



FINAL REPORT

Preliminary Foundation Investigation and Design Report

2nd Concession Road Overpass

Highway 400 to Highway 404 Link (Bradford Bypass)

Simcoe County and York Region

MTO Assignment No. 2019-E-0048

Submitted to:

AECOM Canada

300 Water Street

Whitby, ON L1N 9J2

Submitted by:

WSP Golder

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

+1 905 567 4444

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PART A

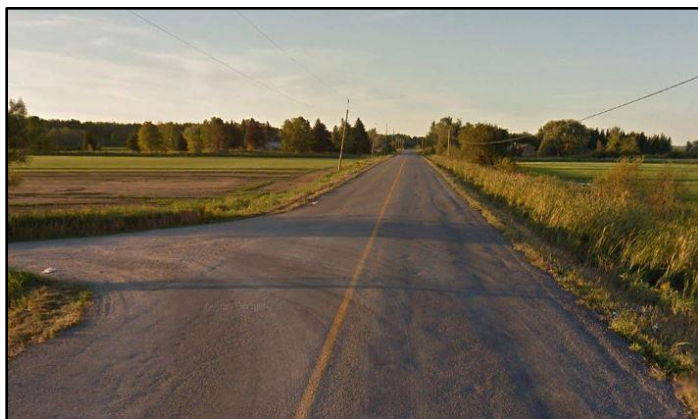
**PRELIMINARY FOUNDATION INVESTIGATION REPORT
2ND CONCESSION ROAD OVERPASS
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
SIMCOE COUNTY AND YORK REGION
MTO ASSIGNMENT NO. 2019-E-0048**

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 2nd Concession Road at the location shown on the Key Plan in Drawing 1.

2.0 SITE DESCRIPTION

The proposed twin single-span bridges will cross 2nd Concession Road north of municipal address number 21024, which is located in the Town of East Gwillimbury in the Region of York. This section of the proposed BBP alignment is oriented in an east-west direction on a slight skew to 2nd Concession Road, which generally runs in a north-south direction. Land use surrounding the site is predominantly agricultural, apart from several residential properties located more than 200 m north and south of the bridge site, including a previous Aerodrome (consisting of a grass airfield and fabric dome structure) that is no longer active located southwest of the site. There is a private laneway just south of the site (See Photograph 1) that provides access to farm fields and intersects the new highway alignment about 200 m west of 2nd Concession Road. There is an existing flood diversion channel that runs in north-south direction about 800 m west of the site which is located in a generally low-lying area. The existing 2nd Concession Road is an undivided arterial road with roadside ditches that carries two lanes of traffic (one lane in each the north and south direction as shown in Photograph 1). The existing road surface has an elevation of about 222 m, and the existing ground surface on both sides of 2nd Concession Road is generally flat (See Photograph 2) and at about Elevation 221 m.



Photograph 1 – looking north on 2nd Concession Road towards proposed bridge crossing site (private laneway entrance on left)



Photograph 2 – looking west on 2nd Concession Road (from the proposed BBP highway alignment centreline)

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on December 21 and 22, 2021, and between October 27 and November 2, 2022, during which time two boreholes (designated 2-1 and 2-2) were advanced at the locations shown on Drawing 1. Borehole 2-1 was advanced on the west side of 2nd Concession Road near the location of the proposed eastbound bridge, and Borehole 2-2 was advanced on the east side of 2nd Concession Road near the location of the proposed westbound bridge.

The boreholes were advanced using 210 mm outside diameter (O.D.) hollow stem augers to depths of approximately 2.3 m (Borehole 2-1) and 3.0 m (Borehole 2-2) below ground surface, followed by wash-rotary techniques (advancement of tricone with water/drilling mud) using a Diedrich D50 track-mounted drill rig. The drill rig was supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The wash-rotary technique was used to counter-balance hydrostatic forces and reduce disturbance at the sampling and testing interval. Water used for the drilling operation was brought to site in totes (portable plastic tanks) by the drilling subcontractor.

Soil samples were generally obtained at 0.75 m, 1.5 m, and 3.0 m intervals of depth using a 50 mm O.D. split spoon sampler driven with an automatic hammer in general accordance with Standard Penetration Test (SPT) procedure (ASTM D1586)¹. The split-spoon samplers used in the investigation generally limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions. In situ field vane shear tests were carried out using an MTO 'N'-vane in the cohesive soils, where practical, to assess peak and remoulded undrained shear strengths in general accordance with ASTM D2573². Additional samples were collected in the compressible clayey soils using 76 mm O.D. thin-walled 'Shelby' Tube samplers (ASTM D1587)³ to obtain relatively undisturbed samples for complex laboratory testing.

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 both boreholes to allow monitoring of the groundwater level. The installed piezometers consist of a 50 mm diameter PVC pipe, with 3.0 m long slotted screens within a filter sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack was backfilled to near ground surface with bentonite pellets in both boreholes. The standpipe piezometers were left sticking up out of the ground and protected with a monument cover.

The field work was monitored on a full-time basis by a member of WSP Golder's engineering staff who located the boreholes in the field, directed the sampling and in-situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in 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. Two consolidation (oedometer) tests were performed on selected Shelby tube samples collected from Boreholes 2-1 and 2-2. All laboratory tests were carried out in general accordance with MTO and / or ASTM Standards, as applicable.

One soil sample from Borehole 2-2 was 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

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

³ ASTM D1587 - Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.

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)		
2-1	4,889,552 (44.145964°)	306,525 (-79.478427°)	220.4	34.1
2-2	4,889,636 (44.146721°)	306,541 (-79.478236°)	221.3	49.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes from the investigation, including the piezometer installation details, water level readings, and the results of the in situ and laboratory tests, are provided on the Record of Borehole sheets in Appendix A. The results of the in situ field tests (i.e., SPT “N”-values and shear strengths from the field vanes) as presented on the borehole records and in Section 4 are the values measured directly in the field and are uncorrected. The detailed results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix B. The results of the analytical testing are provided in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface soils encountered in the boreholes advanced near the proposed 2nd Concession Road overpass consist of a surficial layer of fill underlain by an extensive cohesive deposit of clayey silt-silt to silty clay. The upper portion (upper 18 m) of the deposit is a clayey silt-silt to clayey silt with a firm to very stiff consistency and contains frequent seams and interlayers of non-cohesive soils comprised of compact to dense silt, sand and silty sand. The consistency of the cohesive deposit is generally firm to stiff near the ground surface and is stiff to very stiff down to a depth of about 18 m below ground surface suggesting an apparent “crust”. Below about 18 m depth, the cohesive deposit transitions to a clayey silt to silty clay and is generally firm to stiff. Below a depth of

about 40 m below ground surface, the cohesive deposit consists of very stiff to hard clayey silt-silt to clayey silt interlayered with very dense non-cohesive soils of silt and sand.

A more detailed description of the major soil layers encountered in the boreholes is described in the sections below.

4.2.1 Silt Fill

A 0.7 m thick layer of silt fill was encountered at ground surface in Borehole 2-2. The base of the fill layer extended to Elevation 220.6 m.

A single SPT 'N'-value obtained in the silt fill yielded 7 blows per 0.3 m of penetration, suggesting a loose state of compactness.

4.2.2 Clayey Silt-Silt to Silty Clay

An extensive cohesive deposit of clayey silt-silt to silty clay was encountered at ground surface in Borehole 2-1 and underlying the silt fill in Borehole 2-2. Both boreholes were terminated within the deposit at depths of 34.1 m (Elevation 186.3 m) and 49.4 m (Elevation 171.9 m). The clayey silt-silt to silty clay deposit contained frequent silt / sand seams and interlayers (especially in the upper 18 m below ground surface) as described in subsection 4.2.3.

The SPT 'N'-values measured in the clayey silt-silt to silty clay range from 4 to 72 blows per 0.3 m of penetration. The SPT 'N'-values typically measured 4 to 8 blows within 2 m of the ground surface, suggesting a firm consistency. Beneath this near surface firm zone which contains interlayers of silt and silty sand, the SPT 'N'-values typically measured 8 to 30 blows per 0.3 m of penetration suggesting stiff to very stiff "crust" that was about 15 m thick. Beneath the "crust", the SPT 'N'-values typically measured 4 to 14 blows per 0.3 m of penetration, suggesting a firm to stiff consistency. The bottom portion of the clayey silt-silt to silty clay deposit in Borehole 2-2 measured SPT 'N'-values ranging from 18 to 72 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

Five in situ field vane shear tests were carried out within the clayey silt-silt to silty clay; one test was carried out in Borehole 2-1 at a depth of 31.2 m and four tests were carried out in Borehole 2-2 at depths of 14.5 m, 19.8 m, 22.2 m, and 31.4 m. All tests yielded intact undrained shear strengths in excess of 96 kPa (i.e. the capacity of the torque wrench used for testing). Remoulded field vane shear tests were not carried out since the peak undrained shear strength exceeded the capacity of the testing apparatus. The results of the field vane tests confirms the majority of the clayey silt-silt to silty clay deposit is stiff to very stiff.

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

Atterberg limits testing was carried out on nine samples of the clayey silt-silt to silty clay; the samples typically had liquid limits ranging between 18% and 28%, plastic limits ranging between 12% and 20%, and plasticity indices ranging between 4% and 11%. One test (Borehole 2-2, Sample 20, sampled at a depth of about 30.8 m) had a liquid limit of 50%, a plastic limit of 20%, and a plasticity index of 30%. These results, which are plotted on a plasticity chart on Figure B2, indicate that the deposit generally consists of clayey silt-silt to clayey silt of low plasticity, with the exception of one test that indicates a silty clay of intermediate to high plasticity.

The natural water content measured on selected samples of the clayey silt-silt to silty clay ranges between about 17% and 25%, except for the intermediate to high plasticity silty clay sample that measured 38%.

Two laboratory consolidation (oedometer) tests were carried out on vertically trimmed specimens of the clayey silt obtained from a Shelby tube sample from Borehole 2-1 and 2-2 to assess the compressibility characteristics of the deposit. The details of the test results are shown on Figures B3 to B6 and B7 to B10 for Boreholes 2-1 and 2-2 respectively 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)
2-1 / Sa#20	25.2 – 25.8 / 195.2 – 194.6	267	300 - 325	33 - 58	1.1 to 1.2	0.175	0.010	0.68	0.0017 to 0.0045
2-2 / Sa#16A	19.1 – 19.7 / 202.2 – 201.6	227	240 - 300	13 – 53	1.1 to 1.3	0.185	0.011	0.69	0.0024 to 0.0038

Where: σ_p' = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods) c_v = Coefficient of consolidation (vertical) for approximate stress range $200 \text{ kPa} \leq \sigma_p' \leq 600 \text{ kPa}$

C_c = Compression index C_r = Recompression index

e_o = Initial void ratio OCR = Overconsolidation ratio

σ_{vo}' = Calculated existing vertical effective stress

4.2.3 Silt and Sand (Non-Cohesive Seams / Interlayers)

The extensive cohesive deposit of clayey silt-silt to silty clay contained non-cohesive seams / interlayers comprised of silt, sandy silt, sand, and silty sand in both boreholes.

One layer of silt (0.8 m thick) was encountered at a depth of 1.4 m (Elevation 219.9 m) in Borehole 2-2. Two major layers of silty sand to sand and silty sand to sandy silt were encountered in the upper portion of the clayey deposit at depths of 2.2 m to 3.0 m (Elevations 218.2 m to 218.3 m) and 10.1 m to 11.7 m (Elevations 210.3 m to 209.6 m) and were approximately 0.8 m to 3.4 m thick in both boreholes. A layer of silt (2.3 m thick) was encountered at a depth of 15.5 m in Borehole 2-1.

One major silt and sand layer was encountered in the lower portion of the cohesive deposit in Borehole 2-2 at a depth of 44.5 m (Elevation 176.8 m) and was approximately 3.0 m thick.

The SPT 'N'-values measured in the non-cohesive interlayers range from 4 to 83 blows per 0.3 m of penetration; but typically measured about 12 blows to 42 blows per 0.3 m of penetration indicating a generally compact to dense state of compactness. The 'N' value of 4 was measured in the silt layer at a depth of 1.4 m (Elevation 219.9 m) and the 'N' value of 83 was measured in the silt and sand layer at a depth of 44.5 m (Elevation 176.8 m) in Borehole 2-2.

Grain size distribution testing was carried out on three samples of the non-cohesive interlayers and the results are shown on Figure B11 in Appendix B.

Atterberg limits testing was carried out on the fines portion of two samples of the non-cohesive interlayers and one sample indicated a non-plastic sand to silty sand and the other sample measured a liquid limit of 18%, a plastic limit of 16%, and a plasticity index of 2%, indicating a silt with slight plasticity.

The natural water content measured on selected samples of the non-cohesive interlayers ranges between about 15% and 22%.

4.3 Groundwater Conditions

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

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

Borehole No.	Ground Surface Elevation (m)	Depth (Elevation) of Screen Interval / Sand Pack (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
2-1	220.4	6.1 to 9.1 (El. 214.3 to 211.3)	0.1	220.3	Dec. 21, 2021	Open borehole / inside hollow stem auger
			0.8	219.6	May 12, 2022	Piezometer
			0.8	219.6	May 13, 2022	Piezometer
			0.5	219.9	Feb. 1, 2023	Piezometer
2-2	221.3	45.2 to 48.2 (176.1 to 173.1)	3.2	218.1	Oct. 27, 2022	Open borehole / inside hollow stem auger
			0.7	220.6	Nov. 3, 2022	Piezometer
			2.6	218.7	Nov. 4, 2022	Piezometer
			2.0	219.3	Feb. 1, 2023	Piezometer
			2.8	218.5	Feb. 28, 2023	Piezometer

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

4.4 Analytical Testing of Soil

One soil sample was 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)
2-2, SA 3	7.74	5,700	177	<20 ¹	21

Note 1: Less than reportable detection limit.

5.0 CLOSURE

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

Signature Page

WSP Golder



Mark Henderson, P.Eng.
Geotechnical Engineer



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

MH/KJB/ljv/al

[https://golderassociates.sharepoint.com/sites/120387/project files/6 deliverables/foundations/2nd concession/final/19136074-r-r-rev0-pfidr 2nd conc_2023'10'20.docx](https://golderassociates.sharepoint.com/sites/120387/project%20files/6%20deliverables/foundations/2nd%20concession/final/19136074-r-r-rev0-pfidr%202nd%20conc_2023'10'20.docx)

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
2ND CONCESSION ROAD OVERPASS
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
SIMCOE COUNTY AND YORK REGION
MTO ASSIGNMENT NO. 2019-E-0048**

6.0 DISCUSSION AND PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation recommendations for planning and preliminary design of the Bradford Bypass and 2nd Concession Road overpass structures. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current subsurface exploration.

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

6.2 Project Understanding

Based on the latest Bradford Bypass mainline alignment and profile drawings provided by AECOM (preliminary draft dated May 2023), an Interchange consisting of twin bridge structures is proposed to carry the Bradford Bypass eastbound and westbound lanes over 2nd Concession Road at approximately STA. 23+200. According to the preliminary General Arrangement (GA) drawing (dated March 2023), each single span bridge structure will accommodate two lanes of traffic and a speed change lane in the eastbound and westbound direction (four lanes plus two speed change lanes total) for the interim configuration, with an ultimate configuration to accommodate four lanes and a speed change lane in each direction (eight lanes plus two speed change lanes total) requiring future bridge widenings. The total span length of each bridge is about 50 m in order to accommodate interim and future widening of 2nd Concession Road and multi-use paths. The structural classification of the bridge(s) is defined as “major-route” by the structural designer at this preliminary stage.

Based on the latest preliminary profile drawing, the existing 2nd Concession Road surface between the proposed westbound and eastbound bridges is at about Elevation 222 m, and the existing ground surface adjacent to 2nd Concession Road along the proposed highway centreline is relatively flat and at about Elevation 221 m. The proposed Bradford Bypass highway grade is shown to be up to about Elevation 231 m with proposed approach embankment heights on the order of about 10 m above the ground surface.

Based on the preliminary GA drawing, the interim design configuration will consist of an approximately 17 m to 19 m wide bridge (consisting of two 3.8 m wide lanes, variable width speed change lane, 2.5 m and 3.0 m wide shoulders, and 0.5 m wide concrete barriers) for both the eastbound and westbound directions, and the ultimate configuration will consist of future widening of the bridges toward the highway centreline (essentially join the bridges together less a 1 m gap) with an additional width of about 10 m being added to each bridge (to accommodate two 3.8 m wide lanes, a 3.6 m shoulder, a 0.5 m wide concrete barrier, and a 1,250 mm buffer zone).

In addition, the existing two lane 2nd Concession Road is to be widened from about 10 m to 30 m in order to accommodate speed change lanes, shoulders, boulevards and multi-use paths for the interim condition, and further widened to about 40 m for the ultimate 2nd Concession Road configuration to accommodate two additional lanes and a median.

6.3 General Foundations Design Context

6.3.1 Consequence and Site Understanding Classification

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

In addition, based on the preliminary level of foundation investigation completed to date at this location (see Part A of this report) in comparison to the degree of site understanding, the level of confidence for design of the 2nd Concession Road bridge foundation elements and approach embankments has been assessed as a “low degree of site and prediction model understanding”. At the time of the borehole investigation the locations of the abutment foundations were not confirmed, and site access was limited, thus, 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 preliminary design at this stage. During detail design, additional investigation and testing will need to be performed to increase the level of confidence and modify the geotechnical resistance factors as appropriate. In addition, reference is made to the MTO Material Engineering and Research Office (MERO) Memorandum #2020-01 (dated March 23, 2020) for future foundation, settlement and stability analyses during detail design, as applicable.

6.3.2 Seismic Design

6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance, \bar{N}_{60} and average undrained shear strength, s_u within the upper 30 m of the overburden below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019).

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

6.3.2.2 Spectral Response Values and Seismic Performance Category

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

Seismic Hazard Values for Site Class D	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.039	0.059	0.097
PGV (m/s)	0.040	0.062	0.100
$S_a(0.2)$ (g)	0.066	0.099	0.153
$S_a(0.5)$ (g)	0.054	0.079	0.121
$S_a(1.0)$ (g)	0.034	0.050	0.076
$S_a(2.0)$ (g)	0.016	0.025	0.039
$S_a(5.0)$ (g)	0.003	0.006	0.009
$S_a(10.0)$ (g)	0.001	0.003	0.004

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

6.3.2.3 Soil Liquefaction

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

In general, the soils at these bridge sites consist of firm to very stiff clayey silt-silt to silty clay soils with seams / interlayers of compact to dense silts and sands. Considering the compactness, consistency and relatively low site-specific PGA, the site is estimated to have a low potential for liquefaction during a seismic event based on preliminary liquefaction assessment of the soil and preliminary analyses using the simplified stress-based method as per Section 6.14.8 of the CHBDC (2019). The potential for liquefaction and cyclic mobility (especially for the looser near surface cohesionless soils and upper clayey silt-silt soils with high liquidity index) will need to be assessed when more site-specific foundation soil information and the seismic performance category is confirmed during detail design. The settlement and stability analyses will be to be reassessed, as applicable.

6.4 Foundation Types

Based on the proposed single-span twin structure configuration and subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments. The preliminary recommendations provided herein will be subject to change when more detailed soil information and actual foundation locations are known. Foundation construction should be in general accordance with OPSS.PROV 902 (Excavating and Backfilling – Structures).

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

Shallow foundations are marginally feasible for the abutments if founded below the surficial loose / soft to firm soils on the compact to dense sand to silty sand layer (or on a “perched” compacted granular pad above the sand to silty sand layer) encountered at depths of 2.2 to 3.0 m below ground surface (about Elevation 218.2 m). While shallow foundations are feasible, subexcavation into the founding granular soils with a high groundwater table will require dewatering and a large excavation footprint or temporary excavation support. In addition, significant time dependent settlement of the underlying thick cohesive deposit will occur under the approximately 10 m high approach embankment loading, making this option less practical and not the preferred alternative from a geotechnical/foundations perspective. Deep foundations consisting of driven steel H- or tube piles with the pile cap perched within the approach embankments are preferred from a design and constructability perspective, and this option will permit integral abutments. Caissons are also considered to be a feasible foundation option; however, although this option provides higher geotechnical resistances compared to shallow foundations or driven piles, it would be more costly and would not permit integral abutment design.

6.5 Shallow Foundations

The near surface soft to firm clayey silt soils and loose silt soils are not considered suitable to support shallow foundations or a granular pad (to support shallow foundations). Strip or spread footings founded on the compact to dense sand to silty sand interlayer (at or below the approximate elevation identified below) may be considered for support of the structure abutments. The feasibility of designing 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 2 to 3 m below existing ground surface (and about 2 m below anticipated groundwater level) to remove soft/firm or loose soils and reach the competent founding strata are anticipated to be required. Consideration could be given to subexcavating the unsuitable soils and placing engineered fill such that spread footings could be “perched” within approach embankments to increase geotechnical ultimate resistance values, however, long-term settlements due to embankment loading will still be concern.

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

Anticipated Founding Stratum at Bridge Locations	Founding Elevation ¹	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance ²
Compact to dense sand to silty sand	218.2 m	3 m	500	175
		5 m	600	125
Granular pad on compact to dense sand to silty sand	Min. 3 m of granular fill above El. 218.2 m	3 m	650	250
		5 m	700	175
	Min. 5 m of granular fill above 218.2 m	3 m	750	275
		5 m	800	200

Notes:

1. Subexcavation up to about 3 m 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 compact to dense sand 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 clayey silt to silty clay deposit containing the sand and silt interlayers are considered feasible for support of the new abutments. Steel H-piles extending to the very dense sand / silt or hard clayey silt-silt encountered at a depth of about 45 m (Elevation 177 m) in Borehole 2-2 are considered the preferred pile foundation option for support of the new abutments.

Closed ended steel tube piles are also considered a feasible deep foundation option; however, driven steel H-piles may be preferred over steel tube piles given that H-piles are most commonly used for integral abutment design and pose a lower risk of "hanging up" or being deflected from their vertical or battered orientation during installation if gravel pockets, cobbles or boulders are present (although not anticipated based on the current investigation).

Consideration should be given to "perched" pile caps within the embankment fill to reduce subexcavation and dewatering requirements, although settlement and any associated downdrag loads due to the embankment loading will need to be assessed and mitigated during detail design.

Driven piles will develop the majority of resistance from shaft friction, although increased capacity may be derived from end-bearing on the very dense sand / silt and hard clayey silt-silt deposits encountered below about 45 m depth in Borehole 2-2. The factored ultimate and serviceability geotechnical axial resistances for a range of driven steel H- and tube piles (with corresponding pile tip elevations and anticipated soil strata near pile tip elevation) for the bridges is provided below for preliminary design purposes.

Approximate Pile Length ¹	Estimated Pile Tip Elevation ² (m)	Soil Strata Near Pile Tip Elevation	Pile Type	Factored Ultimate Geotechnical Resistance ^{3,4}	Factored Serviceability Geotechnical Resistance (for 25 mm of Settlement)
30 m	189 m	Firm to stiff clayey silt to silty clay	HP 310x110	800	>800
			HP 360x108	950	>950
			324 mm dia. tube pile (min. 9.5 mm thick)	700	>700
			406 mm dia. tube pile (min. 9.5 mm thick)	1,000	>1,000
45 m	175 m	Very dense silt to sand / Hard clayey silt-silt	HP 310x110	1,600	>1,600
			HP 360x108	1,800	>1,800
			324 mm dia. tube pile (min. 9.5 mm thick)	1,400	>1,400
			406 mm dia. tube pile (min. 9.5 mm thick)	2,000	>2,000

Notes:

1. Measured from underside of pile cap elevation (taken to be approximately 1.5 m below ground surface, i.e., frost depth).
2. Assuming 45 m long piles are driven approximately 1 m to 2 m into very dense sand / silt.
3. Resistance values assume single pile and do not consider pile group efficiency.
4. Consideration should be given to using a heavier H-pile section (310x132 or 360x132) or thicker tube pile (13 mm thickness) if piles are to be driven longer than 30 m. Constructability / feasibility of driving larger size piles more than 30 m should be assessed during detail design.

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

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS). PROV 903 (*Deep Foundations*) as amended by Special Provision 109F57. It is recommended that High-Strain Dynamic testing be specified on at least 10% of the piles or two piles at each foundation element (whichever is greater) in each stage of construction, and re-tapping of piles performed no sooner than 2 weeks after initial driving.

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

- Advanced site-specific investigation during detail design to confirm or adjust axial geotechnical resistances for design based on 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”.

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

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

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

6.6.2 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:

Approximate Caisson Length ¹	Estimated Caisson Base Elevation ²	Soil Strata near Caisson Base Elevation	Caisson Diameter	Factored Ultimate Geotechnical Resistance ^{2,3}	Factored Serviceability Geotechnical Resistance (for 25 mm of Settlement)
30 m	189 m	Firm to stiff clayey silt to silty clay	0.9 m	2,000	>2,000
			1.2 m	2,750	>2,750
			1.5 m	3,500	>3,500

Notes:

1. Measured from existing ground surface.
2. Resistance values assume single caisson and do not take into account caisson group efficiency.

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

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

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

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

Specialized construction techniques would be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (e.g. polymer slurry or other slurry mix) within the open hole / liner to prevent basal heave and disturbance of water-bearing cohesionless layers/interlayers along the shaft and at the base. Although the above drilled shaft capacities have predominantly a shaft friction component, the performance of the drilled shafts in compression will depend to some degree upon the final cleaning and verification of the condition

of the drilled shaft. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection device (SID) or a shaft quantitative inspection device (SQUID); because of the likelihood of water with entrained sediment or the presence of polymer slurry, it is recommended that SQUID testing be specified on some or all caissons to demonstrate and verify base cleaning. Should the inspection indicate that loosened or compressible material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

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

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

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

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

6.6.3 Resistance to Lateral Loads

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

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

For preliminary design, the resistance to static lateral loading in front of a single pile or drilled shaft may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 2002 as referenced in CHBDC, 2006).

For non-cohesive soils:

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

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

For cohesive soils:

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

Where S_u is the undrained shear strength of the soil (kPa); and
 B 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-Silt to Silty Clay (soft to firm) (stiff to very stiff) (very stiff to hard)	- - -	25 - 50 50 – 100 100 – 150
Silts and Sands (loose to compact interlayers) (compact to dense interlayers)	7,000 – 15,000 15,000 – 25,000	- -
Below Elevation 176.8 m (very dense silt and sand, hard clayey silt)	30,000 – 40,000	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 preliminary 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).

6.6.4 Downdrag Loads on Piles / Caissons

Based on the preliminary design, the east and west approach embankments for both bridges are to be about 10 m high with total settlements in the foundation soils estimated to be about 300 mm due to the embankment loading (see Section 6.8.2). Accordingly, depending on the relative timing of embankment fill placement at and near the abutments, and pile installation, the embankment fills could induce significant downdrag loads that will need to be accounted for in the assessment of the structural loading of the piles.

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

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

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

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 10 m 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 fills, organics, topsoil or re-worked soil (although not encountered in Boreholes 2-1 and 2-2 but anticipated based on the surrounding agricultural lands), and any near surface soft/loose soils (anticipated to extend up to about 2 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 Safety⁴ of 1.4 (for the temporary, short-term condition) and 1.6 (for the permanent, long-term condition) were 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 below. 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 composed of OPSS.PROV 1010 Granular 'A' or 'B' Type II with side slopes inclined at 2H:1V (with 2 m wide mid-height berms) and founded on the compact to very dense silty sand to sand layer.

Location	Idealized Stratigraphic Unit	Bulk Unit Weight, γ (kN/m ³)	Effective Friction Angle, ϕ' (°)	Undrained Shear Strength, S_u (kPa)
Eastbound Bridge Approach Embankment (Borehole 2-1)	New Granular Fill (Granular 'A' or 'B' Type II)	21	36	--
	Clayey Silt (Soft to Firm)	19	28	25 to 50
	Silty Sand to Sand (Compact to Dense)	20	34	--
	Clayey Silt (Stiff to Very Stiff)	19	28	50 to 100
	Silt (Compact to Dense)	19	32	--
	Clayey Silt (Firm to Stiff)	19	28	30 to 50
Westbound Bridge Approach Embankment (Borehole 2-2)	New Granular Fill (Granular 'A' or 'B' Type II)	21	36	--
	Silt (Loose)	19	28	--
	Clayey Silt (Firm to Stiff)	19	28	50 to 100
	Silty Sand (Compact to Dense)	20	34	--
	Clayey Silt-Silt (Stiff to Very Stiff)	19	28	100 to 125
	Sandy Silt (Compact to Dense)	20	33	--
	Clayey Silt (Very Stiff to Hard)	20	30	100 to 150
	Clayey Silt (Firm to Very Stiff)	19	28	50 to 100
	Silt to Sand (Very Dense)	21	35	--
	Clayey Silt-Silt (Hard)	21	30	200

Shallower side-slopes (on the order of 2.5H:1V) or toe berms may be required if the loose and/or soft soils are left in-place. 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, which may allow for reduced stripping recommendations.

⁴ 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. $FoS = 1/(\Psi \cdot \phi_{gu})$).

The use of topsoil and seeding as per OPSS.PROV 803 (*Seed and Cover*) or pegged sod should be incorporated into the permanent embankment side slopes to reduce the risk of surficial slope failures.

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:

- Single Embankment Model for Interim Configuration: assumes multistage construction with separate approach embankment (18 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 would induce additional 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 (56 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 sources of total settlement are considered to include the following:

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

As previously mentioned, the surficial loose non-cohesive deposits and soft to firm cohesive deposits extending to depths of up to about 2 m were not incorporated into the model, as it is assumed these softer/looser soils would be stripped prior to the approach embankment construction.

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

The consolidation settlement of the cohesive deposits were assessed using the results of the in situ field vane tests and correlations based on SPT "N"-values to estimate the stress history for the cohesive deposits. The preconsolidation pressure was estimated using the correlation proposed by Mesri (1975). The results of the laboratory index tests were used to assess deformation parameters (i.e., compression and recompression

indices) using empirical correlations proposed in literature by Rendon-Herrero (1980), Bowles (1984), Sowers (1970), Wood and Wroth (1978), and Terzaghi and Peck (1967) in collaboration with the consolidation tests from Boreholes 2-1 and 2-2.

6.8.2.1 Results of Analyses

For a proposed maximum embankment height of 10 m, the estimated magnitude of total settlement for the westbound and eastbound bridge approach embankments are expected to be on the order of about 285 mm to 350 mm, with approximately 15 mm to 25 mm of immediate settlement due to elastic compression of the non-cohesive interlayers and up to about 270 mm to 325 mm of primary consolidation settlements due to time dependent consolidation of the cohesive deposits. Secondary consolidation is anticipated to result in less than 5 mm of settlement.

The lower limit of the estimated settlement is representative of the single embankment model for the interim condition and the higher limit of the estimated settlement is representative of the continuous embankment model for the ultimate configuration. Based on the estimated magnitude of settlement noted above, settlement mitigation options will be required to meet the settlement performance criterion.

6.8.2.2 Mitigation Options

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

- **Preloading (with or without Surcharge):** In cases where the subsurface conditions are sufficiently permeable, such as the upper portion of the extensive cohesive deposit (overconsolidated and containing frequent cohesionless seams / interlayers), preloading to allow excess pore pressures to dissipate to induce settlement in a reasonable period of time (less than 6 months) is ideal. However, given the thickness of the less permeable lower portion of the extensive cohesive deposit, preloading could take several years to reduce settlements to tolerable levels. Depending on the actual time-rate of settlement (i.e. coefficient of consolidation, c_v), surcharging may or may not reduce preload times to a reasonable level (see section 6.8.2.3, below). Additional investigation will need to be carried out during detail design to ascertain whether or not preload and surcharging is a feasible settlement mitigation option.
- **Preload and Surcharging with Vertical Wick Drains:** Prefabricated vertical drains could be installed prior to construction of the embankments to relieve pore pressures and accelerate settlements. Prefabricated vertical drains would typically be installed on a 2 m triangular grid pattern and penetrate to the bottom of the cohesive deposits. Depending on the thickness of the compressible cohesive deposit (which extends to about 45 m below existing ground surface in Borehole 2-2), conventional wick drain installation and effectiveness may not be practical at this site but should be considered during detail design to accelerate the construction schedule.
- **Lightweight Slag or Cellular Concrete:** Various lightweight fill materials are available, from lightweight slag with a unit weight of approximately 14 kN/m³, to cellular concrete with a unit weight between 4 and 7 kN/m³. However, for the volume of fill required for the new embankments, a similar preloading period to using conventional fill materials may still be required to achieve the settlement performance criterion.

- **Lightweight Expanded Polystyrene:** The use of expanded polystyrene (EPS) is another alternative that can be considered to significantly reduce the magnitude of consolidation settlement. Where required, EPS can be used to achieve the settlement performance criterion with limited preloading and therefore, will reduce the length of time for construction. However, the disadvantage of using EPS is the high cost relative to conventional fill or other lightweight fill options. Floatation concerns within the relatively low-lying area (located about 800 m east of an existing flood diversion channel) will also need to be considered.

Based on the above considerations, preloading is considered the technically preferred alternative; however, depending on the time-rate of consolidation (i.e. presence of silt/sand seams and interlayers, effective coefficient of consolidation, and thickness of the lower portion of the compressible cohesive deposit to be determined during detail design), preload and surcharging with vertical wick drains may be necessary to mitigate long-term post-construction settlement at this site. A preliminary assessment of estimated preload times is provided in the next section.

6.8.2.3 Preload and Surcharge Assessment

Based on the following range of estimated coefficient of consolidation (c_v) values and thickness of the lower compressible deposit (below about Elevation 203 m), is estimated that the following preload periods may be required for each approach embankment area to meet the settlement performance criterion, assuming two-way drainage and conventional granular fill for embankment construction.

Estimated Thickness of Lower Compressible Deposit (m)	Estimated Preload Time, days (years)		
	Estimated Coefficient of Consolidation, c_v (cm ² /sec)		
	1.3×10^{-2} (based on overconsolidated clays from literature)	2.0×10^{-3} (low average from consolidation tests)	4.0×10^{-3} (high average from consolidation tests)
20	200 – 350 days (0.5 – 1 year)	1,000 – 1,400 days (2.7 – 3.8 years)	750 – 900 days (2 – 2.5 years)
30	400 – 500 days (1 – 1.4 years)	> 1,450 days (> 4 years)	> 1,450 days (> 4 years)

As noted above, additional investigation (such as the use of Cone Penetration Testing with dissipation tests), particularly to refine the c_v values used to estimate the preload period range noted above and confirm the thickness of the drainage path, will need to be carried out during detail design to ascertain whether or not preload and surcharging are feasible settlement mitigation options.

Based on the available information, it is estimated that conventional preloading times would range from 1 to 3 years. If deep wick drains are installed, the preloading times could potentially be reduced to less than one year.

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

We understand that consideration is being given to constructing each bridge to accommodate the interim 4-lane (plus speed change lanes) configuration with widening of each bridge in the future to accommodate the ultimate 8-lane (plus speed change lanes) highway configuration. It is recommended that the approach embankment geometry for the ultimate bridge(s) configuration be constructed at the interim stage to induce the majority of anticipated settlement at the approach embankments (interim and future). The advantages of constructing the ultimate configuration of the approach embankments as early as possible includes reduced future construction staging and more importantly reduce the impacts of settlement / differential settlement on the interim and future approach embankment configuration and associated settlement sensitive structures (including the proposed widening of 2nd Concession Road). The disadvantages of constructing the ultimate embankment configuration at the interim stage are increased initial magnitudes of settlement, preload times and higher initial costs. Given the relatively long span (about 50 m) and conceptual open abutment configuration, the foundation zone of influence from the embankment loading is not anticipated to have a significant impact on the existing 2nd Concession Road embankment and settlements are estimated to be less than 25 mm. However, given that the proposed widening (interim and ultimate) of 2nd Concession Road extends to within the toe of the proposed approach embankments, the impact of settlement associated with the interim and ultimate approach embankment configurations will need to be considered and addressed during detail design.

As mentioned in Section 6.4 and Section 6.6.4, the settlement of the foundation soils due to the approach embankment loading (and any other foundation locations where the grade is to be raised) will need to be considered for design of any shallow footings (excess settlement in addition to the f-SLS geotechnical resistance) and/or deep foundations (i.e. associated downdrag forces).

6.8.3 Lateral Earth Pressures for Design

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

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in general accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.

- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall in accordance with Figure C6.31(a) of the *Commentary to the CHBDC* (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

6.9 Corrosion Assessment and Protection

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

6.9.1 Potential for Sulphate Attack

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

6.9.2 Potential for Corrosion

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

Borehole 2-2 sample 3 had a resistivity of 5,700 ohm-cm, indicating that the soil corrosiveness is generally Low (6,000 ohm-cm > R > 4,500 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. Ultimately, it is the designer's decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) and the CHBDC (2019) are satisfied.

6.10 Construction Considerations

6.10.1 Subgrade Preparation and Approach Embankment Construction

Prior to construction of the new approach embankments, it is recommended that all unsuitable soils such as topsoil or organics, and existing surficial fill materials or loosened/softened soils (e.g. re-worked soils from existing agricultural activities) be stripped from the embankment footprint and replaced with OPSS Select Subgrade Material (SSM), Granular A or Granular B material. Based on the boreholes, stripping of up to about 2 m below ground surface (and below groundwater level) may be required to remove the unsuitable soils at the approach embankments. As previously discussed, consideration can be given to leaving the existing native soft/firm clayey silt and loose silts in place, although stability and settlement analyses will need to be checked and mitigation measured provided (as applicable) when more detailed geotechnical information is available within the approach embankment footprints.

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

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

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

6.10.2 Temporary Excavations

In general, temporary excavations extending up to about 3 m below ground surface are required for shallow foundations (including subexcavation and replacement with a granular pad for a "perched" spread footing option), if being considered. Temporary excavations can be reduced to about 1.5 m (frost depth) to 2 m deep below ground surface for pile caisson caps and stripping below approach embankment footprints. If pile and/or caisson caps are "perched" within the approach embankments, temporary excavations could be reduced or eliminated at these locations.

All temporary excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. As per OHSA, the existing fill, loose silt, and the firm to stiff clayey silt-silt to clayey silt deposits (above the groundwater level or effectively dewatered) are classified as Type 3 soils, whereas the compact to dense sand to silty sand deposits (above the groundwater level or effectively dewatered) are classified as Type 2 soils. All soils encountered during the current investigation below the groundwater level would be classified as Type 4 soils.

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

within Type 3 and Type 2 soils (sloped to within 1.2 m of the bottom of the excavation) should be made with side slopes no steeper than 1H:1V.

Where required, temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*) and Special Provision 105S09. The lateral movement of the temporary protection systems must meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation.

6.10.3 Control of Groundwater / Surface Water

The groundwater level measured in the shallow piezometer installed in Borehole 2-1 generally fluctuated between about Elevation 219.9 m (about 0.5 m below ground surface) to 219.6 m (about 0.8 m below ground surface) when checked in May 2022 and March 2023. The groundwater level measured in the deep piezometer installed in Borehole 2-2 generally fluctuated between about Elevation 219.3 m (about 2 m below ground surface) to Elevation 218.5 m (about 2.8 m below ground surface) when checked in November 2022 and February 2023. As indicated from the groundwater level readings, the groundwater level at the site is anticipated to fluctuate seasonally and will likely be higher in the spring and fall months.

The excavations for stripping operations or shallow foundations (if applicable) are anticipated to extend about 2 m to 3 m below existing ground surface (about Elevation 219 m to 218 m) and will be 1 m to 2 m below the measured shallow groundwater level. Given the relatively flat, low-lying area, it is expected that advanced dewatering using well points prior to excavation will be required for the foundation excavations. For excavations less than 2 m below ground surface at localized areas for stripping operations, it is likely that groundwater could be controlled by trenching or diversion ditches with sufficient sumps and pumps. It is recommended that the groundwater level be lowered to at least 1 m below the base of the subexcavation level, resulting in temporary groundwater lowering of up to 3 to 4 m below the original ground surface (i.e., to about Elevation 218 m to 217 m). Dewatering operations should be in accordance with OPSS.PROV 517 (*Dewatering*) as referenced in OPSS.PROV 902 (*Excavation and Backfilling – Structures*). Inclusion of a special provision for foundation dewatering will need to be considered in the future contract documents or output specifications to address potential instability / base heave of the foundation subgrade, temporary flow diversion and pre-construction survey requirements, as applicable.

Construction water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self registration on the MECP's Environmental Activity and Sector Registry (EASR), requiring a "Water Taking Plan" and a "Discharge Plan" (to be developed by the Design-Build). A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. The contractor will be responsible for obtaining any required discharge approvals.

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

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

6.11 Recommendations for Additional Work

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

The additional investigation will need to explore the subsurface soil and groundwater conditions at the location of the bridge abutments, approach embankments and any associated retaining walls. Consideration should be given to advancing Cone Penetration Tests (seismic CPTs with dissipation testing) and/or pressuremeter tests to refine the settlement estimates and further characterize the clayey silt-silt to silty clay deposits encountered that are challenging to sample and interpret with conventional push equipment. Consideration could be given to using specialized piston samplers or longer tube samplers (as opposed to conventional thin-walled Shelby tube extraction methods) or mini-block samplers (similar to Sherbrooke sampler but smaller diameter) as an attempt to collect less disturbed samples of the clayey deposits containing silt/sand seams and additional consolidation tests (including larger diameter consolidation tests if larger diameter samples are collected) performed accordingly. In addition, the extent (bottom) and characteristics of the lower cohesive deposit and depth to competent soil (100-blow or hard / very dense soils) should be confirmed across the bridge and approach embankment footprints. Consideration could also be given to constructing a test fill pad with settlement monitoring at / near the site during design to improve prediction of settlement magnitude and rates.

After more detailed foundation investigation is complete, the global stability of the approach embankments and retaining walls will need to be checked and the magnitude of foundation settlements and any mitigation measures (including estimated preload times) will need to be reassessed, especially if the ultimate configuration of the bridge approach embankments is to be constructed at the interim stage. When more details are known on actual loading conditions, the foundation types, sizes and geotechnical resistances will need to be checked and revised as necessary. The use of GSC 5th Generation or 6th Generation seismic hazard maps to define the Site Class should be confirmed for detail design.

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 each borehole 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

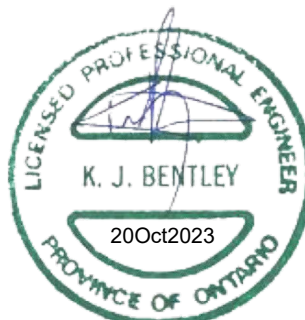
This Preliminary Foundation Design Report was prepared by Mr. Mark Henderson, P.Eng., a Geotechnical Engineer at WSP Golder. Mr. Kevin Bentley, P.Eng., a Geotechnical Engineer and MTO Foundations Designated Contact with WSP Golder, conducted a technical and quality review of the report.

Signature Page

WSP Golder



Mark Henderson, P.Eng.
Geotechnical Engineer



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

MH/KJB/ljv/al

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ASTM International

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

Canadian Standards Association (CSA):

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

Commercial Software:

Settle3 (Version 5.015) by Rocscience Inc.

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 804	Construction Specification for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
Special Provision 109F57	Amendment to OPSS.PROV 903

Ontario Regulations

Ontario Regulation 903 Wells (as amended)

Ontario Regulation 213 Construction Projects (as amended)

Ministry of Transportation, Ontario

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

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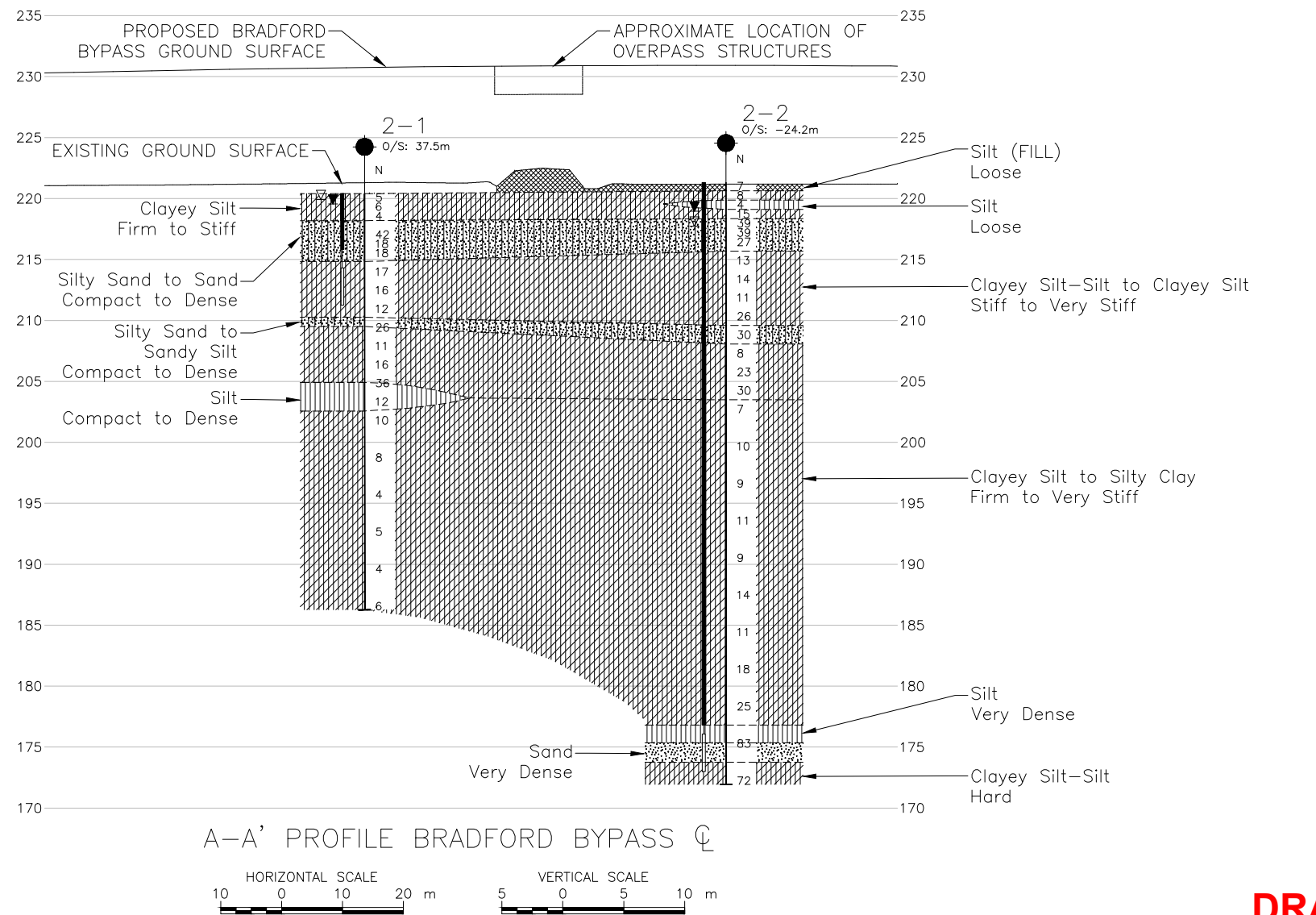
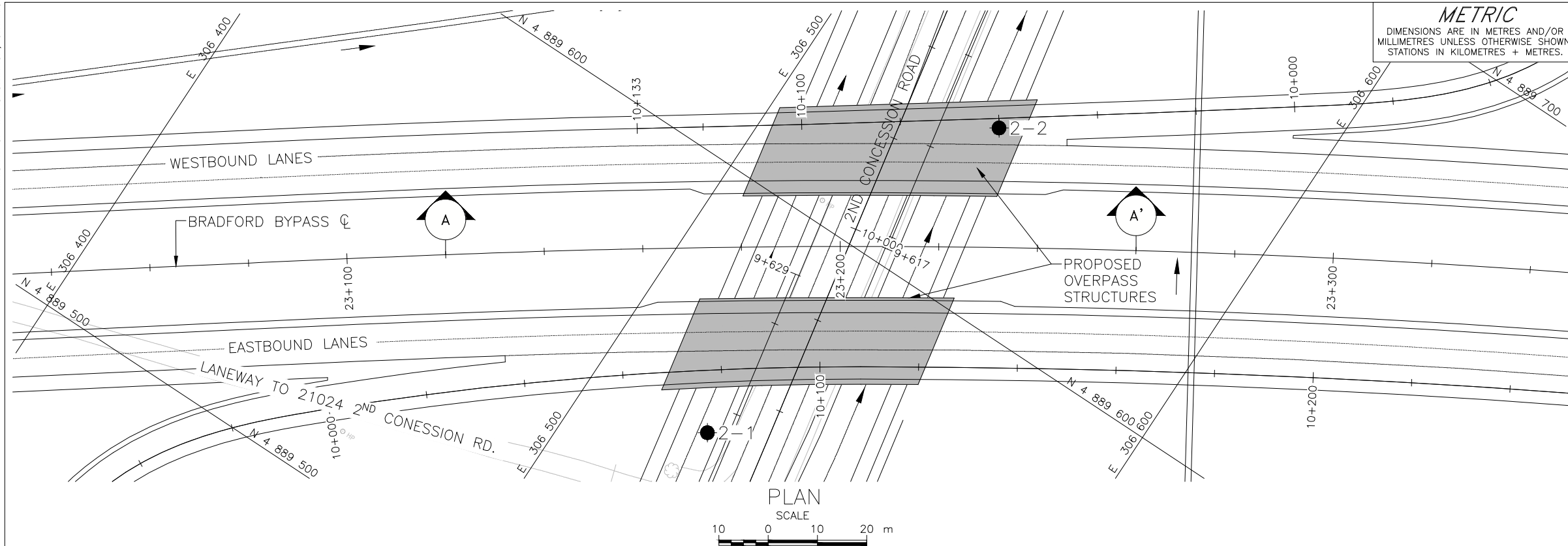
Provincial Engineering Memorandum #20201, Material Engineering and Research Office (MERO), March 23, 2020

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

Table 1: Comparison of Foundation Alternatives - 2nd Concession Road Overpass

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Strip / Spread footings founded on native compact to dense sand to silty sand	Marginally feasible at both bridge locations	<ul style="list-style-type: none">■ Conventional construction■ Founding soils provide adequate geotechnical resistance at f-ULS but marginally adequate f-SLS values.	<ul style="list-style-type: none">■ Excavation of unsuitable soils to about 3 m depth is required to reach competent founding stratum.■ Dewatering in saturated silts and sands will be required to allow for excavation and construction of footings in dry conditions and maintain stable foundation subgrade.■ Temporary protection systems may be needed if 2nd Concession Road is to remain open during construction; alternatively, closure or temporary realignment of 2nd Concession Road may also be required during construction.■ Does not allow for conventional integral abutment design.■ Preloading required to induce settlements from approach embankment loading	<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 may be encountered at preliminary founding level during detail design investigation at actual abutment locations.■ Risk of deeper excavation and increased dewatering and/or temporary shoring efforts.■ Risk of disturbance to founding subgrade if adequate dewatering is not provided in saturated sands and silts■ Preloading required and risk of higher than anticipated settlements and/or preload times from approach embankment loading make this option less practical and not preferred.
“Perched” abutment spread footings founded on a compacted granular pad within approach embankments	Marginally feasible at both bridge locations	<ul style="list-style-type: none">■ Conventional construction■ Granular pad can be constructed within approach embankment for abutment locations to improve geotechnical resistance values, although f-SLS values may still be marginally adequate.■ Founding level can easily be adjusted within approach embankment.■ Depth of excavation, dewatering effort, and height of abutment wall stems (i.e. volume of concrete / steel) can be reduced.	<ul style="list-style-type: none">■ Excavation of unsuitable soils to about 3 m depth is required to reach competent founding stratum.■ Dewatering of saturated silts and sands will be required to allow for subexcavation and placement and compaction of granular pad in dry conditions and maintain stable subgrade.■ Temporary protection systems may be needed if 2nd Concession Road is to remain open during construction; alternatively, closure or temporary realignment of 2nd Concession Road may also be required during construction.■ Does not allow for conventional integral abutment design.■ Preloading required to induce settlements from approach embankment loading	<ul style="list-style-type: none">■ Costs may be comparable to spread footings on native soil option given same costs 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 2nd Concession Road if adequate dewatering and shoring is not provided.■ Preloading required and risk of higher than anticipated settlements and/or preload times from approach embankment loading make this option less practical and not preferred.
Steel H-piles or tube piles driven to 30 m or driven to 45 m into very dense / hard end-bearing soils	Feasible at both bridge locations	<ul style="list-style-type: none">■ Conventional construction methods for driven steel pile foundations, although pile lengths are relatively long.■ Higher axial resistances available compared to shallow footings.■ Pile caps perched within approach embankments can be considered to reduce or eliminate dewatering / subexcavation requirements.■ Allows for integral abutment design.	<ul style="list-style-type: none">■ Noise and vibrations to adjacent properties, although limited residential properties are near the site.■ Dewatering measures may be required at abutments for the construction of pile caps, unless perched in embankment fill at abutments.■ Long piles in excess of 30 m may require heavier sections and larger pile driving equipment.	<ul style="list-style-type: none">■ Lower relative cost than drilled shafts (caissons) and may be comparable to spread footings if dewatering and subexcavation of unsuitable soils can be reduced by designing 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 lower geotechnical resistances during installation for friction pile design in predominantly clayey soils. A longer wait time to allow pore-water pressures to dissipate may be required to meet design values when testing production piles. Alternatively, advanced static load testing may be considered.■ Settlement of approach embankments will cause potential downdrag loads on piles (reduced capacity) which will need to be considered in design and/or mitigated and monitored during construction.■ Low 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).

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Drilled Shafts (Caissons) installed to 30 m	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 with structural columns to the underside of the superstructure.	<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 soil cuttings / slurry during drilled shaft advancement■ Does not allow for conventional integral abutment design.■ Long caissons up to 30 m deep may be challenging and not practical.	<ul style="list-style-type: none">■ Higher relative cost than shallow foundations.■ Higher cost than piles (per element) but reduced dewatering / subexcavation costs if caissons are cast continuously with structural columns to eliminate pile cap.■ Potential higher cost for access roads / platforms for caisson equipment and staging operations.	<ul style="list-style-type: none">■ Reduced impact on design if variable near surface soils is encountered during detailed investigation.■ In-situ testing to confirm design resistance is challenging and expensive (e.g. static pile load test) compared to PDA testing.■ Challenging to inspect the shaft and base of the drilled shafts due to the need for liners, slurry, and tremie concrete methods for relatively long caissons.■ Settlement of approach embankments will cause potential downdrag loads on caissons (reduced capacity) which will need to be considered in design and/or mitigated and monitored during construction.■ Low risk of difficulties penetrating through soil deposits that may contain pockets of gravel or cobbles and boulders (to be confirmed during detail design).



CONT No.
WP No.

BRADFORD BYPASS
2ND CONCESSION ROAD OVERPASS
BOREHOLE LOCATIONS AND SOIL
STRATA

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 November 2022 and February 2023
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
2-2	221.3	4889636.3	306541.2
2-1	220.4	4889552.2	306525.9

NOTES

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

REFERENCE

Base plans provided in digital format by Aecom, drawing file no. X-Base_Bradford Bypass.dwg and BRADFORD BY-PASS OG_Combined.xml, received January 11, 2022.
Horizontal alignment provided in digital format by Aecom, drawing file no. X-60636190-C-DES-All Alignments.xml, received May 12, 2023.
Design plan provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Overall Plan.dwg, received May 12, 2023.
Vertical alignment provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Mainline Profile.dwg, received May 16, 2023.

NO.	DATE	BY	REVISION

Geocres No. 31D00-827

HWY.	PROJECT NO.	DIST.
SUBM'D. KJB	19136074	
DRAWN: DD	DATE: 10/17/2023	SITE:
CHKD. KJB	APPD. KJB	DWG. 1

DRAFT

APPENDIX A

Borehole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

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

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

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	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 = σ'_p / σ'_{vo}

(d) Shear Strength

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

Notes: 1
2

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

PROJECT 19136074

RECORD OF BOREHOLE

No. 2-1

Sheet 1 of 4

METRIC

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

LOCATION N 4889552.2; E 306525.9 NAD83 / MTM Zone 10 (LAT. 44.145964; LONG. -79.478427)

ORIGINATED BY MTI

DIST Central HWY BBP - 2nd Concession

BOREHOLE TYPE Hollow Stem Auger, Mud Rotary

COMPILED BY DP

DATUM CGVD28 Surface Elevation:220.4 m

DATE Dec 21, 2021 - Dec 22, 2021

CHECKED BY KJB

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), trace sand, trace rootlets Soft to firm Brown and Grey Moist Oxidation staining		1	SS	5		220														
			2	SS	6		219														
			3	SS	4												0	2	86	12	
218.2																					
2.2	SILTY SAND to SAND (SM-SP), trace clay, trace gravel Compact to Dense Grey Wet		4	TO			218														See Note 4.
			5	SS	42		217										0	89	10	1	
			6	SS	18		216														
			7	SS	18																
214.8							215														
5.6	CLAYEY SILT (CL), trace sand, trace gravel Stiff to very stiff Grey Moist		8	SS	17		214														
							213														
	- 8.0 m: Silty Sand layer (100 mm thick)		9	SS	16		212														
			10	SS	12		211										0	0	69	31	
	- 9.6 m: Silty Sand layer (100 mm thick)																				

Continued on Next Page

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

PROJECT 19136074

RECORD OF BOREHOLE

No. 2-1

Sheet 2 of 4

METRIC

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

LOCATION N 4889552.2; E 306525.9 NAD83 / MTM Zone 10 (LAT. 44.145964; LONG. -79.478427)

ORIGINATED BY MTI

DIST Central HWY BBP - 2nd Concession

BOREHOLE TYPE Hollow Stem Auger, Mud Rotary

COMPILED BY DP

DATUM CGVD28 Surface Elevation:220.4 m

DATE Dec 21, 2021 - Dec 22, 2021

CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y	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						
210.3 10.1	CLAYEY SILT (CL), trace sand, trace gravel Stiff to very stiff Grey Moist						210														
209.5 10.9	SILTY SAND (SM), Compact Grey Moist		11 A	SS	26																
	CLAYEY SILT (CL), contains sand seams/layers Stiff to hard Grey Moist		11 B				209														
			12	SS	11		208														
							207														
	- 13.8 m: Silty Sand layer (25 mm thick)																				
							206														
204.9 15.5	SILT (ML), trace sand Compact to Dense Grey Moist		14 A	SS	36		205														
			14 B																		
							204														
			15	SS	12		203														
202.6 17.8	CLAYEY SILT (CL), trace sand, trace gravel Firm to stiff Grey Moist						202														
			16	SS	10																
							201														

Continued on Next Page

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

PROJECT	19136074	RECORD OF BOREHOLE	No. 2-1	Sheet 3 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4889552.2; E 306525.9 NAD83 / MTM Zone 10 (LAT. 44.145964; LONG. -79.478427)	ORIGINATED BY	MTI
DIST	Central	HWY	BBP - 2nd Concession	BOREHOLE TYPE	Hollow Stem Auger, Mud Rotary
DATUM	CGVD28 Surface Elevation:220.4 m	DATE	Dec 21, 2021 - Dec 22, 2021	COMPILED BY	DP
				CHECKED BY	KJB

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								
	CLAYEY SILT (CL), trace sand, trace gravel Firm to stiff Grey Moist						200														
			17	SS	8		199										0	1	80	19	
	- 22.1 m: Attempted Shelby Tube sample but no recovery.		18	TO			198														
							197														
			19	SS	4		196														
			20	TO			195														C
							194														
			21	SS	5		193														
							192														
							191														

Continued on Next Page

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

PROJECT 19136074

RECORD OF BOREHOLE

No. 2-1

Sheet 4 of 4

METRIC

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

LOCATION N 4889552.2; E 306525.9 NAD83 / MTM Zone 10 (LAT. 44.145964; LONG. -79.478427)

ORIGINATED BY MTI

DIST Central HWY BBP - 2nd Concession

BOREHOLE TYPE Hollow Stem Auger, Mud Rotary

COMPILED BY DP

DATUM CGVD28 Surface Elevation:220.4 m

DATE Dec 21, 2021 - Dec 22, 2021

CHECKED BY KJB

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								
	CLAYEY SILT (CL), trace sand, trace gravel Firm to stiff Grey Moist		22	SS	4		190														
							189														
							188														
							187														
	- 33.5 m: Transition to silty clay		23	SS	6																
186.3																					
34.1	End of Borehole Note: 1. Hollow stem augers to 2.3 m (Elev. 218.1 m) and then switched to mud rotary. 2. Water level measured at a depth of 0.1 m (Elev. 220.3 m) prior to mud rotary. 3. Water level in standpipe piezometer measured as follows: Depth(m) El. (m) Date 0.77 219.6 May 12, 22 0.81 219.6 May 13, 22 0.47 219.9 Feb 01, 23 4. Attempted shelly tube sample but limited recovery of silty sand.						186														
							185														
							184														
							183														
							182														
							181														

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

PROJECT 19136074

RECORD OF BOREHOLE

No. 2-2

Sheet 1 of 5

METRIC

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

LOCATION N 4889636.3; E 306541.2 NAD83 / MTM Zone 10 (LAT. 44.146721; LONG. -79.478236)

ORIGINATED BY PT

DIST Central HWY BBP - 2nd Concession





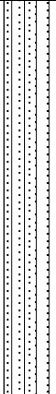

BOREHOLE TYPE Hollow Stem Auger, Mud Rotary

COMPILED BY MCK

DATUM CGVD28 Surface Elevation:221.3 m

DATE Oct 27, 2022 - Nov 02, 2022

CHECKED BY KJB

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	20	40	60	NP Nonplastic						
0.0	SILT (ML), some sand, trace rootlets, (FILL) Loose Brown to grey Moist		1	SS	7		221														
220.6																					
0.7	CLAYEY SILT (CL) Firm Mottled grey and brown, oxidation staining Moist		2	SS	8		220														
219.9																					
1.4	SILT (ML), trace sand Loose Brown to grey, oxidation staining Wet		3	SS	4																
219.1																					
2.2	CLAYEY SILT (CL), contains silty sand layers Stiff Brown to grey, oxidation staining Moist		4	SS	15		219														
218.3																					
3.0	SILTY SAND to SAND (SM-SP) Compact to Dense Grey Wet		5	SS	39		218										0	79	(21)		
			6	SS	39		217														
			7	SS	27																
							216														
215.7																					
5.6	CLAYEY SILT-SILT (CL-ML), trace sand, trace gravel, contains silt and sand seams/layers Stiff to very stiff Grey Moist		8	SS	13		215										0	2	82	16	
							214														
			9	SS	14		213														
			10	SS	11		212														

Continued on Next Page

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

METRIC

CHECKED BY KJB

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

METRIC

CHECKED BY KJB

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

PROJECT 19136074		RECORD OF BOREHOLE No. 2-2		Sheet 4 of 5		METRIC	
G.W.P. Assignment No 2019-E-0048		LOCATION N 4889636.3; E 306541.2 NAD83 / MTM Zone 10 (LAT. 44.146721; LONG. -79.478236)		ORIGINATED BY PT			
DIST Central HWY BBP - 2nd Concession		BOREHOLE TYPE Hollow Stem Auger, Mud Rotary		COMPILED BY MCK			
DATUM CGVD28 Surface Elevation:221.3 m		DATE Oct 27, 2022 - Nov 02, 2022		CHECKED BY KJB			

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 W _p	NMC W	LL W _L						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100					20 40 60								
	CLAYEY SILT (CL) to SILTY CLAY (CI), trace sand Firm to stiff Grey Moist		20	SS	9		191														
							190														
							189														
							188														
			21	SS	14		187														
							186														
186.0 35.4	Sandy CLAYEY SILT (CL), trace to some gravel Stiff to very stiff Grey Moist						185														
			22	SS	11		184														
							183														
							182														
			23	SS	18																

Continued on Next Page

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

PROJECT 19136074

RECORD OF BOREHOLE

No. 2-2

Sheet 5 of 5

METRIC

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

LOCATION N 4889636.3; E 306541.2 NAD83 / MTM Zone 10 (LAT. 44.146721; LONG. -79.478236)

ORIGINATED BY PT

DIST Central HWY BBP - 2nd Concession

BOREHOLE TYPE Hollow Stem Auger, Mud Rotary

COMPILED BY MCK

DATUM CGVD28 Surface Elevation:221.3 m

DATE Oct 27, 2022 - Nov 02, 2022

CHECKED BY KJB

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													
								20 40 60 80 100					20 40 60								
	Sandy CLAYEY SILT (CL), trace to some gravel Stiff to very stiff Grey Moist						181														
179.9							180														
41.5	CLAYEY SILT (CL) Very stiff Grey Moist						179														
			24	SS	25		178														
							177														
176.8							176														
44.5	SILT (ML), some sand Very dense Grey Wet						175														
175.3			25 A	SS	83		174														
46.0	SAND (SP), trace silt Very dense Grey Wet		25 B				173														
173.8							172														
47.5	CLAYEY SILT-SILT (CL-ML) Hard Grey Moist																				
			26	SS	72																
171.9																					
49.4	End of Borehole Note: 1. Hollow stem augers to 3.0 m (Elev. 218.3 m) and then switched to mud rotary. 2. Water level measured at a depth of 3.2 m (Elev. 218.1 m) prior to mud rotary.																				

3. Water level in standpipe piezometer measured as follows:

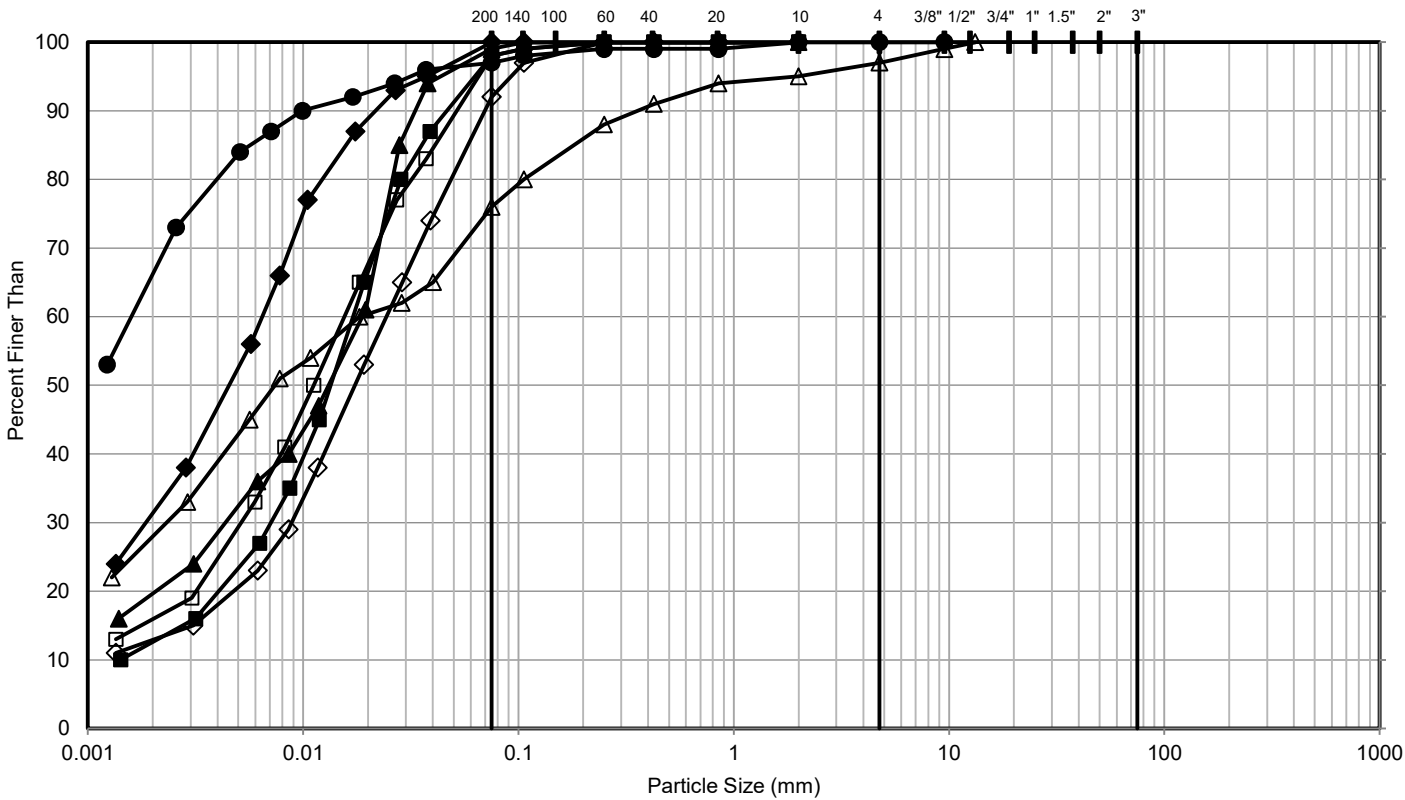
+³, x³ : Numbers refer to Sensitivity o³⁰% STRAIN AT FAILURE

Depth(m)	El. (m)	Date
0.73	220.6	Nov 03, 22
2.63	218.7	Nov 04 22
1.98	219.3	Feb 01, 23
2.77	218.5	Feb 28, 23

APPENDIX B

Geotechnical Laboratory Test Results

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



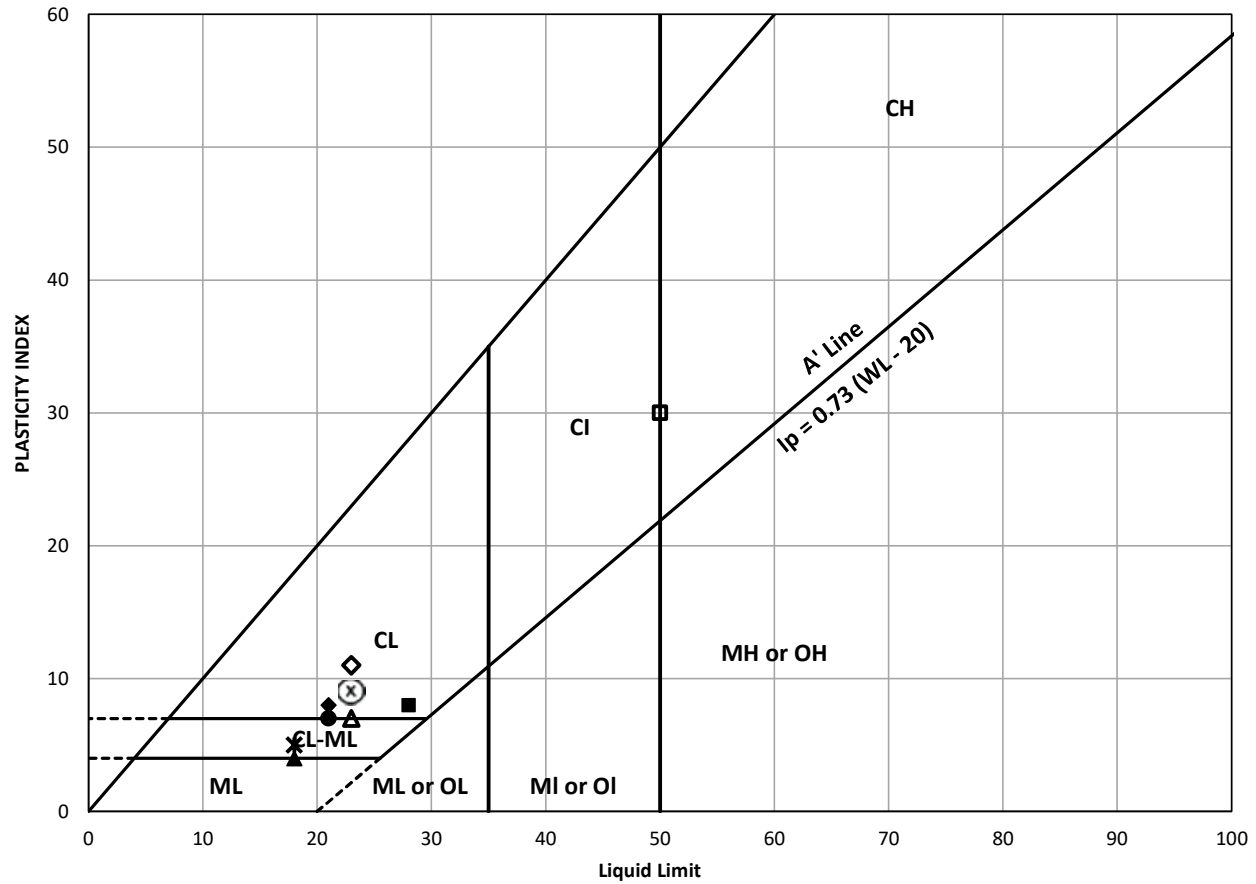
FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	2-1	3	1.5 - 2.1	218.9 to 218.3
◆	2-1	10	9.1 - 9.8	211.3 to 210.7
▲	2-1	17	21.3 - 22.0	199.1 to 198.5
●	2-1	23	33.5 - 34.1	186.9 to 186.3
□	2-2	8	6.1 - 6.7	215.2 to 214.6
◇	2-2	14	15.2 - 15.9	206.1 to 205.5
△	2-2	22	36.6 - 37.2	184.7 to 184.1

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - 2nd Concession Road Overpass	
CONSULTANT	YYYY-MM-DD	2023-03-30	
	DESIGNED	MH	
	PREPARED	MH	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		Plasticity Chart	
		Clayey Silt-Silt (CL-ML) to Silty Clay (CI-CH)	
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B1



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



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	2-1	3	1.5 - 2.1	24.6	28	20	8	
◆	2-1	19	24.4 - 25.0	25.1	21	13	8	
▲	2-2	8	6.1 - 6.7	19.9	18	14	4	
●	2-2	10	9.1 - 9.8	20.2	21	14	7	
✱	2-2	14	15.2 - 15.9	18.3	18	13	5	
⊗	2-2	17	21.3 - 22.0	23.2	23	14	9	
□	2-2	20	30.5 - 31.1	38.1	50	20	30	
◇	2-2	22	36.6 - 37.2	17.4	23	12	11	
△	2-2	26	48.8 - 49.4	16.8	23	16	7	

CLIENT

AECOM / MTO

CONSULTANT

wsp GOLDER

YYYY-MM-DD 2023-03-30

DESIGNED MH

PREPARED MH

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - 2nd Concession Road Overpass

TITLE

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

PROJECT NO.

19136074

CONTROL

0

REV.

0

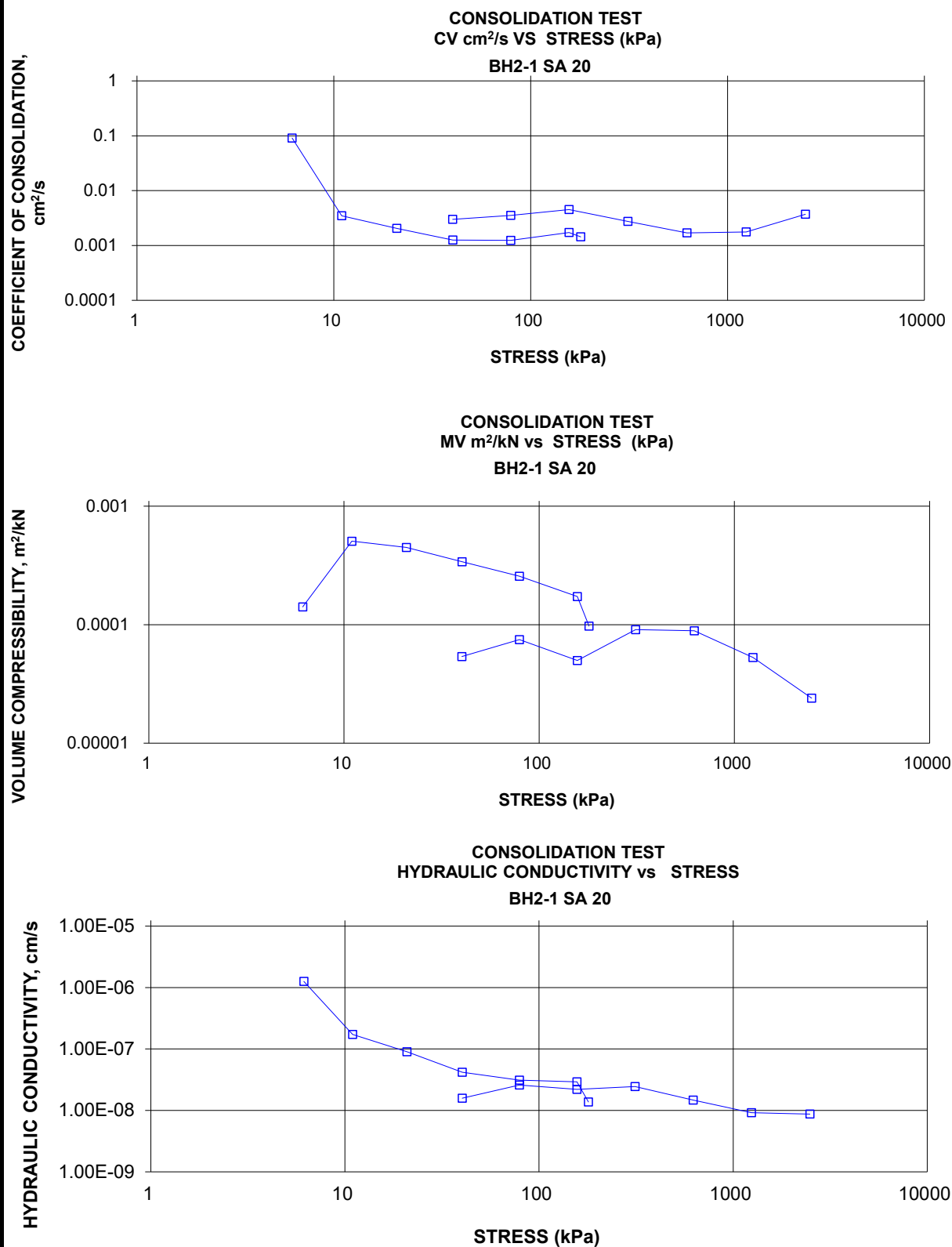
FIGURE

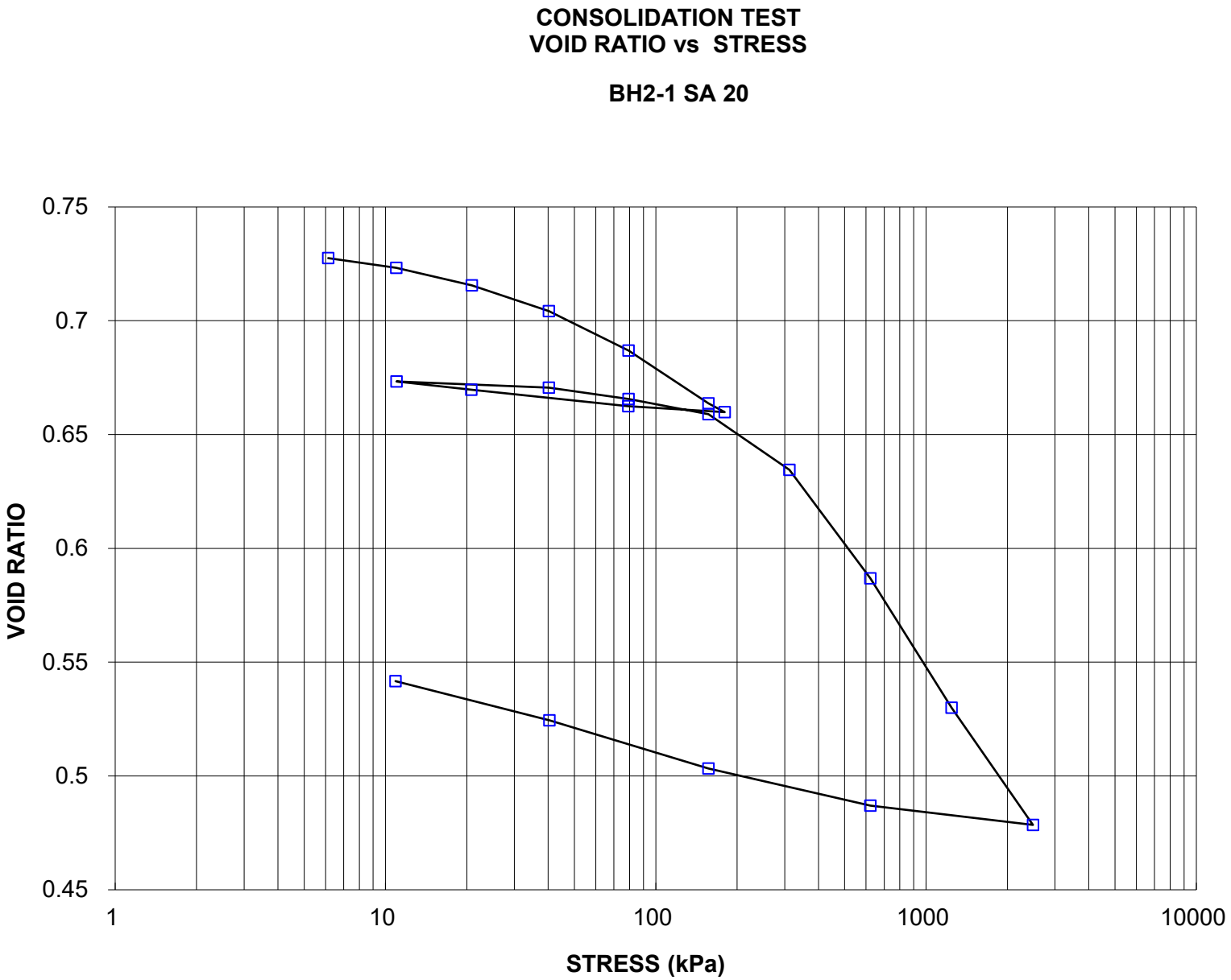
B2

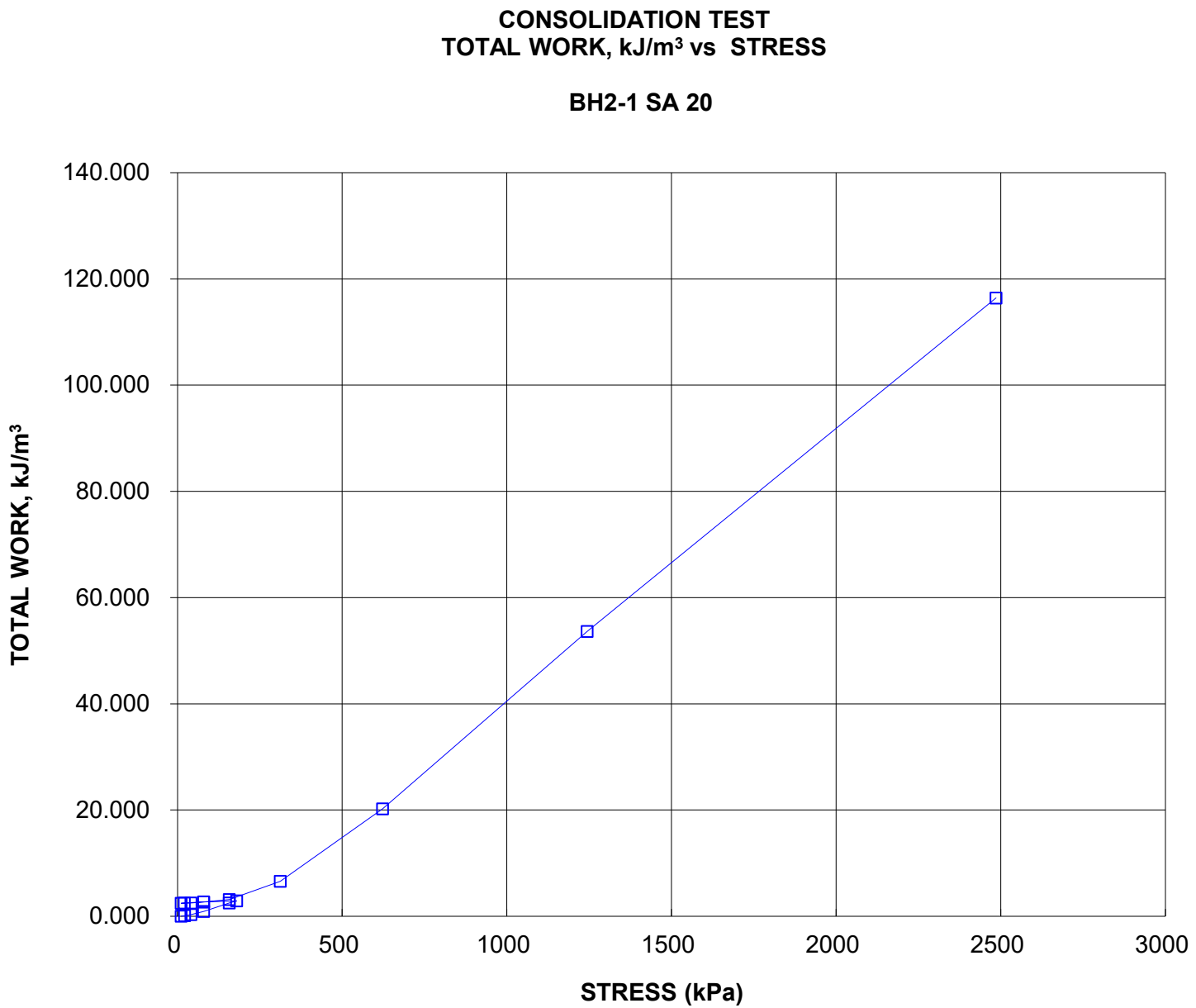
CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE B3		
SAMPLE IDENTIFICATION							
Project Number		19136074		Sample Number		20	
Borehole Number		BH2-1		Sample Depth, m		25.15-25.76	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		4					
Date Started		01/21/2022					
Date Completed		02/07/2022					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.54		Unit Weight, kN/m ³		19.62	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m ³		15.48	
Area, cm ²		31.54		Specific Gravity, measured		2.73	
Volume, cm ³		80.05		Solids Height, cm		1.468	
Water Content, %		26.70		Volume of Solids, cm ³		46.30	
Wet Mass, g		160.14		Volume of Voids, cm ³		33.75	
Dry Mass, g		126.39		Degree of Saturation, %		100.0	
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	2.538	0.729	2.538				
6.14	2.536	0.728	2.537	15	9.10E-02	1.41E-04	1.26E-06
10.96	2.530	0.723	2.533	392	3.47E-03	5.07E-04	1.72E-07
20.88	2.518	0.716	2.524	660	2.05E-03	4.49E-04	9.00E-08
40.24	2.502	0.704	2.510	1066	1.25E-03	3.40E-04	4.17E-08
79.22	2.476	0.687	2.489	1060	1.24E-03	2.57E-04	3.12E-08
156.66	2.442	0.664	2.459	747	1.72E-03	1.73E-04	2.91E-08
179.71	2.437	0.660	2.439	877	1.44E-03	9.73E-05	1.37E-08
79.14	2.440	0.662	2.438				
20.76	2.451	0.670	2.446				
10.99	2.456	0.673	2.454				
40.19	2.452	0.671	2.454	427	2.99E-03	5.38E-05	1.58E-08
79.22	2.445	0.666	2.449	359	3.54E-03	7.47E-05	2.59E-08
156.78	2.435	0.659	2.440	279	4.52E-03	4.98E-05	2.21E-08
312.05	2.399	0.635	2.417	452	2.74E-03	9.08E-05	2.44E-08
622.54	2.329	0.587	2.364	702	1.69E-03	8.88E-05	1.47E-08
1243.78	2.246	0.530	2.288	628	1.77E-03	5.29E-05	9.16E-09
2485.50	2.170	0.479	2.208	279	3.70E-03	2.40E-05	8.71E-09
622.54	2.183	0.487	2.177				
156.78	2.207	0.503	2.195				
40.43	2.238	0.525	2.222				
10.89	2.263	0.542	2.250				
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 6-13cm from bottom of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		2.26		Unit Weight, kN/m ³		20.87	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m ³		17.37	
Area, cm ²		31.54		Specific Gravity, measured		2.73	
Volume, cm ³		71.37		Solids Height, cm		1.468	
Water Content, %		20.20		Volume of Solids, cm ³		46.30	
Wet Mass, g		151.92		Volume of Voids, cm ³		25.08	
Dry Mass, g		126.39					
Prepared By: LH		Golder Associates				Checked By: MM	

CONSOLIDATION TEST SUMMARY

FIGURE B4

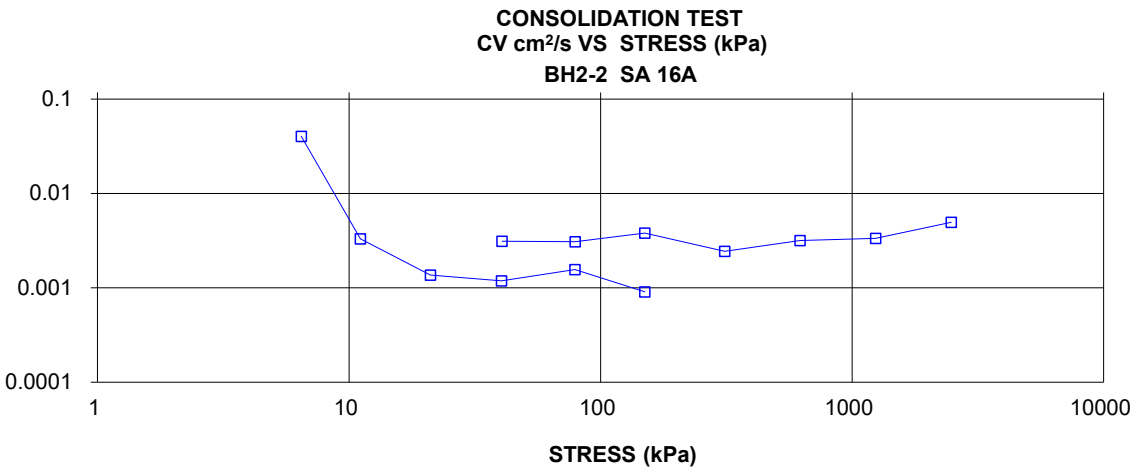




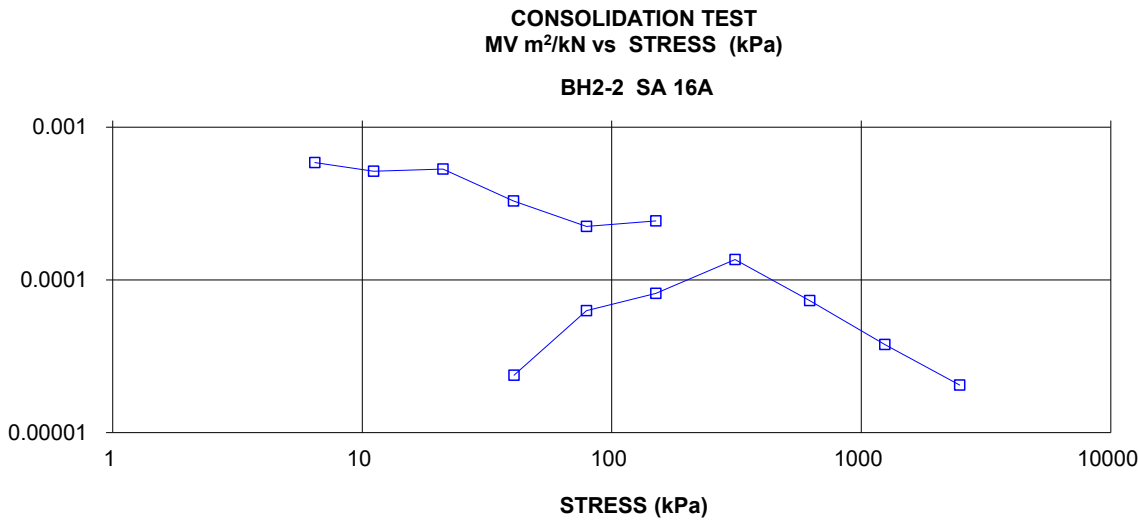


CONSOLIDATION TEST SUMMARY					FIGURE B7		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		19136074		Sample Number		16A	
Borehole Number		2-2		Sample Depth, m		19.05	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		8					
Date Started		02/07/2023					
Date Completed		02/22/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		1.91		Unit Weight, kN/m ³		19.82	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m ³		15.81	
Area, cm ²		31.61		Specific Gravity, measured		2.73	
Volume, cm ³		60.22		Solids Height, cm		1.125	
Water Content, %		25.38		Volume of Solids, cm ³		35.55	
Wet Mass, g		121.69		Volume of Voids, cm ³		24.66	
Dry Mass, g		97.06		Degree of Saturation, %		99.9	
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	1.905	0.694	1.905				
6.46	1.898	0.687	1.901	19	4.03E-02	5.87E-04	2.32E-06
11.10	1.893	0.683	1.896	230	3.31E-03	5.15E-04	1.67E-07
21.06	1.883	0.674	1.888	554	1.36E-03	5.33E-04	7.12E-08
40.38	1.871	0.663	1.877	628	1.19E-03	3.29E-04	3.84E-08
79.15	1.854	0.649	1.863	470	1.57E-03	2.24E-04	3.43E-08
150.08	1.822	0.620	1.838	789	9.08E-04	2.43E-04	2.16E-08
40.57	1.823	0.621	1.822				
11.14	1.830	0.627	1.826				
40.57	1.829	0.626	1.829	228	3.11E-03	2.37E-05	7.23E-09
79.27	1.824	0.622	1.826	230	3.07E-03	6.28E-05	1.89E-08
150.08	1.813	0.612	1.819	184	3.81E-03	8.17E-05	3.05E-08
311.77	1.771	0.575	1.792	279	2.44E-03	1.36E-04	3.25E-08
621.55	1.728	0.536	1.750	205	3.17E-03	7.32E-05	2.27E-08
1241.14	1.683	0.497	1.706	184	3.35E-03	3.77E-05	1.24E-08
2479.31	1.635	0.454	1.659	118	4.95E-03	2.05E-05	9.93E-09
621.55	1.642	0.460	1.639				
156.99	1.657	0.473	1.649				
40.33	1.680	0.494	1.668				
11.14	1.698	0.509	1.689				
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)							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.70		Unit Weight, kN/m ³		21.09	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m ³		17.74	
Area, cm ²		31.61		Specific Gravity, measured		2.73	
Volume, cm ³		53.67		Solids Height, cm		1.125	
Water Content, %		18.93		Volume of Solids, cm ³		35.55	
Wet Mass, g		115.43		Volume of Voids, cm ³		18.11	
Dry Mass, g		97.06					
Prepared By: LH				WSP Golder		Checked By: MM	

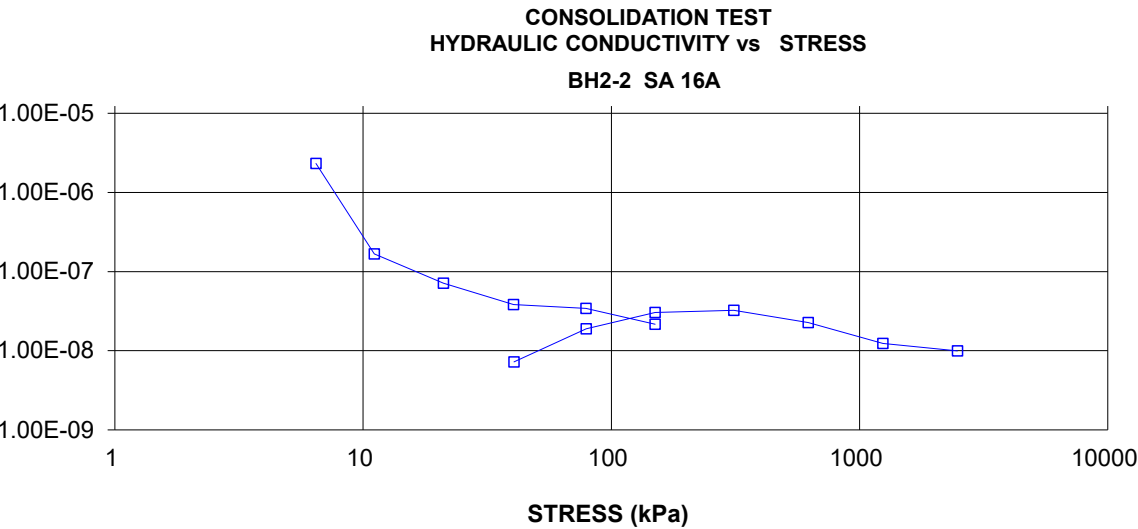
COEFFICIENT OF CONSOLIDATION,
cm²/s

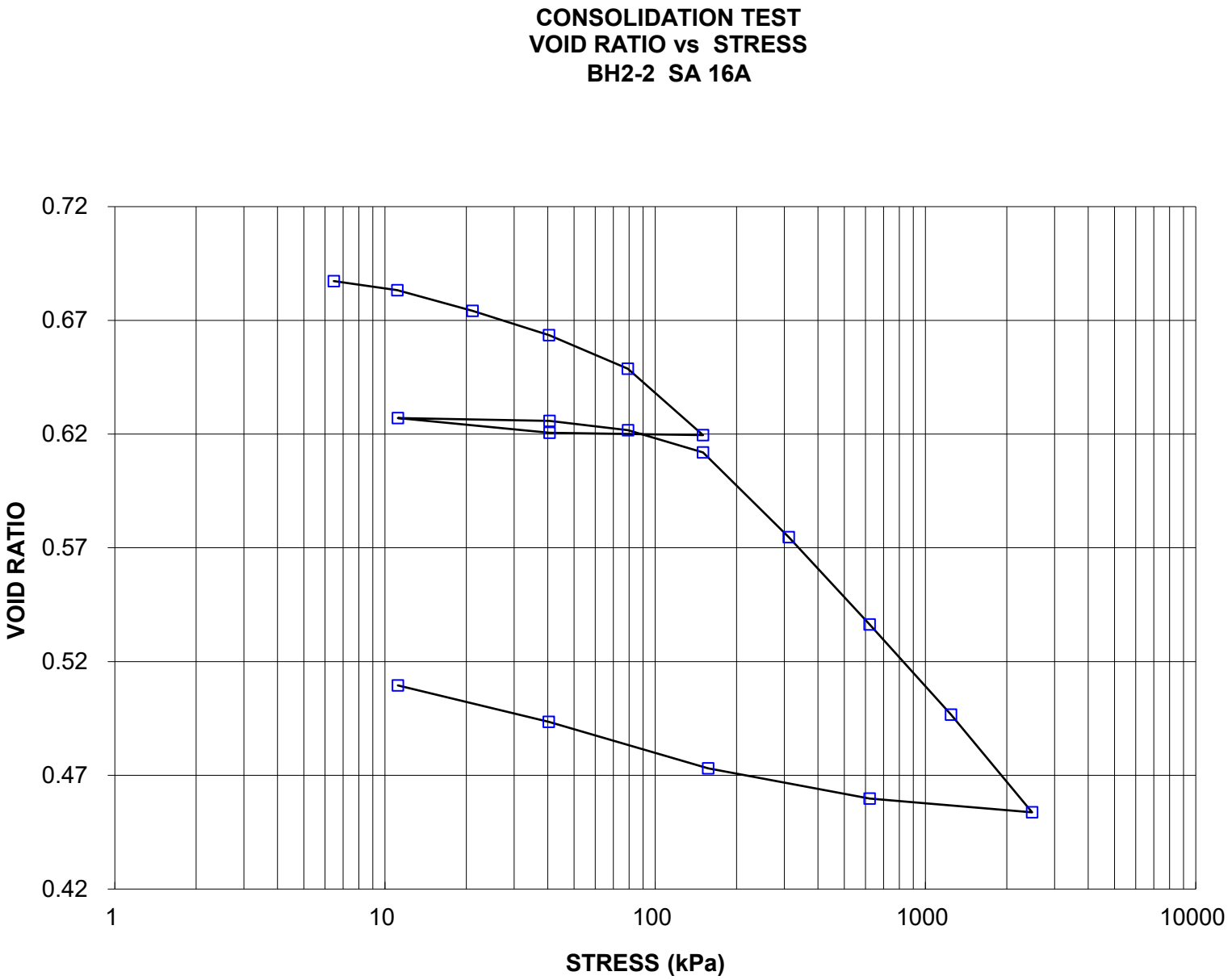


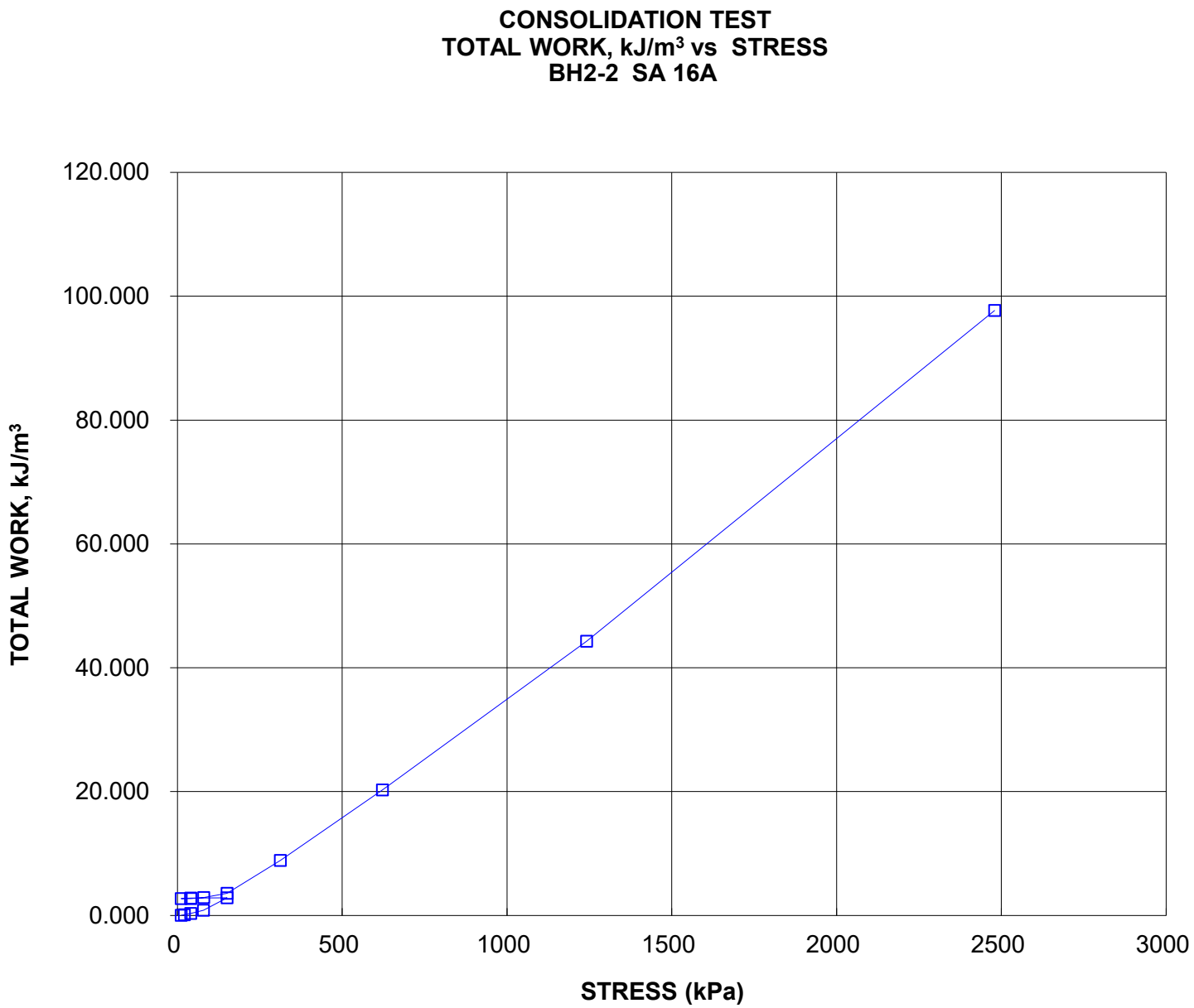
VOLUME COMPRESSIBILITY, m²/kN



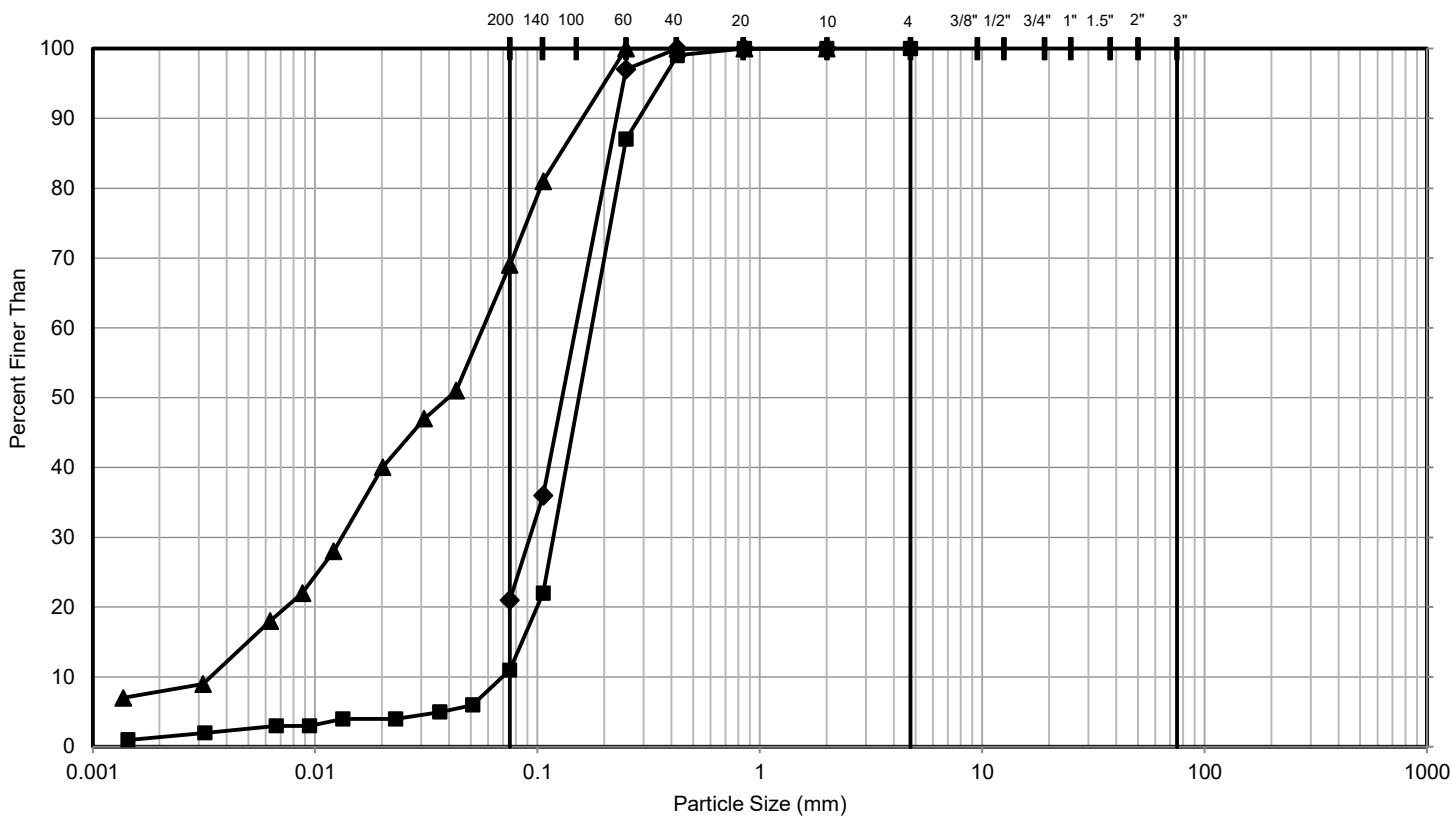
HYDRAULIC CONDUCTIVITY, cm/s







Grain Size Distribution - Sand, Silty Sand, and Sandy Silt (Non-Cohesive Seams / Interlayers)



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	2-1	5	3.1 - 3.7	217.4 to 216.7
◆	2-2	5	3.1 - 3.7	218.3 to 217.7
▲	2-2	12	12.2 - 12.8	209.1 to 208.5

CLIENT

AECOM / MTO

CONSULTANT

WSP

GOLDER

YYYY-MM-DD

2023-03-30

DESIGNED

MH

PREPARED

MH

REVIEWED

KJB

APPROVED

KJB

PROJECT

Bradford Bypass - 2nd Concession Road Overpass

TITLE

Grain Size Distribution
Sand, Silty Sand, and Sandy Silt (Non-Cohesive Seams / Interlayers)

PROJECT NO.

19136074

CONTROL

0

REV.

0

FIGURE

B11

APPENDIX C

Analytical Chemical Test Results



Your P.O. #: 19136074
Your Project #: 19136074
Site Location: BRADFORD
Your C.O.C. #: NA

Attention: Muhammad Talha Irshad

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

Report Date: 2023/03/29
Report #: R7565675
Version: 2 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BUREAU VERITAS JOB #: C2AD661

Received: 2022/12/12, 10:15

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	1	2022/12/16	2022/12/16	CAM SOP-00463	MOE E3013 m
Conductivity	1	2022/12/16	2022/12/16	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	1	N/A	2022/12/16	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	1	N/A	2022/12/19	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	1	2022/12/19	2022/12/19	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2022/12/13	2022/12/16	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	2022/12/16	2022/12/16	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 P.O. #: 19136074
Your Project #: 19136074
Site Location: BRADFORD
Your C.O.C. #: NA

Attention: Muhammad Talha Irshad

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

Report Date: 2023/03/29
Report #: R7565675
Version: 2 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BUREAU VERITAS JOB #: C2AD661

Received: 2022/12/12, 10:15

Encryption Key

Please direct all questions regarding this Certificate of Analysis to:

Ankita Bhalla, Project Manager

Email: Ankita.Bhalla@bureauveritas.com

Phone# (905) 817-5700

=====

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



BUREAU
VERITAS

Bureau Veritas Job #: C2AD661

Report Date: 2023/03/29

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MT

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		UOB264			UOB264		
Sampling Date		2022/10/27			2022/10/27		
COC Number		NA			NA		
	UNITS	2-2 5'-7	RDL	QC Batch	2-2 5'-7 Lab-Dup	RDL	QC Batch
Calculated Parameters							
Resistivity	ohm-cm	5700		8400535			
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	8408097			
Conductivity	umho/cm	177	2	8408336			
Available (CaCl2) pH	pH	7.74		8411397	7.72		8411397
Soluble (20:1) Sulphate (SO4)	ug/g	21	20	8408082			
Sulphide	mg/kg	5.2 (1)	0.5	8417625	4.4	0.5	8417625
Physical Testing							
Moisture-Subcontracted	%	16	0.30	8409298			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Sample contained greater than 10% headspace at time of extraction. Analyzed past method specified hold time							



BUREAU
VERITAS

Bureau Veritas Job #: C2AD661
Report Date: 2023/03/29

WSP Canada Inc.
Client Project #: 19136074
Site Location: BRADFORD
Your P.O. #: 19136074
Sampler Initials: MT

TEST SUMMARY

Bureau Veritas ID: UOB264
Sample ID: 2-2 5'-7
Matrix: Soil

Collected: 2022/10/27
Shipped:
Received: 2022/12/12

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8408097	2022/12/16	2022/12/16	Samuel Law
Conductivity	AT	8408336	2022/12/16	2022/12/16	Gurpartee K AUR
Moisture (Subcontracted)	BAL	8409298	N/A	2022/12/16	Simranjeet Batth
Sulphide in Soil	SPEC	8417625	N/A	2022/12/19	Bailey Morrison
pH CaCl2 EXTRACT	AT	8411397	2022/12/19	2022/12/19	Taslima Aktar
Resistivity of Soil		8400535	2022/12/16	2022/12/16	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8408082	2022/12/16	2022/12/16	Samuel Law

Bureau Veritas ID: UOB264 Dup
Sample ID: 2-2 5'-7
Matrix: Soil

Collected: 2022/10/27
Shipped:
Received: 2022/12/12

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC	8417625	N/A	2022/12/19	Bailey Morrison
pH CaCl2 EXTRACT	AT	8411397	2022/12/19	2022/12/19	Taslima Aktar



BUREAU
VERITAS

Bureau Veritas Job #: C2AD661

Report Date: 2023/03/29

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MT

GENERAL COMMENTS

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

Package 1	4.0°C
-----------	-------

Revised Report [2023/03/29]: Split report required as per client request.

Results relate only to the items tested.



BUREAU
VERITAS

Bureau Veritas Job #: C2AD661

Report Date: 2023/03/29

QUALITY ASSURANCE REPORT

WSP Canada Inc.

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MT

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8408082	Soluble (20:1) Sulphate (SO ₄)	2022/12/16	NC	70 - 130	102	70 - 130	<20	ug/g	3.0	35
8408097	Soluble (20:1) Chloride (Cl ⁻)	2022/12/16	117	70 - 130	107	70 - 130	<20	ug/g	NC	35
8408336	Conductivity	2022/12/16			105	90 - 110	<2	umho/cm	1.5	10
8409298	Moisture-Subcontracted	2022/12/16					<0.30	%	8.1	20
8411397	Available (CaCl ₂) pH	2022/12/19			100	97 - 103			0.24	N/A
8417625	Sulphide	2022/12/19	118	75 - 125	118	75 - 125	<0.5	mg/kg	16	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 #: C2AD661

Report Date: 2023/03/29

WSP Canada Inc.

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VALIDATION SIGNATURE PAGE

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

Veronica Falk, B.Sc., P.Chem., QP, Scientific Specialist, Organics

Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist

Suwan (Sze Yeung) Fock, B.Sc., Scientific Specialist

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



6740 Campobello Road, Mississauga, Ontario L5N 2L8
 Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
 CAM FCD-01191/5

WORK ORDER

CHAIN OF CUSTODY RECORD

Page 1 of 1

Invoice Information		Report Information (If differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: Golder Associates Ltd.		Company Name: same		Quotation #: Golder rates		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name: Kevin Bentley		Contact Name: Muhammad Talha Irshad		P.O. #/ AFE#: 19136074		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address: 6925 Century Ave., Suite 100		Address: 6925 Century Ave., Suite 100		Project #: 19136074		Rush TAT (Surcharges will be applied)	
Mississauga, ON		Mississauga, ON		Site Location: Bradford		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days	
Phone: 905-567-6100 Fax:		Phone: 778-228-5756 Fax:		Site #:		Date Required:	
Email: gld.canadaaccounts@bureauveritas.com; kevin.bentley@bureauveritas.com		Email: muhammad.irshad@bureauveritas.com; 120387@golder.com		Site Location Province: Ontario		Rush Confirmation #:	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS LABORATORIES' DRINKING WATER CHAIN OF CUSTODY							
Regulation 153		Other Regulations		Analysis Requested		LABORATORY USE ONLY	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		# OF CONTAINERS SUBMITTED FIELD FILTERED (CIRCLE) Metals / Hg / CrVI BTEX/ PHC F1 PHC F2 - F4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B) CORROSIVITY PACKAGE (+ SULPHIDE)		CUSTODY SEAL Y / N Present Intact COOLER TEMPERATURES COOLING MEDIA PRESENT: Y / N COMMENTS	
Include Criteria on Certificate of Analysis: Y / N							
SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI
1	2-2 5'-7'	2022/10/27	PM	Soil	2		
2	400-2 0'-2'	2022/11/09	PM	Soil	2		
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)
Muhammad Talha Irshad		2022-12-12		<i>[Signature]</i>			
<i>[Signature]</i>							

MSA with BV Signed May 18, 2020.
 Golder standing offer rates in email from Julie Clement dated Sept 20, 2021.
 Corrosivity package including chloride, conductivity, resistivity, pH, sulphate, sulphide is \$98.60/sample.

12-Dec-22 10:15

Ankita Bhalla

C2AD661

MUM ENV-1744

Unless otherwise agreed to in writing, work submitted on this Chain of Custody is subject to Bureau Veritas Laboratories' standard Terms and Conditions. Signing of this Chain of Custody document is acknowledgment at <http://www.bvlabs.com/terms-and-conditions>



golder.com