



FINAL REPORT

Preliminary Foundation Investigation and Design Report

Metrolinx Overpass

Highway 400 to Highway 404 Link (Bradford Bypass)

Simcoe County and York Region

MTO Assignment No. 2019-E-0048

Submitted to:

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PART A

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

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc. and hereafter referenced as WSP Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the proposed Bradford Bypass (BBP), a 16.3 kilometre (km) rural controlled access freeway connecting Highway 400 to Highway 404, in the County of Simcoe and Regional Municipality of York. This report addresses the proposed overpass (twin single-span structures) to carry the proposed new highway westbound lanes (WBL) and eastbound lanes (EBL) over the Metrolinx right-of-way at the location shown on the Key Plan in Drawing 1.

2.0 SITE DESCRIPTION

The proposed twin single-span bridges will cross the Metrolinx tracks just north of Line 8 (8th Line) and Industrial Road, which is located within the Town of Bradford-West Gwillimbury. The Metrolinx rail tracks generally run in a north-south direction at the site and appear to be founded on a fill embankment up to about 2 m high relative to the adjacent ground surface. The site is bounded by industrial properties directly to the south, forested lands to the north, Artesian Industrial Parkway to the west, and Holland River (West Branch) to the east. The immediate area of the proposed overpass structures is a mix of grassy fields, forested lands, and swampy areas (see Drawing 1 and Photographs 1 and 2). There are a number of industrial properties located along Artesian Industrial Parkway and along Industrial Road / 8th Line. The site is generally flat, rising slightly to the west towards Artesian Industrial Parkway and the existing ground surface is heavily vegetated.



Photograph 1 – Looking east across grassy field and forested area from Artesian Industrial Parkway



Photograph 2 – Looking northwest from Borehole CN-3 towards swampy area (bullrushes and open water in distance)

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between November 17 and November 19, 2021, and between March 8 and March 9, 2022, during which time three boreholes (designated CN-1, CN-2, and CN-3) were advanced at the locations shown on Drawing 1. Borehole CN-1 was advanced near the footprint of the proposed westbound bridge over the Metrolinx rail tracks and Borehole CN-2 was advanced near the footprint of the eastbound bridge, both boreholes were advanced on the west side of the Metrolinx rail tracks. Due to the swampy conditions encountered at the site and access restrictions, Borehole CN-3 was advanced at the closest location on the east side of the railway tracks (about 150 m southeast of the site), through the existing roadway platform of 8th Line.

The boreholes were advanced using 210 mm outside diameter (O.D.) hollow stem augers to a depth of approximately 3.0 m below ground surface, followed by wash-rotary techniques (advancement of tricone with water/drilling mud) using a Diedrich D120 track-mounted drill for Boreholes CN-1 and CN-2, and a Diedrich D56 track-mounted drill for Borehole CN-3. Both drill rigs were 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. The water used for drilling was transported to site in water totes (plastic containers) supplied 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.

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 Boreholes CN-1 and CN-3 to allow monitoring of the groundwater level. The installed piezometers consist of a 50 mm diameter PVC pipe, with 1.5 m (Borehole CN-1) to 3.0 m long (Borehole CN-3) slotted screens within a filter sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack was backfilled to near ground surface with bentonite pellets in Borehole CN-1 and well gravel in Borehole CN-3. The monitoring well in Borehole CN-1 was left sticking up out of the ground with a monument cover and the monitoring well in Borehole CN-3 was capped with a casing installed flush with the 8th Line road surface. Borehole CN-2 was backfilled with a bentonite mixture upon completion in general accordance with Ontario Regulation 903 Wells (as amended), and the ground surface was restored to as near original condition as practicable.

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. One consolidation (oedometer) test was performed on a Shelby tube sample collected in borehole CN-2. All laboratory tests were carried out in general accordance with MTO and / or ASTM Standards, as applicable.

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

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

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

are referenced to Geodetic datum (CGVD28 datum). The borehole locations, including the geographic coordinates, the ground surface elevations, and borehole depths, are summarized below.

Borehole No.	NAD83 MTM (Geographic) Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
CN-1	4,887,976 (44.131768)	300,530 (-79.553359)	222.4	14.0
CN-2	4,887,953 (44.131557)	302,514 (-79.553556)	221.8	12.4
CN-3	4,887,798 (44.130169)	300,708 (-79.551132)	219.8	12.6

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)².

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, water level readings, and the results of the in situ and laboratory tests, are provided on the Record of Borehole sheets in Appendix A. The results of the in situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in 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 Metrolinx overpass consist of surficial deposits of cohesive soils comprised of clayey silt and clayey silt-silt underlain by a deposit of sandy silt to silty sand with a variable state of compactness ranging from very loose (near the upper portion of the deposit) to very dense (towards the lower portion of the deposit). The sandy silt to silty sand deposit contained

² Chapman, L.J. and Putnam, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

seams / interlayers of clayey silt, silt, and organics in the upper portion. The deposit of sandy silt to silty sand is underlain by a very dense / hard glacial till deposit comprised of silt to clayey silt-silt.

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

4.2.1 Gravelly Silty Sand Fill

A 0.6 m thick layer of gravelly silty sand fill was encountered at ground surface in Borehole CN-3. The base of the fill layer extended to Elevation 219.2 m.

A single SPT 'N'-value obtained in the gravelly silty sand fill yielded 109 blows per 0.3 m of penetration, indicating a very dense state of compactness.

4.2.2 Clayey Silt to Clayey Silt-Silt

A cohesive deposit of clayey silt to clayey silt-silt containing trace organics was encountered at ground surface in Boreholes CN-1 and CN-2 and underlying the gravelly silty sand fill in Borehole CN-3. The deposit was approximately 0.7 m to 1.4 m thick, extending to depths of 0.7 m to 1.4 m (Elevations 221.7 m to 218.4 m).

Clayey silt to clayey silt-silt seams / interlayers were encountered within the sandy silt to silty sand deposit (described in subsection 4.2.3, below) in all three boreholes. The clayey silt to clayey silt-silt interlayers were encountered at depths of about 2.2 m to 5.0 m (Elevation 220.2 m to 216.0 m) and ranged from less than 25 mm to 0.8 m thick, extending to depths of about 3.0 m to 5.1 m (Elevation 219.4 m to 215.4 m).

The SPT 'N'-values measured in the clayey silt to clayey silt-silt layers range from 2 to 16 blows per 0.3 m of penetration, suggesting a variable, very soft to very stiff consistency.

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

Atterberg limits testing was carried out on two samples of the clayey silt to clayey silt-silt and measured liquid limits of 18% and 24%, plastic limits of 13% and 14%, and plasticity indices of 5% and 10%. These results, which are plotted on a plasticity chart on Figure B2, indicate that the clayey silt to clayey silt-silt materials are of low plasticity.

The natural water content measured on selected samples of the clayey silt to clayey silt-silt ranges between about 12% and 22%.

A laboratory consolidation (oedometer) test was carried out on a vertically trimmed specimen of the clayey silt interlayer (within the silty sand to sandy silt deposit) obtained from a Shelby tube sample at 5.0 m depth in Borehole CN-2 to assess the compressibility characteristics of the deposit. The details of the test results are shown on Figures B3 to B6 and are summarized below.

Borehole / Sample No.	Sample Depth / Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR (avg.)	C_c	C_r	e_o	c_v (cm ² /s)
CN-2 / Sa#7	4.6 – 5.2 / 217.2 – 216.6	75	244 – 354	169 – 279	3 to 5	0.093	0.003	0.37	0.0082 – 0.011

Where: σ_p' = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)
 C_c = Compression index
 e_o = Initial void ratio
 σ_{vo}' = Calculated existing vertical effective stress
 c_v = Coefficient of consolidation (vertical) for approximate overconsolidated stress range $50 \text{ kPa} \leq \sigma_v' \leq 300 \text{ kPa}$
 C_r = Recompression index
OCR = Overconsolidation ratio

4.2.3 Sandy Silt to Silty Sand

A non-cohesive deposit of sandy silt to silty sand was encountered underlying the deposit of clayey silt to clayey silt-silt in all boreholes. The deposit was encountered at depths of 0.7 m to 1.4 m (Elevations 221.7 m to 218.4 m) and was approximately 5.8 m to 7.3 m thick, extending to depths of 7.2 m to 8.7 m (Elevations 215.2 m to 211.1 m). The sandy silt to silty contained frequent clayey silt to clayey silt-silt seams and interlayers as described in the previous section. In Borehole CN-3, an approximately 0.3 m thick organic sandy silt layer was encountered within the deposit at a depth of about 3.5 m (Elevation 216.3 m).

The SPT 'N'-values measured in the sandy silt to silty sand range from 2 to 104 blows per 0.3 m, indicating a variable, very loose to very dense state of compactness. The SPT 'N'-values typically measured 2 blows up to 25 blows per 0.3 m of penetration, with the SPT 'N'-values increasing markedly towards the bottom 2 m to 3 m of the deposit where clayey silt seams/interlayers were typically not encountered. Within this lower zone of the deposit, the SPT 'N'-values measured 30 upwards to 104 blows per 0.3 m of penetration, indicating a dense to very dense state of compactness.

Grain size distribution testing was carried out on five samples of the sandy silt to silty sand and the results are shown on Figure B7 in Appendix B.

Atterberg limits testing was carried out on the fines portion of five samples of the sandy silt to silty sand and four of the five samples indicated a non-plastic result; with one sample measuring a liquid limit of 16%, a plastic limit of 14%, and a plasticity index of 2%, indicating a slightly plastic silt.

The natural water content measured on selected samples of the sandy silt to silty sand ranges between about 10% and 20%. A sample of the organic sandy silt encountered at Elevation 216.3 m in Borehole CN-3 has a natural water content of about 92%.

4.2.4 Silt / Clayey Silt-Silt (Till)

A deposit of silt to clayey silt-silt (till) was encountered underlying the deposit of sandy silt to silty sand in all boreholes. The deposit was encountered at a depth of 7.2 m in Boreholes CN-1 and CN-2 (Elevations 215.2 m and 214.6 m, respectively) and a depth of 8.7 m (Elevation 211.1 m) in Borehole CN-3. All boreholes were terminated within the till deposit at depths of 12.4 m to 14.0 m (Elevations 209.4 m to 207.2 m).

Within the silt to clayey silt-silt (till) deposit, the split-spoon sampler did not penetrate the entire SPT depth due to refusal conditions. The SPT 'N'-values measured in the till were typically greater than 100 blows for 0.05 m to 0.26 m of penetration suggesting a very dense to hard relative density / consistency. The effective refusal of the split-spoon sampler, observation of auger grinding in Borehole CN-3 at 10.1 m depth (Elevation 209.7 m) and

presence of gravel fragments in a sample from Borehole CN-1, suggest the presence of significant gravel content, cobbles and potentially boulders in the glacially derived till soil.

Grain size distribution testing was carried out on two samples of the silt to clayey silt-silt (till) deposit and the results are shown on Figure B8 in Appendix B.

Atterberg limits testing was carried out on three samples of the silt to clayey silt-silt (till) deposit. One sample indicates the till is non-plastic, and the other two samples had liquid limits of 14% and 18%, plastic limits of 11% and 14%, and corresponding plasticity indices of 3% and 4%. These results, which are plotted on a plasticity chart on Figure B9, indicate that the deposit ranges from non-plastic to low plasticity and supports the silt to clayey silt-silt classification.

The natural water content measured on selected samples of the silt to clayey silt-silt (till) ranges between about 7% and 15%.

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. A standpipe piezometer was installed in Borehole CN-1 and CN-3 to allow monitoring of the stabilized groundwater level at this site. The groundwater levels recorded during drilling (i.e., the unstabilized groundwater levels) and in the piezometers (i.e., the stabilized groundwater levels) are shown on the borehole records in Appendix A and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
CN-1	222.4	2.1	220.3	Nov. 17, 2021	Open borehole / inside hollow stem auger
		1.2	221.2	Dec. 23, 2021	Piezometer
		1.6	220.9	Feb. 4, 2022	Piezometer
		1.6	220.8	Feb. 8, 2022	Piezometer
		1.6	220.8	Feb. 16, 2022	Piezometer
		1.3	221.1	May 12, 2022	Piezometer
CN-2	221.8	1.6	220.2	Nov. 18, 2021	Open borehole / inside hollow stem auger
CN-3	219.8	2.4	217.4	Mar. 8, 2022	Open borehole / inside hollow stem auger
		0.8	219.0	May 12, 2022	Piezometer
		0.8	219.0	May 13, 2022	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 swampy terrain and surrounding low lying areas during the investigation suggests the groundwater level is near the native ground surface and may be influenced by the water level in the Holland River which is about 400 m east of the site.

4.4 Analytical Testing of Soil

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

Borehole No., Sample No.	pH	Resistivity (ohm- cm)	Electrical Conductivity (µmho/cm)	Soluble Chloride (µg/g)	Soluble Sulphate (µg/g)
CN-1, SA 2	7.66	5500	180	22	<20 ¹
CN-2, SA 2	7.45	3900	256	50	<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.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
METROLINX 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 Metrolinx 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 Preliminary General Arrangement (GA) and mainline / profile drawings provided by AECOM (dated April and May, 2023 respectively), twin bridge structures are proposed to carry the Bradford Bypass eastbound and westbound lanes over the Metrolinx tracks at approximately STA. 16+900. 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 preliminary structural classification of the bridge(s) is defined as “major-route” by the structural designer.

Based on Preliminary GA Drawing provided, the existing Metrolinx tracks are supported on an existing fill embankment at approximately Elevation 224 m, and the existing ground surface adjacent to the Metrolinx railway embankment is at about Elevation 223 m on the west side and lowers slightly to the east to about Elevation 222 m. The proposed Bradford Bypass highway grade is shown to be at about Elevation 234 m with proposed approach embankment heights on the order of about 11 m to 12 m above the existing ground surface at the west and east approach embankments, respectively. Based on the preliminary GA drawing, approximately 9 m to 10 m high conventional false abutment RSS walls are proposed adjacent to the Metrolinx right-of-way.

The interim design configuration for the Metrolinx overpass structure will consist of an approximately 13.6 m wide bridge (two 3.8 m wide lanes, 2.5 m and 3.0 m wide shoulders, and a 0.5 m wide concrete barrier) 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 a 3.8 m wide lane, a 3.6 m shoulder, a 0.5 m wide concrete barrier, and a 1,250 mm 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 Metrolinx overpass 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, and site access is provided.

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 the 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.

In addition to meeting the requirements of the CHBDC (2019), the overpass structures and associated foundation systems will need to be in general accordance with the latest version of the Metrolinx “*General Guidelines for Design of Railway Bridges and Structures*”.

6.3.2 Seismic Design

6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance, \bar{N}_{60} and average undrained shear strength, s_u , in combination with the energy-corrected average shear wave velocity (\bar{V}_s) measured from the seismic Cone Penetration Test (CPT) completed at the Holland River site (about 300 m east of Metrolinx site) 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 C in accordance with Table 4.1 of the CHBDC (2019).

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 C were obtained for the bridge site using the NBCC website (earthquakescanada.nrcan.gc.ca) and are summarized below.

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.0393	0.062	0.104
PGV (m/s)	0.0345	0.057	0.101
$S_a(0.2)$ (g)	0.0863	0.135	0.224
$S_a(0.5)$ (g)	0.0606	0.0942	0.156
$S_a(1.0)$ (g)	0.0328	0.0523	0.0887
$S_a(2.0)$ (g)	0.0148	0.0246	0.0429
$S_a(5.0)$ (g)	0.00348	0.00618	0.0115
$S_a(10.0)$ (g)	0.00123	0.00218	0.00401

The values provided above are for the reference ground condition Site Class C and must be 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 soft to very stiff clayey silt to clayey silt-silt soils and generally compact to dense non-cohesive deposits of sandy silt to silty sand, underlain by “100-blow” till materials comprised of silt to clayey silt-silt. Considering the compactness, consistency and liquidity 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 increases in the very loose silty sand to sandy silt cohesionless zones encountered in some of 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. The preliminary design geotechnical resistances will need to be re-evaluated using the appropriate resistance factors corresponding either a “typical” or “high” degree of site and prediction model understanding (as opposed to the current “low” degree of understanding) once further investigation data is available at the foundation elements during detail design. Foundation construction should be in general accordance with OPSS.PROV 902 (Excavating and Backfilling – Structures).

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

Shallow foundations are feasible for the abutments if founded on the lower portion of the dense to very dense sandy silt to silty sand soils (below the clayey silt seams/interlayers) or “perched” on a compacted granular pad founded on the lower compact to very dense sandy silt to silty sand; however, deep subexcavations in flowing soils with a high groundwater table makes this option less practical. Deep foundations consisting of driven steel H- or tube piles with the pile cap perched within the approach embankments is preferred from a constructability perspective, and this option will permit integral abutments and false abutment RSS walls. 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. Continuous flight auger piles are considered feasible but would not permit integral abutment design and may be assessed further during detail design.

6.5 Shallow Foundations

The near surface clayey silt-silt and very loose to compact soils containing clayey silt / silt and organic layers are considered not suitable to support shallow foundations or a granular pad (to support shallow foundations). Strip or spread footings founded on the lower portion of the native dense to very dense sandy silt to silty sand (at or below the approximate elevations identified below) may be considered for support of the structure abutments, particularly at the westbound bridge that appears to be located outside of the swampy area. The feasibility of using shallow foundations will need to be reassessed when actual structure loads, footing sizes, and subexcavation depths required to reach competent soils are confirmed during detail design.

Based on the boreholes, subexcavation of about 4 m (for the westbound bridge) to 6 m (for the eastbound bridge) below ground surface (and below corresponding groundwater level near ground surface) is required to remove the very loose to loose/very soft to soft and organic soils and reach the competent founding strata. 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:

Structure	Anticipated Founding Stratum at Bridge Locations	Founding Elevation ¹	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance ²
Westbound Bridge (Borehole CN-1)	Dense to very dense sandy silt to silty sand	218.5 m	3 m	500	300
			5 m	600	275
	Granular pad on dense to very dense sandy silt to silty sand	Min. 3 m of granular fill above El. 218.5 m	3 m	500	300
			5 m	600	275
		Min. 5 m of granular fill above 218.5 m	3 m	500	300
			5 m	600	275
Eastbound Bridge (Borehole CN-2)	Very dense sandy silt to silty sand	216 m	3 m	600	400
			5 m	700	350
	Granular pad on very dense sandy silt to silty sand	Min. 3 m of granular fill above El. 216 m	3 m	500	300
			5 m	600	275
		Min. 5 m of granular fill above El. 216 m	3 m	500	300
			5 m	600	275

Notes:

1. Subexcavation to about 4 m (for the westbound bridge) and 6 m (for the eastbound bridge) and below groundwater is required to remove unsuitable soils to a competent founding stratum. All design values assume similar vertical stress at founding level.
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, thickness of granular pad (as applicable) and surrounding grade 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, $\tan \delta$, between the cast-in-place concrete footings and the dense to very dense sandy silt to silty sand 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, $\tan \delta$, between the cast-in-place concrete footings and a Granular 'A' pad may be taken as 33° and 0.65, respectively.

6.6 Deep Foundations

6.6.1 Steel H-Pile or Tube Foundations

Driven steel H-piles founded within the "100-blow" till deposit of silt to clayey silt-silt are considered feasible for support of the new abutments. The "100-blow" till deposit was encountered between depths of about 7 m and 9 m below the existing ground surface.

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 and pose a lower risk of "hanging up" or being deflected from their vertical or battered orientation during

installation if cobbles or boulders are present. The presence of potential pockets of gravel, cobbles and/or boulders should be anticipated in the glacially derived silt to clayey silt-silt till deposit.

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 (with corresponding pile tip elevations, based on the elevation of the “100-blow” material encountered in the boreholes) for the bridges is provided below for preliminary design purposes.

Pile Type	Estimated Pile Tip Elevation ¹ (m)	Approximate Pile Length ²	Soil Strata Near Pile Tip Elevation	Factored Ultimate Geotechnical Resistance ³	Factored Serviceability Geotechnical Resistance (for 25 mm of Settlement)
HP 310x110	213 m	7 – 8 m	“100-blow” silt to clayey silt-silt (till)	1,100	>1,100
HP 360x108				1,300	>1,300
324 mm dia. tube pile (min. 9.5 mm thick)				850	>850
406 mm dia. tube pile (min. 9.5 mm thick)				1,350	>1,350

Notes:

1. Assuming piles are driven approximately 1.5 m to 2 m into “100-blow” till material (based on borehole CN-1 and CN-2).
2. Measured from existing ground surface.
3. Resistance values assume single pile and do not consider pile group efficiency.

The estimated factored ultimate geotechnical resistance is calculated on both shaft and tip resistances, but predominantly tip 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.

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

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

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

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

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

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

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

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

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

6.6.2 Drilled Shafts (Caissons)

Drilled shafts (caissons) are also considered feasible for supporting the bridge structure abutments. The following axial geotechnical resistances may be used for preliminary design of the caissons:

Caisson Diameter	Estimated Caisson Base Elevation ¹	Approximate Caisson Length ²	Soil Strata near Caisson Base Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement ³
0.9 m	213 m	7 – 8 m	“100-blow” silt to clayey silt-silt (till)	1,800	>1,800
1.2 m				3,200	>3,200
1.5 m				5,000	>5,000

Notes:

1. Assuming caissons are founded approximately 1.5 m to 2 m into “100-blow” till material (based on borehole CN-1 and CN-2).
2. Measured from existing ground surface.
3. 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 or casing (at least within the upper zone) is expected to be required 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. If a permanent liner is used, the design geotechnical resistance provided above may need to be revised to account for the reduced adhesion between the liner material and surrounding soil along the length of the liner compared to the adhesion between concrete and surrounding soil if temporary liners are used. Specialized construction techniques would be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (polymer slurry) within the open hole and 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 predominantly an 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 base of the drilled shaft. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection device (SID) or a shaft quantitative inspection device (SQUID); because of the likelihood of water with entrained sediment or the presence of polymer slurry, it is recommended that SQUID testing be specified on some or all caissons to demonstrate and verify base cleaning. Should the inspection indicate that loosened or compressible material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

Alternatively, a design based solely on shaft friction may be considered provided the design geotechnical resistances are reduced accordingly and appropriate quality assurance procedures are adopted by the design-builder / contractor. The consistency and characteristics of the drilling slurry (particularly if a bentonite slurry is used) will have an impact on the design geotechnical resistances and this will need to be considered during detail design.

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

- Advanced site-specific investigation during detail design to confirm or adjust axial geotechnical resistances for design based on the use of a typical or high rather than low 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) and MTO's recent special provision should be included in the Design-Build output specifications to address the requirements for supply and installation of drilled shafts (caissons) including the use of temporary or permanent liners/casings and slurry, the placement of concrete by tremie methods, cleaning and inspection of the shafts and base of the drilled shafts as

applicable, and quality control testing. Non-destructive post-construction testing in selected drilled shafts should also be included in the output specification and is recommended to verify the integrity of the concrete given the groundwater conditions, presence of saturated cohesionless soils, and specialized installation methods to counterbalance the hydrostatic pressures.

6.6.3 Continuous Flight Auger Piles

As an alternative to drilled shafts (caissons), Continuous Flight Auger (CFA) piles could be considered. CFA piles are formed by drilling into the ground with hollow-stem continuous flight augers to the target elevation or stratum. Contrary to the installation of drilled shafts (caissons), no casing and/or slurry is required to keep the hole open and the augers are screwed into the soil such that the augers are in direct contact with the surrounding soil (without generating excessive spoils) during the initial drilling process. When the target depth is reached, the auger is withdrawn from the hole while simultaneously pumping concrete or a sand/cement grout mix through the hollow centre of the auger pipe to the base of the auger. Simultaneous pumping of the grout/concrete and withdrawing of the auger provide continuous support of the hole and spoils either removed at the ground surface or displaced into the sides of the hole. Steel reinforcement is placed into the concrete/grout filled hole immediately after the augers are completely removed.

CFA piles are typically designed with diameters ranging from 0.3 m to 0.6 m and lengths up to about 30 m. They are considered to be an intermediate deep foundation option.

Advantages of CFA piles are the potential relatively quick drilling process and cast-in-place method which leads to high production rates and relatively low noise and vibration levels. Disadvantages of CFA piles are the inability to inspect the shaft and base of the piles which leads to a higher level of quality control / assurance during and post installation to verify structural integrity of the piles and confirm the design geotechnical resistance has been achieved. The risk of excessive flighting of soil (i.e., when the auger is rotated too much in proportion to the penetration into the soil, such that too much soil is flighted towards the surface and the auger flights do not maintain adequate soil to provide lateral support to the hole) would need to be addressed via static load testing and/or performance specifications.

For preliminary assessment, a 0.4 m to 0.6 m diameter CFA pile, approximately 7 m to 8 m long and founded within the 100-blow till soils (tip at Elevation 213 m) is estimated to have a factored geotechnical resistance at Ultimate Limit State ranging from about 1,400 kN to 1,600 kN. The factored geotechnical resistance at Serviceability Limit State will depend on installation methods and interpretation of pre-production pile load tests.

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 use of a typical or high rather than low 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).

CFA installation should be in general accordance with OPSS.PROV 903 (Deep Foundations), as applicable, and a special provision will need to be prepared by the design-builder or contractor (and accepted by MTO) to address the requirements for supply and installation of CFA piles including a detailed work method to prevent overexcavation, ensure proper placement of concrete / grout and prevention of heave / loosening of soils at the base and collapse of soils along the shaft during auger removal, and quality control testing. Non-destructive

post-construction pile integrity testing in selected CFA piles should be included in the future contract specifications and is recommended to verify the integrity of the concrete / grout given the installation method, groundwater conditions, presence of saturated cohesionless soils, and specialized installation methods to counterbalance the hydrostatic pressures. Pre-production and production pile load testing will be required (e.g., pile load (proof) tests combined with high strain dynamic testing) prior to finalizing design and during construction.

6.6.4 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:

$$k_h = \frac{n_h z}{B} \quad \text{Where } n_h \text{ is the constant of subgrade reaction (kPa/m);}$$

$$z \text{ is the depth (m); and}$$

$$B \text{ is the pile / caisson diameter or width (m).}$$

For cohesive soils:

$$k_h = \frac{67S_u}{B} \quad \text{Where } S_u \text{ is the undrained shear strength of the soil (kPa); and}$$

$$B \text{ is the pile / caisson diameter or width (m).}$$

Considering the subgrade reaction equations provided above model linear behaviour, they are only considered appropriate where the maximum pile deflections are small (less than 1% of the pile/caisson diameter), where the loading is static (no cycling) and where the pile/caisson material is linear.

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 the ground surface.

Stratigraphic Unit	n_h (kPa/m)	S_u (kPa)
Existing Fill New Granular Fill (New Granular 'A' / 'B' Type II)	5,000 – 7,000 40,000 – 50,000	-
Loose sand within CSP (if applicable)	1,500 – 2,500	
Clayey Silt to Clayey Silt-Silt (very soft to soft)	-	12 - 25
Clayey Silt to Clayey Silt-Silt (stiff to very stiff)	-	50 - 100
Sandy Silt to Silty Sand (very loose to compact)	1,300 – 3,400	-
Sandy Silt to Silty Sand (dense to very dense)	20,000 – 34,000	
Silt to Clayey Silt-Silt ("100-blow" till) ²	34,000 – 61,000	200

Notes:

1. Although parameters are provided for the full depth of the soil stratigraphy, lateral resistance in the upper 1.5 m should be neglected to account for frost action.

2. Where both n_h and S_u parameters are provided, the structural assessment should be completed for both undrained and drained conditions, and the selected design should be based on the more conservative approach.

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).

6.6.5 Downdrag Loads on Piles or Caissons

Based on the preliminary profile drawings, the approach embankments at the Metrolinx overpass structures are up to about 12 m high with total settlements in the foundation soils estimated to range from 100 mm to 150 mm (see section 6.8.2), generally most of which would be considered immediate settlement if subexcavation of the softer zones of cohesive soils (which are up to about 0.7 m to 1.4 m thick and extend to depths of 1.4 m to 2.9 m below ground surface) are carried out prior to pile installation. As a result, downdrag loads are not anticipated to be a major concern, although this must be assessed further during detailed design. Downdrag loads can likely be mitigated by designing piles / caissons to resist the additional load in the structural design and/or reducing downdrag forces by subexcavating compressible soils and/or preloading the foundation soil to induce settlements prior to driving piles or installing caissons.

6.7 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*).

6.8 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 11 m (at the west approach embankments) to 12 m (at the east approach embankments) above the existing ground surface. A 2 m wide mid-height bench should be

incorporated into the design of the embankment slopes, as required for embankment heights greater than 8 m in height in accordance with OPSD 202.010 (*Slope Flattening*).

For preliminary design, it is assumed that prior to construction of the new approach embankments, all topsoil, peat/organics, fills (although not encountered in boreholes CN-1 and CN-2 during the current investigation but anticipated based on the swampy areas within the footprint of the embankments) and near surface very soft to soft / very loose soils (up to about 1.5 m below ground surface) will be stripped from the footprint of the new embankments and replaced with suitable granular fill.

6.8.1 Global Stability

For assessment of global stability, minimum Factors of Safety³ of 1.4 (for the temporary, short term condition) and 1.6 (for the permanent, long-term condition) have been used for the preliminary design for the new approach embankments, given the limited geotechnical information at the site and as per Table 6.2 of CHBDC (2019) and MERO (2020).

The simplified stratigraphy and associated soil parameters employed for the stability analyses are shown in the table in Section 6.8.2. Both undrained (total stress) and drained (effective stress) stability assessment was performed and the results indicate that the minimum target Factors of Safety were achieved for temporary and permanent conditions for embankments with side slopes inclined at 2H:1V and founded on the compact to very dense sandy silt to silty sand soils.

Shallower side-slopes (on the order of 2.5H:1V) or toe berms may be required if very loose and/or softer soils or organic layers (such as those encountered in Borehole CN-3) are confirmed to be present at depth within the embankment footprint (specifically at the eastbound bridge embankments located in the swampy area). Alternatively, subexcavation and replacement of the unsuitable soils (estimated to be up to 4 m below ground surface based on CN-3) or ground improvement measures (e.g. installation of geopiers or staged construction to allow for strength gain and pore pressure dissipation in clayey soils) may be considered to improve global stability of the approach embankments. 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.

To reduce surface water erosion on the granular embankment side slopes, vegetative cover should be established as per OPSS.PROV 803.

6.8.2 Settlement

To estimate the magnitude of settlement of the foundation soils as a result of the proposed new embankments, analyses were carried out near the abutment locations using the commercially available computer program Settle 3 (Version 5.012) from Rocscience Inc, supplemented with hand calculations. 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:

³ The Factor of Safety (FoS) is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} (i.e. $FoS = 1/(\Psi \cdot \phi_{gu})$).

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

The compression of the non-cohesive soil deposits (containing clayey silt seams/ layers in the upper zone) were modelled using established correlations based on SPT "N"-values as presented in Bowles (1984) and by Kulhawy and Mayne (1990), the results of the consolidation test, and engineering judgement from experience with similar soils in this region of Ontario. As previously mentioned, the surficial cohesive deposits of clayey silt to clayey silt-silt extending to depths of up to 1.5 m were not incorporated into the model, as it is assumed these softer soils would be stripped prior to embankment construction.

The foundation engineering parameters used in the settlement analyses are summarized below. The groundwater elevation is assumed to be at the ground surface.

Idealized Stratigraphic Unit	γ (kN/m ³)	ϕ' (°)	S_u (kPa)	E' (MPa)
New Granular Fill (Granular 'A' or 'B' Type II)	21	36	--	--
Sandy Silt to Silty Sand (Very Loose to Compact) and containing clayey silt seams/interlayers	19	28	--	5 – 10
Sandy Silt to Silty Sand (Dense to Very Dense)	20	34	--	50 – 75
Silt to Clayey Silt-Silt ("100-Blow" Till)	21	38	200	200 – 250

For a proposed maximum embankment height of 12 m, the estimated magnitude of total settlement for the westbound and eastbound bridge approach embankments are expected to be on the order of about 100 mm to 150 mm, assuming the use of conventional granular fill for construction and subexcavation of the softer cohesive soils encountered at the site.

Since the majority of the soils encountered during the investigation are non-cohesive or contain relatively thin seams/interlayers of cohesive soil (apart from the "100-blow" till material), the above-noted total settlements are expected to occur immediately or shortly after construction of the embankments. Thus, the estimated post-construction settlement over a 20-year period is expected to be less than 25 mm. The estimated settlements do not account for immediate settlement of the embankment fill itself which is expected to occur during or shortly after construction (within a few months) and would need to be assessed during detail design.

Based on the preliminary investigation and calculated results from the settlement model, post-construction settlements are estimated to be within tolerable values and are not anticipated to be a concern for the approach embankments.

The estimated settlement of the Metrolinx railway and any associated utilities within the Metrolinx right-of-way due to embankment loading will need to be checked during detail design and must be in general accordance with the threshold values provided by Metrolinx (see Section 6.11.6).

6.9 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).

6.10 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete and/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 the soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. The analytical results for the soil samples submitted for testing at this site are summarized in Section 4.4 and the analytical laboratory test reports are included in Appendix C.

6.10.1 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”*) for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S3 (Moderate). Therefore, based on the soil samples tested, when the designer is selecting the exposure class for the structures, the effects of sulphates may not need to be considered.

6.10.2 Potential for Corrosion

The test results indicate a pH of 7.5 to 7.7 and a resistivity of 3,900 to 5,500 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability, and the resistivity indicates that the soil corrosiveness is generally Low to Moderate ($10,000 \text{ ohm-cm} > R > 2000 \text{ ohm-cm}$), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014). Appropriate corrosion protection should be applied to the foundation element / materials and given that the foundations are located adjacent to the highway shoulder / ditches and will be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class for any concrete as defined by CSA A23.1 Table 1.

These recommendations are provided as guidance only; the 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 and determine the appropriate exposure class and ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are satisfied.

6.11 Construction Considerations

6.11.1 Subgrade Preparation and Approach Embankment Construction

Prior to construction of the new approach embankments, it is recommended that all unsuitable soils such as topsoil or organics, and existing surficial fill materials or loosened/softened soils be stripped from the embankment footprint and replaced with OPSS Select Subgrade Material (SSM), Granular A or Granular B material. Based on the boreholes closest to the bridges (borehole CN-1 and CN-2), stripping of about 1.5 m below native ground surface (and below groundwater level) may be required to remove the unsuitable soils at the approach embankments. It is anticipated that thicker unsuitable soils may be encountered in the swampy areas located near the approach embankments at the eastbound bridge, which was not investigated during the current investigation. In borehole CN-3, located about 200 m south but on the east side of the Metrolinx tracks, an organic layer was encountered about 4 m below ground surface.

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

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

6.11.2 Temporary Excavations

In general, temporary excavations up to about 4 m to 6 m are required for shallow foundations and/or subexcavation and replacement with a granular pad. Temporary excavations can be reduced to about 1.5 m below ground surface (i.e. frost depth) for stripping below approach embankments and/or eliminated for pile and caisson caps “perched” within the approach embankments, as applicable.

All temporary excavations must be carried out in accordance with OPSS.PROV 902 (*Excavating and Backfilling*) and Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHS), as amended. For excavation depths of 4 m to 6 m, the very soft to soft clayey silt to clayey silt-silt and very loose sandy silt to silty sand, organic soils, and all soils below the groundwater level that aren't effectively dewatered, are classified as Type 4 soils. The firm to very stiff clayey silt to clayey silt-silt and loose to compact sandy silt to silty sand above the groundwater level are classified as Type 3 soils. As per OHS, temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes of no steeper than 3H:1V in Type 4 soils and 1H:1V in Type 3 soils.

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 generally meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation. Given the proximity of the Metrolinx railway / embankment to the proposed structure foundations, subexcavation for shallow foundations will likely require temporary shoring. Any excavations or crossings within or near the Metrolinx right-of-way will be subject to Metrolinx requirements and permits, and the design-builder is encouraged to engage Metrolinx in early planning and design of any temporary and permanent structures at this site.

6.11.3 Groundwater / Surface Water Control

The highest groundwater level measured in the standpipe piezometers was about 0.8 m below ground surface and was up to about Elevation 221.2 m. In general, the water level appears to slope downward from west to east, from about Elevation 221.2 (1.2 m below ground surface) in Borehole CN-1 at the west side to about Elevation 219.0 m (0.8 m below ground surface) in Borehole CN-3 at the east side.

The excavations for shallow foundations (if considered) are anticipated to extend about 4 m to 6 m below ground surface and will be about 4 m below the measured groundwater levels. Depending on the extent of the excavation and groundwater level at the time of construction, dewatering operations would likely require wells and/or well points supplemented, as necessary, by pumping from properly filtered sumps located within the excavations. The groundwater level would need to be drawn down at least 0.5 m beneath the base of the excavation until the excavation has been backfilled to at least 0.5 m above the static groundwater level. 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 in the swampy areas, 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. In particular, surface water drainage on the west (i.e., high) side of the site must be properly diverted / controlled such that the integrity of any foundation subgrade is maintained. In addition, any standing / ponded water on the east side must be unwatered / drained to allow for construction in the dry.

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.

6.11.4 Temporary Access Routes

Temporary roads / platforms to access and construct the abutment foundations, embankments and provide laydown areas will be required for construction and must be carefully designed within the swampy / low-lying areas (particularly to the east side of the Metrolinx rail embankment). Additional stripping requirements may be required and stability and settlement of the access roads and/or temporary crane platforms will need to be carried out during detail design.

6.11.5 Obstructions during Pile Driving / Caisson Installation

During pile installation through the glacially derived soils, especially the “100-blow” silt to clayey silt-silt (till) deposit at this site, 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. Pre-augering may be considered to reduce the risk of piles “hanging up” or deflecting on potential “100-blow” stratum and if considered, the design geotechnical resistances provided must be reviewed and revised as necessary during detail design. Caisson installation equipment must be capable of penetrating and/or removing obstructions as required. The feasibility of CFA piles will need to be re-evaluated during detail design if significant obstructions are anticipated based on the results of additional investigation at the site.

6.11.6 Vibration / Settlement Monitoring During Construction

Vibration monitoring should be carried out during pile driving, installation of any protection systems, and during operation of vibratory compaction equipment to check that the vibration levels at nearby structures / utilities and on the rail lines are maintained below tolerable levels. Vibrations are not considered to be a major concern if CFA piles or caissons are used for foundations, although the presence of obstructions will increase the potential for higher relative vibration levels.

Commercial buildings are anticipated to be located more than 125 m from the proposed abutment locations. A Peak Particle Velocity (PPV) threshold of 25 mm/s to 50 mm/s is generally considered applicable for vibration impacts on commercial buildings and it is considered unlikely that vibrations induced by conventional construction activities such as pile driving and protection system installation will reach this threshold level. Vibration monitoring on the Metrolinx rail tracks and any associated utilities within the Metrolinx right-of-way will be required during construction in accordance with Metrolinx requirements. In general, a PPV threshold of 25 mm/s and 10 mm/s is

recommended for the rails and any sensitive utilities although the threshold values will need to be checked with Metrolinx.

At this preliminary stage, pre- and post-construction condition surveys and vibration monitoring are recommended at the rails and any associated structures / utilities located within a 100 m radius of any piling operations, and it would be prudent to carry out such monitoring during critical stages of the construction, such as during pile driving operations and during installation of any temporary shoring. For due diligence purposes, supplemental settlement / ground movement monitoring should also be considered where structures / utilities are assessed to have a low tolerance to movement and within the Metrolinx right-of-way as outlined in the “*General Guidelines for Design of Railway Bridges and Structures*” (Metrolinx, 2017).

A special provision which describes the requirements for vibration and settlement monitoring on the adjacent rails and associated structures / utilities is recommended to be included in the future contract documents during detail design.

6.12 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 structures and one borehole located about 200 m south of the site. 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 east side of the Metrolinx railway embankment and to confirm the subsurface soil and groundwater conditions on the west side of the Metrolinx railway embankment, at the location of the bridge foundation elements, approach embankments, and any associated retaining walls. In particular, the foundation soils need to be investigated within the swampy areas which are within the entire footprint of the eastbound bridge and the east side of the westbound bridge. Specialized drilling equipment (portable equipment) and/or construction of temporary access platforms will be required for the foundation investigation within and near the proposed abutment and approach embankment locations.

After more detailed foundation investigation is complete and design details have been finalized (particularly regarding the use of conventional false abutment RSS walls), 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 will need to be reassessed. The use of GSC 5th Generation or 6th Generation seismic hazard maps to define the Site Class should be confirmed for detail design. 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. Additionally, provided that sufficient investigation is carried out, the geotechnical resistances should be re-evaluated with factors associated with a minimum “typical degree of site and prediction model understanding.”

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 piezometers (installed in Boreholes CN-1 and CN-3) should be maintained operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometers 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.

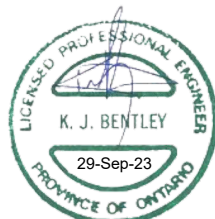
7.0 CLOSURE

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.

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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.

Ontario Provisional Standard Drawing:

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

Ontario Provincial Standard Specifications (OPSS)

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

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

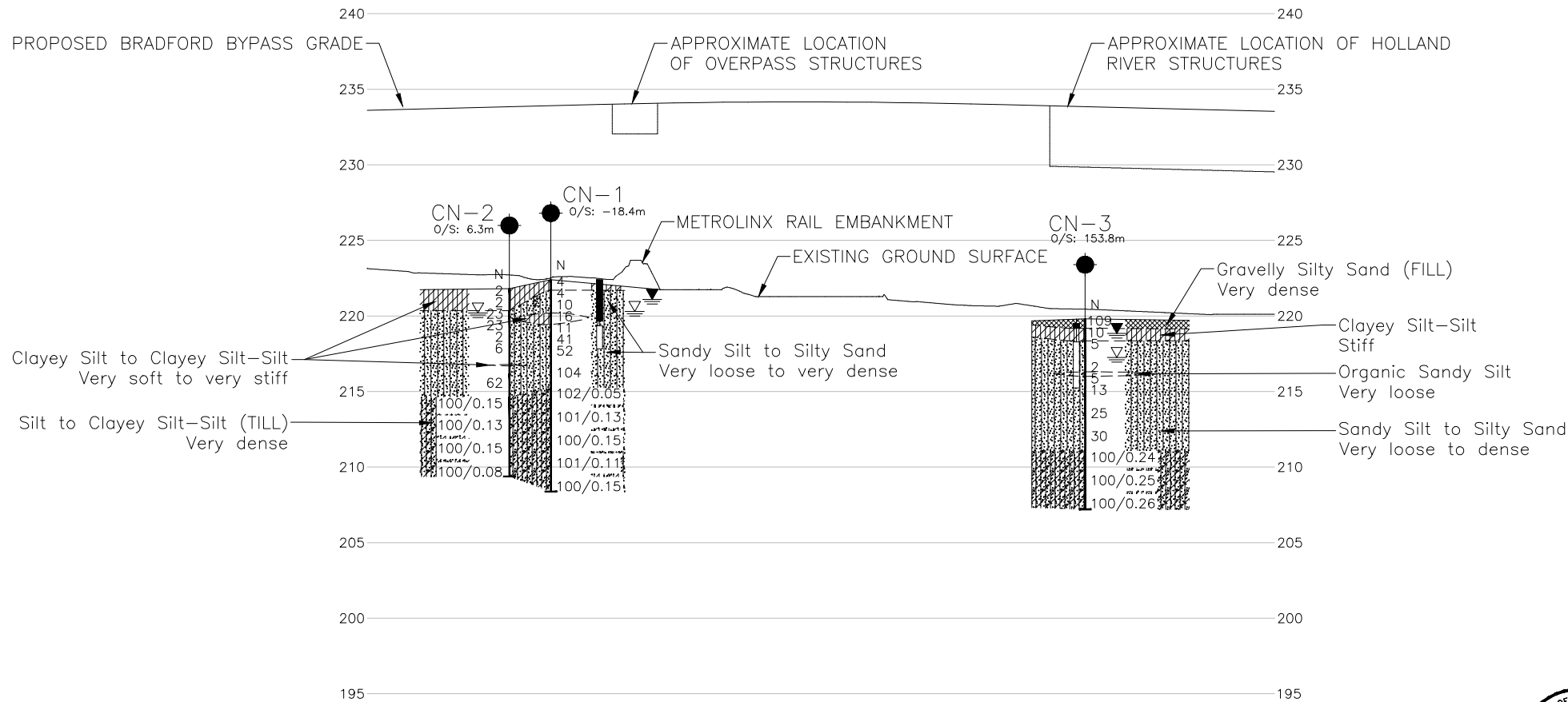
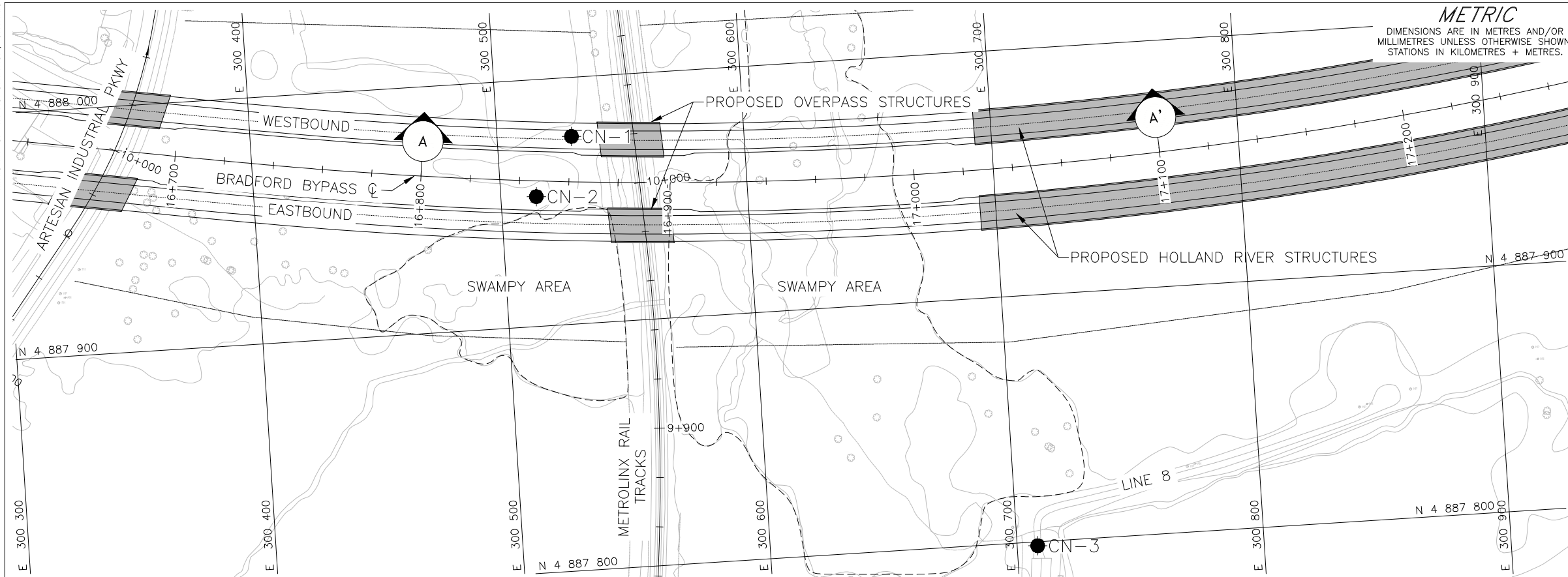
TABLES

Table 1: Comparison of Foundation Alternatives – Metrolinx Overpass

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Strip / Spread footings founded on native compact to very dense sandy silt to silty sand	Marginally Feasible at eastbound bridge Feasible at westbound bridge	<ul style="list-style-type: none">■ Conventional construction■ Founding soils provide adequate geotechnical resistance.	<ul style="list-style-type: none">■ Excavation of unsuitable soils to about 4 m to 6 m depth is required to reach competent founding stratum.■ Extensive dewatering efforts 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 depending on proximity of footings to Metrolinx rail line and to limit extents of excavation.■ Does not allow for conventional integral abutment design.	<ul style="list-style-type: none">■ Costs may be comparable or higher than deep foundation options when additional costs for dewatering and temporary protection systems are considered.	<ul style="list-style-type: none">■ Less competent soils (possibly containing compressible interlayers) 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 and adjacent Metrolinx railway embankment / railway if adequate dewatering and shoring is not provided in saturated silts and sands.■ Anticipated flowing sands and silts make this option not practical for eastbound bridge (depths up to 6 m bgs), and may not be practical for westbound bridge after investigating east side during detail design.
"Perched" abutment spread footings founded on a compacted granular pad within approach embankments	Marginally Feasible at eastbound bridge Feasible at westbound bridge	<ul style="list-style-type: none">■ Conventional construction■ Granular pad can be constructed within approach embankment for abutment locations.■ Founding level can easily be adjusted within approach embankment.■ Engineered fill can be placed and compacted to reduce depth of shallow foundation and reduce construction efforts and volume of concrete / steel used for foundations.	<ul style="list-style-type: none">■ Excavation of unsuitable soils to about 4 m to 6 m depth is required to reach competent founding stratum.■ Extensive 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 depending on proximity of footings to Metrolinx rail line and to limit extents of excavation.■ Does not allow for conventional integral abutment design.■ Geotechnical resistances may decrease compared to spread footings founded deeper on native soils.	<ul style="list-style-type: none">■ Costs may be comparable to spread footings on native soil option given same additional cost for dewatering and temporary protection system, and will depend on balance between increased volume of engineered fill vs. reduced volume of concrete / steel.	<ul style="list-style-type: none">■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual abutment locations; although lower risk of geotechnical resistances being influenced as much as spread footings directly on native.■ Lower risk of deeper excavation and increased dewatering and/or temporary shoring efforts compared to spread footings directly on native soils.■ 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; however, similar risk to disturbing adjacent Metrolinx railway embankment / railway if adequate dewatering and shoring is not provided.■ Anticipated flowing sands and silts make this option not practical for eastbound bridge (depths up to 6 m bgs), and may not be practical for westbound bridge after investigating east side during detail design.
Steel H-piles or tube piles driven into "100-blow" till soils	Feasible at both bridge locations	<ul style="list-style-type: none">■ Conventional construction methods for driven steel pile foundations.■ High axial resistances available■ Pile caps perched within approach embankments can be considered to reduce or eliminate dewatering / subexcavation requirements.■ Allows for integral abutment design and conventional RSS abutment walls.	<ul style="list-style-type: none">■ Noise and vibrations to adjacent properties, although primarily limited to industrial properties.■ Dewatering measures may be required at abutments for the construction of pile caps, unless perched in embankment fill at abutments.	<ul style="list-style-type: none">■ Lower relative cost than drilled shafts (caissons)■ Comparable cost to spread footings considering dewatering and subexcavation of unsuitable soils can be reduced / eliminated, especially with perched pile caps.	<ul style="list-style-type: none">■ 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 in "100-blow" till soils	Feasible at both bridge locations	<ul style="list-style-type: none">■ 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	<ul style="list-style-type: none">■ 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.	<ul style="list-style-type: none">■ Higher relative cost than pile foundations.■ Comparable or slightly higher cost than shallow foundations but reduced dewatering / subexcavation costs if pier caissons are cast continuously with structural columns to eliminate pile cap.	<ul style="list-style-type: none">■ 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.

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
		with structural columns to the underside of the superstructure.	<ul style="list-style-type: none">■ Does not allow for conventional integral abutment design.		
Continuous Flight Auger (CFA) piles founded within "100-blow" till soils.	Feasible at both bridge locations	<ul style="list-style-type: none">■ Offers intermediate geotechnical resistance that is typically higher than driven piles and lower than drilled shafts (caissons).■ Lower noise and lower vibrations compared to driven piles	<ul style="list-style-type: none">■ Special measures and high level of quality control required to reduce potential for soil overexcavation (excessive "flighting" of soil), counterbalance high groundwater pressures to reduce risk of loosening / softening of the sides of the CFA pile and heave / blow-out at base of pile upon completion of drilling and start of concrete / grout placement (through hollow stem).■ Generation and disposal of soil cuttings during auger removal.■ Does not allow for conventional integral abutment design.■ Not a standard installation method for MTO projects, although other experience in Ontario and across U.S. and Europe is well documented.	<ul style="list-style-type: none">■ Less expensive than caissons and comparable to driven piles, although increased quality control testing and pile verification will increase costs.	<ul style="list-style-type: none">■ Reduced impact on design if variable near surface soils and utilities are encountered during detailed investigation.■ Risk of difficulties and/or augers may not be capable of penetrating through soil deposit that may contain pockets of gravel or cobbles and boulders (to be confirmed during detail design).■ Inability to inspect the shaft and base of the CFA piles due to installation method and concrete placement methods requires additional pile load testing and pile integrity testing.■ Higher risk of installation challenges and overexcavation of soil (especially in very loose to loose sands) compared to other options. Higher level of quality control / quality assurance and development of special provision to be accepted by MTO is required.

DRAWINGS



A-A' PROFILE BRADFORD BYPASS CL

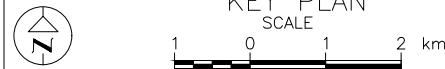
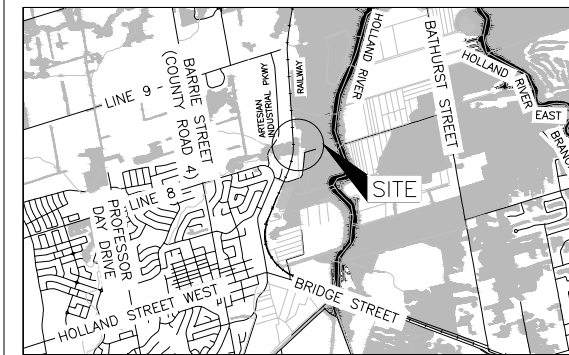


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METROLINX RAIL OVERPASS
BOREHOLE LOCATIONS AND SOIL
STRATA

SHEET



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 May 12 and 13, 2022
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
CN-1	222.4	4887976.1	300530.2
CN-2	221.8	4887952.6	300514.4
CN-3	219.8	4887798.4	300708.3

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.



NO.	DATE	BY	REVISION
Geocres No. 31D00-819			
HWY.		PROJECT NO. 19136074	DIST.
SUBM'D. KJB	CHKD. MH	DATE: 09/26/2023	SITE:
DRAWN: DD	CHKD.	APPD.	DWG. 1

APPENDIX A

Borehole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS^{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 cm² 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, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO 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

Compactness¹

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 grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

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

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal 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

(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
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(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 $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Notes: 1
2

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

PROJECT 19136074

RECORD OF BOREHOLE No. CN-1

Sheet 1 of 2

METRIC

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

LOCATION N 4887976.1; E 300530.2 NAD83 / MTM Zone 10 (LAT. 44.131768; LONG. -79.553359)

ORIGINATED BY MTI

DIST Central HWY BBP - Metrolinx

BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary

COMPILED BY MA/MTI

DATUM CGVD28 Surface Elevation:222.4 m

DATE Nov 17, 2021

CHECKED BY MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m ³	GR SA SI CL				REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W _p	NMC W	LL W _L						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
0.0	CLAYEY SILT (CL), trace sand, trace organics, trace gravel, Soft Dark Brown Moist		1	SS	4		222	20	40	60	80	100	20	40	60						
221.7																					
0.7	SILTY SAND (SM) to Sandy SILT (ML), trace organics, trace gravel Loose to compact Brown, contains oxidation staining Moist		2	SS	4		221														
			3	SS	10																
220.2																					
2.2	CLAYEY SILT (CL), some sand, trace organics, trace gravel Very stiff Brown, contains oxidation staining Moist		4	SS	16		220														
219.4																					
3.0	Sandy SILT (ML) to SILTY SAND (SM), trace to some clay, trace to some gravel Compact to very dense Light brown to grey, contains oxidation staining Moist to wet		5	SS	11		219														
			6	SS	41		218														
			7A	SS	52																
			7B	SS																	
							217														
			8	SS	104		216														
215.2																					
7.2	SILT (ML) to CLAYEY SILT-SILT (CL-ML), some sand, trace gravel, (TILL) Very Dense Grey Moist		9	SS	102/0.05		215														
							214														
	- 9.1 to 9.4 m: gravel fragments encountered in sample		10	SS	101/0.13		213														

Continued on Next Page

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

PROJECT	19136074		RECORD OF BOREHOLE		No. CN-1	Sheet 2 of 2	METRIC
G.W.P.	Assignment No 2019-E-0048		LOCATION	N 4887976.1; E 300530.2 NAD83 / MTM Zone 10 (LAT. 44.131768; LONG. -79.553359)			ORIGINATED BY MTI
DIST	Central	HWY BBP - Metrolinx	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary			COMPILED BY MA/MTI
DATUM	CGVD28 Surface Elevation:222.4 m		DATE	Nov 17, 2021			CHECKED BY MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100	W _p	W						
							○	⊗	×												
							20	40	60	80	100										

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

PROJECT 19136074

RECORD OF BOREHOLE No. CN-2

Sheet 1 of 2

METRIC

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

LOCATION N 4887952.6; E 300514.4 NAD83 / MTM Zone 10 (LAT. 44.131557; LONG. -79.553556)

ORIGINATED BY MTI

DIST Central HWY BBP - Metrolinx

BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary

COMPILED BY MA/MTI

DATUM CGVD28 Surface Elevation:221.8 m

DATE Nov 18, 2021 - Nov 19, 2021


CHECKED BY MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W _p	NMC W	LL W _L		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100					20 40 60								
0.0	CLAYEY SILT (CL), some sand to sandy, trace organics, trace gravel, Very soft to soft Brown Moist		1	SS	2		221														
			2	SS	2																
220.4																					
1.4	Sandy SILT (ML) to SILT (ML), trace organics, trace to some gravel, trace to some clay, contains clayey silt seams/layers Very Loose to Compact Grey Moist to Wet - 2.3 to 2.9 m: No sample recovery - 3.0 to 3.5 m: Clayey Silt layer encountered - 3.0 to 3.7 m: Clayey silt seams/layers and gravel fragments encountered - 3.8 to 4.4 m: Clayey silt-silt seam and trace organics - 4.6 to 5.2 m: Shelby tube sample taken.		3	SS	23		220								NP						
			4	SS	23		219														
			5	SS	2		218														
			6	SS	6																
			7	TO			217								NP						
216.8																					
5.0	Sandy CLAYEY SILT (CL)																				
216.7	Sandy SILT (ML) to SILT (ML), trace gravel Grey Moist																				
5.1																					
216.2																					
5.6	SILTY SAND (SM), trace gravel Very dense Grey Moist		8	SS	62		216														
							215														
214.6																					
7.2	CLAYEY SILT-SILT (CL-ML) to SILT (ML), some sand to sandy, trace gravel (TILL) Very Dense Grey Moist to wet - 9.1 to 9.3 m: Split spoon bouncing		9	SS	100/0.15		214														
							213														
			10	SS	100/0.13		212														

Continued on Next Page


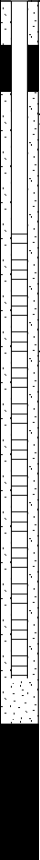
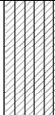
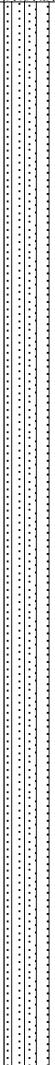

+³, x³ : Numbers refer to Sensitivity o³⁰% STRAIN AT FAILURE

PROJECT	19136074	RECORD OF BOREHOLE	No. CN-2	Sheet 2 of 2	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887952.6; E 300514.4 NAD83 / MTM Zone 10 (LAT. 44.131557; LONG. -79.553556)	ORIGINATED BY	MTI
DIST	Central	HWY	BBP - Metrolinx	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary
DATUM	CGVD28 Surface Elevation:221.8 m	DATE	Nov 18, 2021 - Nov 19, 2021	COMPILED BY	MA/MTI
				CHECKED BY	MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR	SA	SI	CL	REMARKS			
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL									
								Field Vane	Remoulded	Pocket Pen	Quick Triaxial	Unconfined	W _p	W	W _L									
							20	40	60	80	100	NP Nonplastic -----○----- 20 40 60												
209.4 12.4	CLAYEY SILT-SILT (CL-ML) to SILT (ML), some sand to sandy, trace gravel (TILL) Very Dense Grey Moist to wet - 10.7 to 10.9 m: Hard clayey silt layer		11	SS	100/0.15		211								○				1	18	62	19		
								210																
			12	SS	100/0.08												QH							
209.4 12.4	End of Borehole Note: 1. Hollow stem augers to 3.0 m (Elev. 218.8 m) and then switched to mud rotary. 2. Water level measured at a depth of 1.55 m during drilling and prior to mud rotary.						209																	
							208																	
							207																	
							206																	
							205																	
							204																	
							203																	
						202																		

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

PROJECT	19136074	RECORD OF BOREHOLE	No. CN-3	Sheet 1 of 2	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887798.4; E 300708.3 NAD83 / MTM Zone 10 (LAT. 44.130169; LONG. -79.551132)	ORIGINATED BY	MTI
DIST	Central HWY BBP - M Rail	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:219.8 m	DATE	Mar 08, 2022 - Mar 09, 2022	CHECKED BY	MH

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined					W _p	W	W _L						
													NP Nonplastic								
0.0	Gravelly SILTY SAND (SM), trace rootlets, (FILL) Very Dense Brown Moist		1	SS	109																
219.2							219														
0.6	CLAYEY SILT-SILT (CL-ML), trace sand, trace gravel Stiff Grey Moist		2	SS	10																
218.4																					
1.4	Sandy SILT (ML) to SILTY SAND (SM), trace to some clay, trace to some gravel, trace to some rootlets/ organics, contains clayey silt seams/layers Very loose to dense Dark brown to grey Moist		3	SS	5		218														
	- 3.5 to 3.8 m: Organic silt layer encountered		4A	SS	2												4	37	42 17		
			4B														2	37	55 6		
	- 3.8 to 4.4 m: Clayey silt/silt seams and trace organics						216														
			5	SS	5																
	- 4.6 to 5.2 m: Less than 25 mm of sample recovered within split spoon.		6	SS	13		215														
							214														
			7	SS	25																
							213														
			8	SS	30		212										19	39	35 7		
211.1																					
8.7	SILT (ML) to CLAYEY SILT-SILT (CL-ML), some sand, trace gravel, (TILL) Very Dense Grey Moist to wet		9	SS	100/0.24		211														
							210														

Continued on Next Page

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

METRIC

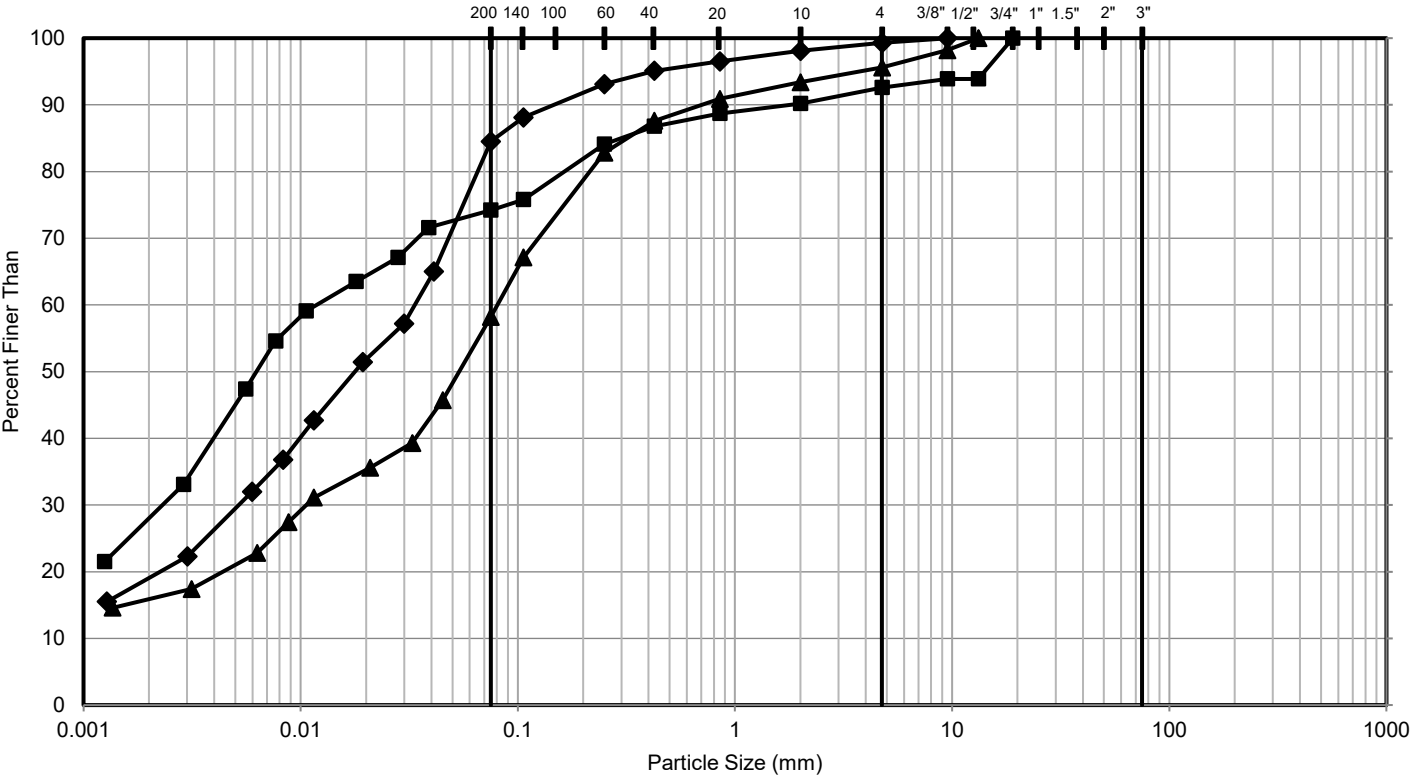
CHECKED BY MH

⁺, x³ : Numbers refer to Sensitivity o^{3%} STRAIN AT FAILURE

APPENDIX B

**Geotechnical Laboratory Test
Results**

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



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

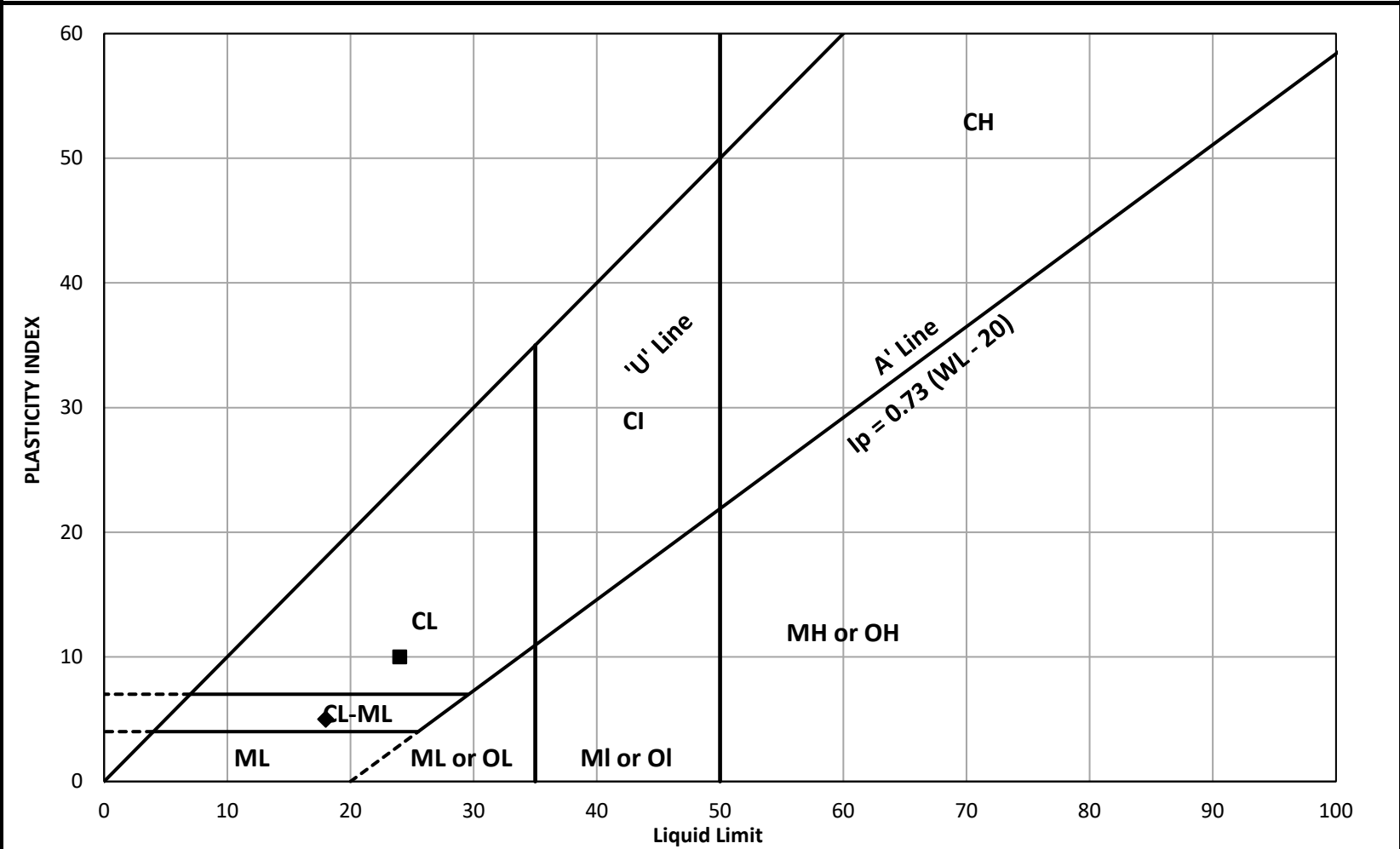
Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	CN-1	4	2.3 - 2.9	220.1 to 219.5
◆	CN-2	5	3.1 - 3.7	218.8 to 218.1
▲	CN-3	4A	3.1 - 3.5	216.8 to 216.3

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Metrolinx Overpass	
CONSULTANT	YYYY-MM-DD	2023-03-06	
	DESIGNED	MH	
	PREPARED	MH	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		Grain Size Distribution	
		Clayey Silt (CL) to Clayey Silt-Silt (CL-ML)	
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B1



PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/M/DRAFT/Appendix B - Lab Results/Working files | FILE NAME: 19136074 M Atterberg Working File.xlsm

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



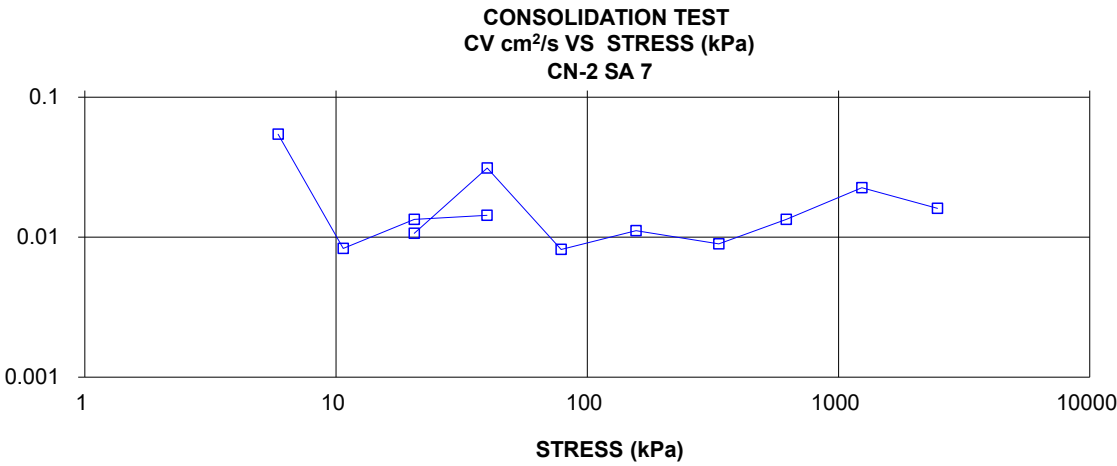
	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	CN-1	4	2.3 - 2.9	17.8	24	14	10	
◆	CN-3	2	0.8 - 1.4	12.3	18	13	5	

CLIENT		
AECOM / MTO		
	CONSULTANT	YYYY-MM-DD
	DESIGNED	2023-03-06
	PREPARED	MH
	REVIEWED	MH
	APPROVED	KJB

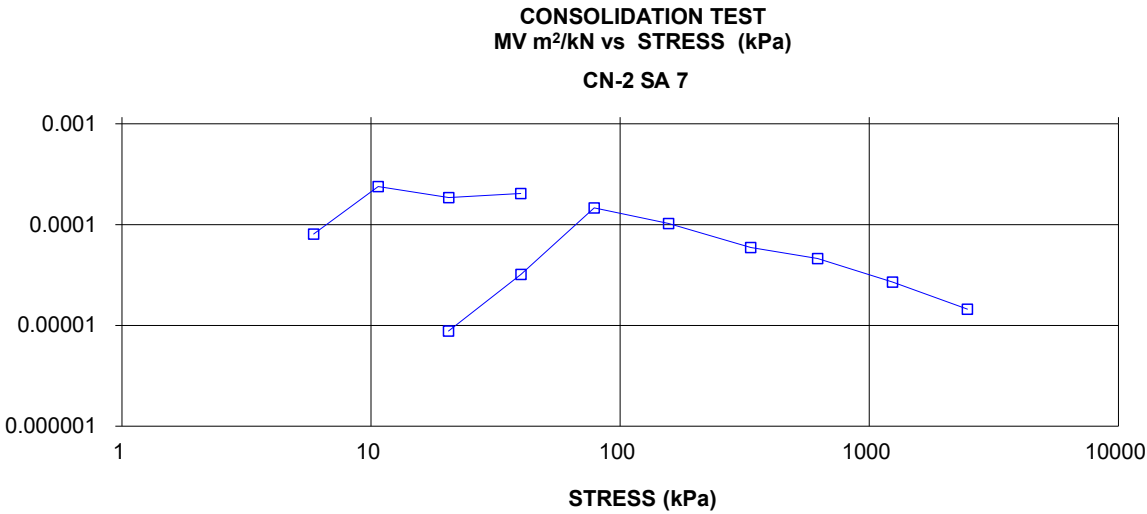
PROJECT			
Bradford Bypass - Metrolinx Overpass			
TITLE			
Plasticity Chart			
Clayey Silt (CL) to Clayey Silt-Silt (CL-ML)			
PROJECT NO.	CONTROL	REV.	FIGURE
0	0	0	B2

CONSOLIDATION TEST SUMMARY					FIGURE B3		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	19136074			Sample Number	7		
Borehole Number	CN-2			Sample Depth, m	4.57-5.18		
TEST CONDITIONS							
Test Type	Laboratory Standard			Load Duration, hr	24		
Oedometer Number	1						
Date Started	01/21/2022						
Date Completed	02/02/2022						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.53			Unit Weight, kN/m ³	22.11		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m ³	19.48		
Area, cm ²	31.61			Specific Gravity, measured	2.72		
Volume, cm ³	80.07			Solids Height, cm	1.849		
Water Content, %	13.50			Volume of Solids, cm ³	58.46		
Wet Mass, g	180.48			Volume of Voids, cm ³	21.61		
Dry Mass, g	159.01			Degree of Saturation, %	99.4		
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	2.533	0.370	2.533				
5.88	2.532	0.369	2.532	25	5.44E-02	8.06E-05	4.29E-07
10.68	2.529	0.367	2.530	163	8.33E-03	2.39E-04	1.95E-07
20.49	2.524	0.365	2.527	101	1.34E-02	1.85E-04	2.43E-07
39.88	2.514	0.359	2.519	94	1.43E-02	2.04E-04	2.86E-07
10.74	2.515	0.360	2.515				
20.47	2.515	0.360	2.515	126	1.06E-02	8.79E-06	9.17E-09
39.93	2.514	0.359	2.514	43	3.12E-02	3.21E-05	9.81E-08
78.70	2.499	0.351	2.506	163	8.17E-03	1.47E-04	1.17E-07
156.22	2.479	0.340	2.489	118	1.11E-02	1.02E-04	1.12E-07
334.25	2.452	0.326	2.466	144	8.95E-03	5.94E-05	5.21E-08
620.16	2.419	0.308	2.436	94	1.34E-02	4.61E-05	6.05E-08
1238.07	2.377	0.285	2.398	54	2.26E-02	2.70E-05	5.96E-08
2477.03	2.331	0.260	2.354	73	1.61E-02	1.45E-05	2.28E-08
620.16	2.335	0.263	2.333				
156.15	2.342	0.266	2.339				
39.88	2.349	0.270	2.346				
10.73	2.359	0.276	2.354				
Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t ₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 33-43cm from top of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	2.36			Unit Weight, kN/m ³	23.17		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m ³	20.91		
Area, cm ²	31.61			Specific Gravity, measured	2.72		
Volume, cm ³	74.57			Solids Height, cm	1.849		
Water Content, %	10.80			Volume of Solids, cm ³	58.46		
Wet Mass, g	176.18			Volume of Voids, cm ³	16.11		
Dry Mass, g	159.01						
Prepared By: LH				WSP Golder		Checked By: MM	

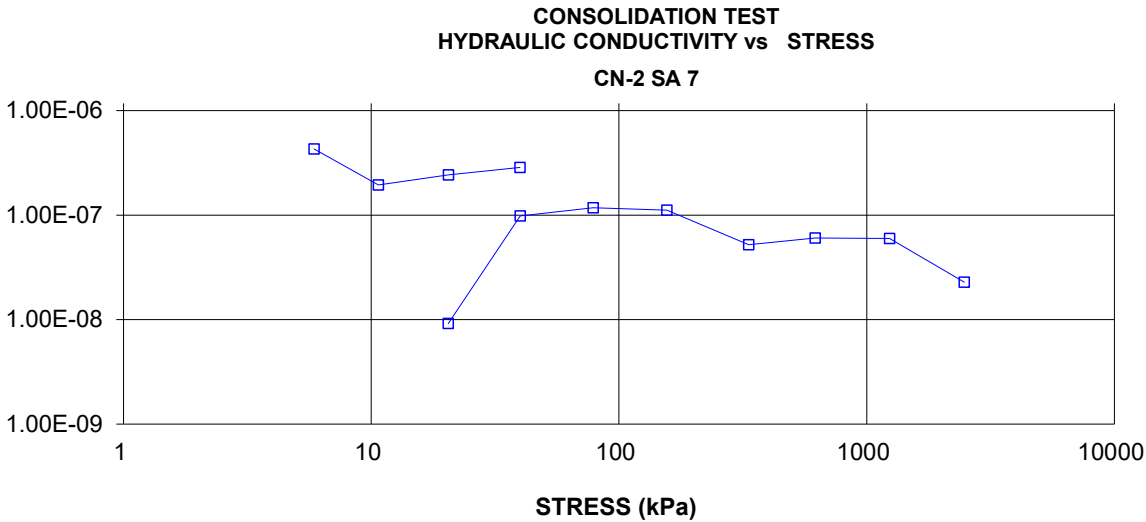
COEFFICIENT OF CONSOLIDATION,
cm²/s

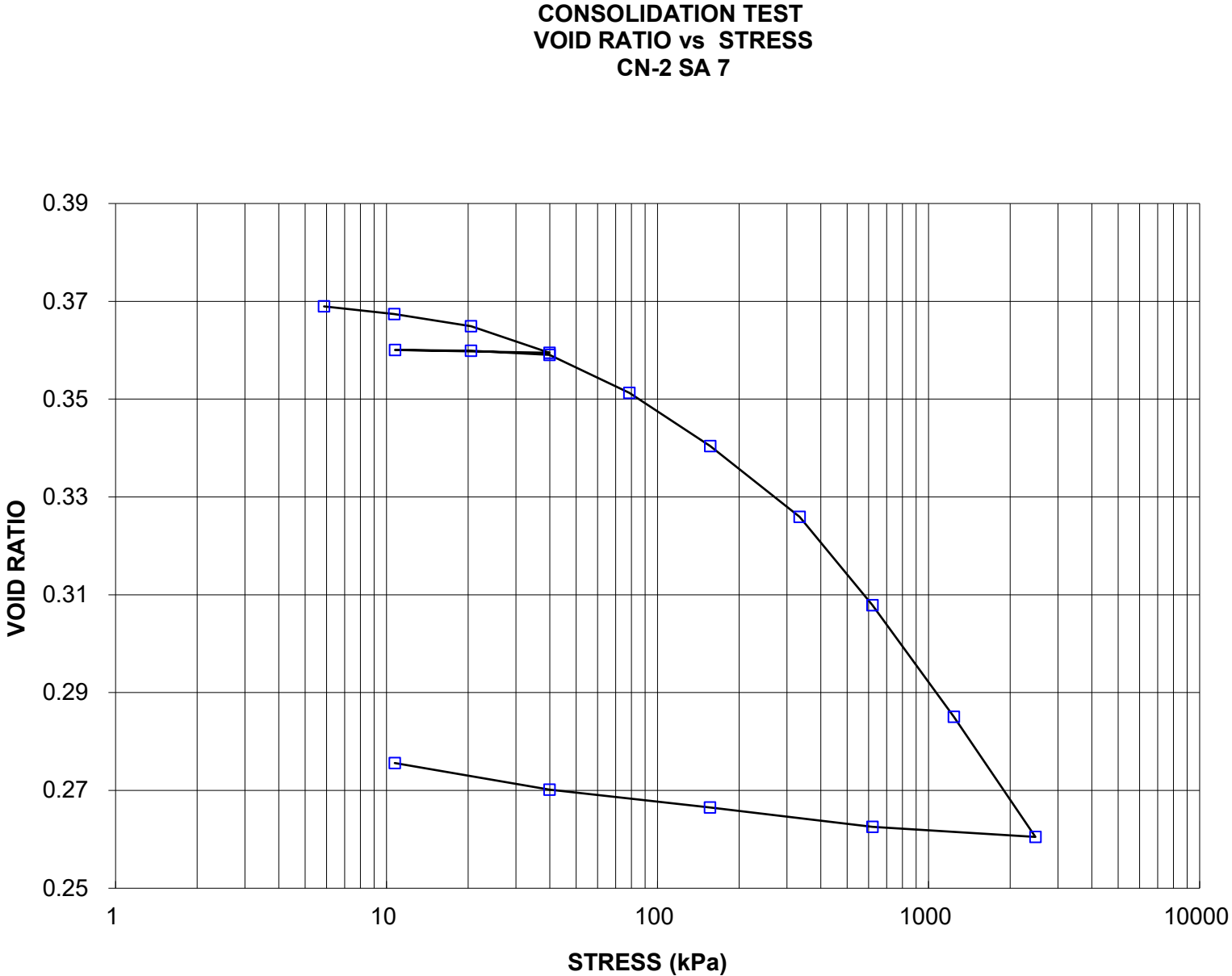


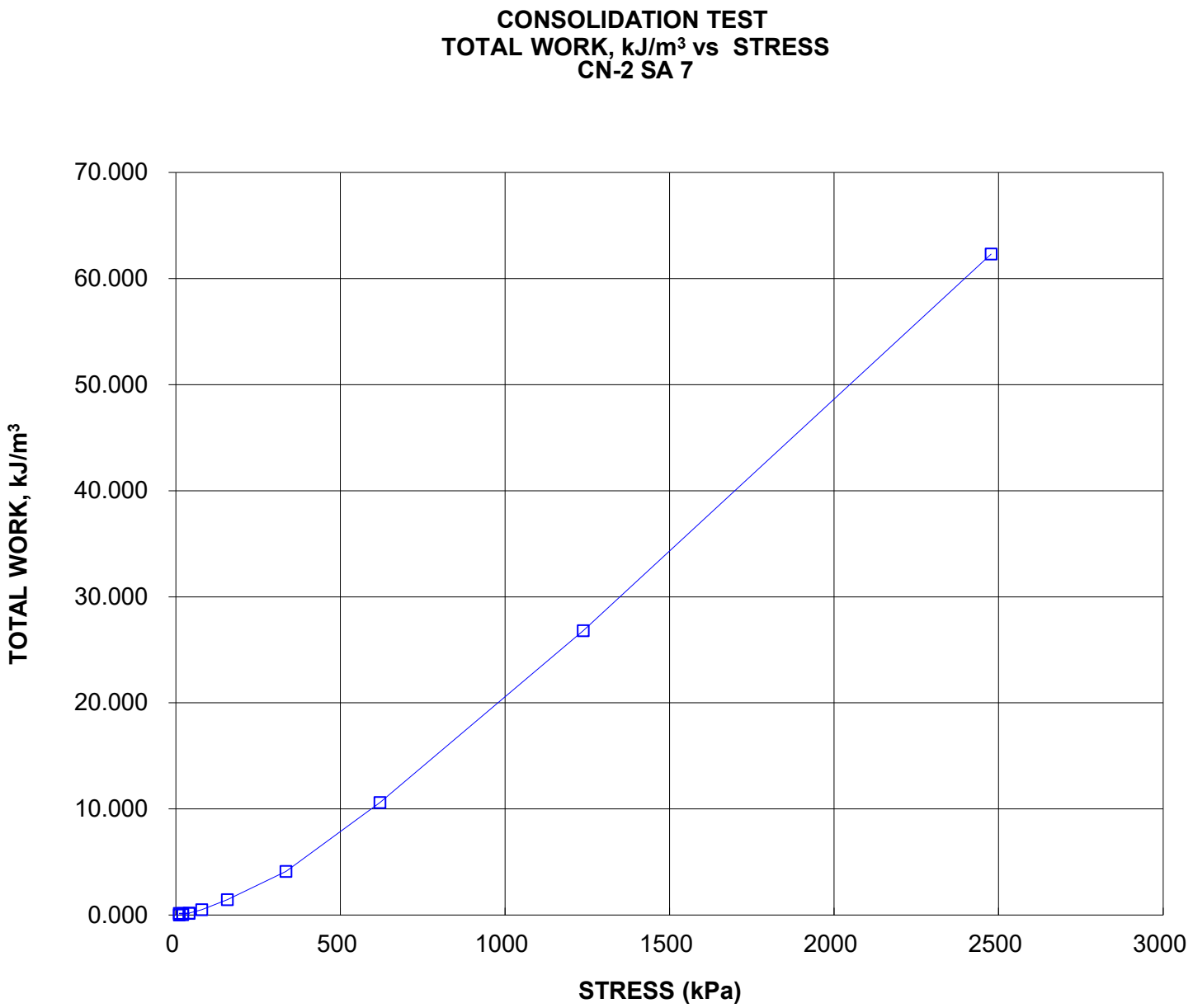
VOLUME COMPRESSIBILITY, m²/kN



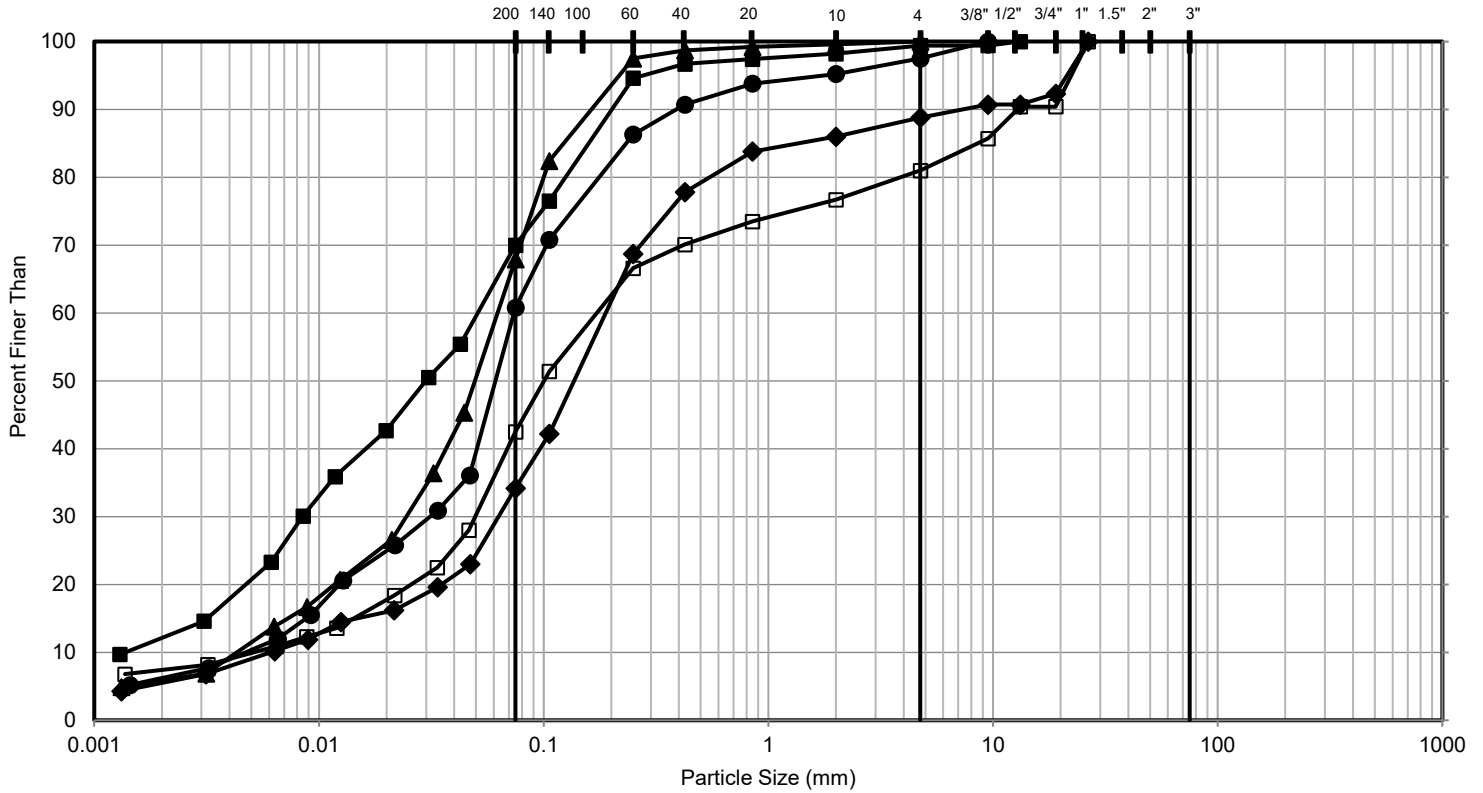
HYDRAULIC CONDUCTIVITY, cm/s







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



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	CN-1	6	3.8 - 4.4	218.6 to 218.0
◆	CN-1	7B	4.9 - 5.2	217.5 to 217.2
▲	CN-1	8	6.1 - 6.6	216.3 to 215.9
●	CN-3	4B	3.5 - 3.7	216.3 to 216.1
□	CN-3	8	7.6 - 8.2	212.2 to 211.6

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2023-03-06

DESIGNED MH

PREPARED MH

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Metrolinx Overpass

TITLE

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

PROJECT NO.

19136074

CONTROL

0

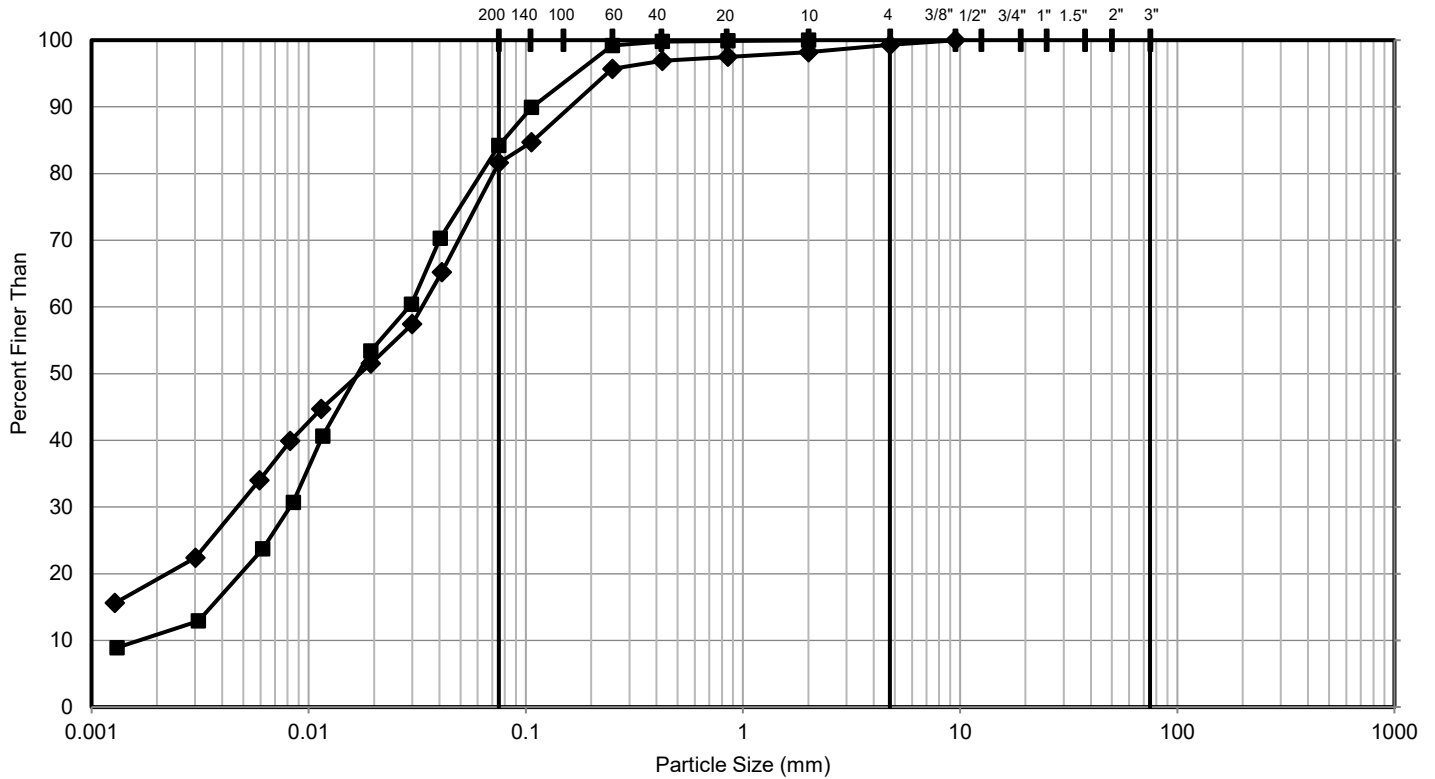
REV.

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FIGURE

B7

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



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	CN-1	13	13.7 - 14.0	208.7 to 208.4
◆	CN-2	11	10.7 - 10.8	211.1 to 211.0

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2023-03-06

DESIGNED MH

PREPARED MH

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Metrolinx Overpass

TITLE

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

PROJECT NO.

19136074

CONTROL

0

REV.

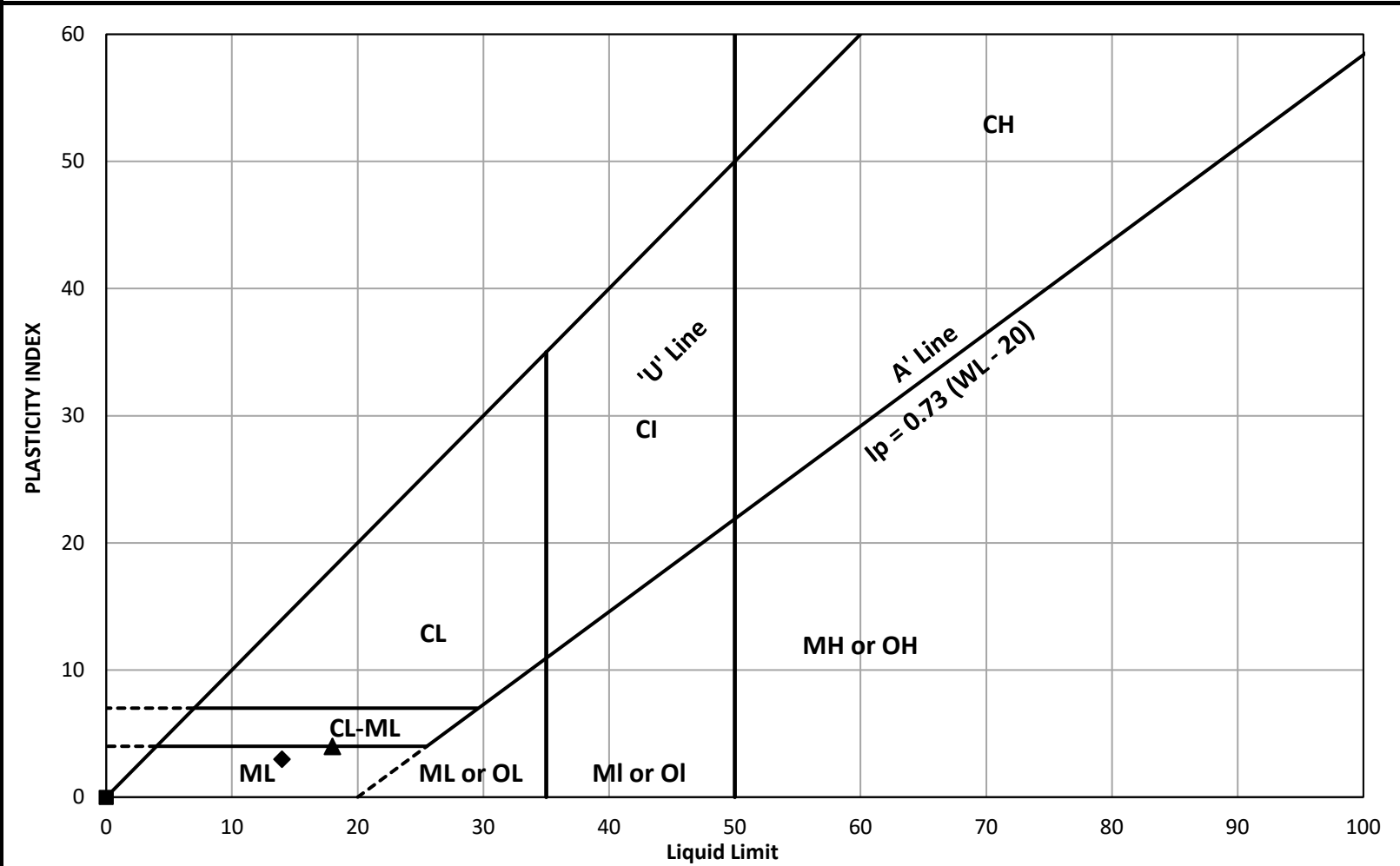
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FIGURE

B8

PATH: https://goldeassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/M/DRAFT/Appendix B - Lab Results/Working files | FILE NAME: 19136074 M Atterberg Working File.xlsm

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



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	CN-1	10	9.1 - 9.4	10.6	0	NP	0	
◆	CN-2	9	7.6 - 7.9	8.9	14	11	3	
▲	CN-2	12	12.2 - 12.4	11.8	18	14	4	

CLIENT

AECOM / MTO

CONSULTANT

wsp

GOLDER

YYYY-MM-DD

2023-03-06

DESIGNED

MH

PREPARED

MH

REVIEWED

KJB

APPROVED

KJB

PROJECT

Plasticity Chart
Bradford Bypass - Metrolinx Overpass

TITLE

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

PROJECT NO.

19136074

CONTROL

0

REV.

0

FIGURE

B9

APPENDIX C

Analytical Laboratory Test Results



Your Project #: 19136074
 Site Location: BRADFORD BYPASS
 Your C.O.C. #: n/a

Attention: Manisha Ahuja

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

Report Date: 2023/03/06
 Report #: R7534803
 Version: 2 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BUREAU VERITAS JOB #: C1Z5211

Received: 2021/12/17, 11:54

Sample Matrix: Soil
 # Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2021/12/20	2021/12/21	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2021/12/20	2021/12/20	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2021/12/23	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2021/12/22	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2021/12/20	2021/12/20	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2021/12/17	2021/12/20	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2021/12/20	2021/12/22	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.

(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE, Calgary, AB, T2E 6P8

(2) Offsite analysis requires that subcontracted moisture be reported.



Your Project #: 19136074
Site Location: BRADFORD BYPASS
Your C.O.C. #: n/a

Attention: Manisha Ahuja

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

Report Date: 2023/03/06
Report #: R7534803
Version: 2 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BUREAU VERITAS JOB #: C1Z5211

Received: 2021/12/17, 11:54

Encryption Key

Please direct all questions regarding this Certificate of Analysis to:

Ankita Bhalla, Project Manager

Email: Ankita.Bhalla@bureauveritas.com

Phone# (905) 817-5700

=====

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SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		RJZ033	RJZ036		
Sampling Date		2021/11/18	2021/11/17		
COC Number		n/a	n/a		
	UNITS	CN-2 SA-02 2'6"-4'6"	CN-1 SA-02 2'6"-4'6"	RDL	QC Batch
Calculated Parameters					
Resistivity	ohm-cm	3900	5500		7736410
Inorganics					
Soluble (20:1) Chloride (Cl ⁻)	ug/g	50	22	20	7741334
Conductivity	umho/cm	256	180	2	7741152
Available (CaCl ₂) pH	pH	7.45	7.66		7741621
Soluble (20:1) Sulphate (SO ₄)	ug/g	<20	<20	20	7741338
Sulphide	mg/kg	4.2 (1)	1.3 (2)	0.5	7752524
Physical Testing					
Moisture-Subcontracted	%	20	15	0.30	7752523
RDL = Reportable Detection Limit QC Batch = Quality Control Batch (1) Sample contained greater than 10% headspace at time of extraction. Sample extracted past method-specified hold time. Analyzed past method specified hold time (2) Sample extracted past method-specified hold time. Analyzed past method specified hold time					



**BUREAU
VERITAS**

Bureau Veritas Job #: C1Z5211
Report Date: 2023/03/06

WSP Canada Inc.
Client Project #: 19136074
Site Location: BRADFORD BYPASS
Sampler Initials: AM

TEST SUMMARY

Bureau Veritas ID: RJZ033
Sample ID: CN-2 SA-02 2'6"-4'6"
Matrix: Soil

Collected: 2021/11/18
Shipped:
Received: 2021/12/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7741334	2021/12/20	2021/12/21	Alina Dobreanu
Conductivity	AT	7741152	2021/12/20	2021/12/20	Kien Tran
Moisture (Subcontracted)	BAL	7752523	N/A	2021/12/23	Kerstin Joyce Luge
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison
pH CaCl ₂ EXTRACT	AT	7741621	2021/12/20	2021/12/20	Taslima Aktar
Resistivity of Soil		7736410	2021/12/20	2021/12/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7741338	2021/12/20	2021/12/22	Avneet Kour Sudan

Bureau Veritas ID: RJZ036
Sample ID: CN-1 SA-02 2'6"-4'6"
Matrix: Soil

Collected: 2021/11/17
Shipped:
Received: 2021/12/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7741334	2021/12/20	2021/12/21	Alina Dobreanu
Conductivity	AT	7741152	2021/12/20	2021/12/20	Kien Tran
Moisture (Subcontracted)	BAL	7752523	N/A	2021/12/23	Kerstin Joyce Luge
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison
pH CaCl ₂ EXTRACT	AT	7741621	2021/12/20	2021/12/20	Taslima Aktar
Resistivity of Soil		7736410	2021/12/20	2021/12/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7741338	2021/12/20	2021/12/22	Avneet Kour Sudan



BUREAU
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2023/03/06

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

GENERAL COMMENTS

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

Package 1	13.3°C
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Revised Report [2023/03/06]: Split report required as per client request.

Results relate only to the items tested.



BUREAU
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2023/03/06

QUALITY ASSURANCE REPORT

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7741152	Conductivity	2021/12/20			98	90 - 110	<2	umho/cm	0.14	10
7741334	Soluble (20:1) Chloride (Cl-)	2021/12/21	NC	70 - 130	103	70 - 130	<20	ug/g	6.9	35
7741338	Soluble (20:1) Sulphate (SO4)	2021/12/22	122	70 - 130	107	70 - 130	<20	ug/g	NC	35
7741621	Available (CaCl2) pH	2021/12/20			100	97 - 103			1.4	N/A
7752523	Moisture-Subcontracted	2021/12/23					<0.30	%		
7752524	Sulphide	2021/12/22	106	75 - 125	108	75 - 125	<0.5	mg/kg	1.9	30

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).



BUREAU
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2023/03/06

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

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

Ghayasuddin Khan, M.Sc., P.Chem., QP, Scientific Specialist, Inorganics

Orla Jorgensen, Organics Lab Manager

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