173					
							172					
							171					
							170					

+³, x³ : Numbers refer to Sensitivity   o³% STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

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



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	B-1	2	0.8 - 1.4	218.1 to 217.5
◆	B-1	3	1.5 - 2.1	217.3 to 216.7
▲	B-1	10	9.1 - 9.7	209.8 to 209.2
●	B-2	3	1.5 - 2.1	217.9 to 217.3
□	B-2	15	16.8 - 17.4	202.7 to 202.1
◇	B-2	17	21.3 - 22.0	198.1 to 197.5

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Bathurst Street

CONSULTANT

WSP GOLDER

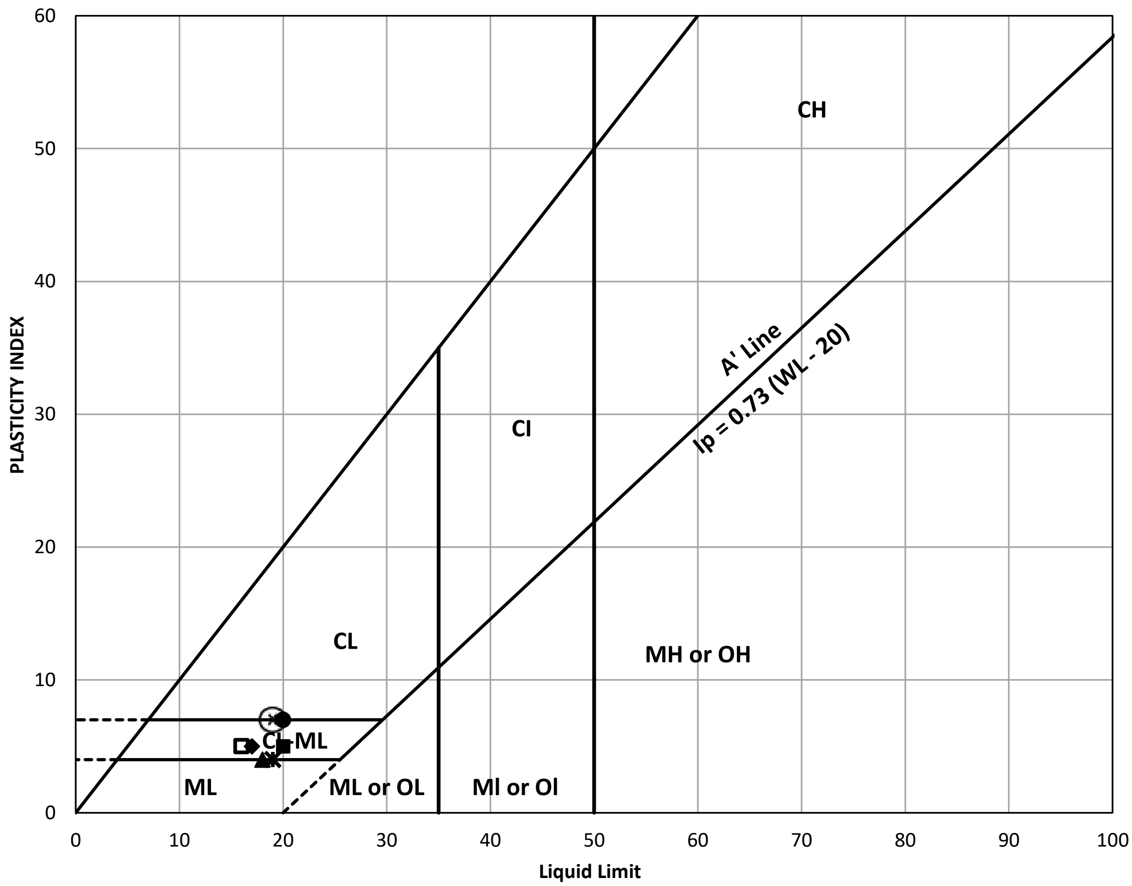
YYYY-MM-DD	2023-02-17
DESIGNED	MH
PREPARED	MH
REVIEWED	KB
APPROVED	KB

TITLE

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

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B1

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



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	B-1	2	0.8 - 1.4	23.4	20	15	5
◆	B-1	3	1.5 - 2.1	19.1	17	12	5
▲	B-1	9	7.6 - 8.2	17.9	18	14	4
●	B-1	15	18.3 - 18.9	20.2	20	13	7
✳	B-1	16	21.6 - 22.2	21.3	19	15	4
⊗	B-2	10	9.1 - 9.8	19.2	19	12	7
□	B-2	15	16.8 - 17.4	17.9	16	11	5

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Bathurst Street

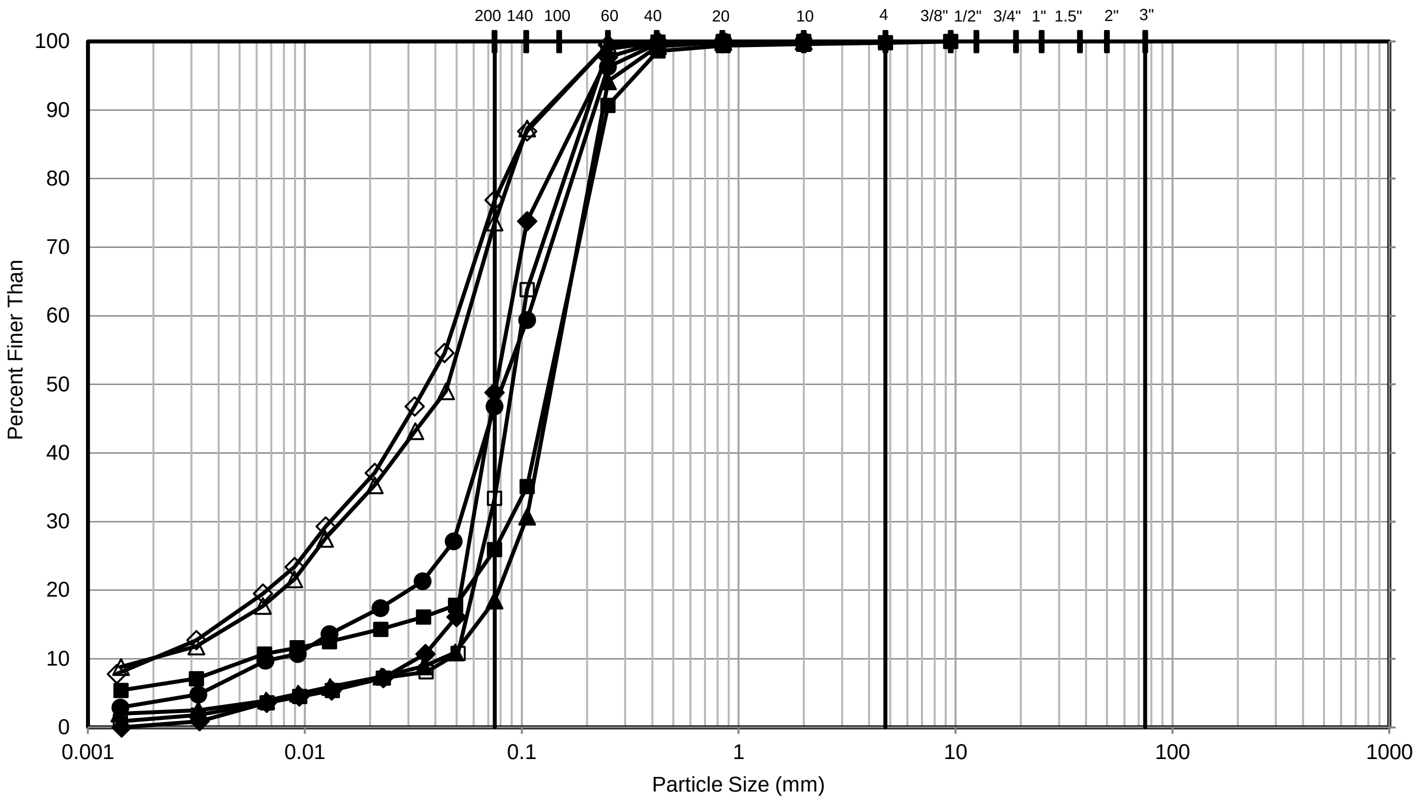
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-02-02
DESIGNED	MTI
PREPARED	MTI
REVIEWED	KB
APPROVED	KB

TITLE
Clayey Silt-Silt to Sandy Clayey Silt-Silt (CL-ML)

PROJECT NO.	CONTROL	FIGURE
19136074	1000	B2

Grain Size Distribution - Silty Sand (SM) to Sandy Silt (ML) - Interlayers



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	B-1	4	2.3 - 2.9	216.6 to 216.0
◆	B-1	5	3.1 - 3.7	215.8 to 215.2
▲	B-1	6	3.8 - 4.4	215.1 to 214.4
●	B-1	7	4.6 - 5.2	214.3 to 213.7
□	B-1	14	15.2 - 15.9	203.6 to 203.0
◇	B-2	7	4.6 - 5.2	214.9 to 214.3
△	B-2	13	13.7 - 14.3	205.7 to 205.1

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Bathurst Street

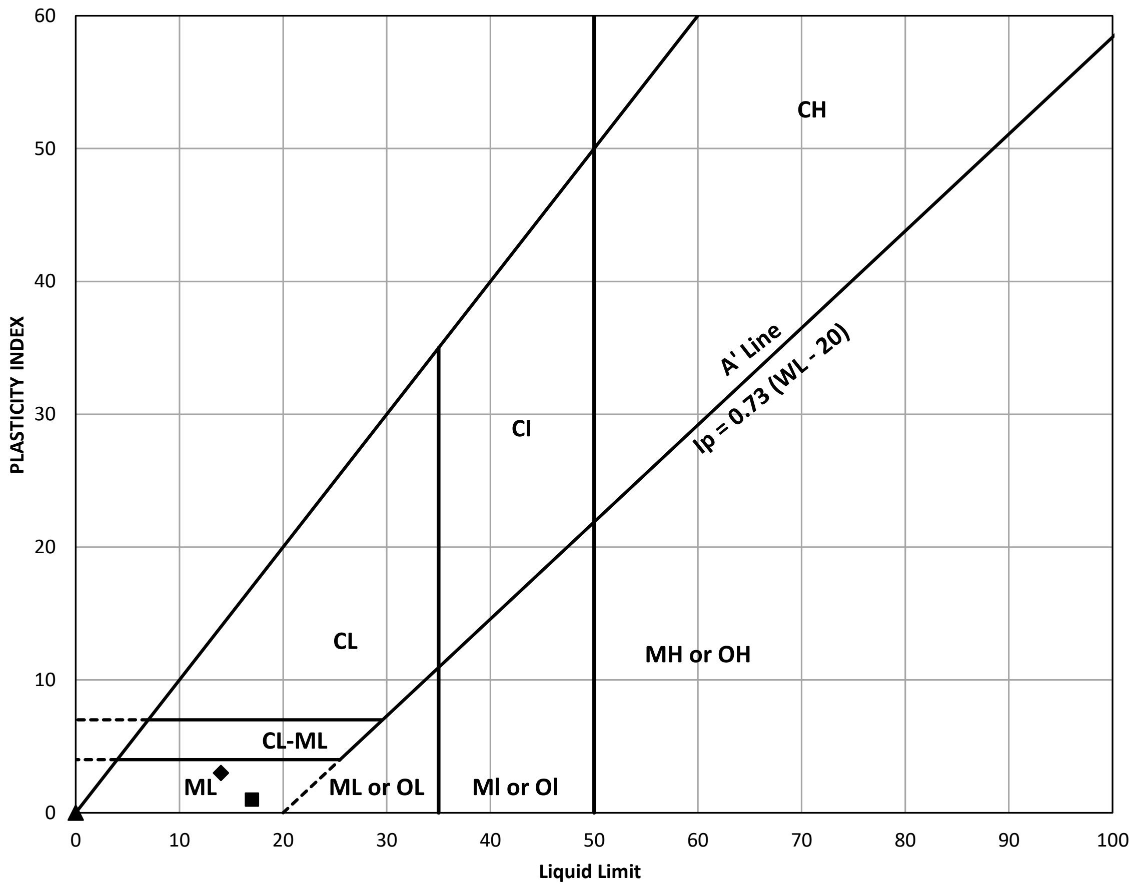
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-02-17
DESIGNED	MH
PREPARED	MH
REVIEWED	KB
APPROVED	KB

TITLE
Grain Size Distribution
Silty Sand (SM) to Sandy Silt (ML) - Interlayers

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B3

Plasticity Chart - Silty Sand (SM) to Sandy Silt (ML) - Interlayers



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	BH B-1	12	12.2 - 12.8	18.1	17	16	1
◆	BH B-2	5	3.1 - 3.7	17.3	14	11	3
▲	BH B-2	8	6.1 - 6.7	17.9	0	NP	0

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Bathurst Street

CONSULTANT
WSP GOLDER

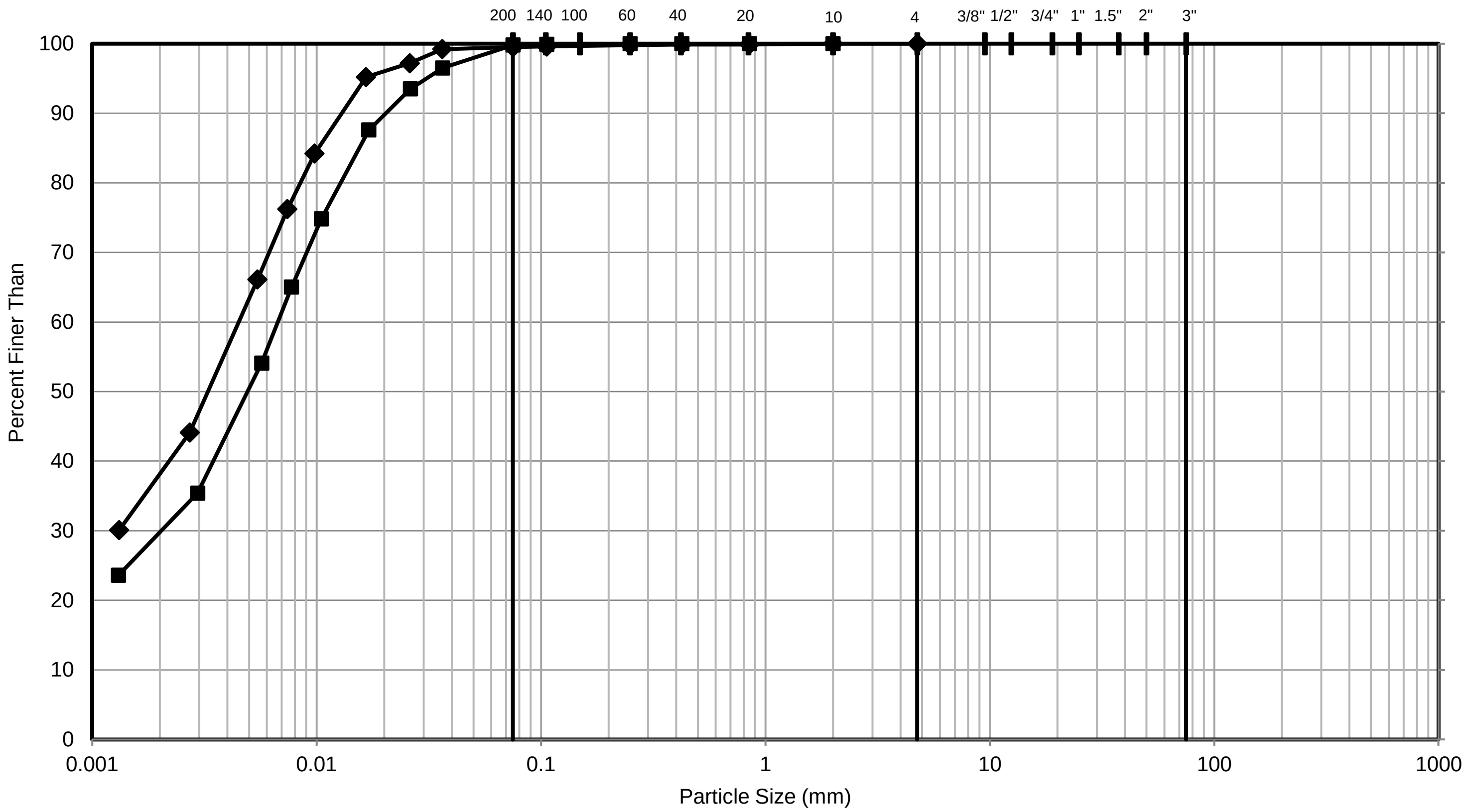
YYYY-MM-DD	2023-02-02
DESIGNED	MTI
PREPARED	MTI
REVIEWED	KB
APPROVED	KB

TITLE
Silty Sand (SM) to Sandy Silt (ML) - Interlayers

PROJECT NO.	CONTROL	FIGURE
19136074	1000	B4

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Bathurst/Appendix B - Lab Results/Working files | FILE NAME: Atterberg Working File - Figure B2 B4 B6 B8 B9.xlsm

Grain Size Distribution - Clayey Silt (CL) to Silty Clay (CI)



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	B-2	18	24.4 - 25.0	195.1 to 194.5
◆	B-2	23	39.6 - 40.2	179.8 to 179.2

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Bathurst Street

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-02-17
DESIGNED	MH
PREPARED	MH
REVIEWED	KB
APPROVED	KB

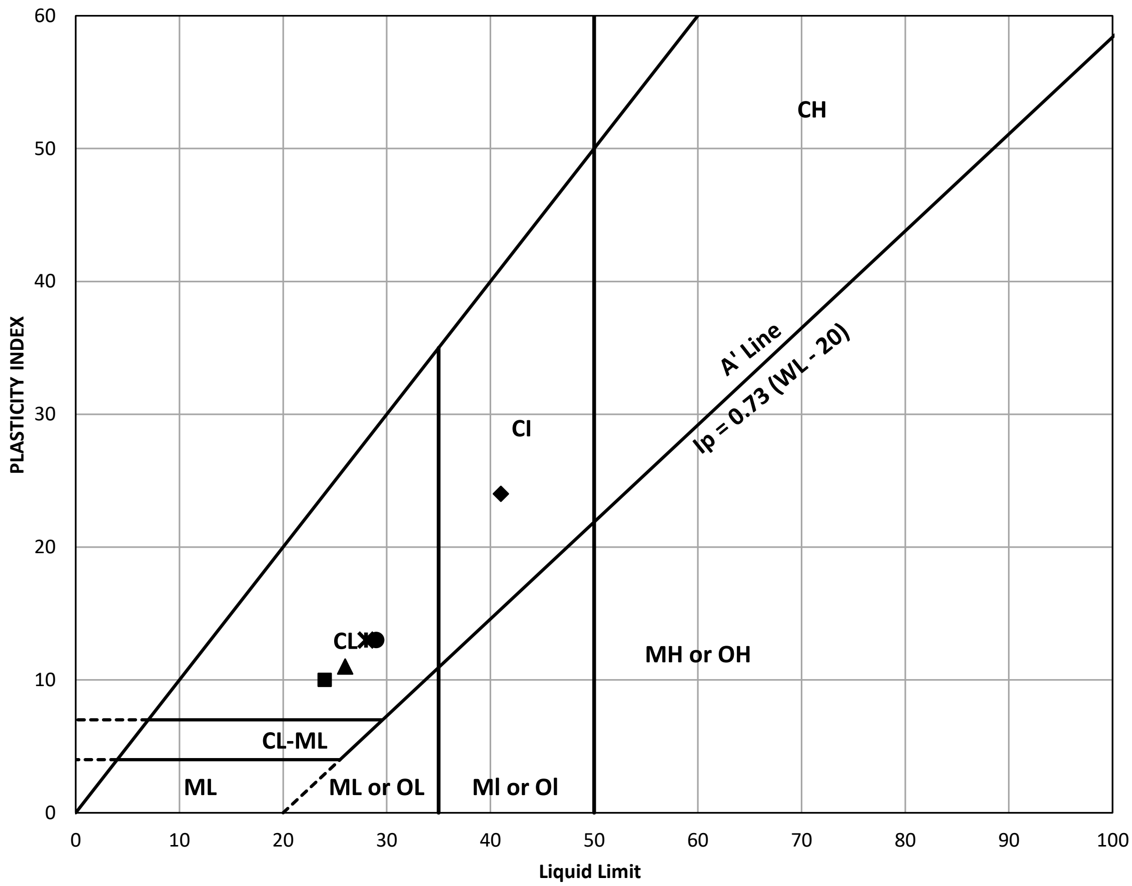
TITLE

Grain Size Distribution
Clayey Silt (CL) to Silty Clay (CI)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B5

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Bathurst/Appendix B - Lab Results/Working files | FILE NAME: GSD Working File - Figure B1 B3 B5 B7.xlsm

Plasticity Chart - Clayey Silt (CL) to Silty Clay (CI)



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	B-1	18	27.4 - 28.0	22.5	24	14	10
◆	B-1	23	42.7 - 43.3	29.3	41	17	24
▲	B-2	20	30.5 - 31.1	22.9	26	15	11
●	B-2	21	33.5 - 34.1	23.5	29	16	13
✱	B-2	23	39.6 - 40.2	24	28	15	13

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Bathurst Street

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-02-02
DESIGNED	MTI
PREPARED	MTI
REVIEWED	MH
APPROVED	KB

TITLE

Clayey Silt (CL) to Silty Clay (CI)

PROJECT NO.	CONTROL	FIGURE
19136074	1000	B6

APPENDIX C

Analytical Chemical Test Results

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

Attention: Carter Comish

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2021/06/25
Report #: R6692694
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C1H2336
Received: 2021/06/22, 16:30

Sample Matrix: Soil
Samples Received: 12

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	12	2021/06/24	2021/06/24	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	12	2021/06/25	2021/06/25	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	12	2021/06/24	2021/06/24	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	12	2021/06/22	2021/06/25	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	12	2021/06/24	2021/06/24	CAM SOP-00464	EPA 375.4 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, MELCC, 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

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2021/06/25
Report #: R6692694
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C1H2336
Received: 2021/06/22, 16:30

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: emese.gitej@bureauveritas.com
Phone# (905)817-5829
==
This report has been generated and distributed using a secure automated process.
BV Labs 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 Signature Page.

BV Labs Job #: C1H2336
Report Date: 2021/06/25

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: CC

RESULTS OF ANALYSES OF SOIL

BV Labs ID		PXF832		PXF833		PXF834	PXF835		
Sampling Date		2021/05/27		2021/05/03		2021/05/17	2021/04/13		
COC Number		827733-01-01		827733-01-01		827733-01-01	827733-01-01		
	UNITS	**B-2 SS3**	**QC Batch**	**10-4 SS2**	**QC Batch**	**Y-1 SS3**	**10-2 SS3**	**RDL**	**QC Batch**
Calculated Parameters									
Resistivity	ohm-cm	1500	7421780	2400	7421780	4500	1400		7421780
Inorganics									
Soluble (20:1) Chloride (Cl-)	ug/g	270	7426573	180	7426733	65	330	20	7426573
Conductivity	umho/cm	651	7429034	416	7429034	220	700	2	7429034
Available (CaCl2) pH	pH	7.66	7426977	7.77	7426977	7.70	7.95		7426977
Soluble (20:1) Sulphate (SO4)	ug/g	<20	7426738	<20	7426744	<20	<20	20	7426738

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

BV Labs ID		PXF835		PXF836		PXF837		PXF838		
Sampling Date		2021/04/13		2021/04/11		2021/06/02		2021/04/12		
COC Number		827733-01-01		827733-01-01		827733-01-01		827733-01-01		
	UNITS	**10-2 SS3 Lab-Dup**	**QC Batch**	**Y-4 SS2**	**RDL**	**L-3 SS3**	**RDL**	**9-1 SS3**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm			3400		380		11000		7421780
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g			130	20	1700	100	<20	20	7426573
Conductivity	umho/cm			295	2	2660	2	90	2	7429034
Available (CaCl2) pH	pH	8.05	7426977	7.81		7.56		7.91		7426977
Soluble (20:1) Sulphate (SO4)	ug/g			<20	20	<20	20	<20	20	7426738

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

BV Labs Job #: C1H2336
Report Date: 2021/06/25

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: CC

RESULTS OF ANALYSES OF SOIL

BV Labs ID		PXF839	PXF840	PXF841	PXF842	PXF843		
Sampling Date		2021/04/29	2021/04/19	2021/04/19	2021/04/30	2021/04/21		
COC Number		827733-01-01	827733-01-01	827733-01-01	827733-01-01	827733-01-01		
	UNITS	**10-1 SS2**	**PDD-1 SS3**	**PDD-2 SS2**	**10-3 SS3**	**B-1 SS3**	**RDL**	**QC Batch**
Calculated Parameters								
Resistivity	ohm-cm	4100	7800	7600	6900	8700		7421780
Inorganics								
Soluble (20:1) Chloride (Cl-)	ug/g	100	<20	<20	25	<20	20	7426573
Conductivity	umho/cm	246	128	131	146	115	2	7429034
Available (CaCl2) pH	pH	7.83	7.74	7.63	7.84	7.88		7426977
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	<20	<20	<20	20	7426738

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

BV Labs ID		PXF843		
Sampling Date		2021/04/21		
COC Number		827733-01-01		
	UNITS	**B-1 SS3 Lab-Dup**	**RDL**	**QC Batch**
Inorganics				
Conductivity	umho/cm	113	2	7429034

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

BV Labs Job #: C1H2336
Report Date: 2021/06/25

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: CC

TEST SUMMARY

BV Labs ID: PXF832
Sample ID: B-2 SS3
Matrix: Soil

Collected: 2021/05/27
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 CaCl2 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

BV Labs ID: PXF833
Sample ID: 10-4 SS2
Matrix: Soil

Collected: 2021/05/03
Shipped:
Received: 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426733	2021/06/24	2021/06/24	Alina Dobreanu
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 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	7426744	2021/06/24	2021/06/24	Avneet Kour Sudan

BV Labs ID: PXF834
Sample ID: Y-1 SS3
Matrix: Soil

Collected: 2021/05/17
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 CaCl2 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

BV Labs ID: PXF835
Sample ID: 10-2 SS3
Matrix: Soil

Collected: 2021/04/13
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 CaCl2 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

BV Labs ID: PXF835 Dup
Sample ID: 10-2 SS3
Matrix: Soil

Collected: 2021/04/13
Shipped:
Received: 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake

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BV Labs Job #: C1H2336
Report Date: 2021/06/25

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: CC

TEST SUMMARY

BV Labs ID: PXF836 **Collected:** 2021/04/11
Sample ID: Y-4 SS2 **Shipped:**
Matrix: Soil **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 CaCl2 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

BV Labs ID: PXF837 **Collected:** 2021/06/02
Sample ID: L-3 SS3 **Shipped:**
Matrix: Soil **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 CaCl2 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

BV Labs ID: PXF838 **Collected:** 2021/04/12
Sample ID: 9-1 SS3 **Shipped:**
Matrix: Soil **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 CaCl2 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

BV Labs ID: PXF839 **Collected:** 2021/04/29
Sample ID: 10-1 SS2 **Shipped:**
Matrix: Soil **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 CaCl2 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

BV Labs ID: PXF840 **Collected:** 2021/04/19
Sample ID: PDD-1 SS3 **Shipped:**
Matrix: Soil **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

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

BV Labs Job #: C1H2336
Report Date: 2021/06/25

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: CC

TEST SUMMARY

BV Labs 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
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 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

BV Labs 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 CaCl2 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

BV Labs ID: PXF842
Sample ID: 10-3 SS3
Matrix: Soil

Collected: 2021/04/30
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 CaCl2 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

BV Labs ID: PXF843
Sample ID: B-1 SS3
Matrix: Soil

Collected: 2021/04/21
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 CaCl2 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

BV Labs ID: PXF843 Dup
Sample ID: B-1 SS3
Matrix: Soil

Collected: 2021/04/21
Shipped:
Received: 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel

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GENERAL COMMENTS

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

Package 1	4.0°C

Results relate only to the items tested.

BV Labs Job #: C1H2336
Report Date: 2021/06/25

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: CC

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7426573	Soluble (20:1) Chloride (Cl-)	2021/06/24	NC	70 - 130	105	70 - 130	<20	ug/g	3.5	35
7426733	Soluble (20:1) Chloride (Cl-)	2021/06/24	NC	70 - 130	104	70 - 130	<20	ug/g	4.4	35
7426738	Soluble (20:1) Sulphate (SO4)	2021/06/24	112	70 - 130	102	70 - 130	<20	ug/g	NC	35
7426744	Soluble (20:1) Sulphate (SO4)	2021/06/24	NC	70 - 130	103	70 - 130	<20	ug/g	19	35
7426977	Available (CaCl2) 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 <= 2x RDL).

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VALIDATION SIGNATURE PAGE

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

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

BV Labs 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 Signature Page.

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6740 Campobello 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:	Golder	Quotation #:	B80683	BV Labs Job #:	Bottle Order #: 827733
Attention:	Accounts Payable	Attention:	Carter Comish	P.O. #:	~~178630~~		
Address:	6925 Century Ave Suite 100 Mississauga ON L5N 7K2	Address:		Project:	19132074	COC #: C#827733-01-01	Project Manager: Ema Gitej
Tel:	(905) 567-4444 Fax: (905) 567-6561	Tel:	Fax:	Project Name:	Bradford Bypass		
Email:	CanadaAccountsPayableInvoices@golder.com	Email:	Carter_Comish@golder.com	Site #: / Sampled By:			

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): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality ☐ PWQO ☐ Reg 406 Table ☐ Other

Special Instructions:

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☒
(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.

Job Specific Rush TAT (if applies to entire submission)
Date Required: Time Required:
Rush Confirmation Number: *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Corrosity pkh short(Cl, SO4, pH, EC/Resistivity)	# of Bottles	Comments
1		B-2 SS3	21/05/27 ~~21/05/18~~	AM	Soil		✓		
2		10-4 SS2	21/05/03	↓	↓		✓		
3		4-1 SS3	21/05/17				✓		
4		10-2 SS3	21/04/13				✓		
5		4-4 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				✓		

22-Jun-21 16:30
Ema Gitej
C1H2336
UDF ENV-929

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted	Time Sensitive	Laboratory Use Only
Carter Comish	21/06/22	4:00pm		2021/06/22	16:30			Temperature (°C) on Recei 5/3/4; Custody Seal Present: Yes; Intact: Yes

* 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.

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CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name:	#1326 Golder Associates Ltd	Company Name:	Golder	Quotation #:	B80683	BV Labs Job #:	Bottle Order #: 827733
Attention:	Accounts Payable	Attention:	Carter Comish	P.O. #:			
Address:	6925 Century Ave Suite 100 Mississauga ON L5N 7K2	Address:		Project:	19136074	COC #: C#827733-02-01	Project Manager: Ema Gitej
Tel:	(905) 567-4444 Fax: (905) 567-6561	Tel:	Fax:	Project Name:	Bradford Bypass		
Email:	CanadaAccountsPayableInvoices@golder.com	Email:	Carter_Comish@golder.com	Site #:			
				Sampled By:			

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): Table 1 / Table 2 / Table 3 / Table ___ ; Res/Park / Ind/Comm / Agri/Other ; Medium/Fine / Coarse / For RSC

Other Regulations: CCME / Reg 558 / MISA / PWQO / Other ; Sanitary Sewer Bylaw / Storm Sewer Bylaw / Municipality ___ / Reg 406 Table ___

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC): Field Filtered (please circle): Metals / Hg / Cr VI ; Corrosity pkh short(Cl, SO4, pH, EC/Resistivity)

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☒
(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.

Job Specific Rush TAT (if applies to entire submission)
Date Required: ___ Time Required: ___
Rush Confirmation Number: ___ *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered / Metals / Hg / Cr VI	Corrosity pkh short(Cl, SO4, pH, EC/Resistivity)	# of Bottles	Comments
1		10-3 SS3	21/04/30	AM	Soil		✓		
2		B-1 SS3	21/04/21	↓	↓		✓		
3									
4									
5									
6									
7									
8									
9									
10									

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted	Time Sensitive	Laboratory Use Only
Carter Comish	21/06/22	4:00pm		21/06/22	16:30			Temperature (°C) on Recei 5/3/4; Custody Seal Present: Yes ✓; Intact: Yes ✓

* 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

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