



**FINAL REPORT**

# **Preliminary Foundation Investigation and Design Report**

*Artesian Industrial Parkway Overpass*

*Highway 400 to Highway 404 Link (Bradford Bypass)*

*Simcoe County and York Region*

*MTO Assignment No. 2019-E-0048*

Submitted to:

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# PART A

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



## 1.0 INTRODUCTION

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

## 2.0 SITE DESCRIPTION

The proposed twin single-span structures will cross AIP just north of 8<sup>th</sup> Line (Dissette Street) in the Town of Bradford-West Gwillimbury in Simcoe County, Ontario. AIP is currently a two-lane local road with one northbound and one southbound lane with several above ground and buried utilities running below and adjacent to the roadway. The site generally falls within an industrial district with buildings located west and northeast of the site. Directly to the east and south, the site consists of grassy fields transitioning into forested area. The existing ground surface generally slopes down from west to east. An existing culvert crosses AIP at the site to drain stormwater from the west side to the east side of the existing road, into a watercourse that drains into a wet swampy area located about 150 m east of site towards Holland River.



Photograph 1 - Proposed bridge site along AIP (looking northwest)



Photograph 2 - Proposed bridge site along AIP (looking southwest)

## 3.0 INVESTIGATION PROCEDURES

The field work for the current investigation was carried out between November 19 and December 1, 2021. A total of three boreholes (designated AIP-2, AIP-3, and AIP-4) were advanced near the proposed structure footprints as shown in Drawing 1. Due to the presence of above ground and buried utilities and private property restrictions at the site, the proposed borehole (AIP-1) that was to be located at the northwest corner of the site was not advanced.

The three boreholes were advanced using 210 mm outside diameter hollow stem augers generally set to a depth of approximately 2.5 m below ground surface, followed by wash-rotary techniques (advancement of tricone and casing with water/drilling mud) using a D120 track-mounted drill (equipped with water tanks/totes) 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. Traffic control for this field investigation was in general accordance with the Ontario Traffic Manual (Book 7, Temporary Conditions) and was provided by PGC Traffic of Stouffville, Ontario and Direct Traffic Management Inc. of Hamilton, Ontario.

Soil samples were generally obtained at 0.75 m, 1.5 m, and 3.0 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven with an automatic hammer in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586)<sup>1</sup>, and using 76 mm outer diameter thin-walled ‘Shelby’ Tube samplers (ASTM D1587)<sup>2</sup> to obtain relatively undisturbed samples in the soil. The split-spoon samplers used in the investigation generally limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

Where encountered, the water level was measured within the hollow stem augers prior to the start of mud rotary operations and a standpipe piezometer was installed directly adjacent to borehole AIP-3 (screened within the sandy clayey silt deposit) to allow monitoring of the groundwater level. The installed piezometer consists of a 50 mm diameter PVC pipe, with 1.5 m long slotted screen within a filter sand pack. The boreholes and annulus surrounding the piezometer pipe above the filter sand were backfilled with a bentonite mixture upon completion in general accordance with Ontario Regulation 903 Wells (as amended), and the ground surface was restored, to as near original condition as practicable.

The field work was monitored on a full-time basis by a member of WSP Golder’s engineering staff who located the boreholes in the field, arranged for clearance of underground services, directed the sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers, and transported to WSP Golder’s laboratory in Mississauga for further visual review and geotechnical laboratory testing. Index and classification testing consisting of natural moisture content, Atterberg limits and grain size distribution were conducted on selected samples. One consolidation (oedometer) test was performed on a Shelby tube sample collected in Borehole AIP-3. All laboratory tests were carried out in general accordance with MTO and / or ASTM Standards, as applicable.

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

The borehole locations were surveyed in the field by WSP Golder personnel using a Trimble Geo 7X Global Positioning System (GPS) unit. The locations given on the borehole records (Appendix A) and shown on Drawing 1 are positioned relative to NAD 83 MTM (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum (CGVD28 datum). The borehole locations, including the geographic coordinates, the ground surface elevations, and borehole depths are summarized below.

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

<sup>2</sup> ASTM D1587 - Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.



Borehole No.	NAD 83 MTM Coordinates (Geographic Coordinates)		Borehole Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
AIP-2	4887958.8 (44.131611)	300326.2 (-79.555908)	225.0	27.7
AIP-3	4887967.1 (44.131686)	300371.1 (-79.555347)	224.8	24.7
AIP-4	4887987.8 (44.131872)	300376.7 (-79.555277)	224.5	27.7

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

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

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

### 4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes from the current investigation including piezometer installation details and water level readings, and the results of the in-situ and laboratory tests are provided on the borehole records in Appendix A. *Abbreviations and Terms Used on Records of Boreholes and Test Pits* and *List of Symbols* sheets are provided in Appendix A to assist in the interpretation of the borehole records. The results of the geotechnical laboratory testing on the soil samples are presented in more detail on the laboratory test figures in Appendix B. The analytical laboratory test results are presented in Appendix C.

The results of the in-situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4 were measured in the field and are considered uncorrected. The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface soils encountered at the AIP overpass site consist of surficial layers of fill underlain by layers of firm to hard clayey silt and silty clay, underlain by a layer of sandy silt to silt. These layers are underlain by a deposit of dense to very dense / hard glacial till consisting of sand and silt / clayey to clayey silt-silt which is further underlain by a very dense silt layer.

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

#### **4.2.1 Asphalt**

An approximately 50 mm thick layer of asphalt was present at the road surface in Borehole AIP-2.

#### **4.2.2 Sand (FILL)**

An approximately 0.7 m thick layer of non-cohesive sand fill was encountered below the asphalt in Borehole AIP-2. The sand contained trace fines and trace gravel.

The SPT 'N'-value measured within the sand fill is 34 blows per 0.3 m of penetration, indicating a dense state of compactness.

The water content measured on a sample of the sand fill was about 5%.

#### **4.2.3 Clayey Silt-Silt (FILL)**

An approximately 0.7 m to 2.3 m thick layer of clayey silt to clayey silt-silt fill was encountered below the sand fill layer in Borehole AIP-2, and at ground surface in Borehole AIP-3 and AIP-4. The cohesive fill consisted of trace to some sand, trace gravel, and contained trace organics near the ground surface.

The SPT 'N'-values measured within this fill layer range from 8 to 34 blows per 0.3 m of penetration suggesting a stiff to hard consistency.

The water content measured on samples of this fill layer range from about 12% to 20%. The results of the Atterberg limits test carried out on a sample of this fill layer measured a liquid limit of 17%, plastic limit of 12%, and plasticity index of 5%.

#### **4.2.4 Silt and Sand to Gravelly Sand / Clayey Silt to Gravel (FILL / Possible FILL)**

An approximately 0.7 m to 0.8 m thick layer of silt and sand to gravelly sand fill was encountered below the clayey silt fill in Boreholes AIP-3 and AIP-4. The silt and sand to gravelly sand fill layer contained trace organics as indicated by the presence of rootlets. Underlying the gravelly sand fill in Borehole AIP-4, interlayers of clayey sand, sand, clayey silt and gravel (ranging from 0.3 m to 0.8 m thick) were encountered and classified as possible fill due to the variable nature of the soils.

The SPT 'N'-values measured within the silt and sand to gravelly sand fill layer range from 13 to 22 blows per 0.3 m of penetration, indicating a compact state of compactness. The SPT 'N'-values measured within the interlayered clayey sand, sand, clayey silt and gravel (possible fill) range from 4 to 28 blows per 0.3 m of penetration, suggesting a very loose to compact / very stiff state of compactness / consistency.

The water content measured on samples of the fill layers ranges from 10% to 15%. The results of grain size distributions carried out on two samples of the silt and sand to gravelly sand fill material are presented on Figure B1 in Appendix B.

#### **4.2.5 Clayey Silt to Clayey Silt-Silt**

A 4.2 m to 6.1 m thick layer of clayey silt to clayey silt-silt was encountered below the fill / possible fill layers in the three boreholes. The clayey silt to clayey silt-silt layer typically contained trace sand and trace gravel with frequent silt seams / interlayers. A zone of sandy clayey silt-silt (resembling glacial till) was encountered within the deposit from about 2.3 m to 5.6 m depth in Borehole AIP-3. Trace organics were also encountered in the upper zone of the deposit (within 2.1 m below ground surface) in Borehole AIP-3.

The SPT 'N'-values measured within the clayey silt to clayey silt-silt typically range from 10 to 67 blows per 0.3 m of penetration suggesting a stiff to hard consistency. Two 'N'-values measured within the sandy clayey silt-silt zone encountered in Borehole AIP-3 were 100 blows for 0.13 m and 0.15 m of penetration, suggesting zones of hard consistency and/or the presence of gravel pockets or cobbles.

The results of grain size distribution and Atterberg limits testing carried out on samples of the clayey silt are presented on Figures B2 and B3, respectively, in Appendix B. Atterberg limits measured liquid limits ranging from 28% to 30%, plastic limits ranging from 16% to 17%, and plasticity indices ranging from 12% to 13%.

The results of grain size distribution and Atterberg limits testing carried out on samples of sandy clayey silt-silt zone in AIP-3 are presented on Figures B4 and B5, respectively, in Appendix B. Atterberg limits measured liquid limits of 19%, plastic limits ranging from 12% to 14%, and plasticity indices ranging from 5% to 7%.

The water content measured on samples of the clayey silt range between about 16% and 23%, with slightly lower water contents of 10% to 14% measured within the sandy clayey silt to clayey silt-silt zone in Borehole AIP-3.

#### **4.2.6 Silty Clay**

Underlying the clayey silt layer in all three boreholes, a 2.3 m to 4.7 m thick deposit of silty clay was encountered. The silty clay typically contained trace sand and sand seams were encountered in the lower portion (below 12.5 m depth) of the layer in Borehole AIP-4.

The SPT 'N'-values measured within the silty clay deposit range from 6 to 31 blows per 0.3 m of penetration, suggesting a firm to hard consistency.

The water content measured on samples of the silty clay range between about 27% and 33%. The results of grain size distribution and Atterberg limits testing carried out on samples of the silty clay are presented on Figures B6 and B7, respectively, in Appendix B. Atterberg limits measured liquid limits ranging from 36% to 42%, plastic limits ranging from 18% to 19%, and plasticity indices ranging from 18% to 23%.

One Shelby tube sample was collected in the silty clay deposit in Borehole AIP-3. Laboratory consolidation (oedometer) testing was carried out on one vertically trimmed specimen of the silty clay to assess the compressibility characteristics of the deposit. The details of the laboratory test results (uncorrected) are shown on Figures B8 to B11, and the interpreted results are summarized below. It was noted that significant swelling of the sample occurred upon initial wetting / saturation and the swelling continued to occur during the first load / unload cycle. The results and interpretations of the testing should be used with caution as it appears the final unloading cycle may also have been influenced by continued swelling of the soil. The swelling potential of the silty clay is classified as medium to high (after Seed et. Al, 1962) and may be attributed to the possible presence of silt seams, high content of clay size particles, and possible history of wetting / drying cycles (desiccation) that may contribute to the high over consolidation ratio.

Borehole / Sample No.	Sample Depth / Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR (avg.)	$C_c$	$C_r$	$e_o$	$c_v$ (cm <sup>2</sup> /s)
AIP-3 / TO12	11.7 / 213.1	120 to 130	>400	270 - 380	> 3	0.17	0.01	0.95	0.001

Where:  $s_{p\phi}$  = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)  $c_v$  = Coefficient of consolidation (vertical) for approximate over consolidated stress range  $125 \text{ kPa} \leq s_v' \leq 300 \text{ kPa}$

$C_c$  = Compression index

$C_r$  = Recompression index

$e_o$  = Initial void ratio

OCR = Overconsolidation ratio

$s_{vo\phi}$  = Calculated existing vertical effective stress

#### 4.2.7 Silt to Sandy Silt

A 2.2 m to 2.3 m thick deposit of silt to sandy silt was encountered below the silty clay layer in all three boreholes.

The SPT 'N'-value measured within the sandy silt deposit ranges from 29 to 67 blows per 0.3 m of penetration, indicating a compact to very dense state of compactness.

The water content measured on samples of the silty soil range between about 20% and 22%. The results of grain size distribution testing carried out on two samples of silt are presented on Figure B12.

#### 4.2.8 Clayey Silt to Clayey Silt-Silt (TILL)

A 2.3 m to 10.5 m thick deposit of clayey silt to clayey silt-silt till was encountered below the silt to sandy silt layer in all three boreholes. The clayey silt to clayey silt-silt till layer typically has trace to some sand, trace gravel and had increasing sand content below a depth of 21.3 m in Borehole AIP-4.

The SPT 'N'-values measured within the till deposit range from 36 blows per 0.3 m of penetration to greater than 100 blows per 0.1 m of penetration, suggesting a hard consistency. Although not specifically encountered or confirmed during the current investigation, the high SPT 'N' values (greater than 100-blows) may suggest the presence of gravel pockets or cobbles (and possibly boulders) and should be expected in the glacially derived till soils.

The results of grain size distribution and Atterberg limits testing carried out on samples of the clayey silt to clayey silt-silt (till) are presented on Figures B13 and B14, respectively, in Appendix B. Atterberg limits measured liquid limits ranging from 19% to 32%, plastic limits ranging from 12% to 18%, and plasticity indices ranging from 4% to 15%. The water content measured on samples of clayey silt to clayey silt-silt till range between about 10% and 20%.

#### 4.2.9 Silt to Silt and Sand (TILL)

A 6.6 m thick deposit of silt to silt and sand (till) was encountered below the clayey silt till in Borehole AIP-2.

The SPT 'N'-values measured within the silt / silt and sand (till) deposit range from 40 to 89 blows per 0.3 m of penetration, indicating dense to very dense state of compactness.

The results of grain size distribution and Atterberg limits testing carried out on samples of the silt and sand to silt till are presented on Figures B15 and B16, respectively, in Appendix B. Atterberg limits performed on the fines portion

of sample measured a liquid limit of 15%, plastic limit of 12%, and plasticity index of 3%, indicating a silt of slight plasticity.

#### 4.2.10 Silt to Silty Sand

Underlying the silt till in Borehole AIP-1 and clayey silt till in Boreholes AIP-3 and AIP-4, a deposit of silt to silty sand was encountered; all three boreholes were terminated after penetrating 1.7 m to 3.3 m into the deposit. The silt deposit in Borehole AIP-3 contains sand seams from a depth of 23.0 m to 24.7 m.

The SPT 'N'-values measured within the silt to silty sand deposit achieved effective refusal measuring more than 100 blows to advance the sampler from 0.1 m to 0.13 m of penetration, indicating a very dense state of compactness. The high 'N'-values suggest the presence of gravel pockets or cobbles, although this was not confirmed during the current investigation.

The results of grain size distribution testing carried out on a sample of the silt deposit is presented on Figure B17. The water content measured on samples of silt to silty sand deposit range from about 17% to 21%. The results of an Atterberg limits test performed on a sample of the silt indicates the deposit is non-plastic.

### 4.3 Groundwater Conditions

The water level was typically not measured in the open boreholes during or upon completion of drilling operations given that water / drilling mud was introduced as part of the wash rotary technique initiated at relatively shallow depth. Where water levels are described on the borehole records, they represent unstabilized groundwater conditions typically measured within the hollow stem augers during drilling operations.

A standpipe piezometer was installed in Borehole AIP-3 to monitor groundwater levels. The standpipe piezometer was screened within the sandy clayey silt to clayey silt-slit as shown in detail on the borehole record in Appendix A. A summary of the groundwater levels measured in the piezometer is provided below.

Borehole No.	Ground Surface Elevation (m)	Depth (Elevation) of Screen Interval / Sand Pack (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
AIP-3	224.8	1.5 to 3.0 (El. 223.3 to 221.8)	1.5	223.3	Dec. 23, 2021	Piezometer
			1.4	223.4	Feb. 4, 2022	
			1.4	223.4	Feb. 8, 2022	
			1.3	223.5	Feb. 16, 2022	
			1.4	223.4	May 12, 2022	

The groundwater level at this site is anticipated to fluctuate seasonally in response to changes in precipitation and should be expected to be higher during the spring season or during/after any period of heavy and/or sustained precipitation.

### 4.4 Analytical Testing Results

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

Borehole No. - Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Chloride (µg/g)	Soluble Sulphate (µg/g)
AIP-2 - 2	7.89	2200	454	96	<20 <sup>1</sup>
AIP-3 - 1	7.65	5900	170	<20 <sup>1</sup>	<20 <sup>1</sup>
AIP-4 - 2	7.81	7200	139	<20 <sup>1</sup>	<20 <sup>1</sup>

Note 1: Less than reportable detection limit.

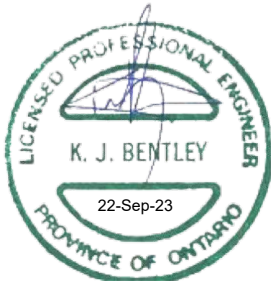
## 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Priyanka Talukdar, an Engineer in Training at WSP Golder. Mr. Kevin Bentley, P.Eng., a Senior Principal and MTO Foundations Designated Contact with WSP Golder conducted a technical review of the report. Ms. Lisa Coyne, P.Eng., a Fellow and MTO Foundations Designated Contact with WSP Golder conducted an independent technical and quality control review of the report.



## Signature Page

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PT/MCK/KJB/LCC/al

[https://golderassociates.sharepoint.com/sites/120387/project files/6 deliverables/foundations/artesian industrial parkway/final/19136074-r-rev0-final fidr-aip\\_22sept2023.docx](https://golderassociates.sharepoint.com/sites/120387/project%20files/6%20deliverables/foundations/artesian%20industrial%20parkway/final/19136074-r-rev0-final%20fidr-aip_22sept2023.docx)

# PART B

**PRELIMINARY FOUNDATION DESIGN REPORT  
ARTESIAN INDUSTRIAL PARKWAY OVERPASS STRUCTURE  
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)  
SIMCOE COUNTY AND YORK REGION  
ASSIGNMENT NO. 2019-E-0048**

## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

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

The Preliminary Foundation Design Report (Part B of this report) including the discussion and recommendations are intended for the use of MTO and their designers for planning and preliminary design and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor or design-build proponents. Contractors undertaking the work must make their own interpretation based on the factual data presented in the Preliminary Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the concept and preliminary design of the project and for which special provisions may be required in the future Contract Documents. Those requiring information on aspects of 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 General Arrangement (GA) drawing and latest Bradford Bypass mainline alignment and profile drawings provided by AECOM (preliminary drafts dated April and May 2023), twin bridge structures are proposed to carry the Bradford Bypass eastbound and westbound lanes over Artesian Industrial Parkway (AIP). Each single-span overpass is proposed to be approximately 38 m long with concrete wingwalls in each quadrant. Initially, the twin overpasses will accommodate two lanes of traffic on Bradford Bypass in the eastbound and westbound direction (four lanes total) for the interim configuration, with an ultimate configuration to accommodate four lanes in each direction (eight lanes total) requiring future bridge widenings. The structural classification of the bridge(s) is defined as “major route” by the structural designer for preliminary design.

Based on the preliminary drawings, the existing AIP road surface is at about Elevation 225 m and the proposed Bradford Bypass highway grade will be at about Elevation 233 m. As a result, the proposed new west and east approach embankments for both structures are anticipated to be up to approximately 8 m high relative to the existing ground surface outside the existing AIP.

Based on the preliminary GA drawing, the interim design configuration will consist of an approximately 14 m wide bridge (two 3.8 m wide lanes, 2.5 m and 3.0 m wide shoulders, and a 0.5 m wide concrete barrier) for both the eastbound and westbound directions, and the ultimate configuration will consist of future widening of the bridges toward the highway centreline (essentially join the bridges together less a 1 m gap) with an additional width of about 10 m being added to each bridge to accommodate an additional two 3.8 m wide lanes, a 3.6 m shoulder, concrete barrier and a buffer zone.

The bridges are designed to facilitate widening of AIP in the future (municipal initiative). The future interim AIP configuration under the overpasses is proposed to consist of one lane in each direction plus shoulders, with provision for future widening to accommodate two lanes in each direction, with multi-use pathways along each side of the road separated by a boulevard. The overpasses may initially be constructed with abutment foreslopes in an

open configuration, with future widening of AIP accomplished via construction of retaining walls within the abutment foreslopes.

## 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 overpass structures and foundation system may be classified as having medium traffic volumes and their performance as having potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design.

In addition, based on the preliminary level of foundation investigation completed to date at this location (see Part A of this report) in comparison to the degree of site understanding, the level of confidence for design of the single span bridge foundation elements and approach embankments has been assessed as a “typical degree of site and prediction model understanding”. At the time of investigation and issue of this report, the locations of the abutment foundations were not confirmed, and based on this together with access considerations, the boreholes are not located directly within the foundation footprints. As such, the recommendations contained in the report are generalized for planning and ongoing preliminary design and further investigation will be required when actual locations of the abutments are known and site access is provided.

Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$  for a typical 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 confirm or possibly increase the level of confidence and geotechnical resistance factors as appropriate. In addition, reference is made to the MTO Material Engineering and Research Office (MERO) Memorandum #2020-01 (dated March 23, 2020) for future foundation, settlement and stability analyses during detail design, as applicable.

### 6.3.2 Seismic Design

#### 6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance,  $\bar{N}_{60}$ , and average undrained shear strength,  $s_u$ , within the upper 30 m of the overburden below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019), in the absence of any geophysical testing. Geophysics testing, such as Multi-Channel Analysis of Surface Waves (MASW) or Vertical Seismic Profiling (VSP), may provide a more favourable Site Class designation, and such testing can be considered during detail design.

The CHBDC (2019) states that the seismic hazard values associated with the design earthquakes should be those established for the *National Building Code of Canada* (NBCC) by the Geological Survey of Canada (GSC). The 2015 seismic hazard maps (referred to as the 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 site using the NBCC website ([earthquakescanada.nrcan.gc.ca](http://earthquakescanada.nrcan.gc.ca)) and are summarized below.

Seismic Hazard Values for Site Class D	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$PGA$ (g)	0.039	0.059	0.095
$PGV$ (m/s)	0.040	0.062	0.098
$S_a(0.2)$ (g)	0.064	0.097	0.151
$S_a(0.5)$ (g)	0.054	0.079	0.119
$S_a(1.0)$ (g)	0.033	0.050	0.074
$S_a(2.0)$ (g)	0.016	0.025	0.039
$S_a(5.0)$ (g)	0.003	0.006	0.009
$S_a(10.0)$ (g)	0.001	0.003	0.004

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

### 6.3.2.3 Soil Liquefaction

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

In general, the soils at these bridge sites consist of interlayered firm to hard clayey silt to clayey silt-silt and silty clay soils, compact to very dense sandy silt to silt, and very dense / hard silts and sands / clayey silt till. Considering the compactness, consistency and liquidity index of the soils and the relatively low site-specific  $PGA$ , the site is considered to have a low potential for liquefaction during a seismic event based on preliminary liquefaction assessment of the soil and preliminary analyses using the simplified stress-based method as per Section 6.14.8 of the CHBDC (2019).

## 6.4 Foundation Types

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

A 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, the use of driven steel tube or H-piles with the pile cap perched within the approach embankments is considered the preferred alternative from a geotechnical/foundations perspective, especially if conventional integral abutments are preferred. It is noted that there are a significant number of buried utilities along and adjacent to AIP at the site, with a fill thickness measured to be up to 4 m below ground surface during the investigation. Based on the presence of buried utilities, variable fill thickness, and relatively high groundwater table, shallow foundations are not preferred at the site. As an alternative to driven piles, caissons could also be considered to extend below the variable fill and silty clay soils and be founded in the glacial till deposit, although caissons would preclude conventional integral abutment design and would require drilling slurry to maintain an open hole during advancement through the sand and silt layers.

### 6.4.1 Shallow Foundations

Strip or spread footings founded below any topsoil or organic soils and existing fills on the native stiff to hard clayey silt to clayey silt-silt (at or below the approximate elevations identified below) are considered feasible for support of the structure abutments. Temporary excavations ranging from about 1.5 m to 4.1 m below existing ground surface are anticipated to be required to extend below the fill and reach the competent clayey silt founding strata.

Consideration can be given to subexcavating the unsuitable soils (topsoil, organics and existing fill soils) and placing engineered fill such that spread footings could be “perched” within approach embankments to increase geotechnical resistance values. However, given that subexcavation depths up to 4.1 m are anticipated (with groundwater estimated to be about 1.3 m below ground surface) near potential environmental areas of concern in this industrial area make this a less favourable option.

The following geotechnical resistances may be used for preliminary design, assuming 3 m and 5 m wide footings:



Structure Founding Element	Anticipated Founding Stratum	Founding Elevation <sup>1</sup>	Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance <sup>2</sup>
Westbound and Eastbound Overpass Abutments	Stiff to hard clayey silt to clayey silt-silt	222 m (west abutments) 223 m to 220 m (east abutments)	3	300 kPa	200 kPa
			5	300 kPa	160 kPa
	Granular pad on Stiff to hard clayey silt to clayey silt-silt	Min. 3 m of granular fill above El. 222 m (west) Min. 3 m of granular fill above El. 223 m to 220 m (east)	3	500 kPa	325 kPa
		Min. 5 m of granular fill above El. 222 m (west) Min. 5 m of granular fill above El. 223 m to 220 m (east)	5	550 kPa	300 kPa

## Notes:

1. Subexcavation to about 4 m (for westbound bridge) and 3 m (eastbound bridge) and below groundwater is required to remove unsuitable soils to a competent founding stratum. All design resistance values assume similar vertical stress at founding level.
2. For 25 mm of settlement due to load on foundation element and independent of any settlements induced by surrounding grade changes / embankment loading. Higher settlements may occur at abutment areas associated with the embankment loading.

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

Resistance to lateral loads / sliding resistance between the new concrete footing and the subgrade should be calculated in accordance with Section 6.10.4 of *CHBDC* (2019), applying the appropriate consequence and degree of site understanding factors as applicable during detailed design. For preliminary design, an effective interface angle of friction between the cast-in-place concrete footings and the native clayey silt to clayey silt-silt may be taken as 26° (with an effective cohesion of zero), and the corresponding unfactored coefficient of friction,  $\tan \delta$ , of 0.49 may be used. An effective angle of friction of 33° and corresponding unfactored  $\tan \delta$  of 0.65 may be used for the resistance between a Granular 'A' pad and cast-in-place concrete.

## 6.4.2 Deep Foundations

### 6.4.2.1 Steel H-Pile or Tube Foundations

Driven steel HP 310x110 piles or 324 mm outer diameter closed ended tube piles (assuming a minimum wall thickness of 9.5 mm) driven into the "100-blow" silt, silt and sand till and/or clayey silt till deposits are considered feasible for the foundations at the bridge structures.

Although not confirmed during the current investigation, the presence of potential pockets of gravel, cobbles and/or boulders should be anticipated within the fill and sandy clayey silt soils (suggested by the auger grinding and "100-blow" SPT "N" values recorded on the borehole records within the fill and clayey silt soils) and will need to be investigated further during detail design.

If integral abutments are being considered, driven steel H-piles are preferred over steel tube piles given that H-piles are most commonly used for integral abutment design and that steel tubes are considered to pose a higher risk of

“hanging up” or being deflected from their vertical or battered orientation during installation if cobbles or boulders are present, due to their larger end area. However, close-ended tube piles are considered to have a higher likelihood of achieving design geotechnical resistances during or shortly after driving due to the increased end-bearing area / resistance if a soil-plug is not achieved when driving the H-piles.

Consideration should be given to “perched” pile caps within the embankment fill to reduce subexcavation and dewatering requirements, although settlement due the embankment will need to be assessed and mitigated during detail design.

The following axial geotechnical resistances may be used for preliminary design of the abutments at both bridges:

Pile Type	Estimated Pile Tip Elevation <sup>1</sup> (m)	Approximate Pile Length <sup>2</sup>	Soil Strata Near Pile Tip Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance <sup>3,4</sup>
HP 310x110	198 m to 201 m	24 m to 27 m	“100-blow” silt and clayey silt to clayey silt-silt (till)	1,300	1,300
HP 360x108				1,500	1,500
324 mm dia. tube pile (min. 9.5 mm thick)				1,000	1,000
406 mm dia. tube pile (min. 9.5 mm thick)				1,600	1,600

Notes:

1. Assuming piles are driven approximately 1.5 m to 2 m into “100-blow” till or silt deposits.
2. Measured from approximate ground surface at closest borehole location.
3. For 25% of settlement independent of any settlements induced by surrounding grade changes / embankment loading
4. Resistance values assume single pile and do not take into account pile group efficiency.

The estimated factored ultimate geotechnical resistances provided above are calculated on both shaft and tip resistances and assume the piles have had sufficient time to “set-up” and allow pore pressures to dissipate after initial driving in order to achieve the design geotechnical resistances.

For preliminary design, driven steel piles spaced at or more than three 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 three 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 ( $R_A$ )
Equal to or greater than 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 10% of the piles or two piles at each foundation element (whichever is greater) in each stage of construction.

In order to optimize the design and reduce the risk of piles not achieving the design geotechnical resistance at the design tip elevation during construction, it is recommended that MTO and/or the design-builder consider a combination of the following options:

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

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

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

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

#### **6.4.2.2 Drilled Shafts (Caissons)**

Drilled shafts (caissons) founded within the "100-blow" silt and clayey silt-silt till deposits are considered feasible for supporting the abutment foundations for both bridges.

Although not confirmed during the current investigation, the presence of potential pockets of gravel, cobbles and/or boulders should be anticipated within the fill and sandy clayey silt soils (suggested by the auger grinding and "100-blow" SPT "N" values recorded on the borehole records within the fill and clayey silt soils) and will need to be investigated further during detail design.

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

Caisson Diameter	Estimated Caisson Base Elevation <sup>1</sup>	Approximate Caisson Length <sup>2</sup>	Soil Strata near Caisson Base Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance <sup>3,4</sup>
0.9 m	198 m to 201 m	24 m to 27 m	"100-blow" silt or clayey silt to clayey silt-silt (till)	2,200	2,200
1.2 m				3,800	3,800
1.5 m				5,900	5,900

## Notes:

1. Assuming caissons are founded approximately 1.5 m to 2 m into "100-blow" till material
2. Measured from existing ground surface.
3. For 25 mm of settlement independent of any settlements induced by surrounding grade changes / embankment loading.
4. Resistance values assume single caisson and do not take into account caisson group efficiency.

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

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

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

If caisson foundations are adopted for support of any of the foundation elements, a temporary liner is likely required (at least within the upper zone) to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils and cohesive soils containing silt and sand interlayers. 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 capacities have both a shaft friction and end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the drilled shaft. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection device (SID) or a shaft quantitative inspection device (SQUID); because of the likelihood of water with entrained sediment or the use of polymer slurry, it is recommended that SQUID testing be specified on some or all caissons to demonstrate and verify base cleaning. Should the inspection indicate that loosened or compressible material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

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

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

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

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

#### **6.4.2.3      *Continuous Flight Auger Piles***

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

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

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

For preliminary assessment, a 0.4 m to 0.6 m diameter CFA pile, approximately 24 m to 27 m long and founded within the 100-blow soils (tip at Elevation 198 m to 201 m) is estimated to have a factored geotechnical resistance at Ultimate Limit State ranging from about 1,600 kN to 1,900 kN. The factored geotechnical resistance at Serviceability Limit State will depend on installation methods and interpretation of pre-production pile load tests.

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

- Advanced site-specific investigation during detail design to confirm or adjust axial geotechnical resistances for design based on the degree of understanding, and/or

- Advanced static pile load test as per ASTM D-1143, bi-directional static load (“Osterberg Cell”) test (CFEM, 2006), or Statnamic Load Test (CFEM, 2006).

CFA installation should be in general accordance with OPSS.PROV 903 (Deep Foundations), as applicable, and a special provision will need to be prepared by the design-builder or contractor (and accepted by MTO) to address the requirements for supply and installation of CFA piles including a detailed work method to prevent overexcavation, ensure proper placement of concrete / grout and prevention of heave / loosening of soils at the base and collapse of soils along the shaft during auger removal, and quality control testing. Non-destructive post-construction pile integrity testing in selected CFA piles should be included in the future contract specifications and is recommended to verify the integrity of the concrete / grout given the installation method, groundwater conditions, presence of saturated cohesionless soils, and specialized installation methods to counterbalance the hydrostatic pressures. Pre-production and production pile load testing will be required (e.g., pile load (proof) tests combined with high strain dynamic testing) prior to finalizing design and during construction.

#### **6.4.2.4 Resistance to Lateral Loads**

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

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

For preliminary design, the resistance to static lateral loading in front of the piles / caissons may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$  (kPa/m), is based on the following equations (CFEM, 2002 as referenced in CHBDC, 2006):

For non-cohesive soils:

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

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



For cohesive soils:

$$k_h = \frac{67S_u}{B} \quad \text{Where } S_u \text{ is the undrained shear strength of the soil (kPa); and } B \text{ is the pile / caisson diameter or width (m).}$$

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

It is understood that an integral abutment design may be considered. Where the integral abutment design includes the installation of 3 m long Corrugated Steel Pipe (CSP) liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-pile will be generally free to flex and move laterally within the limits of the CSP.

The following values of  $n_h$  and  $S_u$  may be assumed in the structural analyses for a single vertical pile or caisson, using the interpreted stratigraphic conditions from the boreholes. The range in the values reflect the variability of the subsurface conditions, the soil properties and groundwater level, and the approximate nature of the linear-elastic subgrade reaction analysis. The groundwater level is assumed to be about 1.5 metres below ground surface.

Soil Unit	$n_h$ (kPa/m)	$S_u$ (kPa)
New Granular Fill (Granular 'A' or 'B' Type II)	40,000 – 50,000	-
Loose Sand within CSP (if applicable)	1,500 - 2,500	-
Compact to Dense Sand and Silt (Existing Fill)	15,000 – 20,000	-
Loose to Compact Interlayered Gravelly Sand, Clayey Sand, Sand and Gravel (Possible Fill)	5,000 – 15,000	-
Stiff to Hard Clayey Silt to Clayey Silt-Silt (Existing Fill)	-	50 - 100
Stiff to Hard Clayey Silt to Clayey Silt-Silt	-	100 - 200
Firm to Hard Silty Clay	-	75 - 150
Compact to Very Dense Sandy Silt to Silt	15,000 - 25,000	-
Hard Clayey Silt to Clayey Silt-Silt (Till)	20,000 - 35,000	175 - 200
Dense to Very Dense Silt to Silt and Sand (Till)	30,000 – 40,000	-
Very Dense "100-blow" Silt	40,000	-

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).

#### 6.4.2.5 Downdrag Loads on Piles / Caissons

Based on the preliminary profile drawings, the proposed approach embankments at the Artesian Industrial Parkway overpass structures are about 8 m high with total settlements in the foundation soils estimated to be approximately

40 mm to 50 mm (see Section 6.6.2). As a result, downdrag loads are not anticipated to be a major concern but must be assessed further during detailed design. Downdrag loads can likely be mitigated by designing piles / caissons to resist the additional load in the structural design and/or reducing downdrag forces by preloading the foundation soil to induce settlements prior to driving piles or installing caissons.

## 6.5 Frost Protection

The spread / strip footing(s) and pile / caisson caps should be founded at a minimum depth of 1.5 m below the lowest surrounding final grade, including any distance measured perpendicular to the sloping ground surface to provide adequate protection against frost penetration (as interpreted from OPSD 3090.101 – *Foundation Frost Penetration Depths for Southern Ontario*).

## 6.6 Approach Embankments

Based on the preliminary profile drawing provided, it is assumed that the approach embankments will be up to 8 m high on the west and east side of both the westbound and eastbound bridges. A 2 m wide mid-height bench should be incorporated into the design of the embankment slopes as required for embankment heights greater than 8 m in height in accordance with OPSD 202.010 (Slope Flattening).

For preliminary design, it is assumed that prior to construction of the new approach embankments, all topsoil, peat/organic soil (although not encountered in the current boreholes), and unsuitable existing fill materials will be stripped from the footprint of the new approach embankments and replaced with suitable granular fill. Based on the borehole information, stripping of unsuitable fill is assumed to range from 1.4 m to 4.1 m below existing ground surface at the approach embankments. Consideration could be given to leaving the existing fill soils in place, although more investigation and analyses will need to be performed during detail design. Additional details regarding embankment construction are provided in Section 6.8.1.

Global stability and settlement analyses were carried out for the maximum embankment height (8 m) anticipated on east and west side of the bridges using the closest borehole information to develop idealized stratigraphy and the foundation engineering parameters summarized below.

Idealized Stratigraphic Unit	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$S_u$ (kPa)	$E'$ (MPa)
New Granular Fill (Granular 'A' or 'B' Type II)	21	36	-	-
Compact to Dense Sand and Silt (Existing Fill)	20	30	-	-
Loose to Compact Interlayered Gravelly Sand, Clayey Sand, Sand and Gravel (Possible Fill)	20	28 - 30	-	-
Stiff to Hard Clayey Silt to Clayey Silt-Silt (Existing Fill)	19	30	50 - 100	-
Stiff to Hard Clayey Silt to Clayey Silt-Silt	20	32	100 - 200	25 - 75
Firm to Hard Silty Clay	19	30	75 - 150	10 - 20
Compact to Very Dense Sandy Silt to Silt	21	33 - 34	-	50 - 75
Hard Clayey Silt to Clayey Silt-Silt (Till)	20	34 - 37	175 - 200	80 - 150
Dense to Very Dense Silt to Silt and Sand (Till)	21	34 - 37	-	120 - 150
Very Dense "100" blow Silt	21	37	-	200

For the preliminary analyses, the groundwater elevation is assumed to be the highest measured water level in the piezometer which was about 1.5 m below ground surface (Elevation 223.5 m).

### 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 level of investigation and typical degree of site understanding, minimum target Factors of Safety of 1.3 and 1.5 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 idealized geometry and results of the stability analyses (modelled using Slide 2 Version 9.017) for the highest embankment on the east side of the bridge (near boreholes AIP-4 and AIP-3) are shown in Figures 1 and 2 for the undrained and drained conditions respectively. The deep existing fill materials encountered in Borehole AIP-4 were modelled to represent the critical section for slope stability. Based on the results, the new approach embankments constructed with suitable granular fill and side slopes no steeper than 2H:1V will have an adequate factor of safety (i.e., greater than 1.3 for short-term “undrained” conditions and greater than 1.5 for long-term “drained” conditions) for global stability. Although not included in the stability model, given the height of the embankment is 8 m, it is recommended that a mid-height 2 m wide bench be incorporated into the final design which will increase the factor of safety against global instability.

### 6.6.2 Settlement

Settlement analyses were carried out for the proposed maximum embankment height (8 m) on the east and west side of the bridge and represent the maximum anticipated values. The settlement analyses assume that any surficial topsoil, organics, and the existing fills or any softened/loosened deposits have been removed and replaced with suitable granular fill.

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 settlement for the east and west approach embankments are presented below, assuming the use of conventional granular fill for construction. The estimated settlements do not account for immediate settlement of the embankment fill itself which is expected to occur during or shortly after construction (within a few months) and would need to be assessed during detail design.

Location	Relevant Boreholes	Proposed Maximum Embankment Height	Estimated Total Settlement (mm)	Estimated Post-Construction Settlement over a 20-Year Period (mm)
East Approach Embankments	AIP-3, AIP-4	8 m	$\delta_{Total} = 40 - 50 \text{ mm}$	$\delta_{20yr} < 25 \text{ mm}$
West Approach Embankments	AIP-2	8 m	$\delta_{Total} = 40 - 50 \text{ mm}$	$\delta_{20yr} < 25 \text{ mm}$

Given that the existing cohesive soils (i.e., clayey silt to clayey silt-silt, silty clay, and clayey silt-silt till) are considered to be over-consolidated and contain seams/interlayers of silt and sand, the majority of the settlement is anticipated

to occur rapidly during or shortly after construction. Based on the preliminary investigation and calculated results above, post-construction settlements are estimated to be within tolerable values and are not anticipated to be a concern at the approach embankments.

If consideration is being given to leaving the existing fill materials in place below existing ground surface, additional investigation and analyses will be required during detail design to estimate the magnitude and time rate of settlement within the fill soils. Given the variability of the fill soils encountered, a preload period of up to about six months is estimated but will require settlement monitoring to confirm settlements have stabilized prior to constructing any settlement sensitive structures and/or installing deep foundations to reduce downdrag loads.

Consideration will need to be given to differential settlements if embankments are to be widened for the future bridge configuration. It is recommended that the full embankment width (to accommodate the future widening and bridge configuration) be constructed during the interim stage to reduce impacts of differential settlement.

### 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 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall in accordance with Figure C6.31(a) of the *Commentary to the CHBDC* (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

## 6.7 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete and/or steel elements (e.g., reinforcing steel) of foundations or related structures buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure can be assessed based on indicators such as the soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride

and sulphate) concentrations. The analytical results for the soil samples submitted for testing at this site are summarized in Section 4.4 and the analytical laboratory test reports are included in Appendix C.

### 6.7.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 ("*Additional requirements for concrete subjected to sulphate attack*") for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S3 (Moderate). Therefore, based on the soil samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

### 6.7.2 Potential for Corrosion

The test results indicate a pH of 7.65 to 7.89 and a resistivity of 2200 to 7200 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability and the resistivity indicates that the soil corrosiveness is generally Very Low to Moderate ( $10,000 \text{ ohm-cm} > R > 2000 \text{ ohm-cm}$ ), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014). Appropriate corrosion protection should be applied to the foundation elements / materials and given that the foundations are located adjacent to the highway shoulder / ditches and will be exposed to de-icing salt, consideration should be given to selection of a "C" type exposure class for any concrete as defined by CSA A23.1 Table 1.

These recommendations are provided as guidance only; the design-builder should take the results of the laboratory testing into consideration for selecting appropriate materials and corrosion susceptibility for design service of the structure foundations and determine the appropriate exposure class and ensure that all aspects of CSA A23.1 Section 4.1.1 "Durability Requirements" are satisfied.

## 6.8 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 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 of about 1.4 m to 4.1 m below ground surface may be required to remove the unsuitable soils at the approach embankments. As previously discussed, consideration can be given leaving the existing fills in place, although the fills will need to be further characterized during detail design to determine the suitability in terms of settlement and stability performance after the new embankments are constructed.

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

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

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

### 6.8.2 Temporary Excavations

In general, temporary excavations up to about 4 m below ground surface will be required for shallow foundations and/or subexcavation and replacement with a granular pad. Temporary excavations could be reduced to about 1.5 m below ground surface (i.e., frost depth) and/or eliminated for pile and caisson caps “perched” within the approach embankments, if the existing fills can be left in place (to be determined during detail design).

All temporary excavations must be carried out in accordance with OPSS.PROV 902 (Excavating and Backfilling) and Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. The existing fill and stiff clayey silt to clayey silt-silt soils (above the groundwater table or effectively dewatered) are classified as Type 3 soils and the very stiff to hard clayey silt to clayey silt-silt is classified as a Type 2 soil. Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes of no steeper than 1H:1V. For Type 2 soils, the excavation may be sloped to within 1.2 m of the bottom of the excavation. Any fills below the groundwater level (if not effectively dewatered) may be classified as Type 4 soils and temporary excavations should be made with side slopes no steeper than 3H:1V.

Where required, temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*) and Special Provision 105S09. The lateral movement of the temporary protection systems must meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities or structures can tolerate this magnitude of deformation. Where existing or relocated utilities/structures are located within the zone of influence of temporary excavations, the owner of the utility/structure should be contacted to check that tolerable levels of movement are designed for and maintained throughout construction. A special provision to include a monitoring plan for adjacent utilities/structures may be required in the future contract documents to check that movement tolerance levels and serviceability of the utilities/structures are maintained throughout and following construction.

### 6.8.3 Groundwater / Surface Water Control

The highest groundwater level measured in the standpipe piezometer installed in AIP-3 (east side) was at about El. 223.5 m (about 1.3 m below ground surface). Based on the topography, the groundwater level is anticipated to be slightly higher on the west side consistent with the slope of the natural ground surface and flow of the watercourse through the existing culvert below Artesian Industrial Parkway at the site.

The excavations for shallow foundations (if considered) or subexcavation and replacement of the existing fills are anticipated to extend up to 4.1 m below ground surface and will be about 2.8 m below the measured groundwater level. Depending on the extent of the excavation and groundwater level at the time of construction, dewatering operations would likely require ditching and pumping from filtered sumps within or adjacent to the excavations and may require shallow wells or well points in areas where cohesionless fills (and utilities) are present. 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 backfilled to at least 0.5 m above the static groundwater level.

Dewatering operations should be in accordance with OPSS.PROV 517 (*Dewatering*) as referenced in OPSS.PROV 902 (*Excavation and Backfilling – Structures*). Inclusion of a special provision for foundation dewatering will need



to be considered in the future contract documents 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 on the west (i.e., high) side and through the existing culvert at the site must be properly diverted / controlled such that the integrity of any foundation subgrade is maintained.

Temporary erosion protection on exposed cuts / fills must be in accordance with OPSS.PROV 804 (Temporary Erosion Control).

#### **6.8.4 Obstructions During Pile/Caisson Installation**

During pile installation through the glacially derived soils, including the existing fills and upper portion of the clayey silt to clayey silt-silt layer that recorded several SPT "N" values of 100, there is a risk of encountering obstructions such as pockets of gravel or cobbles and boulders. It is recommended that steel H-piles or tube piles be reinforced and protected from damage with appropriate driving shoes as per OPSD 3000.100 (Steel H-Pile Driving Shoe) or 3001.100 (Steel Tube Driving Shoe) or equivalent. Pre-augering may be considered to reduce the risk of piles "hanging up" or deflecting on potential "100-blow" stratum and if considered, the design geotechnical resistances provided must be reviewed and revised as necessary during detail design. Caisson installation equipment must be capable of penetrating and/or removing obstructions as required. The feasibility of CFA piles will need to be re-evaluated during detail design if significant obstructions are anticipated based on the results of additional investigation at the site.

#### **6.8.5 Vibration / Settlement Monitoring During Construction**

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

Commercial buildings and associated structures are anticipated to be located as close as 50 m from the proposed abutment locations. A Peak Particle Velocity (PPV) threshold of 25 mm/s is generally considered applicable for vibration impacts on private structures and wells. For utilities in the vicinity of the site, a PPV threshold of 10 mm/s is generally recommended.

At this preliminary stage, pre- and post-construction condition surveys and vibration monitoring are recommended at and near structures located within a 100 m radius of any piling operations (including any critical utilities that are sensitive to vibrations and ground movements), and it would be prudent to carry out such monitoring during critical stages of the construction, such as during pile driving operations and during installation of any temporary shoring. For due diligence purposes, supplemental settlement / ground movement monitoring should also be considered where structures / utilities are assessed to have a low tolerance to movement. A special provision which describes

the requirements for vibration and settlement monitoring on adjacent structures / utilities is recommended to be included in the future contract documents during detail design.

## 6.9 Recommendations for Additional Work

The preliminary foundation recommendations provided in this report are based on the limited available subsurface information in the three boreholes advanced near the proposed structures. Additional subsurface investigation is recommended to be carried out during detail design to further explore the subsurface soil and groundwater conditions closer the bridge foundation elements (abutments), approach embankments, and any associated retaining walls.

In particular, additional exploration is recommended at the northwest quadrant of the site where no information was obtained during the current investigation due to permission to enter limitations, and the southeast quadrant of the site near the existing watercourse which flows into the swampy area further south and east of the site. The swampy area located about 150 m east of the site should be explored as it will be a high fill embankment area located between the Artesian Industrial Parkway bridges and the CN rail bridges further to the east. Specialized (portable) drilling equipment and/or construction of temporary access platforms is likely required for the foundation investigation within and near southeast quadrant of the site and the swampy area located within the high fill area further east. Consideration should be given to advancing Cone Penetration Tests and/or pressuremeter tests to refine the settlement estimates and further characterize the firm to hard clayey silt to clayey silt-silt and silty clay layers encountered that are challenging to sample with conventional push equipment. Consideration could also be given to using specialized piston samplers (as opposed to conventional thin-walled Shelby tube extraction methods) as an attempt to collect less disturbed samples of the clayey deposits containing silt/sand seams and additional consolidation tests performed accordingly.

The presence, thickness and competency of the existing fills encountered within the footprint of the approach embankments should be further investigated to assess the option of leaving the fills in place to support approach embankments and/or appropriate mitigation options provided such that the performance of the foundations and high fill embankments is acceptable.

After more detailed foundation investigation is complete, the global stability of the approach embankments and any retaining walls will need to be checked and the magnitude of foundation settlements and any mitigation measures will need to be reassessed. When more details are known on actual loading conditions, the foundation types, sizes, and geotechnical resistances will need to be checked and revised as necessary. The feasibility of spread footings and CFA piles will need to be checked when further site-specific design information is available.

The use of GSC 5<sup>th</sup> Generation or 6<sup>th</sup> Generation seismic hazard maps to define the Site Class should be confirmed for detail design. Geophysics testing, such as Multi-Channel Analysis of Surface Waves (MASW) or Vertical Seismic Profiling (VSP), may provide a more favourable Site Class designation, and such testing can be considered during detail design. Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*.

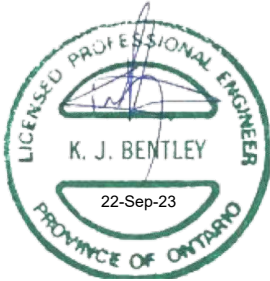
The existing standpipe piezometer (installed in Boreholes AIP-3) should be maintained operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometer will need to be decommissioned in accordance with Ontario Regulation 903 (as amended). Additional piezometers (particularly on the north and west side of the bridges) should be installed near the proposed foundation elements to provide the necessary information to assess dewatering requirements.

## 7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Priyanka Talukdar, a geotechnical EIT with WSP Golder, and reviewed by Mr. Kevin Bentley, P.Eng., Senior Principal and MTO Foundations Designated Contact with WSP Golder. Ms. Lisa Coyne, P.Eng., Fellow and MTO Foundations Designated Contact with WSP Golder, also conducted a technical and quality control review of the report.

## Signature Page

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PT/MCK/KJB/LCC/al

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### ASTM International

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

### Canadian Standards Association (CSA):

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

### Commercial Software:

Settle3 (Version 5.015) by Rocscience Inc.

Slide2 (Version 9.017) by Rocscience Inc.

### Ontario Provisional Standard Drawing:

- |               |   |
|---------------|---|
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario  |
| OPSD 3101.150 | Walls, Abutments, Backfill, Minimum Granular Requirements |
| OPSD 3121.150 | Walls, Retaining, Backfill, Minimum Granular Requirements |

### Ontario Provincial Standard Specifications (OPSS)

- |               |   |
|---------------|---|
| OPSS.PROV 206 | Construction Specification for Grading  |
| OPSS.PROV 501 | Construction Specification for Compacting   |
| OPSS.PROV 517 | Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation |
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems   |

OPSS.PROV 803	Construction Specification for Vegetative Cover
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.PROV 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
Special Provision 109F57	Amendment to OPSS.PROV 903

**Ontario Regulations**

Ontario Regulation 903 Wells (as amended)

Ontario Regulation 213 Construction Projects (as amended)

**Ministry of Transportation, Ontario**

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

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

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

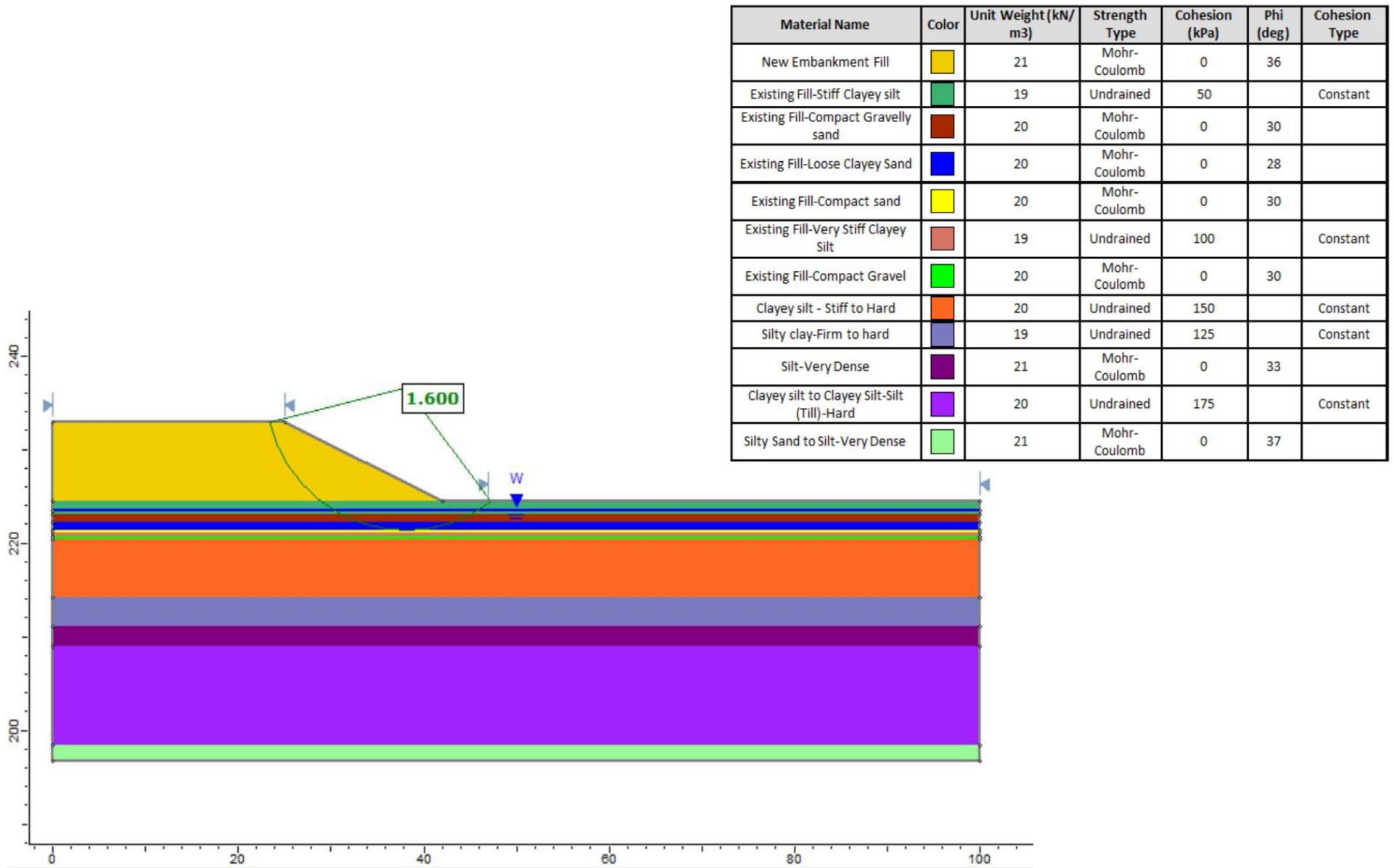
Guideline for MTO Foundation Engineering Services, Version 3, dated April 2022

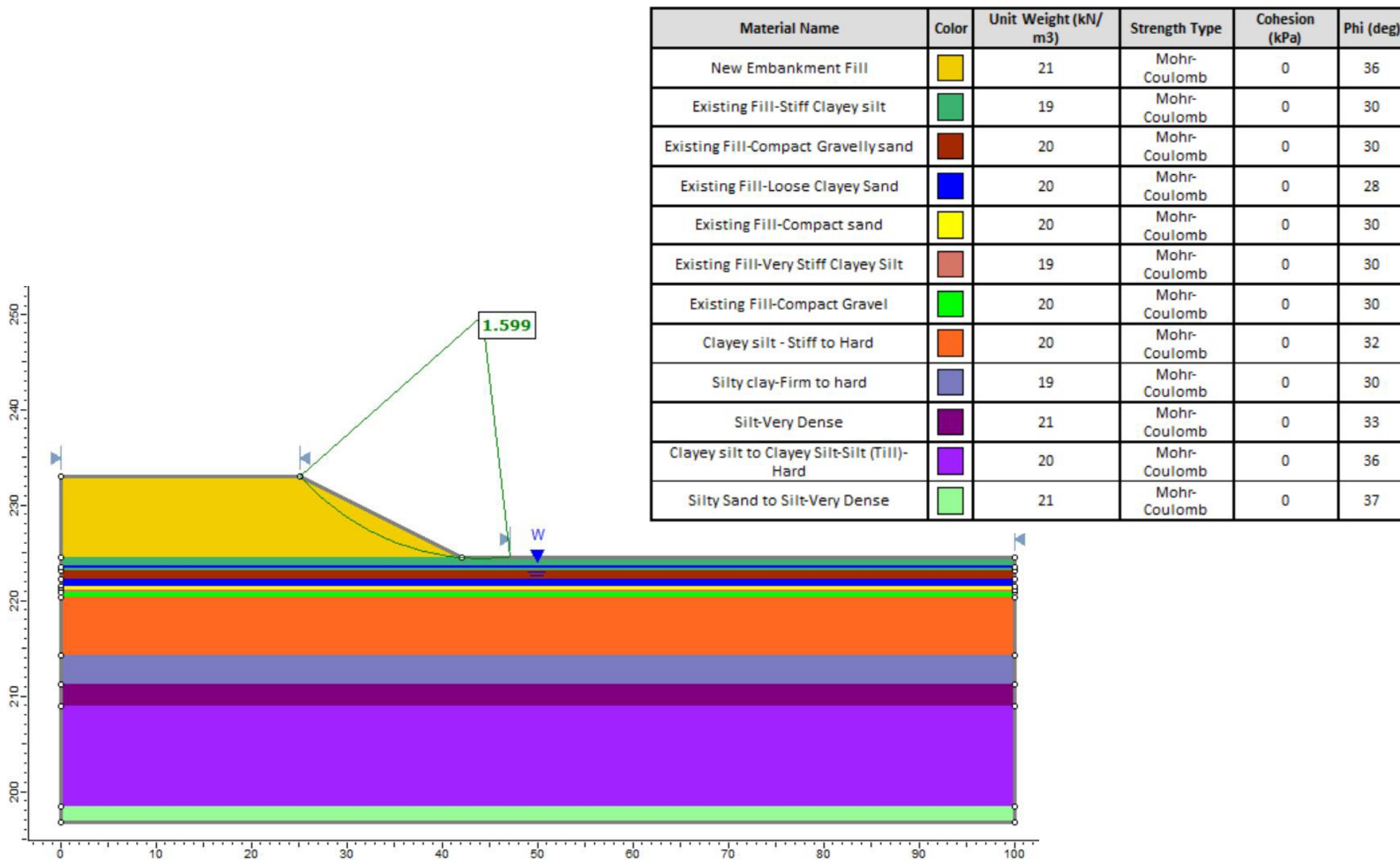
Table 1: Comparison of Foundation Alternatives – Artesian Industrial Parkway Overpass Bridges

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Strip / Spread footings founded on native stiff to hard clayey silt to clayey silt-silt	■ Marginally Feasible	<ul style="list-style-type: none"><li>■ Conventional construction</li><li>■ Relatively competent soils may provide adequate geotechnical resistance depending on actual bridge loads</li></ul>	<ul style="list-style-type: none"><li>■ Relatively low geotechnical resistance compared to deep foundations</li><li>■ Excavation of unsuitable soils to about 4 m depth is required to reach competent founding stratum.</li><li>■ Extensive dewatering (excavations up to 3m below water level) is required to allow for excavation and construction of footings in dry conditions and maintain stable foundation subgrade.</li><li>■ Temporary protection systems may be needed if Artesian Industrial Parkway is to remain open during construction.</li><li>■ Does not allow for conventional integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Lower cost than deep foundations although additional costs for dewatering and temporary protection systems will need to be considered</li></ul>	<ul style="list-style-type: none"><li>■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual abutment locations.</li><li>■ Risk of deeper excavation and increased dewatering and/or temporary shoring efforts.</li><li>■ Significant amount of buried utilities exist along and adjacent to Artesian Industrial Parkway (likely near or within foundation footprint) which may increase founding depth and/or result in utility relocations.</li><li>■ Potential swelling/expansive soils may be encountered near founding level.</li></ul>
“Perched” abutment spread footings founded on a compacted granular pad within approach embankments	■ Marginally Feasible	<ul style="list-style-type: none"><li>■ Conventional construction</li><li>■ Granular pad can be constructed within approach embankment for abutment locations.</li><li>■ Founding level can easily be adjusted within approach embankment.</li><li>■ Depth of abutment wall stems can be reduced compared to founding on native soils.</li><li>■ Increased geotechnical resistance compared to shallow foundation on native deposits.</li></ul>	<ul style="list-style-type: none"><li>■ Relatively low geotechnical resistance compared to deep foundations</li><li>■ Subexcavation and replacement of unsuitable soils to about 4 m depth to reach competent founding stratum is required within foundation zone of influence.</li><li>■ Extensive dewatering (excavations up to 3 m below water level) is required to allow for subexcavation and placement and compaction of granular pad in dry conditions and maintain stable subgrade.</li><li>■ Temporary protection systems needed for subexcavation and replacement if Artesian Industrial Parkway is to remain open during construction.</li><li>■ Does not allow for conventional integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Similar costs for spread footings founded on native soil due to subexcavation and dewatering to construct granular pad.</li></ul>	<ul style="list-style-type: none"><li>■ Less competent soils may be encountered at preliminary founding level during detail design investigation at actual abutment locations.</li><li>■ Risk of deeper excavation and increased dewatering and/or temporary shoring efforts.</li><li>■ Lower risk to foundation performance if subgrade soils are wet / disturbed during construction as compacted granular pad will distribute and decrease loads on native soils.</li><li>■ Significant amount of buried utilities exist along and adjacent to Artesian Industrial Parkway (likely near or within foundation footprint) which may increase founding depth and/or result in utility relocations.</li></ul>
Driven Steel H-piles or tube piles driven into “100-blow” soil.	■ Feasible	<ul style="list-style-type: none"><li>■ Conventional construction methods for driven steel pile foundations.</li><li>■ Higher axial resistances available compared to shallow footings.</li><li>■ Pile caps perched within approach embankments can be considered to reduce or eliminate dewatering / subexcavation requirements.</li><li>■ Allows for integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Noise and vibrations to adjacent properties.</li><li>■ Dewatering measures may be required at abutments for the construction of pile caps, unless perched in embankment fill.</li><li>■ Driving shoes and/or thicker pile section may be required to drive into the “100-blow” glacial till soils that may contain cobbles / boulders.</li></ul>	<ul style="list-style-type: none"><li>■ Lower relative cost than drilled shafts (caissons)</li><li>■ Comparable cost to spread footings if dewatering and subexcavation of unsuitable soils can be reduced by designing perched pile caps.</li></ul>	<ul style="list-style-type: none"><li>■ Reduced impact on design if variable near surface soils and utilities are encountered during detailed investigation.</li><li>■ Risk of piles “hanging up” or being deflected from alignment when driving through fill, clayey silt and glacial deposits that may contain obstructions, pockets of gravel or cobbles and boulders.</li></ul>
Drilled Shafts (Caissons) founded within “100-blow” soil.	■ Feasible	<ul style="list-style-type: none"><li>■ Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements.</li><li>■ May be designed to eliminate pile cap and associated temporary excavations / dewatering as the caissons could be cast continuously with structural columns to the underside of the superstructure.</li><li>■ Low noise and low vibrations compared to driven piles</li></ul>	<ul style="list-style-type: none"><li>■ Temporary (or permanent) liner will be required, at least within upper zone, plus special measures such as use of polymer slurry to counterbalance groundwater pressures to reduce risk of loosening / softening of the sides of the drilled shaft and heave / blow-out at base of shaft during drilling and concrete placement (by tremie methods).</li><li>■ Generation and disposal of soils cuttings / slurry during drilled shaft advancement</li><li>■ Does not allow for conventional integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Higher relative cost than shallow foundations.</li><li>■ Higher cost than piles but reduced dewatering / subexcavation costs if cast continuously with structural columns to eliminate pile cap.</li></ul>	<ul style="list-style-type: none"><li>■ Reduced impact on design if variable near surface soils and utilities are encountered during detailed investigation.</li><li>■ Risk of difficulties penetrating through fill, clayey silt and glacial deposits that may contain obstructions, pockets of gravel or cobbles and boulders.</li><li>■ Challenging to inspect the shaft and base of the drilled shafts due to the need for liners, slurry, and tremie concrete methods.</li></ul>

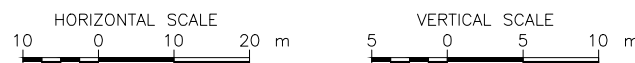
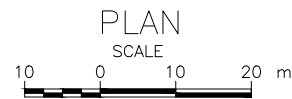


Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Continuous Flight Auger (CFA) piles founded within "100-blow" soil.	<ul style="list-style-type: none"><li>■ Feasible</li></ul>	<ul style="list-style-type: none"><li>■ Offers intermediate geotechnical resistance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





## Drawings



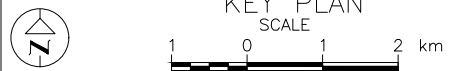
*METRIC*  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No.








BRADFORD BYPASS  
ARTESIAN INDUSTRIAL PARKWAY OVERPASS  
BOREHOLE LOCATIONS AND SOIL  
STRATA

SHEET



### LEGEND

- |   |  |
|---|--|
|  | Borehole – Current Investigation                                   |
|  | Seal   |
|  | Piezometer   |
| N   | Standard Penetration Test Value                                    |
| 16  | Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
|  | WL in piezometer, measured on Feb. 16, 2022                        |
|  | WL upon completion of drilling                                     |

BOREHOLE CO—ORDINATES			
No.	ELEVATION	NORTHING	EASTING
AIP—2	225.0	4887958.8	300326.3
AIP—3	224.8	4887967.0	300371.0
AIP—4	224.5	4887987.8	300376.7

## NOTES

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

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

## REFERENCE

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

Vertical alignment provided in digital format by Aecom, drawing file no. ACAD-X-60636190-C-DES-BBP Mainline Align and Profile.dwg, received September 9, 2022.

Horizontal alignment provided in digital format by Aecom, drawing file no. X-60636190-C-DES-All Alignments.xml, received September 14, 2022.

Design plan provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Overall Plan.dwg, received September 14, 2022.

[illegible]

**APPENDIX A**

# Records of Boreholes

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

## MINISTRY OF TRANSPORTATION, ONTARIO

### PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	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

#### (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

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

Notes: 1  
2

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

PROJECT 19136074

## RECORD OF BOREHOLE No. AIP-2

Sheet 1 of 3

METRIC

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

LOCATION N 4887958.8; E 300326.2 NAD83 / MTM Zone 10 (LAT. 44.131611; LONG. -79.555908)

ORIGINATED BY MTI

DIST Central HWY BBP - AIP

BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary and casing

COMPILED BY PT

DATUM CGVD28 Surface Elevation:225.0 m

DATE Nov 25, 2021 - Dec 01, 2021

CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT	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>						
0.0 225.0 0.0	Asphalt (50 mm) SAND (SP), trace fines, trace gravel (FILL) Dense Brown Moist		1	SS	34			20	40	60	80	100	20	40	60						
224.3 0.7	CLAYEY SILT-SILT (CL-ML), trace to some sand, trace gravel (FILL) Stiff to Hard Brown Moist		2	SS	12		224														
			3	SS	16		223														
	- 2.4 m: - auger grinding noted		4	SS	34																
222.0 3.0	CLAYEY SILT (CL) Very Stiff to Hard Brown to Grey Moist - 3.0 to 3.2 m: - brown in colour		5	SS	18		222														
			6	SS	16		221														
			7	SS	16		220											0	0	50	50
	- 6.2 to 6.6 m: - Contains silt seams / layers (upto 50 mm thick)		8	SS	20		219														
							218														
	- 8.1 m: - Contains silt seams		9	SS	35		217														
216.4 8.6	SILTY CLAY (CI), trace sand Very Stiff Grey Moist		10	SS	17		216														

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. AIP-2

Sheet 2 of 3

METRIC

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

LOCATION N 4887958.8; E 300326.2 NAD83 / MTM Zone 10 (LAT. 44.131611; LONG. -79.555908)

ORIGINATED BY MTI

DIST Central HWY BBP - AIP





BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary and casing

COMPILED BY PT

DATUM CGVD28 Surface Elevation:225.0 m

DATE Nov 25, 2021 - Dec 01, 2021

CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y  kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane	Remoulded	Pocket Pen	Quick Triaxial	Unconfined	W <sub>p</sub>	W	W <sub>L</sub>						
							20	40	60	80	100	20	40	60							
	SILTY CLAY (Cl), trace sand Very Stiff Grey Moist		11	SS	17								○				0	0	30	70	
			12	SS	20																
211.7																					
13.3	SILT (ML), some sand Dense Grey Moist		13	SS	39								○		NP		0	17	79	4	
			14a	SS	49																
209.5			14b																		
15.5	CLAYEY SILT (CL), trace sand to sandy, trace gravel (TILL) Hard Grey Moist																				
			15	SS	50																
207.2																					
17.8	SILT (ML) and sand to SILT (ML), trace sand (TILL) Dense to Very Dense Grey Wet to moist		16	SS	40								○				3	37	49	11	

Continued on Next Page

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

## METRIC

CHECKED BY                      KJB

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

PROJECT 19136074

## RECORD OF BOREHOLE No. AIP-3

Sheet 1 of 3

METRIC

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

LOCATION N 4887967.2; E 300371.1 NAD83 / MTM Zone 10 (LAT. 44.131686; LONG. -79.555347)

ORIGINATED BY MTI

DIST Central HWY BBP - AIP





BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary and casing

COMPILED BY PT

DATUM CGVD28 Surface Elevation:224.8 m

DATE Nov 19, 2021 - Nov 21, 2021

CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y  kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane	Remoulded	Pocket Pen	Quick Triaxial	Unconfined	W <sub>p</sub>	W	W <sub>L</sub>						
0.0	CLAYEY SILT (CL), trace sand, trace gravel, some rootlets (FILL) Stiff Brown Moist		1	SS	11		224														
224.1																					
0.7	SILT (ML) and sand, trace gravel, trace rootlets (FILL) Compact Brown to blackish brown Moist		2	SS	13																
223.4																					
1.4	Sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace sand, trace gravel, Stiff to hard Brown to mottled grey / brown Moist - 2.1 m: -trace organics above 2.1 m bgs - 2.3 m: -resembles glacial till below 2.3 m		3	SS	10		223														
				4	SS	11		222													
				5	SS	21															
				6	SS	100/0.15		221													
				7	SS	100/0.13		220													
219.2																					
5.6	CLAYEY SILT (CL), trace gravel, trace sand Very stiff to Hard Grey Moist  - 8.1 m: -silt seam (25 mm thick)						219														
				8	SS	52		218													
				9	SS	45		217													
							216														
			10	SS	22		215														

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. AIP-3	Sheet 2 of 3	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887967.2; E 300371.1 NAD83 / MTM Zone 10 (LAT. 44.131686; LONG. -79.555347)	ORIGINATED BY	MTI
DIST	Central HWY BBP - AIP	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary and casing	COMPILED BY	PT
DATUM	CGVD28 Surface Elevation:224.8 m	DATE	Nov 19, 2021 - Nov 21, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y	GR	SA	SI	CL	REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <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								
214.6 10.2	CLAYEY SILT (CL), trace gravel, trace sand Very stiff to Hard Grey Moist		11	SS	6		214														
	SILTY CLAY (CI), trace sand Firm to very stiff Grey Moist		12	TO			213														C
212.3 12.5	Sandy SILT (ML), trace gravel Compact to Dense Grey Moist		13a	SS	29												0	1	31	68	
			13b				212														
			14	SS	33		211														
210.0 14.8	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Hard Grey Moist		15	SS	71		210														
							209										0	9	53	38	
			16	SS	40		208														
			17	SS	39		206														
							205														

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. AIP-3	Sheet 3 of 3	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887967.2; E 300371.1 NAD83 / MTM Zone 10 (LAT. 44.131686; LONG. -79.555347)	ORIGINATED BY	MTI
DIST	Central HWY BBP - AIP	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary and casing	COMPILED BY	PT
DATUM	CGVD28 Surface Elevation:224.8 m	DATE	Nov 19, 2021 - Nov 21, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y	GR	SA	SI	CL	REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <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					
201.8	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Hard Grey Moist						204														
			18	SS	107/0.10		203														
							202														
23.0	SILT (ML), some clay, trace sand Very Dense Grey Moist - 23.0 to 24.7 m: -contains sand seams						201														
200.1			19	SS	100/0.13																
24.7	End of Borehole Notes: 1. Hollow stem augers to 2.3 m depth and then switched to mud rotary. 2. Groundwater first encountered at 1.7 m (Elev. 222.2 m) below ground surface before introducing water for mud rotary. 3. Standpipe piezometer (50 mm pipe) installed 1.5 m north of borehole location. Groundwater level measurement(s) in piezometer: Date      Depth (m)      Elev. (m) Dec 23, 2021      1.51      223.3 Feb 4, 2022      1.39      223.4 Feb 8, 2022      1.36      223.4 Feb 16, 2022      1.29      223.5 May 12, 2022      1.43      223.4						200														
							199														
							198														
							197														
							196														
							195														

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



PROJECT 19136074

## RECORD OF BOREHOLE No. AIP-4

Sheet 1 of 3

METRIC

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

LOCATION N 4887987.7; E 300376.7 NAD83 / MTM Zone 10 (LAT. 44.131872; LONG. -79.555277)

ORIGINATED BY MTI

DIST Central HWY BBP - AIP

BOREHOLE TYPE 210 mm Hollow Stem Auger, Mud Rotary and casing

COMPILED BY PT

DATUM CGVD28 Surface Elevation:224.5 m

DATE Nov 23, 2021 - Nov 24, 2021

CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100													
0.0	CLAYEY SILT (CL), trace sand, trace gravel, trace organics (FILL) Stiff Brown Moist		1	SS	8		224														
223.1			2	SS	11																
1.4	Gravelly SAND (SP), trace fines, trace organics (FILL) Compact Brown Moist		3	SS	22		223						○				30	60	8	2	
222.3																					
2.2	CLAYEY SAND (SC), trace gravel, oxidation staining, non-cohesive (Possible FILL) Loose Brown Moist		4	SS	4		222						○								
221.5																					
3.0	SAND (SP), some gravel (Possible FILL) Compact Grey		5a	SS	28																
221.1	Moist		5b				221						○								
220.8	CLAYEY SILT (CL), trace sand, trace gravel (Possible FILL)																				
3.7	Very stiff Brown to grey		6a																		
220.4	Moist																				
4.1	GRAVEL (GP), trace sand (Possible FILL) Compact Grey Moist		6b	SS	21		220						○				3	2	75	20	
	CLAYEY SILT (CL), trace sand, trace gravel Hard Grey Moist		7	SS	47																
							219														
			8	SS	67		218						○								
							217														
			9	SS	58								○								
							216														
			10	SS	54		215														

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. AIP-4	Sheet 2 of 3	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887987.7; E 300376.7 NAD83 / MTM Zone 10 (LAT. 44.131872; LONG. -79.555277)	ORIGINATED BY	MTI
DIST	Central HWY BBP - AIP	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary and casing	COMPILED BY	PT
DATUM	CGVD28 Surface Elevation:224.5 m	DATE	Nov 23, 2021 - Nov 24, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y					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>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100					20 40 60								
214.3 10.2	CLAYEY SILT (CL), trace sand, trace gravel Hard Grey Moist SILTY CLAY (CI), trace sand Very stiff to hard Grey Moist		11	SS	28		214										0	0	37	63	
							213														
	- 12.5 m: -sand seams below a depth of 12.5 m		12	SS	31		212														
211.2 13.3	SILT (ML), trace sand Very dense Grey Moist		13	SS	67		211										0	8	84	8	
							210														
209.0 15.5	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace sand to sandy, trace gravel (TILL) Hard Grey to dark grey Moist		14a	SS	53		209														
			14b																		
							208														
			15	SS	57		207														
							206														
			16	SS	36		205														

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. AIP-4	Sheet 3 of 3	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887987.7; E 300376.7 NAD83 / MTM Zone 10 (LAT. 44.131872; LONG. -79.555277)	ORIGINATED BY	MTI
DIST	Central HWY BBP - AIP	BOREHOLE TYPE	210 mm Hollow Stem Auger, Mud Rotary and casing	COMPILED BY	PT
DATUM	CGVD28 Surface Elevation:224.5 m	DATE	Nov 23, 2021 - Nov 24, 2021	CHECKED BY	KJB

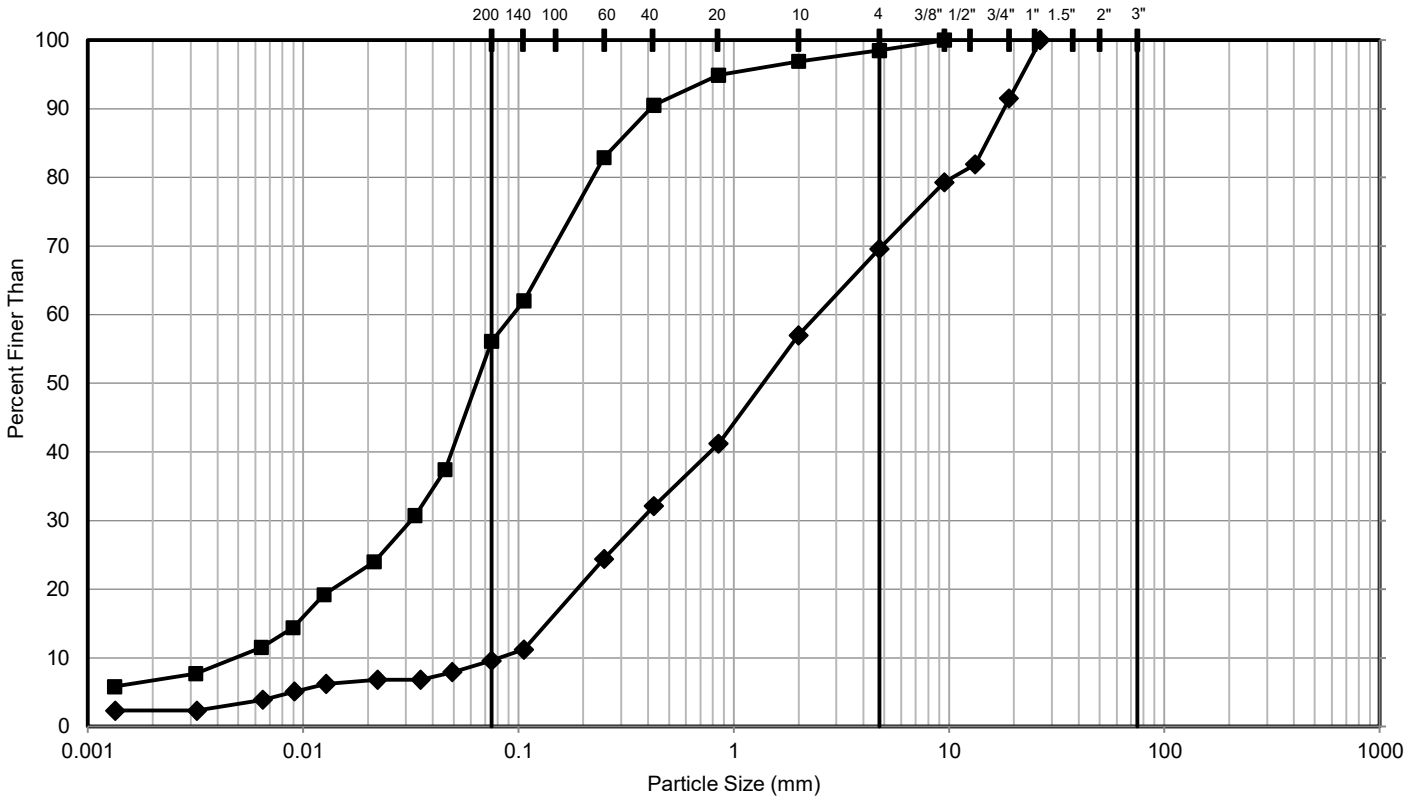
SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y  kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane	Remoulded	Pocket Pen	Quick Triaxial	Unconfined	W <sub>p</sub>	W	W <sub>i</sub>						
								20	40	60	80	100	20	40	60						
	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace sand to sandy, trace gravel (TILL) Hard Grey to dark grey Moist						204														
	- 21.3 m: -increasing sand content below a depth of 21.3 m		17	SS	40		203														
							202														
							201														
			18	SS	100/0.11		200						OH				6	12	64	18	
							199														
198.5																					
26.0	SILTY SAND (SM) to SILT (ML), trace gravel Very dense Grey Moist						198														
							197														
196.8			19	SS	100/0.13																
27.7	End of Borehole Notes 1. Hollow stem augers to 2.4 m below ground surface before switching to mud rotary and casing technique. 2. Groundwater first measured at 1.35 m (Elev. 222.2 m) below ground surface in hollow stem auger.						196														
							195														

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

**APPENDIX B**

# Geotechnical Laboratory Test Results

Grain Size Distribution - Silt and Sand to Gravelly Sand (FILL)

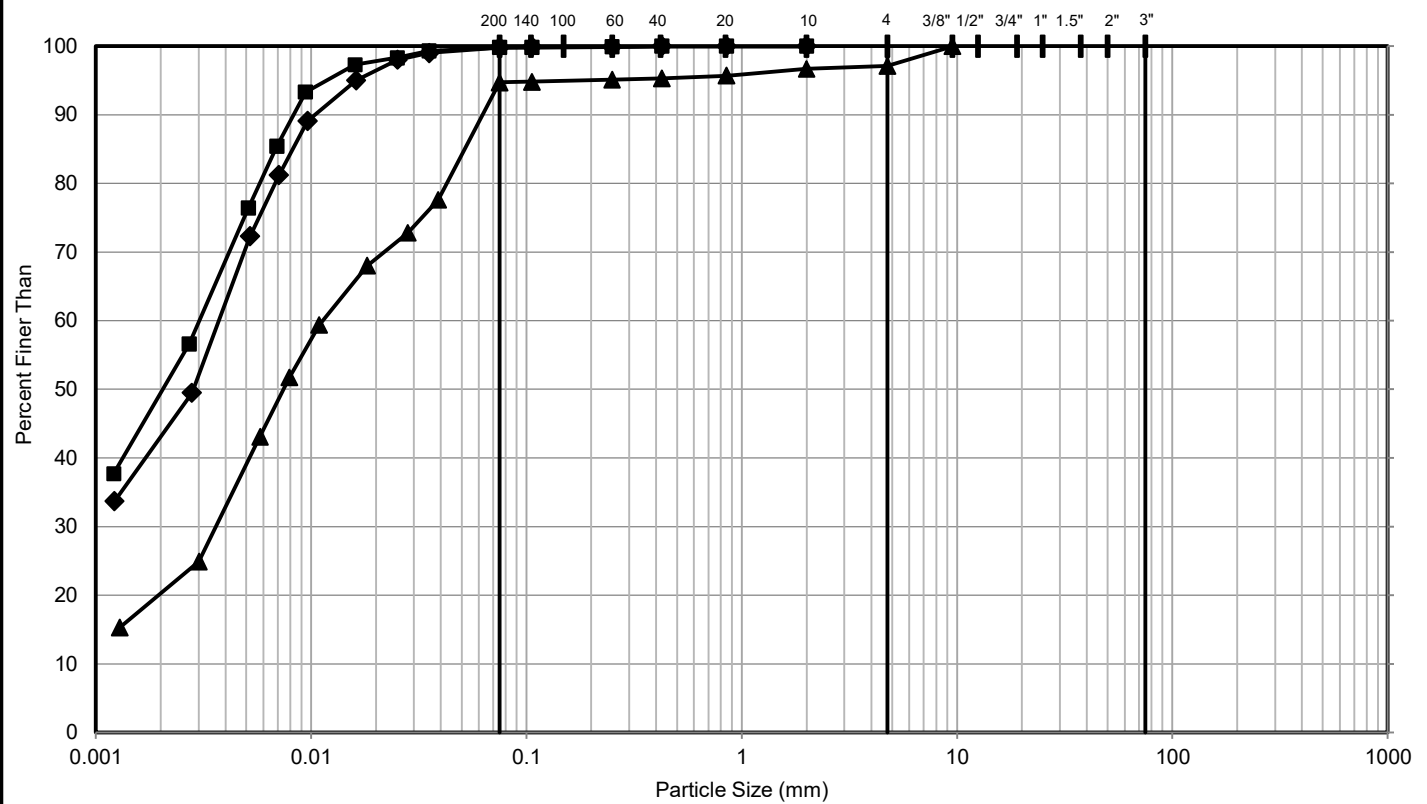


FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-3	2	0.8 - 1.4	224.0 to 223.4
◆	AIP-4	3	1.5 - 2.1	223.0 to 222.4

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Artesian Industrial Parkway	
CONSULTANT	YYYY-MM-DD	2023-03-06	
	DESIGNED	PT	
	PREPARED	PT	
	REVIEWED	KB	
	APPROVED	KB	
		TITLE	
		Grain Size Distribution Silt and Sand to Gravelly Sand (FILL)	
PROJECT NO.		CONTROL	REV.
19136074		1000	0
		FIGURE	
		B1	

Grain Size Distribution - Clayey Silt



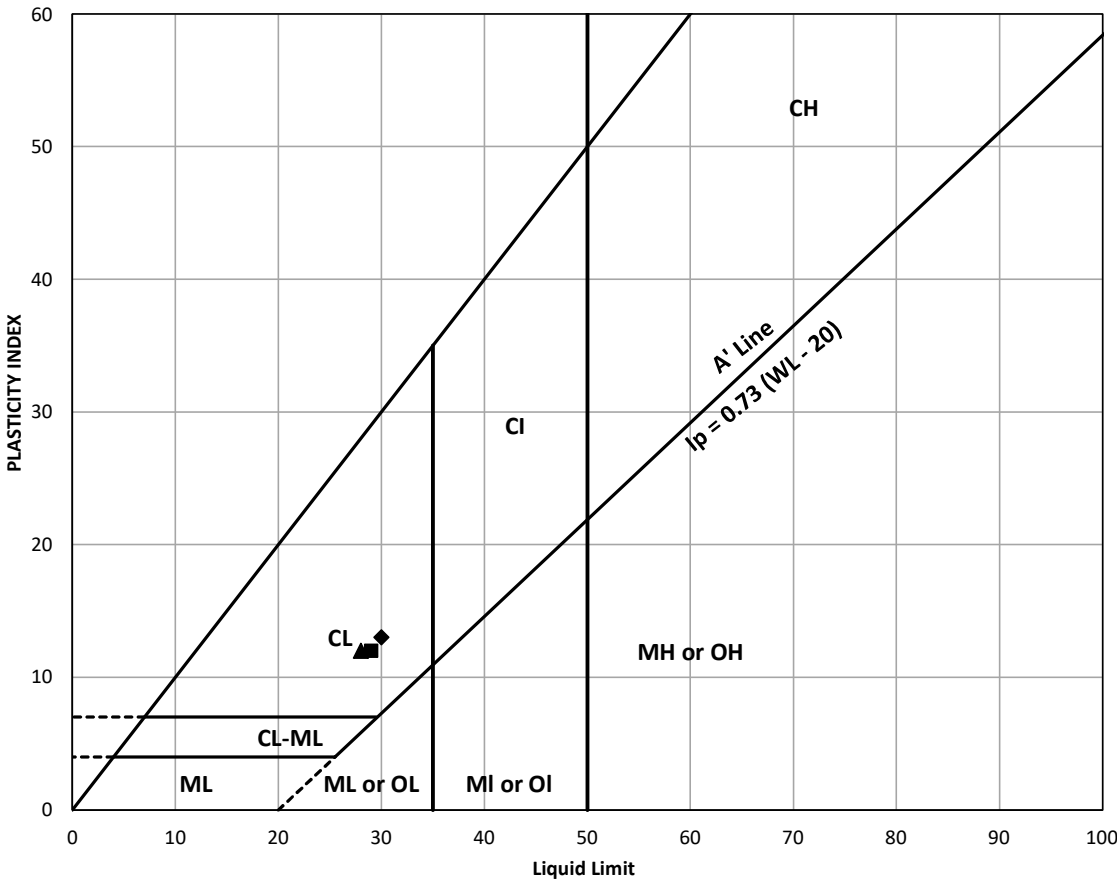
FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-2	7	4.6 - 5.2	220.4 to 219.8
◆	AIP-3	9	7.6 - 8.2	217.2 to 216.6
▲	AIP-4	6b	4.1 - 4.4	220.4 to 220.1

CLIENT			PROJECT			
AECOM / MTO			Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT	YYYY-MM-DD	2023-03-06	TITLE			
	DESIGNED	PT	Grain Size Distribution			
	PREPARED	PT	Clayey Silt			
	REVIEWED	KB				
	APPROVED	KB				
			PROJECT NO.	CONTROL	REV.	FIGURE
			19136074	1000	0	B2

PATH: https://golderasociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Artesian Industrial Parkway/Appendix B Lab Figures/Working Files | FILE NAME: Atterberg Output MTO.xlsm

Plasticity Chart - Clayey Silt

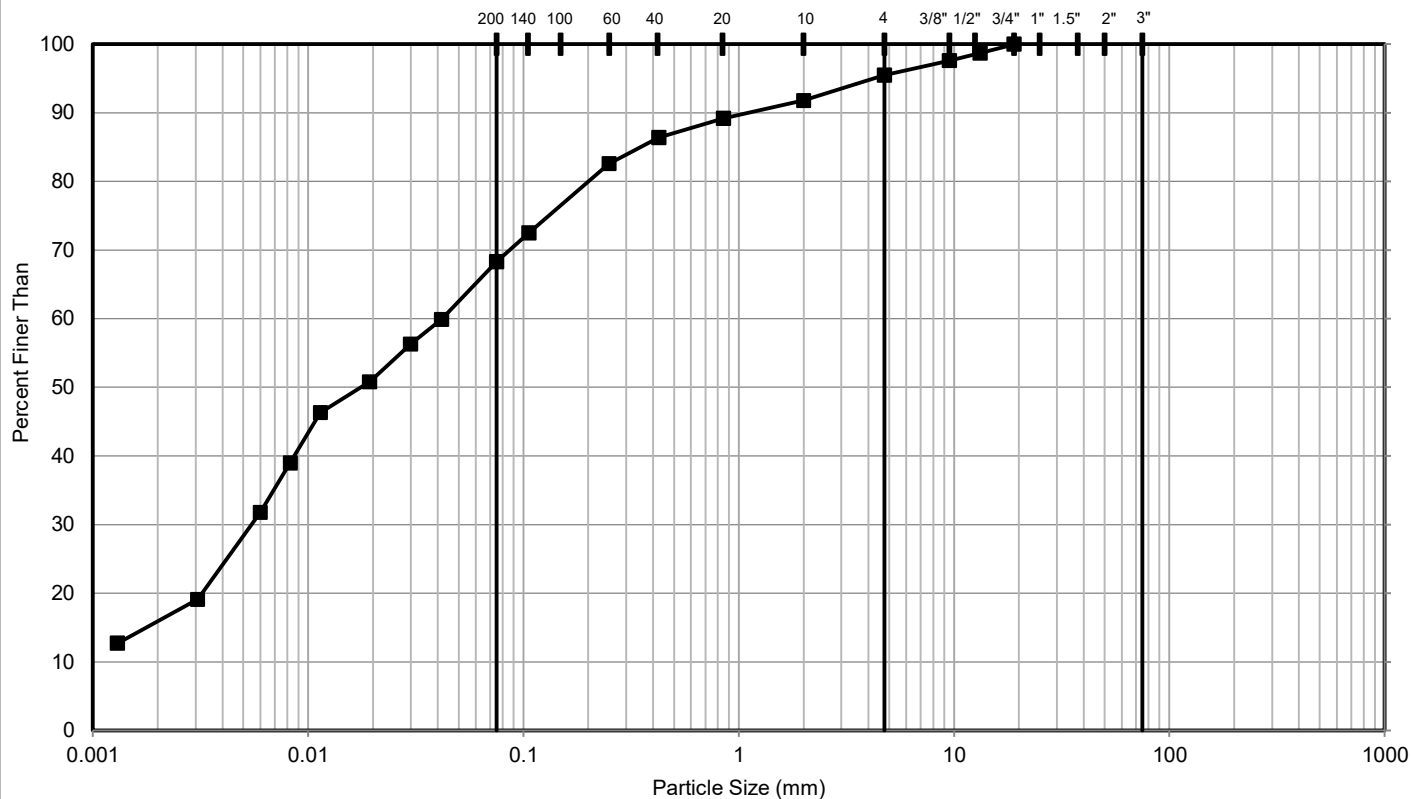


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	AIP-2	6	3.8 - 4.4	24.4	29	17	12
◆	AIP-4	6b	4.1 - 4.4	22.2	30	17	13
▲	AIP-4	9	7.6 - 8.2	19.8	28	16	12

CLIENT				PROJECT			
AECOM / MTO				Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT				TITLE			
				Plasticity Chart - Clayey Silt			
				PROJECT NO.			
				CONTROL			
				FIGURE			
				19136074			
YYYY-MM-DD				1000			
DESIGNED				B3			
PREPARED							
REVIEWED							
APPROVED							



# Grain Size Distribution - Sandy Clayey Silt to Clayey Silt-Silt

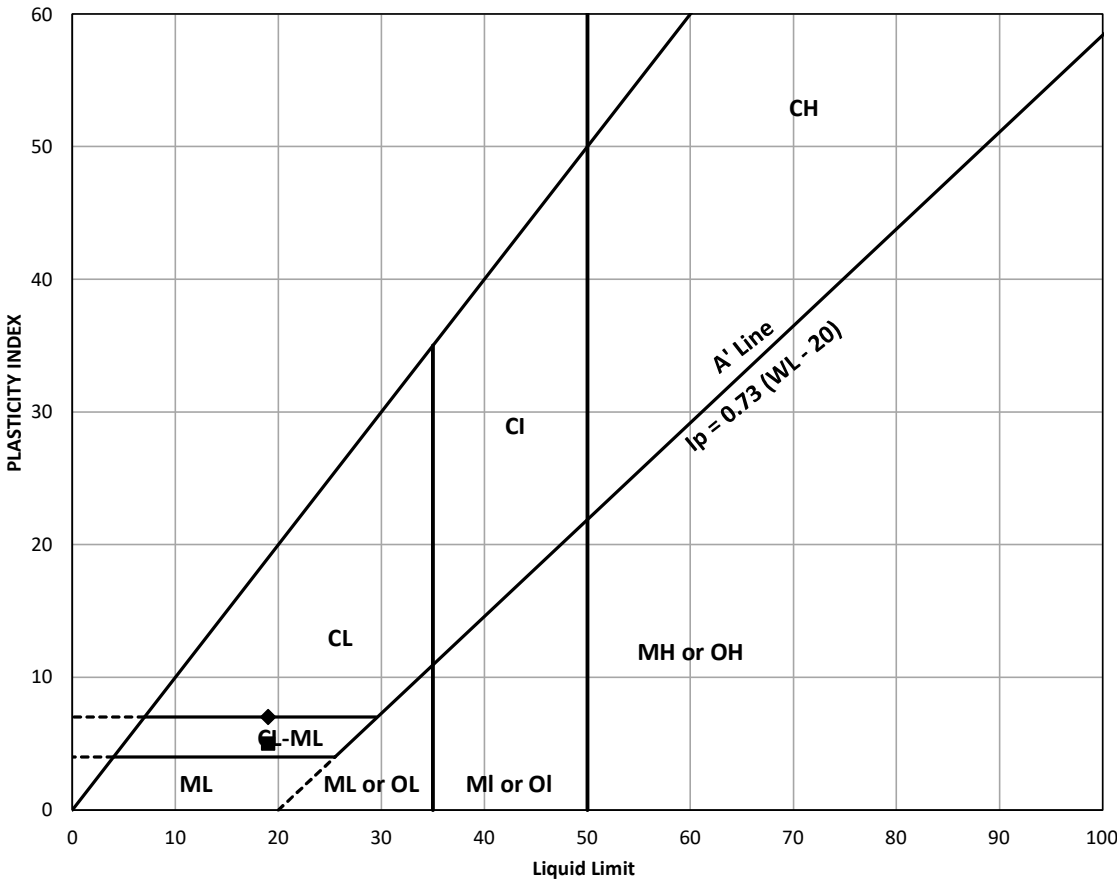


FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-3	4	2.3 - 2.9	222.5 to 221.9

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Artesian Industrial Parkway	
CONSULTANT	YYYY-MM-DD	2023-03-06	
	DESIGNED	PT	
	PREPARED	PT	
	REVIEWED	KB	
	APPROVED	KB	
		TITLE	
		Grain Size Distribution Sandy Clayey Silt to Clayey Silt-Silt	
PROJECT NO.		CONTROL	REV.
19136074		1000	0
		FIGURE	
		B4	

Plasticity Chart - Sandy Clayey Silt to Clayey Silt-Silt

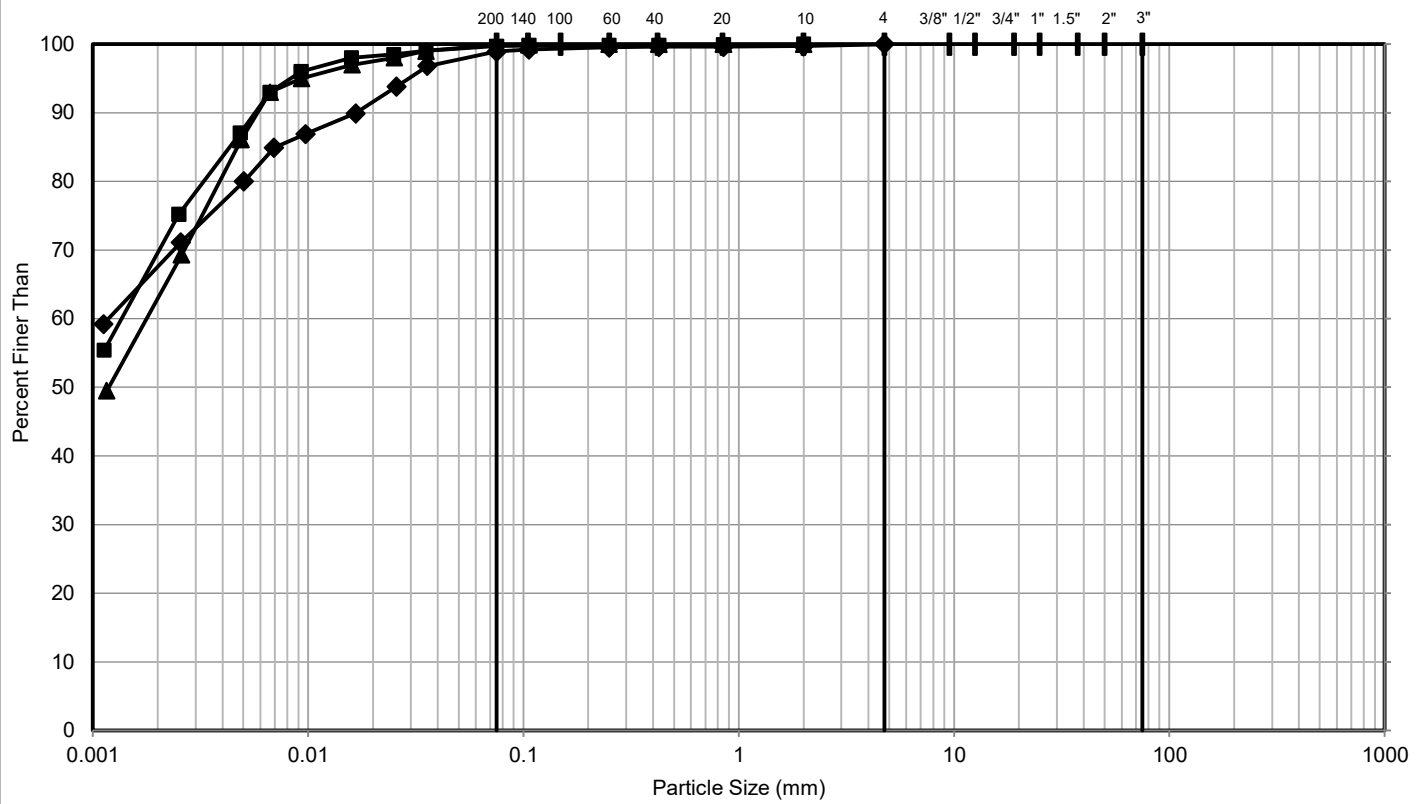


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	AIP-3	4	2.3 - 2.9	13.6	19	14	5
◆	AIP-3	5	3.1 - 3.7	9.8	19	12	7

CLIENT				PROJECT			
AECOM / MTO				Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT		YYYY-MM-DD	2023-03-06	TITLE			
		DESIGNED	PT	Plasticity Chart - Sandy Clayey Silt to Clayey Silt-Silt			
		PREPARED	PT	PROJECT NO.			
		REVIEWED	KB	CONTROL			
		APPROVED	KB	FIGURE			
				19136074		1000	B5

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Artesian Industrial Parkway/Appendix B Lab Figures/Working Files | FILE NAME: Atterberg Output MTO.xlsm

# Grain Size Distribution - Silty Clay

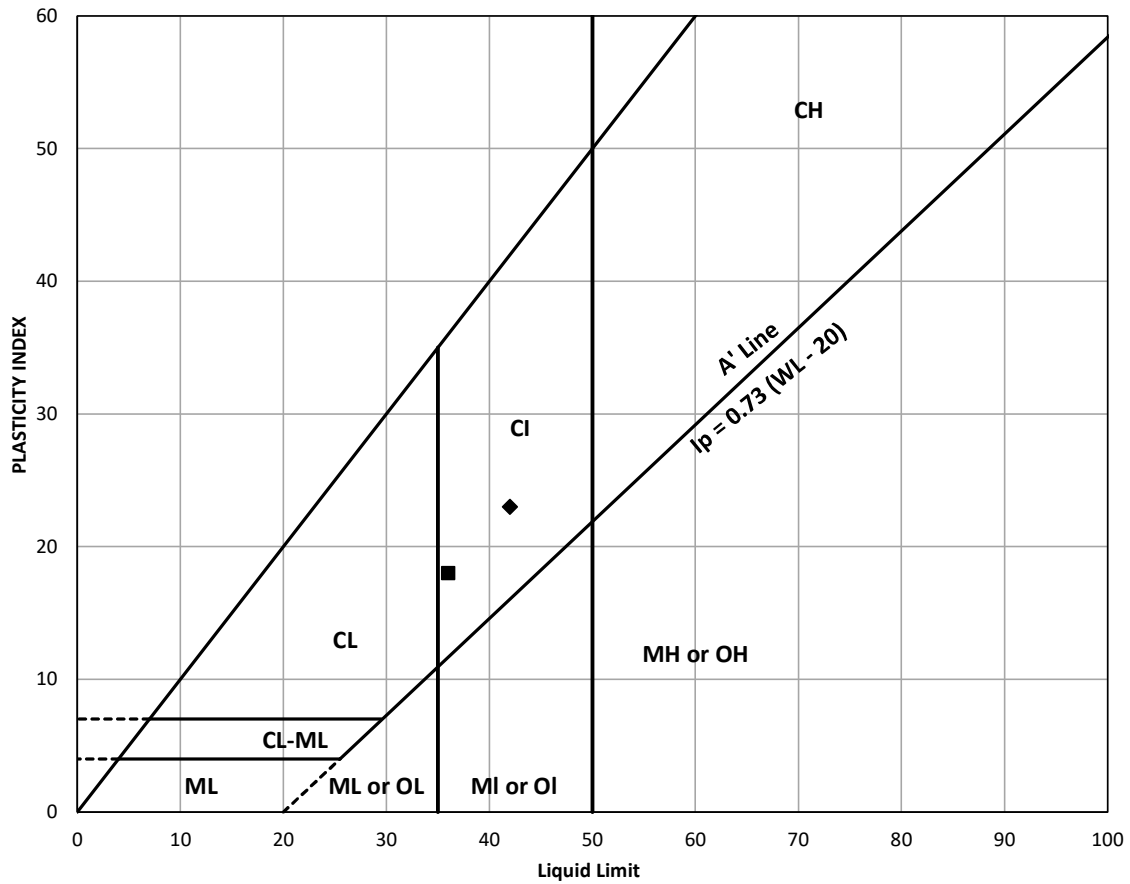


FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-2	11	10.7 - 11.3	214.3 to 213.7
◆	AIP-3	13a	12.2 - 12.5	212.6 to 212.3
▲	AIP-4	11	10.7 - 11.3	213.8 to 213.2

CLIENT			PROJECT			
AECOM / MTO			Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT		YYYY-MM-DD	TITLE			
		2023-03-06	Grain Size Distribution			
		DESIGNED	PT			
		PREPARED	PT			
		REVIEWED	KB			
		APPROVED	KB			
			PROJECT NO.	CONTROL	REV.	FIGURE
			19136074	1000	0	B6

## Plasticity Chart - Silty Clay



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	AIP-2	10	9.1 - 9.8	26.8	36	18	18
◆	AIP-3	13a	12.2 - 12.5	24.6	42	19	23

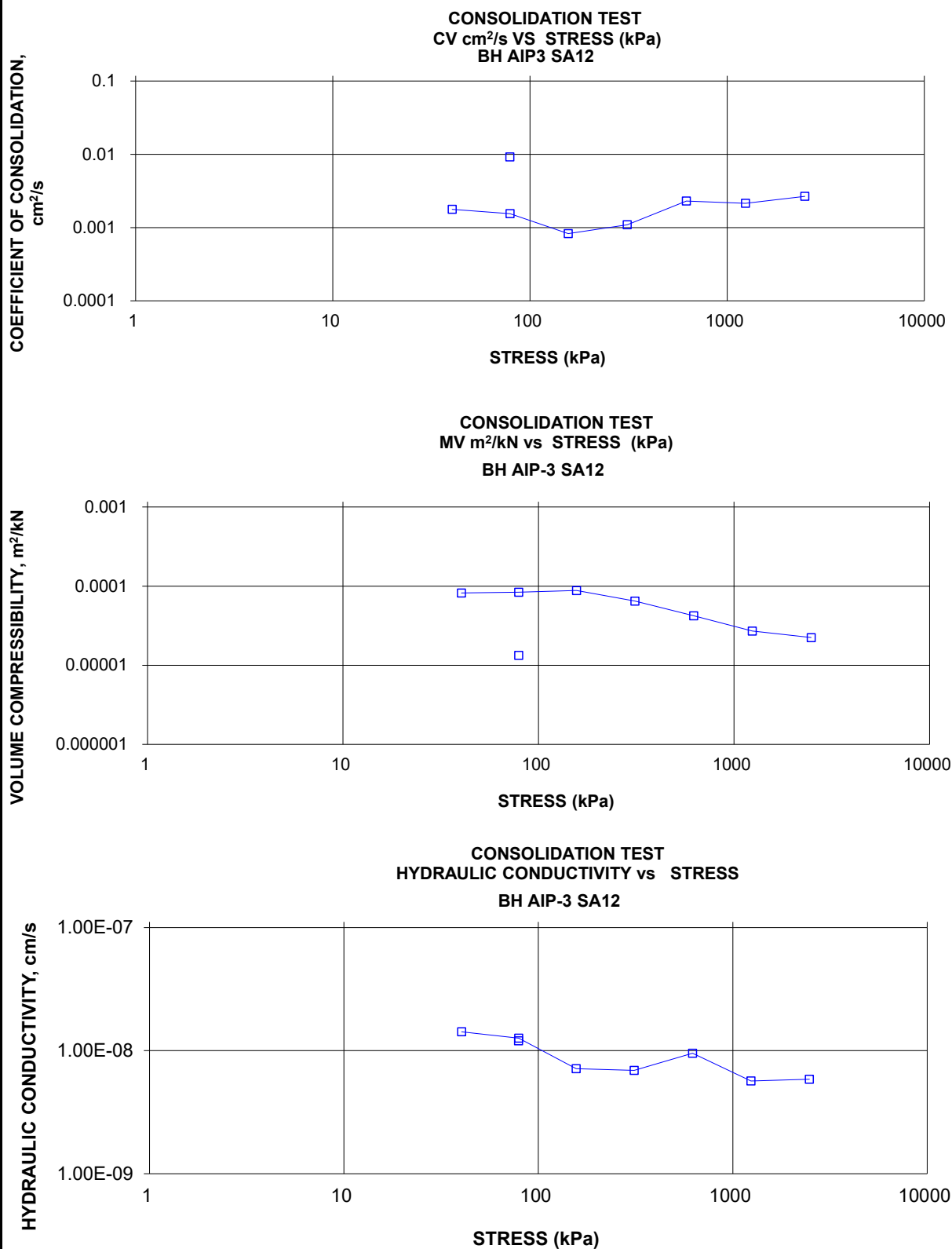
CLIENT				PROJECT			
AECOM / MTO				Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT		YYYY-MM-DD	2023-03-06	TITLE			
		DESIGNED	PT	Plasticity Chart - Silty Clay			
		PREPARED	PT				
		REVIEWED	KB				
		APPROVED	KB				
		PROJECT NO.	19136074	CONTROL	1000	FIGURE	B7

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Artesian Industrial Parkway/Appendix B Lab Figures/Working Files | FILE NAME: Atterberg Output MTO.xlsm

CONSOLIDATION TEST SUMMARY					FIGURE B8		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	19136074			Sample Number	12		
Borehole Number	BH AIP3			Sample Depth, m	11.43-12.04		
TEST CONDITIONS							
Test Type	Laboratory Standard			Load Duration, hr	24		
Oedometer Number	8						
Date Started	01/21/2022						
Date Completed	02/02/2022						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.91		Unit Weight, kN/m <sup>3</sup>	18.84			
Sample Diameter, cm	6.35		Dry Unit Weight, kN/m <sup>3</sup>	14.20			
Area, cm <sup>2</sup>	31.63		Specific Gravity, measured	2.76			
Volume, cm <sup>3</sup>	60.38		Solids Height, cm	1.001			
Water Content, %	32.73		Volume of Solids, cm <sup>3</sup>	31.67			
Wet Mass, g	116.03		Volume of Voids, cm <sup>3</sup>	28.71			
Dry Mass, g	87.42		Degree of Saturation, %	99.7			
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	1.909	0.906	1.909				
6.35	1.913	0.910	1.911				
11.15	1.920	0.917	1.916				
20.93	1.923	0.920	1.921				
40.35	1.930	0.927	1.926				
79.11	1.929	0.926	1.929	86	9.18E-03	1.34E-05	1.20E-08
40.35	1.936	0.933	1.932				
11.16	1.952	0.949	1.944				
40.30	1.947	0.944	1.950	454	1.77E-03	8.18E-05	1.42E-08
79.22	1.941	0.938	1.944	519	1.54E-03	8.36E-05	1.26E-08
156.50	1.928	0.925	1.935	960	8.26E-04	8.81E-05	7.13E-09
311.26	1.909	0.906	1.918	714	1.09E-03	6.47E-05	6.92E-09
620.85	1.884	0.881	1.896	331	2.30E-03	4.23E-05	9.55E-09
1239.81	1.852	0.849	1.868	346	2.14E-03	2.71E-05	5.67E-09
2478.33	1.799	0.796	1.825	265	2.67E-03	2.24E-05	5.85E-09
620.85	1.826	0.823	1.812				
156.50	1.875	0.873	1.850				
40.55	1.921	0.919	1.898				
11.09	1.947	0.945	1.934				
Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 8-17cm from bottom of the tube. Specimen swelled under 40.35kPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.95		Unit Weight, kN/m <sup>3</sup>	18.83			
Sample Diameter, cm	6.35		Dry Unit Weight, kN/m <sup>3</sup>	13.92			
Area, cm <sup>2</sup>	31.63		Specific Gravity, measured	2.76			
Volume, cm <sup>3</sup>	61.59		Solids Height, cm	1.001			
Water Content, %	35.30		Volume of Solids, cm <sup>3</sup>	31.67			
Wet Mass, g	118.28		Volume of Voids, cm <sup>3</sup>	29.92			
Dry Mass, g	87.42						
Prepared By: LH				Golder Associates		Checked By: MM	

# CONSOLIDATION TEST SUMMARY

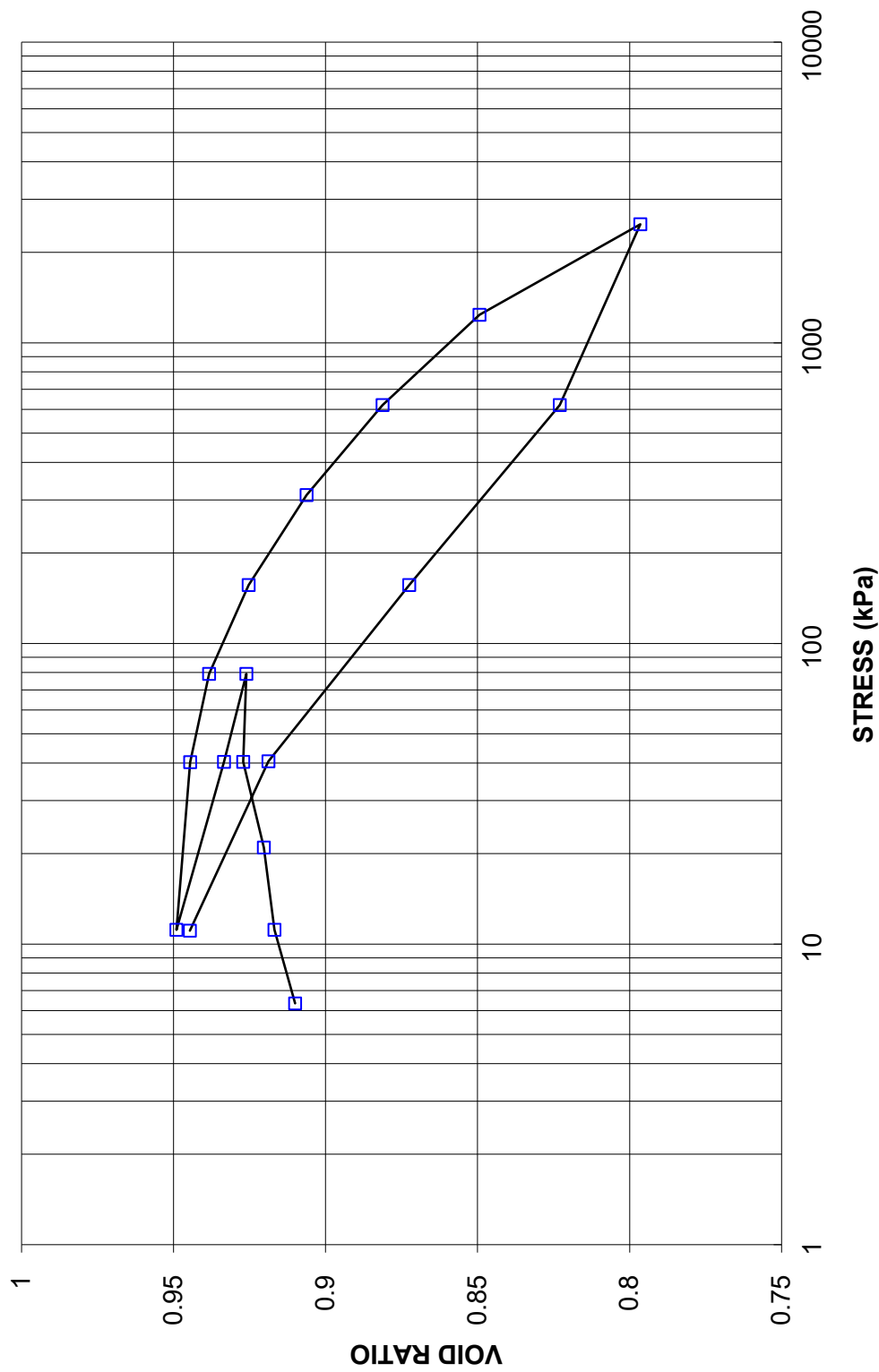
FIGURE B9



CONSOLIDATION TEST  
VOID RATIO VS LOG STRESS

FIGURE B10

CONSOLIDATION TEST  
VOID RATIO vs. STRESS  
BH AIP-3 SA12

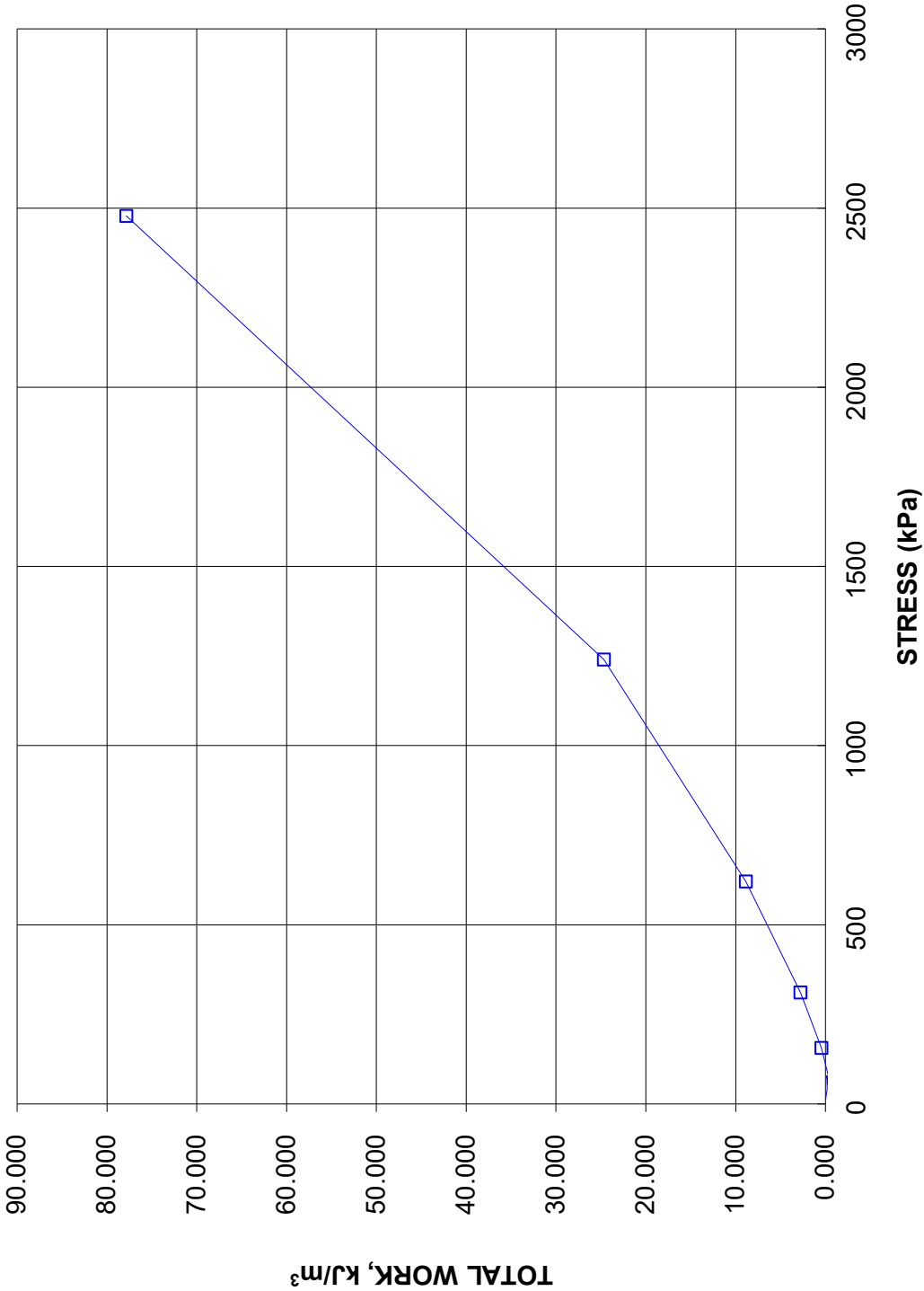




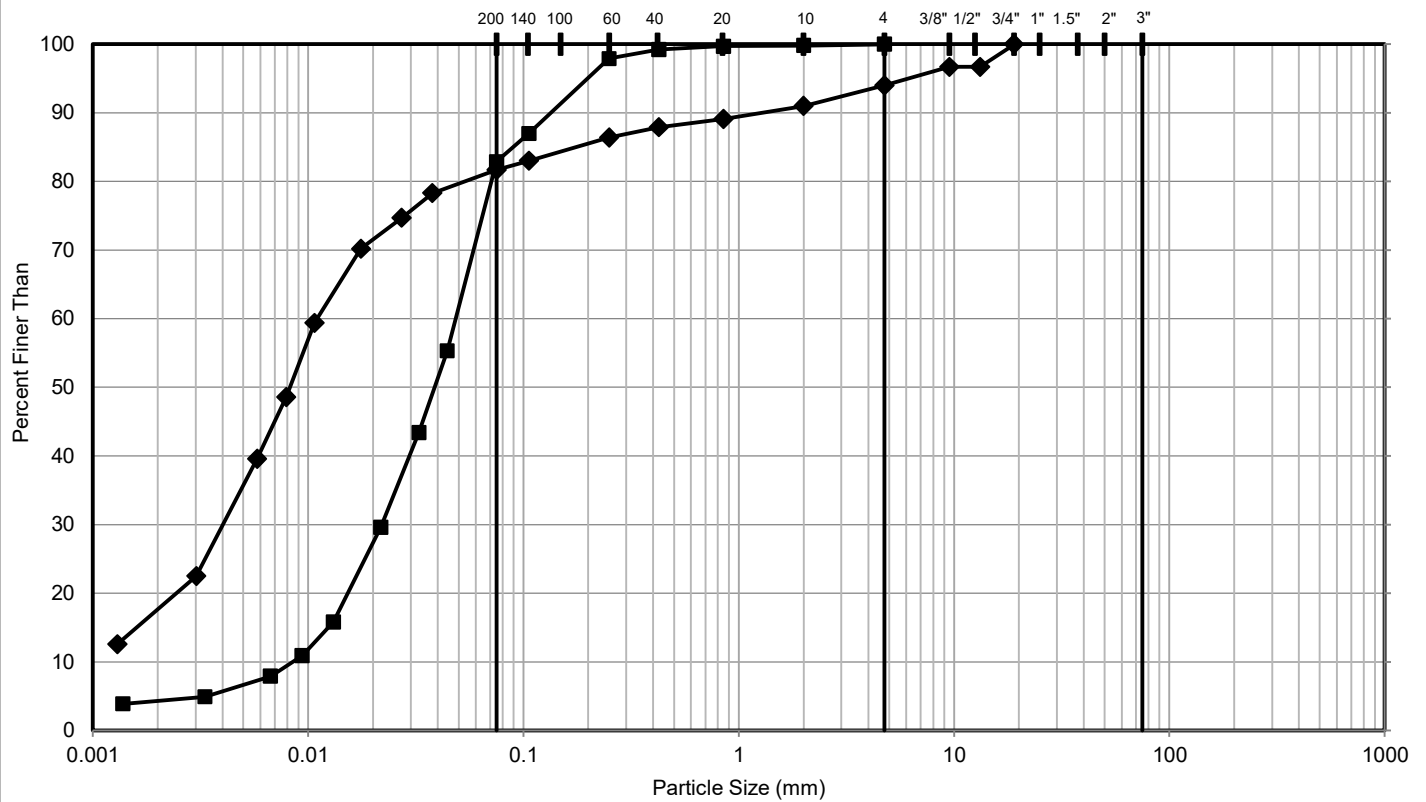
CONSOLIDATION TEST  
TOTAL WORK VS STRESS

FIGURE B11

CONSOLIDATION TEST  
TOTAL WORK, kJ/m<sup>3</sup> vs STRESS  
BH AIP-3 SA12



### Grain Size Distribution - Silt



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-2	13	13.7 - 14.3	211.3 to 210.7
◆	AIP-4	18	24.4 - 24.6	200.1 to 199.9

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Artesian Industrial Parkway

CONSULTANT



YYYY-MM-DD 2023-03-06

DESIGNED PT

PREPARED PT

REVIEWED KB

APPROVED KB

TITLE

Grain Size Distribution  
Silt

PROJECT NO.

19136074

CONTROL

1000

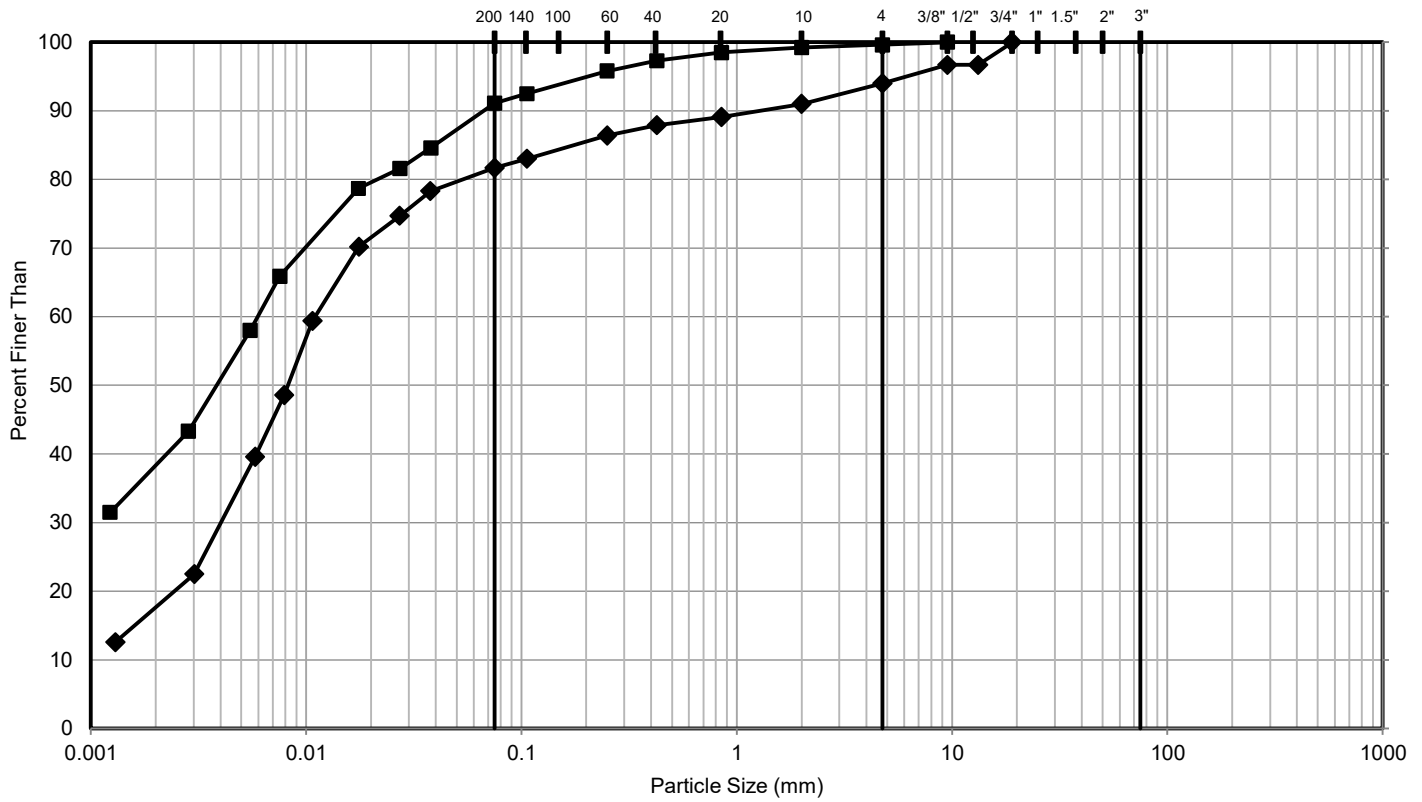
REV.

0

FIGURE

B12

## Grain Size Distribution - Clayey Silt to Clayey Silt-Silt (TILL)



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-3	15	15.2 - 15.9	209.6 to 209.0
◆	AIP-4	18	24.4 - 24.6	200.1 to 199.9

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2023-03-06

DESIGNED PT

PREPARED PT

REVIEWED KB

APPROVED KB

PROJECT

Bradford Bypass - Artesian Industrial Parkway

TITLE

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

PROJECT NO.

19136074

CONTROL

1000

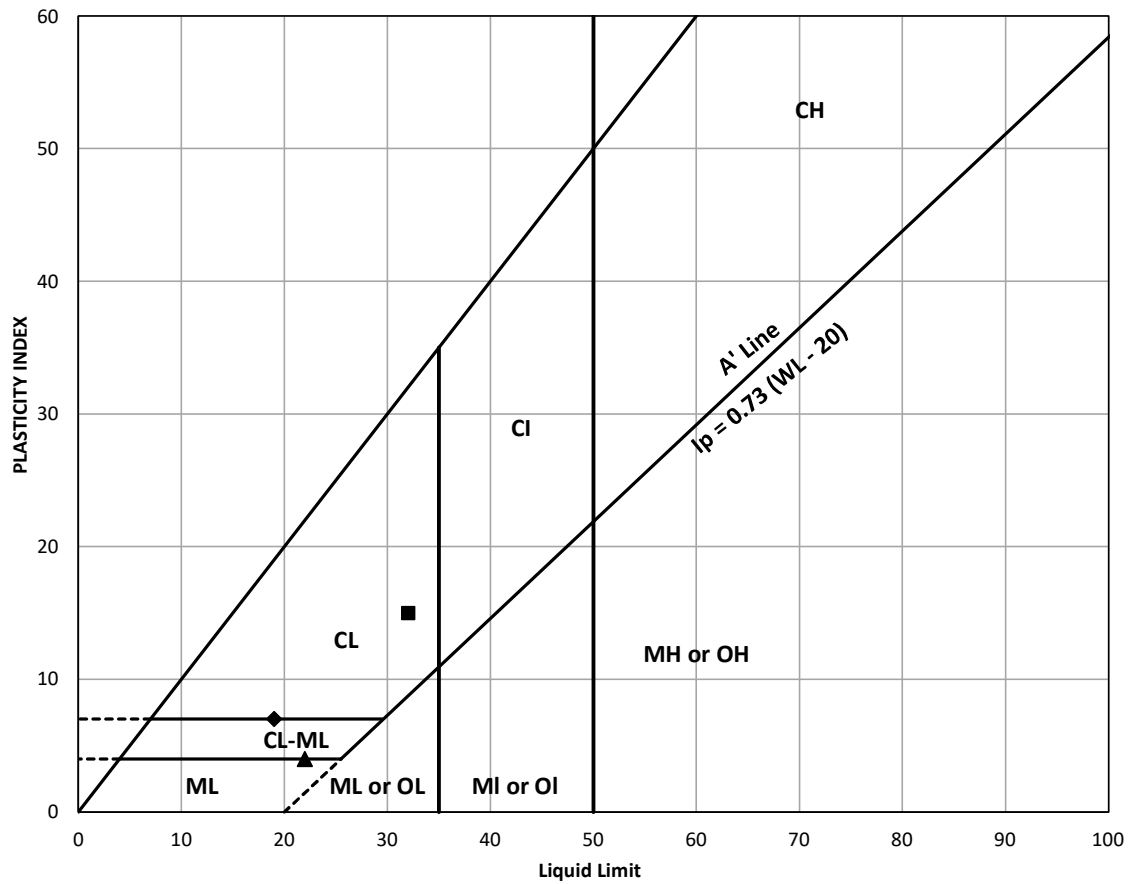
REV.

0

FIGURE

B13

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

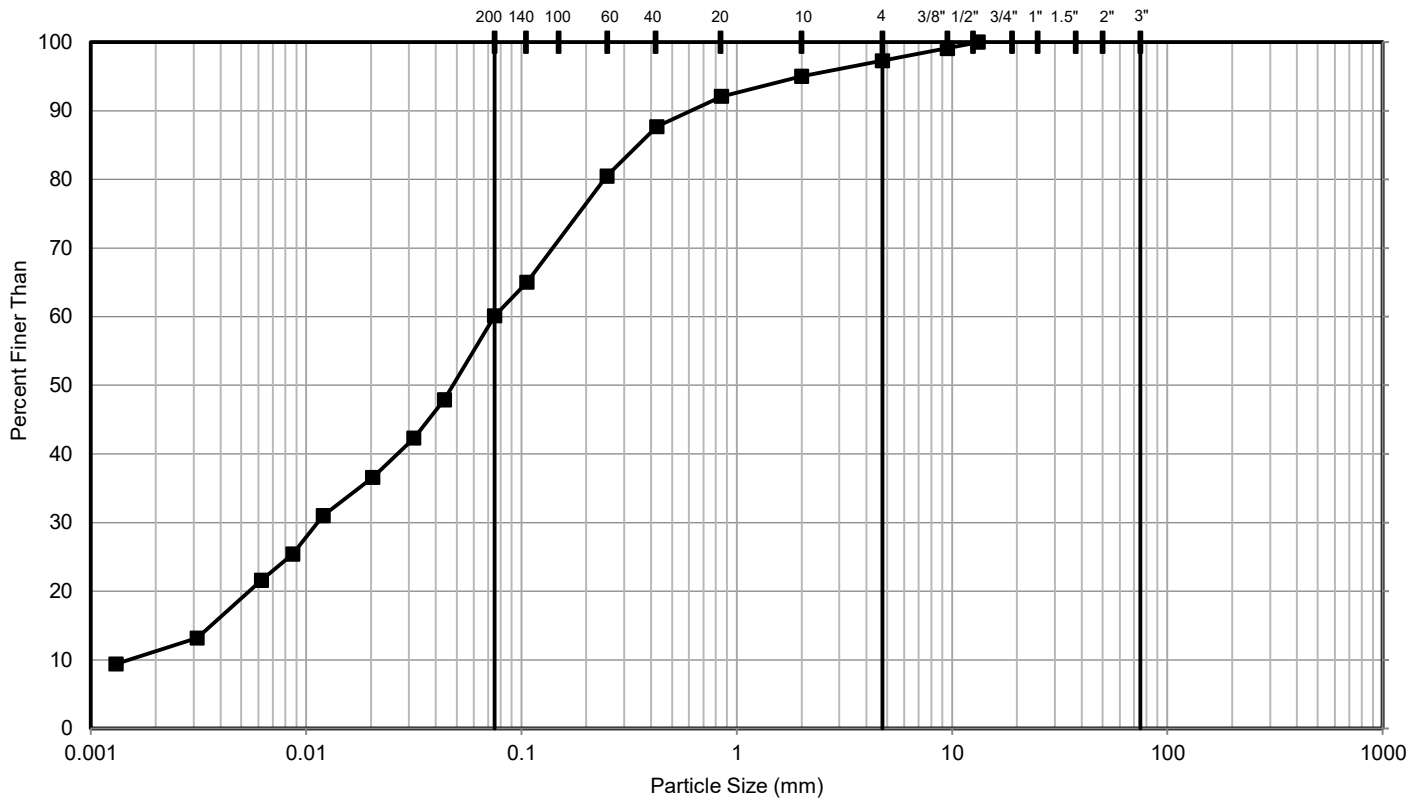


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	AIP-3	15	15.2 - 15.9	19.5	32	17	15
◆	AIP-4	14b	15.5 - 15.9	10.2	19	12	7
▲	AIP-4	18	24.4 - 24.6	14.2	22	18	4

CLIENT				PROJECT			
AECOM / MTO				Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT		YYYY-MM-DD	2023-03-06	TITLE			
		DESIGNED	PT	Plasticity Chart - Clayey Silt to Clayey Silt-Silt (TILL)			
		PREPARED	PT				
		REVIEWED	KB				
		APPROVED	KB				
PROJECT NO.		CONTROL		FIGURE			
19136074		1000		B14			

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Artesian Industrial Parkway/Appendix B Lab Figures/Working Files | FILE NAME: Atterberg Output MTO.xlsm

## Grain Size Distribution - Silt and Sand to Silt (TILL)



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-2	16	18.3 - 18.9	206.7 to 206.1

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2023-03-06

DESIGNED PT

PREPARED PT

REVIEWED KB

APPROVED KB

PROJECT

Bradford Bypass - Artesian Industrial Parkway

TITLE

Grain Size Distribution  
Silt and Sand to Silt (TILL)

PROJECT NO.

19136074

CONTROL

1000

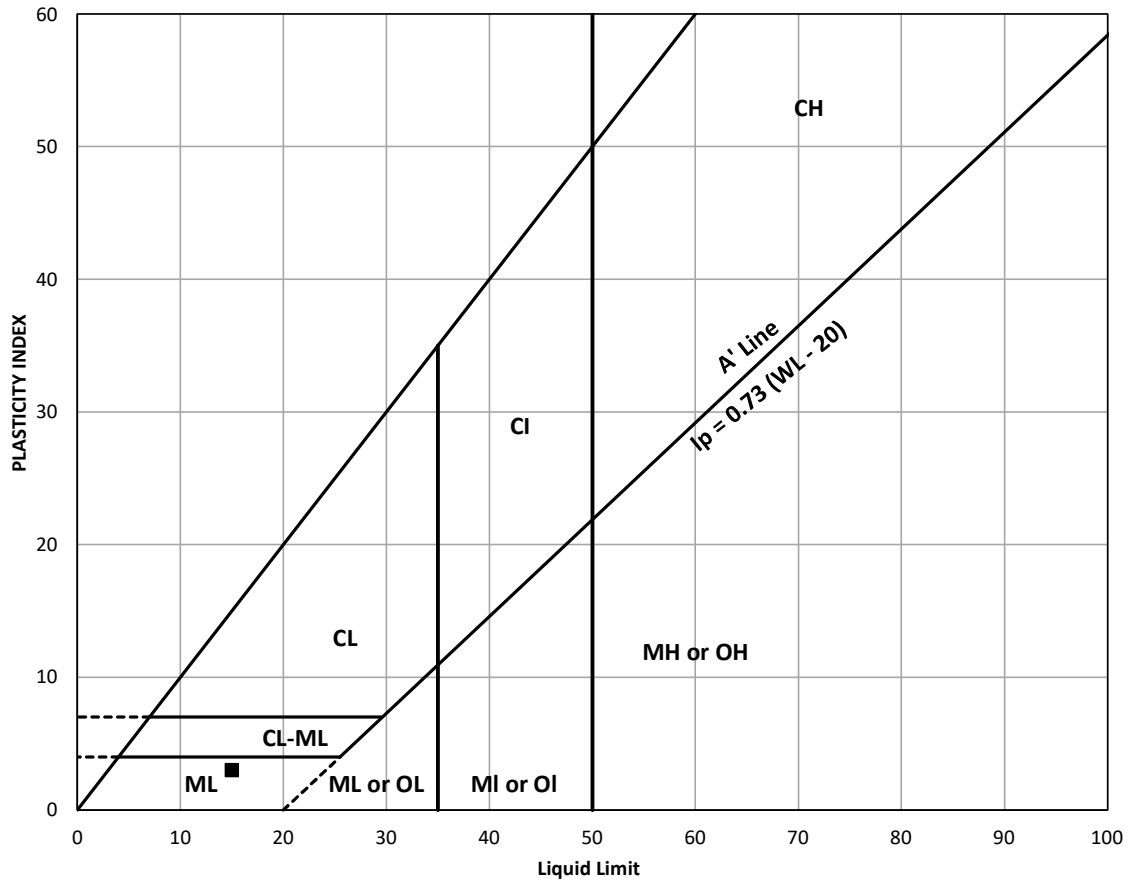
REV.

0

FIGURE

B15

# Plasticity Chart - Silt and Sand to Silt (TILL)

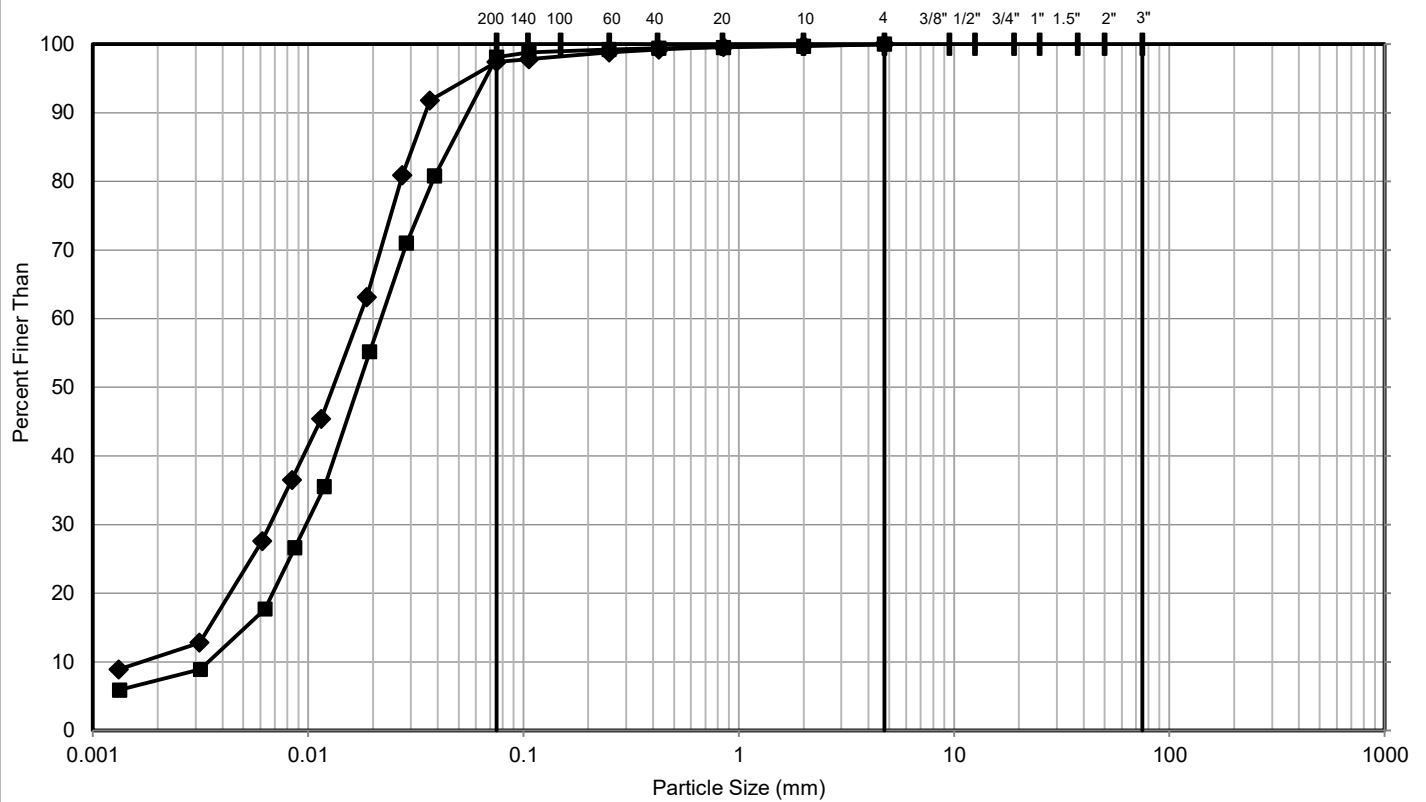


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	AIP-2	16	18.3 - 18.9	9.1	15	12	3

CLIENT				PROJECT			
AECOM / MTO				Bradford Bypass - Artesian Industrial Parkway			
CONSULTANT		YYYY-MM-DD	2023-03-06	TITLE			
		DESIGNED	PT	Plasticity Chart - Silt and Sand to Silt (TILL)			
		PREPARED	PT				
		REVIEWED	KB				
		APPROVED	KB				
		PROJECT NO.	19136074	CONTROL	1000	FIGURE	B16

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Artesian Industrial Parkway/Appendix B Lab Figures/Working Files | FILE NAME: Atterberg Output MTO.xlsm

# Grain Size Distribution - Silt



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	AIP-2	18	24.4 - 24.6	200.6 to 200.4
◆	AIP-3	19	24.4 - 24.7	200.4 to 200.1

CLIENT		PROJECT	
AECOM / MTO		Bradford Bypass - Artesian Industrial Parkway	
CONSULTANT	YYYY-MM-DD	2023-03-06	
	DESIGNED	PT	
	PREPARED	PT	
	REVIEWED	KB	
	APPROVED	KB	
		TITLE	
		Grain Size Distribution Silt	
PROJECT NO.		CONTROL	REV.
19136074		1000	0
		FIGURE	
		B17	

**APPENDIX C**

# Analytical Laboratory Test Results





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

**Attention: Manisha Ahuja**

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

**Report Date: 2021/12/23**  
Report #: R6938140  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C1Z5211**

**Received: 2021/12/17, 11:54**

Sample Matrix: Soil  
# Samples Received: 5

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	5	2021/12/20	2021/12/21	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	5	2021/12/20	2021/12/20	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	5	N/A	2021/12/23	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	5	N/A	2021/12/22	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	5	2021/12/20	2021/12/20	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	5	2021/12/17	2021/12/20	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	5	2021/12/20	2021/12/22	CAM SOP-00464	EPA 375.4 m

**Remarks:**

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

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

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

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

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

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

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

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

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

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



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

**Attention: Manisha Ahuja**

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

**Report Date: 2021/12/23**  
Report #: R6938140  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C1Z5211**

**Received: 2021/12/17, 11:54**

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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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

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

### SOIL CORROSIVITY PACKAGE (SOIL)

<b>Bureau Veritas ID</b>		RJZ032			RJZ032		
<b>Sampling Date</b>		2021/11/24			2021/11/24		
<b>COC Number</b>		n/a			n/a		
	<b>UNITS</b>	<b>AIP-2 SA-02 2'6"-4'6"</b>	<b>RDL</b>	<b>QC Batch</b>	<b>AIP-2 SA-02 2'6"-4'6" Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>							
Resistivity	ohm-cm	2200		7736410			
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl-)	ug/g	96	20	7741334			
Conductivity	umho/cm	454	2	7741152			
Available (CaCl2) pH	pH	7.89		7741621			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	7741338			
Sulphide	mg/kg	3.2 (1)	0.5	7752524	3.3	0.5	7752524
<b>Physical Testing</b>							
Moisture-Subcontracted	%	6.6	0.30	7752523			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Sample contained greater than 10% headspace at time of extraction. Sample extracted past method-specified hold time. Analyzed past method specified hold time							

<b>Bureau Veritas ID</b>		RJZ033	RJZ034		RJZ035		
<b>Sampling Date</b>		2021/11/18	2021/11/19		2021/11/23		
<b>COC Number</b>		n/a	n/a		n/a		
	<b>UNITS</b>	<b>CN-2 SA-02 2'6"-4'6"</b>	<b>AIP-3 SA-01 0'-2'</b>	<b>QC Batch</b>	<b>AIP-4 SA-02 2'6"-4'6"</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>							
Resistivity	ohm-cm	3900	5900	7736410	7200		7736410
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl-)	ug/g	50	<20	7741334	<20	20	7741334
Conductivity	umho/cm	256	170	7741152	139	2	7741152
Available (CaCl2) pH	pH	7.45	7.65	7741621	7.81		7741646
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	7741338	<20	20	7741338
Sulphide	mg/kg	4.2 (1)	3.0 (2)	7752524	2.3 (2)	0.5	7752524
<b>Physical Testing</b>							
Moisture-Subcontracted	%	20	15	7752523	17	0.30	7752523
RDL = Reportable Detection Limit QC Batch = Quality Control Batch (1) Sample contained greater than 10% headspace at time of extraction. Sample extracted past method-specified hold time. Analyzed past method specified hold time (2) Sample extracted past method-specified hold time. Analyzed past method specified hold time							



BUREAU  
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

### SOIL CORROSIVITY PACKAGE (SOIL)

<b>Bureau Veritas ID</b>		RJZ036		
<b>Sampling Date</b>		2021/11/17		
<b>COC Number</b>		n/a		
	<b>UNITS</b>	<b>CN-1 SA-02 2'6"-4'6"</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>				
Resistivity	ohm-cm	5500		7736410
<b>Inorganics</b>				
Soluble (20:1) Chloride (Cl-)	ug/g	22	20	7741334
Conductivity	umho/cm	180	2	7741152
Available (CaCl2) pH	pH	7.66		7741621
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	7741338
Sulphide	mg/kg	1.3 (1)	0.5	7752524
<b>Physical Testing</b>				
Moisture-Subcontracted	%	15	0.30	7752523
RDL = Reportable Detection Limit QC Batch = Quality Control Batch (1) Sample extracted past method-specified hold time. Analyzed past method specified hold time				



BUREAU  
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

## TEST SUMMARY

**Bureau Veritas ID:** RJZ032  
**Sample ID:** AIP-2 SA-02 2'6"-4'6"  
**Matrix:** Soil

**Collected:** 2021/11/24  
**Shipped:**  
**Received:** 2021/12/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7741334	2021/12/20	2021/12/21	Alina Dobreanu
Conductivity	AT	7741152	2021/12/20	2021/12/20	Kien Tran
Moisture (Subcontracted)	BAL	7752523	N/A	2021/12/23	Kerstin Joyce Lague
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison
pH CaCl2 EXTRACT	AT	7741621	2021/12/20	2021/12/20	Taslina Aktar
Resistivity of Soil		7736410	2021/12/20	2021/12/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7741338	2021/12/20	2021/12/22	Avneet Kour Sudan

**Bureau Veritas ID:** RJZ032 Dup  
**Sample ID:** AIP-2 SA-02 2'6"-4'6"  
**Matrix:** Soil

**Collected:** 2021/11/24  
**Shipped:**  
**Received:** 2021/12/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison

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

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

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7741334	2021/12/20	2021/12/21	Alina Dobreanu
Conductivity	AT	7741152	2021/12/20	2021/12/20	Kien Tran
Moisture (Subcontracted)	BAL	7752523	N/A	2021/12/23	Kerstin Joyce Lague
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison
pH CaCl2 EXTRACT	AT	7741621	2021/12/20	2021/12/20	Taslina Aktar
Resistivity of Soil		7736410	2021/12/20	2021/12/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7741338	2021/12/20	2021/12/22	Avneet Kour Sudan

**Bureau Veritas ID:** RJZ034  
**Sample ID:** AIP-3 SA-01 0'-2'  
**Matrix:** Soil

**Collected:** 2021/11/19  
**Shipped:**  
**Received:** 2021/12/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7741334	2021/12/20	2021/12/21	Alina Dobreanu
Conductivity	AT	7741152	2021/12/20	2021/12/20	Kien Tran
Moisture (Subcontracted)	BAL	7752523	N/A	2021/12/23	Kerstin Joyce Lague
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison
pH CaCl2 EXTRACT	AT	7741621	2021/12/20	2021/12/20	Taslina Aktar
Resistivity of Soil		7736410	2021/12/20	2021/12/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7741338	2021/12/20	2021/12/22	Avneet Kour Sudan



**BUREAU  
VERITAS**

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

## TEST SUMMARY

**Bureau Veritas ID:** RJZ035  
**Sample ID:** AIP-4 SA-02 2'6"-4'6"  
**Matrix:** Soil

**Collected:** 2021/11/23  
**Shipped:**  
**Received:** 2021/12/17

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7741334	2021/12/20	2021/12/21	Alina Dobreanu
Conductivity	AT	7741152	2021/12/20	2021/12/20	Kien Tran
Moisture (Subcontracted)	BAL	7752523	N/A	2021/12/23	Kerstin Joyce Luge
Sulphide in Soil	SPEC	7752524	N/A	2021/12/22	Bailey Morrison
pH CaCl <sub>2</sub> EXTRACT	AT	7741646	2021/12/20	2021/12/20	Taslima Aktar
Resistivity of Soil		7736410	2021/12/20	2021/12/20	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7741338	2021/12/20	2021/12/22	Avneet Kour Sudan

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

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

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



BUREAU  
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

### GENERAL COMMENTS

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

Package 1	13.3°C
-----------	--------

Results relate only to the items tested.

BUREAU  
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

## QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7741152	Conductivity	2021/12/20			98	90 - 110	<2	umho/cm	0.14	10
7741334	Soluble (20:1) Chloride (Cl-)	2021/12/21	NC	70 - 130	103	70 - 130	<20	ug/g	6.9	35
7741338	Soluble (20:1) Sulphate (SO4)	2021/12/22	122	70 - 130	107	70 - 130	<20	ug/g	NC	35
7741621	Available (CaCl2) pH	2021/12/20			100	97 - 103			1.4	N/A
7741646	Available (CaCl2) pH	2021/12/20			100	97 - 103			0.11	N/A
7752523	Moisture-Subcontracted	2021/12/23					<0.30	%		
7752524	Sulphide	2021/12/22	106	75 - 125	108	75 - 125	<0.5	mg/kg	1.9	30

N/A = Not Applicable

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

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

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

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

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

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





BUREAU  
VERITAS

Bureau Veritas Job #: C1Z5211

Report Date: 2021/12/23

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD BYPASS

Sampler Initials: AM

### VALIDATION SIGNATURE PAGE

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

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

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

Orla Jorgensen, Organics Lab Manager

---

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

## WORK ORDER CHAIN OF CUSTODY RECORD

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name:	Golder Associates Ltd.	Company Name:	Golder Associates Ltd.	Quotation #:		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name:	Canada Accounts Payable	Contact Name:	Manisha Ahuja	P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address:	6925 Century Ave. Suite 100 Mississauga, ON	Address:	6925 Century Ave. Suite 100 Mississauga, ON L5N 7K2	Project #:	19136074	Rush TAT (Surcharges will be applied)	
Phone:	905-567-4444 Fax: 905-567-6561	Phone:	365-292-1471 Fax: 905-567-6561	Site Location:	Bradford Bypass	<input type="checkbox"/> 1 Day	<input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days
Email:	canadaaccounts payableinvoices@golder.com	Email:	Manisha_Ahuja@golder.com	Site Location Province:	Bradford Ontario	Date Required:	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS DRINKING WATER CHAIN OF CUSTODY				Sampled By:	AM	Rush Confirmation #:	
<b>Regulation 153</b>		<b>Other Regulations</b>		<b>Analysis Requested</b>			
<input checked="" type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQU Region _____ <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED) <input type="checkbox"/> REG 406 Table _____		# OF CONTAINERS SUBMITTED FIELD FILTERED (CIRCLE) Metals / Hg / CrVI BTEX / PHC F1 PHCs F2 - F4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cr, VI, ICPMS Metals, HWS - B) Corrosivity Package (+ Sulphide) HOLD - DO NOT ANALYZE			
Include Criteria on Certificate of Analysis: Y / N		SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS		LABORATORY USE ONLY			
CUSTODY SEAL Y / N		COOLER TEMPERATURES		Present Intact			
19/11/15							
COOLING MEDIA PRESENT: Y / N							
COMMENTS							
2 Jars, no redox.							
1 Jars, no redox.							
2 Jars, no redox.							
2 Jars, no redox.							
2 Jars, no redox.							
MSA with BV Signed May 18, 2020. Golder standing offer rates in email from Julie Clement dated Sept 20, 2021. Corrosivity package including chloride, conductivity, resistivity, pH, sulphate, sulphide is \$98.60/sample.							
RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)		
<i>[Signature]</i>	2021/12/17	12:00	<i>[Signature]</i>	2021/12/17	11:54		
C1Z5211							

NDA ENV 1270



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