FINAL REPORT

Preliminary Foundation Investigation and Design Report

Bradford Bypass / Highway 400 Interchange Ramp Structures (E-S Ramp Over Highway 400, N-E Ramp Over Highway 400 / E-S Ramp)

Simcoe County and York Region

MTO Assignment No. 2019-E-0048

Submitted to:

AECOM Canada Ltd.

300 Water Street
Whitby, Ontario L1N 9J2

Submitted by:

WSP Canada Inc.

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

+1 905 567 4444

GEOCRES No.: 31D-832

Latitude: 44.117691°

Longitude: -79.635318°

19136074 (BBP Hwy 400, Rev 0.)

September 29, 2023

Distribution List

1 Electronic Copy - MTO Central Region

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
BRADFORD BYPASS AND HIGHWAY 400 INTERCHANGE RAMP STRUCTURES (E-S RAMP OVER HIGHWAY 400, N-E RAMP OVER HIGHWAY 400 / E-S RAMP)
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
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 foundation engineering services for the proposed Bradford Bypass (BBP), a 16.3 km rural controlled access freeway connecting Highway 400 to Highway 404, in the County of Simcoe and Regional Municipality of York. This report presents the results of the foundation investigation carried out for planning and preliminary design of the following proposed ramp structures at the BBP/Highway 400 Interchange as shown on the Key Plan in Drawing 1.

- **E-S Ramp over Highway 400**: multi-span structure carrying BBP/Hwy 400 E-S ramp over Highway 400 located between approximately Station 11+735 and 11+952
- **N-E Ramp over Highway 400 and the E-S Ramp**: multi-span structure carrying the Hwy 400/BBP N-E Ramp over Highway 400 and over the E-S Ramp located between approximately Station 10+709 to 10+952.

2.0 SITE DESCRIPTION

The site of the proposed BBP/Hwy 400 Interchange is located in the County of Simcoe and in the Town of Bradford / West Gwillimbury, Ontario. Highway 400 is currently a six-lane highway with three northbound and three southbound lanes separated by a concrete median. The proposed interchange is located between 8th Line and 9th Line along Highway 400 and the proposed E-S and N-E ramp structures are located approximately 1.8 km and 2.2 km north of Simcoe Road 88, respectively. The general site (east and west of Highway 400) consists of farmland. The existing ground surface generally slopes down from north to south at the proposed interchange location. At the N-E Ramp location the ground slopes down from west to east, with the Highway 400 grade appearing to have been constructed in a partial cut in this area (see Photograph 1). The typical farmland on the east side of Highway 400 is shown in Photograph 2. The existing Highway 400 over 9th Line overpass structure is located to the north of the proposed Bradford Bypass and Highway 400 interchange structures and an existing structural culvert crosses below Highway 400 to the south of the proposed structures.

Photograph 1 – Proposed west side of N-E Ramp structure location (looking northwest from Hwy 400 SBL)

Photograph 2 – Proposed east side of E-S Ramp and N-E Ramp structure locations (looking northeast from Hwy 400 NBL ditch)

3.0 INVESTIGATION PROCEDURES

The field work for the current investigation was carried out between December 7 and 20, 2021 and November 9 and 17, 2022, during which time a total of four boreholes (designated Boreholes 400-1 to 400-4) were advanced at the locations shown on Drawing 1.

All boreholes were advanced using 210 mm outside diameter (O.D.) hollow stem augers followed by wash-rotary techniques (advancement of tricone with water/drilling mud) using a Diedrich D120 and D50 track-mounted drill supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The wash-rotary technique was used to counter-balance hydrostatic forces and reduce disturbance at the sampling and testing interval. 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 the 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.

The water level was typically not measured in the open boreholes due to the introduction of water during drilling operations, unless otherwise noted on the drilling records. Standpipe piezometers were installed in Boreholes 400-2 and 400-3 and were screened within a silty sand and clayey silt deposit, respectively. The installed piezometers consist of a 50 mm diameter PVC pipe, with a 3 m long slotted screen within a filter sand pack. The boreholes and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with bentonite pellets in general accordance with Ontario Regulation 903 Wells (as amended)[2]. The monitoring wells were capped with monument casings.

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

[2] Ontario Regulation 903 Wells (as amended)

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 laboratory consolidation (oedometer) tests were performed on samples collected form borehole 400-1 using 76 mm O.D. thin walled 'Shelby' tubes (ASTM D1587[3]) to obtain relatively undisturbed samples of the soil. All laboratory tests were carried out in general accordance with MTO and / or ASTM Standards, as applicable.

One soil sample obtained from each borehole was submitted to a specialist analytical laboratory (Bureau Veritas Laboratories of Mississauga, Ontario) under chain of custody procedures for testing of electrical conductivity / resistivity, pH, and chemical analysis of 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 borehole records and shown on Drawing 1 are positioned relative to NAD 83 MTM (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum (CGVD28 datum; HT2 Geoid Model). The borehole locations, including the geographic (Latitude / Longitude) coordinates, the ground surface elevations, and borehole depths are summarized below.

Borehole No.	NAD 83 MTM Northing (m) (Latitude, °)	NAD 83 MTM Easting (m) (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
400-1	4,886,282.5 (44.116458)	294,024.6 (-79.634623)	255.7	33.8
400-2	4,886,611.9 (44.119421)	293,903.7 (-79.636141)	268.3	49.4 [1]
400-3	4886491.8 (44.118343)	294131.0 (-79.633298)	258.4	21.6
400-4	4886102.3 (44.114835)	293976.8 (-79.635217)	253.8	40.0

Note: 1. Borehole 400-2 was the only borehole that did not terminate upon refusal (SPT 'N'-value of 100 blows per 0.3 m of penetration or greater).

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

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984[4]), the general site lies near the border of three physiographic regions of Southern Ontario known as the Peterborough Drumlin field, the Schomberg Clay Plains, and Simcoe Lowlands.

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

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

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

The overall topography of the area indicates the northwest portion of the interchange site is located on the side of a rolling hill, possibly the edge of a till plain. The subsurface conditions encountered during the current investigation are generally consistent with the variable regional geology described above and may explain the variable nature of the soils encountered in the boreholes.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes from the current investigation, including piezometer installation details and water level readings, and the results of the in-situ and laboratory tests, are provided on the borehole records in Appendix A. The results of the in-situ field tests (i.e., SPT "N"-values) as presented on the borehole records and in Section 4 are uncorrected and are based on use of an automatic hammer. 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 Drawings 1 and 2 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 soil encountered at this site consists of firm to hard clayey silt to silty clay deposit (with interlayers of silty sand and clayey silt-silt till) underlain by a non-cohesive till deposit composed of silt and sand to silty sand to gravelly silty sand, except for the northwest portion of site at Borehole 400-2. In Borehole 400-2 (advanced from a higher elevation on the side of the hill in the northwest portion of the site), interlayered upper deposits of

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[4] Chapman, L.J. and Putnam, D. F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

cohesive and non-cohesive till and silt to silty sand layers were encountered above the clayey silt to silty clay deposit.

More detailed descriptions of the major soil layers encountered in the boreholes are provided in the following sections.

4.2.1 Silt

A 0.7 m thick layer of silt was encountered at ground surface (Elevation 268.3 m) in Borehole 400-2 and extended to about Elevation 267.6 m.

The SPT 'N'-value measured in the silt deposit was 10 blows per 0.3 m of penetration, indicating a loose degree of compactness.

The natural water content measured on a sample of the silt deposit was about 21%.

4.2.2 Upper Clayey Silt Till

A 2.5 m thick clayey silt till deposit was encountered at a depth of 0.7 m (Elevation 267.6 m) in Borehole 400-2 underlying the silt layer. The upper clayey till deposit extended to a depth of about 3.2 m below ground surface (Elevation 265.1 m).

The SPT 'N'-values measured in the clayey silt till range from 17 to 28 blows per 0.3 m of penetration, suggesting a very stiff consistency.

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

Atterberg limits testing was carried out on a sample of the clayey silt till and the sample had a liquid limit of 24%, plastic limit of 14%, and plasticity index of 10%, indicating a clayey silt of low plasticity. The results of the Atterberg limits test carried out on the upper clayey silt till deposit are plotted on the plasticity chart on Figure B2.

The natural water content measured on selected samples of the clayey silt till range from about 12% to 15%.

4.2.3 Upper Silty Sand Till

A 3.1 m thick upper silty sand till deposit was encountered underlying the upper clayey silt till deposit in Borehole 400-2. The deposit was encountered at a depth of 3.2 m (Elevation 265.1 m) and extended to a depth 6.3 m below ground surface (Elevation 262.0 m).

The SPT 'N'-values measured in the silty sand till range from 40 blows per 0.3 m of penetration to 100 blows per 0.23 m of penetration indicating a dense to very dense degree of compactness.

Grain size distribution testing was carried out on a sample of the silty sand till and the results are shown on Figure B3 in Appendix B.

Atterberg limits testing carried out on the silty sand till had a liquid limit of 14%, plastic limit of 12%, and plasticity index of 2%. These results, which are plotted on a plasticity chart on Figure B4, indicate that the fines portion of the deposit contains silt of slight plasticity.

The natural water content measured on selected samples of the silty sand till ranges between about 7% and 10%.

4.2.4 Silty Sand

A 5.3 m thick deposit of upper silty sand was encountered underlying the upper till deposits in Borehole 400-2. The upper deposit was encountered at a depth of 6.3 m (Elevation 262.0), and extends to a depth 11.6 m below ground surface (Elevation 256.7 m). A 2.3 m thick lower silty sand deposit was encountered beneath the clayey silt-silt till layer (described in the next section) in Borehole 400-2. The lower deposit was encountered at a depth of 17.8 m below ground surface (Elevation 250.5 m) and extended to a depth of about 20.1 m (Elevation 248.2 m).

The SPT 'N'-values measured in the silty sand deposit ranged from 57 blows per 0.3 m of penetration to 100 blows for 0.26 m of penetration indicating a very dense degree of compactness.

Grain size distribution testing was carried out on a sample of the silty sand deposit and the results are shown on Figure B5 in Appendix B.

Atterberg limits testing was carried out on a sample of silty sand and the results indicate the fines portion of the deposit is non-plastic.

The natural water content measured on selected samples of the silty sand range from about 9% to 18%.

4.2.5 Upper Clayey Silt-Silt Till

A 6.2 m thick layer of clayey silt-silt till was encountered in Borehole 400-2 underlying the silty sand layer at a depth of 11.6 m (Elevation 256.7 m). The base of the layer extended to a depth of 17.8 m (Elevation 250.5 m) and sand seams were encountered throughout the layer.

The SPT 'N'-values measured in the clayey silt-silt till range from 43 to 101 blows per 0.3 m of penetration, suggesting a hard consistency.

Grain size distribution testing was carried out on a sample of the clayey silt-silt till and the results are shown on Figure B6 in Appendix B.

Atterberg limits testing was carried out on a sample of the clayey silt-silt till and the sample had a liquid limit of 14%, plastic limit of 9%, and plasticity index of 5%. These results, which are plotted on a plasticity chart on Figure B7, indicate that the till deposit consists of clayey silt-silt of low plasticity.

The natural water content measured on selected samples of the clayey silt-silt till range from about 8% to 9%.

4.2.6 Clayey Silt to Silty Clay

A deposit of clayey silt to silty clay was encountered at ground surface in Boreholes 400-1, 400-3, and 400-4 (between Elevation 258.4 m and 253.8 m), and underlying the silty sand deposit in Borehole 400-2 at a depth of 20.1 m (Elevation 248.2 m). In Boreholes 400-1, 400-3 and 400-4, the layer was approximately 5.6 m to 11.7 m thick and extended from ground surface to Elevations 252.8 m to 244.0 m. In Borehole 400-2, the clayey silt to silty clay layer was penetrated for a length of 29.3 m before the borehole was terminated within the layer at a depth of 49.4 m (Elevation 218.9 m). The cohesive layer typically contained frequent silt and/or sand seams / laminations. In Borehole 400-4, the deposit contained a 1.3 m thick interlayer of silty sand encountered at a depth of 0.9 m (Elevation 252.9 m) and a 1.6 m thick interlayer of sandy clayey silt-silt till which was encountered at a depth of 5.6 m (Elevation 248.2 m).

The SPT 'N'-values measured in the clayey silt to silty clay deposit generally range from 5 to 29 blows per 0.3 m of penetration, except for in Borehole 400-2 where the deposit was encountered below the upper till and silty sand deposits where the measured SPT 'N'-values range from 30 to 64 blows per 0.3 m of penetration. One 'N'-value of 3 was measured in the surficial deposit in Borehole 400-4. In general, the SPT 'N'-values suggest the clayey silt to silty clay deposit has a firm to very stiff consistency, except the area below the till and silty sand (Borehole 400-2), where the 'N' values suggest a hard consistency.

Grain size distribution testing was carried out on eight samples of the clayey silt to silty clay deposit and the results are shown on Figure B8 in Appendix B.

Atterberg limits testing was carried out on ten samples of the clayey silt to silty clay deposit and measured liquid limits ranging from 27% to 39%, plastic limits ranging from 15% to 19%, and plasticity indices ranging from 10% to 21%. These results, which are plotted on a plasticity chart on Figure B9, indicate that the deposit ranges from clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water content measured on selected samples of the clayey silt to silty clay range between about 16% and 28%.

Laboratory consolidation tests were carried out on two specimens of the clayey silt deposit in Borehole 400-1. The preconsolidation stresses provided below were estimated from the void ratio versus logarithmic stress plot and from the total work versus stress plot. The bulk unit weight of the test specimens were measured to be about 19.8 kN/m^3 and 21.6 kN/m^3, and the specific gravity was 2.73 and 2.72 in samples TO7 and TO9, respectively. Details of the test results are shown on Figures B10 and B11 in Appendix B 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)
400-1 / TO7	4.9 / 250.8	65	450	360	7.0	0.229	0.027	0.699	0.0094
400-1 / TO9	7.2 / 248.5	88	250	120	2.9	0.105	0.012	0.433	0.0037

Where:

σ_p' = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)

c_v = Coefficient of consolidation (vertical) for approximate overconsolidated stress range 80 kPa ≤ σ_v' ≤ 600 kPa

C_c = Compression index

C_r = Recompression index

e_o = Initial void ratio

OCR = Overconsolidation ratio

σ_{vo}' = Calculated existing vertical effective stress

The SPT 'N'-values measured in the silty sand interlayer within the clayey silt to silty clay deposit range from 3 to 15 blows per 0.3 m of penetration indicating a very loose to compact degree of compaction. The natural water content measured on a sample of the silty sand interlayer was 7%.

The SPT 'N'-values measured in the sandy clayey silt-silt till interlayer within the clayey silt to silty clay deposit was 12 blows per 0.3 m of penetration suggesting a stiff consistency. A grain size distribution test was carried out on a sample of the sandy clayey silt-silt till interlayer and the results are shown on Figure B6 in Appendix B. An Atterberg limits test was carried out on a sample of the sandy clayey silt-silt till interlayer, the sample had a liquid limit of 15%, plastic limit of 9%, and corresponding plasticity index of 6%. These results, which are plotted on a plasticity chart on Figure B7, indicate that the deposit is a clayey silt-silt of low to slight plasticity. The natural water content measured on a sample of the sandy clayey silt-silt till was 11%.

4.2.7 Silt and Sand to Silty Sand to Gravelly Silty Sand - Till

A non-cohesive till deposit consisting of silt and sand to silty sand to gravelly silty sand was encountered underlying the clayey silt to silty clay deposit in Boreholes 400-1, 400-3, and 400-4. The deposit was encountered at a depth of 11.7 m (Elevation 244.0 m), 5.6 m (Elevation 252.8 m), and 9.8 m (Elevation 244.0 m) in Boreholes 400-1, 400-3 and 400-4, respectively. The non-cohesive till deposit had a thickness of 10.7 m in Borehole 400-3 and in Boreholes 400-1 and 400-4 measured a thickness of 22.1 m and 30.2 m respectively, before the boreholes were terminated within the deposit at depths of 33.8 m to 40.0 m below ground surface (Elevations 221.9 m to 213.8 m). In Borehole 400-3 the bottom 0.7 m of the deposit (Elevation 242.1 m to 242.8 m) transitioned to a clayey sit till, and in Borehole 400-4 a 3 m thick interlayer of sandy clayey silt-silt till (described separately in the next section) was encountered at a depth of 30.8 m (Elevation 223.0 m) and a 1.5 m thick interlayer of silty sand was encountered at a depth of 33.8 m (Elevation 220.0 m).

The SPT 'N'-values measured in the silt and sand to silty sand (till) generally range from 5 blows to 35 blows per 0.3 m of penetration. Two lower SPT 'N'-values of 3 blows per 0.3 m of penetration were encountered within the silt and sand portion of the till deposit in Borehole 400-1. In the gravelly silty sand till at the bottom of Borehole 400-1 and in the silty sand till at the bottom of Borehole 400-4, higher SPT 'N'-values were encountered ranging between about 100 blows per 0.15 m of penetration and 100 blows per 0.23 m of penetration. The SPT 'N'-values indicate the till has a generally loose to dense degree of compactness, with some areas of the deposit having a very loose or very dense degree of compactness.

The SPT 'N'-values measured in the silty sand interlayer in Borehole 400-4 was 87 blows per 0.3 m of penetration, indicating a very dense degree of compactness. The natural water content measured on a sample of the silty sand layer was 15%.

Grain size distribution testing was carried out on eight samples of the non-cohesive till deposit and the results are shown on Figure B12 in Appendix B.

Atterberg limits testing was carried out on seven samples of the silt and sand to silty sand to gravelly silty sand (till) deposit and measured liquid limits ranging from 11% to 13%, plastic limits ranging from 9% to 10%, and corresponding plasticity indices ranging from 1% to 4%. These results, which are plotted on a plasticity chart on Figure B13, indicate that the fines portion of the non-cohesive till deposit is silt of slight plasticity.

The natural water content measured on selected samples of the non-cohesive till range between about 4% and 13%, however the natural water content is generally between 9% and 13%.

4.2.8 Sandy Clayey Silt-Silt Till (Interlayer)

A sandy clayey silt-silt till interlayer was encountered within the deposit of silt and sand to silty sand to gravelly silty sand (till) in Borehole 400-4. The deposit was encountered at a depth of 30.8 m (Elevation 223.0 m) and extended 3.0 m to a depth of 33.8 m (Elevation 220.0 m).

The SPT 'N'-values measured in the sandy clayey silt-silt till interlayer in Borehole 400-4 were 13 and 87 blows per 0.3 m of penetration suggesting a stiff to hard consistency.

Grain size distribution testing was carried out on a sample of the sandy clayey silt-silt till layer and the results are shown on Figure B14 in Appendix B. Atterberg limits testing was carried out on a sample of the sandy clayey silt-silt till layer and measured a liquid limit of 17%, plastic limit of 10%, and corresponding plasticity index of 7%, indicating that the material is a clayey silt-silt of low plasticity as shown on a plasticity chart on Figure B15 in Appendix B.

The natural water content measured on a sample of the sandy clayey silt-silt till was 9%.

4.2.9 Lower Clayey Silt

A lower 3.6 m thick clayey silt deposit was encountered in Borehole 400-3 underlying the till deposit at a depth of 16.3 m (Elevation 242.1 m) and extending to a depth of 19.9 m (Elevation 238.5 m).

The SPT ‘N’-values measured in the clayey silt were 118 blows per 0.18 m and 102 blows per 0.15 m of penetration, suggesting a hard consistency.

A grain size distribution test was carried out on a sample of the lower clayey silt and the results are shown on Figure B16 in Appendix B.

A natural water content measured on a sample of the clayey silt was 17%.

4.2.10 Lower Gravelly Sand

A gravelly sand layer was encountered underlying the lower clayey silt deposit in Borehole 400-3. The deposit was encountered at a depth of 19.9 m (Elevation 238.5 m) and the borehole was advanced for a length of 1.7 m before terminating within the deposit at a depth of 21.6 m (Elevation 236.8 m).

The SPT ‘N’-value measured in the gravelly sand was 100 blows per 0.15 m of penetration indicating a very dense degree of compaction.

A grain size distribution test was carried out on a sample of the lower gravelly sand deposit and the results are shown on Figure B17 in Appendix B.

The natural water content measured on a sample of the gravelly sand was 8%.

4.3 Groundwater Conditions

The water levels measured in the open boreholes at the time of the investigation are shown on the borehole records and are not considered representative of the hydrostatic water levels at the site due to the addition of drilling fluids/water into the boreholes and/or considering the water levels did not have sufficient time to stabilize.

Standpipe piezometers were installed in Boreholes 400-2 and 400-3 to allow monitoring of the groundwater level at this site. The groundwater levels recorded in the piezometers are shown on the borehole records in Appendix A and are summarized below.

Borehole No. (Piezometer)	Depth (Elevation of Screen Interval / Sand Pack)	Depth (bgs) to Water Level (m)	Water Level Elevation (m)	Date of Water Level Reading
400-2	8.9 – 12.0 (259.4 – 256.3)	3.7	264.6	01-Feb-23
400-3	2.4 – 5.5 (255.9 – 252.9)	0.9	257.5	28-Feb-23

The groundwater level observations 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 Results

Three soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the test results are summarized below:

Borehole No. – Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Chlorides (µg/g)	Soluble Sulphates (µg/g)
400-1 – 3	7.77	1100	893	480	< 20
400-2 – 1 to 3	7.60	5100	196	< 20	< 20
400-3 – 2	7.63	5300	188	< 20	< 20
400-4 – 3	7.88	2300	435	180	< 20

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Muhammad Talha Irshad, E.I.T. and was reviewed by Madison Kennedy, P.Eng., a Geotechnical Engineer at WSP Golder. Kevin Bentley, P.Eng., a Geotechnical Engineer with WSP Golder and MTO Foundations Designated Contact conducted a technical and quality control review of the report.

WSP Golder

Muhammad Talha Irshad, E.I.T.
Geotechnical Engineering Intern

Madison C. Kennedy, P.Eng.
Geotechnical Engineer

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

MTI/MCK/KJB/al

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
BRADFORD BYPASS AND HIGHWAY 400 INTERCHANGE RAMP STRUCTURES (E-S RAMP OVER HIGHWAY 400, N-E RAMP OVER HIGHWAY 400 / E-S RAMP)
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
MTO ASSIGNMENT NO. 2019-E-0048

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for planning and preliminary design of the Bradford Bypass and Highway 400 Interchange ramp 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 relied upon for any other purpose or by any other parties, including the construction contractor or design-build proponents. Contractors undertaking the work must make their own interpretation based on the factual data presented in the Preliminary Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the concept and preliminary design of the project and for which special provisions may be required in the future Contract Documents. Those requiring information on aspects of 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 General Arrangement drawings and Bradford Bypass mainline alignment / profile drawings provided by AECOM (dated March and April 2023, respectively), the proposed BBP / Highway 400 Interchange will consist of two ramp structures as follows:

- **E-S Ramp over Highway 400**: Four-span structure carrying the BBP/Hwy 400 E-S Ramp over Highway 400 located between Station 11+735 and 11+952 (about 217 m long and 14 m wide). The west and east approach embankments are anticipated to be about 9 m and 11 m high.
- **N-E Ramp over Highway 400 and E-S Ramp**: Five-span structure carrying Highway 400/BBP N-E ramp over Highway 400 and the E-S Ramp located between Station 10+709 to 10+952 (about 243 m long and 14 m wide). The east and west approach embankments are anticipated to be up to about 16 m high and 5 m high above existing ground surface (adjacent to where Highway 400 is constructed in a partial cut) respectively.

The preliminary General Arrangement drawings indicate that the ultimate configuration of each ramp structure will be constructed (as opposed to the interim configuration of the typical overpass structures that are to widened to the ultimate configuration in the future) and consist of up to two travelled lanes with a shoulder on each side. The structural classification of the bridge(s) is defined as “major-route” by the structural designer and to be confirmed by the owner as per Section 4.4.2 of the CHBDC (2019).

6.3 General Foundation Design Context

6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code CAN/CSA S6-19* (CHBDC, 2019) and its *Commentary*, the ramp structures and 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.

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 multi-span ramp foundation elements and approach embankments has been assessed as a "low degree of site and prediction model understanding". At the time of foundation investigation, the locations of the abutments and pier foundations were not confirmed and permissions to enter properties closer to the alignment was restricted, and hence boreholes are not located at/near each of the proposed ramp structure abutments. 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 and piers are confirmed.

Accordingly, the ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, $\boldsymbol{\Psi}$, and geotechnical resistance factors, $\boldsymbol{\phi_{gu}}$ and $\boldsymbol{\phi_{gs}}$ for a low degree of site understanding, from Tables 6.1 and 6.2 of CHBDC (2019) have been used at this stage of preliminary design. During detail design, additional investigation and testing must 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 developing future geotechnical resistance values during detail design, as applicable.

6.3.2 Seismic Design

6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance, $\overline{N}_{60}$ and average undrained shear strength, s_u within the upper 30 m of the overburden below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D for the N-E Ramp structure and Site Class E for the E-S Ramp structure in accordance with Table 4.1 of the CHBDC (2019), in the absence of any geophysical testing (i.e. shear wave velocity measurements). Geophysics testing, if carried out, may provide a more favourable Site Class designation and can be considered during detail design.

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

6.3.2.2 Spectral Response Values and Seismic Performance Category

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

Site Class D – Peak Ground Acceleration, Peak Ground Velocity, and Spectral Response

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.037	0.059	0.095
PGV (m/s)	0.040	0.060	0.097

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$S_a(0.2)$ (g)	0.064	0.097	0.151
$S_a(0.5)$ (g)	0.053	0.078	0.119
$S_a(1.0)$ (g)	0.033	0.050	0.074
$S_a(2.0)$ (g)	0.016	0.025	0.039
$S_a(5.0)$ (g)	0.003	0.006	0.009
$S_a(10.0)$ (g)	0.001	0.003	0.004

Site Class E – Peak Ground Acceleration, Peak Ground Velocity, and Spectral Response

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.052	0.083	0.134
PGV (m/s)	0.067	0.101	0.163
$S_a(0.2)$ (g)	0.085	0.128	0.200
$S_a(0.5)$ (g)	0.089	0.131	0.200
$S_a(1.0)$ (g)	0.059	0.090	0.135
$S_a(2.0)$ (g)	0.029	0.046	0.073
$S_a(5.0)$ (g)	0.006	0.012	0.018
$S_a(10.0)$ (g)	0.003	0.005	0.008

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

6.3.2.3 Soil Liquefaction

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

In general, the soils at these bridge sites consist of firm to hard clayey silt, and generally loose to very dense silt and sand to silty sand tills with generally hard clayey silt till interlayers. Based on the compactness and consistency of the soils and the relatively low site-specific PGA, the soils at this site are considered to have a low potential for liquefaction during a seismic event. Further assessment of liquefaction potential of the loose silt and sand to silty sand deposit at the E-S Ramp structure should be considered during detail design when the Seismic Performance Category is confirmed. Additionally, assessment of the cyclic mobility of the cohesive deposit(s) encountered at this site should be carried out during detail design and the associated impacts on stability and settlement should be assessed, as required.

6.4 Foundation Types

Based on the structure configurations (multi-span structures with total span lengths ranging from 217 m to 243 m) and subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments and piers. The preliminary recommendations provided herein will be subject to change when more detailed soil information and actual foundation locations are known, and when the geotechnical resistance factors can be increased on the basis of an increased level of site understanding.

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. A summary of the general advantages and disadvantages associated with each option and the preferred option is provided below.

For abutment foundations, driven steel tube or H-piles with the pile cap perched within the approach embankments is considered the preferred alternative from a geotechnical/foundations perspective. Provided that settlements can be mitigated and design geotechnical resistances are adequate, shallow foundations “perched” on a compacted granular pad above the stiff to very stiff clayey silt to clayey silt till may be considered for the west abutment of the N-E Ramp structure, but are not considered practical at the other foundation locations. Driven piles are considered the preferred option for piers at this stage given that limited information is available near the anticipated pier locations to justify shallow foundations. Caissons could also be considered at the abutment and pier locations for both bridges. Caissons are preferred at the pier locations at the centerline and directly adjacent to Highway 400, as they can be designed and constructed without the need for temporary protections systems.

The feasibility of shallow foundations for abutments depends to a large degree on settlement of the foundation soils due approach embankment loading. Steel H-piles or tube piles driven into the “100-blow” soils (where encountered) will range from about 20 m to 30 m below ground surface, and friction piles may need to be designed at some foundation elements. Caissons will provide higher capacities but may be too long (in excess of 30 m) for practical installation purposes at many locations in order to achieve design capacities and will require drilling slurry and temporary casings to maintain an open hole during advancement through the saturated sand and silt deposits.

6.4.1 Shallow Foundations

Strip or spread footings founded on the very stiff clayey silt, compact silty sand, and very stiff to hard clayey silt till (at or below the approximate elevations identified below and resulting in up to 1.8 m of subexcavation below ground surface) are considered marginally feasible for support of the ramp structure abutments and piers due to the anticipated high loads and relatively low geotechnical resistances. Strip or spread footings may be founded on a compacted Granular 'A' pad placed above the stiff to very stiff clayey silt or very stiff to hard clayey silt till deposits to increase geotechnical resistances. For the granular pad option, settlements in the foundation soils due to any additional granular fill placement is anticipated and an increase in the factored serviceability geotechnical resistance may not be substantial enough to increase the feasibility of shallow foundations at most locations.

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

Structure Name	Founding Element	Founding Elevation(s)	Reference Borehole, Founding Stratum	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance[1]
E-S Ramp over Highway 400	West Abutment	252 m	BH 400-4, Compact Silty Sand / Firm to Stiff Clayey Silt	3 m	210 - 220 kPa	50 - 100 kPa
				5 m	215 - 225 kPa	40 - 80 kPa
			BH 400-4, 3 m Compacted Granular Pad over Very Stiff Clayey Silt over Stiff Sandy Clayey Silt-Silt Till	3 m	280 - 325 kPa	85 - 125 kPa
				5 m	290 - 330 kPa	65 - 100 kPa
	Piers	1.5 m below ground surface[2]	BH 400-1, Firm to Stiff Clayey Silt	3 m	125 – 150 kPa	30 - 75 kPa
				5 m	125 – 150 kPa	30 - 75 kPa
			BH 400-1, 3 m Compacted Granular Pad over Firm to Stiff Clayey Silt	3 m	280 - 320 kPa	85 - 100 kPa
				5 m	270 - 300 kPa	60 - 75 kPa
	East Abutment	257 m	BH 400-3, Stiff to Very Stiff Clayey Silt	3 m	210 - 220 kPa	50 - 100 kPa
				5 m	215 - 225 kPa	40 - 80 kPa

Structure Name	Founding Element	Founding Elevation(s)	Reference Borehole, Founding Stratum	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance[1]
E-S Ramp over Highway 400	East Abutment	257 m	BH 400-3, 3 m Compacted Granular Pad over Stiff Clayey Silt over Loose to Compact Silty Sand Till	3 m	320 - 340 kPa	100 - 125 kPa
				5 m	290 - 310 kPa	75 - 100 kPa
N-E Ramp over Highway 400 and E-S Ramp	West Abutment and West Piers	267 m (west abutment[3]) 1.5 m below ground surface at west piers[2]	BH 400-2, Very Stiff Clayey Silt Till	3 m	475 kPa	250 kPa
				5 m	500 kPa	200 kPa
			BH 400-2, 3 m Compacted Granular Pad over Dense to Very Dense Silty Sand Till	3 m	600 kPa	300 kPa
				5 m	625 kPa	225 kPa
	East Abutment and East Piers	257 m (east abutment) 1.5 m below ground surface at east piers [2]	BH 400-3, Very Stiff Clayey Silt	3 m	220 kPa	50 - 100 kPa
				5 m	225 kPa	40 - 80 kPa
			BH 400-3, 3 m Compacted Granular Pad over Stiff Clayey Silt over Loose to Compact Silty Sand Till	3 m	320 - 340 kPa	100 - 125 kPa
				5 m	290 - 310 kPa	75 - 100 kPa

Notes:
1. For 25 mm of settlement independent of any settlements induced by surrounding grade changes / embankment loading.
2. The final founding elevation and associated founding stratum at the pier locations will need to be confirmed with additional investigation at detail design.
3. Ground surface at the proposed west abutment of the N-E Ramp is about 5 m higher than elevation of the borehole (BH 400-2) advanced during current exploration. It is assumed Highway 400 has been constructed in a cut in this area and competent soils are likely present at higher elevation, similar to those observed in the closest borehole (BH 400-2). Actual founding elevations for structure foundations on the west side of Highway 400 must be investigated further and checked during detail design; the location of the footing relative to the Highway 400 cut slope must also be taken into account in assessing the geotechnical resistance.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width, founding elevation, and thickness of granular pad (as applicable) and as such, the geotechnical resistances must be reviewed and revised if the footing width varies from that specified above or if the founding soils differ from that given in the previous section. In general, for larger footing sizes, higher factored ultimate and lower factored serviceability geotechnical resistances would apply. The preliminary factored geotechnical resistances should also be re-evaluated using geotechnical resistance factors for a typical degree of understanding once further investigation data is available at the foundation elements.

Given the variability in the founding soils at this site it is anticipated that there will be differential settlement across and between foundation elements. Additional investigation will be required to confirm the subsurface conditions within the footprints of the abutments and piers. Depending on the subsurface conditions, different foundation options or mitigation measures may be required to reduce the potential for differential settlement across each foundation, and between foundation elements of the structure(s).

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. Assuming that the founding soils (firm to very stiff clayey silt, compact silty sand, or very stiff to hard clayey silt till) are not loosened or disturbed during excavation and construction, an effective interface angle of friction between the cast-in-place concrete footings and founding soils of 24° and corresponding unfactored coefficient of friction, tan δ, of 0.45 may be used for preliminary design. An effective angle of friction of 33° and corresponding unfactored coefficient of friction, tan δ, of 0.65 may be used between the cast-in-place concrete footings and the Granular 'A' pad.

6.4.2 Deep Foundations

6.4.2.1 Steel H-Pile or Tube Foundations

Steel piles (HP section or closed ended tube piles) driven into the "100-blow" gravelly sand and gravelly silty sand till, or clayey silt to silt deposit are considered feasible for the foundations at the ramp structures. At some locations, although competent "100-blow" end-bearing soil was encountered during the preliminary investigation, the thickness and consistency of the "100-blow" soil will need to be confirmed during detail design. At other locations, long friction piles (35 m to 40 m) are proposed as no significant thickness of "100-blow" soil was encountered within the drilled depth (up to 50 m below ground surface) for end-bearing pile design.

Although not specifically encountered or confirmed during the current investigation, the presence of potential pockets of gravel or cobbles and/or boulders should be anticipated within the glacially derived till and silty sand deposits (specifically where SPT 'N'-values of "100-blow" were encountered in the upper deposits in Borehole 400-2) and will need to be considered during detail design.

The following factored geotechnical resistances may be used for preliminary design:

Structure Name(s)	Foundation Location (Associated Borehole)	Approximate Pile Length[4]	Estimated Pile Tip Elevation (Soil Strata Near Pile Tip)	Pile Type	Factored Ultimate Geotechnical Resistance[1]	Factored Serviceability Geotechnical Resistance[1,2]
E-S Ramp over Hwy 400	East Abutment (BH400-3)	19 m	238 m (Very Dense "100-blow" Gravelly Sand)	324 mm dia. tube pile	1,000 kN	Does Not Govern
				HP 310x110	1,200 kN	Does Not Govern
				HP 360x108	1,500 kN	Does Not Govern

Structure Name(s)	Foundation Location (Associated Borehole)	Approximate Pile Length[4]	Estimated Pile Tip Elevation (Soil Strata Near Pile Tip)	Pile Type	Factored Ultimate Geotechnical Resistance[1]	Factored Serviceability Geotechnical Resistance[1,2]
E-S Ramp over Hwy 400	Piers (BH400-1)	30 m	224 m (Very Dense (“100-blow” Gravelly Silty Sand Till)	324 mm dia. tube pile	1,100 kN	Does Not Govern
				HP 310x110	1,300 kN	Does Not Govern
				HP 360x108	1,600 kN	Does Not govern
	West Abutment (BH400-4)	35 m	217 m (Compact Silty Sand Till)	324 mm dia. tube pile	1000 kN	Does Not Govern
				HP 310x110	1,200 kN	Does Not Govern
				HP 360x108	1,500 kN	Does Not Govern
N-E Ramp over Hwy 400 and E-S Ramp	East Abutment and East Piers (piers east of Hwy 400 centreline) (BH400-3)	19 m (abutment) 30 m (piers)[3]	At or below 238 m (Very Dense “100-blow” Gravelly Sand)	324 mm dia. Tube pile	1,000 kN	Does Not Govern
				HP 310x110	1,200 kN	Does Not Govern
				HP 360x108	1,500 kN	Does Not Govern
	West Abutment and West Piers (piers west of and including Hwy 400 centreline) (400-2)	40 m (abutment) 35 m (piers)	228 m (Hard Clayey Silt)	324 mm dia. Tube pile	900 kN	Does Not Govern
				HP 310x110	1,100 kN	Does Not Govern
				HP 360x108	1,300 kN	Does Not Govern

Notes:
1. Resistance values assume single pile and do not take into account pile group efficiency.
2. Does Not Govern: SLS geotechnical resistance value for 25 mm of settlement is greater than the ULS value and does not govern the design. The SLS value for 25 mm of settlement does not account for settlement of foundation soils due to surrounding grade changes / embankment loading.
3. Longer piles may be required at the pier locations to achieve the founding resistances and/or reach the very dense gravelly sand soil strata near the pile tip.
4. Assuming the pile cap is approximately 1.5 m below existing grade.

The estimated factored ultimate geotechnical resistances provided above are calculated on both shaft and tip resistances, and assume piles have had sufficient time to “set-up” and allow pore pressures to dissipate after initial driving in order to achieve the design geotechnical resistances. If higher capacities are required, consideration can be given to further increasing the size of the piles.

Considering the anticipated high loads for the multi-span bridges, pile groups at each foundation element are likely required. For preliminary design, driven steel piles spaced at 3 pile diameters (centre-to-centre) can be assumed to act as single piles with no group interaction effects with regards to axial resistance. For piles spaced less than 3 diameters, the total pile axial resistance should be reduced by a group reduction factor (R_A) (Reese, 2006) as follows:

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

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

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) PROV 903 *(Deep Foundations)* as amended by Special Provision 109F57 with High-Strain Dynamic testing specified on at least 20% of the piles or two piles at each foundation element (whichever is greater) in each stage of construction.

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

- Advanced site-specific investigation during detail design to confirm or adjust axial geotechnical resistances for design based on the use of a typical rather than low degree of understanding;
- High-strain dynamic testing (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/or
- Evaluation of strength gain with time (via PDA testing or static pile load testing or both) to ascertain the potential gain, if any, in geotechnical resistance.

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 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.

6.4.2.2 Drilled Shafts (Caissons)

Caissons founded within the silty sand, silt and sand to silty sand till deposit and clayey silt to silt deposit are feasible for supporting the ramp structure abutments and piers. Long friction caissons (>19 m) penetrating into the cohesive clayey silt deposit and loose to compact section of the silt and sand till to silty sand till deposit have been evaluated for preliminary design. At the N-E Ramp west abutment and west pier locations, consideration could be given to founding shorter caissons in the very dense upper silty sand or hard clayey silt layers encountered between depths of 6.3 m and 20.1 m below ground surface (Elevations 262.0 m and 248.2 m) in Borehole 400-2; however, as these layers were absent in the other boreholes advanced at this site these recommendations are only applicable for the current proposed location of the N-E Ramp west abutment and surrounding piers. As borehole coverage is limited at this stage, it is recommended that evaluation of alternatives be completed in detail design subject to additional investigation. If adopted, caissons founded within the cohesionless silty sand would require use of an appropriate method to control basal heave (e.g. polymer slurry) and achieve the design geotechnical resistances.

Although not specifically encountered or confirmed during the current investigation, consideration must be given to the presence of potential cobbles and boulders that may be present within the glacially derived till and silty sand deposits.

The following geotechnical resistances may be used for preliminary design at the associated foundation locations and pile lengths, based on geotechnical resistance factors for a low degree of site understanding:

Structure Name(s)	Foundation Location (Associated Borehole)	Approximate Caisson Length	Estimated Caisson Base Elevation	Caisson Diameter	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance[1]
E-S Ramp over Hwy 400	East Abutment (BH400-3)	19 m	238 m (Very Dense "100-blow" Gravelly Sand)	0.9 m	2,500 kN	Does Not Govern
				1.5 m	5,900 kN	Does Not Govern
	Piers (BH400-1)	30 m	224 m (Very Dense "100-blow" Gravelly Silty Sand Till)	0.9 m	3,500 kN	Does Not Govern
				1.5 m	7,700 kN	Does Not Govern
	West Abutment (BH400-4)	20 m	232 m (Loose to Compact Silt and Sand Till)	0.9 m	1,500 kN	Does Not Govern
				1.5 m	2,900 kN	Does Not Govern
		35 m	217 m (Compact Silty Sand Till)	0.9 m	2,500 kN	Does Not Govern
				1.5 m	5,900 kN	Does Not Govern

Structure Name(s)	Foundation Location (Associated Borehole)	Approximate Caisson Length	Estimated Caisson Base Elevation	Caisson Diameter	Factored Ultimate Geotechnical Resistance[1]	Factored Serviceability Geotechnical Resistance[1,2]
N-E Ramp over Hwy 400 and E-S Ramp	East Abutment and East Piers (piers east of Hwy 400 centreline) (400-3)	19 m (abutment) 30 m (east piers)[3]	At or below 238 m (Very Dense "100-blow" Gravelly Sand)	0.9 m	2,500 kN	Does Not Govern
				1.5 m	5,900 kN	Does Not Govern
	West Abutment and West Piers (piers west of and including Hwy 400 centreline) (400-2)	15 m (abutment)[4] 10 m (west piers)[4]	258 m (Very Dense Silty Sand)	0.9 m	1,400 kN	Does Not Govern
				1.5 m	3,500 kN	Does Not Govern
		30 m (abutment) 25 m (west piers)	238 m (Hard Clayey Silt)	0.9 m	2,100 kN	Does Not Govern
				1.5 m	2,800 kN	Does Not Govern

Notes:
1. Resistance values assume single caisson and do not take into account caisson group efficiency.
2. Does Not Govern: SLS geotechnical resistance value for 25 mm of settlement is greater than the ULS value and does not govern the design. The SLS value for 25 mm of settlement does not account for settlement of foundation soils due to surrounding grade changes / embankment loading.
3. Longer piles may be required at the pier locations to achieve the founding resistances and/or reach the very dense gravelly sand strata near the pile tip.
4. Short caisson option to be confirmed during detail design for west abutment and west pier foundation elements for N-E Ramp over Hwy 400 and E-S Ramp structure; the presence, depth and thickness of the very dense silty sand needs to be checked and confirmed at actual foundation element location. It is noted that west abutment location is about 5 m higher than the ground surface at the borehole location (BH 400-2).
5. Assumes the caisson length is measured from approximately 1.5 m below existing grade.

For preliminary design, drilled shafts (caissons) 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 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 seams / interlayers. If a permanent liner is used, the design geotechnical resistances provided above may need to be revised to account for the reduced adhesion between the liner material and surrounding soil along the length of the liner compared to the adhesion between concrete and the surrounding soil if temporary liners are used. Specialized construction techniques would be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (e.g. polymer slurry or other slurry mix) within the liner / open hole to prevent basal heave and disturbance of water-bearing cohesionless layers/interlayers (along shaft and at base). Given that the above drilled shaft geotechnical resistances have both a shaft friction and end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the drilled shaft. The design geotechnical resistances provided may need to be revised depending on the proposed construction method and specifications, particularly if a bentonite slurry is allowed to be used, as it may reduce the shaft friction component of the geotechnical resistance. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection device (SID) or given the use of a drilling slurry, a shaft quantitative inspection device (SQUID). Should the inspection indicate that loosened material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

Alternatively, a design based solely on shaft friction may be considered provided the design geotechnical resistances are reduced accordingly and appropriate quality assurance procedures are adopted by the design-builder / contractor. The consistency and characteristics of the drilling slurry (particularly if bentonite slurry is being considered) or use of permanent liners (if not specified in the design drawings) will have an impact on the design geotechnical resistances and this will need to be considered during detail design and included in the 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 use of a typical rather than low degree of understanding, and/or
- Advanced static pile load test as per ASTM D-1143, bi-directional static load (“Osterberg Cell”) test (CFEM, 2006), or Statnamic Load Test (CFEM, 2006).

Caisson installation must be in accordance with OPSS.PROV 903 (Deep Foundations) and MTO’s recent special provision should be included in the future contract documents to address the requirements for supply and installation of drilled shafts (caissons) including the use of temporary or permanent liners/casings and slurry, the placement of concrete by tremie methods, cleaning and inspection of the shafts as applicable, and quality control testing. Non-destructive post-construction testing in selected drilled shafts should also be included in the future contract specifications 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.

6.4.2.3 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 or caissons, the resistance to lateral loading will have to be derived from the soil in front of the piles.

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 the piles / caissons may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 2002 as referenced in CHBDC, 2006):

For non-cohesive soils:

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

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

For cohesive soils:

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

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

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.

The following values of n_h and s_u may be assumed in the structural analyses for a single vertical pile or caisson, 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. The groundwater level is assumed to be about 1 metre below ground surface.

Soil Unit	n_h [3] (kPa/m)	S_u (kPa)
New Granular Fill (Granular ‘A’ or ‘B’ Type II)	40,000 – 50,000	-
Clayey Silt to Clayey Silt-Silt Till	-	200
Very Loose to Compact Silty Sand	9,500 – 13,000	-
Compact to Very Dense Silty Sand	13,000 – 14,500	
Soft to Stiff Clayey Silt to Silty Clay	-	50 - 100
Very Stiff to Hard Clayey Silt to Silty Clay	-	150
Loose to Very Dense Silt and Sand to Silty Sand to Gravelly Silty Sand Till	15,000 - 20,000	-

Soil Unit	n_h [3] (kPa/m)	S_u (kPa)
Very Dense Lower Gravelly Sand	20,000	-

Notes:
1. Although parameters are provided for the full depth of the soil stratigraphy, lateral resistance in the upper 1.5 m should be neglected to account for frost action.
2. Where both n_h and s_u parameters are provided, the structural assessment should be completed for both undrained and drained conditions, and the selected design should be based on the more conservative approach.
3. Values of n_h provided are based on material below the groundwater table, with the exception of new granular fill.

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).

6.4.2.4 Downdrag Loads on Piles / Caissons

Based on the preliminary profile drawings, the approach embankments at the two bridges range from approximately 5 m high to 16 m high with total post-construction settlements in the foundation soils estimated to range from 5 mm to 300 mm (see Section 6.6.2). As a result, downdrag loads will need to be assessed further during detailed design. Downdrag loads can likely be mitigated by designing piles / caissons to resist the additional load in the structural design and/or reducing downdrag forces by preloading the foundation soil to induce settlements prior to driving piles or installing caissons.

6.5 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)*.

6.6 Approach Embankments

Based on the preliminary profile alignments, the approximate height of the approach embankments and anticipated foundation soils at the two proposed bridge structures are summarized below.

Structure	Height of Approach Embankment (Abutment Location)	Anticipated Foundation Soils
E-S Ramp over Hwy 400	11 m (east abutment) 9 m (west abutment)[1]	Firm to Very Stiff Clayey Silt to Silty Clay over Loose to Dense Silt and Sand to Silty Sand Till over “100-blow” soils (east abutment) Firm to Very Stiff Clayey Silt to Silty Clay over Loose to Very Dense Silt and Sand to Silty Sand Till (west abutment)

Structure	Height of Approach Embankment (Abutment Location)	Anticipated Foundation Soils
N-E Ramp over Highway 400 and E-S Ramp	16 m (east abutment) 5 m (west abutment)[1]	Firm to Very Stiff Clayey Silt to Silty Clay over Loose to Dense Silt and Sand to Silty Sand Till over “100-blow” soils (east abutment) Very Stiff to Hard / Very Dense Clayey Silt to Silty Sand (Till) with Very Dense Silty Sand interlayers over Hard Clayey Silt to Silty Clay (west abutment)

Note:

1. Relatively low west approach embankment height located on top of “hill” adjacent to Highway 400 which is constructed in a partial cut.

A 2 m wide bench should be incorporated into the design of the embankment slopes as required for uninterrupted embankment heights greater than 8 m in accordance with OPSD 202.010 (Slope Flattening). At the east approach embankment of the N-E Ramp structure, two benches may be required if the total embankment height exceeds 16 m.

For preliminary design, it is assumed that prior to construction of the new approach embankments, all topsoil, peat/organic soil, existing unsuitable fill materials and any soft/loose surficial deposits (possibly disturbed by farming activities) will be stripped from the footprint of the new embankments and replaced with suitable granular fill. Based on the borehole information, stripping of unsuitable soil is estimated to be up to about 1.5 m below ground surface, however, organics were noted to extend up to about 2 m below existing ground surface at some locations and will need to be further investigated during detail design. Additional details regarding embankment construction are provided in Section 6.8.1.

Conventional embankment construction is considered feasible at the site. Where space limitations exist, consideration can be given to designing RSS embankments or retaining walls as required.

Global stability and settlement analyses were carried out at the critical locations identified to be the east and west approach embankments of the N-E Ramp bridge, and the west approach embankment of the E-S Ramp bridge using the closet borehole information. At the west abutment of the N-E Ramp bridge, although only 5 m of fill is proposed, it is important to note that Highway 400 has been constructed in a cut with the existing ground surface sloping down towards the existing highway such that the proposed approach embankment slope height is actually about 16.5 m high (consisting of 5 m of new approach embankment fill and 11.5 m of existing cut slope). It is noted that a borehole was not advanced directly at the proposed west abutment location of the N-E Ramp structure, which is located about 5 m above borehole 400-2, and the soil conditions (between approximately Elevation 268 m and 274 m) were interpreted to be consistent the subsurface conditions encountered in Borehole 400-2. Additional investigation will need to be carried out at the west abutment to confirm the stability and settlement analysis at this location.

For the both the stability and settlement analyses, the groundwater elevation was generally assumed to be the highest measured water level in the closest borehole / piezometer which ranged from about 0.9 m to 3.7 m below ground surface.

6.6.1 Stability

The Factor of Safety for global stability is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_{gu} (i.e. $FoS = 1/(\Psi \cdot \phi_{gu})$. Accordingly, given the limited geotechnical information at the site and low degree of site understanding, minimum target Factors of Safety of 1.4 and 1.6 have been used for the preliminary design of the approach embankment slopes for the temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2019) and MERO (2020).

The foundation engineering parameters for the new embankment fill and major soil types encountered below the embankment footprints for the proposed bridge structures are summarized below.

Idealized Stratigraphic Unit	γ (kN/m^3)	φ' (°)	S_u (kPa)
New Granular Fill (Granular ‘A’ or ‘B’ Type II)	21	36	--
Soft to Firm Clayey Silt	19	28	50 / 75
Firm to Stiff Clayey Silt to Silty Clay	19	30	100
Very Stiff to Hard Clayey Silt to Silty Clay	19	30	150
Very Loose to Compact Silty Sand	20	30	--
Compact to Very Dense Silty Sand	20	32	--
Loose to Very Dense Silt and Sand to Silty Sand (Till)	21	34	--
Very Stiff to Hard Clayey Silt to Clayey Silt-Silt (Till)	21	34	200

where: γ = bulk unit weight
ϕ’ = effective friction angle
S_u = undrained shear strength

The idealized geometry and results of the stability analyses (modelled for circular slip surfaces using *Slide 2 (Version 9.014)*) for the critical sections (i.e., highest approach embankment on the west and east side of Highway 400 for the N-E Ramp and west side for the E-S Ramp) are shown in Figures 1 to 6. Based on the results, the new approach embankments for the N-E Ramp bridge and E-S Ramp bridge constructed with suitable granular fill and 2H:1V side slopes (with a mid-height 2 m wide bench) will have an adequate factor of safety (i.e., greater than 1.4 for short-term conditions and greater than 1.6 for long-term conditions) for global stability.

Structure	Location (Relevant Borehole)	Slope Height (Embankment Material)	Slope Gradient[1]	Static Global Stability Limit State	Calculated Factor of Safety
E-S Ramp	East Approach Embankment (400-3)	11 m (new granular fill)	2H : 1V	Temporary (Undrained) Condition	>1.4
				Permanent (Drained) Condition	>1.6
	West Approach Embankment (400-4)	9 m (new granular fill)	2H : 1V	Temporary (Undrained) Condition	>1.4
				Permanent (Drained) Condition	>1.6

Structure	Location (Relevant Borehole)	Slope Height (Embankment Material)	Slope Gradient[1]	Static Global Stability Limit State	Calculated Factor of Safety
N-E Ramp	East Approach Embankment (400-3)	16 m (new granular fill)	2H:1V	Temporary (Undrained) Condition	>1.4
				Permanent (Drained) Condition	>1.6
	West Approach Embankment (400-2)	16.5 m (5 m of new granular fill above 11.5 m high cut slope)	2H:1V	Temporary (Undrained) Condition	>1.4
				Permanent (Drained) Condition	>1.6

1. Including 2 m wide mid-height bench.

When more detailed foundation investigation is completed at the site (typical or high level of understanding), the resistance factor can be increased and the target Factor of Safety for the temporary and permanent conditions can be decreased accordingly.

6.6.2 Settlement

Settlement analyses were carried out for the proposed maximum fill thickness (fill height) at the east and west approach embankments for the N-E Ramp and E-S Ramp bridge structures. The thickness of the compressible foundation soils and the height of the approach embankments will vary along the approach embankment alignment, and as such the settlements along the length of the alignment will similarly vary; however, the settlements estimated from the settlement analysis represent the maximum anticipated value near the abutments.

The settlement analyses assume that topsoil, surficial deposits containing excessive organic material, any disturbed soils from farming activities, or any other deleterious materials (i.e., approximately the surficial 1.5 m of soil) have been removed and re-compacted or replaced with suitable granular fill. The settlement analyses were carried out using the commercially available program Settle3 (Version 5.012), developed by Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on Westergaard's (1938) solution.

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). Due to the generally overconsolidated and stiff to very stiff nature of the clayey soils, secondary compression is considered to be relatively negligible (less than 5 to 10 mm) and is not considered for preliminary analysis.

The immediate compression of the non-cohesive deposits were modelled by estimating an elastic modulus of deformation (E') based on the SPT "N"-values using correlations proposed by Bowles (1984), Kulhawy and Mayne

(1990), and Peck et al. (1974), as well engineering judgement from experience with similar soils in this region of Ontario.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests near the site to estimate the stress history and deformation parameters for the cohesive deposits.

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was estimated using the results of the laboratory consolidation tests.

The foundation engineering parameters used in the settlement analyses for the major soil types encountered below the embankment footprints for the proposed bridge structures are summarized below.

Idealized Stratigraphic Unit	γ (kN/m^3)	Compressibility Parameters						
		E' (MPa)	C_c	C_r	e_o	σ_p'	OCR	c_v (cm^2/s)
Firm to Hard Clayey Silt to Silty Clay	19	--	0.229 to 0.105	0.027 to 0.012	0.699 to 0.433	450 to 250	7 to 2.9	9.4×10^{-3} to 3.7×10^{-3}
Compact to Very Dense Silty Sand	20	50 - 75	-	-	-	-	-	-
Very Stiff to Hard Clayey Silt to Clayey Silt-Silt Till	21	50 - 100	-	-	-	-	-	-
Very Loose to Very Dense Silt and Sand to Silty Sand Till	21	25 - 100	-	-	-	-	-	-
Very Dense Gravelly Sand to Gravelly Silty Sand Till	21	200	-	-	-	-	-	-

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 estimated magnitude of immediate, post-construction, and total settlement of the foundations soils for the highest anticipated approach embankment near the east and west abutment locations for the N-E Ramp and E-S Ramp bridges are presented below, assuming the use of conventional granular fill for construction. The estimated settlements do not account for immediate settlement of the embankment fill itself.

Structure	Location (Relevant Borehole)	Proposed Maximum Embankment Thickness	Settlement (mm)		
			Immediate	Estimated Post-Construction Settlement over a 20-Year Period	Total
E-S Ramp	East Approach (400-3)	11 m	35 - 50	*145 - 210*	*180 - 260*

Structure	Location (Relevant Borehole)	Proposed Maximum Embankment Thickness	Settlement (mm)		
			Immediate	Estimated Post-Construction Settlement over a 20-Year Period	Total
E-S Ramp	West Approach (400-4)	9 m	70 - 100	*100 - 125*	*170 - 225*
N-E Ramp	East Approach (400-3)	16 m	50 - 75	*150 - 300*	*200 - 375*
	West Approach (400-2)	5m	25 - 50	*5*	*30 - 55*

Based on the estimated magnitude of settlement above, settlement mitigation options will be required at both approach embankments for the E-S Ramp structure, and the east approach embankment of the N-E Ramp structure for the to meet the settlement performance criterion.

6.6.2.1 Mitigation Options

Given that the compressible soils (i.e. clayey silt to silty clay) are considered to be over consolidated, the majority of the settlement is anticipated to occur rapidly during or shortly after construction (see Section 6.6.2.2). Several settlement mitigation options have been considered to meet the settlement performance criterion and a brief discussion on these alternatives is provided in the bullet points below. Full sub-excavation and replacement is not considered suitable or cost effective due to the size of the footprint of the embankment, and thickness and depth of the compressible deposits. Other ground improvement measures such as the use of wick drains, rammed aggregate piers, deep soil mixing, and dynamic compaction are considered feasible and should be investigated during detail design as applicable.

- **Preloading:** Due to the thickness of the cohesive layers observed, and the drainage boundaries (i.e. cohesionless layers) observed throughout the cohesive deposits at the site, preloading is expected to be effective in reaching the settlement performance criterion. A settlement instrumentation and monitoring plan would be required during construction to assess when the settlement performance criterion has been achieved.

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

- **Lightweight Expanded Polystyrene:** The use of expanded polystyrene (EPS) is another alternative that can be considered to significantly reduce the magnitude of consolidation settlement. Where required, EPS can be used to achieve the settlement performance criterion without preloading and therefore, will reduce the length of time for construction. Given the relatively short preload time anticipated for most approach embankments with using conventional fill (see next section), the impact on the construction schedule may not be significant and given the additional handling requirements and high cost of EPS compared to other lightweight and conventional granular fills, this option may not be practical.

Based on the above considerations, preloading is considered the technically preferred alternative to mitigate long-term post-construction settlement at this site.

6.6.2.2 Preloading

Based on the estimated coefficient of consolidation (c_v) of between 3.7x10^{-3} and 9.4x10^{-3} cm^2/s for the over consolidated cohesive deposit, it is estimated that the following preload periods will be required for each approach embankment area to meet the settlement performance criterion assuming the embankments are constructed of granular fill.

Structure / Location	Height of Embankment (m)	Estimated Preload Period[1] (days)
E-S Ramp / East Approach Embankment	11	60 - 90
E-S Ramp / West Approach Embankment	9	30 - 60
N-E Ramp / East Approach Embankment	16	60 - 90

Notes:

1. Time for preload to remain in place to reduce future primary consolidation settlements to less than 25 mm over 20 year period.

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

As mentioned in Sections 6.4.1 and 6.4.2.4, 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 spread footings (excess settlement in addition to the f-SLS geotechnical resistance) and/or deep foundations (i.e., associated downdrag forces).

Consideration will need to be given to differential settlements if the ultimate configuration of the approach embankments are not constructed at the same time (i.e., if consideration is being given to constructing an interim configuration to be widened in the future).

6.6.3 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 OPSD 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).

6.7 Corrosion Assessment and Protection

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

6.7.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-19 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* for potential sulphate attack on concrete. The sulphate concentration measured in the tested samples was less than 20 µg/g (< 0.002%) and are below the exposure class of S-3 (Moderate). Therefore, based on the soil samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

6.7.2 Potential for Corrosion

The test results indicate a pH ranging from 7.6 to 7.9 and a resistivity ranging from 1100 to 5300 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to durability. However, the resistivity indicates that the soil corrosiveness ranges from Low (6000 ohm-cm > R > 4500 ohm-cm) to Severe (R < 2000 ohm-cm), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and appropriate corrosion protection should be applied to the foundation element / materials. Further, given that the foundations are located adjacent to the highway shoulder and will be exposed to de-icing salt, consideration should be given to selection of a "C" type exposure class 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 the design service of the structure foundations and determine the appropriate exposure class and ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.8 Construction Considerations

6.8.1 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. from farming activities) be stripped from the embankment footprint and replaced with OPSS Select Subgrade Material (SSM), Granular A or Granular B soils. Based on the boreholes, stripping up to about 1.5 m below ground surface (possibly up to 2 m near borehole 400-1) may be required to remove the unsuitable soils at the approach embankments; stripping requirements must be confirmed following completion of additional boreholes during detail design.

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 may also be considered where sufficient volumes are available. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). Permanent embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular fill. Where earth fill is used, slightly flatter side slopes on the order of 2.25H:1V may be necessary depending on the composition of the material to reduce the potential for shallow surficial failures and should be assessed during detail design.

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

To reduce surface water erosion on the granular embankment side slopes, vegetative cover should be established as per OPSS.PROV 803. Depending on the time of year, temporary erosion control measures such as mulch, bonded fibre matrix (BFM), fiber reinforced matrix (FRM), or erosion control blankets (ECB), should be applied as per OPSS.PROV 804 (*Temporary Erosion Control*) as soon as possible after construction of the embankments.

6.8.2 Temporary Excavations

Temporary excavations up to 1.5 m are anticipated for construction of pile or caisson caps, with excavations up to 1.8 m required for shallow foundations (if being considered).

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. The soft clayey silt and very loose to loose silty sand and silt (encountered in the upper 2 m of Boreholes 400-4 and 400-2) are classified as Type 4 soils, as are silty sand deposits below the ground water table. The existing fill and the native firm to very stiff clayey silt to silt clay deposits are classified as Type 3 soils. Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes of no steeper than 1H:1V sloped from the bottom of the excavation for Type 3 soils, and with side slopes no steeper than 3H:1V sloped from the bottom of the excavation for Type 4 soils.

Temporary protection systems may be required for the construction of the pier foundations adjacent to Highway 400. 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.

6.8.3 Groundwater / Surface Water Control

The groundwater level measured during the foundation investigation varied between 252.9 m and 264.6 m across the Bradford Bypass-Highway 400 Interchange area. Near the west abutment of the N-E Ramp bridge (in the monitoring well installed in Borehole 400-2) the groundwater level was measured at about Elevation 264.6 m (about 3.7 meters below ground surface (mbgs)). Near the east abutment of both bridges (in the monitoring well installed in Borehole 400-3) the groundwater level was measured at about Elevation 257.5 m (about 0.9 mbgs), and near the west abutment of the E-S Ramp bridge (Borehole 400-4) the unstabilized groundwater level was measured at Elevation 252.9 m (about 0.9 mbgs) in the open borehole during drilling operations.

At this preliminary stage it is anticipated that temporary excavations for shallow foundations (if considered) or pile caps may extend below the shallow groundwater table on the east side of Highway 400 and at the west abutment of the E-S ramp bridge. The temporary excavations for the N-E Ramp bridge west abutment will likely be above the groundwater table. As it is expected that limited excavation (less than 1.8 m deep) will be required for foundations, groundwater seepage into the foundation excavations can likely be adequately controlled by ditching and pumping from filtered sumps within or adjacent to the excavations. Dewatering efforts are anticipated to increase near the south end of the site (near the west abutment of the E-S Ramp bridge) which is located in a lower lying area and where a creek and culvert crossing Highway 400 are located about 200 m south of the abutment location.

If the excavation operations are carried out in the wet season, the groundwater level could be higher (especially at the west abutment of the E-S ramp) and more extensive groundwater control measures may be required depending on the excavation requirements.

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 during detail design to address potential instability / base heave of the foundation subgrade, temporary flow diversion and pre-construction survey requirements, as applicable.

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

Surface water must be directed away from the excavations at all times. In particular, surface water drainage at the west abutment of the N-E Ramp must be 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).

6.8.4 Obstructions during Pile Driving / Caisson Installation

During pile installation through the glacially derived soils, especially the till and the “100-blow” silty sand soil layers at this site, there is a risk of encountering pockets of gravel and/or cobbles and boulders. It is recommended that steel H-piles or tube piles be reinforced and protected from damage with appropriate driving shoes as per OPSD 3000.100 (Steel H-Pile Driving Shoe) or 3001.100 (Steel Tube Driving Shoe) or equivalent. Pre-augering may be considered where 100-blow soils are present at shallow depth (as at Borehole 400-2), to reduce the risk of piles “hanging up” on potential “100-blow” stratum. If pre-augering is considered, the design geotechnical resistances provided must be reviewed and revised as necessary during detail design. Caisson installation equipment must be capable of penetrating and/or removing obstructions as required.

6.9 Recommendations for Additional Work

The preliminary foundation recommendations provided in this report are based on the limited available subsurface information in the four boreholes advanced near the proposed 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 configurations.

The additional investigation will need to explore the subsurface soil and groundwater conditions closer to and at the location of the bridge foundation elements (abutments and pier locations), approach embankments, and any associated retaining walls. In particular, the locations of both abutments of the proposed E-S Ramp bridge should be investigated as the closest boreholes are more than 100 m from the foundation footprint. The west abutment of the N-E Ramp bridge should also be investigated (on the apparent “hill” where the ground surface at the proposed abutment is about 5 m higher than the ground surface at the closest borehole) to check and confirm foundation soils and groundwater levels. The locations of the abutments and piers should be confirmed and boreholes advanced closer to the foundation elements accordingly, particularly on the east side of Highway 400 where a watercourse flows directly adjacent to the highway and at pier locations near Highway 400. Boreholes should be advanced below the anticipated pile tip elevations and beyond 30 m depth to confirm the presence and thickness of the “100-blow” soils and confirm long friction pile assumptions as required for detail design. In-situ vane tests and undisturbed samples of the cohesive deposits should be collected to carry out a sufficient number of complex laboratory tests (i.e. consolidation tests, and triaxial tests, as applicable) to characterize the cohesive deposits and till deposits encountered on this site. It is recommended that seismic Cone Penetration Testing also be performed through the clayey silt to silty clay deposit to provide more detailed information to assess anticipated settlement and rates of consolidation. Also, pressuremeter testing is recommended in the very loose to compact silt and sand to silty sand (till) soils to better predict actual magnitudes of settlement and risks associated with staged construction (i.e. differential settlement) and downdrag forces on deep foundations.

Additionally, given that the seismic Site Class based on $\overline{N}_{60}$ indicated that the site ranges from a Site Class D to E, geophysics testing should be considered to measure shear wave velocities. Geophysics testing, such as Multi-Channel Analysis of Surface Waves (MASW) or Vertical Seismic Profiling (VSP), may provide a more favourable and consistent Site Class designation across the site, and such testing should be considered during detail design. The use of GSC 5th Generation or 6th Generation seismic hazard maps to define the Site Class should be confirmed for detail design.

After more detailed foundation investigation is complete, the global stability of the approach embankments and any retaining walls will need to be checked and the magnitude and time-rate of settlements (including mitigation measures) will need to be reassessed. When more details are known on actual loading conditions, the foundation types, sizes and geotechnical resistances will need to checked and revised as necessary. Given the variable subsurface conditions at this site, differential settlement across and between founding elements of the structure(s) should also be assessed.

Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*. The existing standpipe piezometers (installed in Boreholes 400-2 and 400-3) should be maintained operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometers will need to be decommissioned in accordance with Ontario Regulation 903 (as amended). Additional piezometers (particularly near the E-S ramp bridge abutments) should be installed near the proposed foundation elements to provide additional information for assessment of dewatering requirements.

7.0 CLOSURE

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

WSP Golder

Madison C. Kennedy, P.Eng.
Geotechnical Engineer

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

MTI/MCK/KJB/al

https://golderassociates.sharepoint.com/sites/120387/project files/6 deliverables/foundations/highway 400/final/19136074 hwy 400-bbp interchange pfidr-rev0_29sept23.docx

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Canadian Standards Association (CSA) Group, 2019. Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-19.

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

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Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Westergaard, H.M. 1938. A problem of elasticity suggested by a problem in soil mechanics: Soft material reinforced by numerous strong horizontal sheets. Contributions to the Mechanics of Solids, Stephen Timoshenko 60th birthday anniversary volume, Macmillan, New York. Pages 268-277.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.
ASTM D1143	Standard Test Methods for Deep Foundation Elements Under Static Axial Compressive Load

Commercial Software:

Settle3 (Version 5.012) by Rocscience Inc.

Slide2 (Version 9.014) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 803	Construction Specification for Vegetative Cover
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.PROV 902	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 213	Construction Projects (as amended)
Ontario Regulation 903	Wells (as amended)

Ministry of Transportation, Ontario

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

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

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

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

TABLES

Table 1: Comparison of Foundation Alternatives – Bradford Bypass / Highway 400 Interchange Ramp Structures

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread footings founded on native compact silty sand, stiff to very stiff clayey silt to silty clay, or very stiff clayey silt till	■ Marginally feasible except at west abutment of N-E bridge	■ Conventional construction ■ Relatively competent soils may provide adequate geotechnical resistance at west abutment of N-E bridge	■ Anticipated high loading requires large foundation widths and native foundation soils can only offer a low geotechnical resistance at f-SLS and likely not feasible. ■ Anticipated settlement / consolidation of foundation soils due to embankment loading will exceed tolerable limits (25 mm) at abutments and will need to be mitigated (e.g. ground improvement such as preloading). ■ Subexcavation up to 1.8 m bgs anticipated for abutments; however deeper excavations may be required for piers near existing watercourse on east side of Hwy 400. Temporary protection systems likely required to limit footprint and control stability / unbalanced hydrostatic pressures adjacent to Highway 400. ■ Low geotechnical resistance compared to deep foundations ■ Less competent near surface soils (presence of and thicker compressible soils) may exist at actual abutment and/or pier locations.	■ Lower cost than deep foundations where feasible at N-E west abutment.	■ High anticipated structure loads will require large footing widths resulting in reduced f-SLS geotechnical resistances (compared to smaller footing widths) that will govern design. ■ Risk of excess total and differential settlement due to anticipated high foundation loads, approach embankment loads, and variable soil conditions. Settlement mitigation and monitoring required. ■ Risk of variable soil conditions and increased subexcavation depth of unsuitable soils (e.g. compressible soils or organics) near low-lying areas near watercourse on east and west side of Hwy 400 for E-S Ramp bridge.
Driven Steel Piles	■ Feasible for all foundation elements	■ Conventional construction methods for driven H-pile foundations. ■ Higher axial resistances compared to shallow footings. ■ Larger H-piles or tube piles can be considered to increase axial resistance. ■ Perched abutments can be considered to reduce dewatering / subexcavation for pile caps. ■ Likely feasible and preferred if deeper unsuitable soil deposits are encountered near ground surface within footprint.	■ Dewatering measures may be required for pile caps if they cannot be perched. ■ Relatively long (greater than 30 m) piles will be required at some locations and will be designed mainly on skin friction as there was no confirmed hard / very dense end-bearing stratum encountered within a 50 m depth. ■ Presence of and thickness of “100-blow” soil needs to be confirmed during detail design; otherwise, resistances may need to be reduced and/or longer piles required.	■ Lower relative cost than drilled shafts (caissons)	■ Variable soil conditions (deeper soft or very loose or unsuitable soil deposits) closer to foundation locations may lead to longer pile lengths. ■ Risk of lower geotechnical resistances during installation for friction pile design at some locations. A longer wait time to allow pore-water pressures to dissipate may be required when testing production piles. Alternatively, advanced static load testing may be considered. ■ Settlement of approach embankments could cause potential downdrag loads on piles (reduced capacity) unless mitigation and monitoring is provided during construction. ■ Risk of damage to pile due to driving >30 m at some locations and through hard / very dense deposits possibly containing cobbles / boulders. Driving shoes and/or thicker pile section, and possible pre-augering required at some locations.
Drilled Shafts (Caissons)	■ Feasible to marginally feasible	■ Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. ■ Larger diameter caissons can be considered to increase axial resistance. ■ May be designed to eliminate caisson caps and temporary excavations / protection systems as the caisson could be cast continuously with structural columns to underside of superstructure. Associated dewatering efforts reduced compared to shallow foundation and pile cap construction.	■ Long drilled shafts (in excess of 30 m) likely required at some locations and will be challenging from constructability perspective (may not be feasible). ■ Given that there was no confirmed hard / very dense end-bearing stratum encountered within a 50 m depth at some locations, the caisson design is based mainly on skin friction and offers limited increase in resistance compared to driven piles. ■ Temporary or permanent liner will be required, plus special measures such as use of polymer slurry to counterbalance hydrostatic head to reduce risk of loosening / softening of the sides of excavation and “blow-out” at base of shaft during drilling and concrete placement (by tremie methods). ■ Generation, containment and disposal of soil cuttings / slurry during caisson advancement.	■ Higher relative cost than driven piles.	■ Variable soil conditions (deeper compressible or unsuitable soil deposits) closer to foundation elements may lead to longer caissons (>30 m) which may not be practical from constructability perspective. ■ Risk of lower geotechnical resistances during detail design and installation procedures for friction caisson design. Higher geotechnical capacities could be considered if advanced load testing is considered (e.g. Osterberg Cell Test or Static Load Test). ■ Settlement of approach embankments will cause potential downdrag loads on caissons (reduced capacity) unless mitigation and monitoring is provided during construction. ■ Challenges associated with inspection of shaft walls and base may lead to conservative friction design and longer caissons.

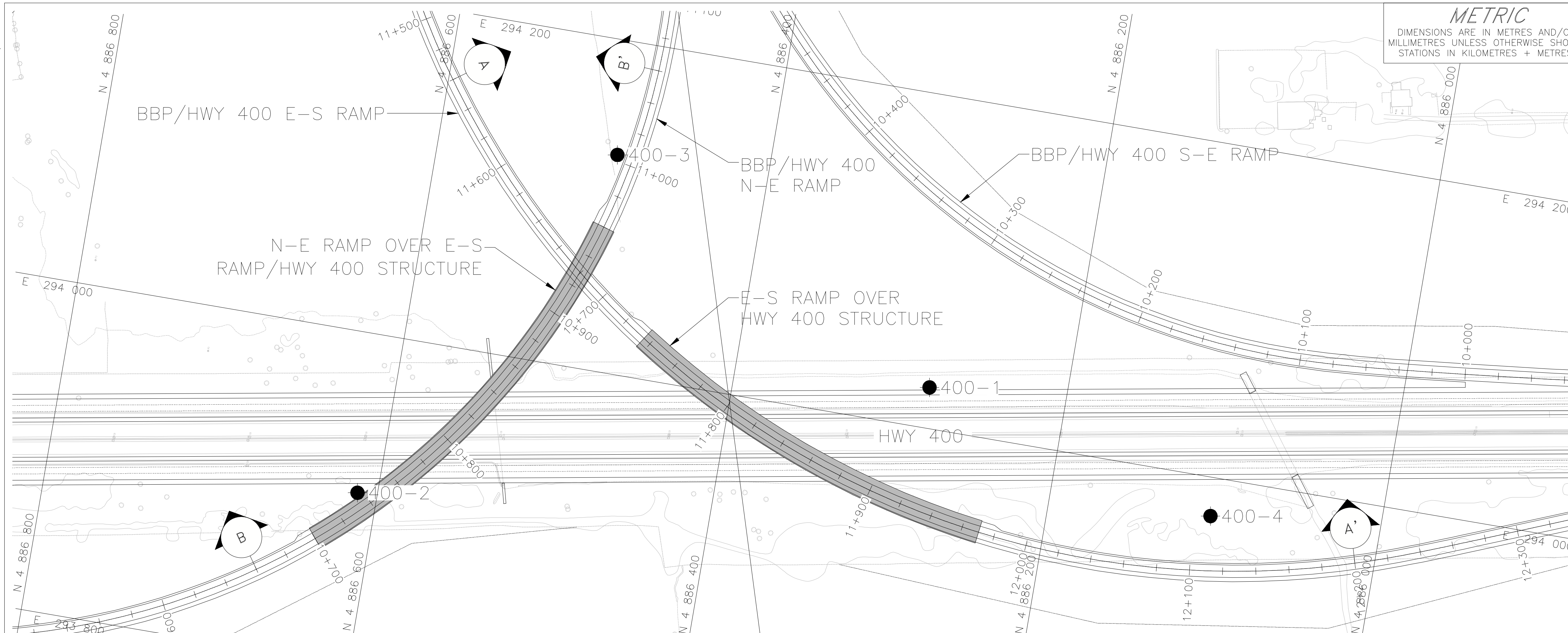
DRAWINGS

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

CONT No.
WP No.

BRADFORD BYPASS
HWY 400 INTERCHANGE RAMP STRUCTURES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



PLAN
SCALE
30 0 30 60 m

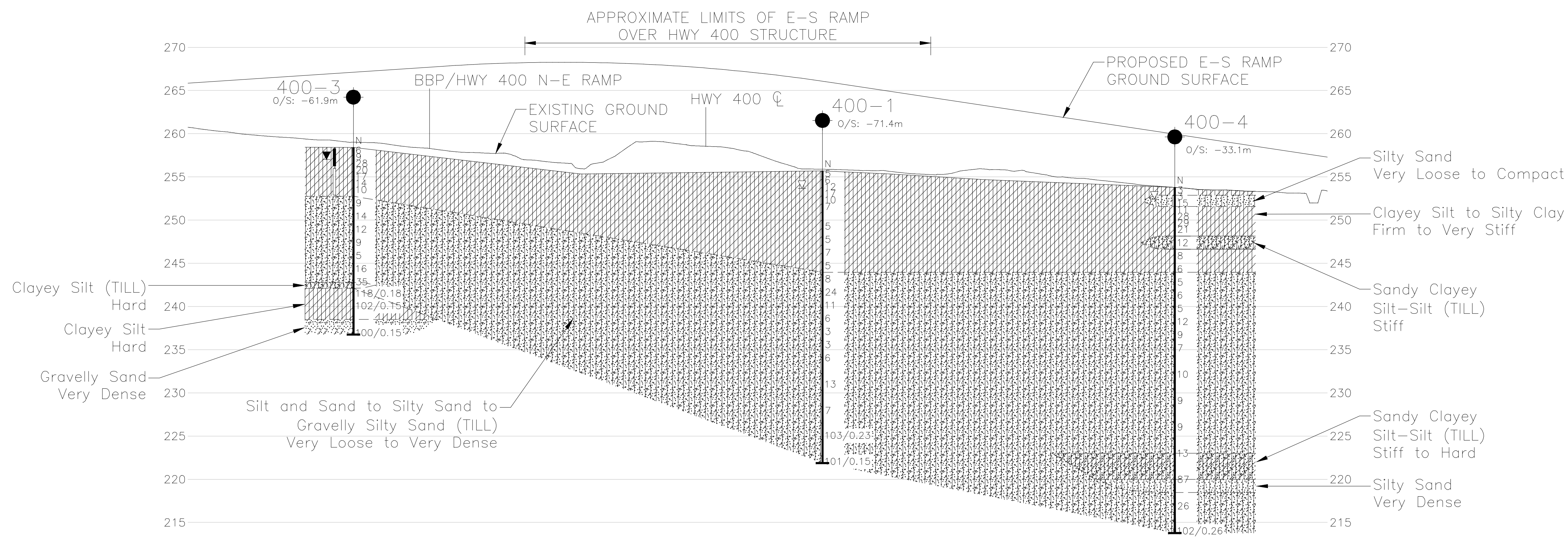
KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
400-1	255.7	4886282.5	294024.6
400-2	268.3	4886611.9	293903.7
400-3	258.4	4886491.8	294131.0
400-4	253.8	4886102.3	293976.8



PROFILE A-A' – BBP/HWY 400 E-S RAMP ℄

HORIZONTAL SCALE
30 0 30 60 m

VERTICAL SCALE
6 0 6 12 m

NOTES

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

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

REFERENCE

Base plans provided in digital format by Aecom, drawing file nos. 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 September 14, 2022.
Design plan provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Overall Plan.dwg, received September 14, 2022.
Vertical alignment provided in digital format by Aecom, drawing file no. ACAD-X-60636190-C-DES-Hwy 400 IC Plan and Profile.dwg, received February 21, 2023.

LICENSED PROFESSIONAL ENGINEER M. C. KENNEDY 100547814 Sep. 29, 2023 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER K. J. BENTLEY Sep. 29, 2023 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D-832			
HWY. BRADFORD BYPASS	PROJECT NO. 19136074		DIST.
SUBM'D. MCK	CHKD. MCK	DATE: 09/29/2023	SITE:
DRAWN: DD	CHKD. KJB	APPD. KJB	DWG. 1

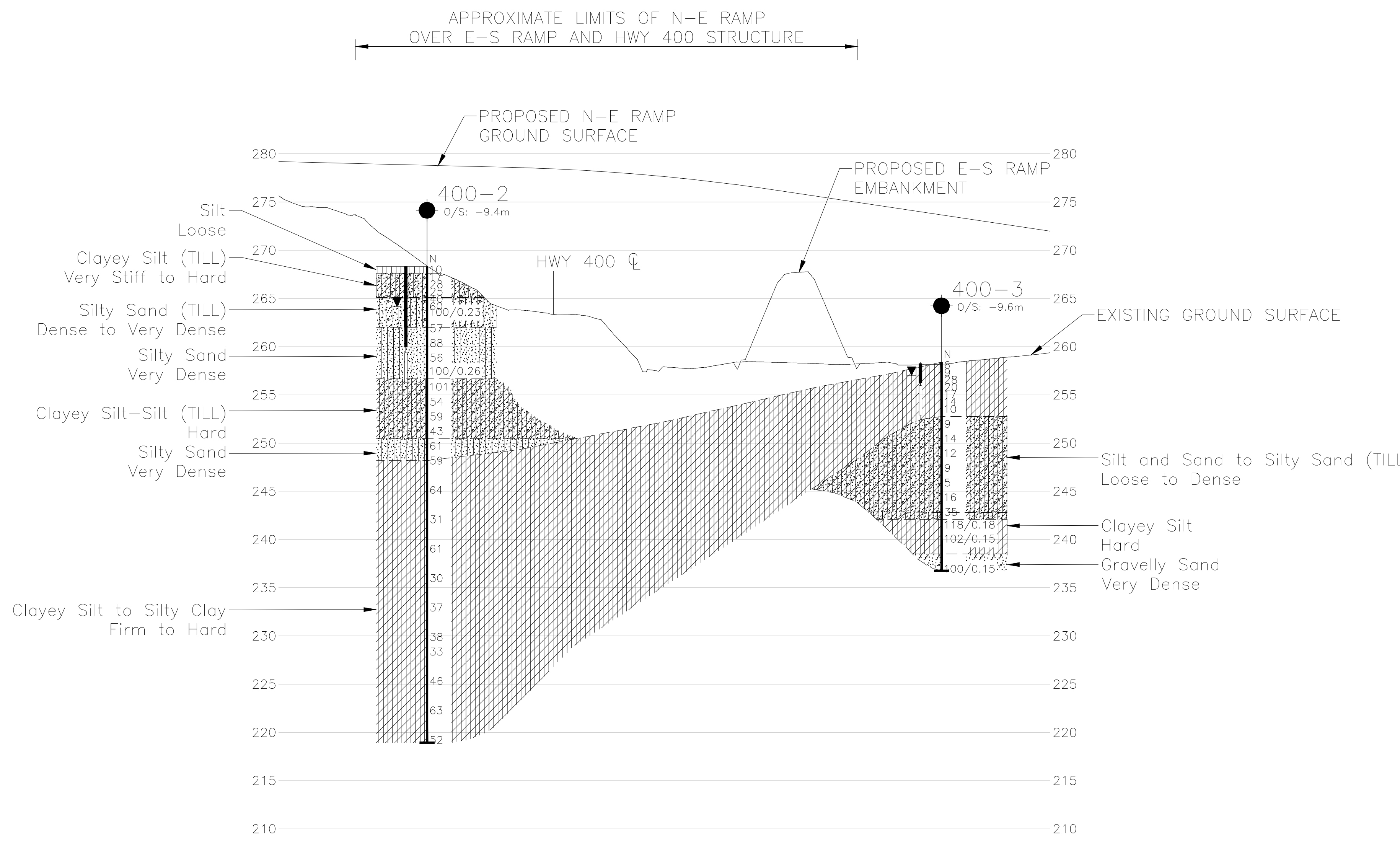
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.

BRADFORD BYPASS
HWY 400 INTERCHANGE RAMP STRUCTURES

SOIL STRATA

SHEET



PROFILE B–B' – BBP/HWY 400 N–E RAMP ℄

HORIZONTAL SCALE 30 0 30 60 m

VERTICAL SCALE 6 0 6 12 m

KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer
- WL upon completion of drilling

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
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400–2	268.3	4886611.9	293903.7
400–3	258.4	4886491.8	294131.0
400–4	253.8	4886102.3	293976.8

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REFERENCE

Vertical alignment provided in digital format by Aecom, drawing file no. ACAD–X–60636190–C–DES–Hwy 400 IC Plan and Profile.dwg, received February 21, 2023.

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LICENSED PROFESSIONAL ENGINEER K. J. BENTLEY Sep. 29, 2023 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D–832

HWY. BRADFORD BYPASS	PROJECT NO. 19136074		DIST.
SUBM'D. MCK	CHKD. MCK	DATE: 09/29/2023	SITE:
DRAWN: DD	CHKD. KJB	APPD. KJB	DWG. 2

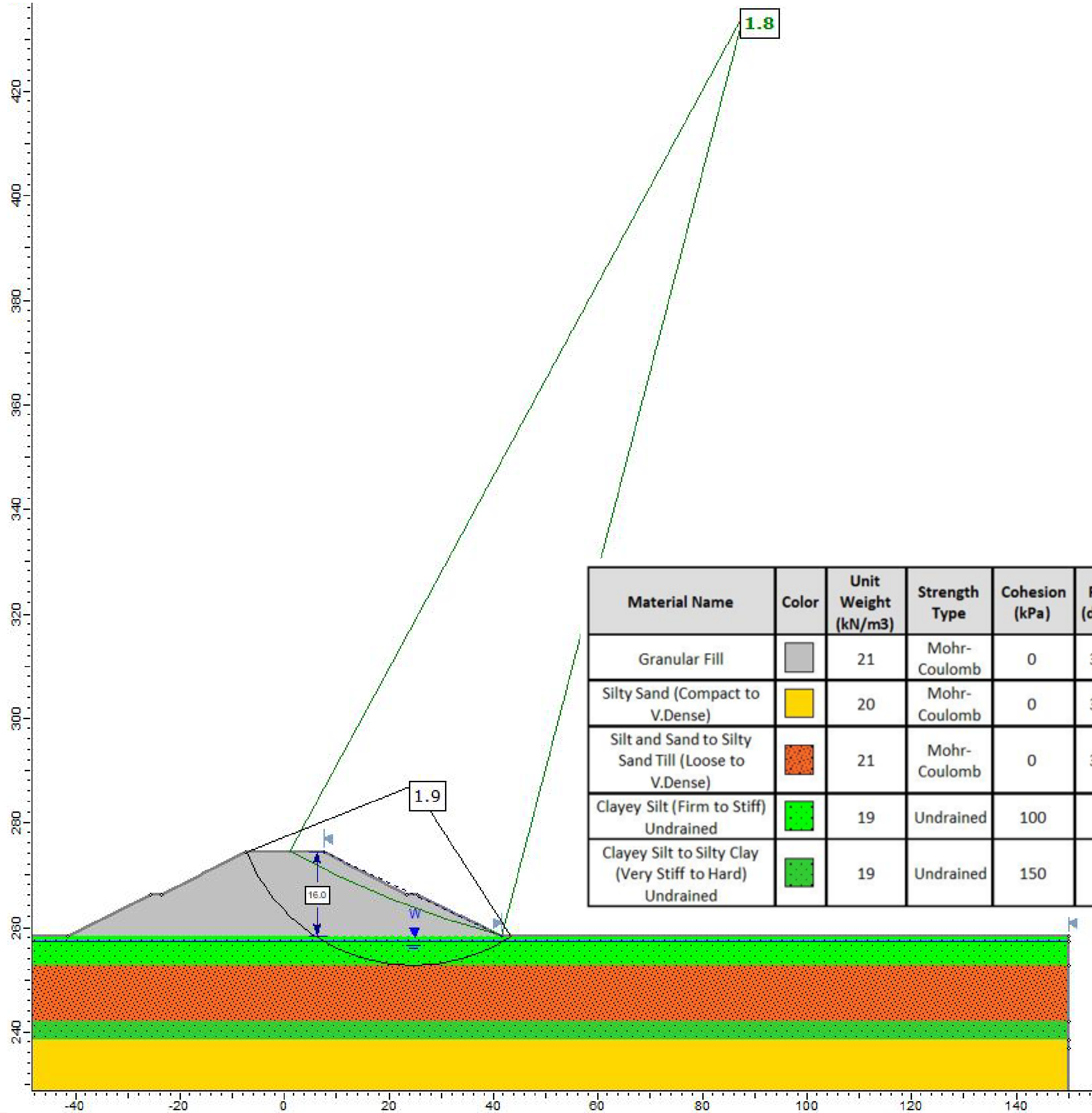
PLOT DATE: September 29, 2023
FILENAME: S:\Clients\MTO\Bradford_Bypass\99_PROJ\19136074_Aecom\40_PROD\0007_HWY_400_New_Interchange_BR\19136074-0007-BG-0002.dwg

FIGURES

Stability Analysis Results (Undrained Condition)

N-E Ramp East Approach Embankment - Highway 400

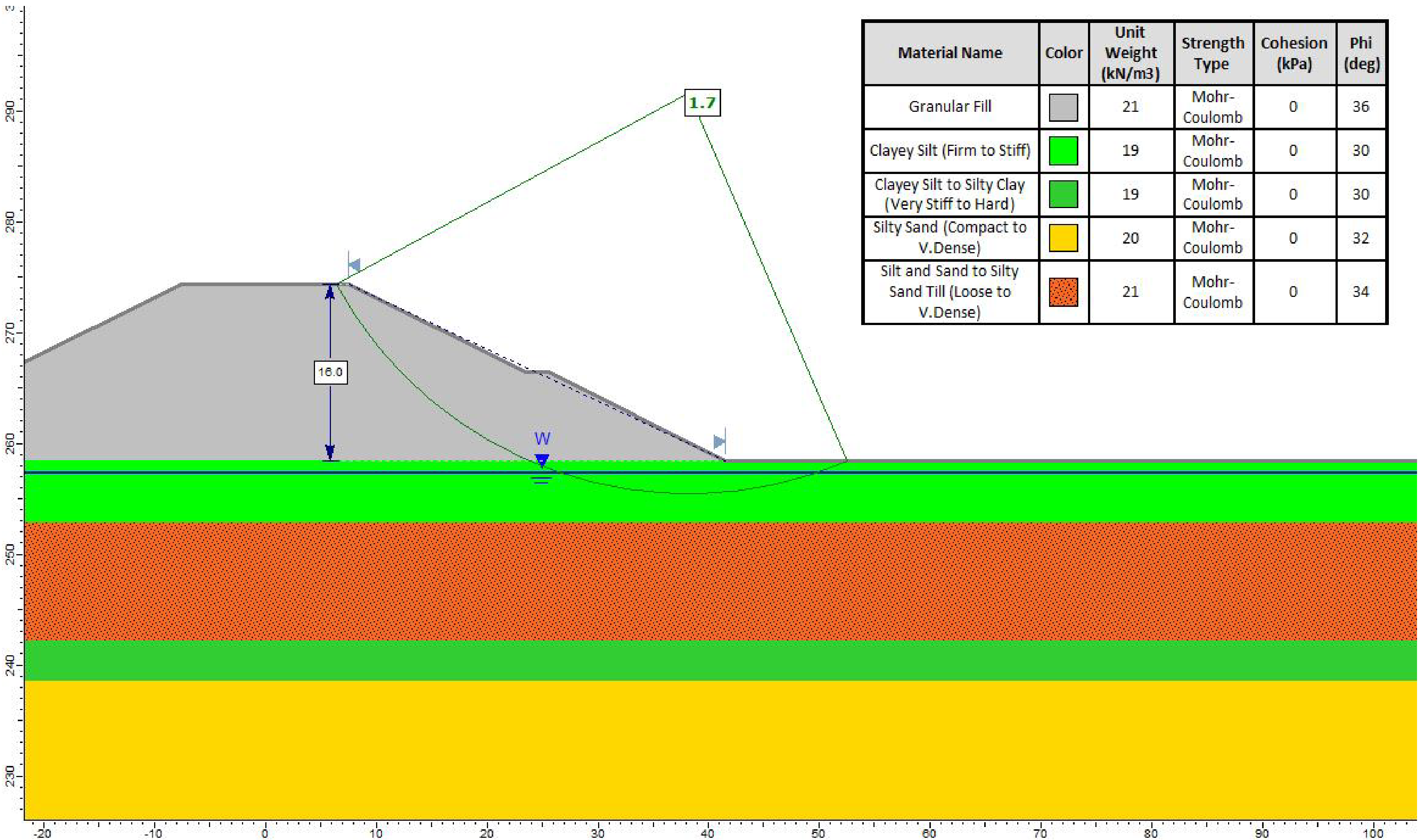
Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill		21	Mohr-Coulomb	0	36
Silty Sand (Compact to V.Dense)		20	Mohr-Coulomb	0	32
Silt and Sand to Silty Sand Till (Loose to V.Dense)		21	Mohr-Coulomb	0	34
Clayey Silt (Firm to Stiff) Undrained		19	Undrained	100	
Clayey Silt to Silty Clay (Very Stiff to Hard) Undrained		19	Undrained	150	

Stability Analysis Results (Drained Condition)
N-E Ramp East Approach Embankment - Highway 400

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill		21	Mohr-Coulomb	0	36
Clayey Silt (Firm to Stiff)		19	Mohr-Coulomb	0	30
Clayey Silt to Silty Clay (Very Stiff to Hard)		19	Mohr-Coulomb	0	30
Silty Sand (Compact to V.Dense)		20	Mohr-Coulomb	0	32
Silt and Sand to Silty Sand Till (Loose to V.Dense)		21	Mohr-Coulomb	0	34

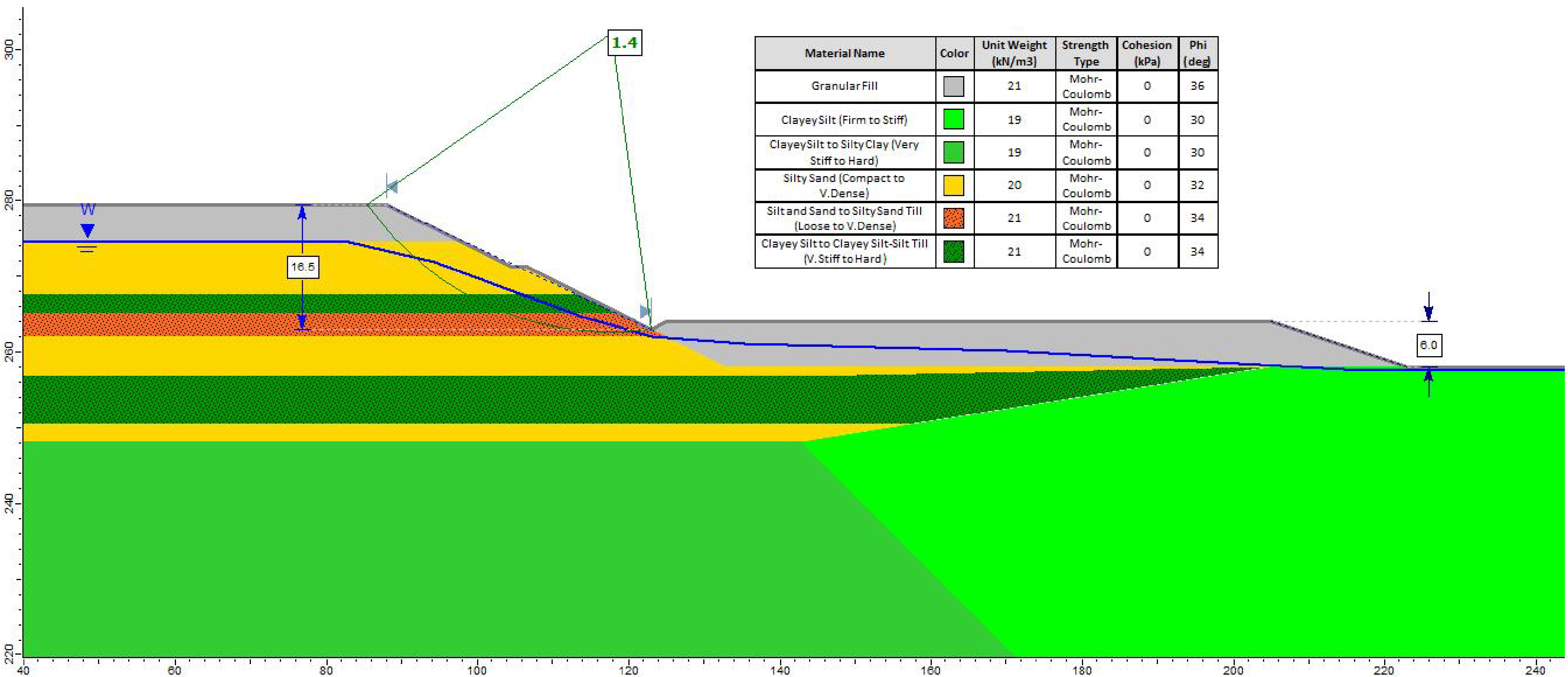
Date: September 2023
Project No: 19136074

Analysis By: MCK **Reviewed By:** KJB

Stability Analysis Results (Undrained Condition)

N-E Ramp West Approach Embankment - Highway 400

Figure 3



Date: September 2023

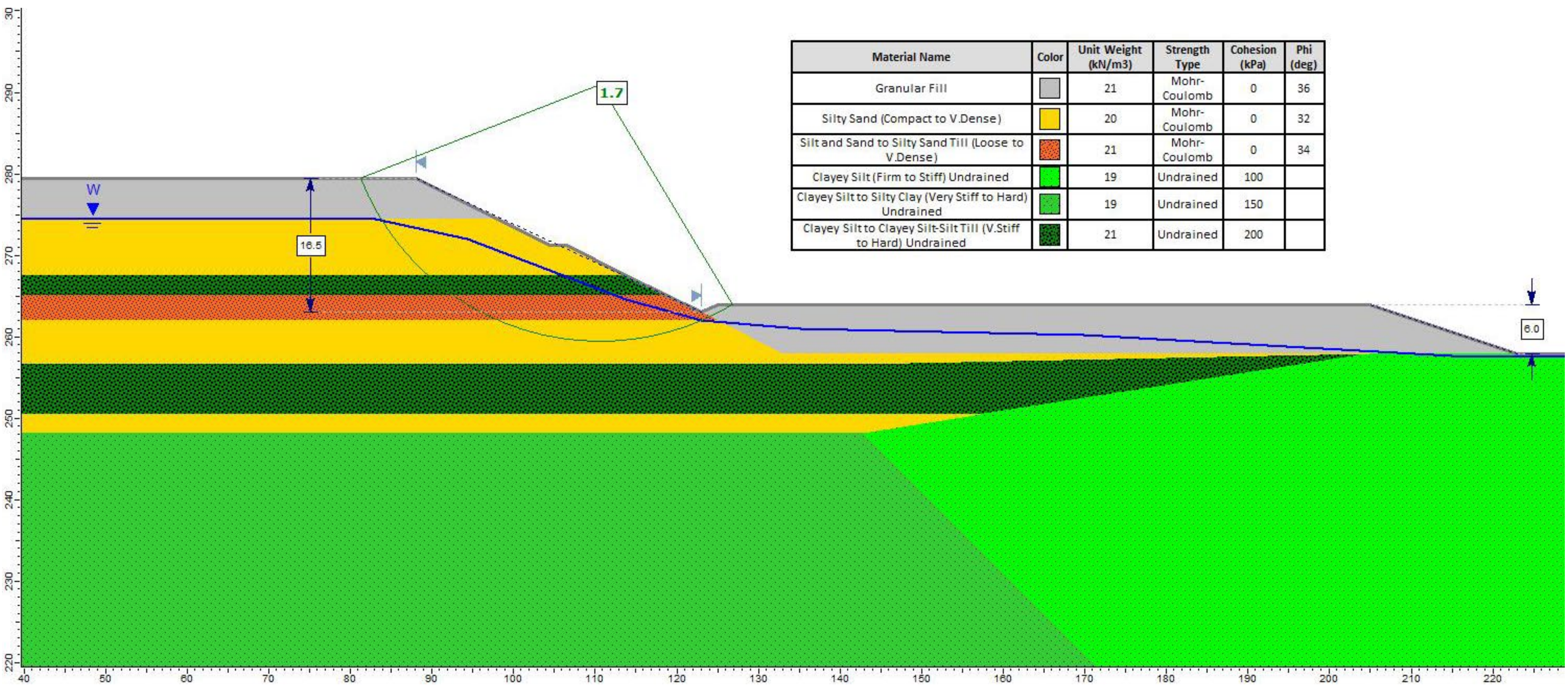
Project No: 19136074

Analysis By: MCK **Reviewed By:** KJB

WSP GOLDER

Stability Analysis Results (Drained Condition)
N-E Ramp West Approach Embankment - Highway 400

Figure 4



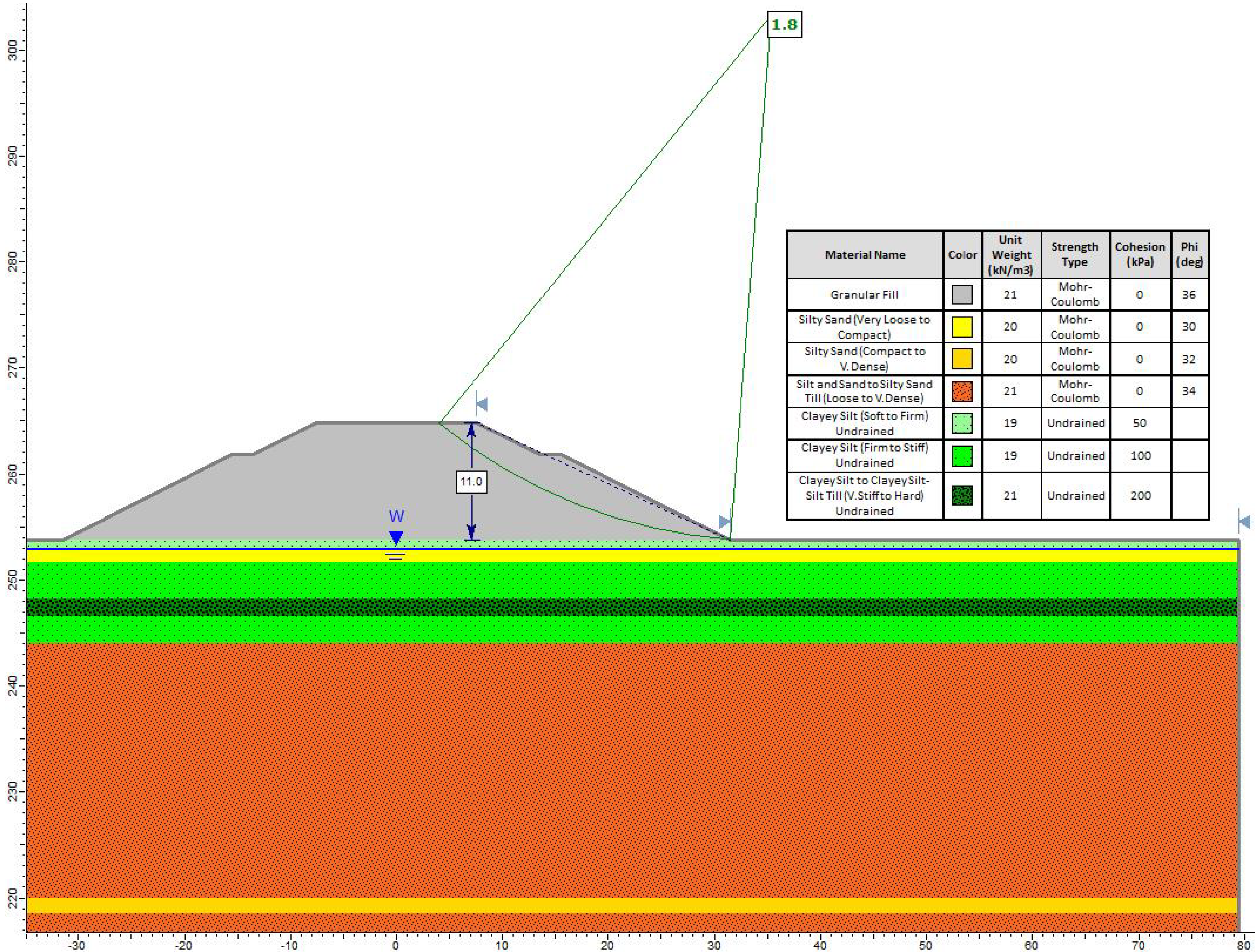
Date: September 2023
Project No: 19136074

Analysis By: MCK **Reviewed By:** KJB

Stability Analysis Results (Undrained Condition)

E-S Ramp West Approach Embankment - Highway 400

Figure 5



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill		21	Mohr-Coulomb	0	36
Silty Sand (Very Loose to Compact)		20	Mohr-Coulomb	0	30
Silty Sand (Compact to V. Dense)		20	Mohr-Coulomb	0	32
Silt and Sand to Silty Sand Till (Loose to V. Dense)		21	Mohr-Coulomb	0	34
Clayey Silt (Soft to Firm) Undrained		19	Undrained	50	
Clayey Silt (Firm to Stiff) Undrained		19	Undrained	100	
Clayey Silt to Clayey Silt-Silt Till (V. Stiff to Hard) Undrained		21	Undrained	200	

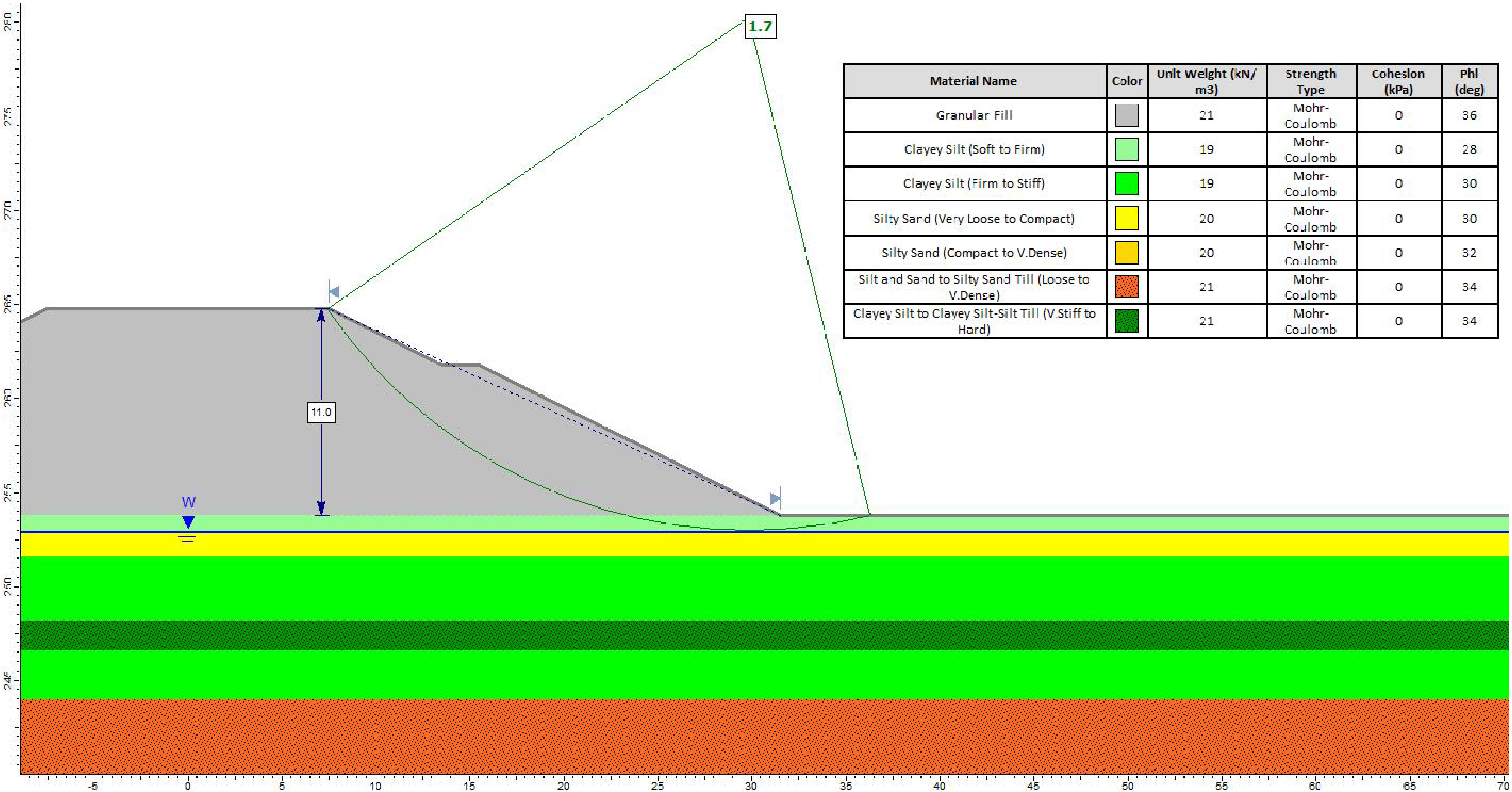
Date: September 2023

Project No: 19136074

Analysis By: MCK **Reviewed By:** KJB

Stability Analysis Results (Drained Condition)
E-S Ramp West Approach Embankment - Highway 400

Figure 6



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill		21	Mohr-Coulomb	0	36
Clayey Silt (Soft to Firm)		19	Mohr-Coulomb	0	28
Clayey Silt (Firm to Stiff)		19	Mohr-Coulomb	0	30
Silty Sand (Very Loose to Compact)		20	Mohr-Coulomb	0	30
Silty Sand (Compact to V.Dense)		20	Mohr-Coulomb	0	32
Silt and Sand to Silty Sand Till (Loose to V.Dense)		21	Mohr-Coulomb	0	34
Clayey Silt to Clayey Silt-Silt Till (V.Stiff to Hard)		21	Mohr-Coulomb	0	34

Date: September 2023
Project No: 19136074

Analysis By: MCK **Reviewed By:** KJB

APPENDIX A

Boreholes

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

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

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

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

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

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

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

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

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

Ontario

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PROJECT	19136074		RECORD OF BOREHOLE No. 400-1		Sheet 1 of 4		METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886282.5; E 294024.6 NAD83 / MTM Zone 10 (LAT. 44.116458; LONG. -79.634623)			ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary			COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:255.7 m	DATE	Dec 07, 2021 - Dec 09, 2021			CHECKED BY	KJB

Dynamic cone penetration resistance plot: 20, 40, 60, 80, 100. Shear strength (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, × Field Vane. Water content (%): PL (W_p), NMC (W), LL (W_l); 20, 40, 60; NP Nonplastic. Unit weight γ kN/m³.

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	GR	SA	SI	CL	REMARKS
0.0	CLAYEY SILT (CL), some sand, trace rootlets, trace organics Firm Brown Moist	1	SS	5						
		2	SS	6						
254.2 / 1.4	- 1.4 to 2.3 m: Oxidation staining SILTY CLAY (CI), trace sand, trace gravel, containing rootlets to a depth of 2.1 m Stiff to very stiff Dark brown Moist - 1.6 m: Becoming light brown	3	SS	12	∇ (approx. 254.1)					
		4	SS	17		0	0	49	51	
252.7 / 3.0	CLAYEY SILT (CL), trace sand, trace gravel Firm to stiff Grey Moist	5	SS	10						
		6	SS	7		0	0	69	31	
		7	TO							C
		8	SS	5						
		9	TO							C
		10	SS	5		0	2	50	48	
		11	SS	7						

Elevation scale: 255, 254, 253, 252, 251, 250, 249, 248, 247, 246

Continued on Next Page

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

Ontario

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PROJECT	19136074	RECORD OF BOREHOLE No. 400-1	Sheet 2 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION: N 4886282.5; E 294024.6 NAD83 / MTM Zone 10 (LAT. 44.116458; LONG. -79.634623)	ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE: 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:255.7 m	DATE: Dec 07, 2021 - Dec 09, 2021	CHECKED BY	KJB

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100. Shear Strength (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane. Water Content (%): PL (W_p), NMC (W), LL (W_l); NP Nonplastic; 20 40 60. Unit Weight γ kN/m³. Grain size: GR SA SI CL.

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL	REMARKS
	CLAYEY SILT (CL), trace sand, trace gravel Firm to stiff Grey Moist				245					
	- 11.0 to 11.1 m: Becoming sandy	12	SS	5						
244.0 / 11.7	SILT (ML) and sand, trace gravel (TILL) Very loose to compact Grey Moist				244					
		13	SS	8	243					
		14	SS	24	242					
					241					
		15	SS	11	240					
					239					
		16	SS	6		5	45	40	10	
					238					
		17	SS	3	237					
236.4 / 19.4	SILTY SAND (SM), trace gravel (TILL) Very loose to compact Grey Wet				236					

Continued on Next Page

$+^3$, x^3 : Numbers refer to Sensitivity o^3% STRAIN AT FAILURE

Ontario

wsp

PROJECT	19136074	RECORD OF BOREHOLE No. 400-1	Sheet 3 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886282.5; E 294024.6 NAD83 / MTM Zone 10 (LAT. 44.116458; LONG. -79.634623)	ORIGINATED BY MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY MCK
DATUM	CGVD28 Surface Elevation:255.7 m	DATE	Dec 07, 2021 - Dec 09, 2021	CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	SILTY SAND (SM), trace gravel (TILL) Very loose to compact Grey Wet	18	SS	3						
						235				
		19	SS	6		234				
						233				
						232				
		20	SS	13		231			9 50 31 10	
						230				
						229				
		21	SS	7		228				
						227				
226.4 29.3	Gravelly SILTY SAND (SM) (TILL) Very dense Grey Moist					226				

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

Continued on Next Page

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

Ontario

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PROJECT	19136074	RECORD OF BOREHOLE No. 400-1	Sheet 4 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION: N 4886282.5; E 294024.6 NAD83 / MTM Zone 10 (LAT. 44.116458; LONG. -79.634623)	ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE: 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:255.7 m	DATE: Dec 07, 2021 - Dec 09, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	Gravelly SILTY SAND (SM) (TILL) Very dense Grey Moist								
		22	SS	103/0.23	225	○		21 47 26 6	
					224				
					223				
221.9		23	SS	101/0.15	222				
33.8	End of Borehole								
					221				
					220				
					219				
					218				
					217				
					216				

Notes:
1. Water level measured at a depth of 1.6 m (Elev. 254.1 m) prior to initiation of mud rotary drilling for borehole advancement below a depth of 3.0 m below ground surface.
2. Water level not recorded upon completion of drilling due to the introduction of drilling mud.

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

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

Ontario

wsp

PROJECT 19136074 | **RECORD OF BOREHOLE No. 400-2** | Sheet 1 of 5 | **METRIC**

G.W.P. Assignment No 2019-E-0048 | LOCATION N 4886611.9; E 293903.7 NAD83 / MTM Zone 10 (LAT. 44.119421; LONG. -79.636141) | ORIGINATED BY KR/MTI/DR

DIST Central | HWY BBP - HWY 400 | BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary | COMPILED BY MCK

DATUM CGVD28 Surface Elevation:268.3 m | DATE Nov 09, 2022 - Nov 17, 2022 | CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL	REMARKS
0.0	SILT (ML) some sand, trace organics Loose Dark brown Moist	1	SS	10	268					
267.6 0.7	CLAYEY SILT (CL), some sand, trace gravel, (TILL) Very stiff to hard Brown Moist	2	SS	17	267					
		3	SS	28	266					
		4	SS	25		3	20	50	27	
265.1 3.2	SILTY SAND (SM), trace gravel, (TILL) Dense to very dense Brown Moist	5A			265					
		5B	SS	40						
		6	SS	60	264					
		7	SS	100/0.23	263	8	50	36	6	
262.0 6.3	- 6.3 to 6.7 m: Containing clay laminations and oxidation staining SILTY SAND (SM) trace gravel, Very dense Brown Wet	8A			262					
		8B	SS	57	261					
		9	SS	88	260					
		10	SS	56	259	0	85	14	1	

Sample 10 water content: NP

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

Continued on Next Page

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

Ontario

RECORD OF BOREHOLE No. 400-2

METRIC

PROJECT	19136074		
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886611.9; E 293903.7 NAD83 / MTM Zone 10 (LAT. 44.119421; LONG. -79.636141)
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary
DATUM	CGVD28 Surface Elevation:268.3 m	DATE	Nov 09, 2022 - Nov 17, 2022
ORIGINATED BY	KR/MTI/DR	COMPILED BY	MCK
CHECKED BY	KJB		

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
	SILTY SAND (SM) trace gravel, Very dense, Brown, Wet
256.7 / 11.6	- 11.6 to 12.7 m: Containing sand seams
	CLAYEY SILT-SILT (CL-ML) and sand, trace gravel, (TILL), Hard, Grey, Moist
	- 16.8 to 17.4 m: Containing sand seams
250.5 / 17.8	SILTY SAND (SM), trace gravel, Very dense, Grey, Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
11	SS	100/0.26				
12	SS	101				
13	SS	54				
14	SS	59	4	36	45	15
15	SS	43				
16	SS	61				
17 A						

ELEVATION SCALE: 258, 257, 256, 255, 254, 253, 252, 251, 250, 249

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

REMARKS

Continued on Next Page

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

Ontario

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PROJECT 19136074

RECORD OF BOREHOLE No. 400-2

METRIC

G.W.P. Assignment No 2019-E-0048	LOCATION N 4886611.9; E 293903.7 NAD83 / MTM Zone 10 (LAT. 44.119421; LONG. -79.636141)	ORIGINATED BY	KR/MTI/DR
DIST Central HWY BBP - HWY 400	BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM CGVD28 Surface Elevation:268.3 m	DATE Nov 09, 2022 - Nov 17, 2022	CHECKED BY	KJB

Shear strength (kPa) legend: + Field Vane, × Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined. Dynamic cone penetration resistance plot scale: 20 40 60 80 100. Water content (%): PL (W_p), NMC (W), LL (W_l); NP Nonplastic; scale 20 40 60. Unit weight γ kN/m³.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL	REMARKS
248.2 / 20.1	SILTY SAND (SM), trace gravel Very dense Grey Wet		SS	59						
	CLAYEY SILT (CL), trace sand; containing silt laminations Hard Grey Moist	17 B			248					
					247					
					246					
		18	SS	64	245	0	0	45	55	
					244					
					243					
		19	SS	31	242					
					241					
					240					
	- 29.0 to 29.6 m: Containing sand seams	20	SS	61	239					

Continued on Next Page

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

Ontario

PROJECT	19136074	RECORD OF BOREHOLE No. 400-2	Sheet 4 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION: N 4886611.9; E 293903.7 NAD83 / MTM Zone 10 (LAT. 44.119421; LONG. -79.636141)	ORIGINATED BY	KR/MTI/DR
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE: 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:268.3 m	DATE: Nov 09, 2022 - Nov 17, 2022	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL), trace sand; containing silt laminations Hard Grey Moist						238				
							237				
			21	SS	30		236			0 0 61 39	
							235				
							234				
	- 35.1 to 35.7 m: Containing silt seams		22	SS	37		233				
							232				
							231				
			23	SS	38		230				
							229				
			24	SS	33						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

Continued on Next Page

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

Ontario

PROJECT	19136074	RECORD OF BOREHOLE No. 400-2	Sheet 5 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION N 4886611.9; E 293903.7 NAD83 / MTM Zone 10 (LAT. 44.119421; LONG. -79.636141)	ORIGINATED BY	KR/MTI/DR
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:268.3 m	DATE Nov 09, 2022 - Nov 17, 2022	CHECKED BY	KJB

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL), trace sand; containing silt laminations Hard Grey Moist						228					
							227					
							226					
			25	SS	46		225					
223.8 44.5	SILTY CLAY (CI); containing silt laminations Hard Grey Moist						224					
							223					
			26	SS	63		222		PL–W–LL plotted			
							221					
							220					
218.9 49.4	End of Borehole		27	SS	52		219					

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

Notes:
1. Borehole dry prior to initiation of mud rotary drilling for borehole advancement below a depth of 3.0 m below ground surface.
2. Water level not recorded upon completion of drilling due to the introduction of drilling mud.
3. A monitoring well was installed approximately 1.5 m east of Borehole 400-2 (N 4886612.4; E 293905.1).
4. Water level in piezometer measured at a depth of 3.7 m (Elev. 264.6 m) on February 1, 2023

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

Ontario

RECORD OF BOREHOLE No. 400-3

METRIC

PROJECT	19136074				
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886491.8; E 294131 NAD83 / MTM Zone 10 (LAT. 44.118343; LONG. -79.633298)	ORIGINATED BY	MTI
DIST	Central HWY BBP- HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:258.4 m	DATE	Dec 17, 2021 - Dec 20, 2021	CHECKED BY	KJB

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

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
0.0	CLAYEY SILT (CL), trace sand, trace gravel, containing rootlets to a depth of 1.4 m Firm to very stiff Brown mottled grey with oxidation staining Moist	1	SS	6				
		2	SS	9				
		3	SS	28				
256.2 / 2.2	CLAYEY SILT (CL) Stiff to very stiff Brown to grey with oxidation staining to a depth of 4.4 m Moist	4	SS	20				
		5	SS	17	0	0	64	36
		6	SS	14				
		7	SS	10				
252.8 / 5.6	SILTY SAND (SM), trace to some gravel (TILL) Loose to compact Grey Moist to wet	8	SS	9				
		9	SS	14				
		10	SS	12	19	39	33	9

Elevation scale: 258, 257, 256, 255, 254, 253, 252, 251, 250, 249

Continued on Next Page

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

Ontario

wsp

PROJECT	19136074	RECORD OF BOREHOLE	No. 400-3	Sheet 2 of 3		METRIC	
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886491.8; E 294131 NAD83 / MTM Zone 10 (LAT. 44.118343; LONG. -79.633298)			ORIGINATED BY	MTI
DIST	Central HWY BBP- HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary			COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:258.4 m	DATE	Dec 17, 2021 - Dec 20, 2021			CHECKED BY	KJB

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100. Shear Strength (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane.

Water Content (%): PL (W_p), NMC (W), LL (W_l); NP Nonplastic; 20 40 60.

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
	SILTY SAND (SM), trace to some gravel (TILL) Loose to compact Grey Moist to wet				248							
		11	SS	9		○						
					247							
		12	SS	5	246	◑		7	46	37	10	
					245							
		13	SS	16	244							
243.5 14.9	SILT (ML) and sand, trace gravel (TILL) Dense Grey Moist	14 A	SS	35	243	○		8	42	39	11	
242.8 15.6	CLAYEY SILT (CL), trace sand, trace gravel (TILL) Hard Grey Moist	14 B										
242.1 16.3	CLAYEY SILT (CL) trace sand, trace gravel Hard Grey Moist				242							
		15	SS	118/0.18		○		0	1	74	25	
					241							
		16	SS	102/0.15	240							
					239							
238.5 19.9	Gravelly SAND (SW), trace fines Very dense Grey Moist											

Continued on Next Page

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

Ontario

PROJECT	19136074	RECORD OF BOREHOLE No. 400-3	Sheet 3 of 3	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886491.8; E 294131 NAD83 / MTM Zone 10 (LAT. 44.118343; LONG. -79.633298)	ORIGINATED BY MTI
DIST Central	HWY BBP- HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY MCK
DATUM	CGVD28 Surface Elevation:258.4 m	DATE	Dec 17, 2021 - Dec 20, 2021	CHECKED BY KJB

ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa) 20 40 60 80 100	WATER CONTENT (%) PL NMC LL 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	Gravelly SAND (SW), trace fines Very dense Grey Moist						238					
236.8			17	SS	100/0.15		237		○		27 66 (7)	
21.6	End of Borehole						236					
							235					
							234					
							233					
							232					
							231					
							230					
							229					

Notes:
1. Water level measured at a depth of 0.9 m (Elev. 257.5 m) prior to initiation of mud rotary drilling for borehole advancement below a depth of 3.0 m below ground surface.
2. Water level not recorded upon completion of drilling due to the introduction of drilling mud.
3. A monitoring well was installed approximately 12 m west of Borehole 400-3 (N 4886489.7; E 294117.2; Elev. 258.3 m).
4. Water level in piezometer measured at a depth of 0.9 m (Elev. 257.5 m) on February 28, 2023.

Legend: X Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined; W_p |----o----| W_l; NP Nonplastic

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

Ontario

RECORD OF BOREHOLE No. 400-4

Sheet 1 of 5 — METRIC

PROJECT 19136074

G.W.P. Assignment No 2019-E-0048 — LOCATION N 4886102.3; E 293976.8 NAD83 / MTM Zone 10 (LAT. 44.114835; LONG. -79.635217) — ORIGINATED BY MTI

DIST Central — HWY BBP - HWY 400 — BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary — COMPILED BY MCK

DATUM CGVD28 Surface Elevation:253.8 m — DATE Dec 14, 2021 - Dec 16, 2021 — CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL	REMARKS
0.0	CLAYEY SILT (CL), some sand, trace rootlets, trace organics Soft Brown Moist	1	SS	3							
252.9 / 0.9	SILTY SAND (SM) of slight plasticity, trace to some gravel Very loose to compact Brown Wet	2	SS	3	▽	253					
		3	SS	15		252					
	- 2.0 m: Large pieces of gravel encountered around auger										
251.6 / 2.2	CLAYEY SILT (CL), some sand, trace gravel Stiff to very stiff Brown to brownish grey Moist	4	SS	11		251					
		5	SS	28			0	11	49	40	
		6	SS	29		250					
		7	SS	21		249					
248.2 / 5.6	Sandy CLAYEY SILT-SILT (CL-ML), trace gravel, (TILL) Stiff Grey Moist					248					
		8	SS	12			7	35	43	15	
	- 6.5 to 6.7 m: Increased sand content					247					
246.6 / 7.2	CLAYEY SILT (CL), trace sand, trace gravel Firm to stiff Grey Moist to wet	9	SS	8		246	1	4	57	38	
						245					
	SILT (ML) and sand. trace gravel, (TILL) Loose to compact Grey Moist	10	SS	6							
244.0 / 9.8	- 9.9 to 10.5 m: Sample 11 recovery 50%					244					

Continued on Next Page

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

Ontario

wsp

PROJECT	19136074	RECORD OF BOREHOLE No. 400-4	Sheet 2 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION N 4886102.3; E 293976.8 NAD83 / MTM Zone 10 (LAT. 44.114835; LONG. -79.635217)	ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:253.8 m	DATE Dec 14, 2021 - Dec 16, 2021	CHECKED BY	KJB

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

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	GR SA SI CL
	SILT (ML) and sand. trace gravel, (TILL) Loose to compact Grey Moist	11	TO				
					243		
		12	SS	5		○	
					242		
		13	SS	6			
					241		
					240		
		14	SS	5		⊖	6 41 44 9
					239		
		15	SS	12			
					238		
					237		
		16	SS	9		○	
					236		
		17	SS	7			
					235		
					234		

Continued on Next Page

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

Ontario

PROJECT	19136074	RECORD OF BOREHOLE No. 400-4	Sheet 3 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION N 4886102.3; E 293976.8 NAD83 / MTM Zone 10 (LAT. 44.114835; LONG. -79.635217)	ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:253.8 m	DATE Dec 14, 2021 - Dec 16, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	SILT (ML) and sand. trace gravel, (TILL) Loose to compact Grey Moist					233				
		18	SS	10		232				
						231				
						230				
		19	SS	9		229				
						228				
						227				
	- 27.9 to 28.0 m: Gravelly seam encountered	20	SS	9		226				
						225				
						224				

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

Continued on Next Page

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

Ontario

RECORD OF BOREHOLE No. 400-4

METRIC

PROJECT	19136074				
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4886102.3; E 293976.8 NAD83 / MTM Zone 10 (LAT. 44.114835; LONG. -79.635217)	ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:253.8 m	DATE	Dec 14, 2021 - Dec 16, 2021	CHECKED BY	KJB

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

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	GR	SA	SI	CL
	SILT (ML) and sand. trace gravel, (TILL) Loose to compact Grey Moist	21 A	SS	13	223					
223.0 / 30.8	Sandy CLAYEY SILT-SILT (CL-ML), trace gravel (TILL) Stiff to hard Grey Moist	21 B				plotted	8	32	42	18
					222					
					221					
		22 A	SS	87	220					
220.0 / 33.8	SILTY SAND (SM), trace gravel Very dense Grey Moist to wet	22 B				plotted				
					219					
218.5 / 35.3	SILTY SAND (SM), trace gravel (TILL) Compact to very dense Grey Moist				218					
		23	SS	26	217	plotted	6	45	38	11
					216					
					215					
		24	SS	102/0.26	214					

Continued on Next Page

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

Ontario

WSP

PROJECT	19136074	RECORD OF BOREHOLE No. 400-4	Sheet 5 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION N 4886102.3; E 293976.8 NAD83 / MTM Zone 10 (LAT. 44.114835; LONG. -79.635217)	ORIGINATED BY	MTI
DIST	Central HWY BBP - HWY 400	BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:253.8 m	DATE Dec 14, 2021 - Dec 16, 2021	CHECKED BY	KJB

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	STRATA PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane	WATER CONTENT (%) PL W_p NMC W LL W_l; NP Nonplastic; 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
213.8 40.0	SILTY SAND (SM), trace gravel (TILL) Compact to very dense Grey Moist											
	End of Borehole											
							213					
							212					
							211					
							210					
							209					
							208					
							207					
							206					
							205					
							204					

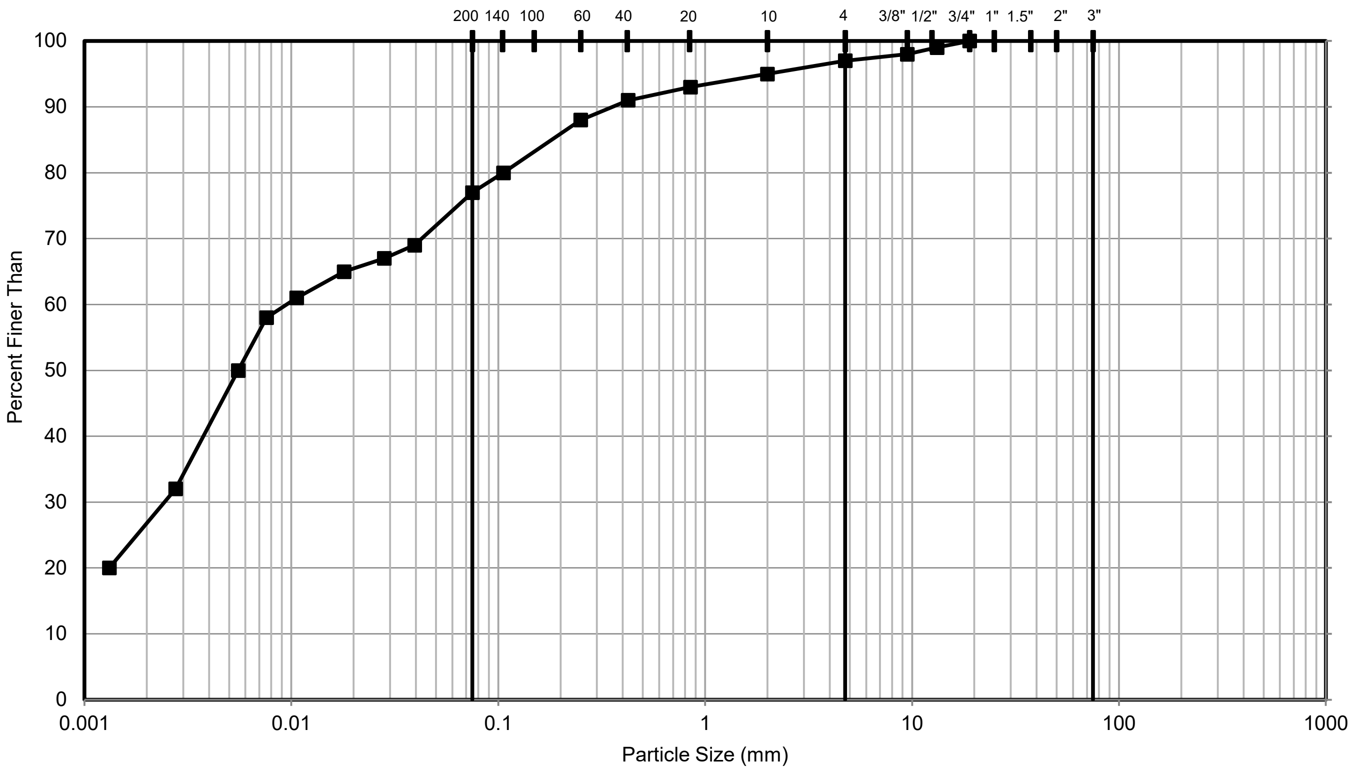
Notes:
1. Water level measured at a depth of 0.9 m (Elev. 252.9 m) prior to initiation of mud rotary drilling for borehole advancement below a depth of 3.0 m below ground surface.
2. Water level not recorded upon completion of drilling due to the introduction of drilling mud.

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

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-2	4	2.3 - 2.9	266.0 to 265.4

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

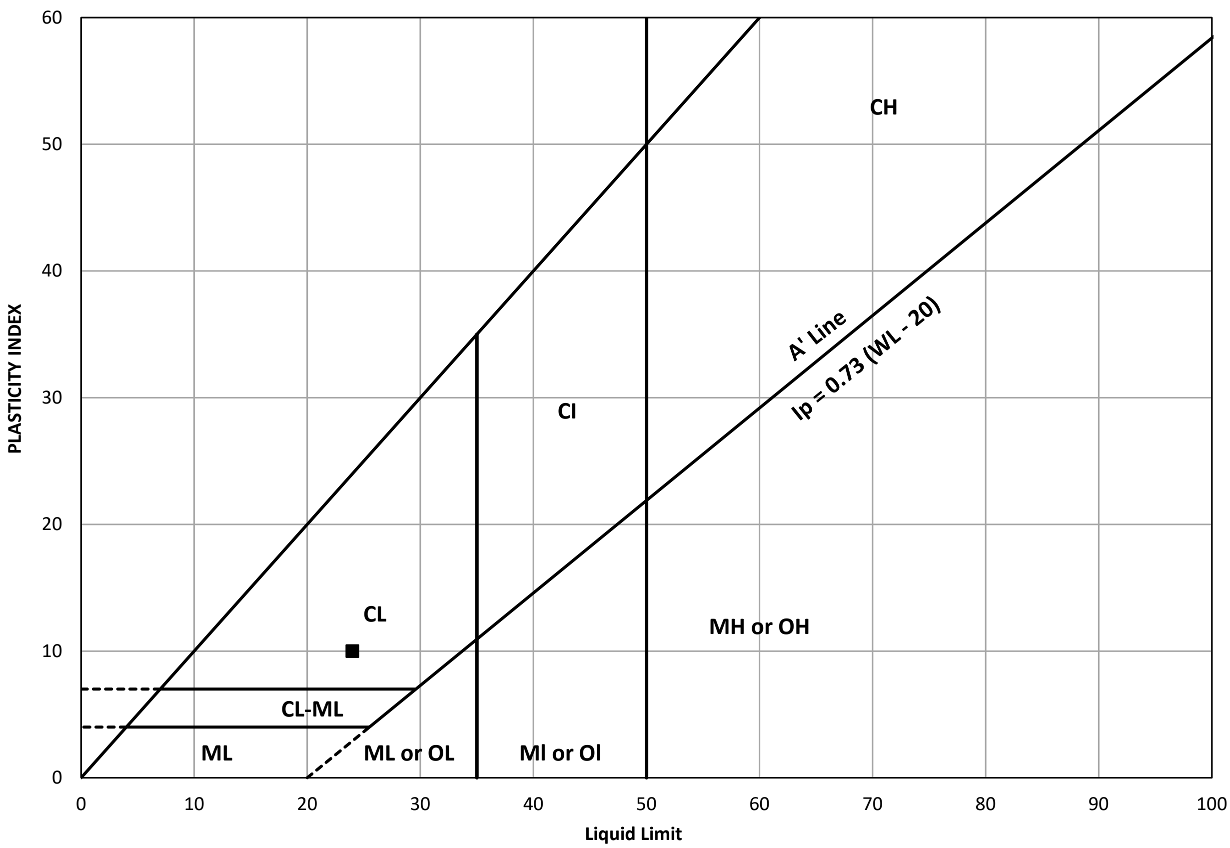
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

UPPER CLAYEY SILT (CL) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B1

PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	400-2	4	266.0 to 265.4	15	24	14	10

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Highway 400 Interchange

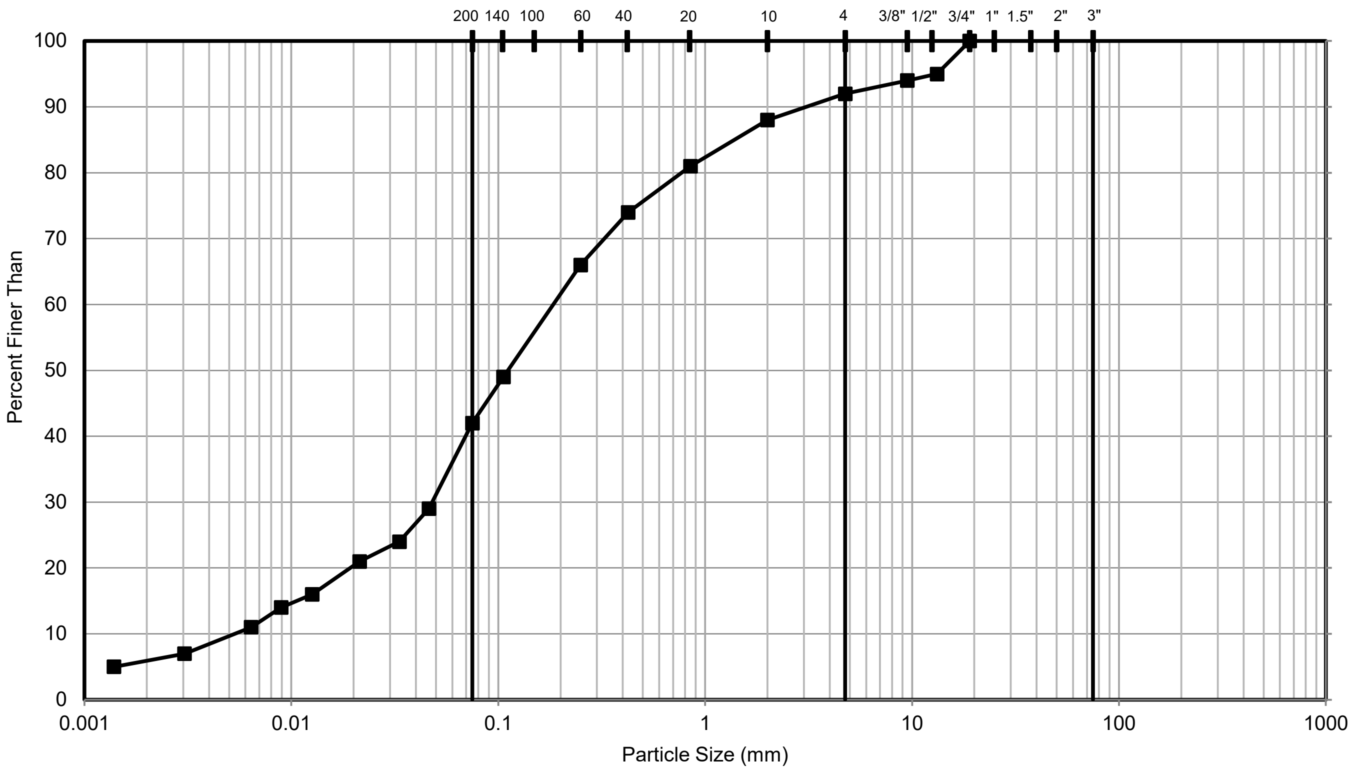
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE
UPPER CLAYEY SILT (CL) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B2

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-2	7	4.6 - 5.0	263.7 to 263.3

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

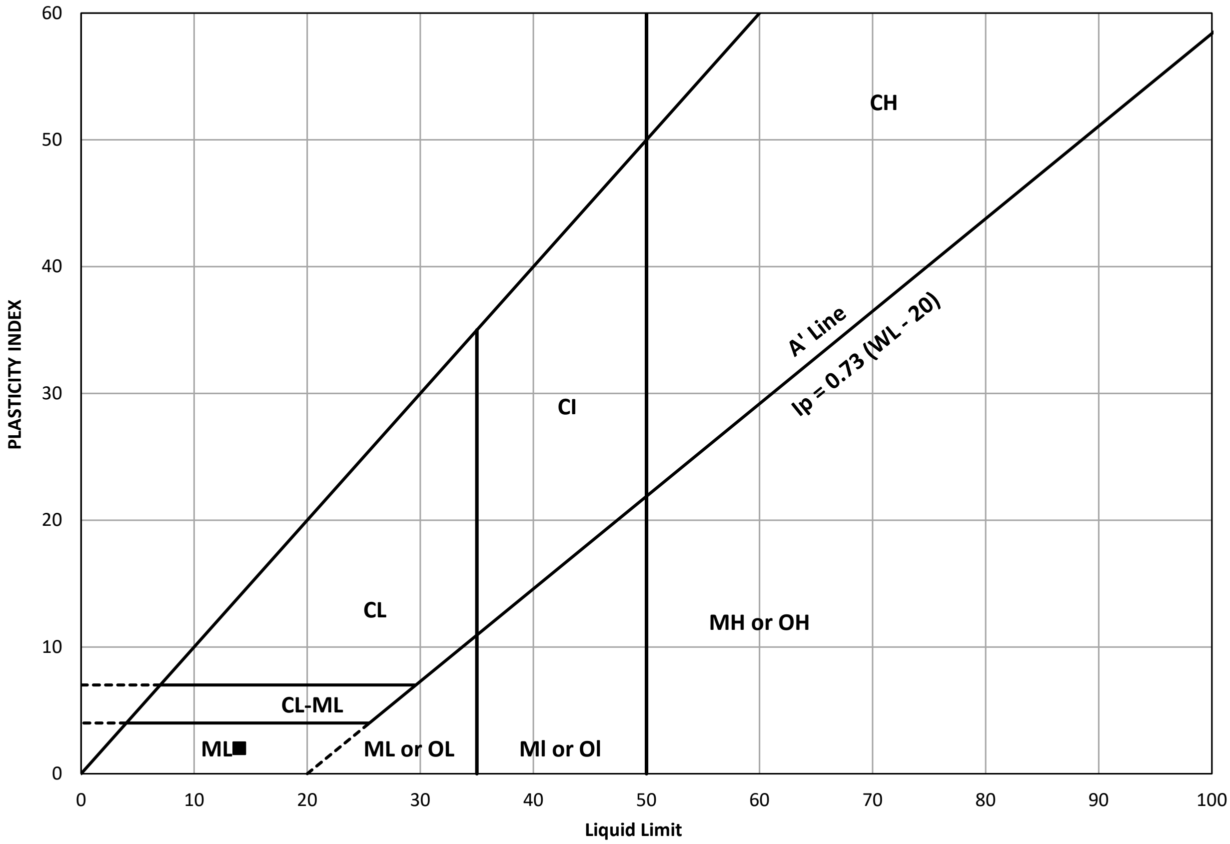
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

UPPER SILTY SAND (SM) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B3

PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	400-2	7	263.7 to 263.3	7.3	14	12	2

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Highway 400 Interchange

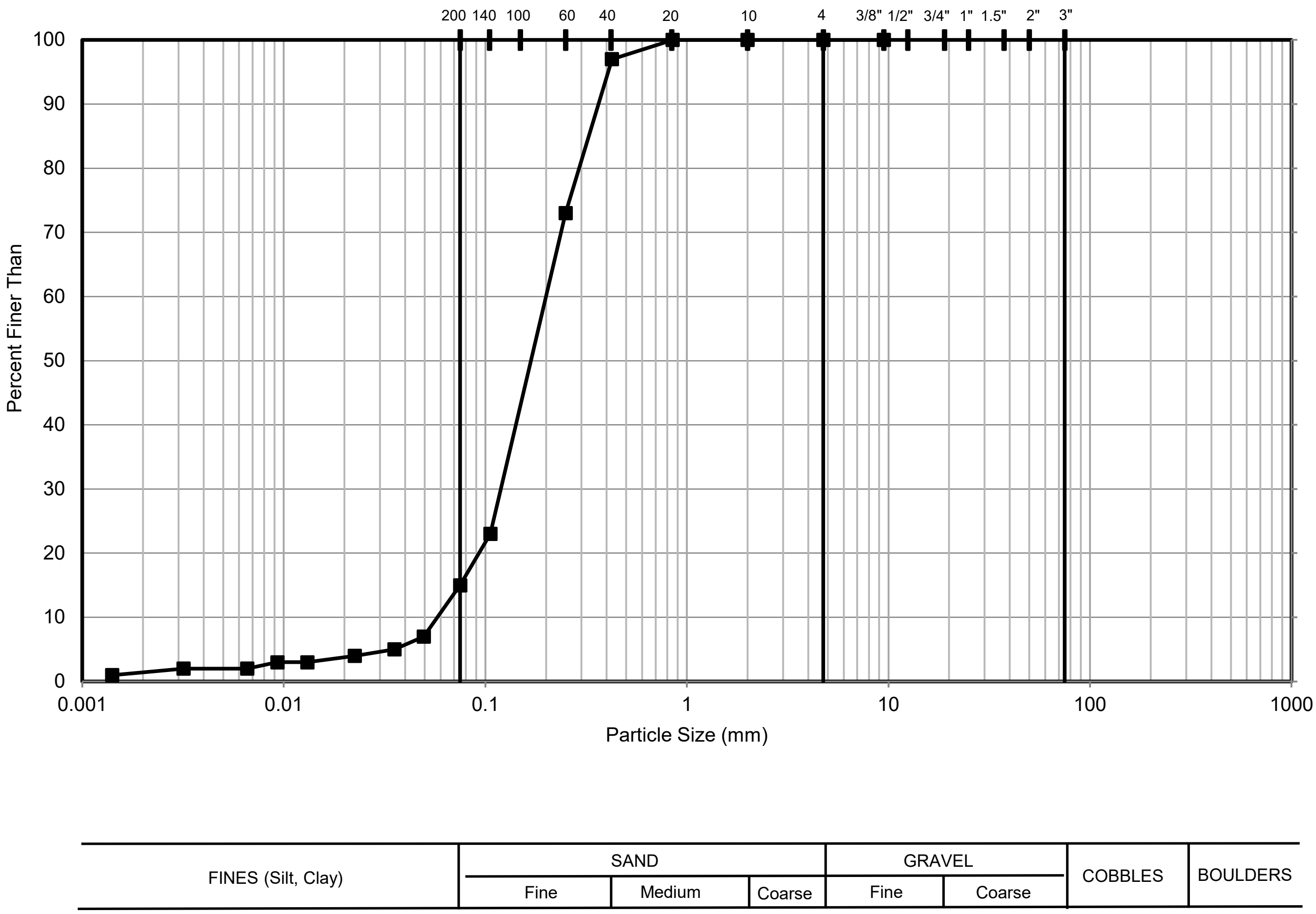
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE
UPPER SILTY SAND (SM) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B4

GRAIN SIZE DISTRIBUTION



Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-2	10	9.1 - 9.8	259.1 to 258.5

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

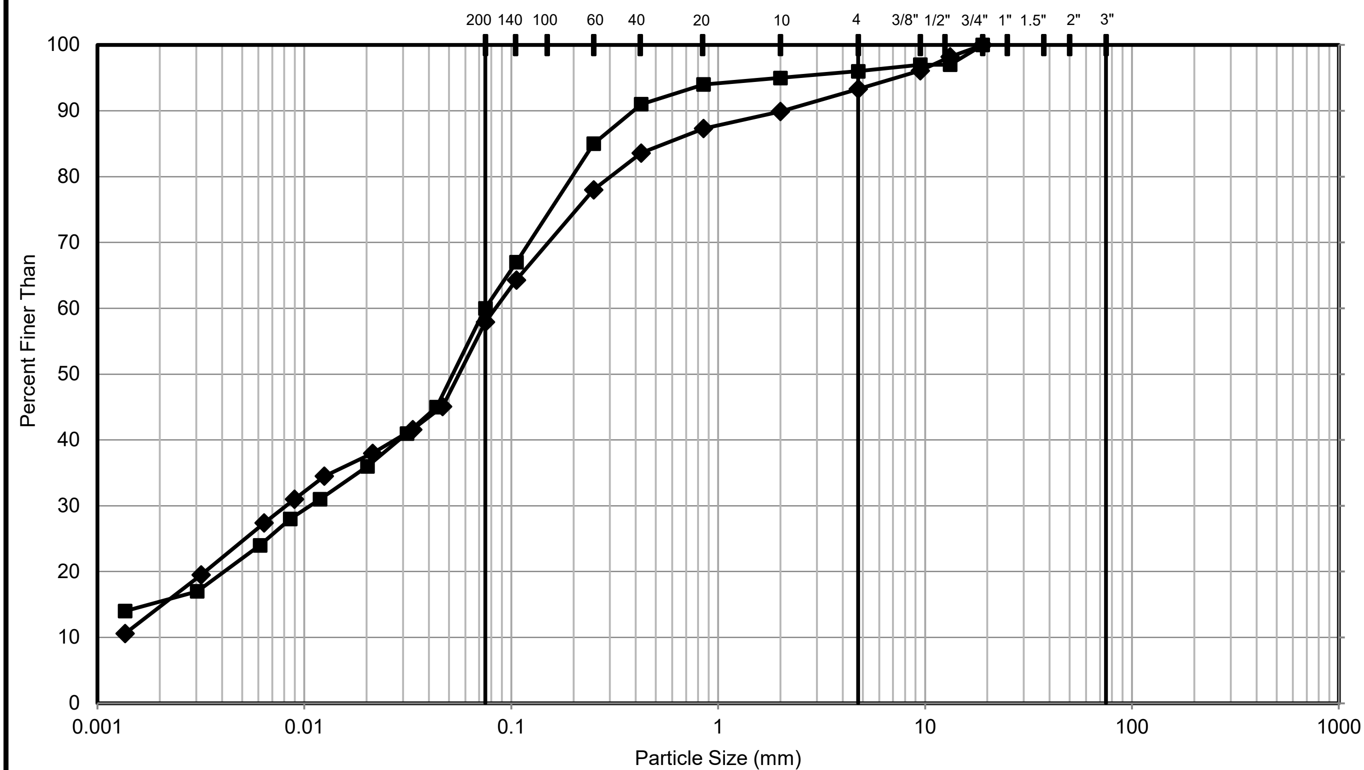
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

UPPER SILTY SAND (SM)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B5

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-2	14	15.2 - 15.9	253.1 to 252.4
◆	400-4	8	6.1 - 6.7	247.7 to 247.1

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

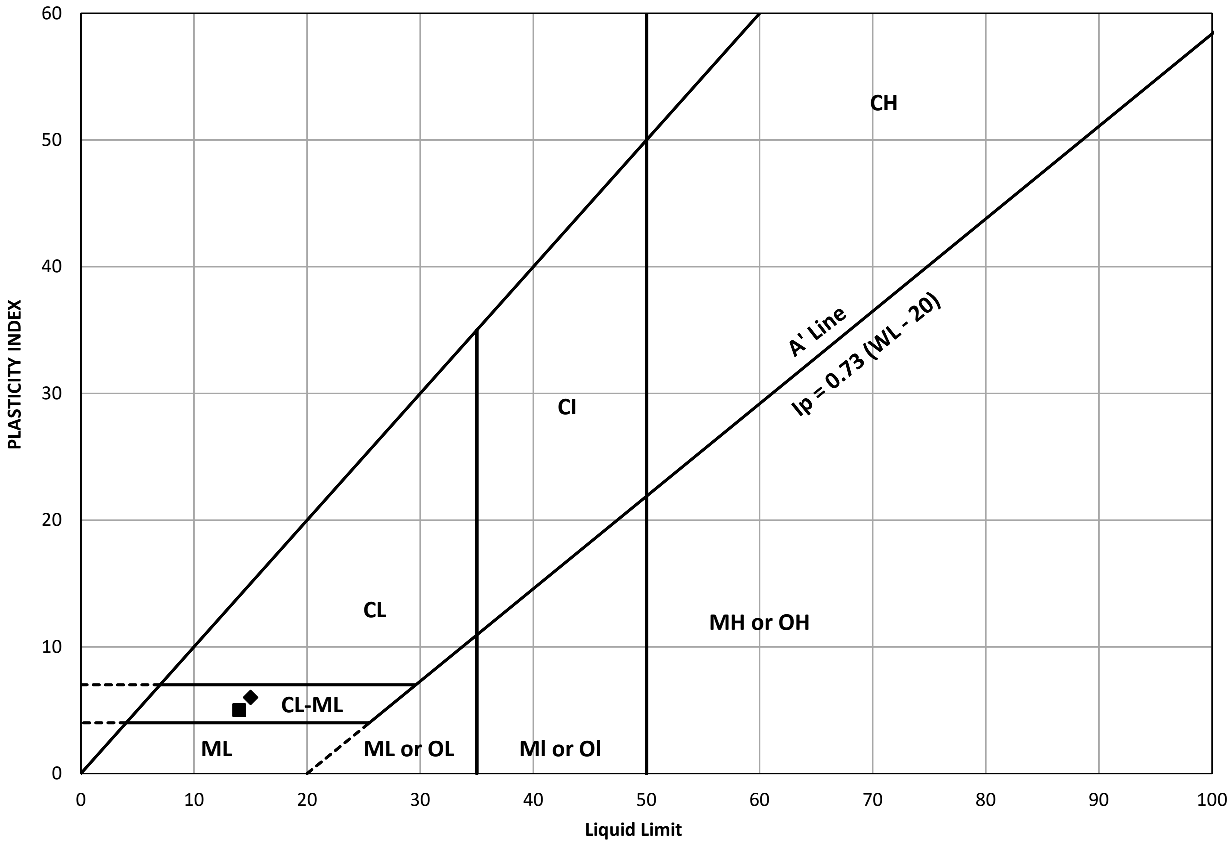
TITLE

UPPER CLAYEY SILT-SILT (CL-ML) Till

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B6

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Highway 400/RevB (Draft to MTO)/Appendix B - Lab Figures | FILE NAME: Figures - PSD.xlsm

PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	400-2	14	253.1 to 252.4	7.9	14	9	5
◆	400-4	8	247.7 to 247.1	11.1	15	9	6

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Highway 400 Interchange

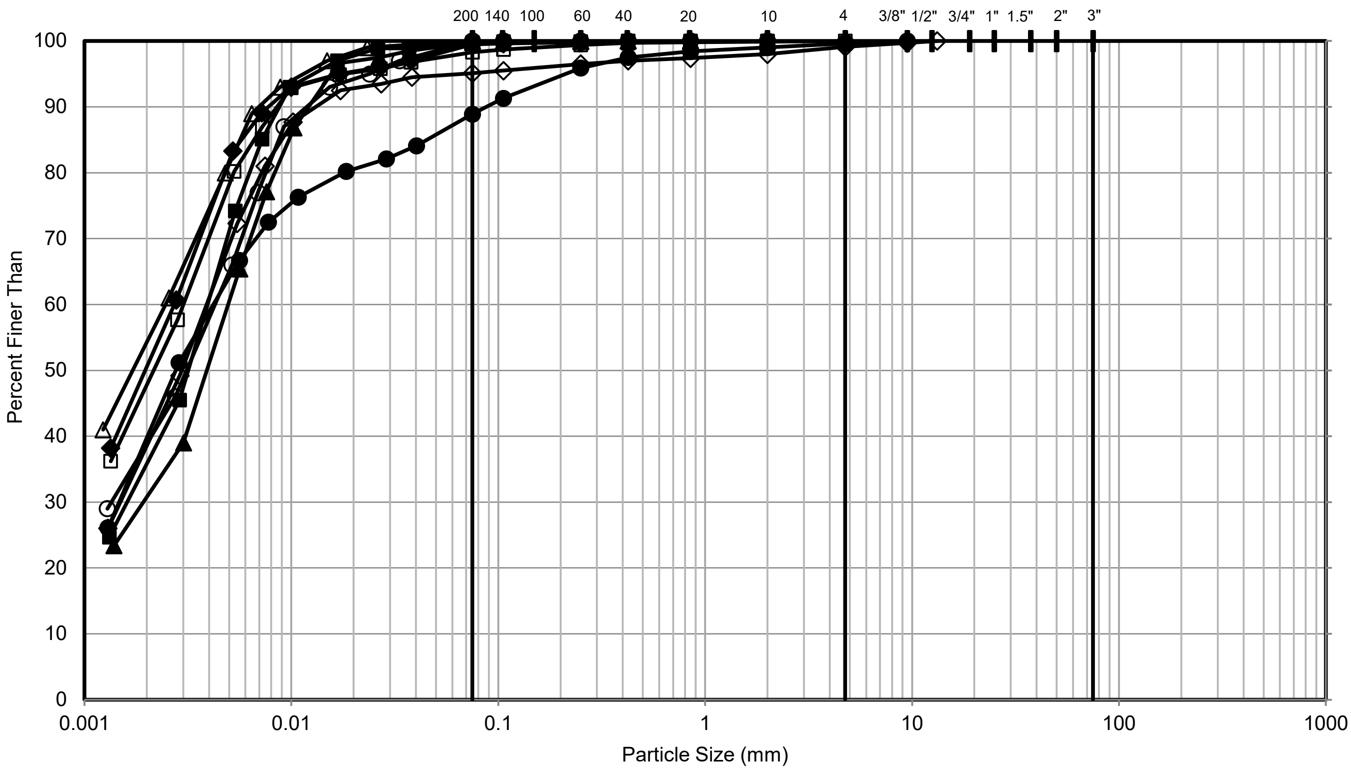
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE
UPPER CLAYEY SILT-SILT (CL-ML) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B7

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-3	5	3.1 - 3.7	255.4 to 254.7
◆	400-1	4	2.3 - 2.9	253.4 to 252.8
▲	400-1	6	3.8 - 4.4	251.9 to 251.3
●	400-4	5	3.1 - 3.7	250.8 to 250.1
□	400-1	10	7.6 - 8.2	248.1 to 247.5
◇	400-4	9	7.6 - 8.2	246.2 to 245.6
△	400-2	18	22.9 - 23.5	245.4 to 244.8
○	400-2	21	32.0 - 32.6	236.3 to 235.7

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

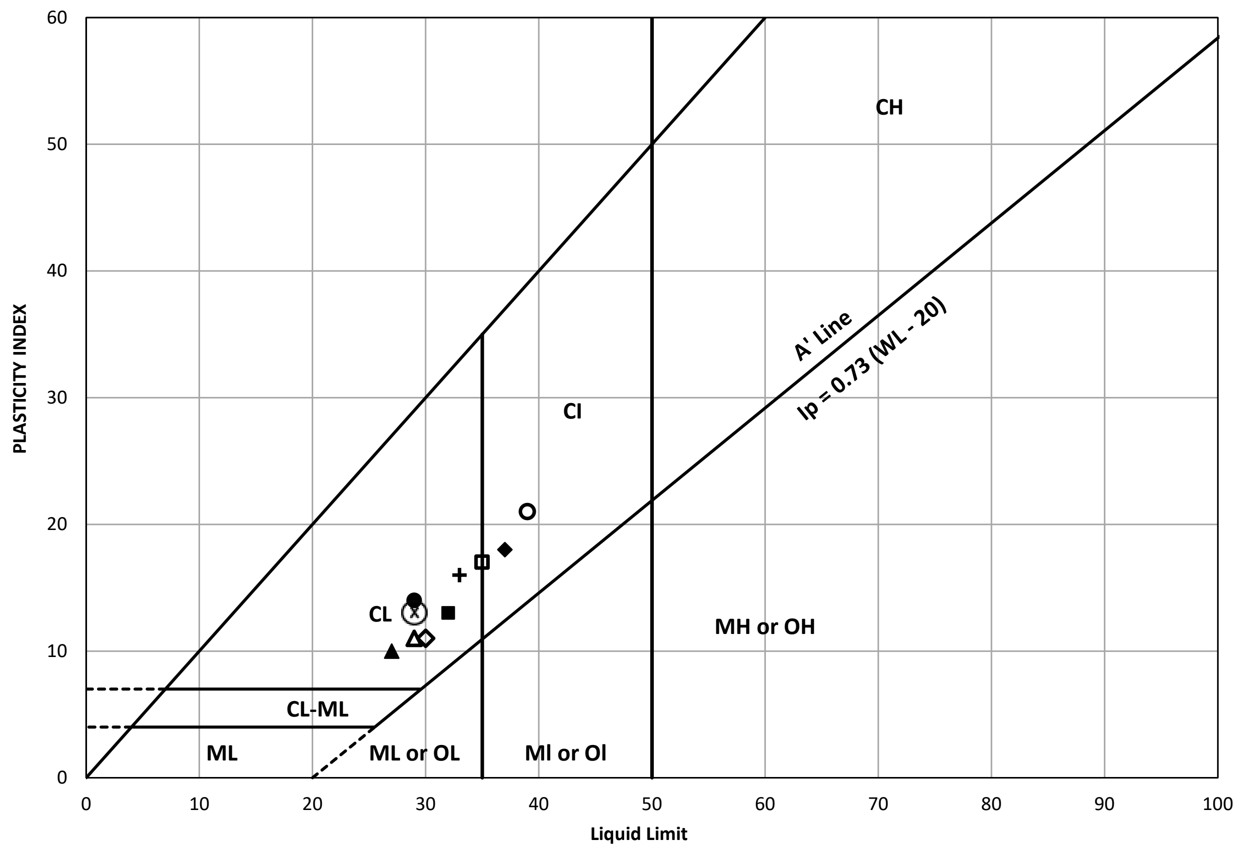
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

CLAYEY SILT (CL) to SILTY CLAY (CI)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B8

PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	400-3	5	255.4 to 254.7	22.4	32	19	13
◆	400-1	4	253.4 to 252.8	24.5	37	19	18
▲	400-1	6	251.9 to 251.3	23.2	27	17	10
●	400-4	5	250.8 to 250.1	15.9	29	15	14
+	400-1	10	248.1 to 247.5	26.5	33	17	16
⊗	400-4	9	246.2 to 245.6	28	29	16	13
□	400-2	18	245.4 to 244.8	21.5	35	18	17
◇	400-2	21	236.3 to 235.7	24.3	30	19	11
△	400-2	23	230.2 to 229.6	26.2	29	18	11
○	400-2	26	222.6 to 222.0	26.9	39	18	21

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Highway 400 Interchange

CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE
CLAYEY SILT (CL) to SILTY CLAY (CI)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B9

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE B10

SAMPLE IDENTIFICATION

Project Number	19136074	Sample Number	7
Borehole Number	400-1	Sample Depth, m	4.57-5.18

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	01/21/2022		
Date Completed	02/02/2022		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	19.79
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	15.76
Area, cm^2	31.52	Specific Gravity, measured	2.73
Volume, cm^3	80.12	Solids Height, cm	1.496
Water Content, %	25.56	Volume of Solids, cm^3	47.17
Wet Mass, g	161.69	Volume of Voids, cm^3	32.95
Dry Mass, g	128.77	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.542	0.699	2.542				
6.03	2.541	0.698	2.542	7	1.96E-01	5.22E-05	1.00E-06
10.78	2.541	0.698	2.541	38	3.60E-02	3.31E-05	1.17E-07
20.53	2.540	0.698	2.541	73	1.87E-02	1.61E-05	2.96E-08
40.11	2.536	0.695	2.538	194	7.04E-03	9.04E-05	6.24E-08
10.72	2.537	0.695	2.536				
20.76	2.537	0.695	2.537	163	8.37E-03	1.57E-05	1.29E-08
40.05	2.536	0.695	2.536	187	7.29E-03	1.63E-05	1.17E-08
79.11	2.527	0.689	2.531	135	1.01E-02	8.96E-05	8.84E-08
156.54	2.512	0.678	2.519	154	8.74E-03	7.77E-05	6.66E-08
311.82	2.490	0.664	2.501	144	9.21E-03	5.47E-05	4.94E-08
622.85	2.445	0.634	2.468	135	9.56E-03	5.69E-05	5.33E-08
1244.52	2.345	0.567	2.395	331	3.67E-03	6.35E-05	2.28E-08
2487.77	2.257	0.508	2.301	277	4.05E-03	2.78E-05	1.10E-08
622.85	2.267	0.515	2.262				
156.50	2.290	0.530	2.278				
40.24	2.315	0.547	2.302				
10.72	2.349	0.570	2.332				

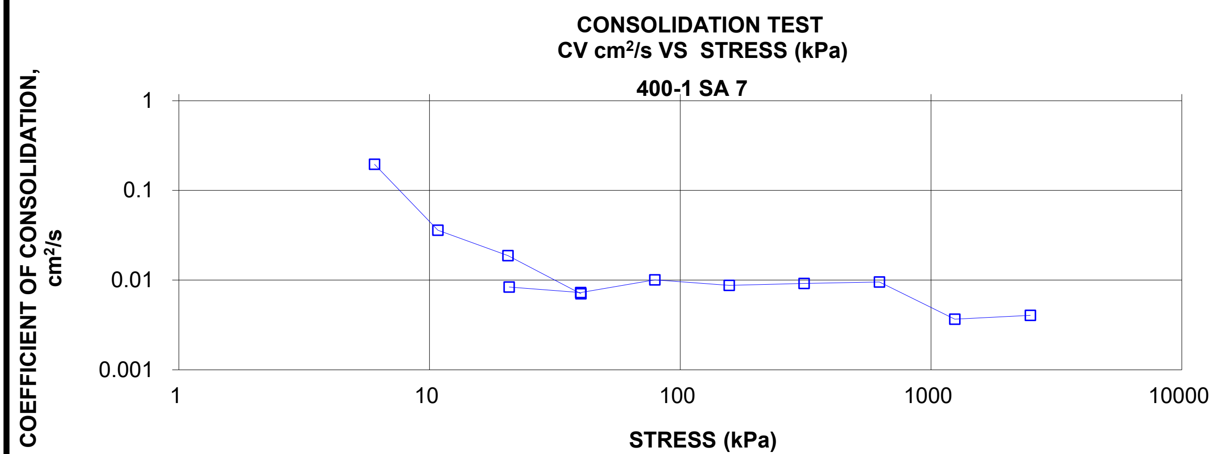
Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen taken 16.5-26.5cm from top of the tube.

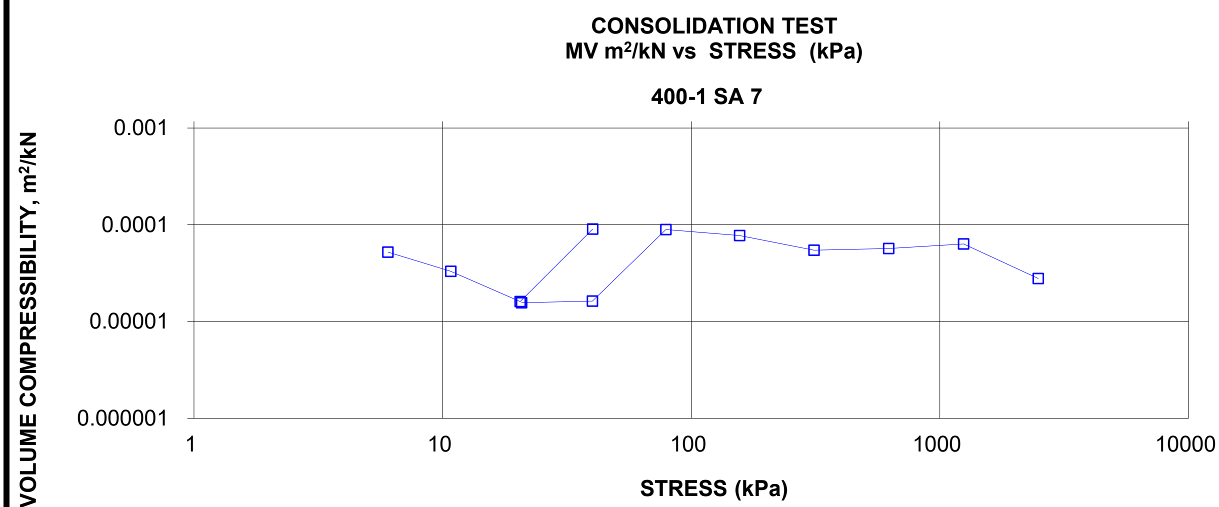
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

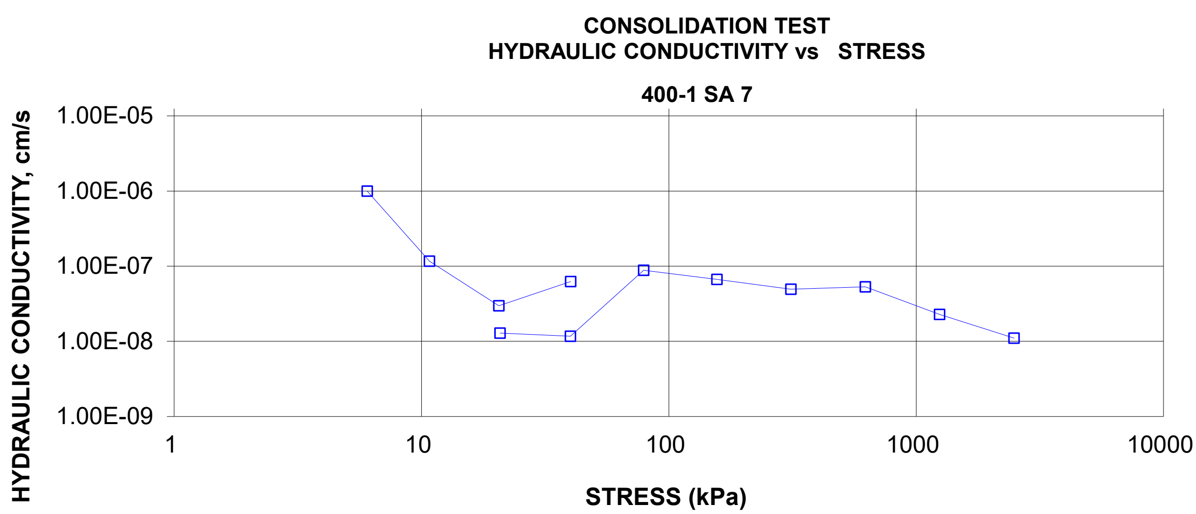
Sample Height, cm	2.35	Unit Weight, kN/m^3	20.76
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	17.06
Area, cm^2	31.52	Specific Gravity, measured	2.73
Volume, cm^3	74.03	Solids Height, cm	1.496
Water Content, %	21.70	Volume of Solids, cm ^{3}	47.17
Wet Mass, g	156.71	Volume of Voids, cm ^{3}	26.87
Dry Mass, g	128.77		

Prepared By: LH **Golder Associates** Checked By: MM

CONSOLIDATION TEST SUMMARY	FIGURE B10







Project No. 19136074

Prepared By: LH

Golder Associates

Checked By: MM

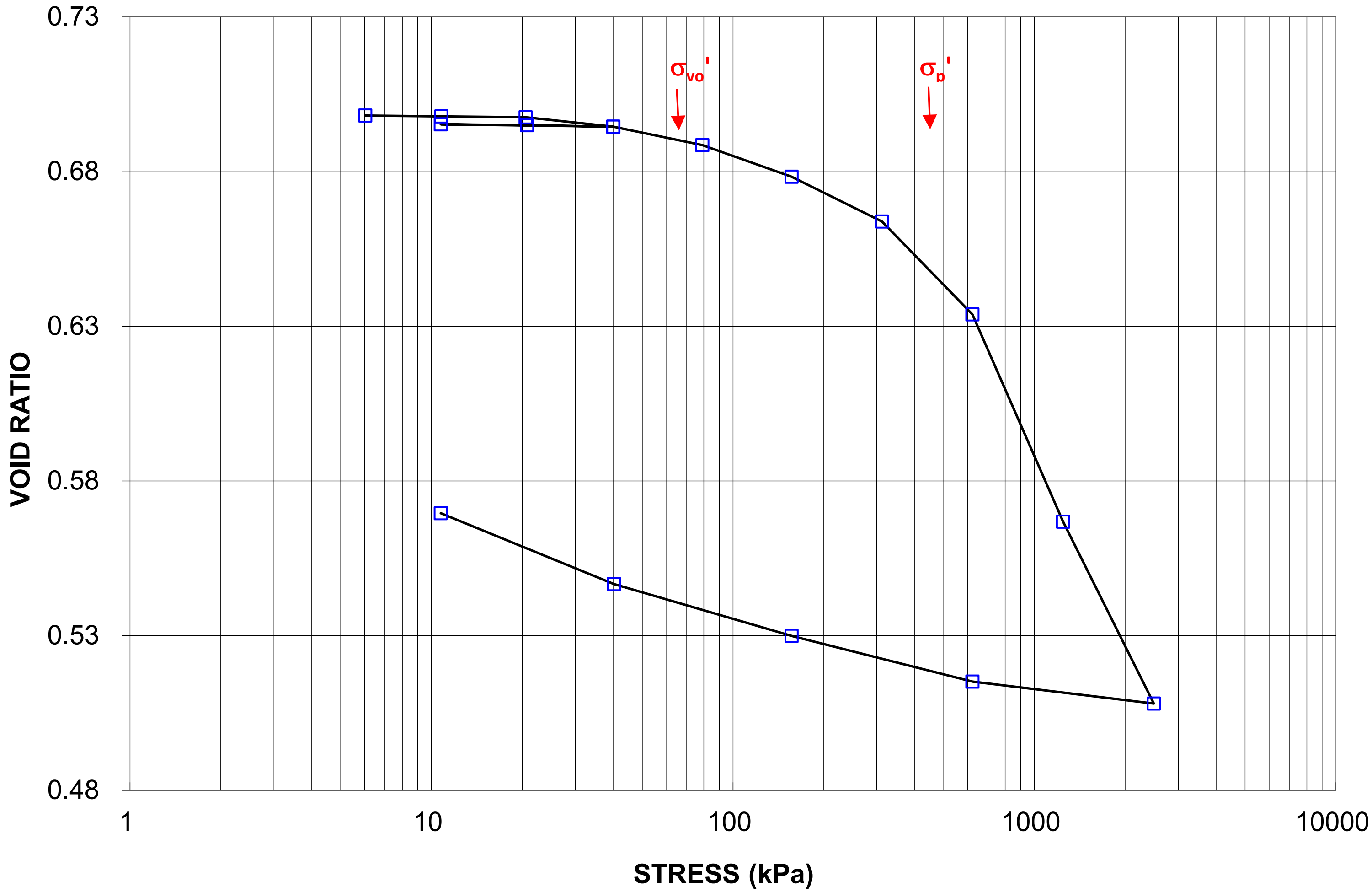
Project No. 19136074

Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO vs STRESS
400-1 SA 7



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B10

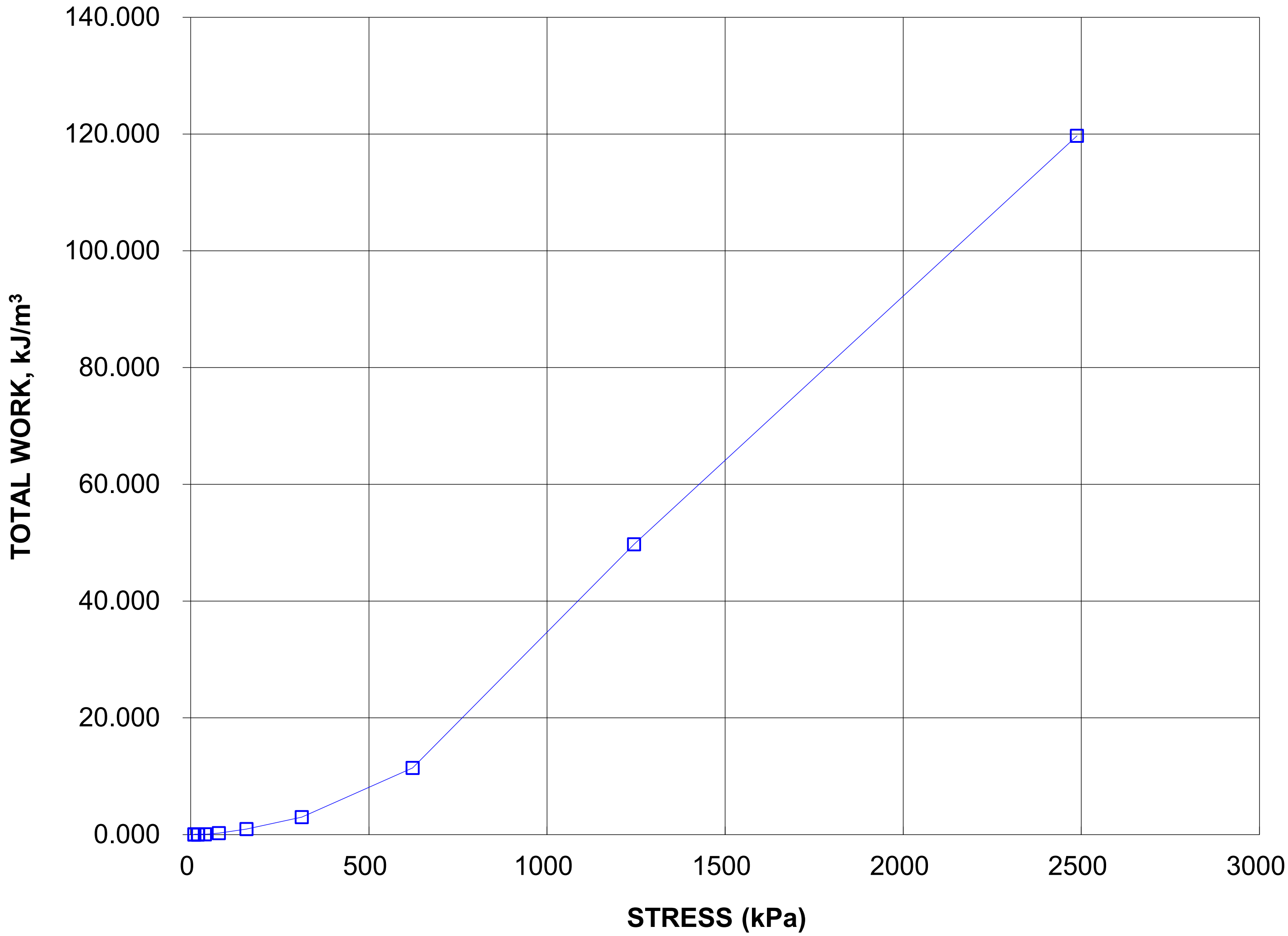
Project No. 19136074

Prepared By: LH

Checked By: MM

Golder Associates

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
400-1 SA 7



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B10

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE B11

SAMPLE IDENTIFICATION

Project Number	19136074	Sample Number	9
Borehole Number	400-1	Sample Depth, m	6.86-7.47

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	01/21/2022		
Date Completed	02/02/2022		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	21.57
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	18.62
Area, cm^2	31.52	Specific Gravity, measured	2.72
Volume, cm^3	80.19	Solids Height, cm	1.776
Water Content, %	15.86	Volume of Solids, cm^3	55.97
Wet Mass, g	176.37	Volume of Voids, cm^3	24.22
Dry Mass, g	152.23	Degree of Saturation, %	99.7

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.544	0.433	2.544				
5.83	2.544	0.433	2.544	9	1.52E-01	2.43E-05	3.63E-07
10.76	2.543	0.432	2.544	194	7.07E-03	1.91E-05	1.33E-08
20.76	2.541	0.431	2.542	240	5.71E-03	8.65E-05	4.84E-08
40.24	2.533	0.426	2.537	265	5.15E-03	1.72E-04	8.65E-08
10.73	2.533	0.426	2.533				
20.68	2.533	0.426	2.533	154	8.83E-03	1.22E-06	1.05E-09
40.24	2.531	0.425	2.532	406	3.35E-03	3.35E-05	1.10E-08
78.87	2.515	0.417	2.523	558	2.42E-03	1.61E-04	3.81E-08
156.50	2.492	0.403	2.504	694	1.91E-03	1.18E-04	2.22E-08
311.85	2.461	0.386	2.477	217	5.99E-03	7.74E-05	4.55E-08
621.79	2.421	0.364	2.441	290	4.36E-03	5.06E-05	2.16E-08
1242.67	2.372	0.336	2.397	375	3.25E-03	3.11E-05	9.89E-09
2485.01	2.328	0.311	2.350	390	3.00E-03	1.42E-05	4.17E-09
621.79	2.331	0.313	2.329				
156.54	2.337	0.316	2.334				
40.05	2.349	0.323	2.343				
10.84	2.366	0.332	2.357				

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 27-36cm from top of the tube.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

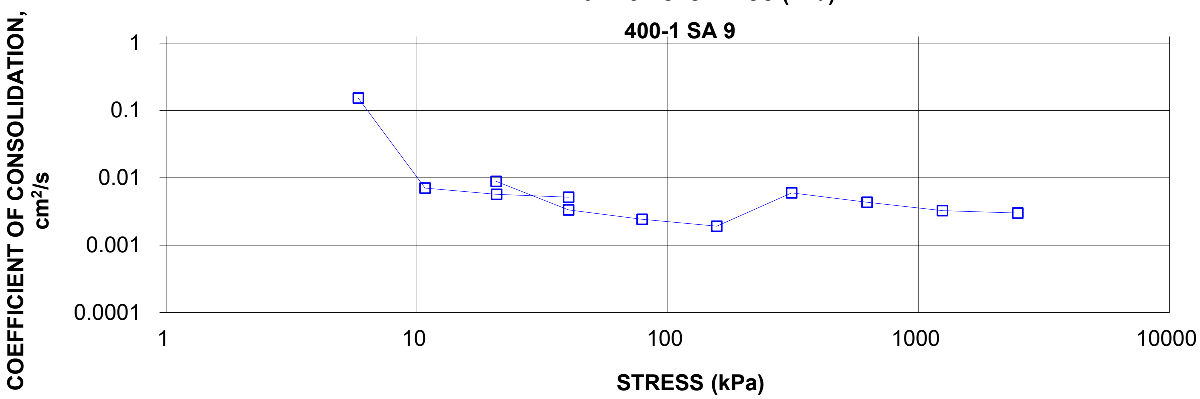
Sample Height, cm	2.37	Unit Weight, kN/m^3	22.58
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	20.02
Area, cm^2	31.52	Specific Gravity, measured	2.72
Volume, cm^3	74.56	Solids Height, cm	1.776
Water Content, %	12.78	Volume of Solids, cm^3	55.97
Wet Mass, g	171.68	Volume of Voids, cm^3	18.59
Dry Mass, g	152.23		

Prepared By: LH **Golder Associates** Checked By: MM

CONSOLIDATION TEST SUMMARY	FIGURE B11

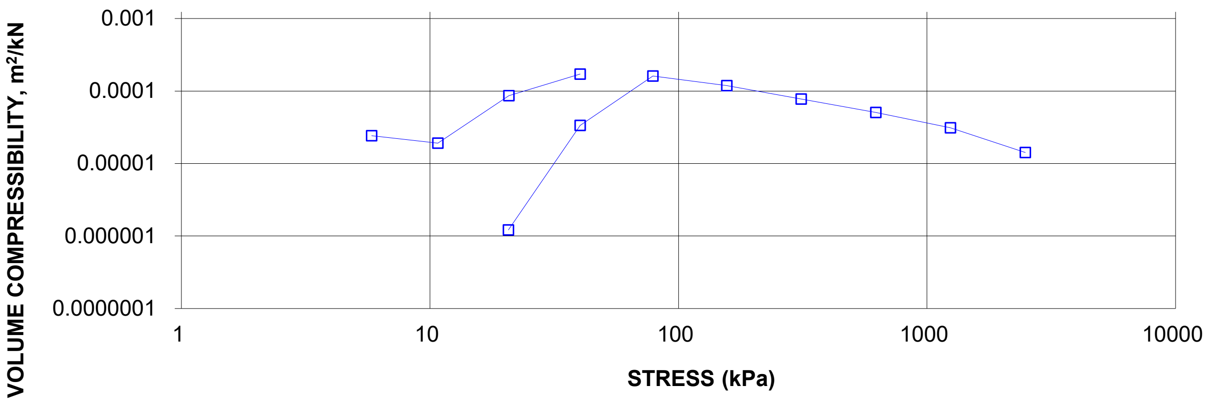
CONSOLIDATION TEST
CV cm^2/s VS STRESS (kPa)

400-1 SA 9



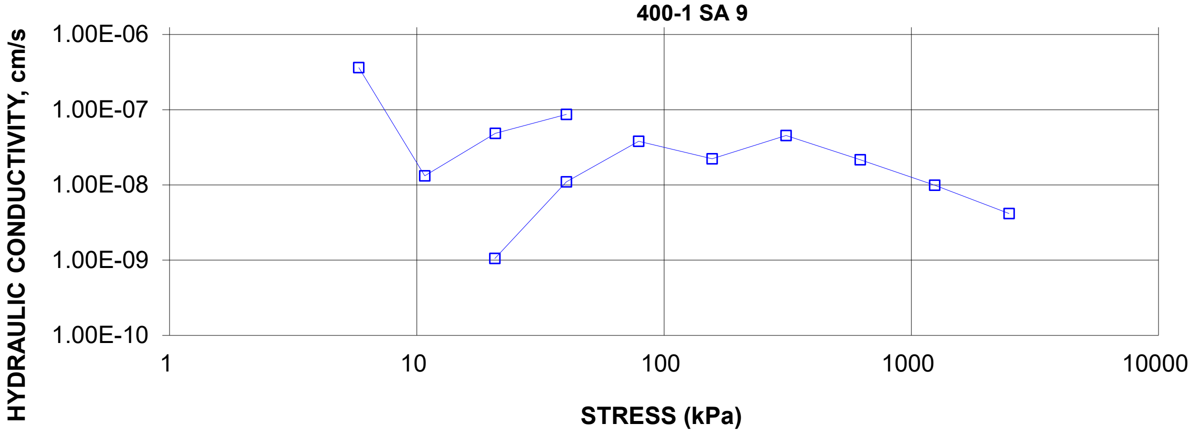
CONSOLIDATION TEST
MV m^2/kN vs STRESS (kPa)

400-1 SA 9



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS

400-1 SA 9



Project No. 19136074

Prepared By: LH

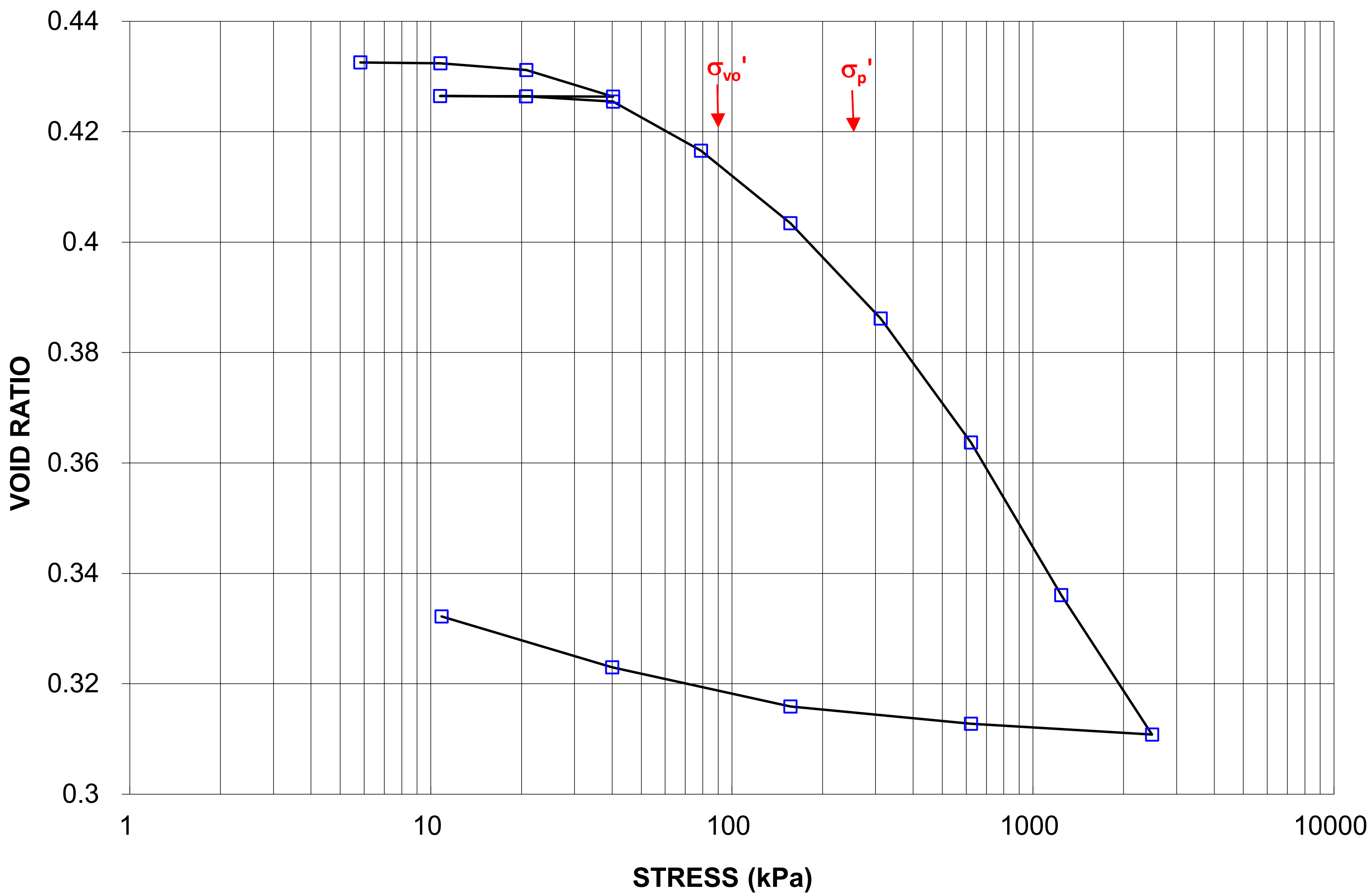
Golder Associates

Checked By: MM

Project No. 19136074

Prepared By: LH

CONSOLIDATION TEST
VOID RATIO vs STRESS
400-1 SA 9



Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B11

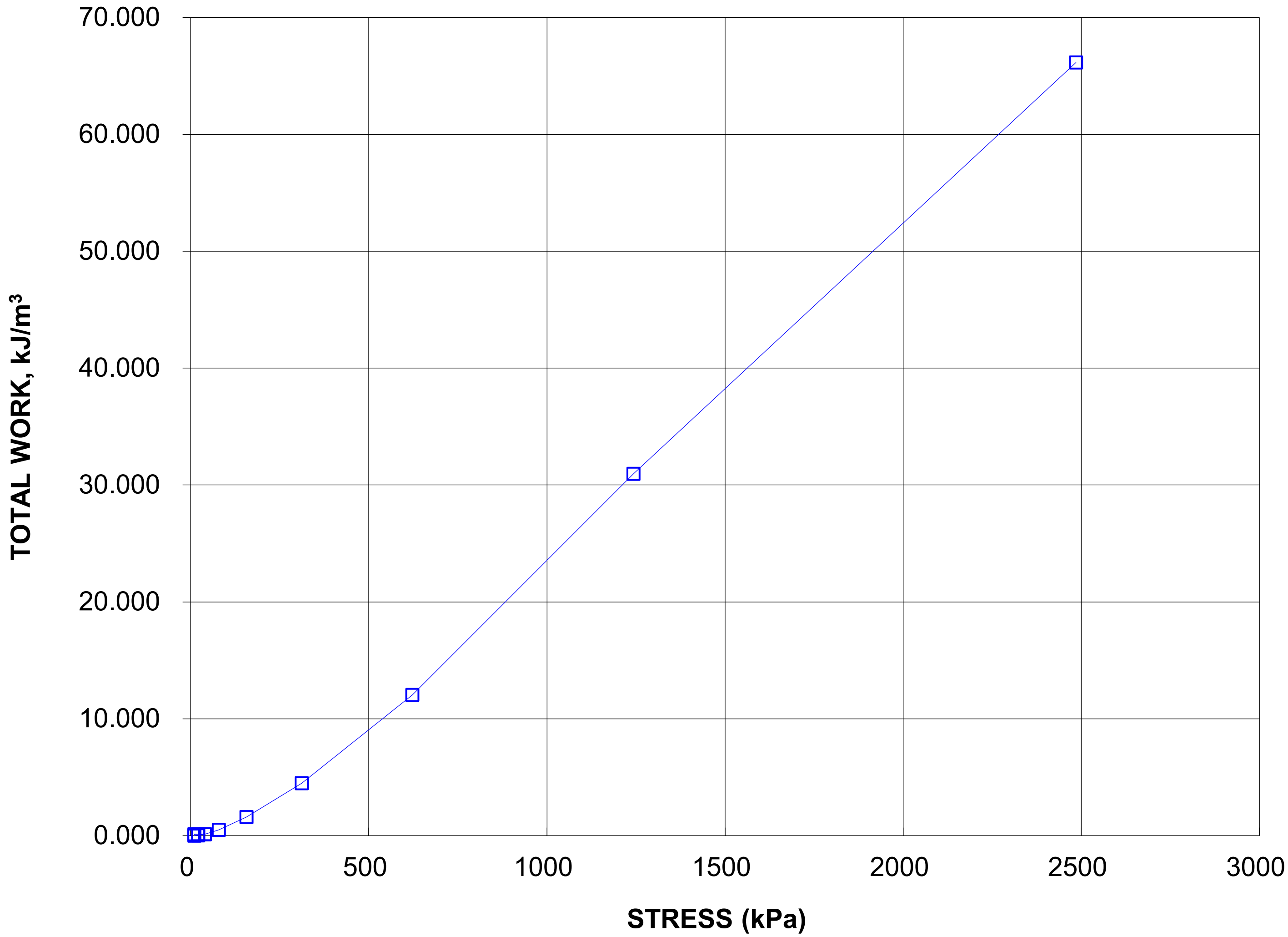
Project No. 19136074

Prepared By: LH

Golder Associates

Checked By: MM

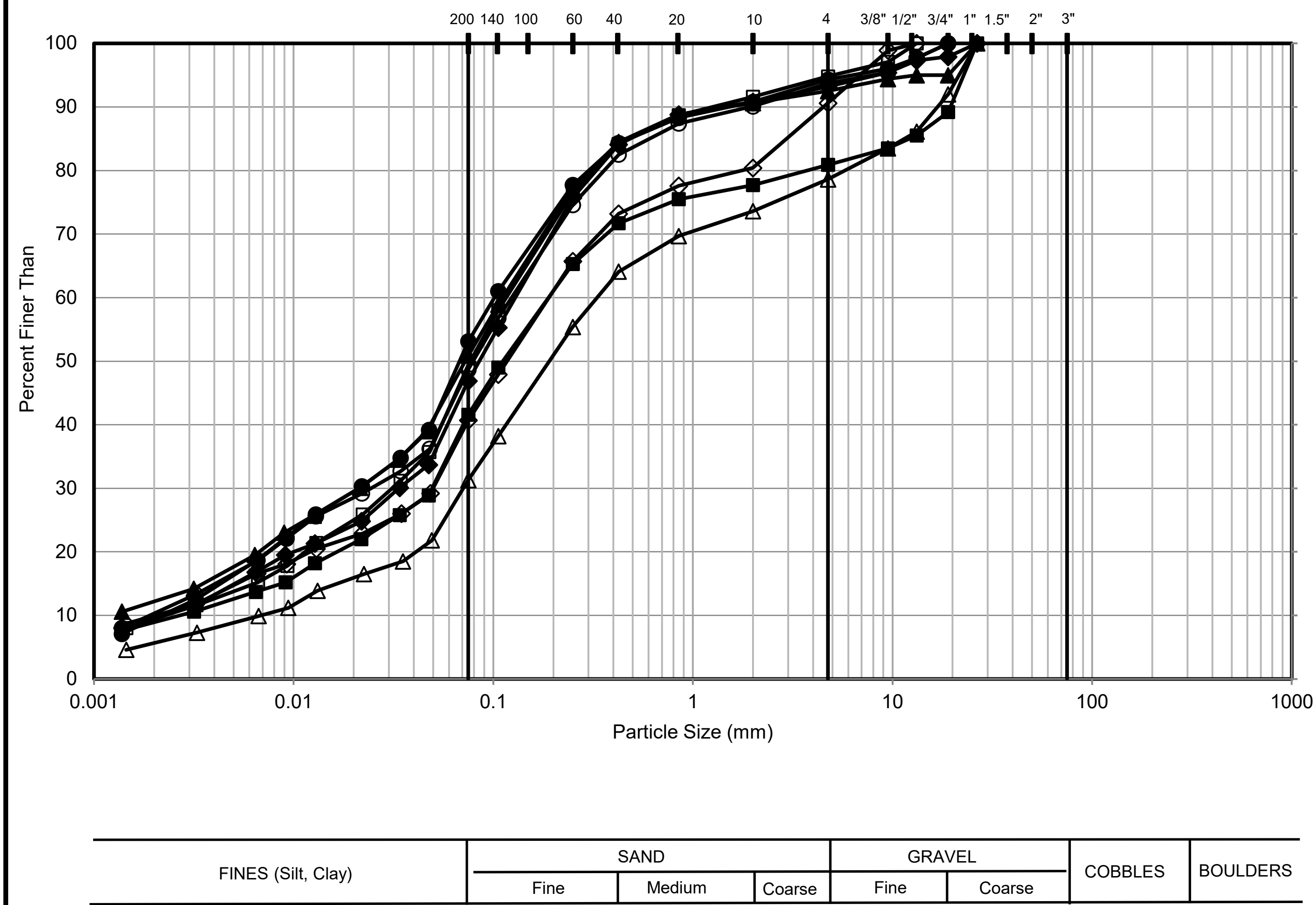
CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
400-1 SA 9



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B11

GRAIN SIZE DISTRIBUTION



Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-3	10	9.1 - 9.8	249.3 to 248.7
◆	400-3	12	12.2 - 12.8	246.2 to 245.6
▲	400-3	14A	15.2 - 15.6	243.2 to 242.8
●	400-4	14	13.7 - 14.3	240.1 to 239.5
□	400-1	16	16.8 - 17.4	238.9 to 238.3
◇	400-1	20	24.4 - 25.0	231.3 to 230.7
△	400-1	22	30.5 - 30.9	225.2 to 224.8
○	400-4	23	36.6 - 37.2	217.2 to 216.6

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

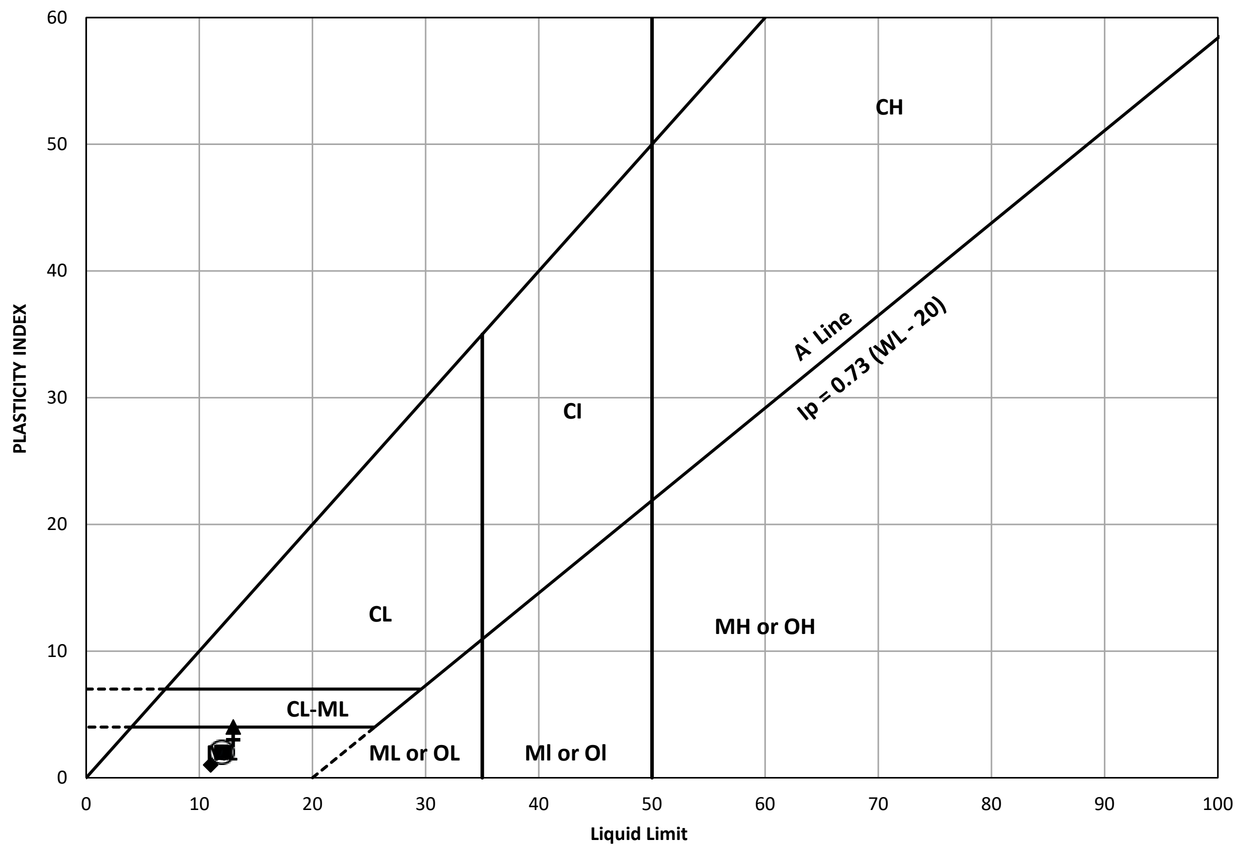
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

SILT AND SAND (ML) TO SILTY SAND TO GRAVELLY SILTY SAND (SM) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B12

PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	400-3	8	252.3 to 251.7	12.8	12	10	2
◆	400-3	12	246.2 to 245.6	10	11	10	1
▲	400-4	14	240.1 to 239.5	9.1	13	9	4
●	400-1	16	238.9 to 238.3	9.7	12	10	2
+	400-1	18	235.9 to 235.3	10.3	13	10	3
⊗	400-4	18	232.5 to 231.9	10.3	12	10	2
□	400-1	20	231.3 to 230.7	11.2	12	10	2

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Highway 400 Interchange

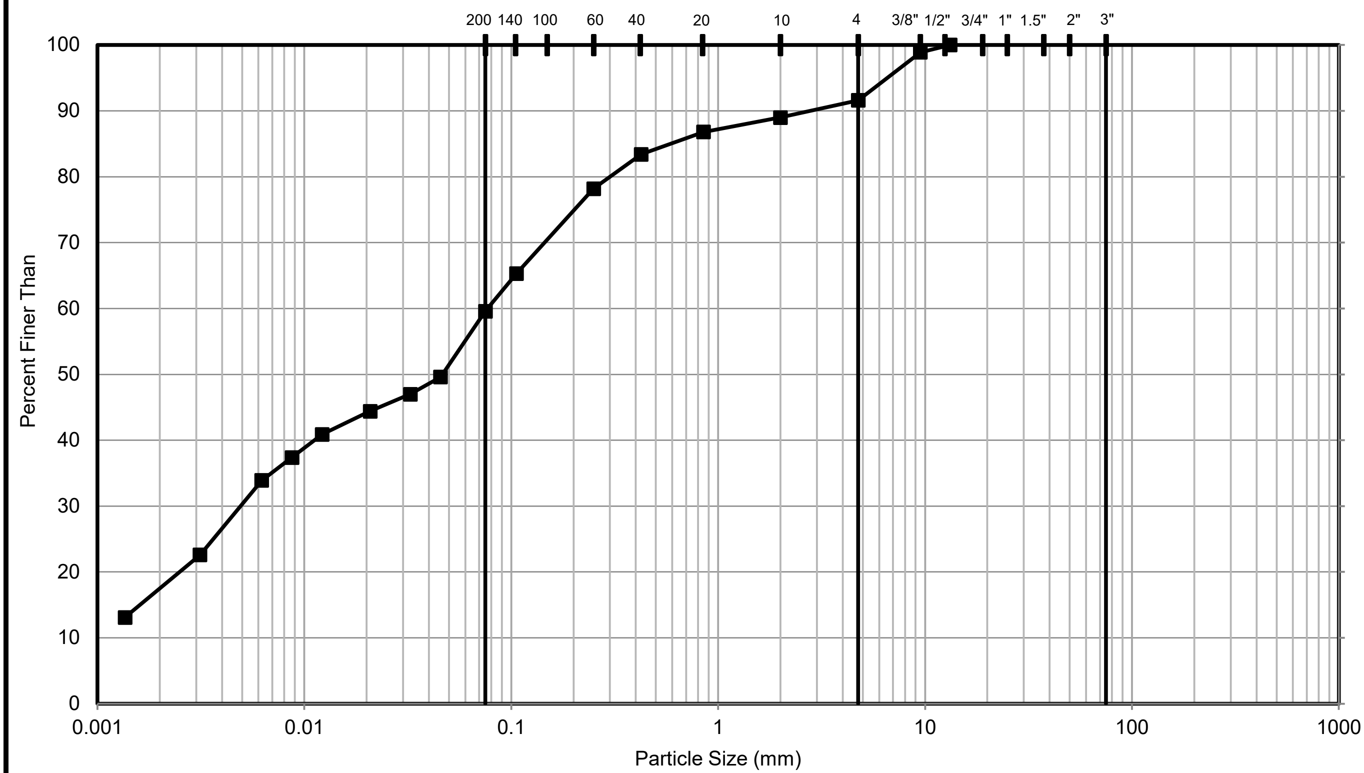
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE
SILT AND SAND (ML) TO SILTY SAND (SM) TILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B13

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-4	21B	30.8 - 31.1	223.0 to 222.7

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

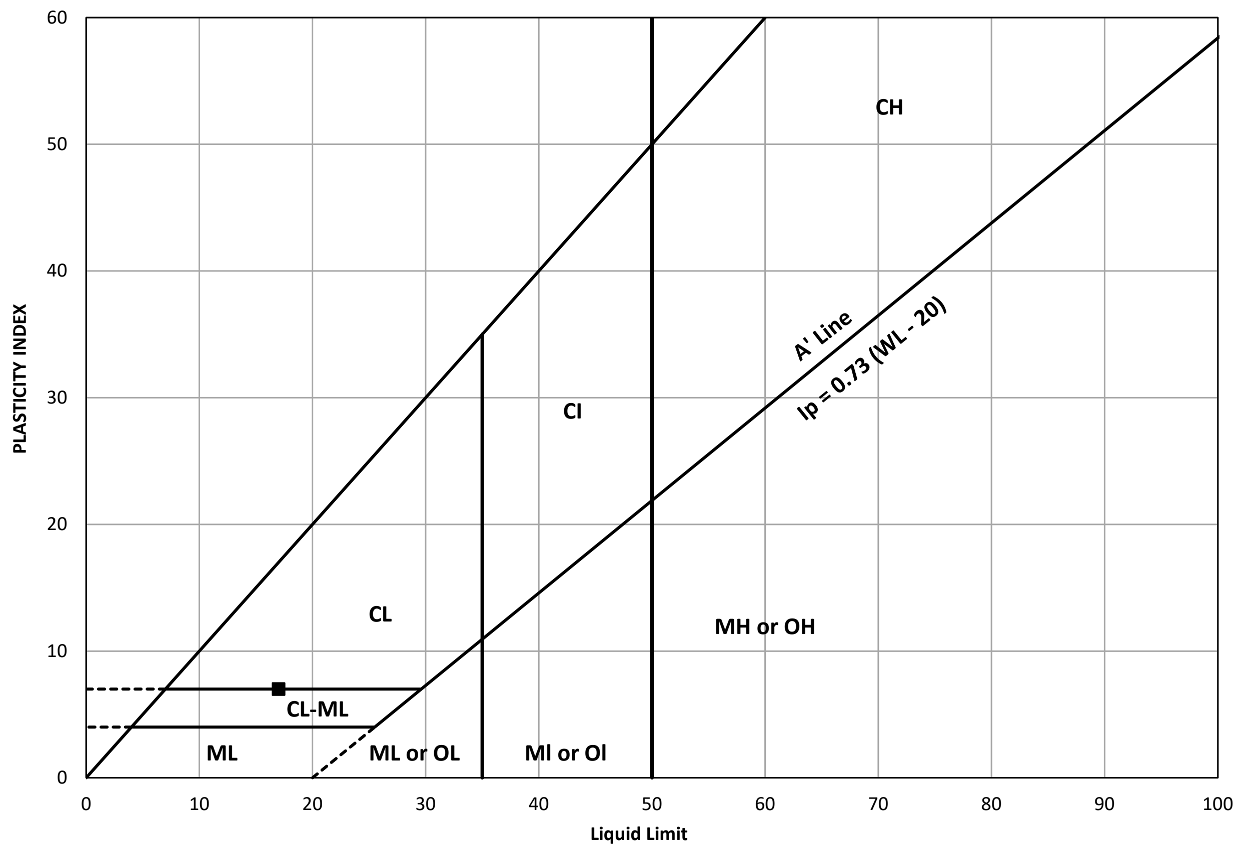
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

SANDY CLAYEY SILT-SILT (CL-ML) TILL - INTERLAYER

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B14

PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	400-4	21B	223.0 to 222.7	8.5	17	10	7

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Highway 400 Interchange

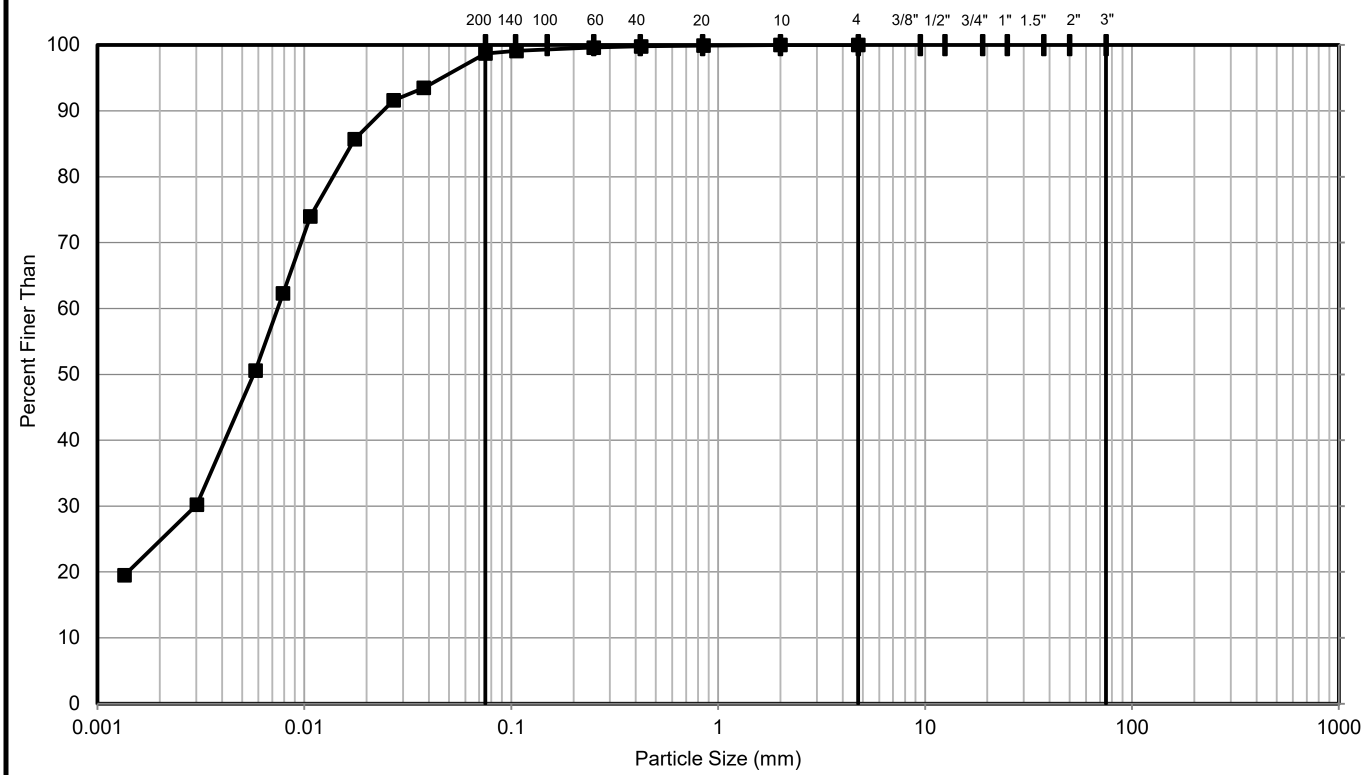
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE
SANDY CLAYEY SILT-SILT (CL-ML) TILL - INTERLAYER

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B15

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-3	15	16.8 - 17.1	241.6 to 241.3

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

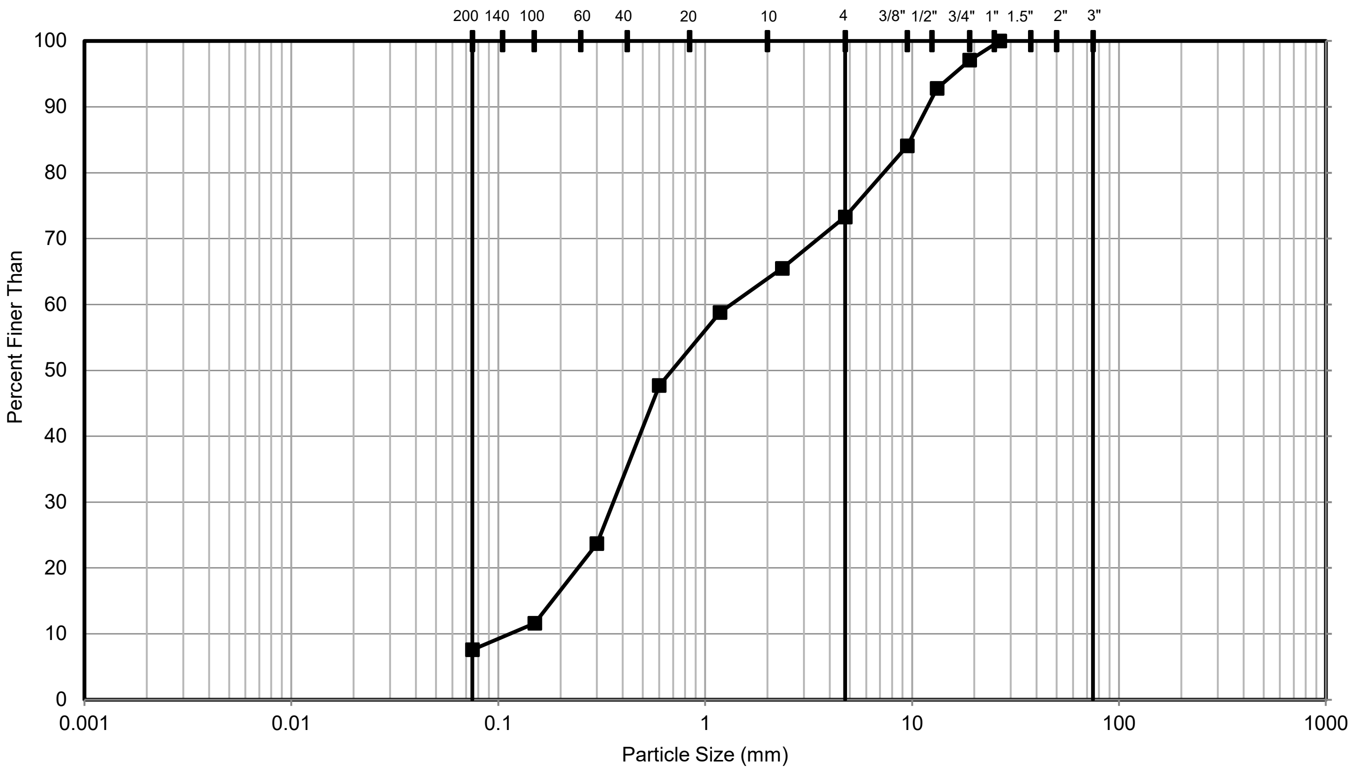
YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

LOWER CLAYEY SILT (CL)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B16

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	400-3	17	21.3 - 21.6	237.1 to 236.8

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Highway 400 Interchange

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-04-14
DESIGNED	MCK
PREPARED	MCK
REVIEWED	KJB
APPROVED	KJB

TITLE

LOWER GRAVELLY SAND (SW)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	B17

APPENDIX C

Soil Analytical Test Results

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

Attention: Manisha Ahuja

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

Report Date: 2022/01/07
Report #: R6953535
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C1Z9994
Received: 2021/12/20, 17:36

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	3	2021/12/29	2022/01/06	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	3	2021/12/29	2021/12/29	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	3	N/A	2021/12/23	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	3	N/A	2021/12/22	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2021/12/22	2021/12/22	CAM SOP-00413	EPA 9045 D m
pH CaCl2 EXTRACT	1	2021/12/23	2021/12/23	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2021/12/20	2021/12/29	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	2021/12/29	2022/01/07	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE , Calgary, AB, T2E 6P8
(2) Offsite analysis requires that subcontracted moisture be reported.

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

Attention: Manisha Ahuja

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

Report Date: 2022/01/07
Report #: R6953535
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C1Z9994
Received: 2021/12/20, 17:36

Encryption Key AUTHORIZED REPORT RAPPORT AUTORISÉ Bureau Veritas 07 Jan 2022 18:58:33

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: emese.gitej@bureauveritas.com
Phone# (905)817-5829
==
This report has been generated and distributed using a secure automated process.
BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Bureau Veritas Job #: C1Z9994
Report Date: 2022/01/07

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

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		RLD018		RLD019		
Sampling Date		2021/12/07		2021/12/17		
COC Number		n/a		n/a		
	UNITS	**BH400-1 SA-03 5'-7'**	**QC Batch**	**BH400-3 SA-02 2'6''-4'6''**	**RDL**	**QC Batch**
Calculated Parameters						
Resistivity	ohm-cm	1100	7744133	5300		7744133
Inorganics						
Soluble (20:1) Chloride (Cl-)	ug/g	480	7756920	<20	20	7756920
Conductivity	umho/cm	893	7757558	188	2	7757558
Available (CaCl2) pH	pH	7.77	7748024	7.63		7750875
Soluble (20:1) Sulphate (SO4)	ug/g	<20	7756946	<20	20	7756946
Sulphide	mg/kg	3.3 (1)	7752526	5.8	0.5	7752526
Physical Testing						
Moisture-Subcontracted	%	23	7752565	21	0.30	7752565

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
(1) Sample extracted past method-specified hold time. Analyzed past method specified hold time

Bureau Veritas ID		RLD019			RLD020		
Sampling Date		2021/12/17			2021/12/14		
COC Number		n/a			n/a		
	UNITS	**BH400-3 SA-02 2'6''-4'6'' Lab-Dup**	**RDL**	**QC Batch**	**BH400-4 SA-03 5'-7'**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm				2300		7744133
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	7756920	180	20	7756920
Conductivity	umho/cm				435	2	7757558
Available (CaCl2) pH	pH	7.66		7750875	7.88		7748024
Soluble (20:1) Sulphate (SO4)	ug/g				<20	20	7756946
Sulphide	mg/kg				4.1 (1)	0.5	7752526
Physical Testing							
Moisture-Subcontracted	%				12	0.30	7752565

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
(1) Sample extracted past method-specified hold time. Analyzed past method specified hold time

Bureau Veritas Job #: C1Z9994
Report Date: 2022/01/07

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

TEST SUMMARY

Bureau Veritas ID: RLD018
Sample ID: BH400-1 SA-03 5'-7'
Matrix: Soil

Collected: 2021/12/07
Shipped:
Received: 2021/12/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7756920	2021/12/29	2022/01/06	Alina Dobreanu
Conductivity	AT	7757558	2021/12/29	2021/12/29	Kien Tran
Moisture (Subcontracted)	BAL	7752565	N/A	2021/12/23	Parveer Singh
Sulphide in Soil	SPEC	7752526	N/A	2021/12/22	Bailey Morrison
pH CaCl2 EXTRACT	AT	7748024	2021/12/22	2021/12/22	Taslima Aktar
Resistivity of Soil		7744133	2021/12/29	2021/12/29	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7756946	2021/12/29	2022/01/07	Avneet Kour Sudan

Bureau Veritas ID: RLD019
Sample ID: BH400-3 SA-02 2'6''-4'6''
Matrix: Soil

Collected: 2021/12/17
Shipped:
Received: 2021/12/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7756920	2021/12/29	2022/01/06	Alina Dobreanu
Conductivity	AT	7757558	2021/12/29	2021/12/29	Kien Tran
Moisture (Subcontracted)	BAL	7752565	N/A	2021/12/23	Parveer Singh
Sulphide in Soil	SPEC	7752526	N/A	2021/12/22	Bailey Morrison
pH CaCl2 EXTRACT	AT	7750875	2021/12/23	2021/12/23	Taslima Aktar
Resistivity of Soil		7744133	2021/12/29	2021/12/29	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7756946	2021/12/29	2022/01/07	Avneet Kour Sudan

Bureau Veritas ID: RLD019 Dup
Sample ID: BH400-3 SA-02 2'6''-4'6''
Matrix: Soil

Collected: 2021/12/17
Shipped:
Received: 2021/12/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7756920	2021/12/29	2022/01/06	Alina Dobreanu
pH CaCl2 EXTRACT	AT	7750875	2021/12/23	2021/12/23	Taslima Aktar

Bureau Veritas ID: RLD020
Sample ID: BH400-4 SA-03 5'-7'
Matrix: Soil

Collected: 2021/12/14
Shipped:
Received: 2021/12/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7756920	2021/12/29	2022/01/06	Alina Dobreanu
Conductivity	AT	7757558	2021/12/29	2021/12/29	Kien Tran
Moisture (Subcontracted)	BAL	7752565	N/A	2021/12/23	Parveer Singh
Sulphide in Soil	SPEC	7752526	N/A	2021/12/22	Bailey Morrison
pH CaCl2 EXTRACT	AT	7748024	2021/12/22	2021/12/22	Taslima Aktar
Resistivity of Soil		7744133	2021/12/29	2021/12/29	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7756946	2021/12/29	2022/01/07	Avneet Kour Sudan

GENERAL COMMENTS

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

Package 1	2.7°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

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

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7748024	Available (CaCl2) pH	2021/12/22			100	97 - 103			0.25	N/A
7750875	Available (CaCl2) pH	2021/12/23			100	97 - 103			0.44	N/A
7752526	Sulphide	2021/12/22	106	75 - 125	108	75 - 125	<0.5	mg/kg		
7752565	Moisture-Subcontracted	2021/12/23					<0.30	%		
7756920	Soluble (20:1) Chloride (Cl-)	2022/01/06	114	70 - 130	103	70 - 130	<20	ug/g	NC	35
7756946	Soluble (20:1) Sulphate (SO4)	2022/01/07	NC	70 - 130	107	70 - 130	<20	ug/g	3.8	35
7757558	Conductivity	2021/12/29			98	90 - 110	<2	umho/cm	3.0	10

N/A = Not Applicable

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

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

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

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

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

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

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

VALIDATION SIGNATURE PAGE

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

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

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

Orla Jorgensen, Organics Lab Manager

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
CAM FCD-01191/6

WORK ORDER

CHAIN OF CUSTODY RECORD

Invoice Information

Company Name: Golder Associates Ltd.
Contact Name: Canada Accounts Payable
Address: 6925 Century Ave. Suite 100, Mississauga, ON
Phone: 905-567-4444 Fax: 905-567-6561
Email: canadaaccountspayableinvoices@golder.com

Report Information (if differs from invoice)

Company Name: Golder Associates Ltd.
Contact Name: Manisha Ahuja
Address: 6925 Century Ave. Suite 100, Mississauga, ON L5N 7K2
Phone: 365-292-1471 Fax: 905-567-6561
Email: Manisha_Ahuja@golder.com

Project Information (where applicable)

Quotation #:
P.O. #/ AFE#:
Project #: 19136074
Site Location: Bradford Bypass
Site #:
Site Location Province: Bradford Ontario
Sampled By: ______ AM

Turnaround Time (TAT) Required

[X] Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)
[ ] 1 Day [ ] 2 Days [ ] 3-4 Days

Date Required:
Rush Confirmation #:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS DRINKING WATER CHAIN OF CUSTODY

Regulation 153

[X] Table 1 [ ] Res/Park [ ] Med/ Fine
[ ] Table 2 [ ] Ind/Comm [ ] Coarse
[ ] Table 3 [ ] Agri/ Other
[ ] Table ____
FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] MISA [ ] Storm Sewer Bylaw
[ ] PWQO Region
[ ] Other (Specify)
[ ] REG 558 (MIN. 3 DAY TAT REQUIRED)
[ ] REG 406 Table ____

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrosivity Package (+ Sulphide)	HOLD- DO NOT ANALYZE	COMMENTS
1	BH 400-1 SA-03 5'-7'	2021-12-07	PM	SOIL	2								X		2 Jars, no redox.
2	BH 400-3 SA-02 2'6"-4'6"	2021-12-17	PM	SOIL	1								X		1 Jar, no redox.
3	BH 400-4 SA-03 5'-7'	2021-12-14	PM	SOIL	2								X		2 Jars, no redox.
4															
5															
6															
7															
8															
9															
10															

20-Dec-21 17:36
Ema Gitej
C1Z9994
ATM ENV-1464

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.

LABORATORY USE ONLY

CUSTODY SEAL Y / N
Present | Intact
COOLER TEMPERATURES: 4 | 2 | 2
COOLING MEDIA PRESENT: Y / N

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	BV JOB #
/ Albert M	2021/12/20	18:00	VEDHT BHAKAR	2021/12/20	17:36	

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

Attention: Muhammad Talha Irshad

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

Report Date: 2022/12/21
Report #: R7440310
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C2AD661
Received: 2022/12/12, 10:15

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	2	2022/12/16	2022/12/16	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2022/12/16	2022/12/16	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2022/12/16	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2022/12/19	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2022/12/19	2022/12/19	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2022/12/13	2022/12/16	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	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

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

Report Date: 2022/12/21
Report #: R7440310
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C2AD661
Received: 2022/12/12, 10:15

Encryption Key

Ankita Bhalla
Project Manager
22 Dec 2022 09:03:17

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 Job #: C2AD661
Report Date: 2022/12/21

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD
Your P.O. #: 19136074
Sampler Initials: MT

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		UOB264			UOB264			UOB265		
Sampling Date		2022/10/27			2022/10/27			2022/11/09		
COC Number		NA			NA			NA		
	UNITS	**2-2 5'-'7**	**RDL**	**QC Batch**	**2-2 5'-'7 Lab-Dup**	**RDL**	**QC Batch**	**400-2 0'-'7**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	5700		8400535				5100		8400535
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	8408097				<20	20	8408097
Conductivity	umho/cm	177	2	8408336				196	2	8408336
Available (CaCl2) pH	pH	7.74		8411397	7.72		8411397	7.60		8411449
Soluble (20:1) Sulphate (SO4)	ug/g	21	20	8408082				<20	20	8408082
Sulphide	mg/kg	5.2 (1)	0.5	8417625	4.4	0.5	8417625	4.7 (1)	0.5	8417625
Physical Testing										
Moisture-Subcontracted	%	16	0.30	8409298				14	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 ID		UOB265	
Sampling Date		2022/11/09	
COC Number		NA	
	UNITS	**400-2 0'-'7 Lab-Dup**	**QC Batch**
Inorganics			
Available (CaCl2) pH	pH	7.57	8411449

QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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

Bureau Veritas Job #: C2AD661
Report Date: 2022/12/21

Golder Associates Ltd
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	Gurparteek KAUR
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 ID: UOB265
Sample ID: 400-2 0'-'7
Matrix: Soil

Collected: 2022/11/09
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	Gurparteek KAUR
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	8411449	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: UOB265 Dup
Sample ID: 400-2 0'-'7
Matrix: Soil

Collected: 2022/11/09
Shipped:
Received: 2022/12/12

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	8411449	2022/12/19	2022/12/19	Taslima Aktar

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

BUREAU VERITAS

Bureau Veritas Job #: C2AD661
Report Date: 2022/12/21

Golder Associates Ltd
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

Results relate only to the items tested.

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

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

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8408082	Soluble (20:1) Sulphate (SO4)	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 (CaCl2) pH	2022/12/19			100	97 - 103			0.24	N/A
8411449	Available (CaCl2) pH	2022/12/19			100	97 - 103			0.36	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).

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Bureau Veritas Job #: C2AD661
Report Date: 2022/12/21

Golder Associates Ltd
Client Project #: 19136074
Site Location: BRADFORD
Your P.O. #: 19136074
Sampler Initials: MT

VALIDATION SIGNATURE PAGE

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

Veronica Falk

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

Eva Pranjic
CHARTERED
Eva Pranjic
CHEMIST

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

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

BUREAU VERITAS

WORK ORDER

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

CHAIN OF CUSTODY RECORD

Invoice Information

Company Name: Golder Associates Ltd.
Contact Name: Kevin Bentley
Address: 6925 Century Ave., Suite 100, Mississauga, ON
Phone: 905-567-6100 Fax:
Email: gld.canadaaccountspayableinvoices@wsp.com; kevin.bentley@wsp.com

Report Information (if differs from invoice)

Company Name: same
Contact Name: Muhammad Talha Irshad
Address: 6925 Century Ave., Suite 100, Mississauga, ON
Phone: 778-228-5756 Fax:
Email: muhammad.irshad@wsp.com; 120387@golder.com

Project Information (where applicable)

Quotation #: Golder rates
P.O. #/ AFE#: 19136074
Project #: 19136074
Site Location: Bradford
Site #:
Site Location Province: Ontario
Sampled By:

Turnaround Time (TAT) Required

[X] Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)
[ ] 1 Day [ ] 2 Days [ ] 3-4 Days

Date Required:
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

[ ] Table 1 [ ] Res/Park [ ] Med/ Fine
[ ] Table 2 [ ] Ind/Comm [ ] Coarse
[ ] Table 3 [ ] Agri/ Other
[ ] Table ____
FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] MISA [ ] Storm Sewer Bylaw
[ ] PWQO Region
[ ] Other (Specify)
[ ] REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	CORROSIVITY PACKAGE (+ SULPHIDE)	HOLD- DO NOT ANALYZE	COMMENTS
1	2-2 5'-7'	2022/10/27	PM	Soil	2								X		2 JARS. NO REDOX POTENTIAL.
2	400-2 0'-7'	2022/11/09	PM	Soil	2								X		2 JARS. NO REDOX POTENTIAL.
3															
4															
5															
6															
7															
8															
9															
10															

LABORATORY USE ONLY

CUSTODY SEAL Y / N Present	Intact	COOLER TEMPERATURES
✓	N	5/7/5

COOLING MEDIA PRESENT: (Y) / N

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.

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 / Talha	2022-12-12		Pam FB	2022/12/12	10:15

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 acknov
at http://www.bvlabs.com/terms-and-conditions

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