

**FINAL REPORT**

# **Preliminary Foundation Investigation and Design Report**

*Bradford Bypass / Highway 400 Interchange Structures Over 9th Line (N-E Ramp over 9th Line, Highway 400 over 9th Line Replacement, E-N Ramp over 9th Line)  
Simcoe County and York Region  
MTO Assignment No. 2019-E-0048*

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# PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
BRADFORD BYPASS / HIGHWAY 400 INTERCHANGE STRUCTURES OVER  
9<sup>TH</sup> LINE (N-E RAMP OVER 9<sup>TH</sup> LINE, HIGHWAY 400 OVER 9<sup>TH</sup> LINE  
REPLACEMENT, E-N RAMP OVER 9<sup>TH</sup> LINE)  
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 structures to carry the BBP/Highway 400 Interchange ramps and widened Highway 400 over 9<sup>th</sup> Line as shown in Drawing 1.

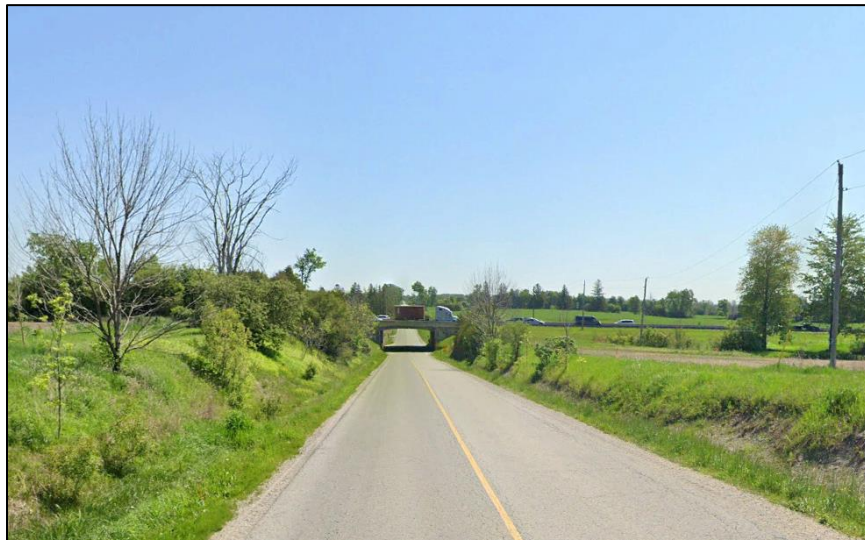
- **N-E Ramp over 9<sup>th</sup> Line:** a new single-span structure carrying the BBP/Hwy 400 N-E ramp over 9<sup>th</sup> Line located between about Station 10+187 and 10+216 relative to the N-E Ramp stationing. The centre of the proposed bridge is at about Station 9+935 relative to the 9<sup>th</sup> Line stationing.
- **Highway 400 over 9<sup>th</sup> Line Replacement:** the interim configuration consists of replacing the existing single-span structure with two new single-span structures carrying the Hwy 400 southbound lanes and northbound lanes respectively over 9<sup>th</sup> Line. The proposed bridges are located between about Station 18+872 and 18+900 relative to the Highway 400 stationing with a gap of about 17 m between each bridge. The centre point between the two interim structures is at about Station 10+000 relative to 9<sup>th</sup> Line stationing. Future ultimate widening of Highway 400 southbound and northbound lanes towards the Highway 400 centreline is proposed in the future, basically filling the gap between the interim bridges.
- **E-N Ramp over 9<sup>th</sup> Line:** a new single-span structure carrying the BBP/Hwy 400 E-N ramp over 9<sup>th</sup> Line located between about Station 10+910 and 10+938 relative to the E-N Ramp stationing. The centre of the proposed bridge is located at about Station 10+070 relative to the 9<sup>th</sup> Line stationing.

## 2.0 SITE DESCRIPTION

The site of the proposed structures crossing over 9<sup>th</sup> Line 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 existing Hwy 400 / 9<sup>th</sup> Line overpass is a single span bridge with an approximate span of 11 m. It was constructed in the early 1950's under Contract 50-157 and the abutments are founded on spread footings at about Elevation 269.9 m. There are existing concrete toe walls located at each corner of the existing bridge near the bottom (toe) of the approach embankments. There was no obvious signs of major settlement or erosion observed at the bridge location during the site investigation, although minor tilting of the southeast toe wall (towards 9<sup>th</sup> Line) was observed. The existing 9<sup>th</sup> Line is currently a paved two-lane arterial road with gravel shoulders and ditches.

The general site (east and west of Highway 400) consists of farmland. The existing ground surface generally slopes down from west to east, with the Highway 400 grade appearing to have been constructed near ground level at the southbound lanes (see Photograph 1) and on a partial fill embankment on the northbound lanes (see Photograph 2). As a result, 9<sup>th</sup> Line appears to have been constructed predominantly in a cut on the west side of the bridge and near ground surface or partial cut on the east side of the bridge.





*Photograph 1 – Proposed west side of N-E Ramp structure location  
(looking east from 9<sup>th</sup> Line)*



*Photograph 2 – Proposed east side of E-N Ramp structure location  
(looking west from 9<sup>th</sup> Line)*



### 3.0 INVESTIGATION PROCEDURES

#### 3.1 Previous Borehole Investigation

Two boreholes were advanced at the 9<sup>th</sup> Line site as part of a previous Golder Associates Ltd. (Golder) geotechnical investigation in 2000 for the preliminary design of the 9<sup>th</sup> Line (formerly 9<sup>th</sup> Concession) overpass in support of the ultimate widening of Highway 400 (Golder, 2001)<sup>1</sup>. Boreholes B4-1 and B4-2 were advanced east and west of Highway 400, respectively, from the 9<sup>th</sup> Line cut grade. The boreholes were advanced to depths of 20.4 m and 23.3 m below ground surface. The Record of Boreholes and associated laboratory test results are provided in Appendix A and the borehole locations are shown on Drawing 1.

Both Borehole B4-1 and B4-2 were advanced using 108 mm diameter solid stem augers. Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure<sup>2</sup>.

The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B4-2. The details of the boreholes including locations in MTM NAD83 (Zone 10) northing and easting coordinates, geographic (Latitude / Longitude) coordinates, ground surface elevations referenced to Geodetic datum and drilled depths are summarized below.

Borehole Number	NAD 83 MTM Northing (m) (Latitude, °)	NAD 83 MTM Easting (m) (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
B4-1	4,887,127.3 (44.124068)	293,870.9 (-79.636565)	271.7	23.3
B4-2	4,887,124.8 (44.124044)	293,822.5 (-79.637169)	272.1	20.4

#### 3.2 Current Borehole Investigation

The field work for the current investigation was carried out between April 12 and 13, 2021, during which time one borehole (designated Borehole 9-1) was advanced to a depth of 33.9 m at the location shown on Drawing 1. A copy of the borehole record is provided in Appendix B.

The borehole was 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 D54 track-mounted drill supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The wash-rotary technique was used to counter-balance hydrostatic forces and reduce disturbance at the sampling and testing interval. Water used for the drilling operation was brought to site in totes (portable plastic tanks) by the drilling subcontractor.

Soil samples were generally obtained at 0.75 m, 1.5 m, and 3.0 m intervals of depth using a 50 mm O.D. split spoon sampler driven with an automatic hammer in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586<sup>2</sup>). The split-spoon samplers used in the investigation generally limit the maximum

<sup>1</sup> Golder Associates Ltd. 2001. *Preliminary foundation investigation and design report, Ninth concession overpass structure site 30-308, Highway 400 widening from York/Simcoe boundary to 1 km south of Highway 89, G.W.P. 40-00-00.*

<sup>2</sup> ASTM D1586 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils

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 not measured in the open borehole due to the introduction of water during drilling operations. A standpipe piezometer was installed approximately 3 m north of Borehole 9-1 and was screened within a silty sand till deposit. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 3 m long slotted screen within a filter sand pack. The borehole and the 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<sup>3</sup> (as amended). The monitoring well was capped with a monument casing.

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

One soil sample obtained from the 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 location was surveyed in the field by Golder personnel using a Trimble Geo 7X Global Positioning System (GPS) unit. The location given on the borehole record and shown on Drawing 1 is positioned relative to NAD 83 MTM (Zone 10) northing and easting coordinates and the ground surface elevation is 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)
9-1	4,887,082.5 (44.123655)	293,780.6 (-79.637689)	275.6	33.9

<sup>3</sup> Ontario Regulation 903 Wells

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)<sup>4</sup>, 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 subsurface conditions encountered during the current investigation are generally consistent with the variable regional geology described above.

### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes from the current and previous 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 and B. 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. The detailed results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix A and C. The results of the analytical testing are provided in Appendix D.

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

In summary, the soil encountered at this site consists of cohesive and non-cohesive fill overlying an upper till deposit consisting predominantly of silty sand, and transitioning to a clayey silt below the N-E Ramp and western portion of Highway 400 (Boreholes 9-1 and B4-2). A silty sand to sand layer was encountered below the upper till. Below the silty sand to sand layer at the N-E Ramp location (Borehole 9-1) a deposit of clayey silt to silty clay was encountered, and at the Highway 400 and E-N Ramp location (Boreholes B4-1 and B4-2) a lower deposit of till consisting of clayey silt and silty sand was encountered.

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<sup>4</sup> Chapman, L. J. and Putnam, D. F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition.

More detailed descriptions of the major soil layers encountered in the boreholes as well as a summary of laboratory results are provided in the following sections.

#### **4.2.1 Fill**

A 0.7 m thick layer of sandy clayey silt fill was encountered at ground surface (Elevation 275.6 m) in Borehole 9-1, and a 1.4 m thick layer of sand and gravel to silty sand fill was encountered at ground surface (Elevation 271.7 m) in Borehole B4-1.

The SPT 'N'-value measured in the clayey silt fill was 8 blows per 0.3 m of penetration, suggesting a stiff consistency. The SPT 'N'-values measured in the sand and gravel to silty sand fill were 9 and 22 blows per 0.3 m of penetration, indicating a loose to compact degree of compactness.

#### **4.2.2 Upper Silty Sand Till**

A 7.3 m to 10.8 m thick silty sand till deposit was encountered below the fill in Boreholes 9-1 and B4-1, and at ground surface (Elevation 272.1 m) in Borehole B4-2. The silty sand till was described as a sand and silt till in the previous borehole records.

Slow advancement and grinding of the augers was observed in the silty sand till in Borehole 9-1 and could suggest the presence of gravel pockets, cobbles, or boulders within the glacial till deposit.

The SPT 'N'-values measured in the silty sand till range from 7 to 74 blows per 0.3 m of penetration, indicating a loose to very dense degree of compactness.

Grain size distribution testing was carried out on three samples of the silty sand till deposit and the results are shown on Figure A1 in Appendix A and Figure C1 in Appendix C.

Atterberg limits testing was carried out on two samples of the upper silty sand till from the current investigation and four samples from the previous investigation, and indicated liquid limits of 12% to 14%, plastic limits of 10% to 11%, and plasticity indices of 2% to 4%, indicating the fines portion of the deposit is a silt of slight plasticity. The results of the Atterberg limits tests carried out on the upper silty sand till deposit from the current investigation are plotted on the plasticity chart on Figure C2 in Appendix C, and the results from the previous investigation are shown on the Record of Boreholes in Appendix A.

The natural water content measured on selected samples of the upper silty sand till deposit range from about 6% to 11%.

#### **4.2.3 Upper Clayey Silt Till**

Underlying the upper silty sand till, a 1.4 m to 1.5 m thick layer of clayey silt till was encountered in Boreholes 9-1 and B4-2.

The SPT 'N'-values measured in the clayey silt till layer were 11 and 12 blows per 0.3 m, suggesting a stiff consistency.

A grain size distribution test was carried out on a sample of the clayey silt till deposit and the results are shown in Figure C3 in Appendix C.

An Atterberg limits test carried out on a sample of the clayey silt till layer indicated a liquid limit of 26%, a plastic limit of 14% and a plasticity index of 12%. The results, which are plotted on a plasticity chart on Figure C4, indicate that the deposit is a clayey silt of low plasticity.

The natural water content measured on a sample of the clayey silt till layer was 21%.

#### **4.2.4 Sand to Silty Sand**

A 1.5 m to 1.7 m thick sand to silty sand layer was encountered underlying the upper clayey silt till in Boreholes B4-2 and 9-1, and below the upper silty sand till in Borehole B4-1.

The SPT 'N'-values measured in the sand to silty sand range from 6 to 39 blows per 0.3 m of penetration indicating a loose to dense degree of compactness.

The natural water content measured on selected samples of the sand to silty sand layer range from about 10% to 11%.

#### **4.2.5 Lower Silty Sand Till**

A 1.4 to 9.6 m thick deposit of silty sand till was encountered underlying the sand to silty sand layer in Boreholes B4-1 and 9-1. A 0.9 m thick sand interlayer was encountered directly below the silty sand till layer in Borehole 9-1. Borehole B4-1 was terminated within the lower silty sand till deposit.

The SPT 'N'-values measured in the silty sand till deposit range from 20 blows per 0.3 m of penetration to 119 blows for 0.23 m of penetration, indicating a compact to very dense degree of compactness. The SPT 'N'-value measured in the sand layer was 20 blows per 0.3 m of penetration, indicating a compact degree of compactness.

Grain size distribution testing was carried out on a sample of the lower silty sand till deposit and the results are shown on Figure C5 in Appendix C.

Atterberg limits testing was carried out on a sample of the lower silty sand till deposit and measured a liquid limit of 12%, plastic limit of 11% and plasticity index of 1%. The results, which are plotted on a plasticity chart on Figure C6, indicate that the fines portion of the deposit is a silt of slight plasticity.

The natural water content measured on selected samples of the silty sand till range from about 7% to 11%. The natural water content measured on the sand layer sample was about 14%.

#### **4.2.6 Lower Clayey Silt Till**

A clayey silt till deposit was encountered in Borehole B4-2 underlying the sand to silty sand layer and was penetrated for a length of 10.2 m before borehole termination.

The SPT 'N'-values measured in the lower clayey silt till range from 27 to 117 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The natural water content measured on selected samples of the clayey silt till range from about 9% to 17%.

#### **4.2.7 Clayey Silt to Silty Clay**

A 14.2 m thick deposit of clayey silt to silty clay was encountered below the lower silty sand till and sand interlayer in Borehole 9-1. The upper 1.4 m of the deposit is described as sandy, and slow drilling was observed within the upper 2.6 m of the deposit. The lower portion of the deposit (below about Elevation 249.4 m) is described as containing silt laminations / seams.

The SPT 'N'-values measured in the clayey silt to silty clay deposit generally range from 25 to 53 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

Grain size distribution testing was carried out on a sample of the silty clay portion of the deposit and the results are shown on Figure C7 in Appendix C.

Atterberg limits testing was carried out on a sample of the clayey silt to silty clay deposit and measured a liquid limit of 37%, plastic limit of 18% and plasticity index of 19%. The results, which are plotted on a plasticity chart on Figure C8, indicate that the sample tested is a silty clay of intermediate plasticity.

The natural water content measured on selected samples of the clayey silt to silty clay deposit range from about 22% to 27%.

#### 4.2.8 Sandy Silt

A sandy silt deposit was encountered below the clayey silt to silty clay deposit in Borehole 9-1. The sandy silt deposit was penetrated for a length of 4.0 m before borehole termination.

The SPT 'N'-values measured in the sandy silt deposit were 102 blows per 0.22 m of penetration and 100 blows per 0.15 m of penetration, indicating a very dense degree of compactness.

Grain size distribution testing was carried out on a sample of the sandy silt deposit and the results are shown on Figure C9 in Appendix C.

The natural water content measured on selected sample of the non-cohesive sandy silt till is 22%

### 4.3 Groundwater Conditions

The water levels measured in the open boreholes at the time of the investigation from the previous investigation are shown on the borehole records in Appendix A and are not considered representative of the hydrostatic water levels at the site. The water level in Borehole 9-1 advanced during the current investigation was not recorded during drilling due to the addition of drilling fluids/water into the borehole during mud rotary operations.

Standpipe piezometers were installed in Boreholes B4-2 and 9-1 to allow monitoring of the stabilized hydrostatic groundwater level at this site. The groundwater levels recorded in the piezometers are shown on the borehole records in Appendix A and B and are summarized below.

Borehole No. (Piezometer)	Depth (bgs) (Elevation) of Screen Interval / Sand Pack (m)	Depth (bgs) to Water Level (m)	Water Level Elevation (m)	Date of Water Level Reading
9-1	6.2 – 9.2 (269.4 – 266.4)	1.5	274.1	December 10, 2021
B4-2	6.1 – 7.6 (266.0 – 264.5)	-(0.3) <sup>a,b</sup>	272.4	January 18, 2001
		0	272.1	March 20, 2001
		-(0.2) <sup>a</sup>	272.3	June 19, 2001

Notes: a. Above ground surface and indicates artesian conditions  
b. Water frozen in piezometer

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

One soil sample was 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 D and the test results are summarized below:

Borehole No., Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity ( $\mu$ mho/cm)	Soluble Chlorides ( $\mu$ g/g)	Soluble Sulphates ( $\mu$ g/g)
9-1, Sa#3	7.91	11,000	90	<20 *	< 20 *

Note: \* Less than reportable detection limit.

## 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Lina Mohamed 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.

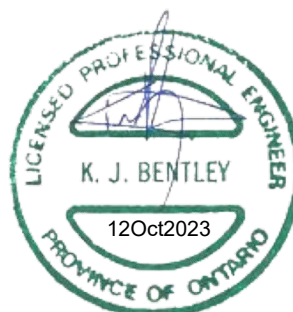
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# PART B

**PRELIMINARY FOUNDATION DESIGN REPORT  
BRADFORD BYPASS / HIGHWAY 400 INTERCHANGE STRUCTURES OVER  
9<sup>TH</sup> LINE (N-E RAMP OVER 9<sup>TH</sup> LINE, HIGHWAY 400 OVER 9<sup>TH</sup> LINE  
REPLACEMENT, E-N RAMP OVER 9<sup>TH</sup> LINE)  
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 9<sup>th</sup> Line structures associated with the Bradford Bypass and Highway 400 Interchange ramps and widening of Highway 400. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced as part of the current and previous 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 Bradford Bypass mainline alignment, profile drawings and General Arrangement (GA) drawings provided by AECOM (preliminary draft dated July, February, and May 2023, respectively), both Highway 400 and 9<sup>th</sup> Line will be widened to interim configurations, with future widening planned for the ultimate configurations.

For the 9<sup>th</sup> Line interim configuration, it is proposed that 9<sup>th</sup> Line will be lowered to accommodate two lanes with shoulders and the grade lowered beyond the shoulders. The new BBP / Hwy 400 Interchange ramp bridges are to be designed to cross over 9<sup>th</sup> Line to accommodate the ultimate configuration. The ultimate widening of 9<sup>th</sup> Line is a municipal initiative and is proposed to carry four lanes that will also include additional boulevards and multi-use paths in both the east and west directions.

For the Highway 400 interim configuration, it is proposed that the existing bridge structure be replaced with two new structures to accommodate the current 6 lanes of traffic (3 lanes plus shoulders in both the northbound and southbound direction). The new bridges will be separated by about 17 m to allow for future bridge widening (towards the Hwy 400 median) to the ultimate configuration of Highway 400 that will accommodate 10 lanes of traffic (5 lanes plus shoulders in both northbound and southbound directions).

Based on the Preliminary GA drawings, the proposed 9<sup>th</sup> Line structures associated with the BBP / Highway 400 Interchange and Highway 400 widening will consist of two new ramp structures, as well as replacement of the Highway 400 / 9<sup>th</sup> Line overpass, the details are provided below:

- **N-E Ramp over 9<sup>th</sup> Line:** single-span structure (about 29 m long and 12 m wide) carrying BBP/Hwy 400 N-E ramp over 9<sup>th</sup> Line located between about Station 10+187 and 10+216 relative to the N-E Ramp stationing and at about Station 9+935 relative to 9<sup>th</sup> Line stationing. At this location, 9<sup>th</sup> line is proposed to be widened and the road grade lowered by about 2 m. Localized cut depths for the widening will be up to about 6.5 m near and adjacent to the front of the closed abutment walls. At the approach embankments, the north side will match the existing ground surface and south side will require about a 2 m high grade raise. The closed

abutments will have wing walls / retaining walls surrounding all four quadrants of the bridge to accommodate lowering of 9<sup>th</sup> Line and the slight grade raise on north side.

- **Highway 400 over 9<sup>th</sup> Line Replacement:** existing Hwy 400 / 9<sup>th</sup> Line overpass to be removed and replaced with two single-span structures (each structure having a span of about 27.5 m and about 17 m wide) to carry Hwy 400 over 9<sup>th</sup> Line. The new structures are to be located between about Station 18+872 and 18+900 relative to Highway 400 chainage and the mid-point between the two structures is at about Station 10+000 relative to 9<sup>th</sup> Line chainage. At this location, the existing 9<sup>th</sup> line grade will not change significantly although deeper cuts up to about 7 m are required near the front of the closed abutment walls at the west bridge location, where 9<sup>th</sup> Line is constructed in cut and within the existing Hwy 400 embankment, to accommodate the future widening for the ultimate configuration of 9<sup>th</sup> Line. For the east bridge approach embankments, where the existing ground surface is near the 9<sup>th</sup> Line grade, the north and south approach embankments are anticipated to require fills up to about 6 m high. For the west bridge approach embankments, the north and south approach embankments are anticipated to be similar to the N-E ramp bridge configuration. The closed abutments will have wing walls / retaining walls surrounding all four quadrants of each bridge.
- **E-N Ramp over 9<sup>th</sup> Line:** single-span structure (about 27.5 m long and 12 m wide) carrying BBP/Hwy 400 E-N ramp over 9<sup>th</sup> Line located between about Station 10+910 and 10+938 relative to the E-N Ramp chainage and the centre of which is at about Station 10+070 relative to 9<sup>th</sup> Line stationing. The proposed approach embankments are planned to be about 6 m high at the north and south abutments. The closed abutments will have wing walls / retaining walls surrounding all four quadrants of each bridge.

The structural classification of the proposed bridges is defined as “major-route” by the structural designer at this preliminary design stage.

## 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 structures and foundation systems 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 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 configuration of the interchange and location of the ramps was not confirmed and boreholes were located in the general area of the proposed structures. As such, recommendations contained in the report are generalized for planning and ongoing preliminary design and further investigation will be required when detailed locations of the abutments are confirmed.

Accordingly, the ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$  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,  $\bar{N}_{60}$  within the upper 30 m of the overburden below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019), in the absence of any geophysical testing. Site-specific testing, such as vertical seismic profiling or multi-channel analysis of surface waves (MASW) may be used to assess the average shear wave velocity of the soils, and it may be possible (although not guaranteed) to upgrade this preliminary seismic site class.

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 5<sup>th</sup> generation seismic hazard maps) have been used for preliminary design for this project, as referenced in the CHBDC (2019).

### 6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the 2019 CHBDC, the peak ground acceleration ( $PGA$ ), peak ground velocity ( $PGV$ ) and 5% damped spectral response acceleration ( $S_a(T)$ ) values for Site Class D were obtained for the bridge sites using the NBCC website ([earthquakescanada.nrcan.gc.ca](http://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	0.058	0.093
$PGV$ (m/s)	0.038	0.060	0.097
$S_a(0.2)$ (g)	0.064	0.095	0.149
$S_a(0.5)$ (g)	0.053	0.078	0.118
$S_a(1.0)$ (g)	0.033	0.050	0.074
$S_a(2.0)$ (g)	0.016	0.025	0.039
$S_a(5.0)$ (g)	0.003	0.006	0.009
$S_a(10.0)$ (g)	0.001	0.003	0.004

The Site Class and associated design spectral values will need to be reassessed and modified based on site-specific shear wave velocity measurements, if applicable, along with the importance category (defined as “major-route” by the structural designer for preliminary design 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 generally compact to very dense silty sand and very stiff to hard clayey silt tills with generally compact to dense sand to silty sand interlayers or very stiff to hard clayey silt to silty clay. 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. Additionally, assessment of the cyclic mobility of the cohesive deposit(s) encountered at this site should be carried out during detail design when more site-specific foundation soil information is available and when the seismic performance category is confirmed, and the associated impacts on stability and settlement should be reassessed, as required.

## 6.4 Foundation Types

Based on the structure configurations (single-span structures with total span lengths less than 29 m) and subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments. The preliminary recommendations provided herein will be subject to change when more detailed soil information is 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, strip or spread footings founded on the generally compact to very dense upper silty sand till deposit or “perched” on a compacted granular pad above the compact to very dense silty sand till within the approach embankments is considered the preferred alternative from a geotechnical/foundations perspective. Driven piles are also considered feasible for the structures and may provide higher capacities compared to shallow foundations and are preferred if integral abutments are to be designed. Caissons are also considered feasible, however they are not considered to have a significant advantage over shallow foundations or piles given there is no competent end-bearing stratum at shallow depth, and unless they can be cast continuously with columns so as to avoid construction of pile caps.

Shallow foundations “perched” on a compacted granular pad will reduce or eliminate concerns with the relatively high (possible artesian) groundwater level. Steel H-piles or tube piles driven into the “100-blow” soils will range from about 20 m to 25 m below the 9<sup>th</sup> line cut grade, and consideration should be given to “perched” pile caps within the approach embankment to reduce dewatering concerns. Caissons founded 20 to 25 m below ground surface will provide higher capacities than piles but will require drilling slurry and temporary casings to maintain an open hole during advancement through the saturated silty sand and sand deposits, and challenges may be anticipated if artesian groundwater conditions are present.

### 6.4.1 Shallow Foundations

Strip or spread footings founded on the compact to dense silty sand till (at or below the approximate elevations identified below) are considered feasible for support of the ramp structure abutments and Highway 400 structure abutments. Based on the proposed 9<sup>th</sup> Line grade lowering and assuming closed abutments, subexcavation depths are anticipated to range from 1.5 m below ground surface (bgs) (east side) to about 8 m bgs (west side).



The following geotechnical resistances may be used for the preliminary design of the new ramp structures and Highway 400 replacement structure at 9<sup>th</sup> Line, assuming a 3 m or 5 m wide footing:

Structure	Founding Stratum	Founding Elevation <sup>2,3</sup>	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance <sup>1</sup>
N/E Ramp and E/N Ramp over 9 <sup>th</sup> Line	Compact to Very Dense Silty Sand Till	270 m	3	450 kPa	300 kPa
			5	600 kPa	225 kPa
	Min. 3 m Compacted Granular Pad above Compact to Very Dense Silty Sand Till	At or below 270 m	3	600 kPa	475 kPa
			5	800 kPa	325 kPa
Highway 400 over 9 <sup>th</sup> Line (both structures)	Compact to Very Dense Silty Sand Till	270 m	3	450 kPa	250 kPa
			5	600 kPa	175 kPa
	Min. 3 m Compacted Granular Pad above Compact to Very Dense Silty Sand Till	At or below 270 m	3	600 kPa	375 kPa
			5	800 kPa	225 kPa

Notes:

1. For 25 mm of settlement independent of any settlements induced by surrounding grade changes / embankment loading. SLS values assume ultimate configuration dimensions.
2. Actual founding elevations for the E-N ramp structure foundations on the east side of Highway 400 must be investigated further and checked during detail design when more geotechnical information is available.
3. Foundations must meet the requirements for cover for frost protection, see Section 6.5.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and length, founding elevation, and thickness of granular pad (as applicable) and as such, the geotechnical resistances must be reviewed and revised if the footing width or length 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.

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 (compact to dense silty sand 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 26° and corresponding unfactored coefficient of friction,  $\tan \delta$ , of 0.5 may be used for preliminary design. An effective angle of friction of 33° and corresponding unfactored coefficient of friction,  $\tan \delta$ , 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” sandy silt, clayey silt till or silty sand till deposit are considered feasible for the foundations at the ramp and Highway 400 structures. Although competent “100-blow” end-bearing soil was encountered during the preliminary investigation, the thickness and consistency of the “100-blow” soil at the foundation elements will need to be confirmed during detail design.

During the current investigation slow augering and auger grinding was noted in the upper silty sand till deposit, suggesting the presence of potential pockets of gravel or cobbles and/or boulders. Cobbles and/or boulders and gravel pockets should be anticipated within the glacially derived till deposits and will need to be considered during detail design.

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

Structure	Approximate Pile Length	Estimated Pile Tip Elevation (Soil Strata Near Pile Tip)	Pile Type	Factored Ultimate Geotechnical Resistance <sup>1</sup>	Factored Serviceability Geotechnical Resistance <sup>1,2</sup>
N-E Ramp over 9 <sup>th</sup> Line	25 m	245 m (Very Dense “100-blow” Sandy Silt)	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
Highway 400 (both structures) and E-N Ramp over 9 <sup>th</sup> Line	20 m	250 m (“100-blow” Very Dense Silty Sand Till or Hard Clayey Silt Till)	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

Notes:

- Resistance values assume single pile and do not take into account pile group efficiency.
- 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.

The estimated factored ultimate geotechnical resistances provided above are calculated on both shaft and tip resistance, and assume piles have had sufficient time to “set-up” or “relax” and allow pore pressures to dissipate after initial driving in order to achieve the design geotechnical resistances. Although the estimated time for pore pressures to dissipate is difficult to predict in fine-grained soils, it is recommended that test loading of piles should not be carried out for at least two weeks after driving (CFEM, 2006). If higher capacities are required, consideration can be given to further increasing the size of the piles.

Considering the anticipated high loads for the highway bridges, pile groups at the foundation elements 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 till, clayey silt till or very dense sandy silt deposits are feasible for supporting the abutments for the Highway 400 and ramp structures. At the E-N Ramp and Highway 400 structures, 20 m long caissons founded on the very dense silty sand till or hard clayey silt till have been evaluated for preliminary design. For the N-E Ramp structure, 25 m long caissons penetrating into the very dense sandy silt deposit been evaluated for preliminary design. As borehole coverage is limited at this stage, it is recommended that evaluation of alternatives be completed during detail design subject to additional investigation.

Consideration must be given to the presence of potential gravel pockets, cobbles and boulders that may be present within the glacially derived till and non-cohesive deposits, as suggested by slow augering and auger grinding observed during borehole advancement during the current investigation.

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

Structure	Approximate Caisson Length	Estimated Caisson Base Elevation	Caisson Diameter	Factored Ultimate Geotechnical Resistance <sup>1</sup>	Factored Serviceability Geotechnical Resistance <sup>1,2</sup>
N-E Ramp over 9 <sup>th</sup> Line	25 m	245 m (Very Dense “100-blow” Sandy Silt)	0.9 m	2,500 kN	Does Not Govern
			1.2 m	3,500 kN	Does Not Govern
			1.5 m	5,500 kN	Does Not Govern
Highway 400 (both structures) and E-N Ramp over 9 <sup>th</sup> Line	20 m	250 m (Very Dense “100-blow” Silty Sand Till or Hard Clayey Silt Till)	0.9 m	2,500 kN	Does Not Govern
			1.2 m	3,500 kN	Does Not Govern
			1.5 m	5,500 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.

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 to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils (possibly artesian) and cohesive soils containing sand/silt seams. From an installation perspective, a permanent liner may be preferred over a temporary liner (particularly in the case of relatively deep shaft excavations) since there is no requirement to withdraw multiple casing strings and therefore allows for a faster installation time. Other drilled shaft construction methods such as polymer slurry drilling, which only requires a temporary “starter” casing to be withdrawn upon completion of concrete placement, could also be considered but would require a higher level of quality control / quality assurance and development of special provisions. From a design perspective, use of a permanent liner would decrease the available frictional

resistance and corresponding design geotechnical resistance due to the difference in adhesion between the liner material and soil versus the adhesion between concrete and soil which would need to be considered during detail design. The preferred use of a temporary versus permanent liner depends on the Contractors experience, equipment and selected means and methods.

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) 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. 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 polymer 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) used with temporary liners or use of permanent liners 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 (potential artesian) 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:

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

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

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

For cohesive soils:

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

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

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

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 at existing ground surface.

Soil Unit	Location Relative to Groundwater	$n_h$ (kPa/m)	$S_u$ (kPa)
New Granular Fill (Granular 'A' or 'B' Type II)	Above GW	40,000 – 50,000	-
Loose to Very Dense Silty Sand (Till)	Below GW	15,000 – 20,000	-
Loose to Dense Silty Sand to Sand	Below GW	8,000	-
Stiff to Hard Clayey Silt (Till)	Below GW	-	150-200
Very Stiff to Hard Clayey Silt to Silty Clay	Below GW	-	150
Very Dense Sandy Silt	Below GW	9,500 – 13,000	-

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.

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 GA drawings, the fill heights of the approach embankments at the bridges range from approximately 2 m high to 7 m high with total post-construction settlements in the foundation soils estimated to be less than 25 mm (see Section 6.6.2). As a result, downdrag loads are not anticipated to be a concern but must be assessed further during detail design.

### 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*). For preliminary purposes, 25 mm of rigid polystyrene foam insulation can be used as an equivalent to a 0.3 m reduction in soil cover, if required.

### 6.6 Approach Embankments

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

Structure	Height of Approach Embankment (Abutment Location)	Anticipated Foundation Soils
N-E Ramp over 9 <sup>th</sup> Line	Fill Height Up to 2 m (north abutment)  No significant cut / fill (south abutment)  *Cut Depth up to 6.5 m and 4 m required adjacent to south and north abutment wingwall to accommodate 9 <sup>th</sup> Line grade lowering	Compact to Very Dense Silty Sand Till
Highway 400 over 9 <sup>th</sup> Line	West Bridge – same as above  East Bridge - Fill Height Up to 6 m (north and south abutment)	Compact to Very Dense Silty Sand Till
E-N Ramp over 9 <sup>th</sup> Line	Fill Height Up to 6 m (north and south abutment)	Compact to Very Dense Silty Sand Till

\* Proposed cut is for 9<sup>th</sup> Line grade lowering and will be adjacent to approach embankment wing wall, and no cut is required for approach embankment itself.

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. Boreholes were not advanced at all approach embankment locations (within the fields north and south of 9<sup>th</sup> line and through the existing Hwy 400 embankment) and additional investigation in these areas will be

needed during detail design to confirm the depth of stripping required. 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 6 m high fills at the Highway 400 east bridge and E-N Ramp bridge approaches. Global stability was also carried out to check the feasibility of the proposed permanent cut slope on the south side 9<sup>th</sup> Line, directly adjacent to the proposed wing walls of the Highway 400 west bridge and N-E Ramp bridge.

For both the stability and settlement analyses, the groundwater elevation was assumed to be at the highest measured groundwater level in the piezometers at the closest borehole location and typically ranged from ground surface to 1.5 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,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$  (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	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$S_u$ (kPa)
New Granular Fill (Granular 'A' or 'B' Type II)	21	36	--
Loose to Very Dense Silty Sand (Till)	21	34	--
Loose to Dense Silty Sand to Sand	20	30	--
Stiff to Hard Clayey Silt (Till)	21	34	150
Very Stiff to Hard Clayey Silt to Silty Clay	19	30	150
Very Dense Sandy Silt	20	32	--

where:  $\gamma$  = bulk unit weight  
 $\phi'$  = effective friction angle  
 $S_u$  = undrained shear strength

The idealized geometry and results of the stability analyses (modelled for circular slip surfaces using *Slide2* (Version 9.020)) for the critical sections (i.e. highest approach embankment with fill heights up to 6 m on the east side of Highway 400 and the E-N Ramp, and the 6.5 m cut slope adjacent to the abutment wing walls for the Highway 400 and N-E Ramp structures) are shown in Figures 1 to 3.

Based on the results, the approach embankments for the N-E Ramp, E-N Ramp and Highway 400 bridges constructed with suitable granular fill for the embankment and 2H:1V (2 Horizontal : 1 Vertical) side slopes will have an adequate factor of safety (i.e., greater than 1.4 for short-term and greater than 1.6 for long-term as shown on Figure 1) for global stability. The permanent cut slopes along 9<sup>th</sup> Line adjacent to the abutment wing walls, through the native silty sand till and with 2.5H:1V (2.5 Horizontal : 1 Vertical) side slopes, were also calculated to have an adequate factor of safety for global stability for the short-term and long-term condition (Figure 2 and 3).

The water level in the stability models varied from 1.5 m below ground surface (Elevation 274.1 m measured in the piezometer in Borehole 9-1) to the existing or proposed ground surface (Elevation 272 m for the approach embankment fill and Elevation 271 m for the proposed bottom of the cut slope adjacent to the abutment wing walls). The interface between the existing Highway 400 embankment fill and native soil for the Highway 400 (east bridge) stability model is estimated and will need to be checked, updated and the stability analysis re-evaluated when detailed geotechnical information is obtained during detail design.

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. As a result, side-slopes of 2H:1V may be acceptable for the permanent cut if soil conditions are similar to the results of the preliminary investigation.

### 6.6.2 Settlement

Settlement analyses were carried out for the proposed maximum fill thickness (fill height) at the N-E Ramp, Highway 400, and E-N Ramp approach embankments. 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 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 post-construction settlement for the highest embankments at each structure site are presented below, assuming the use of conventional granular fill for construction. The estimated settlements do not account for the immediate settlement of the embankment fill and foundation soils which is expected to occur during or shortly after construction and would need to be assessed during detail design.

Structure	Proposed Maximum Embankment Fill	Estimated Post-Construction Settlement over a 20-Year Period
N-E Ramp over 9 <sup>th</sup> Line	2 m	$\delta_{total} < 25 \text{ mm}$
Highway 400 over 9 <sup>th</sup> Line (both structures)	6 m	$\delta_{total} < 25 \text{ mm}$
E-N Ramp over 9 <sup>th</sup> Line	6 m	$\delta_{total} < 25 \text{ mm}$

Based on the preliminary investigation and calculated results above, post-construction settlements are not anticipated to be a concern at the approach embankments for the structures at 9<sup>th</sup> Line.

### 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 sample submitted for testing are summarized in Section 4.4 and the analytical laboratory test report is included in Appendix D.

### 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 of 7.9 and a resistivity of 11,000 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to durability. The resistivity indicates that the soil corrosiveness is extremely low, above the range indicated in Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014). 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 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.

To reduce surface water erosion on the granular embankment side slopes and any permanent cut 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 or excavation of deep cuts.

### 6.8.2 Temporary Excavations

Temporary excavations up to 1.5 m are anticipated for construction of pile or caisson caps, and shallow foundations if considered. Permanent cut slopes up to about 6.5 m are also required for the widening of 9<sup>th</sup> Line.

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 existing fill and the native loose to very dense silty sand till deposits above the groundwater level 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. Below the groundwater level, the

loose to very dense silty sand till deposit may be classified as Type 4 soil, and temporary excavations should be made with side slopes no steeper than 3H:1V.

Temporary protection systems may be required to facilitate construction of the new bridge structures while maintaining traffic on the existing Highway 400 bridge and/or 9<sup>th</sup> Line. Additional investigation will need to be completed at the location of any temporary protections systems. 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 current and previous foundation investigation varied between 274.1 m (west side of site) and 272.1 m (middle portion of site). The monitoring well installed in Borehole 9-1 measured a groundwater level at about Elevation 274.1 m (about 1.5 m below ground surface) where the ground surface is near the existing Highway 400 grade. Within the existing 9<sup>th</sup> Line cut grade (west of existing Highway 400), the monitoring well installed in Borehole B4-2 measured a groundwater level between Elevation 272.1 m and 272.4 m (corresponding to ground surface and 0.3 m above ground surface). As result, in areas where the grade has been or will be cut to lower 9<sup>th</sup> Line, artesian groundwater conditions can be expected.

At this preliminary stage it is anticipated that temporary excavations for shallow foundations or pile caps will extend below the shallow groundwater table. Given the high groundwater level, depending on the extent of the excavation and groundwater level at the time of construction, dewatering will likely be required and may require wells or well point systems within the sand to silty sand deposits. The groundwater level would need to be drawn down to at least 0.5 m below the base of the excavation until the excavation has been properly backfilled. Consideration should be given to advancing temporary and/or permanent cuts along 9<sup>th</sup> Line to passively drain the groundwater prior to final grading and foundation construction. Ditching and/or counterfort drains may be required for temporary and/or permanent groundwater control in the deep cut areas along 9<sup>th</sup> Line and adjacent to the structure foundations. Depending on the founding level and if artesian conditions are present, additional dewatering / depressurization efforts may be required to facilitate construction of the foundations.

If the excavation operations are carried out in the wet season, the groundwater level could be higher (especially at the west side of Highway 400) 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 in the cut areas of the Site must be properly diverted / controlled such that the integrity of any foundation subgrade is maintained.

To reduce erosion of the permanent embankment or cut 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 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. 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 two existing boreholes and one new borehole 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 and 9<sup>th</sup> Line widening 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), approach embankments, retaining walls, and temporary protection systems. In particular, boreholes should be advanced at the abutments of the proposed E-N Ramp location as no boreholes were advanced at the location of this structure. Boreholes should be advanced below the anticipated pile tip elevations to confirm the presence and thickness of the “100-blow” soils. The deep cut area along 9<sup>th</sup> Line (west of and below Highway 400) should be investigated to confirm soil conditions and more importantly, assess the groundwater regime and presence of any artesian conditions / confined aquifers. A sufficient number of index tests should be carried out in boreholes advanced at this site, such that they meet the minimum requirements outlined in the *Guideline for MTO Foundation Engineering Services*. Consideration could be given to carrying out pressuremeter testing within the till deposits to refine soil parameters for settlement analysis.

Additionally, given that the seismic Site Class based on  $\bar{N}_{60}$  indicated that the site is Site Class D, geophysics testing should be considered and may provide a more favourable Site Class designation. 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 deep cut areas, and any retaining walls will need to be checked. When more details are known on actual loading conditions, the foundation types, sizes and geotechnical resistances will need to be checked and revised as necessary.

Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*. The existing standpipe piezometer (installed in Borehole 9-1) should be maintained operational to allow for continued monitoring of the groundwater level during

detail design and up to construction, at which time the piezometer will need to be decommissioned in accordance with Ontario Regulation 903 (as amended). Additional piezometers (particularly on the east side near the E-N Ramp bridge abutments, and on the west side of the site to assess potential artesian conditions where significant cuts are anticipated) should be installed near the proposed foundation elements to provide additional information for assessment of dewatering requirements. A hydrogeological assessment including in-situ testing (e.g. slug tests or pump tests) should be carried out in monitoring wells installed in the upper silty sand till deposit to further assess groundwater impacts and quantify the hydraulic conductivity of the deposit for the design of dewatering / depressurization systems.

## 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.

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### ASTM International

- |            |   |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils. |
| ASTM D1143 | Standard Test Methods for Deep Foundation Elements Under Static Axial Compressive Load  |

### Canadian Standards Association (CSA):

- CAN/CSA A23.1-19 Concrete Materials and Methods of Concrete Construction

### Commercial Software:

Settle3 (Version 5.012) by Rocscience Inc.

Slide2 (Version 9.020) by Rocscience Inc.

### Ontario Provisional Standard Drawing:

- |               |   |
|---------------|---|
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario  |
| OPSD 3101.150 | Walls, Abutments, Backfill, Minimum Granular Requirements |
| OPSD 3190.100 | Walls, Retaining and Abutment, Wall Drain                 |
| OPSD 3121.150 | Walls, Retaining, Backfill, Minimum Granular Requirements |
| OPSD 3000.100 | Foundation, Piles, Steel H-Pile, Driving Shoe             |
| OPSD 3001.100 | Foundation, Piles, Steel Tube Pile, Driving Shoe          |

### Ontario Provincial Standard Specifications (OPSS)

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

OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 803	Construction Specification for Vegetative Cover
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.PROV 902	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 #2020-01, Material Engineering and Research Office (MERO), March 23, 2020.

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

## TABLES

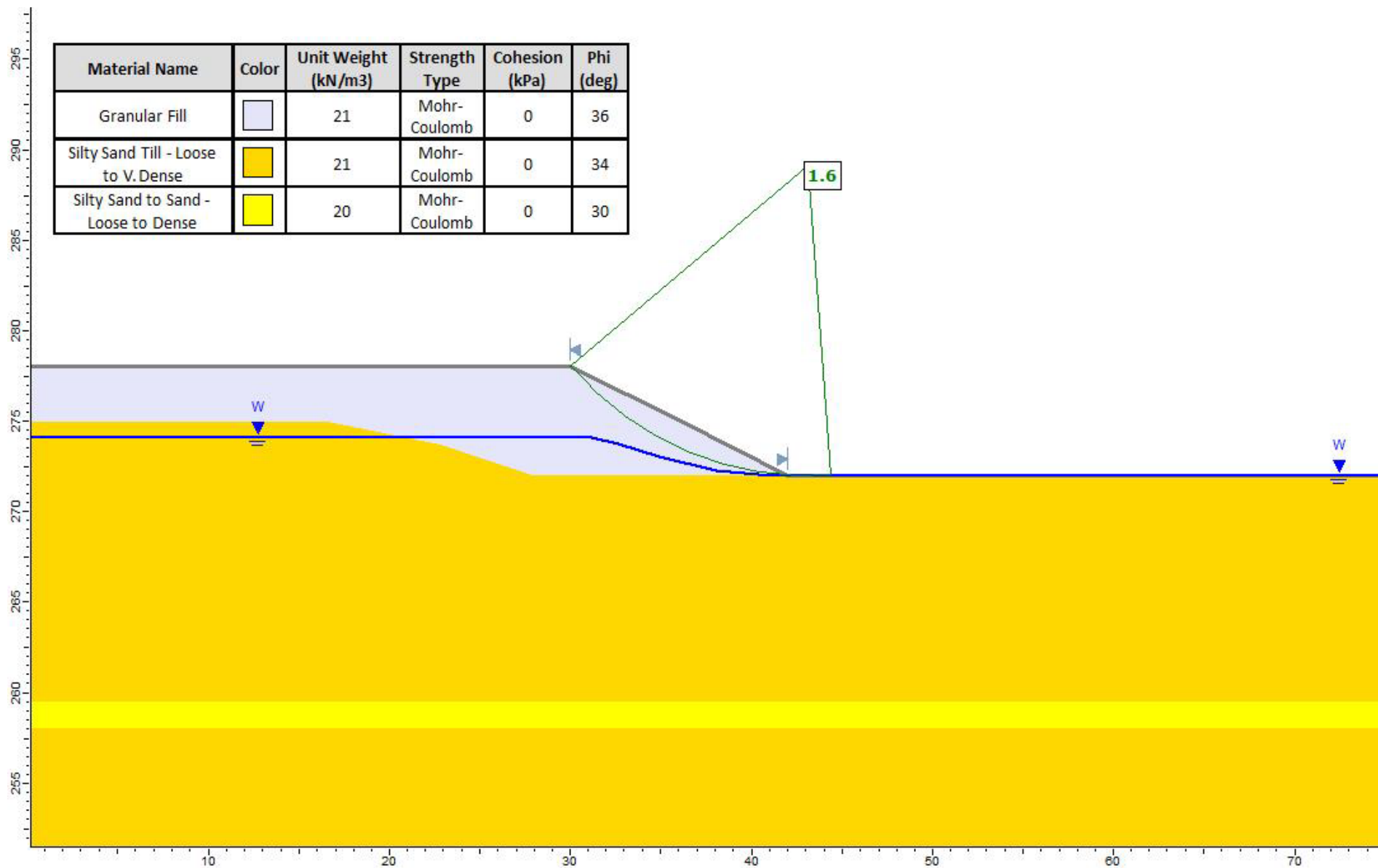
Table 1: Comparison of Foundation Alternatives – Bradford Bypass / Highway 400 Interchange Structures Over 9<sup>th</sup> Line

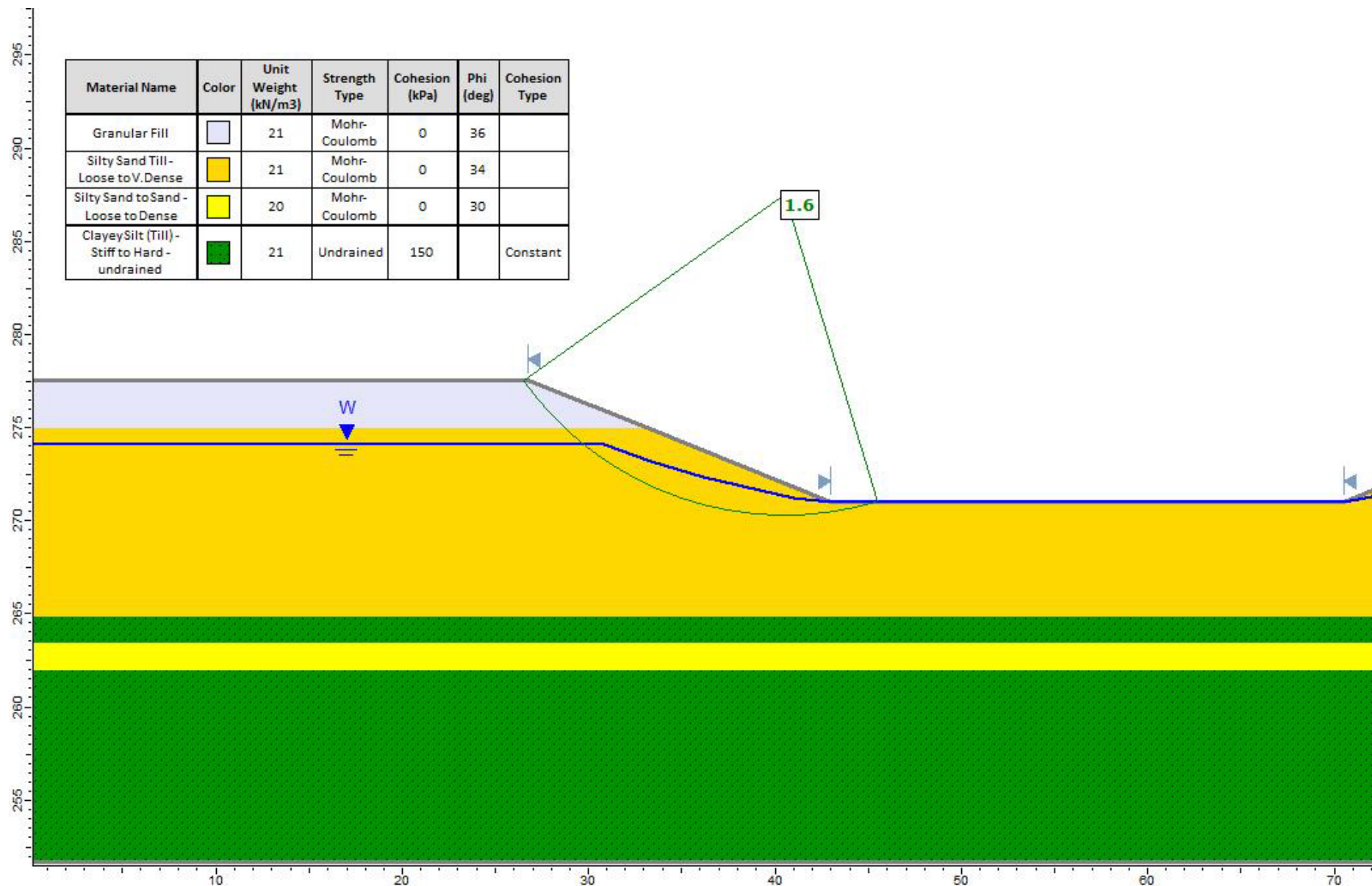
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread footings founded on native compact to very dense silty sand till	<ul style="list-style-type: none"><li>Feasible for all foundation elements</li></ul>	<ul style="list-style-type: none"><li>Conventional construction.</li><li>Relatively competent soils may provide adequate geotechnical resistances.</li></ul>	<ul style="list-style-type: none"><li>Subexcavation up to 1.5 m bgs on east side and up to 8 m bgs on west side anticipated for closed abutments; temporary protection systems may be required to limit footprint and control stability / unbalanced hydrostatic pressures within Highway 400 and 9<sup>th</sup> Line.</li><li>Will require advanced dewatering (possibly extensive) to allow for excavation and construction of footings in dry conditions and maintain stable foundation subgrade on west side of site.</li><li>Low geotechnical resistance compared to deep foundations.</li><li>Less competent near surface soils (presence of compressible soils) may exist at actual abutment locations, particularly at the E-N Ramp structure where no information is available.</li><li>Does not allow for conventional integral abutment design at abutments.</li></ul>	<ul style="list-style-type: none"><li>Lower cost than deep foundations</li><li>Costs for subexcavation and dewatering need to be considered.</li></ul>	<ul style="list-style-type: none"><li>Risk of excess total and differential settlement due to anticipated high foundation loads.</li><li>Variable and less competent soil conditions near founding level at actual foundation element footprint may decrease preliminary geotechnical resistance values and may make this option not feasible.</li><li>Risk of variable soil conditions and increased subexcavation depth of unsuitable soils (e.g. compressible soils or organics) at the E-N Ramp location east of Highway 400.</li><li>Artesian conditions, if present, will increase risk of disturbing foundation during construction and may require mitigation measures or make this option not feasible.</li></ul>
Spread footings founded on a compacted granular pad over native compact to very dense silty sand till	<ul style="list-style-type: none"><li>Feasible for all foundation elements, but less practical for closed-type abutments</li></ul>	<ul style="list-style-type: none"><li>Conventional construction.</li><li>Granular pad can be constructed within approach embankment for abutments.</li><li>Founding level can be adjusted within approach embankment.</li><li>Reduces depth of excavation and height of abutment wall stems, and reduces dewatering efforts.</li><li>Increases geotechnical resistance compared to shallow foundations on native deposits.</li></ul>	<ul style="list-style-type: none"><li>May require some dewatering to allow for excavation and construction of granular pad in dry conditions and maintain stable foundation subgrade.</li><li>Low geotechnical resistance compared to deep foundations.</li><li>Less competent near surface soils (presence of and thicker compressible soils) may exist at actual abutment locations, particularly at the E-N Ramp structure.</li><li>Does not allow for conventional integral abutment design at abutments.</li></ul>	<ul style="list-style-type: none"><li>Lower cost than deep foundations</li><li>Lower relative cost for subexcavation and dewatering than shallow footing on native soil.</li></ul>	<ul style="list-style-type: none"><li>Risk of excess total and differential settlement due to anticipated high foundation loads.</li><li>Variable and less competent soil conditions near founding level at actual foundation element footprint may decrease preliminary geotechnical resistance values and may this option not feasible.</li><li>Risk of variable soil conditions and increased subexcavation depth of unsuitable soils (e.g. compressible soils or organics) at the E-N Ramp location east of Highway 400.</li><li>Lower risk of impact if artesian conditions are encountered.</li></ul>
Driven Steel Piles	<ul style="list-style-type: none"><li>Feasible for all foundation elements</li></ul>	<ul style="list-style-type: none"><li>Conventional construction methods for driven H-pile or tube pile foundations.</li><li>Higher axial resistances compared to shallow footings.</li><li>Larger H-piles or tube piles can be considered to increase axial resistance.</li><li>Perched abutments can be considered to reduce dewatering / subexcavation for pile caps.</li></ul>	<ul style="list-style-type: none"><li>Subexcavation up to 1.5 m bgs on east side and up to 8 m bgs on west side anticipated for pile cap construction, unless “perched” within approach embankments. As a result, temporary protection systems may be required to limit footprint and control stability / unbalanced hydrostatic pressures within Highway 400 and 9<sup>th</sup> Line</li><li>Will require advanced dewatering measures (possible extensive) to allow for excavation and construction of pile caps on west side unless “perched” in embankment fill.</li></ul>	<ul style="list-style-type: none"><li>Lower relative cost than drilled shafts (caissons)</li><li>Higher cost than shallow footings but reduced dewatering / excavation costs if “perched” pile caps are designed.</li></ul>	<ul style="list-style-type: none"><li>If encountered during detail design, variable soil conditions (deeper soft or very loose or unsuitable soil deposits) closer to foundation locations will have less impact compared to shallow foundations.</li><li>Low risk of damage to pile due to driving through hard / very dense deposits possibly containing gravel pockets, cobbles / boulders. Driving shoes and/or thicker pile section, and possible pre-augering may be required at some locations.</li><li>Presence of and thickness of “100-blow” soil needs to be confirmed at foundation elements during detail design and could lead to longer piles.</li></ul>
Drilled Shafts (Caissons)	<ul style="list-style-type: none"><li>Feasible for all foundation elements</li></ul>	<ul style="list-style-type: none"><li>Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements.</li><li>Larger diameter caissons can be considered to increase axial resistance.</li><li>May be designed to eliminate caisson caps and temporary excavations / protection systems as the caisson could</li></ul>	<ul style="list-style-type: none"><li>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).</li><li>Generation, containment and disposal of soil cuttings / slurry during caisson advancement.</li></ul>	<ul style="list-style-type: none"><li>Higher relative cost than driven piles</li><li>Higher cost than shallow footings but reduced dewatering / excavation if cast continuously with</li></ul>	<ul style="list-style-type: none"><li>If encountered during detail design, variable soil conditions (deeper soft or very loose or unsuitable soil deposits) closer to foundation locations will have less impact compared to shallow foundations.</li><li>Challenges associated with inspection of shaft walls and base due to the need for liners, slurry, and tremie concrete methods may require the use of SID and SQUID to inspect the shaft and base.</li></ul>

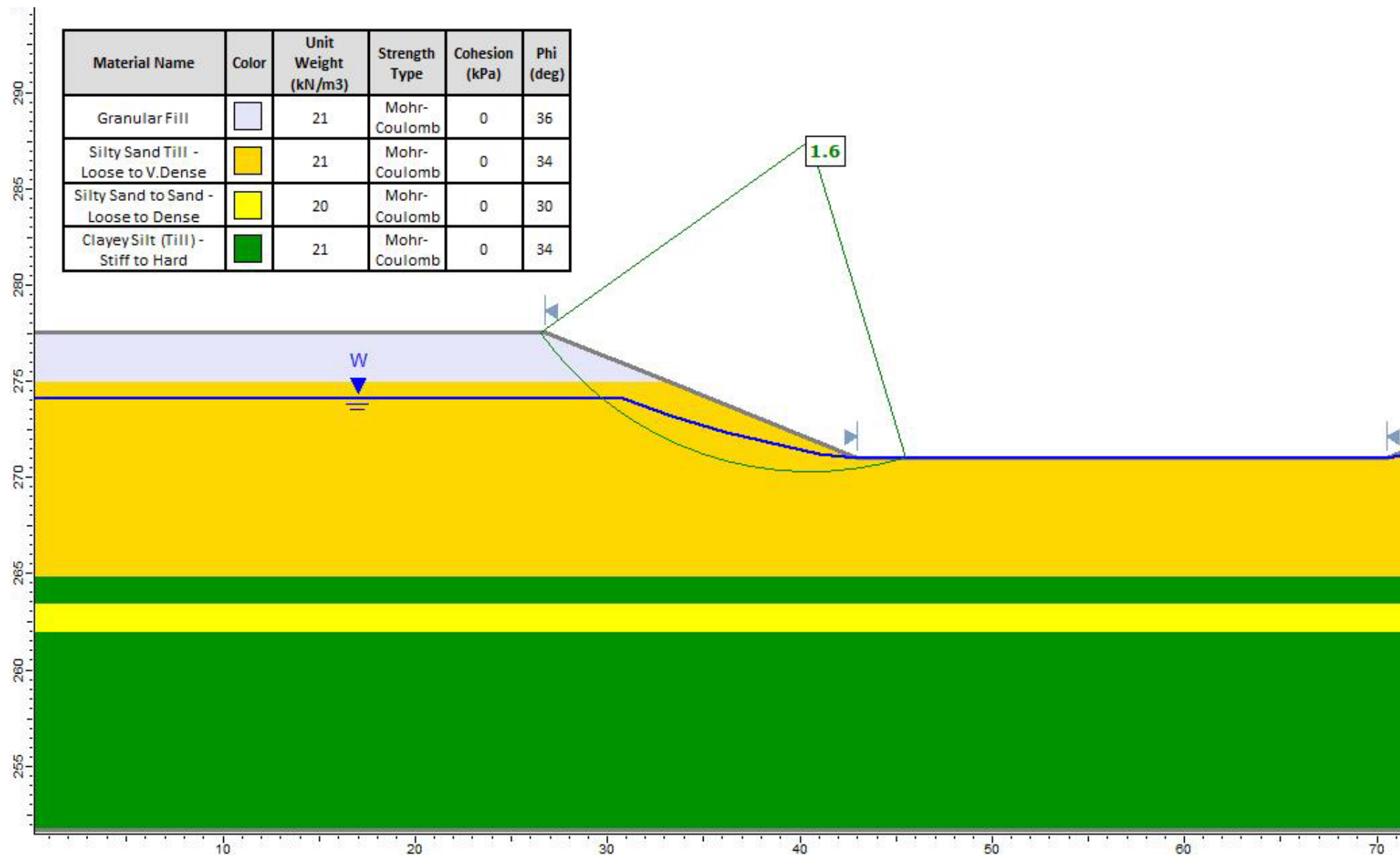
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
		<p>be cast continuously with structural columns to underside of superstructure. Associated subexcavation and dewatering efforts reduced compared to shallow foundation and pile cap construction.</p> <ul style="list-style-type: none"><li>Contractor could consider contiguous caisson wall design at west side to eliminate subexcavation and dewatering concerns related to 9<sup>th</sup> Line grade lowering.</li></ul>	<ul style="list-style-type: none"><li>Does not allow for conventional integral abutment design.</li></ul>	<p>columns or “perched” pile caps.</p>	<ul style="list-style-type: none"><li>Artesian groundwater conditions, if present, may pose challenges during excavation and/or concrete placement. Deep dewatering / depressurization measures may be required.</li><li>Low risk of excavating through potential gravel pockets, cobbles / boulders based on preliminary investigation.</li><li>Presence of and thickness of “100-blow” soil needs to be confirmed at foundation elements during detail design and could lead to longer caissons</li></ul>



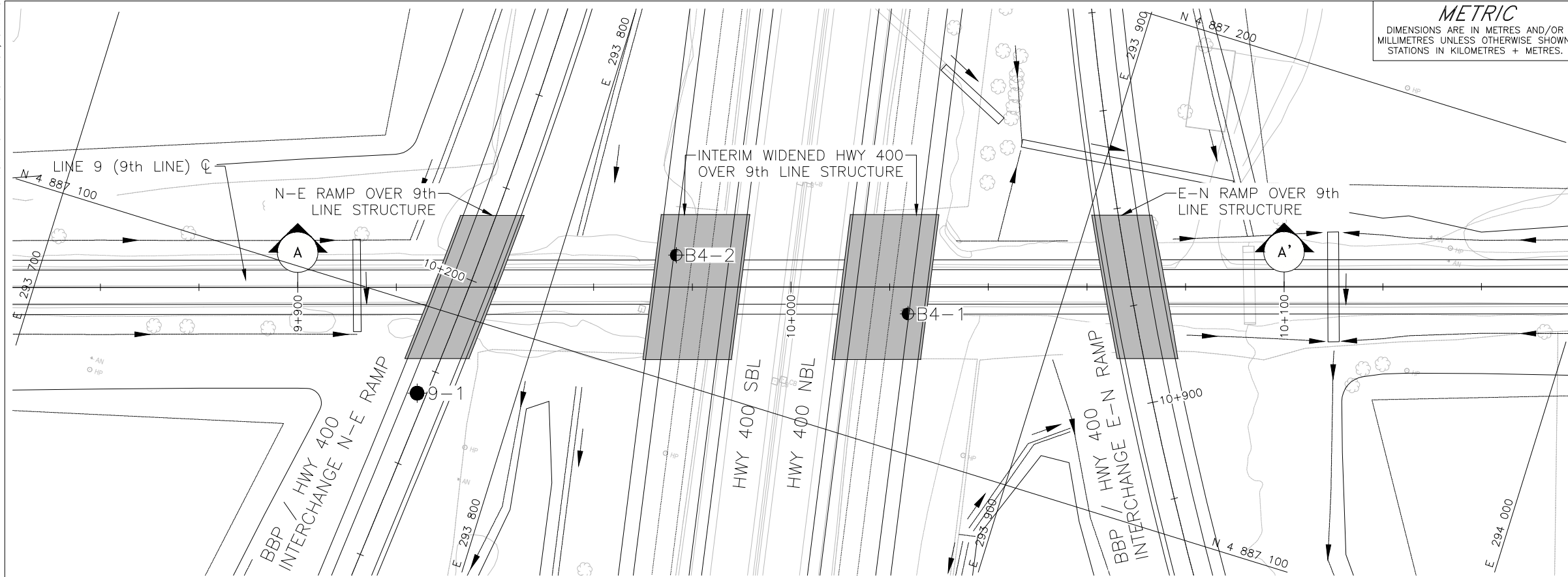
## FIGURES







## DRAWINGS



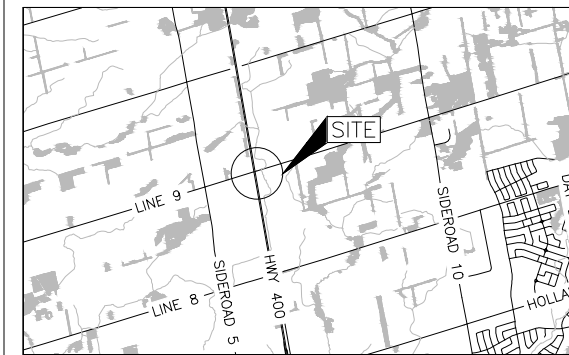
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No.

BRADFORD BYPASS  
BRADFORD BYPASS/HWY 400 INTERCHANGE RAMP  
STRUCTURES OVER 9TH LINE  
BOREHOLE LOCATIONS AND SOIL  
STRATA

10/12/2023

SHEET



KEY PLAN  
SCALE  
1 0 1 2 km

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Artesian WL in piezometer, measured on June, 19, 2001
- WL in piezometer, measured on Dec. 10, 2021
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
9-1	275.6	4887082.5	293780.6
B4-2	272.1	4887124.8	293822.5
B4-1	271.7	4887127.3	293870.9

**NOTES**

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

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

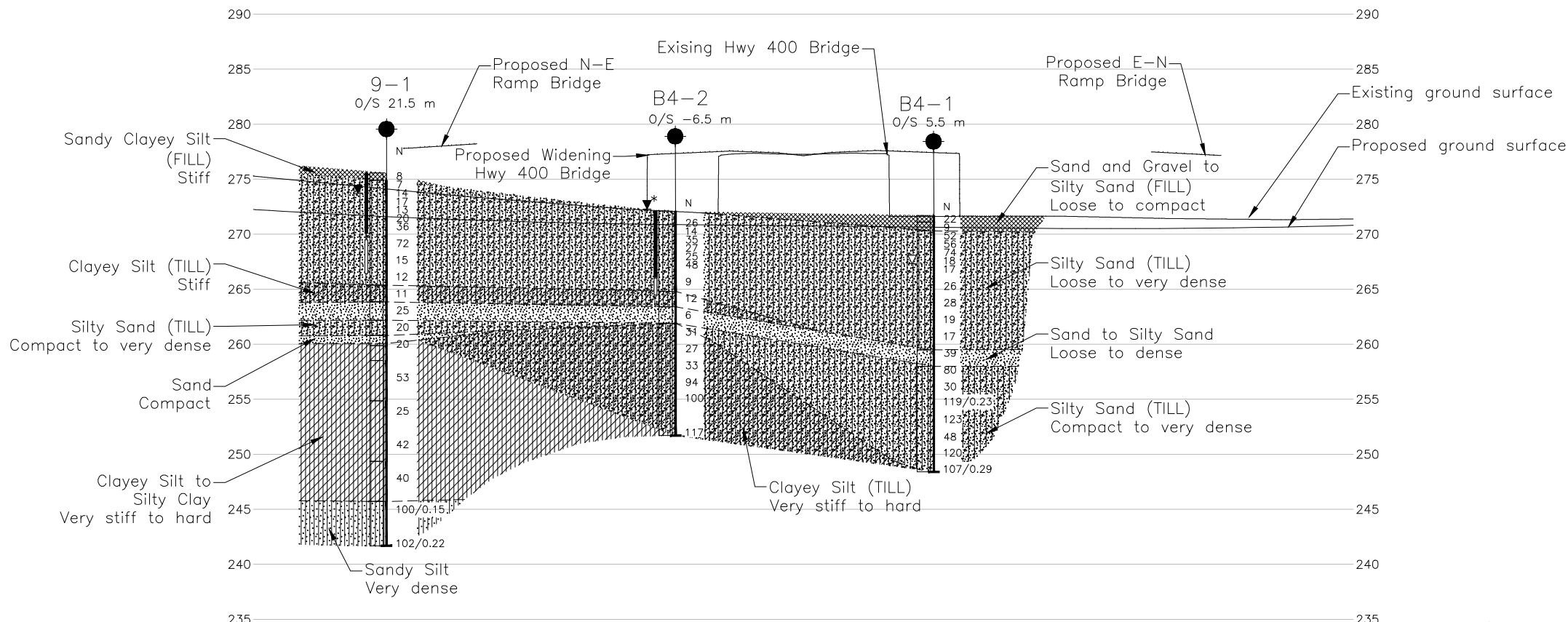
**REFERENCE**

Base plans provided in digital format by Aecom, drawing file no. X-Base\_Bradford Bypass.dwg and BRADFORD BY-PASS OG\_Combined.xml, received January 11, 2022.

Horizontal alignment provided in digital format by Aecom, drawing file no. X-60636190-C-DES-All Alignments.xml, 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.

Design plan provided in digital format by Aecom, drawing file no. BBP Mainline Plan and Profile\_To WSP\_230705.dwg, received July 5, 2023.



A-A' PROFILE 9TH LINE



NO.	DATE	BY	REVISION
Geocres No. 31D-834			
HWY.	PROJECT NO. 19136074	DIST.	
SUBM'D. MCK	CHKD. MCK	DATE: 10/12/2023	SITE:
DRAWN: DD/SA	CHKD. MCK	APPD. KJB	DWG. 1

**APPENDIX A**

**Borehole Records and  
Geotechnical Laboratory Test  
Results (Golder 2001 Investigation)**



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

### II. 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.) drive open sampler for a distance of 300 mm (12 in.)

#### (b) Cohesive Soils

Consistency	$c_u, s_u$ kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

#### Piezo-Cone Penetration Test (CPT)

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
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	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note: 1** Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

S:\FINALDATA\ABBREV\2000\LOFA-D00.DOC

## LIST OF SYMBOLS

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

### I GENERAL

$\pi$	= 3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10} x$ or $\log x$ ,	logarithm of x to base 10
$g$	acceleration due to gravity
$t$	time
$F$	factor of safety
$V$	volume
$W$	weight

### II STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
$u$	porewater pressure
$E$	modulus of deformation
$G$	shear modulus of deformation
$K$	bulk modulus of compressibility

### III SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
$e$	void ratio
$n$	porosity
$S$	degree of saturation
*	Density symbol is $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

#### (a) Index Properties (con't.)

$w$	water content
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	plasticity Index $= (w_l - w_p)$
$w_s$	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)

#### (c) 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

#### (d) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (overconsolidated range)
$C_s$	swelling index
$C_\alpha$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
$U$	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

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

ON MOT 001-1151.GPJ ON MOT.GDT 7/12/01

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

PROJECT 001-1151				RECORD OF BOREHOLE No B4-1				2 OF 2				METRIC					
W.P. 40-00-00				LOCATION N 4887127.3; E 293870.9				ORIGINATED BY SB									
DIST SW HWY 400				BOREHOLE TYPE 108mm Diameter Solid Stem Augers				COMPILED BY LCC									
DATUM Geodetic				DATE December 21-22, 2000				CHECKED BY ASP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N* VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
248.4	Silty Sand, trace to some gravel, trace to some clay (Till) Dense to very dense Grey Moist		17	SS	48												
							251										
							250										
			18	SS	120												
			19	SS	107												
23.3	END OF BOREHOLE																
	Note: Water level in open borehole on completion of drilling at 4.0m depth (Elev.267.7m).																

ON\_MOT 001-1151.GPJ ON\_MOT.GDT 7/12/01

PROJECT 001-1151

# RECORD OF BOREHOLE No B4-2

1 OF 2

METRIC

W.P. 40-00-00

LOCATION N 4887124.8; E 293822.5

ORIGINATED BY GPD

DIST SW HWY 400

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

COMPILED BY LCC

DATUM Geodetic

DATE December 19-21, 2000

CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
272.1	GROUND SURFACE													
0.0	Sand and Silt, trace clay and gravel (Till) Loose to dense Grey Moist		1	SS	26		272							
			2	SS	14		271							
			3	SS	35		270							
			4	SS	27		269							
			5	SS	25		268							
			6	SS	47		267							
			7	SS	9		266							
264.8							265							
7.3	Clayey Silt, trace sand and gravel (Till) Stiff Grey Moist		8	SS	12		264							
263.4							263							
8.7	Silty Sand, trace clay and gravel Loose Grey Moist		9	SS	6		262							
261.9							261							
10.2	Clayey Silt, trace to some sand, trace gravel (Till) Very stiff to hard Grey Moist		10	SS	31		260							
			11	SS	27		259							
			12	SS	33		258							
			13	SS	94		257							
			14	SS	100		256							
							255							
							254							
							253							

Continued Next Page

+<sup>3</sup> X<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ON\_MOT\_001-1151.GPJ ON\_MOT.GDT 7/12/01

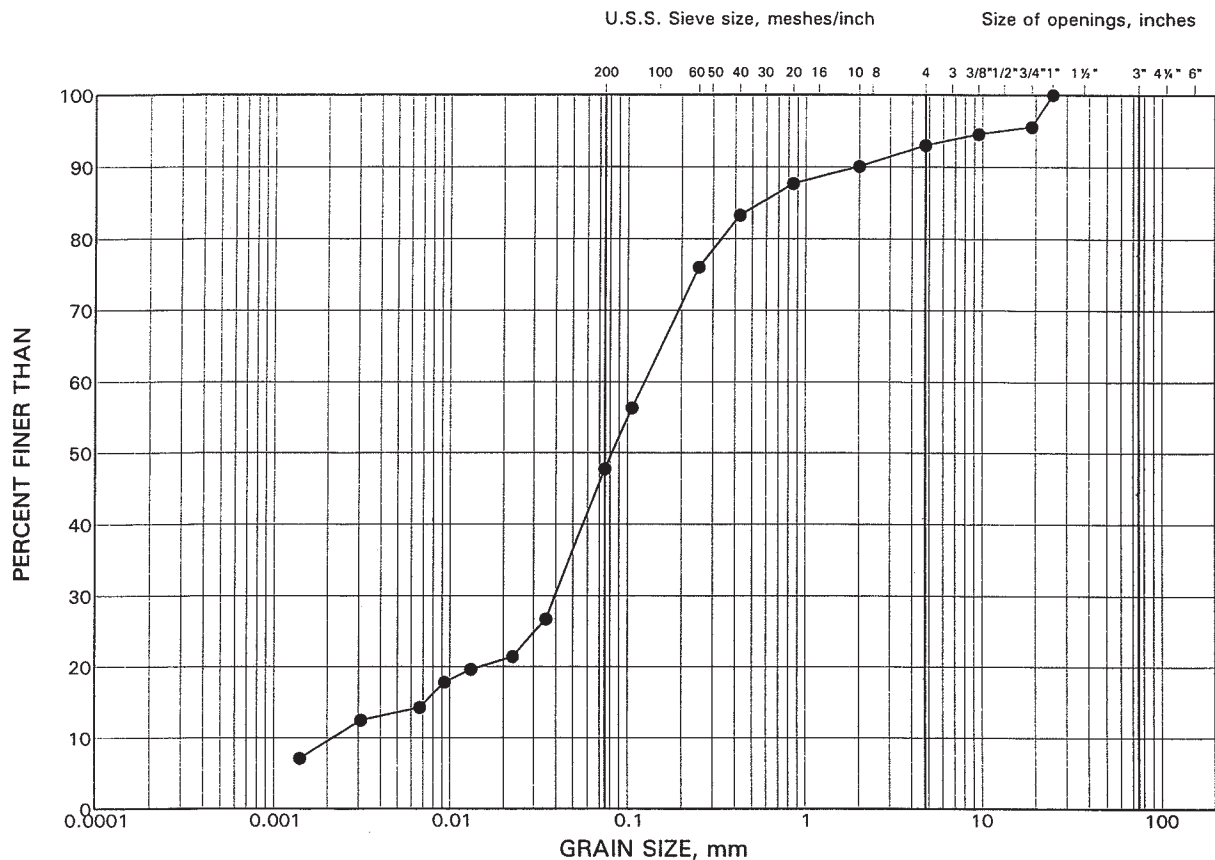
PROJECT 001-1151			RECORD OF BOREHOLE No B4-2				2 OF 2		METRIC								
W.P. 40-00-00			LOCATION N 4887124.8; E 293822.5				ORIGINATED BY GPD										
DIST SW HWY 400			BOREHOLE TYPE 108mm Diameter Solid Stem Augers				COMPILED BY LCC										
DATUM Geodetic			DATE December 19-21, 2000				CHECKED BY ASP										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	— CONTINUED FROM PREVIOUS PAGE —																
251.7			15	SS	117		252										
20.4	END OF BOREHOLE  Notes: 1. Water level in open borehole on completion of drilling at 0.3m depth (Elev.271.8m) 2. The piezometer was installed in a second borehole, drilled 1m west of this deep borehole. 3. The water level in the piezometer was frozen at 0.3m above ground surface (Elev.272.4m) on January 18, 2001, and at ground surface (Elev.272.1m) on March 20, 2001. The water level in the piezometer was 0.2m above ground surface (Elev.272.3m) on June 19, 2001.																

ON\_MOT 001-1151.GPJ ON\_MOT.GDT 7/12/01

# GRAIN SIZE DISTRIBUTION TEST RESULT

~~Sand and Silt Till~~  
Silty Sand

FIGURE 1  
A1



SILT AND CLAY SIZES				FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED				SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B4-1	3	269.6



**APPENDIX B**

**Borehole Records (WSP  
Golder 2021  
Investigation)**

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

## MINISTRY OF TRANSPORTATION, ONTARIO

### PARTICLE SIZES OF CONSTITUENTS

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

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

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (i.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<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q<sub>t</sub>), porewater pressure (u) and sleeve friction (f<sub>s</sub>) are recorded electronically at 25 mm penetration intervals.

#### Dynamic Cone Penetration Resistance (DCPT); N<sub>d</sub>:

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

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

### COARSE-GRAINED SOILS

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

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

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

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

### FINE-GRAINED SOILS

#### Consistency

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

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

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

### Field Moisture Condition

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

# LIST OF SYMBOLS

## MINISTRY OF TRANSPORTATION, ONTARIO

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

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

### III. SOIL PROPERTIES

#### (a) Index Properties

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

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

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

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

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

Notes: 1  
2

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

PROJECT 19136074

## RECORD OF BOREHOLE No. 9-1

Sheet 1 of 4

METRIC

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

LOCATION N 4887082.5; E 293780.6 NAD83 / MTM Zone 10 (LAT. 44.123655; LONG. -79.637689)

ORIGINATED BY SS

DIST Central HWY BBP - 9th Line

BOREHOLE TYPE Hollow Stem Auger, Mud Rotary with Casing

COMPILED BY MCK

DATUM CGVD28 Surface Elevation:275.6 m

DATE Apr 12, 2021 - Apr 13, 2021

CHECKED BY MTI/KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m <sup>3</sup>	GR	SA	SI	CL	REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>						
0.0	Sandy CLAYEY SILT (CL), trace gravel, trace organics, (FILL) Stiff Brown Moist		1	SS	8		275	20	40	60	80	100	20	40	60						
274.9																					
0.7	SILTY SAND (SM), trace clay, trace gravel to gravelly, (TILL) Loose to very dense Brown to Grey, iron oxide staining Moist to wet		2	SS	7		274														
			3	SS	14		273														
			4	SS	17		272														
			5	SS	13		271														
			6	SS	20		270														
			7	SS	36		269														
			8	SS	72		268														
			9	SS	15		267														
			10	SS	12		266														

Continued on Next Page

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

PROJECT	19136074	RECORD OF BOREHOLE	No. 9-1	Sheet 2 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887082.5; E 293780.6 NAD83 / MTM Zone 10 (LAT. 44.123655; LONG. -79.637689)	ORIGINATED BY	SS
DIST	Central HWY BBP - 9th Line	BOREHOLE TYPE	Hollow Stem Auger, Mud Rotary with Casing	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:275.6 m	DATE	Apr 12, 2021 - Apr 13, 2021	CHECKED BY	MTI/KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined					W <sub>p</sub>	W	W <sub>L</sub>						
													NP Nonplastic								
265.4 10.2	SILTY SAND (SM), trace clay, trace gravel to gravelly, (TILL) Loose to very dense Brown to Grey, iron oxide staining Moist to wet CLAYEY SILT (CL), some sand, trace gravel, (TILL) Stiff Grey Moist - 11.0 m: Sand seam encountered within sample		11	SS	11	265											0	15	46	39	
263.8 11.7	SAND (SP), trace gravel Compact Grey Moist		12	SS	25	264															
262.2 13.4	SILTY SAND (SM), trace clay, some gravel (TILL) Compact Grey Moist		13	SS	20	263															
260.8 14.8	SAND (SP), trace gravel Compact Grey Wet		14 A	SS	20	262															
259.9 15.7	Sandy CLAYEY SILT (CL), trace gravel Very Stiff Grey Wet  - 16.8 to 18.3 m: Slow drilling.		14 B			261															
258.5 17.1	CLAYEY SILT (CL), trace sand, trace gravel; Hard Grey Wet		15	SS	53	260															
						259															
						258															
						257															
						256															

Continued on Next Page

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

PROJECT	19136074	RECORD OF BOREHOLE	No. 9-1	Sheet 3 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887082.5; E 293780.6 NAD83 / MTM Zone 10 (LAT. 44.123655; LONG. -79.637689)	ORIGINATED BY	SS
DIST	Central HWY BBP - 9th Line	BOREHOLE TYPE	Hollow Stem Auger, Mud Rotary with Casing	COMPILED BY	MCK
DATUM	CGVD28 Surface Elevation:275.6 m	DATE	Apr 12, 2021 - Apr 13, 2021	CHECKED BY	MTI/KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100					20 40 60								
254.9	CLAYEY SILT (CL), trace sand, trace gravel; Hard Grey Wet						255														
20.7	SILTY CLAY (CI) Very stiff to hard Grey Moist to wet		16	SS	25		254										0	0	44	56	
							253														
							252														
							251														
			17	SS	42		250														
249.4	CLAYEY SILT (CL) containing laminations Hard Moist Grey						249														
26.2							248														
	- 28.0 m: contains silt seams/layers		18	SS	40		247														
							246														
245.7	Sandy SILT (ML), trace clay Very Dense Grey																				
29.9	Wet																				

Continued on Next Page

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



## METRIC

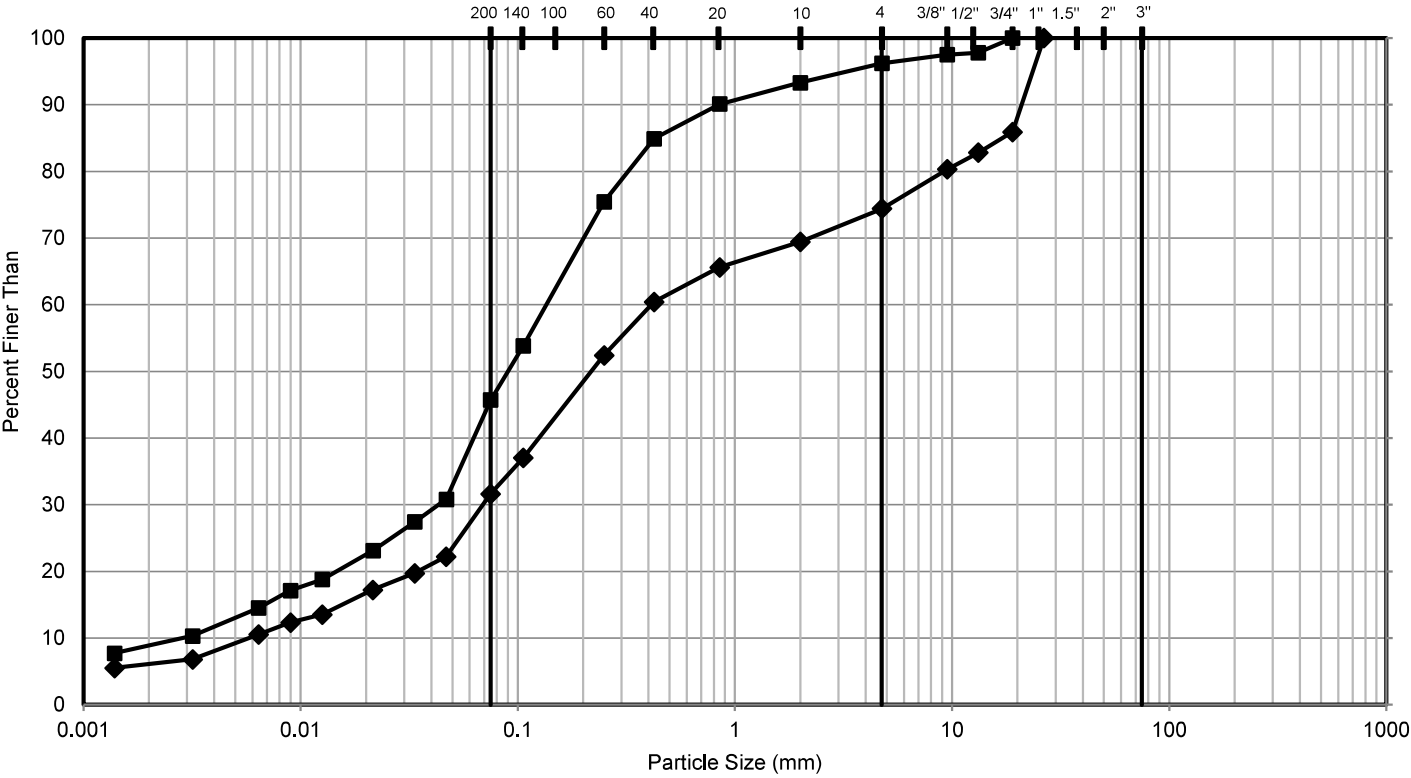
CHECKED BY MTI/KJB

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

**APPENDIX C**

**Geotechnical Laboratory Test  
Results (WSP Golder 2021  
Investigation)**

GRAIN SIZE DISTRIBUTION



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

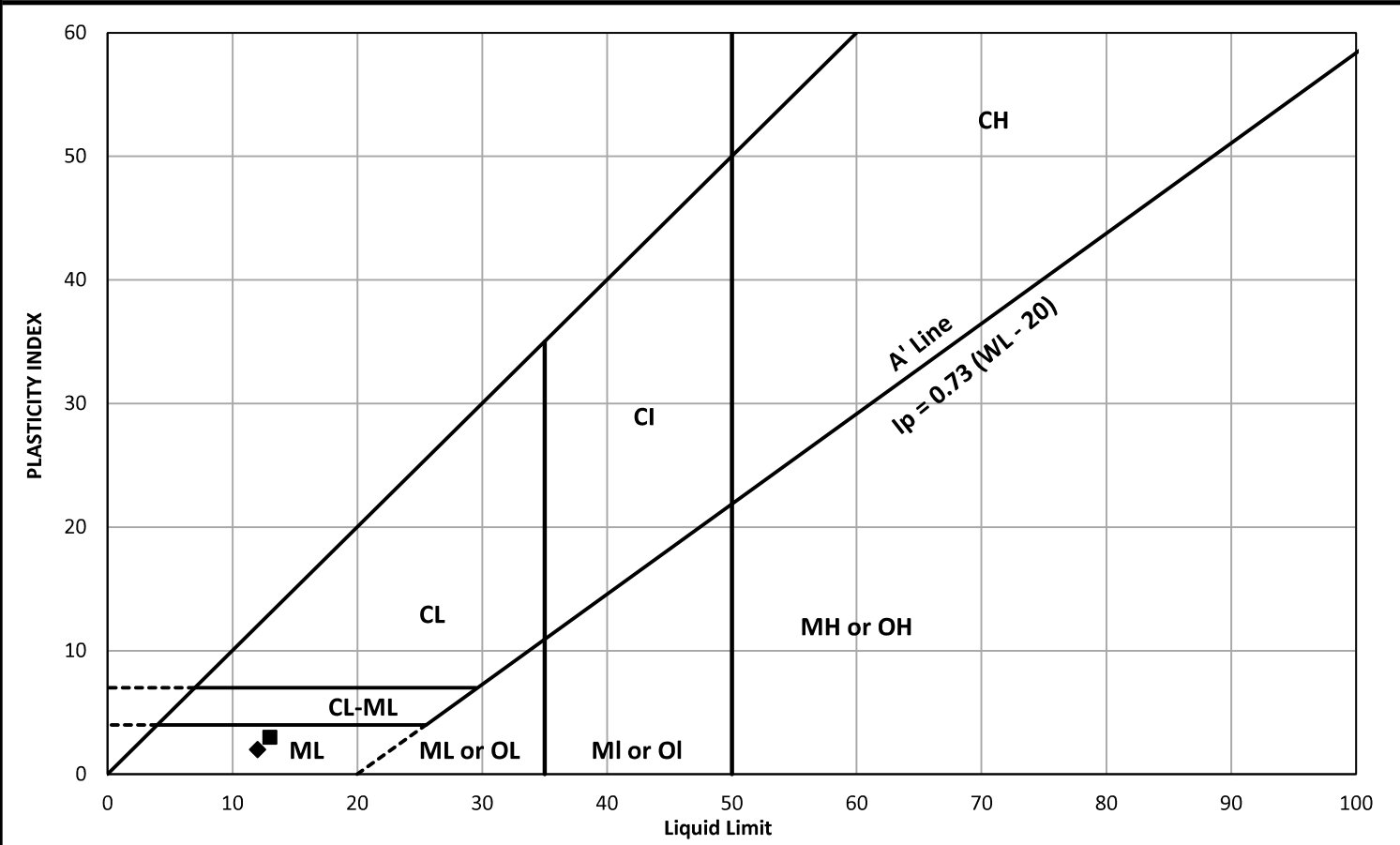
Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	9-1	4	2.3 - 2.9	273.3 to 272.7
◆	9-1	8	6.1 - 6.7	269.5 to 268.9

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - 9th Line	
CONSULTANT	YYYY-MM-DD	2023-08-09	
	DESIGNED	MCK	
	PREPARED	MCK	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		UPPER SILTY SAND (SM) TILL	
		PROJECT NO.	CONTROL
		19136074	1000
		REV.	FIGURE
		0	C1



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PLASTICITY CHART

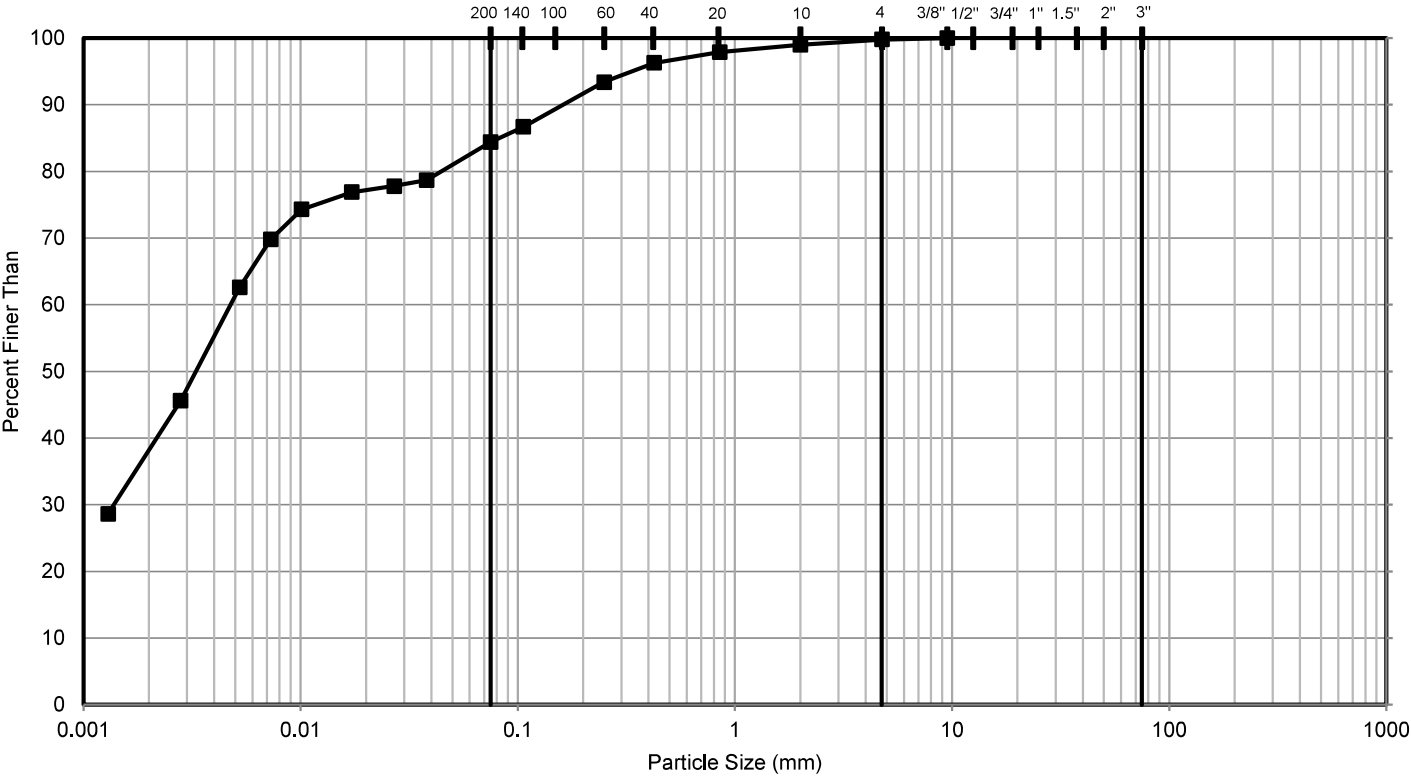


Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	9-1	4	273.3 to 272.7	9.8	13	10	3
◆	9-1	8	269.5 to 268.9	6.2	12	10	2

CLIENT			
AECOM / MTO			
	CONSULTANT	YYYY-MM-DD	2023-08-09
	DESIGNED		MCK
	PREPARED		MCK
	REVIEWED		KJB
	APPROVED		KJB


PROJECT			
Bradford Bypass - 9th Line			
TITLE			
UPPER SILTY SAND (SM) TILL			
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	C2

GRAIN SIZE DISTRIBUTION



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

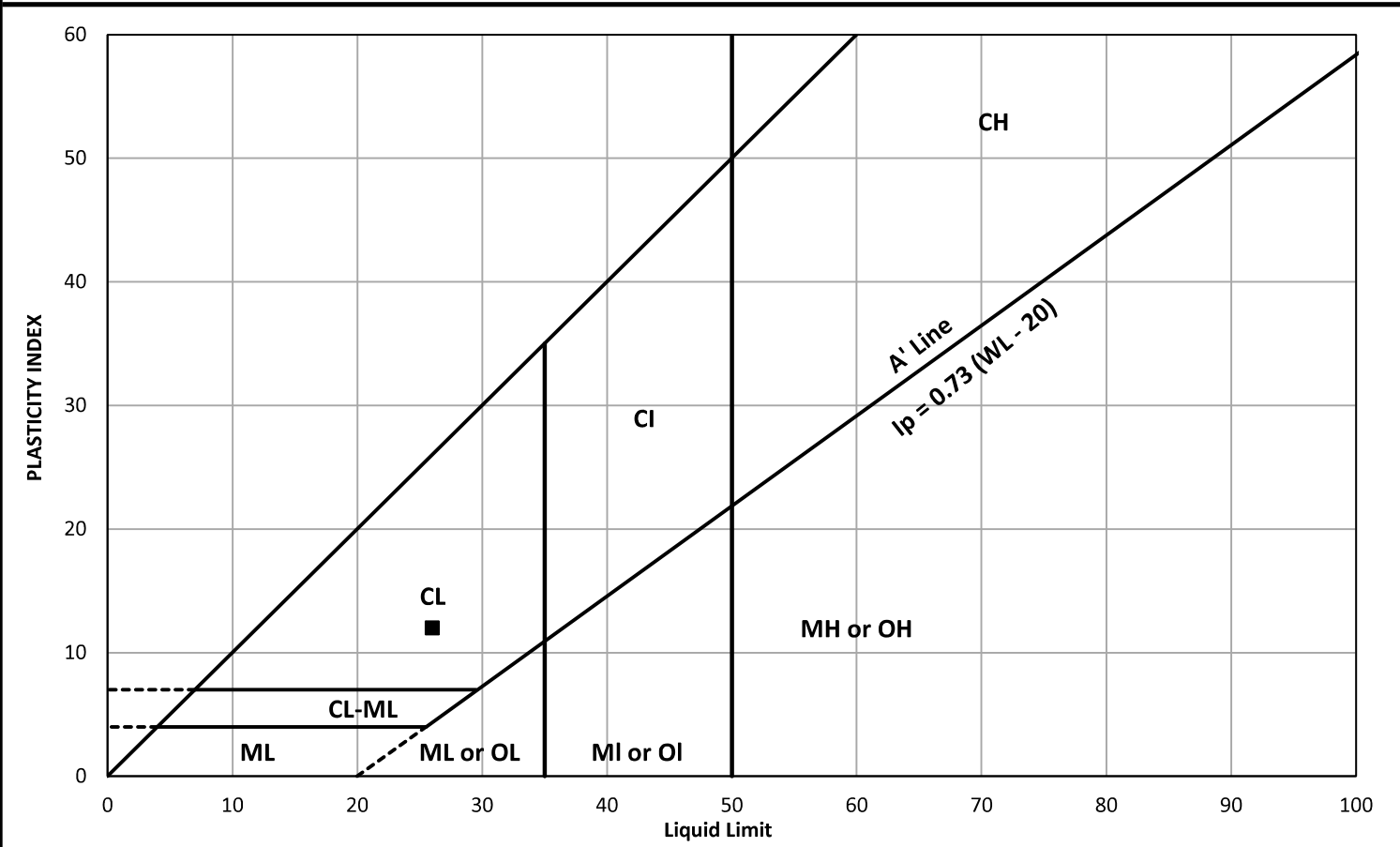
Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	9-1	11	10.7 - 11.3	264.9 to 264.3

CLIENT		PROJECT			
AECOM / MTO		Bradford Bypass - 9th Line			
CONSULTANT	YYYY-MM-DD	2023-08-09			TITLE
	DESIGNED	MCK			
	PREPARED	MCK			
	REVIEWED	KJB			
	APPROVED	KJB			
 <b>GOLDER</b>		UPPER CLAYEY SILT (CL) TILL			
		PROJECT NO.	CONTROL	REV.	FIGURE
		19136074	1000	0	C3



PATH: https://wsponline-my.sharepoint.com/personal/madison\_kennedy\_wsp\_com/Documents/Documents/19136074 - Bradford Bypass/9th Line | FILE NAME: Alterberg Output MTO for Bradford Bypass -BH 9-1.xlsm

PLASTICITY CHART



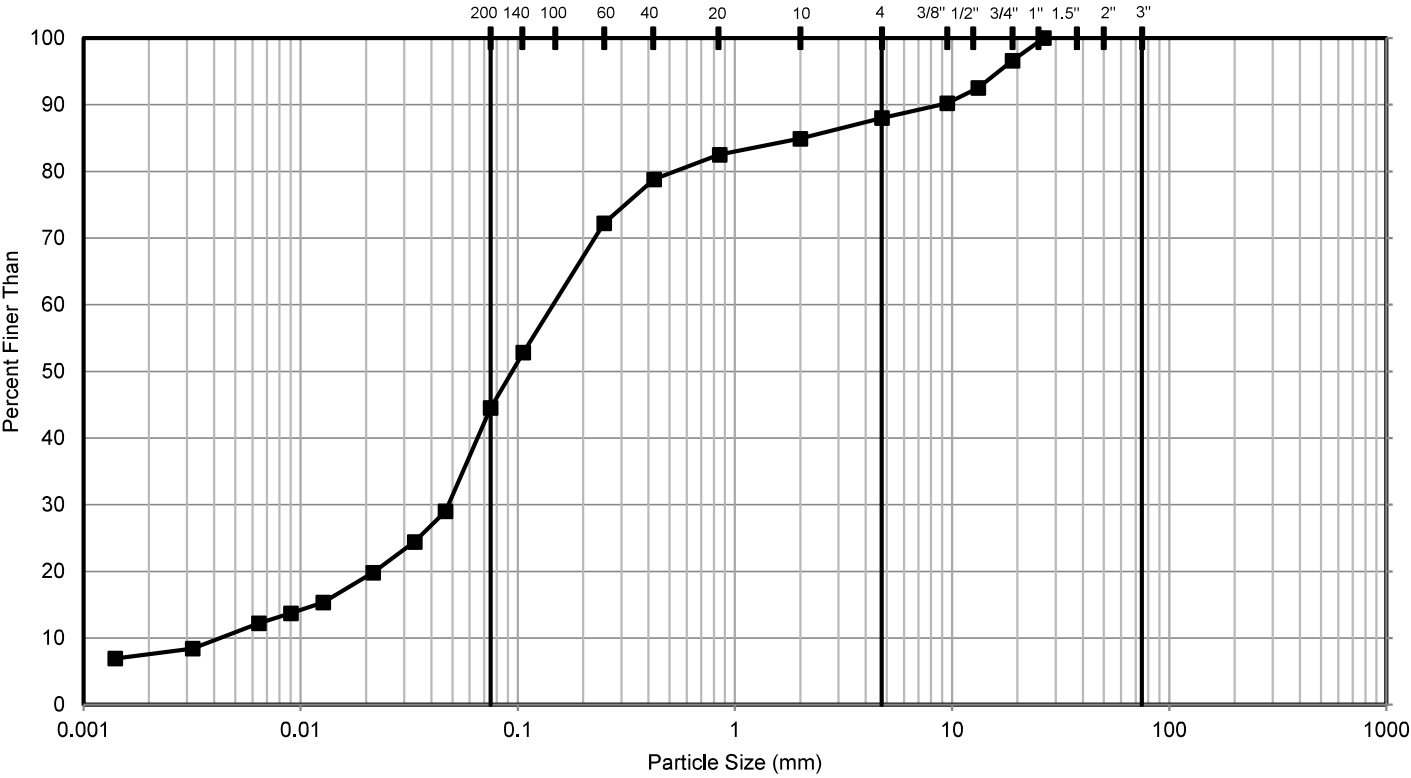
Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	9-1	11	264.9 to 264.3	21.1	26	14	12

CLIENT		
AECOM / MTO		
CONSULTANT	YYYY-MM-DD	2023-08-09
	DESIGNED	MCK
	PREPARED	MCK
	REVIEWED	KJB
	APPROVED	KJB



PROJECT			
Bradford Bypass - 9th Line			
TITLE			
UPPER CLAYEY SILT (CL) TILL			
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	0	C4

GRAIN SIZE DISTRIBUTION



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

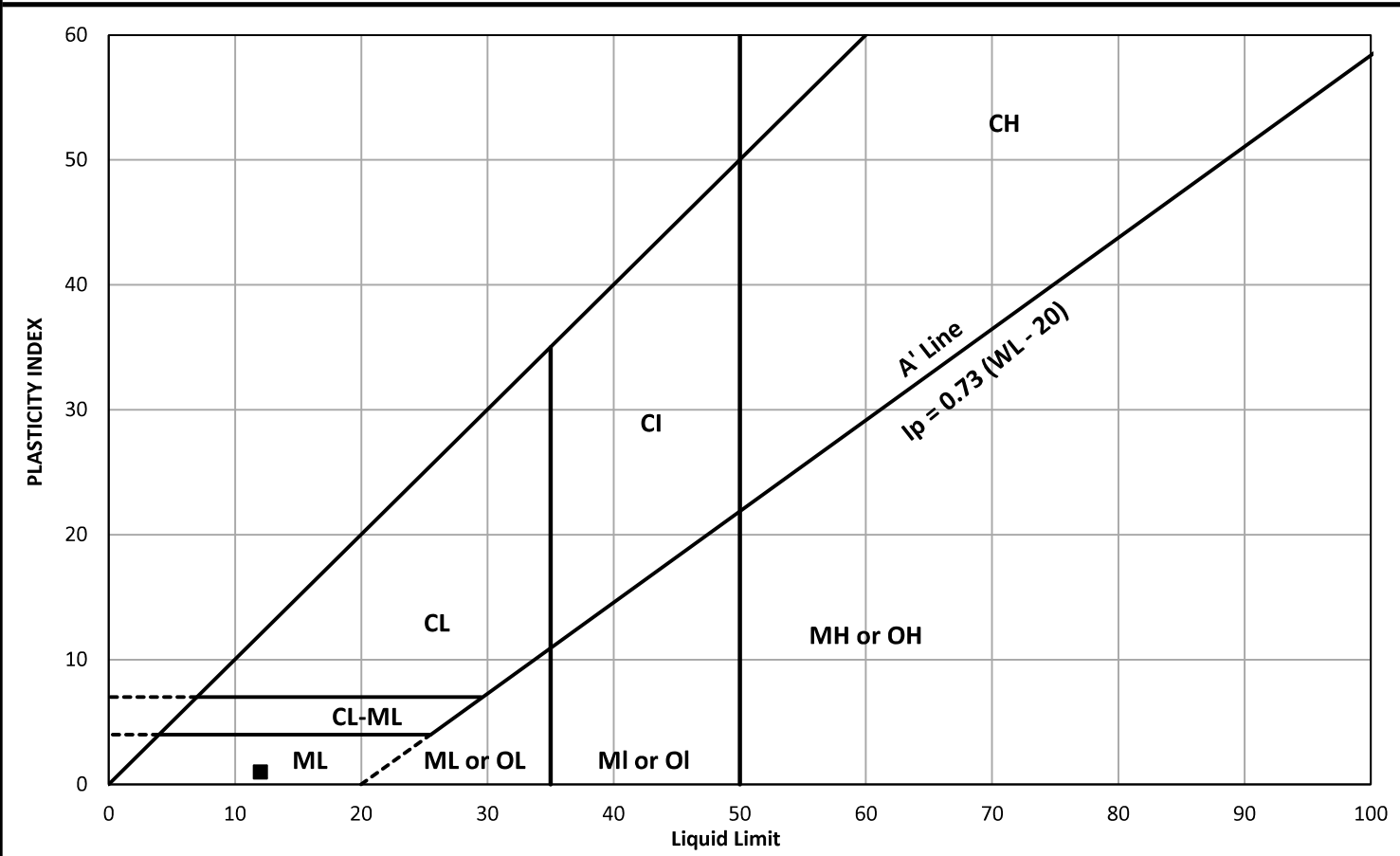
Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	9-1	13	13.7 - 14.3	261.9 to 261.3

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - 9th Line	
CONSULTANT	YYYY-MM-DD	2023-08-09	
	DESIGNED	MCK	
	PREPARED	MCK	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		LOWER SILTY SAND (SM) TILL	
		PROJECT NO.	CONTROL
		19136074	1000
		REV.	FIGURE
		0	C5





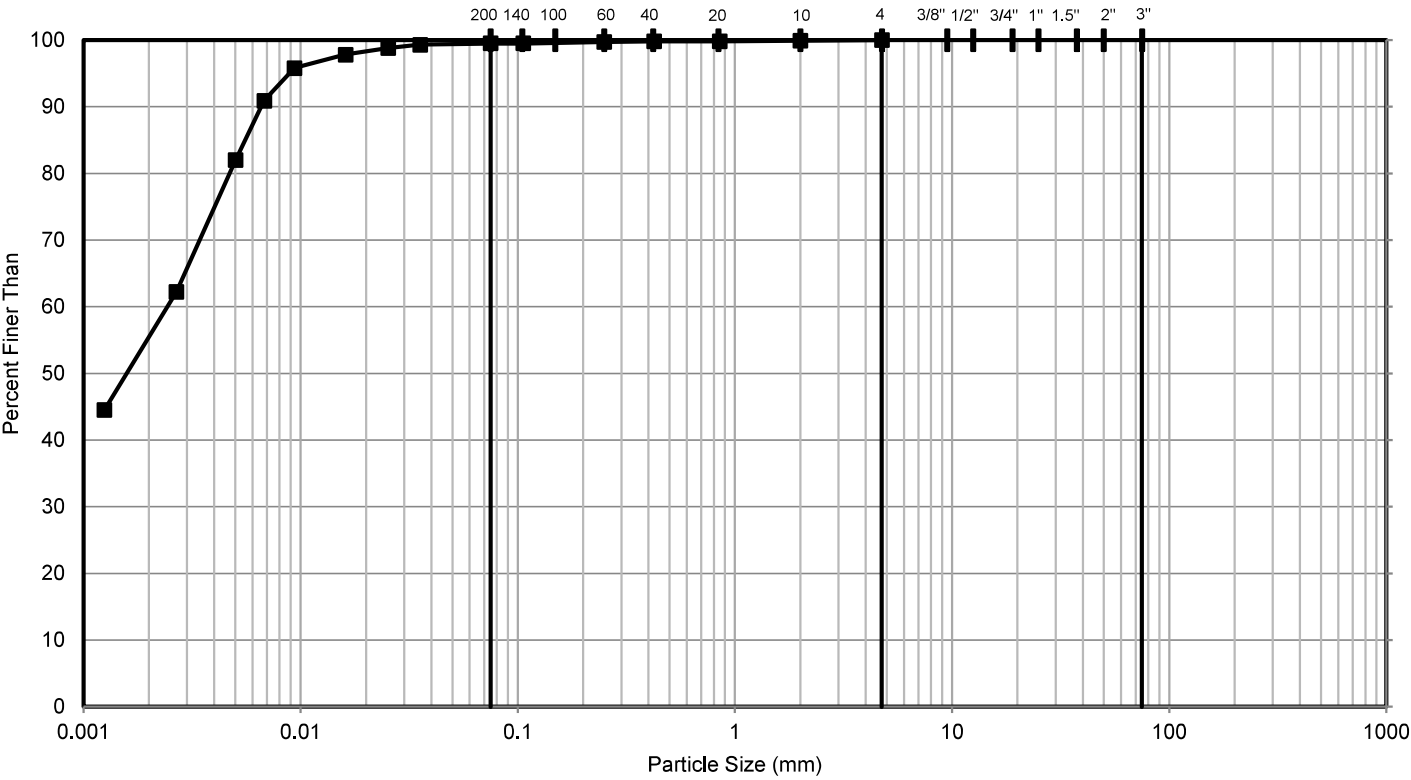
PLASTICITY CHART



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	9-1	13	261.9 to 261.3	11.1	12	11	1

CLIENT				PROJECT			
AECOM / MTO				Bradford Bypass - 9th Line			
	CONSULTANT	YYYY-MM-DD	2023-08-09	TITLE LOWER SILTY SAND (SM) TILL			
	DESIGNED		MCK				
	PREPARED		MCK				
	REVIEWED		KJB				
	APPROVED		KJB				
PROJECT NO.		CONTROL		REV.		FIGURE	
19136074		1000		0		C6	

GRAIN SIZE DISTRIBUTION



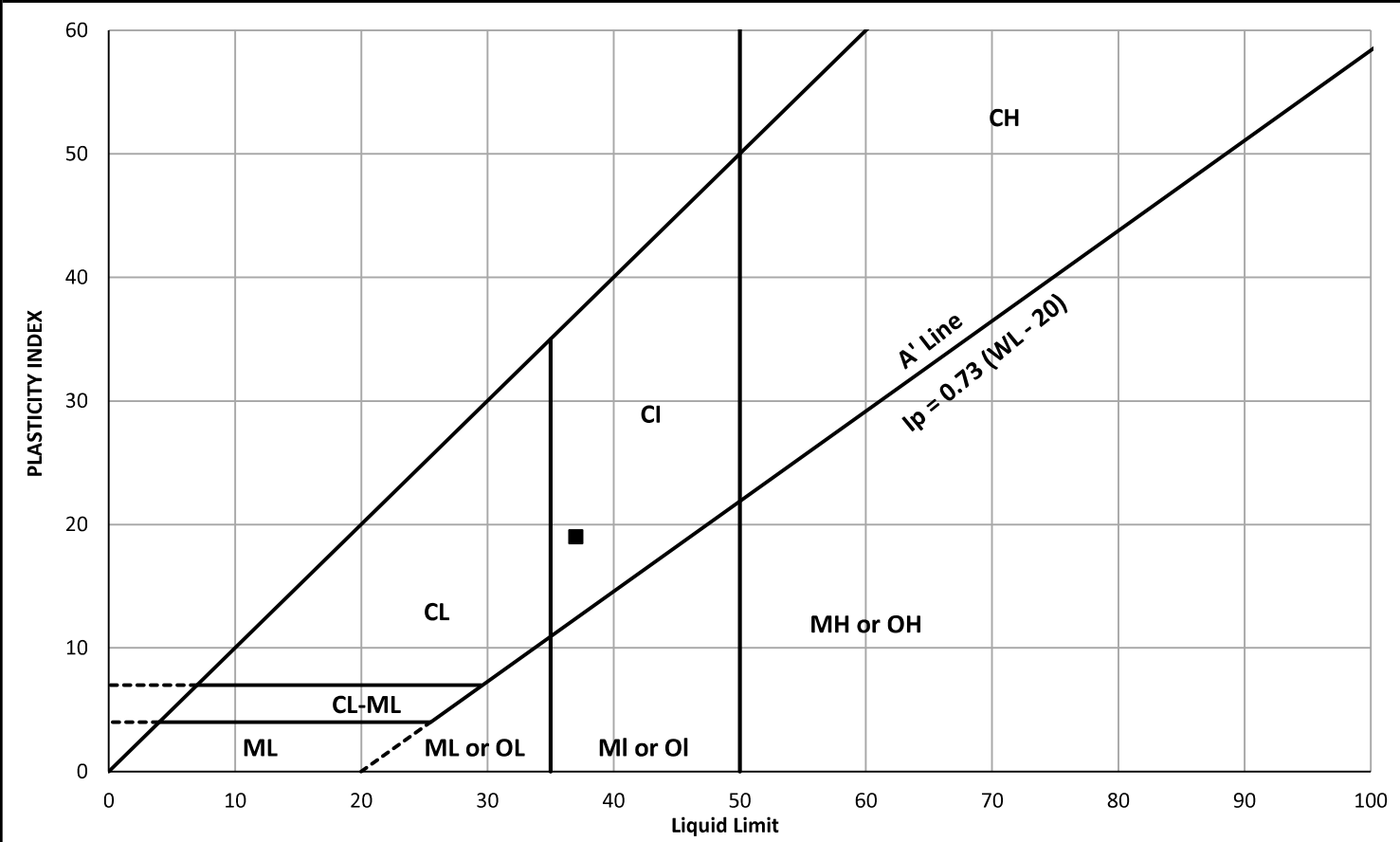
FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	9-1	16	21.3 - 22.0	254.2 to 253.6

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - 9th Line	
CONSULTANT	YYYY-MM-DD	2023-08-09	
	DESIGNED	MCK	
	PREPARED	MCK	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		SILTY CLAY (CI)	
		PROJECT NO.	CONTROL
		19136074	1000
		REV.	FIGURE
		0	C7



PLASTICITY CHART




Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	9-1	16	254.2 to 253.6	26.6	37	18	19

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD

2023-08-09

DESIGNED

MCK

PREPARED

MCK

REVIEWED

KJB

APPROVED

KJB

PROJECT

Bradford Bypass - 9th Line

TITLE

SILTY CLAY (CI)

PROJECT NO.

19136074

CONTROL

1000

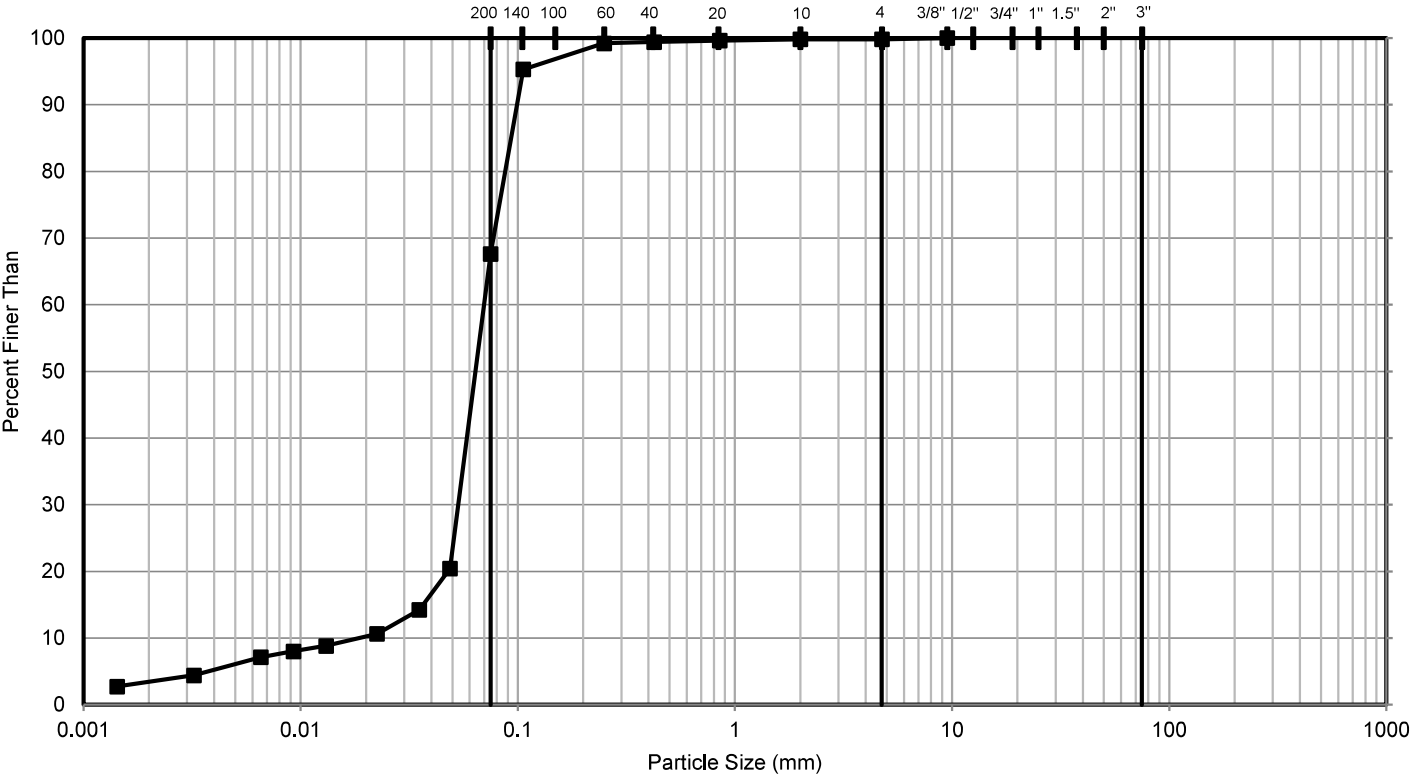
REV.

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FIGURE

C8

GRAIN SIZE DISTRIBUTION



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	9-1	19	30.5 - 30.8	245.1 to 244.8

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - 9th Line	
CONSULTANT	YYYY-MM-DD	2023-08-09	
	DESIGNED	MCK	
	PREPARED	MCK	
	REVIEWED	KJB	
	APPROVED	KJB	
TITLE		SANDY SILT (ML)	
		PROJECT NO.	CONTROL
		19136074	1000
		REV.	0
		FIGURE	C9



**APPENDIX D**

**Soil Analytical Test Results (WSP  
Golder 2021 Investigation)**



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

**Attention: Carter Comish**

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

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

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C1H2336**

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

Sample Matrix: Soil  
# Samples Received: 12

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

**Remarks:**

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

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

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

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

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

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

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

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



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

**Attention: Carter Comish**

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

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

**CERTIFICATE OF ANALYSIS**

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

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.  
Ema Gitej, Senior Project Manager  
Email: emese.gitej@bureauveritas.com  
Phone# (905)817-5829

=====

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BUREAU  
VERITASBV Labs Job #: C1H2336  
Report Date: 2021/06/25Golder Associates Ltd  
Client Project #: 19136074  
Site Location: BRADFORD BYPASS  
Sampler Initials: CC

## RESULTS OF ANALYSES OF SOIL

BV Labs ID		PXF832		PXF833		PXF834	PXF835		
Sampling Date		2021/05/27		2021/05/03		2021/05/17	2021/04/13		
COC Number		827733-01-01		827733-01-01		827733-01-01	827733-01-01		
	UNITS	B-2 SS3	QC Batch	10-4 SS2	QC Batch	Y-1 SS3	10-2 SS3	RDL	QC Batch

## Calculated Parameters

Resistivity	ohm-cm	1500	7421780	2400	7421780	4500	1400		7421780
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## Inorganics

Soluble (20:1) Chloride (Cl-)	ug/g	270	7426573	180	7426733	65	330	20	7426573
Conductivity	umho/cm	651	7429034	416	7429034	220	700	2	7429034
Available (CaCl2) pH	pH	7.66	7426977	7.77	7426977	7.70	7.95		7426977
Soluble (20:1) Sulphate (SO4)	ug/g	<20	7426738	<20	7426744	<20	<20	20	7426738

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

BV Labs ID		PXF835		PXF836		PXF837		PXF838		
Sampling Date		2021/04/13		2021/04/11		2021/06/02		2021/04/12		
COC Number		827733-01-01		827733-01-01		827733-01-01		827733-01-01		
	UNITS	10-2 SS3 Lab-Dup	QC Batch	Y-4 SS2	RDL	L-3 SS3	RDL	9-1 SS3	RDL	QC Batch

## Calculated Parameters

Resistivity	ohm-cm			3400		380		11000		7421780
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## Inorganics

Soluble (20:1) Chloride (Cl-)	ug/g			130	20	1700	100	<20	20	7426573
Conductivity	umho/cm			295	2	2660	2	90	2	7429034
Available (CaCl2) pH	pH	8.05	7426977	7.81		7.56		7.91		7426977
Soluble (20:1) Sulphate (SO4)	ug/g			<20	20	<20	20	<20	20	7426738

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

Lab-Dup = Laboratory Initiated Duplicate



BUREAU  
VERITAS

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

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

### RESULTS OF ANALYSES OF SOIL

BV Labs ID		PXF839	PXF840	PXF841	PXF842	PXF843		
Sampling Date		2021/04/29	2021/04/19	2021/04/19	2021/04/30	2021/04/21		
COC Number		827733-01-01	827733-01-01	827733-01-01	827733-01-01	827733-01-01		
	UNITS	10-1 SS2	PDD-1 SS3	PDD-2 SS2	10-3 SS3	B-1 SS3	RDL	QC Batch
<b>Calculated Parameters</b>								
Resistivity	ohm-cm	4100	7800	7600	6900	8700		7421780
<b>Inorganics</b>								
Soluble (20:1) Chloride (Cl <sup>-</sup> )	ug/g	100	<20	<20	25	<20	20	7426573
Conductivity	umho/cm	246	128	131	146	115	2	7429034
Available (CaCl <sub>2</sub> ) pH	pH	7.83	7.74	7.63	7.84	7.88		7426977
Soluble (20:1) Sulphate (SO <sub>4</sub> )	ug/g	<20	<20	<20	<20	<20	20	7426738
RDL = Reportable Detection Limit								
QC Batch = Quality Control Batch								

BV Labs ID		PXF843		
Sampling Date		2021/04/21		
COC Number		827733-01-01		
	UNITS	B-1 SS3 Lab-Dup	RDL	QC Batch
<b>Inorganics</b>				
Conductivity	umho/cm	113	2	7429034
RDL = Reportable Detection Limit				
QC Batch = Quality Control Batch				
Lab-Dup = Laboratory Initiated Duplicate				



BUREAU  
VERITAS

BV Labs Job #: C1H2336

Report Date: 2021/06/25

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

## TEST SUMMARY

**BV Labs ID:** PXF832

**Sample ID:** B-2 SS3

**Matrix:** Soil

**Collected:** 2021/05/27

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF833

**Sample ID:** 10-4 SS2

**Matrix:** Soil

**Collected:** 2021/05/03

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426733	2021/06/24	2021/06/24	Alina Dobreanu
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426744	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF834

**Sample ID:** Y-1 SS3

**Matrix:** Soil

**Collected:** 2021/05/17

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF835

**Sample ID:** 10-2 SS3

**Matrix:** Soil

**Collected:** 2021/04/13

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF835 Dup

**Sample ID:** 10-2 SS3

**Matrix:** Soil

**Collected:** 2021/04/13

**Shipped:**

**Received:** 2021/06/22

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



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VERITAS

BV Labs Job #: C1H2336

Report Date: 2021/06/25

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

## TEST SUMMARY

**BV Labs ID:** PXF836

**Sample ID:** Y-4 SS2

**Matrix:** Soil

**Collected:** 2021/04/11

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF837

**Sample ID:** L-3 SS3

**Matrix:** Soil

**Collected:** 2021/06/02

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF838

**Sample ID:** 9-1 SS3

**Matrix:** Soil

**Collected:** 2021/04/12

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF839

**Sample ID:** 10-1 SS2

**Matrix:** Soil

**Collected:** 2021/04/29

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF840

**Sample ID:** PDD-1 SS3

**Matrix:** Soil

**Collected:** 2021/04/19

**Shipped:**

**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan



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

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

## TEST SUMMARY

**BV Labs ID:** PXF840  
**Sample ID:** PDD-1 SS3  
**Matrix:** Soil

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

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

**BV Labs ID:** PXF841  
**Sample ID:** PDD-2 SS2  
**Matrix:** Soil

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

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

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

**Collected:** 2021/04/30  
**Shipped:**  
**Received:** 2021/06/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

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

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

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7426573	2021/06/24	2021/06/24	Avneet Kour Sudan
Conductivity	AT	7429034	2021/06/25	2021/06/25	Khushbu Vijay kumar Patel
pH CaCl2 EXTRACT	AT	7426977	2021/06/24	2021/06/24	Neil Dassanayake
Resistivity of Soil		7421780	2021/06/25	2021/06/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7426738	2021/06/24	2021/06/24	Avneet Kour Sudan

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

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

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



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BV Labs Job #: C1H2336

Report Date: 2021/06/25

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

### GENERAL COMMENTS

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

Package 1	4.0°C
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Results relate only to the items tested.



BUREAU  
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BV Labs Job #: C1H2336

Report Date: 2021/06/25

## QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7426573	Soluble (20:1) Chloride (Cl-)	2021/06/24	NC	70 - 130	105	70 - 130	<20	ug/g	3.5	35
7426733	Soluble (20:1) Chloride (Cl-)	2021/06/24	NC	70 - 130	104	70 - 130	<20	ug/g	4.4	35
7426738	Soluble (20:1) Sulphate (SO4)	2021/06/24	112	70 - 130	102	70 - 130	<20	ug/g	NC	35
7426744	Soluble (20:1) Sulphate (SO4)	2021/06/24	NC	70 - 130	103	70 - 130	<20	ug/g	19	35
7426977	Available (CaCl2) pH	2021/06/24			100	97 - 103			1.3	N/A
7429034	Conductivity	2021/06/25			100	90 - 110	<2	umho/cm	1.6	10

N/A = Not Applicable

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

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

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

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

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

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





BUREAU  
VERITAS

BV Labs Job #: C1H2336

Report Date: 2021/06/25

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: CC

### VALIDATION SIGNATURE PAGE

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

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

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

# CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:			REPORT TO:			PROJECT INFORMATION:			Laboratory Use Only:						
Company Name: #1326 Golder Associates Ltd			Company Name: Golder			Quotation #: B80683			BV Labs Job #:						
Attention: Accounts Payable			Attention: Carter Comish			P.O. #:			Bottle Order #:						
Address: 6925 Century Ave Suite 100			Address:			Project: 14136074			COC #:						
Mississauga ON L5N 7K2			Tel: (905) 567-4444 Fax: (905) 567-6561			Project Name: B70683 B4255			Project Manager:						
Email: CanadaAccountsPayableInvoices@golder.com			Email: Carter_Comish@golder.com			Site #:			Ema Gitej						
Sampled By:			C8827733-02-01												
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY															
Regulation 153 (2011)				Other Regulations				Special Instructions							
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Medium/Fine				<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw											
<input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse				<input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw											
<input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC				<input type="checkbox"/> MISA Municipality											
<input type="checkbox"/> Table				<input type="checkbox"/> PWQO <input type="checkbox"/> Reg 406 Table											
<input type="checkbox"/> Other															
Include Criteria on Certificate of Analysis (Y/N)?															
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle):	Metals / Hg / Cr / VI	Corrosivity pH short (Cl, SO4, pH, EC/Resistivity)	ANALYSIS REQUESTED (PLEASE BE SPECIFIC)				Turnaround Time (TAT) Required:			
1	10-3 SS3	21/04/22	AM	Soil									Regular (Standard) TAT:		
2	B-1 SS3	21/04/21											(will be applied if Rush TAT is not specified):		
3													Standard TAT = 5-7 Working days for most tests.		
4													Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.		
5													Job Specific Rush TAT (if applies to entire submission)		
6													Date Required: Time Required:		
7													Rush Confirmation Number: (call lab for #)		
8													# of Bottles		
9													Comments		
10															
* RELINQUISHED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	# jars used and not submitted	Laboratory Use Only						
Carter Comish		21/06/22	4:00 PM	Carter Comish		21/06/22	1:00 PM		Time Sensitive	Temperature (°C) on Reel	Custody Seal	Yes	No		
										5/3/4	Present				
											Intact				
* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.												White: BV Labs		Yellow: Client	
* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.												SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS			
** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.															

Bureau Veritas Canada (2019) Inc.

**wsp** GOLDER

**wsp.com**