



REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 401/Dingman Drive Underpass Replacement (Site No. 19X-0368/B0)

London, Ontario

MTO GWP 3025-18-00, Assignment No. 3018-E-0011

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PART A

**FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/DINGMAN DRIVE UNDERPASS REPLACEMENT
(SITE No. 19X-0368/B0)
LONDON, ONTARIO
MTO GWP 3025-18-00, ASSIGNMENT No. 3018-E-0011**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the expansion of Highway 401 from Tilbury to London, Ontario as part of MTO Assignment No. 3018-E-0011.

The overall project is divided into six GWPs. Dingman Drive underpass, the subject of this report, is part of GWP 3025-18-00, which addresses widening/reconstruction of approximately 10 km of Highway 401 from 1 km west of Highway 4 (Colonel Talbot Road) to Wellington Road in London, Ontario, including associated structure improvements.

This report presents the results of a foundation investigation conducted in support of detail design for the replacement of the Dingman Drive underpass (MTO Structure Site No. 19X-0368/B0) crossing Highway 401, approximately 1.5 km west of Wellington Road in London, Ontario. The purpose of the work is to assess the subsurface conditions at the location of the proposed underpass replacement by drilling a limited number of boreholes, completing in situ testing, and completing geotechnical and analytical laboratory testing on selected soil samples obtained from the boreholes, to support the development of foundation engineering recommendations for the detail design.

The terms of reference for the scope of work are provided in MTO's Request for Proposal for Assignment No. 3018-E-0011, dated December 2018, Section 3.7.1 – Foundation Engineering of Stantec's Technical Proposal, and change orders to address the shift from design-build-ready to detail design, including the addition of boreholes at the centre pier and for embankment widening.

2.0 SITE DESCRIPTION

The topography in the area of the existing underpass at Highway 401/Dingman Drive consists of flat to undulating land typically used for agriculture. Dingman Creek is located immediately to the west of the Dingman Drive underpass site and drains this local area.

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of this report is referenced to project north and therefore may differ from magnetic north. For the purposes of this report, Highway 401 is considered to be oriented east-west and Dingman Drive to be oriented north-south. The Dingman Drive underpass site is located approximately 1.5 km west of Wellington Road, crossing over Highway 401 at a skew. The existing bridge is a 10.4 m wide by 74.9m long, four-span prestressed concrete girder bridge spanning six lanes of Highway 401.

The natural ground surface in the immediate vicinity of the site varies from approximately Elevation 254 m to 257 m. Highway 401 has been constructed on a low embankment with its grade at approximately Elevation 259 m at the structure site, while the existing Dingman Drive grade is at approximately Elevation 265 m at the underpass. The existing approach embankments on Dingman Drive are up to approximately 8.5 m high relative to the original ground surface, with the highest point immediately behind the bridge abutments, transitioning to approximately 4.5 m in height approximately 100 m north and south of the respective bridge abutments.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation at the proposed Dingman Drive underpass replacement was carried out on October 10, 2019, November 14, 2019 and between April 20 and May 1, 2020, during which time a total of seven sampled boreholes (designated as Boreholes BH-101 to BH-107) were advanced as near as practicable to the footprint of the proposed replacement structure. The locations of the boreholes are shown on Drawing 1 following the text of this report.

The borehole investigation was carried out using both truck-mounted and track-mounted drilling equipment, supplied and operated by specialist drilling contractors. The boreholes were advanced through the overburden to depths of 18.8 m to 40.9 m, using 108 mm inside diameter solid stem augers, and open-hole tri-cone drilling methods. Soil samples were 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 by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)¹, and using 76 mm O.D thin-walled 'Shelby' Tube samplers (ASTM D1587-00)² to obtain relatively undisturbed samples in the cohesive soils. Field vane shear tests were carried out in the cohesive soils for assessment of undrained shear strength (ASTM D2573)³ using an MTO standard N-size vane. The results of the in situ field tests (i.e. SPT "N" values and undrained shear strengths from the field vane tests) as presented on the borehole records in Appendix A and in Section 4 are uncorrected.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations, and a vibrating wire piezometer was installed in Borehole BH-103 to permit monitoring of the groundwater level at the site. The borehole annulus surrounding the piezometer was backfilled to the ground surface with a cement-bentonite grout. The piezometer has not been decommissioned and it is recommended that the piezometer remain in place for use by the Contractor to obtain accurate water level readings at the time of construction, and for this piezometer to be decommissioned in accordance with Ontario Regulation 903, Wells (as amended) as part of the construction contract. All remaining boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was observed by members of Golder's technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in-situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's London geotechnical laboratory where the samples underwent further visual examination. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples, and one-dimensional consolidation (oedometer) testing was carried out on two samples of the cohesive deposits. All of the geotechnical laboratory tests were carried out to MTO and/or ASTM Standards, as applicable, and the results are presented in Appendix B.

Three selected soil samples were submitted to AGAT Laboratories Ltd. (AGAT), a Standards Council of Canada (SCC) accredited laboratory, in Mississauga, Ontario for chemical analysis. The soil samples were analyzed for a suite of corrosivity parameters, including conductivity, resistivity, soluble chloride, soluble sulphate and pH. The results of the chemical analyses are presented in Appendix C.

¹ ASTM D1586-08a - Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.

² ASTM D1587-15 - Standard Practice For Thin-Walled Tube Sampling Of Fine-Grained Soils For Geotechnical Purposes.

³ ASTM D2573-15 - Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

The as-drilled borehole locations and the ground surface elevations were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The locations given on the borehole/drillhole records and shown on Drawings 1 to 3 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations including both northing and easting coordinates and geographic coordinates of latitude and longitude, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Termination Depth (m)	Borehole Termination Elevation (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)			
BH-101	4,753,324.7 (42.912607)	408,904.5 (-81.224971)	263.8	21.8	242.0
BH-102	4,753,331.6 (42.912663)	408,946.6 (-81.224455)	258.1	40.9	217.2
BH-103	4,753,306.5 (42.912432)	408,985.3 (-81.223986)	258.4	35.1	223.3
BH-104	4,753,312.9 (42.912484)	409,027.8 (-81.223465)	264.6	18.8	245.8
BH-105	4,753,345.2 (42.912795)	408,877.7 (-81.225296)	256.3	18.8	237.5
BH-106	4,753,330.8 (42.912651)	408,985.3 (-81.223981)	258.9	40.0	218.9
BH-107	4,753,332.3 (42.912654)	409,059.5 (-81.223073)	257.5	20.3	237.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The Dingman Drive underpass site lies within the physiographic region of southwestern Ontario known as the Mount Elgin Ridges⁴. The soils generally consist of moraines of clay or silty clay with vales of alluvium consisting of gravel, sand and silt.

According to the Ontario Department of Mines Preliminary Map P.0606 entitled “Pleistocene Geology of the St. Thomas Area (East Half), Southern Ontario”, the site sits between the Westminster and Ingersoll moraines. The predominant soils at the site consist of glacial tills deposited by the Erie Lobe of the St. Thomas moraine during the Late Wisconsin period of glaciation. From Dingman Drive easterly to the east project limit, the soils consist of the

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

Port Stanley silty clay and clayey silt till, which is locally overlaying by thin layers of lacustrine silts. At Dingman Creek immediately west of the underpass site, deposits of modern alluvium consisting of gravel, sand and silt are present. From Dingman Creek westerly, lacustrine or pond deposits of sand and silty sand predominate and are intersected by localized valleys.

The bedrock in the area of the site is reported to consist of limestone of the Dundee Formation of the Hamilton Group, and its surface is estimated to be approximately 55 metres below ground surface or at about Elevation 205 m.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the current investigation, together with the results of the geotechnical laboratory tests and in situ testing carried out, are presented on the borehole records in Appendix A; the geotechnical laboratory test sheets are provided in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile and cross-sections on Drawings 2 and 3 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 and moreover, the interpreted stratigraphy shown on Drawings 2 and 3 represents a simplification of the subsurface conditions. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions in the area of the proposed bridge replacement structure consist of asphalt or topsoil and fill associated with the existing highway and local road embankments, underlain by a thick deposit of stiff to hard clayey silt to silty clay that contains a 2 m to 4 m thick silt to silty sand interlayer, and which is subsequently underlain by deposits of hard sandy clayey silt to silt till, and a very dense lower silty sand to sand. A more detailed description of the subsurface conditions encountered in the boreholes from the current investigation is provided in the following sections.

4.2.1 Asphalt

Boreholes BH-101 and BH-104 were advanced through the existing Dingman Drive pavement and encountered asphalt with a thickness of 180 mm and 240 mm, respectively. Borehole BH-106 was advanced from the eastbound Highway 401 lanes and encountered 480 mm of asphalt.

4.2.2 Topsoil

In Boreholes BH-102, BH-103, BH-105 and BH-107, topsoil with a thickness ranging between about 90 mm and 210 mm was encountered immediately below the ground surface.

Layers of buried topsoil were encountered below the fill materials (discussed below) in Boreholes BH-103 and BH-104 at depths of 1.8 m and 7.9 m below ground surface, respectively, with the surface of this buried topsoil at approximately Elevation 256.6 m in both boreholes. These topsoil layers are about 0.5 m and 1.1 m thick as encountered in the boreholes.

4.2.3 Fill

A layer of non-cohesive fill was encountered underlying the asphalt/topsoil in Boreholes BH-101 and BH-103 to BH-106; this non-cohesive fill is variable in composition and consists of sandy silt to silty sand to sand and gravel. A layer of non-cohesive fill consisting of clayey silt to sandy clayey silt was encountered underlying the asphalt/topsoil in Borehole BH-102, and underlying the non-cohesive fill in Boreholes BH-101, BH-103, BH-104 and BH-106. The total fill thickness ranges between 1.7 m and 9.2 m in Boreholes BH-101 to BH-106, and the base of the fill material varies between approximately Elevation 254.1 m and 256.6 m as encountered in these boreholes.

The Standard Penetration Test (SPT) “N”-values measured within the non-cohesive fill range from 3 blows to 40 blows per 0.3 m of penetration, indicating that the non-cohesive fill has a loose to dense state of compactness. The SPT “N”-values measured within the cohesive fill range from 5 blows to 19 blows per 0.3 m of penetration, suggesting that the cohesive fill layers have a firm to very stiff consistency.

Grain size distribution tests were carried out on four samples of the cohesive fill and the results are shown on Figure B-1 in Appendix B. Atterberg limits testing was carried out on four samples of the cohesive fill and measured liquid limits ranging from about 24 to 34 per cent, plastic limits ranging from about 15 to 18 per cent, and plasticity indices ranging from about 9 to 16 per cent. The Atterberg limits test results are shown on the plasticity chart on Figure B2 in Appendix B and indicate that the material is typically classified as a clayey silt of low plasticity. The natural water content measured on samples of the non-cohesive fill ranges from about 5 to 23 per cent. The natural water content measured on samples of the cohesive fill ranges from about 10 to 25 per cent.

4.2.4 Surficial Sand

A 0.8 m thick layer of sand and gravel, some silt was encountered underlying the buried topsoil layer in Borehole BH-104 at a depth of 8.4 m below ground surface (Elevation 256.2 m).

The natural water content measured on a sample of the sand layer is about 21 per cent.

4.2.5 Clayey Silt to Silty Clay

In all of the boreholes advanced at the site, a cohesive deposit ranging in composition from clayey silt to silty clay, trace to some sand was encountered below the topsoil, fill or surficial sand layer. The top of the deposit was encountered between Elevation 257.4 m and 254.1 m. The thickness of the deposit ranges between 12.7 m and 25.8 m in the boreholes where it was fully penetrated (i.e., Boreholes BH-101 and BH-104). A 1.5 m to 4.5 m thick silt to silty sand interlayer (discussed in Section 4.2.6) was encountered within the clayey silt to silty clay deposit in Boreholes BH-102, BH-103, BH-106 and BH-107.

The SPT “N”-values measured within the cohesive deposit range between 4 blows and 58 blows per 0.3 m of penetration. In-situ field vane tests carried out within the cohesive stratum measured undrained shear strengths ranging from about 79 kPa to 153 kPa, as well as greater than 144 kPa, with a calculated sensitivity between about 1.4 and 2.6. The field vane test results along with the measured SPT “N”-values indicate that the clayey silt to silty clay deposit has a soft to hard consistency, but is generally stiff to very stiff. Typically, the upper few metres of the deposit is very stiff, with a firm to stiff zone typically between about Elevation 250 m and 242 m (near the top of the silt sandy silt interlayer), below which the deposit becomes very stiff to hard.

The results of grain size distribution tests carried out on 16 samples of the clayey silt to silty clay deposit are shown on Figures B-3.1 and B-3.2 in Appendix B. Atterberg limits tests were carried out on 21 samples of this deposit and measured liquid limits ranging between about 21 and 40 per cent, plastic limits ranging between about 11 and 19 per cent, and plasticity indices ranging between about 8 and 21 per cent. These results, which are plotted on

plasticity charts on Figures B-4.1 and B-4.2 in Appendix B, indicate that the material ranges in classification from a clayey silt of low plasticity to silty clay of intermediate plasticity. The natural water content measured on samples of the cohesive deposit ranges from about 10 to 31 per cent.

Laboratory consolidation tests were carried out on two samples of the clayey silt to silty clay deposit obtained from Shelby tube samples. Preconsolidation stresses ranging between 250 kPa and 420 kPa were estimated from the void ratio versus logarithmic stress plots, indicating an overconsolidation ratio (OCR) between 1.6 and 2.8. Bulk unit weights ranging between about 19.7 kN/m³ and 20.7 kN/m³ and a specific gravity between 2.75 and 2.77 were measured on the consolidation test specimens. Details of the consolidation test results are included in Figures B5 and B6 in Appendix B, and the test results are summarized below. The compressibility characteristics will vary with depth in accordance with the moisture content and shear strength profiles.

Borehole Sample No.	Sample Depth/Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	C_v^* (cm ² /s)
Borehole BH-103 Sample 12	10.9 m / 247.5 m	160	250	90	1.6	0.138	0.007	0.55	3.1×10^{-4}
Borehole BH-107 Sample 11	10.9 m / 246.6 m	150	420	270	2.8	0.161	0.016	0.76	2.3×10^{-3}

* For stress range between approximately the in-situ effective overburden stress and final stress due to proposed embankment construction.

where: σ_{vo}' is the effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
 OCR is overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is initial void ratio
 C_v is the coefficient of consolidation in cm²/s

4.2.6 Silt to Silty Sand Interlayer

A non-cohesive interlayer ranging in composition from silt to sandy silt to silty sand was encountered within the clayey silt to silty clay deposit in Boreholes BH-102, BH-103, BH-105, BH-106 and BH-107 (as discussed in Section 4.2.5). The surface of the interlayer was encountered between Elevation 241.6 m and 238.6 m, and the thickness of the interlayer is between 1.4 m and 4.5 m in the boreholes where it was fully penetrated.

The SPT "N"-values measured within the silt to silty sand interlayer range from 16 blows to 56 blows per 0.3 m of penetration, indicating the layer has a compact to very dense state of compactness.

The results of grain size distribution tests carried out on five samples from the silt to silty sand interlayer are shown on Figure B-7 in Appendix B. The natural water content measured on samples of the interlayer ranges from about 14 to 22 per cent.

4.2.7 Sandy Clayey Silt to Clayey Silt to Silt Till

A glacial till deposit consisting of sandy clayey silt to clayey silt to silt, trace to some gravel, was encountered underlying the clayey silt to silty clay deposit in Boreholes BH-102, BH-103, BH-106 and BH-107. The surface of the till was encountered between Elevation 238.5 m and 228.7 m. Boreholes BH-103 and BH-107 were terminated in this deposit at depths of 35.0 m and 20.3 m below ground surface (Elevation 223.4 m and 237.2 m), respectively. The thickness of the deposit is between 6.3 m and 7.6 m in the boreholes where it was fully penetrated.

The SPT “N”-values measured within the sandy clayey silt to silt till deposit range from 26 blows to 103 blows per 0.3 m of penetration, with four SPT “N”-values greater than 100 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

The result of grain size distribution tests carried out on three samples from the till deposit are shown on Figure B-8 in Appendix B. Atterberg limits tests were carried out on three samples of this deposit and measured liquid limits ranging between about 15 and 21 per cent, plastic limits ranging between about 9 and 12 per cent, and plasticity indices range between about 6 and 9 per cent. These results, which are plotted on a plasticity chart on Figure B-9 in Appendix B, indicate that the cohesive material ranges in classification from a clayey silt-silt to clayey silt of low plasticity. The natural water content measured on samples of the cohesive till deposit ranges from about 9 to 21 per cent.

4.2.8 Lower Silty Sand to Sand

A non-cohesive deposit of silty sand to sand, some silt, trace gravel, was encountered underlying the sandy clayey silt to silt till deposit in Boreholes BH-102 and BH-106, the two deepest boreholes advanced at the site. The surface of this lower silty sand to sand deposit was encountered between Elevation 222.7 m and 221.1 m. Both of the boreholes were terminated in this deposit at depths of 40.9 m and 40.0 m below ground surface (Elevation 217.2 m and 218.9 m), respectively.

The SPT “N”-values measured within the lower silty sand to sand deposit were greater than 100 blows for 0.3 m of penetration, indicating a very dense state of compactness, with the exception of the first sample within this deposit in Borehole BH-102, where the SPT “N”-value of 47 blows per 0.3 m of penetration suggests a dense state of compactness.

The results of grain size distribution tests carried out on two samples of the lower silty sand to sand deposit are shown on Figure B-10 in Appendix B. The natural water content measured on samples of this lower silty sand to sand deposit ranges from about 16 per cent to 22 per cent.

4.3 Groundwater Conditions

The groundwater levels were measured in the open boreholes upon completion of drilling operations. The details of these measurements are shown on the borehole records contained in Appendix A; however, it is noted that measurements recorded on completion of drilling are not considered to represent the stabilized groundwater level at the site.

A Vibrating Wire Piezometer (VWP) was installed in Borehole BH-103 to permit monitoring of the groundwater level at the site. Details of the piezometer installation and measured groundwater levels are shown on the borehole record in Appendix A. The measured groundwater levels in this VWP are summarized in the following table:

Borehole No.	Location	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)	Screened Deposit
BH-103	South Abutment	258.4	4.1	254.3	11/20/2019	Silt
			4.2	254.2	12/13/2019	
			4.3	254.1	2/7/2020	
			3.8	254.6	5/19/2020	

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

The VWP has not been removed from the site and we recommend that it remain in place for the Contractor to use to measure/monitor the groundwater level at the site. The VWP does not require decommissioning in accordance with Ontario Regulation 903 (Wells, as amended); only removal of the surface box and wires is required during construction.

4.4 Analytical Testing of Soil Samples

As discussed in Section 3.0, three soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the following table summarizes the results of the testing:

Parameter	BH-103 Sample 5 (Elev. 255.1 m)	BH-102 Sample 5 (Elev. 253.2 m)	BH-106 Sample 6 (Elev. 254.1 m)
pH	8.15	7.85	7.98
Resistivity (ohm-cm)	1,190	2,050	2,210
Electrical Conductivity (umho/cm)	840	488	452
Chlorides (ug/g)	364	164	130
Soluble Sulphates (ug/g)	8	43	54

5.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., a senior geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality review of this report.

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PART B

**FOUNDATION DESIGN REPORT
HIGHWAY 401/DINGMAN DRIVE UNDERPASS REPLACEMENT
(SITE No. 19X-0368/B0)
LONDON, ONTARIO
MTO GWP 3025-18-00, ASSIGNMENT No. 3018-E-0011**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides discussion and recommendations on geotechnical/foundation aspects for the detail design of the Dingman Drive underpass replacement and associated approach embankments, as well as the nearby overhead sign. The recommendations are based on Golder's interpretation of the factual information obtained during the field explorations and geotechnical laboratory testing. The discussion and recommendations presented are intended to provide the designers with information to assess the feasible design and construction alternatives and to design the bridge foundations, retaining walls / wing walls and raised approach embankments.

The Foundation Design Report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) 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 Foundation Design Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects which could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

The existing bridge is a 10.4 m wide by 74.9 m long, four-span prestressed concrete girder bridge spanning six lanes of Highway 401. The existing bridge is supported on shallow foundations bearing on native soil deposits at about Elevation 255.5 m at the piers and perched within the approach embankments at about Elevation 261.5 m at the abutments. Based on visual observations made during the field investigation, there is no visual evidence of differential settlement between the foundation elements, nor of global stability issues on the approach embankment side slopes.

Based on the General Arrangement drawing provided by Stantec (dated March 2020), it is understood that the two-span replacement structure is to be constructed on a shifted alignment approximately 8 m to the east of the existing bridge, resulting in the need to widen the existing embankments to the east in the immediate vicinity of the structure. A grade raise on the order of 1.5 m is also planned; the proposed embankment height is approximately 8.8 m at the approaches at the north and south abutments, transitioning to 6.5 m to 7 m approximately 100 m north and south of the abutments.

It is understood that integral, semi-integral and conventional abutments are all structurally viable alternatives for design of the structure; however, integral abutments are preferred. The suitability of integral or semi-integral abutments is influenced by the length, type and geometry of the structure, abutment and wingwall heights, number of spans and the subsurface soil conditions. Provided the abutment heights and wingwall lengths are limited to a maximum of 6 and 7 m, respectively, use of integral or semi-integral abutments at the site is considered geotechnically feasible. Integral abutments are typically supported by driven steel H-piles installed with single or double corrugated steel pipe (CSP) liners filled with sand over the top 3 m. Consideration may also be given to supporting integral abutments on concrete-filled steel tube piles provided the increased stiffness can be accommodated in the design. Conventional or semi-integral or hybrid-integral abutments may be founded on spread footings bearing on native soils, steel H-piles or drilled concrete shafts (caissons).

Both shallow and deep foundation options have been considered for support of the replacement structure. Temporary protection systems may be required along the east side of the existing Dingman Drive to facilitate construction of the new approach embankments and foundations for the new abutments, although the requirement for protection systems may be lessened if construction proceeds under a road closure and detour approach.

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

- **Strip or spread footings founded on the stiff to very stiff silty clay:** Shallow footings are feasible at this site; the existing structure abutments are founded on spread footings and have performed well, based on observations at the time of the site investigation. However, this option would result in settlements on the order of 75 mm or more at the abutments and pier, and would need to be designed and constructed to control differential settlements across the foundation elements; therefore, shallow foundations present greater risks than deep foundation options. This option would require excavation to a depth of about 1.2 m below the existing grade at the toe of the existing approach embankments (and full height excavation through the existing embankment and side slopes) to found footings on the stiff to very stiff silty clay deposit. This option does not allow for the construction of integral abutments.
- **Footings “perched” on a compacted granular pad in the approach embankments:** Shallow footings “perched” within the proposed approach embankments are technically feasible for this site, although similar to the footing option described above, these foundations would be affected by up to 75 mm of settlement under the approach embankment loading and at the centre pier. While such total settlements could be controlled to produce a solution with acceptable differential settlements for structure performance, this option presents higher risk than deep foundation options. This perched footing option would minimize the depth of excavation below the existing grade (notwithstanding the requirement to remove the existing substructure where interference occurs). This option does not allow for the construction of integral abutments.
- **Driven steel H-piles or pipe piles founded within the very dense/hard glacial till deposits:** Driven steel H-piles or steel pipe (tube) piles could also be considered for support of the proposed bridge foundations. In this case, the piles would develop their resistance from both side friction and end-bearing. Design tip elevations will vary across the structure, and will require piles that are about 30 m to 40 m in length. As inferred from grinding of the augers during borehole advancement and given that the site soils are glacially derived, the presence of cobbles and boulders within the native soil deposits should be anticipated which could affect deep foundation installation, although damage to piles may be mitigated by using driving shoes and/or heavier pile sections.
- **Drilled shafts founded within the dense silt or hard clayey silt deposit at a depth of approximately 20 m:** Drilled shafts (caissons) are feasible for support of the abutments (although they would not permit integral abutment construction) and pier for the proposed new structure. They may be advantageous at the pier because they can eliminate the need for a below-grade pile cap, and in general they require a smaller footprint in plan than either a shallow foundation or driven pile foundation would require; however, it is understood that the median working zone is not significantly constrained during construction staging. In general, temporary liners filled with water, or controlled-density drilling fluids, as well as tremie concrete may be required during caisson installation to control the ground and groundwater within the water-bearing zones. As inferred from grinding of the augers during borehole advancement and given that the site soils

are glacially derived, the presence of cobbles and boulders within the native soil deposits should be anticipated which could affect deep foundation installation.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments on driven steel H-piles founded within the very dense/hard till deposits, particularly if integral abutments are preferred, and to found the new center pier on drilled shafts or driven piles to minimize differential settlement relative to the abutments.

6.3 Design Considerations

6.3.1 Consequence and Site Understanding Classification

The proposed bridge crosses over Highway 401, which carries large volumes of traffic with the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2019 *Canadian Highway Bridge Design Code* CAN/CSA S6-19 and its Commentary (CHBDC 2019), the proposed bridge and its foundation system is considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the CHBDC have been used for design.

6.3.2 Seismic Design

6.3.2.1 Seismic Parameters

The new bridge is in Seismic Performance Category (SPC) 1 and therefore seismic analysis of bridges in SPC 1 is not a requirement of the CHBDC (Clause 4.4.5.1). However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clauses 4.4.10.2 and 4.4.10.5.

6.3.2.2 Seismic Hazard Assessment and Liquefaction Potential

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.⁵ The characteristics of the cohesive soils indicate that they are not susceptible to liquefaction. Although layers of saturated granular materials are present, they are relatively thin. The liquefaction potential is considered low based on the soil profile type, age of the deposits, relative density/consistency and the historically low regional seismicity. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted.

6.3.2.3 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation and in situ testing. Based on the energy-corrected average penetration resistance, \bar{N}_{60} below the founding level, the site may be classified as Site Class D in accordance with Table 4.1 of the 2019 CHBDC, in the absence of any geophysical testing. Geophysics testing, if carried out, may provide a more favourable Site Class designation.

⁵ FHWA, 1997: “Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles.” *Geotechnical Engineering Circular No. 3*:FHWA-SA-97-076, Washington, D.C.

The 2019 CHBDC 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 current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2015.

6.3.2.4 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 per cent damped spectral response acceleration ($S_a(T)$) values for Site Class C are presented below.

Site Class C Spectral Values for Subject Site

Seismic Hazard Values	2% Probability of Exceedance in 50 years (2,475-year return period)
PGA (g)	0.067
PGV (m/s)	0.056
$S_a(0.2)$ (g)	0.111
$S_a(0.5)$ (g)	0.071
$S_a(1.0)$ (g)	0.041
$S_a(2.0)$ (g)	0.021
$S_a(5.0)$ (g)	0.005
$S_a(10.0)$ (g)	0.002

The values given above are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given above in this report (Site Class D) in accordance with Section 4.4.3 of the CHBDC. The corresponding site-specific Site Class D seismic hazard values given in the table below can be used for design.

Site Class D Spectral Values for Subject Site

Seismic Hazard Values	2% Probability of Exceedance in 50 Years (2,475-year)
PGA (g)	0.086
PGV (m/s)	0.082
$S_a(0.2)$ (g)	0.138
$S_a(0.5)$ (g)	0.104
$S_a(1.0)$ (g)	0.064
$S_a(2.0)$ (g)	0.033
$S_a(5.0)$ (g)	0.008
$S_a(10.0)$ (g)	0.003

6.4 Shallow Foundations

6.4.1 Founding Elevation and Geotechnical Resistances

The abutments and retaining walls for the new bridge may be founded on conventional spread/strip footings. All footings should be provided with a minimum of 1.2 m of earth cover or thermal equivalent for frost protection purposes, per Ontario Provincial Standard Drawing (OPSD) 3090.101. Alternatively, the abutment footings could be constructed on free-draining granular engineered fill with reduced cover provided that an appropriate subdrain and outlet is provided to drain the granular materials. If this minimum soil cover is not provided for the footing such that the depth of embedment is reduced, the factored ultimate geotechnical resistances should be re-evaluated.

A factored ultimate geotechnical resistance of 350 kPa and a factored serviceability geotechnical resistance of 150 kPa may be used for abutment footings founded on the stiff to very stiff silty clay at approximately Elevation 256.8 m, based on a 6 m wide footing. The foundation for the median pier may be designed using a factored ultimate geotechnical resistance of 500 kPa; for a 500 kPa load at the pier footing, the pier footing is expected to settle approximately 100 mm; for context, the factored serviceability geotechnical resistance for 25 mm of settlement at the pier would be 150 kPa. The factored serviceability geotechnical resistance for the abutments is not necessarily applicable as the existing ground has the potential to settle as much as approximately 75 mm in conjunction with the widened/raised embankment loading, and the embankment load and settlement will control footing performance in this case. Thus, the effects of about 25 mm of differential settlement between the abutments and the centre pier (with greater settlement at the pier) should be considered in the structural design if shallow foundations are considered for all foundation elements. If a settlement mitigation strategy is adopted to reduce the settlement under the embankment loading, greater differential settlements would apply between the strip-footing-supported abutments and centre pier.

Alternatively, “perched” abutments may be used where these are founded on engineered fill approach embankments. The engineered fill should be constructed on the properly prepared stiff to very stiff silty clay at about Elevation 258 m and should consist of a minimum thickness of 2 m of Ontario Provincial Standard Specifications (OPSS).PROV 1010 Granular A placed and compacted in accordance with OPSS.PROV 501. The engineered fill should extend a minimum of 1 m plus the fill thickness beyond the footing and should be sloped at an inclination of 1 horizontal to 1 vertical (1H:1V) outward from the underside of the footing to meet the prepared subgrade. Abutment footings founded on engineered fill as described above may be designed using a factored ultimate geotechnical resistance at ULS of 600 kPa, assuming 6 m wide footings. As indicated above, the factored serviceability geotechnical resistance is not necessarily applicable given the embankment settlement impacts without mitigation, and the effects of differential settlement should be considered in the structural design depending on the option(s) adopted.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width or founding elevation vary from that given above. The factored ultimate geotechnical resistances provided are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, eccentricity and inclination of the load should be considered.

The stiff to very stuff subgrade could be susceptible to disturbance and degradation on exposure to water and construction traffic and therefore, if this foundation option is adopted, a concrete working slab is recommended to be placed over the subgrade to protect the integrity of the foundation soils.

6.4.2 Resistance to Lateral Forces/Sliding Resistance

Resistance to lateral forces/sliding between the concrete spread/strip footings and the subgrade should be calculated in accordance with Section 6.10.4 of CHBDC 2019. Assuming that the founding soils are not loosened or disturbed during excavation and footing construction, the unfactored coefficient of friction ($\tan \phi'$) values between the cast-in-place concrete footings and the inspected and approved subgrade may be taken as follows:

Subgrade Material	Coefficient of Friction, $\tan \phi'$
Cast-in-place footing or working slab on compacted Granular 'A'	0.70
Cast-in-place footing or working slab on stiff to very stiff silty clay	0.58

6.5 Driven Piles

6.5.1 Tip Elevations and Geotechnical Resistances

Based on the boreholes advanced at each foundation element, the elevation at which practical refusal to pile driving is expected, and the associated pile lengths at each foundation element, are summarized in the table below.

Foundation Element	Pile Cap Elevation (m)	Anticipated Pile Refusal Elevation (m)	Approximate Pile Length (m)
North Abutment	261.4	220 to 219	42 to 43
Pier	256.6	228.5 to 227.5	29 to 30
South Abutment	261.3	226.5 to 225.5	35 to 36

For the design of HP 310x110 or HP 360x108 piles driven to practical refusal at these design tip elevations, the factored ultimate geotechnical axial resistance and factored serviceability geotechnical resistance provided in the following table may be used. It is understood that the design team has selected HP 310x125 H-piles for design of the abutments based on structural considerations. These piles can be considered to be geotechnically equivalent to HP 310x110 piles for design and the values given in this report for HP 310x110 can also be used for the HP 310x125 piles.

Foundation Element	Pile Type	Founding Strata	Maximum (Highest) Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
North Abutment	HP 310 x 110	Very dense sand	220	1,700	N/A*
	HP 360 x 108			1,800	N/A*
Pier	HP 310 x 110	Hard clayey silt till	228	1,600	N/A*
	HP 360 x 108			1,700	N/A*

Foundation Element	Pile Type	Founding Strata	Maximum (Highest) Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
South Abutment	HP 310 x 110	Hard clayey silt till	226	1,700	N/A*
	HP 360 x 108			1,800	N/A*

* The factored serviceability geotechnical resistances for 25 mm of settlement will be higher than the factored ultimate geotechnical resistance values given, so Serviceability Limit States (SLS) will not govern the design.

Downdrag loads arising from settlement under the widened/raised approach embankments, as described in Section 6.5.3, must also be considered in the structural design of the abutment piles.

Pile installation should be in accordance with OPSS 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile. The set criteria must therefore be established at the time of construction once the piling equipment is confirmed. The pile capacity should be verified in the field by the use of both the Hiley formula (MTO Standard Drawing SS103-11) during the final stages on all piles, and high strain dynamic testing (more commonly known as pile dynamic analyzer (PDA) testing) on a minimum of two piles at each foundation element. The following note from MTO's Structural Manual should be shown on the Contract Drawing, assuming the use of HP 310x110 or HP 310x125 piles and that a resistance factor of 0.5 is applied to the use of the Hiley formula and PDA test results:

- Piles to be driven in accordance with Standard SS103-11 and/or Pile Dynamic Analyzer (PDA) testing using an ultimate geotechnical resistance of 3,400 kN per pile at the abutments, and 3,200 kN per pile at the pier.
[Note to structural designers: Stantec to adjust the stated ultimate geotechnical resistance to reflect two times the design load, if less than the geotechnical resistances values recommended herein.]

Assessment of the ultimate geotechnical resistance by the Hiley formula and high-strain dynamic testing should commence once the pile reaches a depth of not less than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48-hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation. SSP 109F57 has been modified to amend OPSS.PROV 903 (*Deep Foundations*) to address the 48-hour wait period between initial driving and re-tapping, as well as the requirement for PDA testing, for inclusion in the Contract Documents (see Appendix D).

6.5.2 Other Details – Driving Shoes, CSP Liners and Frost Protection

The silty clay to clayey silt and till deposits are known to contain cobbles and boulders that may interfere with driving of the piles or cause damage to pile tips. Flange plates in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) or driving shoes should be specified on the foundation drawings to reduce the potential for damage to the piles. Where flange plates are adopted, it is recommended that they be ground flush with the pile flanges after welding, to minimize the potential for reduction in the shaft friction component of the pile capacity; alternatively, driving shoes that are flush with the pile flanges are recommended as the preferred pile tip reinforcement. Further, consideration could be given to using a heavier H-pile section such as HP 310x125, HP

310x132 or HP 360x132, to reduce the potential for damage to the piles during driving to the design tip elevation and ultimate resistance, along with the use of flange plate reinforcement or pile driving shoes.

Piles supporting integral abutments require placement of a corrugated steel pipe (CSP) liner filled with loose uniform sand around the upper 3 m of the pile to allow appropriate pile flexibility to accommodate thermal effects and other forces at the bridge deck. Depending on the site grades, pre-augering may be required to install these CSPs. A Non-Standard Special Provision (NSSP) for the CSPs detailing the sand gradation should be included in the Contract Documents where applicable.

Pile caps should be provided with a minimum frost cover of 1.2 metres of soil cover or thermal equivalent.

6.5.3 Downdrag Loads at Abutments

The eastward widening and grade raise on the approach embankments will cause long-term consolidation settlement of the underlying silty clay to clayey silt deposits. The consolidation settlement is time-dependent and will not completely occur during the construction period unless the embankments are placed well in advance (i.e., more than one to two years in advance) of bridge construction or other settlement mitigation measures are adopted (e.g., surcharging in conjunction with preloading, or the use of lightweight fill to construct some portions of the embankments). Post-construction settlement of the silty clay to clayey silt deposits relative to the piles will result in development of negative skin friction acting on the piles.

Based on the results of the investigation, the unfactored downdrag load acting on an HP 310x110 or HP 310x125 pile driven to the design elevations given in the preceding sections has been assessed as approximately 450 kN per pile, based on the Alpha method outlined in the *Canadian Foundation Engineering Manual* (CFEM, 2006) together with engineering judgment. The neutral plane is estimated to be at about 16 m below ground surface. The downdrag load has no effect on the geotechnical axial capacity of the pile and should not be included in the design check that considers the factored ultimate geotechnical resistance. At Ultimate Limit States (ULS), the pile structural capacity is to be checked using the following:

$$P_f > 1.25 Q_d + \gamma_p DF$$

where: P_f = factored axial compression resistance of the pile

Q_d = permanent dead load/sustained load on the pile

γ_p = load factor for drag force = 1.25

DF = drag force on the pile

Should drilled shafts be used to support the abutments, downdrag loads will be significantly higher than those provided above for H-piles; Golder will assess these values if a drilled shaft option is adopted at the abutments.

6.5.4 Resistance to Lateral Loads

Resistance to lateral loading may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can

be estimated using subgrade reaction theory (as outlined below). However, the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are most appropriate where the maximum pile deflections are less than 1% of the pile width or diameter, where the loading is static (no cycling) and where the pile material is linear as per the *Canadian Foundation Engineering Manual* (CFEM, 2006). Where these conditions are not met, and/or where required for the structural engineering model, the non-linear lateral behavior of the soil should be considered using P-y curves. P-y curves for a single laterally loaded pile at the abutment and pier locations are provided on Figures 1 and 2, respectively, for an HP310x110 pile installed to the tip elevations given in Section 6.5.1.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory, where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equations given below, as described by Terzaghi (1955) and the *Canadian Foundation Engineering Manual* (CFEM, 1992).

For non-cohesive soils:

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

where:

- n_h = coefficient related to soil density (kPa/m)
- z = depth below pile cap for semi-integral abutment and bottom of CSP for integral abutments (m), and
- B = width of pile or diameter of drilled shaft (m)

For cohesive soils:

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

where:

- s_u = undrained shear strength of the soil (kPa), and
- B = width of pile or diameter of drilled shaft (m)

The following values of n_h (Terzaghi, 1955) and s_u may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties, the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection). In the table, the clayey silt to clay stratum is referred to as the silty clay deposit for simplicity.

Foundation Element	Soil Unit	Approximate Elevation Interval (m)	n_h (kPa/m)	s_u (kPa)
North Abutment	Firm to stiff clayey silt fill	257 to 254	--	50
	Stiff to very stiff clayey silt	254 to 251	--	100
	Soft to stiff silty clay	251 to 242	--	50
	Compact to very dense silt	242 to 237	6,000	--
	Very stiff to hard clayey silt till	237 to 225	--	200
	Dense to very dense sand	Below 225	7,000	--
Center Pier	Firm to stiff clayey silt fill	259 to 254.5	--	50
	Stiff to hard clayey silt to silty clay	254.5 to 241	--	100
	Compact to dense silt	241 to 238	5,000	--
	Very stiff to hard clayey silt	238 to 229	--	200
	Hard clayey silt till	Below 229	--	200

Foundation Element	Soil Unit	Approximate Elevation Interval (m)	n_h (kPa/m)	s_u (kPa)
South Abutment	Stiff to hard clayey silt fill	258 to 256	--	100
	Firm to very stiff clayey silt	256 to 241	--	100
	Dense to very dense sandy silt	241 to 238	6,000	--
	Very stiff to hard silty clay	238 to 230	--	200
	Hard clayey silt till	Below 230	--	200

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the piles should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the abutment wall for units supporting the abutments (Section C6.11.2.2 of the *Commentary to CHBDC* (2019)).

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 efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to CHBDC* (2019).

6.6 Drilled Shafts (Caissons) for Centre Pier

6.6.1 Tip Elevations and Geotechnical Resistances

The proposed bridge pier may also be supported on drilled shafts (caissons). From a constructability perspective, drilled shafts up to 2 m in diameter installed to an approximate depth range of 20 m are constructible without the use of specialty foundation contractors from outside of southwestern Ontario. For this site, consideration may be given to founding drilled shafts for the centre pier within the dense silt/hard clayey silt at approximately Elevation 235 m, using the following factored ultimate geotechnical resistances and factored serviceability geotechnical resistances:

Foundation Element	Founding Elevation (m)	Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
Centre Pier	235	1.2	3,750	N/A*
	235	1.5	5,000	N/A*
	235	2.0	7,000	N/A*

* The factored serviceability geotechnical resistances for 25 mm of settlement are higher than the factored ultimate geotechnical resistance values given, so the SLS condition will not govern the design.

Higher geotechnical resistances could be achieved by extending the lengths of the drilled shafts into the lower till deposit at this site, if required.

Drilled shaft foundations should be constructed in accordance with OPSS.PROV 903 (*Deep Foundations*). If drilled shaft foundations are adopted for the centre piers at this site, temporary liners should be used to support the overburden soils during construction to minimize disturbance to the side walls, particularly where saturated cohesionless soils are present (such as the silt deposit in which the drilled shafts may be founded). The liner should

be advanced while filled with slurry to minimize the potential for non-cohesive materials to migrate into the drillhole and to control base disturbance / basal heave due to groundwater pressures / seepage. In addition, placement of concrete by tremie methods is recommended.

Consideration must also be given to the potential presence of cobbles and boulders within the native soil deposits. Appropriate construction equipment and techniques must be selected to handle the anticipated cobbles and boulders.

The performance of drilled shafts will depend upon the final cleaning and verification of the subgrade quality at the base of the drilled shaft. Each drilled shaft excavation should be cleaned to remove all loosened debris. The inspection of the base of the drilled shafts can be accomplished by means of a Shaft Inspection Device (SID). If conditions in the drilled shaft do not allow for use of a SID, a Shaft Quantitative Inspection Device (SQUID) should be used to verify the quality of the drilled shaft base. Should the inspection indicate that loosened/unacceptable soil is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected. Following construction of the drilled shafts, in particular if drilling slurry and tremie concrete are used, it is recommended that non-destructive testing of the shafts be carried out, which could include pile integrity testing, cross-hole sonic logging, or thermal integrity testing. Alternatively, it is recommended that a dynamic load test (i.e., APPLE test) be specified for at least one drilled shaft per foundation element.

An NSSP is provided in Appendix D for inclusion in the Contract Documents, if drilled shafts are adopted at the centre pier, to alert the Contractor to the presence of potential flowing soil conditions and potential obstructions in the native soils and to address requirements for inspection. This NSSP will be modified further to require the use of a SID or SQUID and potentially to include non-destructive testing, based on MTO construction contract experience and direction, if drilled shafts are adopted.

6.6.2 Downdrag Loads

Downdrag loads are not expected to apply for drilled shafts for the centre pier.

6.6.3 Resistance to Lateral Loads

The resistance to lateral loads for drilled shafts at the centre pier may be assessed using the equations and parameters provided in Section 6.5.4.

6.7 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wingwalls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, 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 abutments, in accordance with the CHBDC. These lateral earth pressure recommendations assume that the ground above the wall will be flat, not sloping; if the inclination of the slope above the wall is sloping, or if the slope changes, lateral earth pressures will need to be adjusted.

- Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type II or Type III should be used as backfill behind the abutments and walls. This fill should be placed and compacted in accordance with OPSS.PROV 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with *CHBDC* (2019) Section 6.12.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501. 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 1.2 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).
- For restrained walls, the pressures are based on the proposed new embankment fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM) for the general embankment fill:

Fill Type	Unit Weight of Material (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K _o	Active, K _a
Earth Fill / Select Subgrade Material	20	0.50	0.33

- For an unrestrained wall, the pressures are based on the properties of the granular backfill and the following parameters (unfactored) may be used:

Fill Type	Unit Weight of Material (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K _o	Active, K _a
Granular 'A'	22	0.43	0.27
Granular 'B' Type II or III	21	0.43	0.27

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the *Commentary to the CHBDC* (2019).
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.2 of the *CHBDC* (2019).

6.8 Approach Embankments

Based on the General Arrangement drawing provided by Stantec (dated March 2020), it is understood that the new structure is to be constructed on a shifted alignment approximately 8 m to the east of the existing bridge, resulting in the need to widen the existing embankments (which have side slopes oriented at approximately 2H:1V) to the east in the immediate vicinity of the structure. A grade raise on the order of 1.5 m is also planned; the proposed embankment height is approximately 8.8 m at the approaches at the north and south abutments, transitioning to 6.5 m to 7 m approximately 100 m north and south of the abutments.

The following sections address subgrade preparation and embankment construction, global stability and settlement of the proposed widened/raised approach embankments. The critical sections used in the analyses are located just behind the abutments where the embankments are highest at each side of the bridge. The piezometric conditions used in the analyses are based on the groundwater level as encountered during the subsurface investigation and recorded in the vibrating wire piezometer (VWP) installed in Borehole BH-103; a groundwater level of Elevation 254.5 m was used adjacent to the north and south abutments.

6.8.1 Subgrade Preparation and Embankment Construction

Prior to construction of the realigned approach embankments, it is recommended that all topsoil/organic soils and any loose/soft deleterious fill be stripped from within the footprint of the embankments and from the side slopes of the existing embankments where the new embankments will encroach on the existing.

From a geotechnical perspective, it is recommended that fill for construction of the new/widened approach embankments consist of granular fill or OPSS.PROV.1010 Select Subgrade Material (SSM). If there is a surplus of earth fill on the construction contract, or if there is a significant local source of earth fill at the time of construction, use of earth fill may be considered for the embankment widening; however, its suitability would be dependent on the quality of the source, including its plasticity and water content, and in this regard the native clayey silt to silty clay deposit that predominates in this local area may be prone to sloughing/slumping on the embankment side slopes if used as earth fill.

Fill materials should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading) and should be benched into the existing embankment side slopes in accordance with OPSD 208.010 (Benching of Earth Slopes). Embankments greater than 8 m should incorporate into the side slopes a minimum 2 m wide bench at mid-height for all fill heights greater than 8 m as suggested in OPSD 202.010 (Slope Flattening), to promote surficial stability and erosion protection on the embankment side slopes.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.8.2 Global Stability

6.8.2.1 Methods and Parameters

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 8), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses,

the factors of safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of forces tending to resist failure to the driving forces tending to cause the failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor, ψ , and the geotechnical resistance factor ϕ_{gu} (i.e. $FoS = 1/(\psi * \phi_{gu})$). Accordingly, a target minimum Factor of Safety of 1.3 has been used for design of the temporary embankment side slopes, and a FoS of 1.5 for the design of the final embankment configuration as per Table 6.2 of CHBDC (2019) for the total stress (short-term undrained) and effective stress (long-term drained) condition, as applicable.

The simplified stratigraphy together with the associated strength and unit weights employed for the different soil types at the critical sections are summarized for all soil layers in the following table.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle (°)	Effective Cohesion (c') (kPa)	Undrained Shear Strength (s _u) (kPa)
New granular fill	21	32	2	--
Stiff to hard clayey silt fill	19	30	0	100
Stiff to hard clayey silt to silty clay	19	32	0	100
Soft to stiff silty clay	19	28	0	50
Compact to very dense silt	20	32	0	--
Very stiff to hard clayey silt till	20	34	0	200
Dense to very dense sand	19	35	0	--

The subsoils encountered are composed of a combination of cohesive deposits (clayey silt to clay) and granular deposits (silt and sand). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle) for the granular soils were estimated from empirical correlations using the results of the SPT "N"-values as suggested by NAVFAC (1986) and Kulhawy and Mayne (1990), in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the short-term, undrained analyses while effective stress parameters were employed in the long-term, drained analyses.

6.8.2.2 Analysis Results

For embankments up to approximately 8.8 m in height, with side slopes oriented no steeper than 2 horizontal to 1 vertical (2H:1V), the factor of safety for global stability will be greater than 1.3 in temporary/short-term conditions during construction, and greater than 1.5 in long-term/permanent conditions.

6.8.3 Settlement

6.8.3.1 Methods and Parameters

To estimate the magnitude of the expected settlements under the widened/raised approach embankment loading, analyses were carried out at the critical sections of the proposed approach embankments using the commercially available program Settle3D (Version 4.015) produced by Rocscience Inc., combined with spreadsheet calculations where appropriate, using the Boussinesq stress distribution method.

The sources of settlement include immediate settlement of the granular soils, and primary time dependent consolidation of the cohesive deposits. Due to the over-consolidated nature of the cohesive soils at this site, secondary compressions (creep) is not anticipated.

The immediate compression of the native cohesionless soil layers was modelled by estimating an elastic modulus of deformation based on the SPT “N”-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in CHBDC (2006) and adjusted, as necessary.

The consolidation settlement of the cohesive deposit was assessed by evaluating compressibility parameters (i.e. σ_p' , C_r , C_c , $C_{\alpha\epsilon}$) from the results of the laboratory consolidation tests and the in-situ field vane tests, along with the results of the laboratory index tests and using empirical correlations proposed in literature by Terzaghi and Peck (1967), Nishida (1956) and Azzouz et al. (1976). The coefficient of consolidation, c_v , use in the time/rate settlement analysis for the over-consolidated soils at this site is estimated to be $1.1 \times 10^{-2} \text{ cm}^2/\text{s}$, based on the consolidation tests and correlations with natural water content.

The simplified stratigraphy together with the associated compressibility parameters and unit weights employed for the different soil types at the approach embankments are presented.

Soil Type	γ (kN/m ³)	σ_p' (kPa)	e_o	C_c	C_r	Elastic Modulus (MPa)
Stiff to hard clayey silt fill	19	σ_{vo}'	0.5	0.09	0.009	--
Stiff to hard clayey silt to silty clay	19	300	0.5	0.09	0.009	--
Soft to stiff silty clay*	19	300	0.5	0.09 to 0.17	0.009 to 0.017	--
Compact to very dense silt	20	--	--	--	--	50
Very stiff to hard clayey silt till	20	--	--	--	--	150
Dense to very dense sand	20	--	--	--	--	150

* The properties of this deposit vary with depth

6.8.3.2 Settlement Performance Criteria

The settlement performance criteria for design of high fill embankments are in accordance with MTO's Guideline “Embankment Settlement Criteria for Design” (2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75
	>75 m	<100

The above criteria, and limiting differential settlement to 200:1, have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankment grade raise. The settlement criteria are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These settlement criteria form part of the overall design performance for the approach embankments approaching the new replacement bridge. As such, the critical sections for the settlement analysis are at the abutments and at 20 m, 50 m and 75 m beyond the abutments.

6.8.3.3 Analysis Results

A summary of the results of the settlement analysis at the critical sections below the embankment widening/grade raise, assuming embankment construction with granular fill, is presented below.

Approx. Station	Location	Approximate Proposed Embankment Height (m)	Settlement Criteria (mm)	Estimated Total Settlement (mm)	Estimated Immediate Settlement (mm)	Estimated Post-Construction Settlement (mm)
North Abutment and Approach						
9+960	North abutment	8.8	25	75	20	55
9+940	20 m north	8.5	50	70	20	50
9+910	50 m north	7.7	75	60	15	45
9+885	75 m north	7.0	100	60	15	45
South Abutment and Approach						
10+050	South abutment	8.7	25	75	20	55
10+070	20 m south	8.4	50	70	20	50
10+100	50 m south	7.5	75	60	15	45
10+125	75 m south	6.5	100	55	10	45

As shown above, only within the first 20 m from the abutment is the post-construction primary consolidation settlement estimated to be greater than MTO's settlement performance criteria. The following sections discusses potential mitigation measures for this area.

6.8.3.4 Settlement Mitigation

Settlement calculations indicate that about 75 mm of settlement may occur as a result of embankment construction; the founding soils are relatively stiff and over-consolidated, but the cohesive deposits are relatively thick and stresses induced by the new embankments penetrate deeply because of the wide and high embankment geometry. It is estimated that about one-third of this settlement will occur during construction and the remainder of the settlement should be complete within about two to three years following completion of embankment construction. Even though these settlement estimates are based on site-specific information, published research and recognised analysis methods, settlement estimates in geotechnical engineering tend to be subject to a degree of uncertainty on the order of $\pm 20\%$ even with a significant body of site-specific detailed laboratory and field testing. In addition, the magnitude of post-construction settlement could have some impact(s) on the design of an approach slab/sleeper slab system.

Potential methods for mitigating post-construction settlements have been assessed and are summarized below:

- **Preload/Surcharge with Vertical Drains** – Prefabricated vertical drains (wick drains) are typically used in softer cohesive soils to rapidly dissipate excess pore water pressures and, hence, accelerate settlements. This alternative is not considered to be appropriate or cost-effective at this site due to the relatively stiff and over-consolidated nature of the site soils, and the relatively small magnitude of estimated settlement under the embankment loading. In softer soils, wick drains can be rapidly installed by pushing them into the ground with a mandrel system; however, at this site, the stiff to very stiff silty clay to clayey silt strata are not suitable for installation by this method, and pre-drilled boreholes would be required. It is estimated that this would take one to two months to complete, followed by another two to four months to reduce post-construction settlements to meet MTO's performance criteria; however, it is understood that this project is expected to have an accelerated schedule to achieve structure replacement, so this likely cannot be accommodated in the schedule. It is estimated that the cost of the embankments would approximately triple to include the drains, compared to conventional construction with no special settlement mitigation measures.
- **Construct Approaches with Lightweight Fill** – Portions of the new approach embankments could be constructed using lightweight fill materials such as expanded polystyrene (EPS) or lightweight cellular concrete. In order to meet MTO's settlement performance criteria, the lightweight fill would be placed to about two-thirds of the embankment height and extend back from the abutments at least 10 m. Beyond this 10 m zone, the lightweight fill height would be tapered to intersect the original ground at an inclination of about 2H:1V, to provide a settlement and fill type transition zone. Conventional fill would be placed over the lightweight fill to construct the remainder of the embankments to grade, construct the pavements and flatten the slopes. While this alternative will address the short- and long-term settlement concerns, it is estimated that embankment construction costs would be about three to five times greater compared to conventional earth-fill embankment construction without settlement mitigation measures.
- **"Do Nothing" with Approach Slabs and Provision to Correct Road Profile** – Because the magnitude of the estimated settlements is not excessive, conventional embankment construction using SSM, granular or earth fill and without any settlement mitigation could be considered. For this option, it is recommended to monitor ground settlement from initial embankment construction through to paving; this construction-phase settlement monitoring would permit refinement of the estimate of post-construction ground and embankment settlement performance. Following paving, long-term settlement points would be strategically located to permit monitoring of any finished roadway settlements with time. Provided that a provision for milling and paving of the surface course asphalt and any adjustment to guiderails and the like are included in the contract to be

completed about two to three years after construction, correction of the roadway profile to meet MTO performance criteria could be carried out. There remains some probability that this treatment may not even be necessary depending on the actual settlement behaviour of the cohesive deposits beyond completion of the embankment filling and roadway paving. To reduce the potential for needing to carry out repairs, it is recommended that the approach slab/sleeper slab design be developed to accommodate approximately 50 mm of post-construction settlement and a subsequent mill and overlay treatment, if this design approach is implemented.

Based on this assessment, from a geotechnical/foundations engineering perspective, the “do nothing” alternative with an appropriate approach slab design and with provision for future grade correction is likely to be the most cost-effective alternative and, in Golder’s opinion, represents the preferred alternative.

At the south approach, the existing City of London watermain crosses under Highway 401 and ends under the proposed footprint of the widened embankment. An assessment of the anticipated settlement impacts on the watermain under the loading from the south approach embankment widening (assuming the use of conventional granular fill for the embankment widening) has been carried out. Golder’s analyses estimate a maximum of about 25 mm of settlement where the watermain will be under the new eastern side slope of the widened embankment.

It is understood that this amount of settlement cannot be tolerated by the watermain and that prior to construction of the embankment widening the watermain will be relocated to the east to beyond the limits of the widened embankment.

6.9 Overhead Sign Foundations

As part of the replacement of the Dingman Drive underpass a cantilever overhead sign is to be constructed approximately 30 m east of the Dingman Drive underpass at approximately Station 22+078. These foundation recommendations presented in this section of the report are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the nearby Dingman Drive underpass site.

6.9.1 Design of Sign Support Foundations

Caisson foundations for sign supports should be designed in accordance with the requirements in MTO’s *Sign Support Manual* (MTO, 2015). The *Sign Support Manual* includes standard caisson foundation designs for each sign type as follows:

- **Cantilever Signs:** Cantilever Static Sign Supports, Section 3 and Standard Drawings SS118-3, SS118-4 and SS118-5.
- **Trichord Overhead Signs:** Tri-Chord Static Sign Supports, Section 4 and Standard Drawings SS118-3, SS118-4 and SS118-5.

In the standard caisson foundation design, the caisson is extended 5 m to 6.5 m below the design frost depth (taken as 1.2 m as per OPSD 3090.101 Foundation Frost Penetration Depths for Southern Ontario) resulting in a total length of 6.2 to 7.7 m below final grade depending on the sign class and corresponding caisson diameter. The standard sign foundation designs presented in the MTO’s *Sign Support Manual* have been developed based on the minimum soil conditions given below.

- **Case 1 (Non-Cohesive Soils):** Sand with a friction angle of 28 degrees surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and sand with a friction angle of 30 degrees surrounding the lower third of the portion of the caisson below the design frost depth.
- **Case 2 (Cohesive Soils):** Soft clay with an undrained shear strength of 25 kPa surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and “soft” clay with an undrained shear strength of 50 kPa surrounding the lower third of the portion of the caisson below the design frost depth.

The standard foundation design provided in MTO’s *Sign Support Manual* does not apply to sites where extensive poor fill materials or materials looser or softer than those of Case 1 or Case 2 are present. The standard foundation design is also not applicable where bedrock is encountered within the standard foundation depth. For such subsurface conditions, a site-specific design is required.

Based on the subsurface conditions in the boreholes advanced closest to the proposed sign support location, a standard design is applicable at this site, provided that the sign board (i.e., wind loading area) is within the limits outlined in MTO’s *Sign Support Manual*.

6.9.1.1 Site-Specific Caisson Foundation Design in Soil

If the sign board is larger than the standards outlined in MTO’s *Sign Support Manual*, a site-specific caisson foundation design may be carried out by the structural engineer using the geotechnical design parameters given in the following table.

Stratum	Depth Relative to Proposed Ground Surface (m) ¹	Elevation (m)	Design Groundwater Elevation (m)	Design Parameters ²					
				S_u (kPa)	Φ'	γ (kN/m ³)	γ' (kN/m ³)	K_p	$K_{p2:1}$
Stiff to firm clayey silt fill	0.0 – 4.0	258.1 – 254.1	254.6	50	28	20	10	2.8	1.1
Stiff to very stiff clayey silt to silty clay	4.0 – 12.5	254.1 – 241.6		100	32	21	11	3.3	1.3

1. If the finished ground surface elevation surrounding the sign foundation varies from that given above, the elevations and depths must be adjusted accordingly.
2. Where both undrained and effective stress parameters are provided, both cases should be checked for design.

In the design of the sign foundation, the passive resistance within the upper 1.2 m below ground surface should be neglected to account for frost action. The unfactored lateral resistance should be calculated assuming an equivalent width equal to three times the caisson diameter. A resistance factor of 0.5 should be applied to this unfactored lateral resistance to obtain the factored lateral geotechnical resistance at Ultimate Limit Status (ULS).

The General Arrangement drawing indicates that the sign support will be constructed in an area of relatively flat ground; however, where sign support foundations are located on a slope or within approximately two diameters of the crest of a slope in the direction of loading, there would be unbalanced earth pressures around the foundation due to it being located within sloping ground. For such a case, the passive earth pressure coefficient must be adjusted to account for sloping ground; the $K_{p2:1}$ values provided above address 2H:1V sloping ground in the direction of loading.

6.10 Analytical Testing for Construction Materials

The results of an analytical test on three samples of the clayey silt deposit are presented in Section 4.4 and in Appendix C. The analytical test results were compared to CSA A23.1 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 three samples of soil tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

The analytical test results of the soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The chloride concentrations and the resistivity measured in the soil samples indicate "corrosive" to "moderately corrosive". Based on the results of the samples tested, and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a "C" type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 "Durability Requirements" are followed.

6.11 Construction Considerations

6.11.1 Excavations and Groundwater Control

Excavations for pile caps and/or abutments will penetrate the existing fill materials (including those of the existing embankment) and extend into the silty clay to clayey silt deposits. The groundwater level is expected to be at about Elevation 254.5 m and will fluctuate seasonally. The excavations may extend below the groundwater level; however, seepage volumes from the cohesive founding soils are expected to be low, although excavations may encounter some groundwater "perched" within non-cohesive fill or native materials atop the clayey soils. If necessary, groundwater control for such seepage may be achieved by pumping from properly constructed and filtered sumps in the base of the excavation in accordance with OPSS.PROV 517, with sumps maintained outside of the foundation limits. Special Provision (SP) 517F01 is not strictly required from a foundations perspective, but the designers may wish to fill in the applicable storm event return period in this SP to address precipitation/surface water, although surface water runoff should be directed away from the excavations at all times. The Contractor shall be responsible for the selection, performance and detailed design of the design of dewatering, unwatering, and temporary flow passage system.

Excavations should be completed in accordance with OPSS 902 as amended by SP 109S12, and in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The existing fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The upper portion of the native clayey silt to silty clay deposit would be classified as Type 2 soils.

6.11.2 Temporary Protection Systems

To support the excavation sides and permit the use of vertical cuts, temporary protection systems will be required where space is restricted and will not permit the use of open cuts. These systems are to be designed by and the limits determined by the contractor.

Temporary protection systems could consist of soldier piles and lagging, where the H-piles would be driven or installed within a pre-bored hole to a suitable depth and horizontal lagging installed as the excavation proceeds, or

driven steel sheet piling. Support of the system(s) could be in the form of struts and walers in the case of pile cap/abutment excavations or rakers and anchors. The protection system must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as address the impact(s) of sloping ground behind the system. The lateral movement of the temporary support system should meet Performance Level 2 as specified in OPSS.PROV 539.

Vibrations will be induced by the installation of temporary protection systems, and vibration monitoring is recommended as discussed further in Section 6.10.6 and 6.10.7 below.

6.11.3 Shallow Foundations – Subgrade Protection

The founding soils are sensitive to disturbance and softening due to water seepage and/or ponding and construction equipment or foot traffic when moist to wet. Placement of a concrete working slab (100 mm thick, 20 MPa concrete) is recommended at the base of the excavations for the footing areas. If shallow foundations are adopted, the NSSP for this item is provided in Appendix D, for inclusion in the Contract Documents.

6.11.4 Deep Foundations

Cobbles and boulders should be expected in the soils at the site and which may impact pile driving or caisson drilling operations. An NSSP or Notice to Contractor should be added to the Contract Documents to alert the contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile or caisson installation; a draft Notice to Contractor is presented in Appendix D.

Deep foundations should be installed and monitored in accordance with OPSS.PROV 903 and Ontario Provincial Standard Drawing (OPSD) 3000.150 and 3001.150 for H-piles or tube piles, respectively, as well as SS103-11 (Pile Driving Control). It is also recommended that Pile Dynamic Analyzer (PDA) testing be completed on a representative portion of piles driven at each foundation element and in each stage, to augment or replace pile driving control using the Hiley formula. We have modified SP 903S06, which amends OPSS.PROV 903 (*Deep Foundations*), to address a wait period of up to 48 hours between initial driving and re-tapping, as well as the requirement for PDA testing, for inclusion in the Contract Documents (see Appendix D).

6.11.5 Ground and Groundwater Control for Drilled Shaft Installation

As discussed in Section 6.6, running or flowing of water-bearing cohesionless soils (silt deposits) could occur during or after drilling of drilled shafts. If drilled shaft foundations are adopted for support of any of the foundation elements, temporary liners potentially in conjunction with drilling fluids would be required to support the soils during construction and permit inspection and cleaning of the base. It is recommended that an NSSP or Notice to Contractor be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; a draft Notice to Contractor is presented in Appendix D.

6.11.6 Existing and Relocated Watermain

If the existing City of London watermain were to remain in place, operational constraints or other vibration mitigation measures and vibration monitoring would be required during pile driving. However, based on settlement considerations under the eastward embankment widening, this existing watermain is to be relocated to the east, outside of the zone of influence of vibration impacts and settlement. Therefore, no vibration or settlement monitoring is required on the relocated watermain nor on the portions of the existing watermain that will remain in place beyond the limits of the relocation.

7.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng. and an independent technical and quality control review of this report was carried out by Ms. Lisa Coyne, P.Eng., a Principal and MTO Designated Foundations Contact for Golder.

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- Ontario Department of Mines and Northern Affairs, *Quaternary Geology of the Goderich Area, Southern Ontario*, Prelim. Map P1232, Geol. Ser., scale 1:50,000. 1977.
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- Ministry of Transportation, *MTO Gravity Pipe Design Guidelines*, April 2014
- National Resources Canada Earthquake Hazard Website. <http://earthquakescanada.nrcan.gc.ca/hazard-alea/index-eng.php>. Accessed on December, 2018.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
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Ontario Provincial Standard Drawings

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specifications

OPSS.PROV 206	Construction Specifications for Grading.
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling – Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous

OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 109S12	Amendment to OPSS 902
SP 517F01	Amendment to OPSS 517

Ontario Water Resources Act

Ontario Regulation 903, Wells (as amended)

Ontario Occupational Health and Safety Act

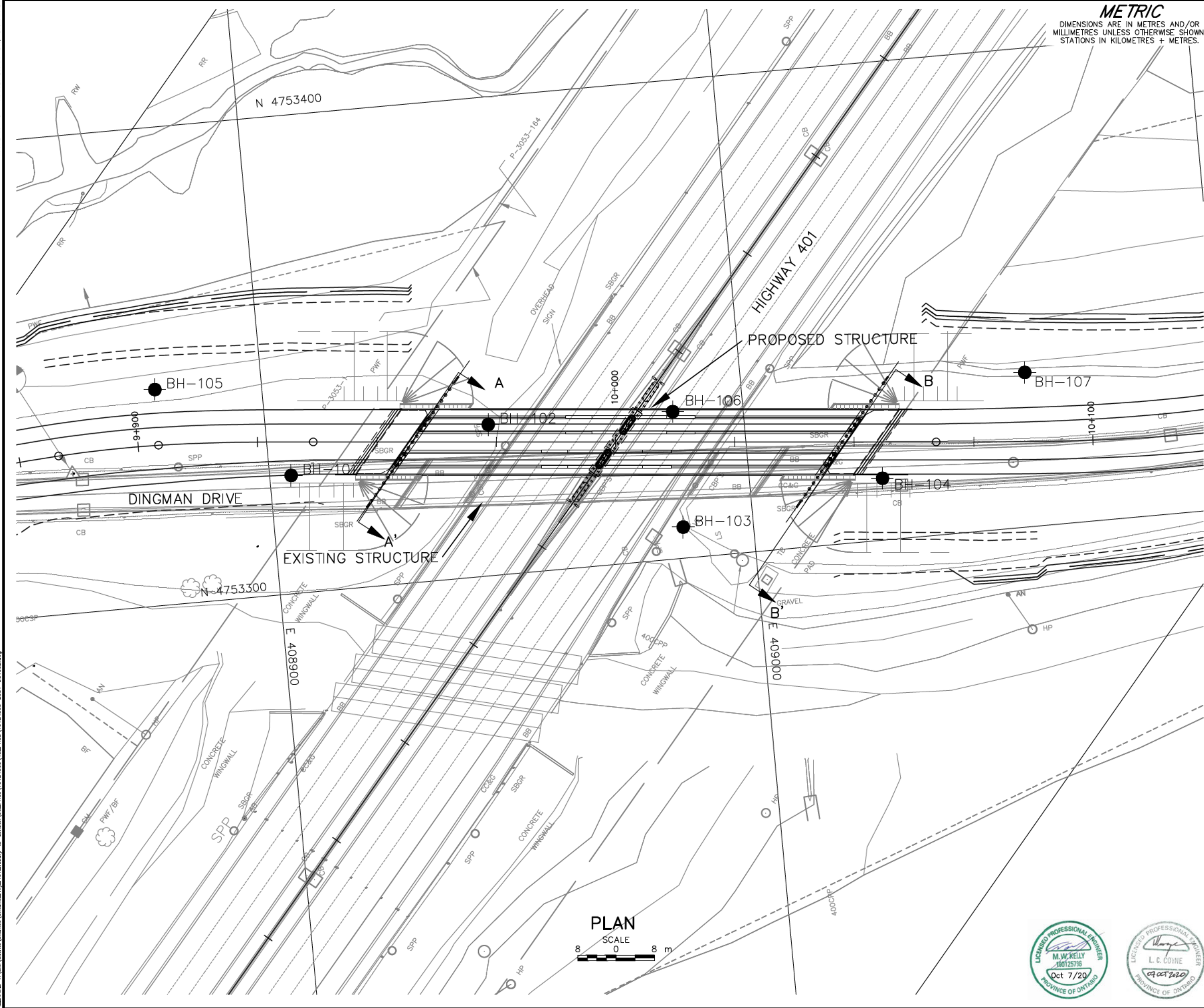
Ontario Regulation 213/91, Construction Projects (as amended)

**TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – HIGHWAY 401 / DINGMAN DRIVE UNDERPASS STRUCTURE
G.W.P. 3025-18-00**

Option	Feasibility	Advantages	Disadvantages	Geotechnical Risks/Consequences	Constructability	Relative Costs
Strip or spread footings founded on the stiff to very stiff clayey silt to silty clay	<ul style="list-style-type: none"> Feasible for pier and abutment foundations, although would have to design for total and differential settlements 	<ul style="list-style-type: none"> Existing abutments are supported on shallow foundations, and appear to have performed well although they have likely experienced total settlements on the order of 75 mm. Only minor groundwater seepage anticipated, so pumping from filtered sumps expected to provide adequate groundwater control. 	<ul style="list-style-type: none"> Low geotechnical resistances for 25 mm of settlement Construction of approach embankments would result in settlements of up to about 75 mm that would impact abutments. Temporary protection system required along Dingman Drive for construction of abutments. Precludes use of integral foundations; potentially greater maintenance required. 	<ul style="list-style-type: none"> Greater risks than deep foundation options for total and differential settlement between and across foundation elements. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations, excluding protection systems.
Strip footings perched in approach embankments on granular pad	<ul style="list-style-type: none"> Feasible for abutment foundations, although would have to design for total and differential settlements 	<ul style="list-style-type: none"> No groundwater seepage anticipated. Higher ultimate bearing resistance than for shallow footings founded on native soil deposits, although similar limitations related to serviceability/settlement. 	<ul style="list-style-type: none"> Construction of approach embankments would result in settlements of up to about 75 mm; impacts serviceability resistance for footings. Temporary protection system may be required along Dingman Drive for construction of abutments. Precludes use of integral foundations; potentially greater maintenance required. 	<ul style="list-style-type: none"> Greater risks than deep foundation options for potential differential settlement between and across foundation elements. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations, excluding protection systems.

Option	Feasibility	Advantages	Disadvantages	Geotechnical Risks/Consequences	Constructability	Relative Costs
Driven steel H-piles or pipe piles founded within the very dense/hard glacial till deposits	<ul style="list-style-type: none"> Feasible and preferred for support of bridge abutments and potentially centre pier, given sufficient working area in highway median 	<ul style="list-style-type: none"> Negligible post-construction settlement of foundation elements, although must accommodate downdrag loads Can be used for support of integral abutments. Minor groundwater seepage anticipated in pile cap excavations, so pumping from filtered sumps will provide adequate groundwater control. 	<ul style="list-style-type: none"> Vibration monitoring of nearby watermain would be required during construction if watermain remains in place Possibility of piles “hanging up” on or being damaged by cobbles/boulders Long piles, potentially with a splice will be required to reach “100-blow” materials 	<ul style="list-style-type: none"> Negligible risk of post-construction settlements Moderate to high risks around vibration impacts to existing watermain during pile driving unless relocated Risk (albeit low) of piles “hanging up” on or being damaged by cobbles/boulders and not achieving design resistance 	<ul style="list-style-type: none"> Conventional construction methods for driven piles 	<ul style="list-style-type: none"> Higher costs than strip footings Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction Potentially lower life cycle costs
Drilled shafts founded within dense silt or hard clayey silt deposit at a depth of approximately 20 m	<ul style="list-style-type: none"> Feasible for support of bridge foundations 	<ul style="list-style-type: none"> Potentially reduced number of deep foundation elements compared to driven piles Potential for lower vibrations particularly if liners installed using oscillatory equipment. Negligible post construction settlement. Pile caps could potentially be eliminated if pier columns extend from top of drilled shafts. Minor groundwater seepage in pile cap excavations, so pumping from filtered sumps will provide adequate groundwater control. 	<ul style="list-style-type: none"> Temporary liners filled with water or controlled-density drilling fluids required during construction to control the ground and groundwater within the water-bearing cohesionless soils Concrete would have to be placed by tremie methods Cleaning of the base below the water table could be difficult Not suitable for integral abutment design Greater risk of encountering obstructions due to larger size of drill hole required 	<ul style="list-style-type: none"> Moderate to high risk of disturbance of water-bearing non-cohesive soils, requiring use of temporary liners and tremie concrete Some risk of vibration impacts on nearby watermain (unless relocated) depending on installation techniques for liners; specifications can be developed to mitigate Negligible risk of post-construction settlement of bridge foundations 	<ul style="list-style-type: none"> Conventional construction methods for drilled shaft foundations; temporary liners required for ground and groundwater control; may require specialized liner installation to minimize vibrations if existing watermain is not relocated 	<ul style="list-style-type: none"> Higher cost than steel H-piles Installation cost could be impacted by need for liner to minimize disturbance and loss of ground and for tremie concrete placement. Estimated cost is approximately \$1000/m length for caisson installation and \$600/m³ for pile cap construction.

Drawings



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

CONT No. 2020-3062
WP No. 3199-19-01

DINGMAN DRIVE UNDERPASS
HIGHWAY 401
BOREHOLE LOCATIONS

SHEET

KEY PLAN

LEGEND

Borehole (Current Investigation)

No.	ELEVATION	CO-ORDINATES (MTM ZONE 11)	
		NORTHING	EASTING
101	263.80	4 753 324.7	408 904.5
102	258.07	4 753 331.6	408 946.6
103	258.43	4 753 306.5	408 985.3
104	264.55	4 753 312.9	409 027.8
105	256.31	4 753 345.2	408 877.7
106	258.90	4 753 330.8	408 985.3
107	257.50	4 753 332.3	409 059.5

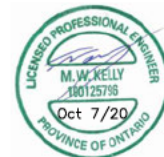
NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

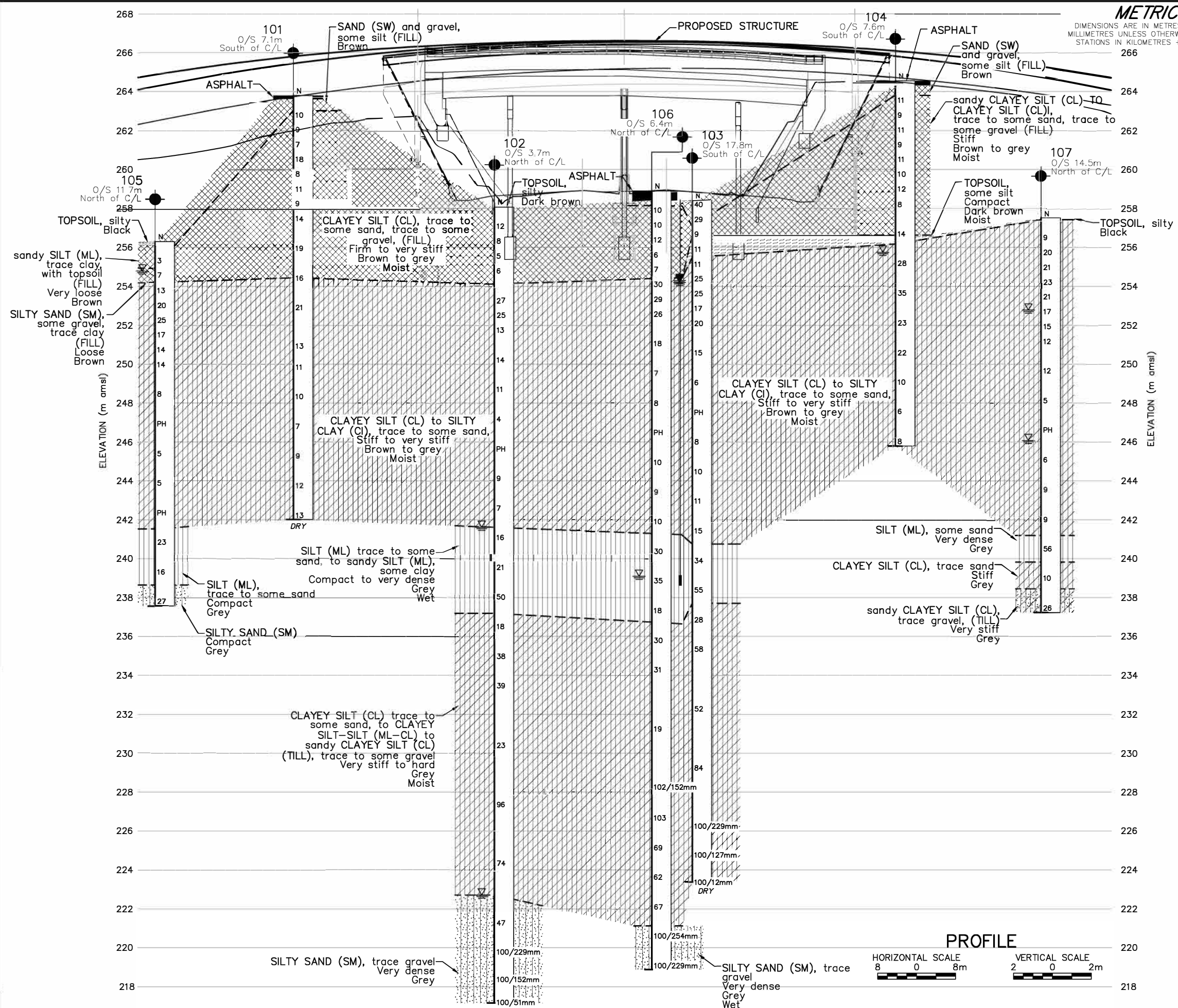
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Stantec.



NO.	DATE	BY	REVISION
1	2020	ZJB	ISSUED FOR CONSTRUCTION
Geocres No. 4014-194			
HWY.	401	PROJECT NO.	19124560
SUBM'D.	AC	CHKD.	MWK
DRAWN:	ZJB	CHKD.	MWK
DATE:	Oct 7/20	APPD.	LCC
SITE:	19X-0368/BO	DWG.	1

CONT No. 2020-3062
WP No. 3199-19-01DINGMAN DRIVE UNDERPASS
HIGHWAY 401
SOIL STRATA

SHEET



LEGEND

- Borehole (Current Investigation)
- N** Standard Penetration Test Value
- 16** Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured
- WL encountered during drilling
- DRY** Water level not established
- Vibrating Wire Piezometer

No.	ELEVATION	CO-ORDINATES (MTM ZONE 11)	
		NORTHING	EASTING
101	263.80	4 753 324.7	408 904.5
102	258.07	4 753 331.6	408 946.6
103	258.43	4 753 306.5	408 985.3
104	264.55	4 753 312.9	409 027.8
105	256.31	4 753 345.2	408 877.7
106	258.90	4 753 330.8	408 985.3
107	257.50	4 753 332.3	409 059.5

NOTES

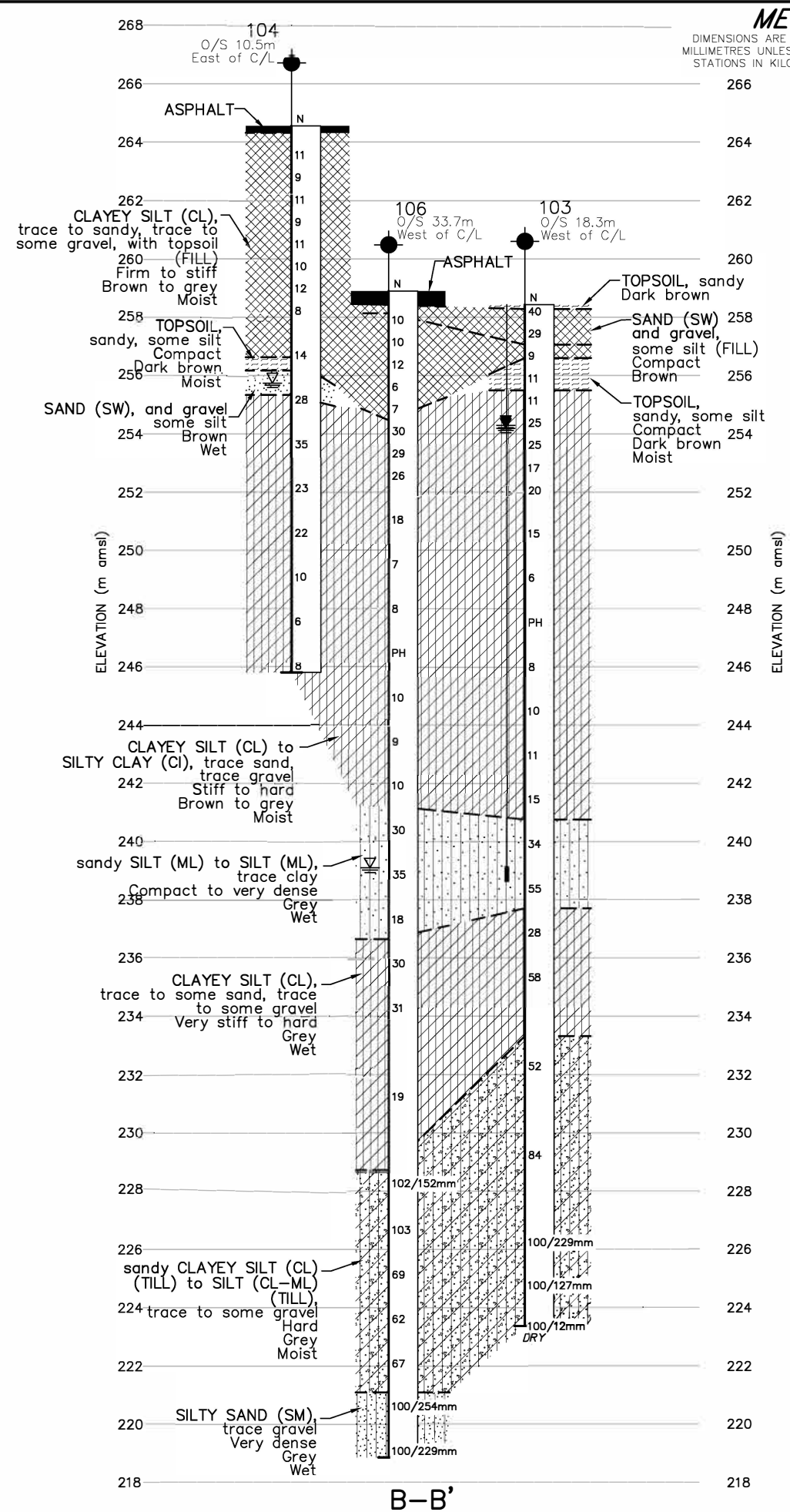
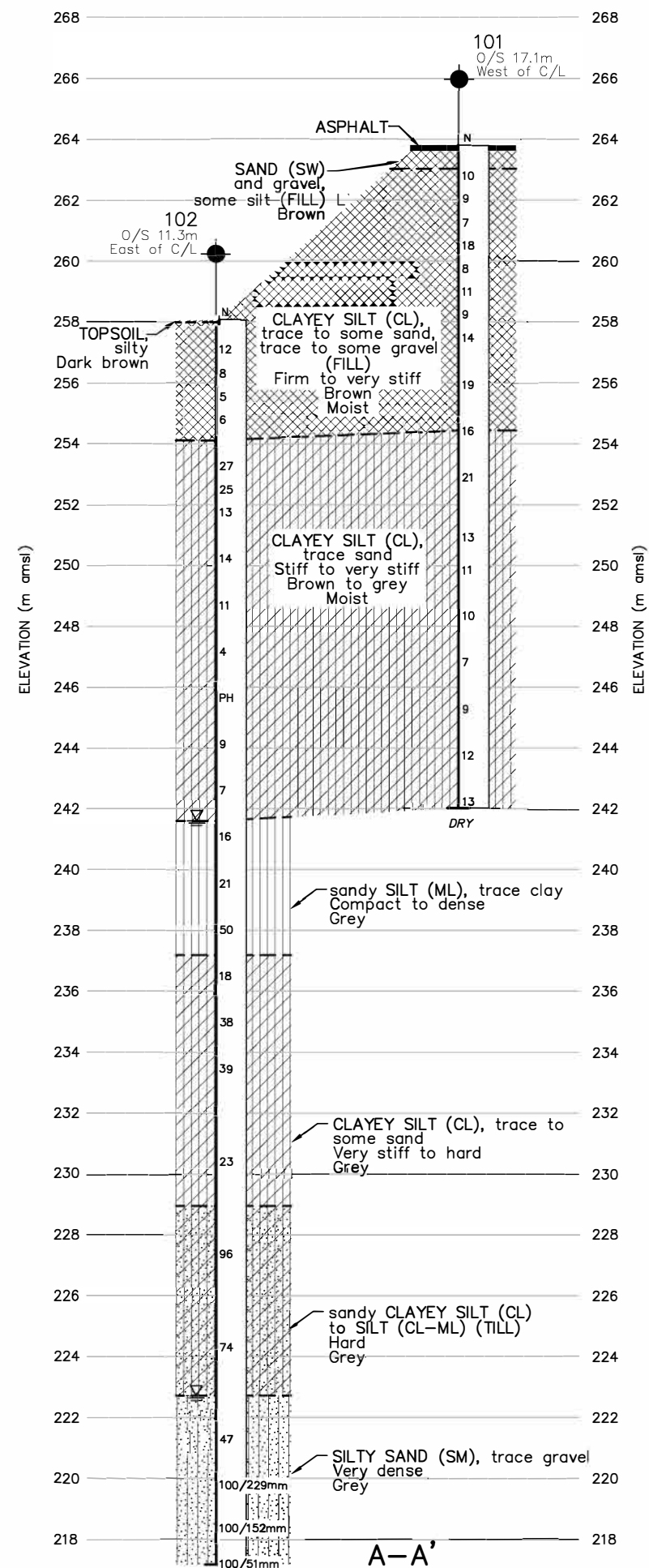
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The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by STANTEC.

NO.	DATE	BY	REVISION
Geocres No. 40114-194			
HWY.	401	PROJECT NO.	19124560
SUBM'D.	AC	CHKD.	MWK
DRAWN:	ZJB	CHKD.	MWK
DATE:	Oct 7/20	APPD.	LCC
SITE:	19X-0368/BO	DWG.	2

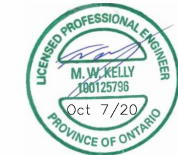
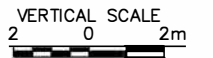
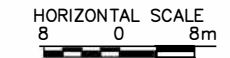


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.





CONT No. 2020-3062
WP No. 3199-19-01

DINGMAN DRIVE UNDERPASS
HIGHWAY 401
CROSS-SECTIONS A-A' AND B-B'

SHEET



LEGEND

- | | |
|---|--|
|  | Borehole (Current Investigation) |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
|  | WL measured |
|  | WL encountered during drilling |
| DRY | Water level not established |
|  | Vibrating Wire Piezometer |

No.	ELEVATION	CO-ORDINATES (MTM ZONE 11)	
		NORTHING	EASTING
101	263.80	4 753 324.7	408 904.5
102	258.07	4 753 331.6	408 946.6
103	258.43	4 753 306.5	408 985.3
104	264.55	4 753 312.9	409 027.8
106	258.90	4 753 330.8	409 059.5

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REFERENCE

Base plans provided in digital format by STANTEC.

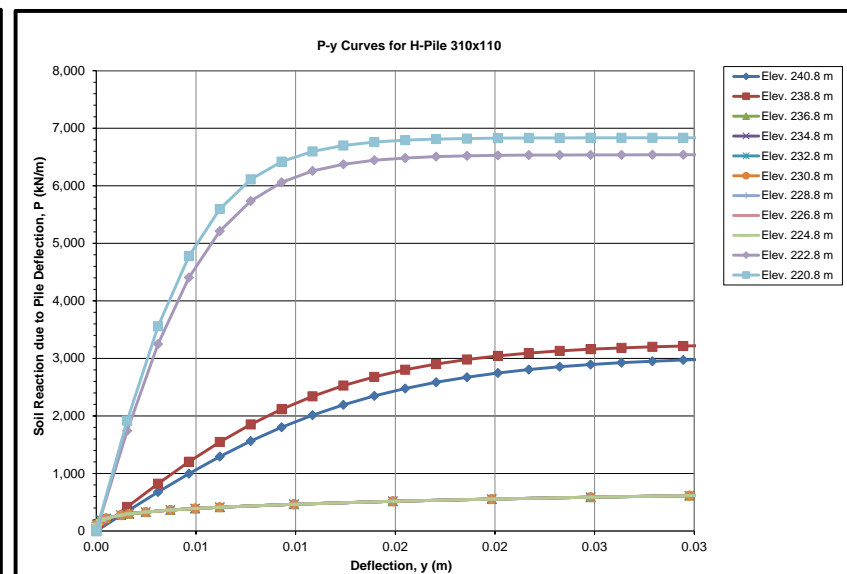
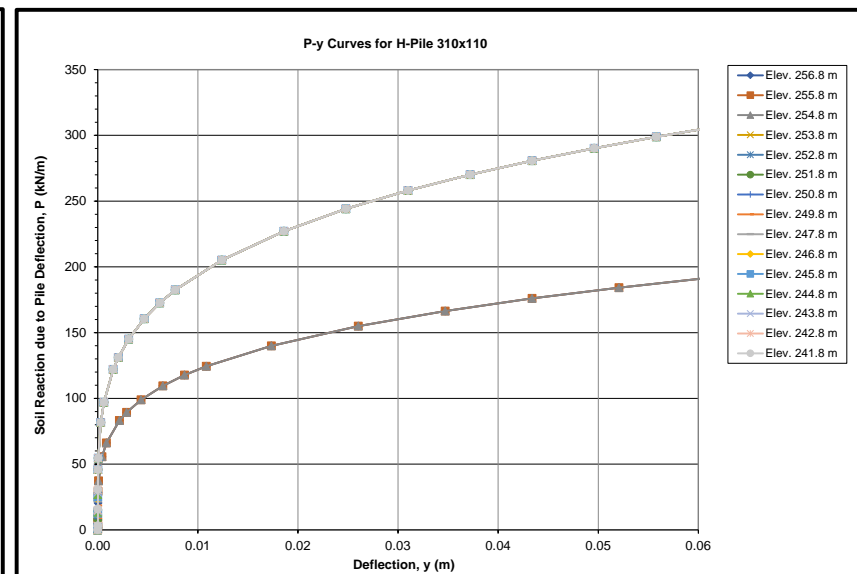
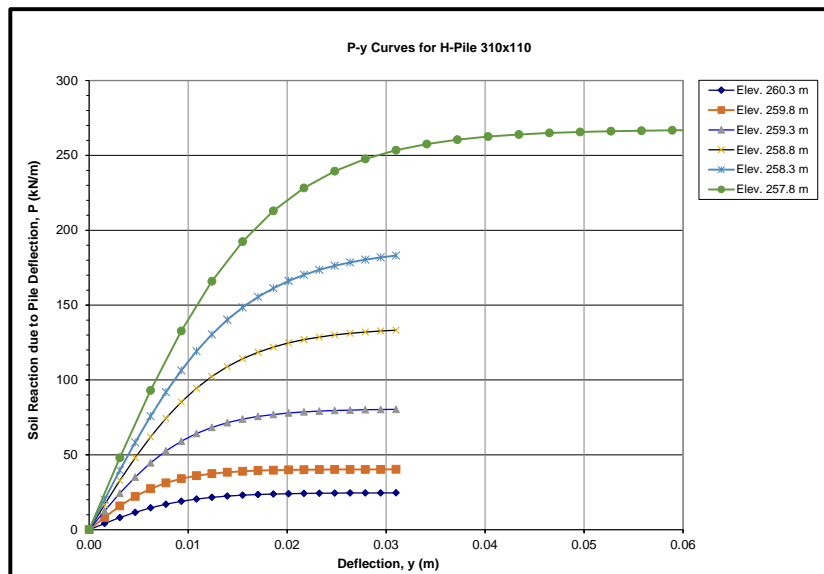
NO.	DATE	BY	REVISION
Geocres No. 4014-194			
HWY. 401	PROJECT NO. 19124560		DIST.
SUBM'D. AC	CHKD. MWK	DATE: Oct 7/20	SITE: 19X-0368/B0
DRAWN: ZJB	CHKD. MWK	APPD. LCC	DWG. 3

Figures

19124560 Dingman Drive Underpass Abutment HP 310x110

FIGURE 1

Description Depth (z) * Elevation P-y Curves	Loose Fill												Firm to Very Stiff Clayey Silt Fill						Stiff to Very Stiff Silty Clay to Clayey Silt																			
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 4.0 m		z= 5.0 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m			
	Elev. 260.3 m		Elev. 259.8 m		Elev. 259.3 m		Elev. 258.8 m		Elev. 258.3 m		Elev. 257.8 m		Elev. 256.8 m		Elev. 255.8 m		Elev. 254.8 m		Elev. 253.8 m		Elev. 252.8 m		Elev. 251.8 m		Elev. 250.8 m		Elev. 249.8 m		Elev. 248.8 m		Elev. 247.8 m		Elev. 246.8 m		Elev. 245.8 m			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)		
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
0.00155	1.41517	0.00155	8.25066667	0.00155	12.455	0.00155	16.654333	0.00155	20.0122	0.0031	48.01807	8.7E-07	0.4102429	8.7E-07	0.585875	8.7E-07	0.703075	6.2E-07	1.1720335	6.2E-07	1.339	6.2E-07	1.5069665	6.2E-07	1.6743415	6.2E-07	1.8403333	6.2E-07	2.0086667	6.2E-07	2.176	6.2E-07	2.3443333	6.2E-07	2.511			
0.0031	8.06207	0.0031	15.832	0.0031	24.33	0.0031	32.809333	0.0031	39.5762	0.0062	93.02767	4.3E-06	2.0515714	4.3E-06	2.92925	4.3E-06	3.51525	3.1E-06	4.5891675	3.1E-06	6.696	3.1E-06	7.5330335	3.1E-06	8.3717073	3.1E-06	9.2036667	3.1E-06	10.042381	3.1E-06	10.88	0.000031	11.717619	3.1E-06	12.552			
0.00465	11.5702	0.00465	22.278667	0.00465	35.149	0.00465	48.022333	0.00465	58.285	0.0093	132.7313	8.7E-06	4.1024286	8.7E-06	5.85875	8.7E-06	7.03075	6.2E-06	11.720335	6.2E-06	13.39	6.2E-06	15.069665	6.2E-06	16.743415	6.2E-06	18.403333	6.2E-06	20.086667	6.2E-06	21.76	0.0000062	23.443333	6.2E-06	25.11			
0.0062	14.5647	0.0062	27.41	0.0062	44.605	0.0062	61.955667	0.0062	75.7998	0.0124	165.9207	4.3E-05	20.515714	4.3E-05	29.2925	4.3E-05	31.29	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	0.000031	45.89	3.1E-05	45.89			
0.00775	17.0173	0.00775	31.287333	0.00775	52.574	0.00775	74.395667	0.00775	91.8766	0.0155	192.4547	8.7E-05	37.21	8.7E-05	37.21	8.7E-05	37.21	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	0.000062	54.58	6.2E-05	54.58	
0.0093	18.961	0.0093	34.103	0.0093	59.087	0.0093	85.253	0.0093	106.369	0.0186	212.898	0.00043	55.64	0.00043	55.64	0.00043	55.64	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	
0.01085	20.4593	0.01085	36.091333	0.01085	64.277	0.01085	94.542667	0.01085	119.22	0.0217	228.2347	0.00087	66.17	0.00087	66.17	0.00087	66.17	0.00062	97.05	0.00062																		

[illegible]

3 There are no pile group effects (i.e. analysis is based on a single pile)

Date: August 2020
Project No: 19124560

Prepared By: CC
Checked By: MWK



SUMMARY OF P-y CURVES FOR A H-Pile 310x110 at Abutments

Description Depth (z) * Elevation P-y Curves	Firm to Very Stiff Clayey Silt Fill												Stiff to Very Stiff Silty Clay to Clayey Silt																								
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 4.0 m		z= 5.0 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		
	Elev. 256.8 m		Elev. 256.3 m		Elev. 255.8 m		Elev. 255.3 m		Elev. 254.8 m		Elev. 254.3 m		Elev. 253.3 m		Elev. 252.3 m		Elev. 251.3 m		Elev. 250.3 m		Elev. 249.3 m		Elev. 248.3 m		Elev. 247.3 m		Elev. 246.3 m		Elev. 245.3 m		Elev. 244.3 m		Elev. 243.3 m				
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.7E-07	0.05859	8.7E-07	0.1171923	8.7E-07	0.175757	8.7E-07	0.234365	8.7E-07	0.292957	8.7E-07	0.339887	6.2E-07	0.6696239	6.2E-07	0.836962	6.2E-07	1.00416901	6.2E-07	1.1721338	6.2E-07	1.3390282	6.2E-07	1.5069155	6.2E-07	1.674	6.2E-07	1.8411429	6.2E-07	2.0094	6.2E-07	2.175857	6.2E-07	2.344	6.2E-07	2.510571		
4.3E-06	0.29298	4.3E-06	0.5858846	4.3E-06	0.878763	4.3E-06	1.1717762	4.3E-06	1.464483	4.3E-06	1.699035	3.1E-06	3.3486197	3.1E-06	4.1848099	3.1E-06	5.02183803	3.1E-06	5.8588662	3.1E-06	6.6959718	3.1E-06	7.5330845	3.1E-06	8.3698333	3.1E-06	9.2075	3.1E-06	10.042	3.1E-06	10.88429	0.0000031	11.72	3.1E-06	12.55286		
8.7E-06	0.58587	8.7E-06	1.1719231	8.7E-06	1.757566	8.7E-06	2.3436504	8.7E-06	2.929566	8.7E-06	3.398867	6.2E-06	6.6962394	6.2E-06	8.3696197	6.2E-06	10.0416901	6.2E-06	11.721338	6.2E-06	13.390282	6.2E-06	15.069155	6.2E-06	16.74	6.2E-06	18.411429	6.2E-06	20.094	6.2E-06	21.75857	0.0000062	23.44	6.2E-06	25.10571		
4.3E-05	2.92977	4.3E-05	5.8588462	4.3E-05	8.787629	4.3E-05	11.717762	4.3E-05	14.64483	4.3E-05	16.99035	3.1E-05	33.486197	3.1E-05	41.848099	3.1E-05	48.59	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	3.1E-05	45.89	0.000031	45.89	3.1E-05	45.89
8.7E-05	5.85869	8.7E-05	11.719231	8.7E-05	17.57566	8.7E-05	23.436504	8.7E-05	29.29566	8.7E-05	33.86902	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58	6.2E-05	54.58
0.00043	24.2885	0.00043	30.088462	0.00043	35.68986	0.00043	41.06021	0.00043	46.41986	0.00043	50.72028	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61	0.00031	81.61
0.00087	28.9162	0.00087	35.779231	0.00087	42.44545	0.00087	48.831818	0.00087	55.20545	0.00087	60.31909	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05	0.00062	97.05
0.00217	36.3638	0.00217	44.990769	0.00217	53.3728	0.00217	61.395804	0.00217	69.4128	0.00217	75.84441	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122	0.00155	122
0.00289	39.0692	0.00289	48.338462	0.00289	57.35056	0.00289	65.979161	0.00289	74.59056	0.00289	81.49888	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1	0.00207	131.1
0.00434	43.2392	0.00434	53.498462	0.00434	63.46818	0.00434	73.013706	0.00434	82.54818	0.00434	90.19161	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1	0.0031	145.1
0.00651	47.8562	0.00651	59.209231	0.00651	70.23979	0.00651	80.805315	0.00651	91.35378	0.00651	99.80839	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6	0.00465	160.6
0.00868	51.4223	0.00868	63.624615	0.00868	75.47545	0.00868	86.832797	0.00868	98.15748	0.00868	107.249	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6	0.0062	172.6
0.01085	54.3746	0.01085	67.276154	0.01085	79.8072	0.01085	91.814196	0.01085	103.7951	0.01085	113.3895	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5	0.00775	182.5
0.01736	61.15	0.01736	75.66	0.01736	89.7586	0.01736	103.22699	0.01736	116.7545	0.01736	127.5706	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2	0.0124	205.2
0.02604	67.6769	0.02604	83.733846	0.02604	99.33203	0.02604	114.27902	0.02604	129.214	0.02604	141.172	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1	0.0186	227.1
0.03472	72.7208	0.03472	89.978462	0.03472	106.7336	0.03472	122.75944	0.03472	138.8336	0.03472	151.7126	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1	0.0248	244.1
0.0434	76.8992	0.0434	95.141538	0.0434	112.8531	0.0434	129.83007	0.0434	146.8133	0.0434	160.3734	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1	0.031	258.1
0.05208	80.4854	0.05208	99.573846	0.05208	118.1329	0.05208	135.9007	0.05208	153.6329	0.05208	167.8343	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1	0.0372	270.1
0.06076	83.6462	0.06076	103.50615	0.06076	122.7727	0.06076	141.28112	0.06076	159.6727	0.06076	174.4748	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7	0.0434	280.7
0.06944	86.48	0.06944	106.96923	0.06944	126.9524	0.06944	146.06154	0.06944	165.0923	0.06944	180.3951	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2	0.0496	290.2
0.07812	89.0654	0.07812	110.20769	0.07812	130.7322	0.07812	150.35175	0.07812	170.0322	0.07812	185.7357	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9	0.0558	298.9
0.0868	91.4423	0.0868	113.14615	0.0868	134.2119	0.0868	154.43217	0.0868	174.572	0.0868	190.7559	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9	0.062	306.9
0.089	91.442	0.089	113.146	0.089	134.212	0.089	154.432	0.089	174.572	0.089	190.756	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9	0.06355	306.9

Description Depth (z) * Elevation P-y Curves	Compact to Very Dense Silt										Very Stiff to Hard Clayey Silt to Clayey Silt-Silt									
	z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 22.0 m		z= 24.0 m		z= 26.0 m		z= 28.0 m			
	Elev. 241.3 m		Elev. 240.3 m		Elev. 239.3 m		Elev. 238.3 m		Elev. 237.3 m		Elev. 235.3 m		Elev. 233.3 m		Elev. 231.3 m		Elev. 229.3 m			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)		
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
	0.00155	243.037	0.00155	267.97	0.000155	292.8833	0.00155	317.79333	0.00155	342.68	5E-07	5.6394	5E-07	6.1754	5E-07	6.7096667	5E-07	7.2458		
	0.0031	475.67	0.0031	523.63	0.000151	571.49	0.0031	619.25667	0.0031	666.95	2.5E-06	28.194	2.5E-06	30.880667	2.5E-06	33.544667	2.5E-06	36.2313333		
	0.00465	689.2	0.00465	756.84	0.00465	824.18	0.00465	891.26	0.00465	958.13	5E-06	56.394	5E-06	61.754	5E-06	67.096667	5E-06	69.75		
	0.0062	877.787	0.0062	961.02	0.0062	1043.667	0.0062	1125.8	0.0062	1207.6	2.5E-05	104.3	2.5E-05	104.3	2.5E-05	104.3	2.5E-05	104.3		
	0.00775	1038.77	0.00775	1133.5	0.00775	1227.233	0.00775	1320.3333	0.00775	1412.7	5E-05	124	5E-05	124	5E-05	124	5E-05	124		
	0.0093	1172.23	0.0093	1274.7	0.0093	1376.033	0.0093	1476.4	0.0093	1576	0.00025	185.5	0.00025	185.5	0.00025	185.5	0.00025	185.5		
	0.01085	1280.2	0.01085	1387.6	0.01085	1493.533	0.01085	1598.4333	0.01085	1702.4	0.0005	220.6	0.0005	220.6	0.0005	220.6	0.0005	220.6		
	0.0124	1365.83	0.0124	1475.9	0.0124	1584.5	0.0124	1691.8667	0.0124	1798.3	0.00124	277.4	0.00124	277.4	0.00124	277.4	0.00124	277.4		
	0.01395	1432.8	0.01395	1544	0.01395	1653.733	0.01395	1762.3	0.01395	1870.	0.00165	298	0.00165	298	0.00165	298	0.00165	298		
	0.0155	1484.4	0.0155	1595.9	0.0155	1705.933	0.0155	1814.8333	0.0155	1922.8	0.00248	329.8	0.00248	329.8	0.00248	329.8	0.00248	329.8		
	0.01705	1523.87	0.01705	1635	0.01705	1744.833	0.01705	1853.5333	0.01705	1961.5	0.00372	365	0.00372	365	0.00372	365	0.00372	365		
	0.0186	1553.77	0.0186	1664.3	0.0186	1773.867	0.0186	1882	0.0186	1989.7	0.00496	392.2	0.00496	392.2	0.00496	392.2	0.00496	392.2		
	0.02015	1576.4	0.02015	1686.2	0.02015	1794.867	0.02015	1902.7	0.02015	2010	0.0062	414.7	0.0062	414.7	0.0062	414.7	0.0062	414.7		
	0.0217	1593.37	0.0217	1702.4	0.0217	1810.433	0.0217	1917.8333	0.0217	2024.7	0.00992	466.4	0.00992	466.4	0.00992	466.4	0.00992	466.4		
	0.02325	1606.07	0.02325	1714.4	0.02325	1821.833	0.02325	1928.7333	0.02325	2035.2	0.01488	516.2	0.01488	516.2	0.01488	516.2	0.01488	516.2		
	0.0248	1615.6	0.0248	1723.2	0.0248	1830.167	0.0248	1936.7	0.0248	2042.8	0.01984	554.7	0.01984	554.7	0.01984	554.7	0.01984	554.7		
	0.02635	1622.67	0.02635	1729.8	0.02635	1836.3	0.02635	1942.4	0.02635	2048.3	0.0248	586.5	0.0248	586.5	0.0248	586.5	0.0248	586.5		
	0.0279	1627.97	0.0279	1734.6	0.0279	1840.733	0.0279	1946.5667	0.0279	2052.1	0.02976	613.9	0.02976	613.9	0.02976	613.9	0.02976	613.9		
	0.02945	1631.93	0.02945	1738.1	0.02945	1844	0.02945	1949.5667	0.02945	2054.9	0.03472	638	0.03472	638	0.03472	638	0.03472	638		
	0.031	1634.83	0.031	1740.8	0.031	1846.333	0.031	1951.6667	0.031	2056.9	0.03968	659.7	0.03968	659.7	0.03968	659.7	0.03968	659.7		
											0.04464	679.4	0.04464	679.4	0.04464	679.4	0.04464	679.4		
											0.0496	697.5	0.0496	697.5	0.0496	697.5	0.0496	697.5		
											0.05084	697.5	0.05084	697.5	0.05084	697.5	0.05084	697.5		

APPENDIX A

Borehole Records

PROJECT <u>19124560</u>		RECORD OF BOREHOLE No BH-101		2 OF 2	METRIC
W.P. <u>3025-18-00</u>	LOCATION <u>N 4753324.7 , E 408904.5 (Lat 42.912607 , Long -81.224971)</u>	ORIGINATED BY <u>MA</u>			
DIST <u></u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, 108mm ID HOLLOW STEM</u>	COMPILED BY <u>AMS</u>			
DATUM <u>GEODETIC</u>	DATE <u>October 10, 2019</u>	CHECKED BY <u></u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _p	W	W _L						○ UNCONFINED
	CLAYEY SILT (CL), trace sand very stiff Brown to grey at about elev. 253.1m Moist		14	SS	10		248														
242.01			18	SS	13																
21.79	END OF BOREHOLE Borehole dry upon completion of drilling																				

PROJECT		19124560	
W.P.		3025-18-00	
DIST		HWY 401	
DATUM		GEODETIC	
LOCATION		N 4753331.6 , E 408946.6 (Lat 42.912663 , Long -81.224455)	
BOREHOLE TYPE		POWER AUGER, 108mm ID HOLLOW STEM	
DATE		April 23 - 24, 2020	
ORIGINATED BY		MR	
COMPILED BY		ZJB	
CHECKED BY			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
258.07	GROUND SURFACE													
0.09	silty TOPSOIL Dark brown CLAYEY SILT (CL), trace sand, trace gravel (FILL) Stiff to firm Brown		1	SS	12									
			2	SS	8									
			3	SS	5									
			4	SS	6									
254.11	CLAYEY SILT (CL), to SILTY CLAY (CI), trace sand Stiff to very stiff Grey		5	SS	27									
3.96			6	SS	25									
			7	SS	13									
			8	SS	14									
			9	SS	11									
			10	SS	4									
			11	SS	PH									
			12	SS	9									

Continued Next Page

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

PROJECT 19124560		RECORD OF BOREHOLE No BH-102		2 OF 3	METRIC
W.P. 3025-18-00		LOCATION N 4753331.6 , E 408946.6 (Lat 42.912663 , Long -81.224455)		ORIGINATED BY MR	
DIST _____ HWY 401		BOREHOLE TYPE POWER AUGER, 108mm ID HOLLOW STEM		COMPILED BY ZJB	
DATUM GEODETIC		DATE April 23 - 24, 2020		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						w _p w w _L																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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 + ³, × ³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 19124560		RECORD OF BOREHOLE No BH-102		3 OF 3	METRIC
W.P. 3025-18-00		LOCATION N 4753331.6 , E 408946.6 (Lat 42.912663 , Long -81.224455)		ORIGINATED BY MR	
DIST _____ HWY 401		BOREHOLE TYPE POWER AUGER, 108mm ID HOLLOW STEM		COMPILED BY ZJB	
DATUM GEODETIC		DATE April 23 - 24, 2020		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W _p	W	W _L		

LDN_MTO_06 19124560-2001.GPJ LDN_MTO.GDT 09/07/20

PROJECT 19124560		RECORD OF BOREHOLE No BH-103		3 OF 3	METRIC
W.P. 3025-18-00		LOCATION N 4753306.5 , E 408985.3 (Lat 42.912432 , Long -81.223986)		ORIGINATED BY JK/AC	
DIST _____ HWY 401		BOREHOLE TYPE POWER AUGER, 108mm ID HOLLOW STEM, TRICONE MUD ROTARY		COMPILED BY AMS	
DATUM GEODETIC		DATE November 14, 2019		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×	LAB VANE	w _p	w		w _L			
	sandy CLAYEY SILT (CL), trace to some gravel (TILL) Hard Grey Moist																			
			23	SS	100/ 229mm															
			24	SS	100/ 127mm															
223.38 35.05	Rock fragments in tip of spoon END OF BOREHOLE		25	SS	100/ 12mm															
	Borehole dry upon completion of drilling																			
	VWP 1903700 Water level measured at 254.3m on November 20, 2019 Water level measured at 254.2m on December 13, 2019 Water level measured at 254.1m on February 7, 2020 Water level measured at 254.6m on May 19, 2020																			

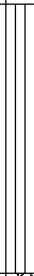
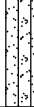
PROJECT 19124560		RECORD OF BOREHOLE No BH-105		1 OF 2	METRIC
W.P. 3025-18-00		LOCATION N 4753345.2 , E 408877.7 (Lat 42.912795 , Long -81.225296)		ORIGINATED BY MR	
DIST HWY 401		BOREHOLE TYPE POWER AUGER, 108mm ID HOLLOW STEM		COMPILED BY ZJB	
DATUM GEODETIC		DATE April 28 - 29, 2020		CHECKED BY	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE					
256.31	GROUND SURFACE													
0.00	silty TOPSOIL													
0.21	Black sandy SILT (ML), trace clay, with topsoil (FILL) Very loose Brown		1	SS	3									
254.94														
1.37	SILTY SAND (SM), some gravel, trace clay (FILL) Loose Brown		2	SS	7									
254.18														
2.13	CLAYEY SILT (CL), some sand, trace gravel Stiff to very stiff Grey		3	SS	13									
			4	SS	20									
			5	SS	25									
			6	SS	17									
			7	SS	14									
			8	SS	14									
249.30														
7.01	SILT (ML), some sand Compact Grey		9	SS	8									
			10	SS	PH									
			11	SS	5									
			12	SS	5									
			13	SS	PH									

Continued Next Page

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

PROJECT <u>19124560</u>		RECORD OF BOREHOLE No BH-105		2 OF 2	METRIC
W.P. <u>3025-18-00</u>		LOCATION <u>N 4753345.2 , E 408877.7 (Lat 42.912795 , Long -81.225296)</u>		ORIGINATED BY <u>MR</u>	
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, 108mm ID HOLLOW STEM</u>		COMPILED BY <u>ZJB</u>	
DATUM <u>GEODETIC</u>		DATE <u>April 28 - 29, 2020</u>		CHECKED BY <u> </u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					w _p — w — w _L								
							20	40	60	80	100										
	SILT (ML), some sand Compact Grey		14	SS	23									○							
			15	SS	16									○							
238.64																					
17.67	SILTY SAND (SM) Compact Grey																				
237.56			16	SS	27									○							
18.75	END OF BOREHOLE Groundwater encountered at about elev. 254.8m during drilling on April 28, 2020.																				

PROJECT 19124560		RECORD OF BOREHOLE No BH-106		2 OF 3	METRIC
W.P. 3025-18-00		LOCATION N 4753330.8 , E 408985.3 (Lat 42.912651 , Long -81.223981)		ORIGINATED BY MR	
DIST HWY 401		BOREHOLE TYPE POWER AUGER, 108mm ID HOLLOW STEM		COMPILED BY ZJB	
DATUM GEODETIC		DATE April 20 - 27, 2020		CHECKED BY	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
	CLAYEY SILT (CL) Very stiff Grey Moist		14	SS	9		243														
			15	SS	10		242														
241.23	sandy SILT (ML), some clay Compact to dense Grey Wet						241														
17.67		16	SS	30	240																
							239														
			17	SS	35		238														
							237														
			18	SS	18																
236.65																					
22.25	CLAYEY SILT (CL), some sand, trace gravel Very stiff to hard Grey Moist		19	SS	30		236														
								235													
				20	SS		31	234													
								233													
								232													
							231														
			21	SS	19																
								230													
								229													

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

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT 19124560		RECORD OF BOREHOLE No BH-107		1 OF 2	METRIC
W.P. 3025-18-00		LOCATION N 4753332.3 , E 409059.5 (Lat 42.912654 , Long -81.223073)		ORIGINATED BY MR	
DIST HWY 401		BOREHOLE TYPE POWER AUGER, 108mm ID HOLLOW STEM		COMPILED BY ZJB	
DATUM GEODETIC		DATE April 30 - May 1, 2020		CHECKED BY	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)					
257.50	GROUND SURFACE						20 40 60 80 100							
0.09	TOPSOIL, silty Black CLAYEY SILT (CL) trace to some sand Stiff to very stiff Brown to grey at about elev. 253.9m													
			1	SS	9									
			2	SS	20									
			3	SS	21									
			4	SS	23									
			5	SS	21									
			6	SS	17									
			7	SS	15									
			8	SS	12									
			9	SS	12									
			10	SS	5									

LDN_MTO_06 19124560-2001.GPJ LDN_MTO.GDT 09/07/20

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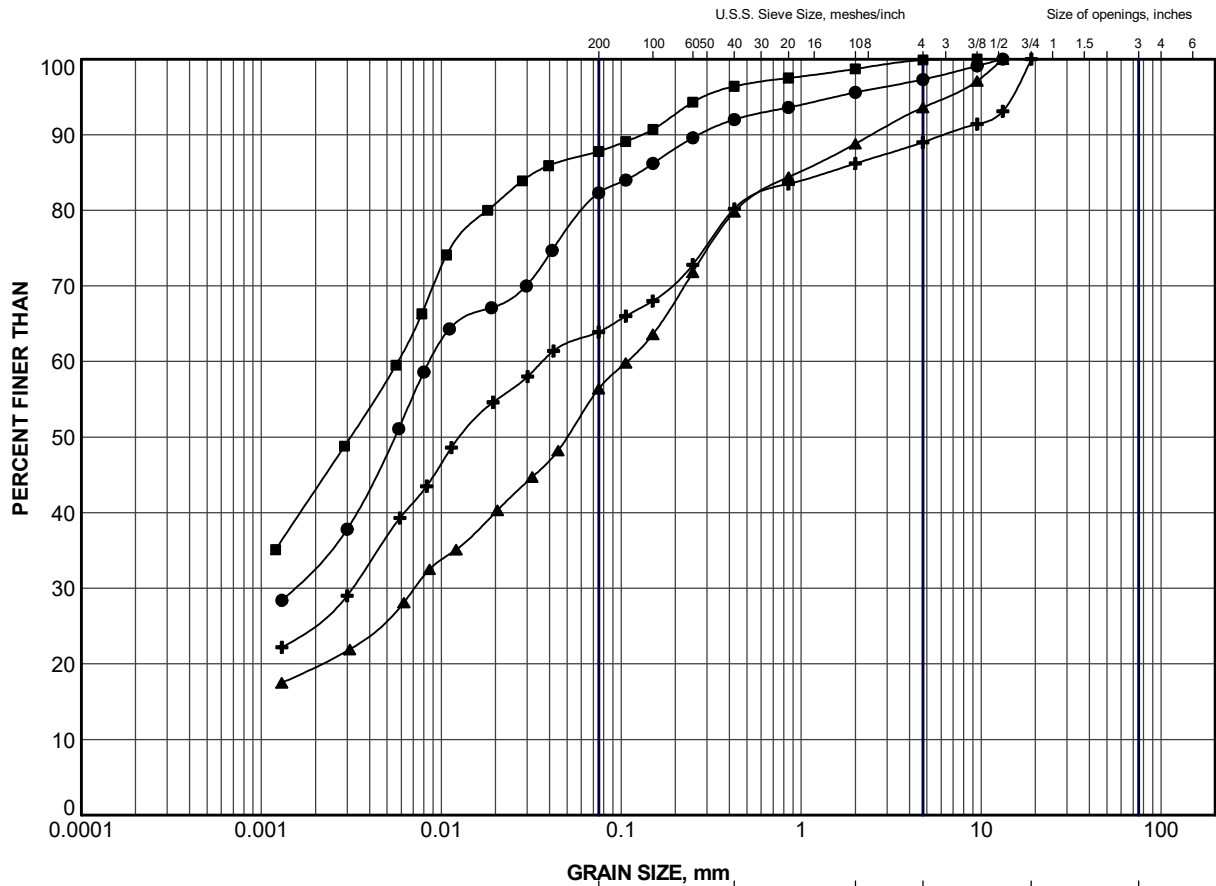
 +³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>19124560</u>		RECORD OF BOREHOLE No BH-107		2 OF 2	METRIC
W.P. <u>3025-18-00</u>	LOCATION <u>N 4753332.3 , E 409059.5 (Lat 42.912654 , Long -81.223073)</u>	ORIGINATED BY <u>MR</u>			
DIST <u></u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, 108mm ID HOLLOW STEM</u>	COMPILED BY <u>ZJB</u>			
DATUM <u>GEODETIC</u>	DATE <u>April 30 - May 1, 2020</u>	CHECKED BY <u></u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W _p	W	W _L		
241.19	CLAYEY SILT (CL) trace to some sand Stiff to very stiff Brown to grey at about elev. 253.9m		14	SS	9												
16.31	SILT (ML), some sand Very dense Grey		15	SS	56												
239.82	CLAYEY SILT (CL), trace sand Stiff Grey		16	SS	10												
238.45	sandy CLAYEY SILT (CL), trace gravel (TILL) Very stiff Grey		17	SS	26												
237.23	END OF BOREHOLE																
20.27	Groundwater encountered at about 252.8m and 246.1m during drilling on April 30, 2020.																

APPENDIX B

Geotechnical Laboratory Test Results

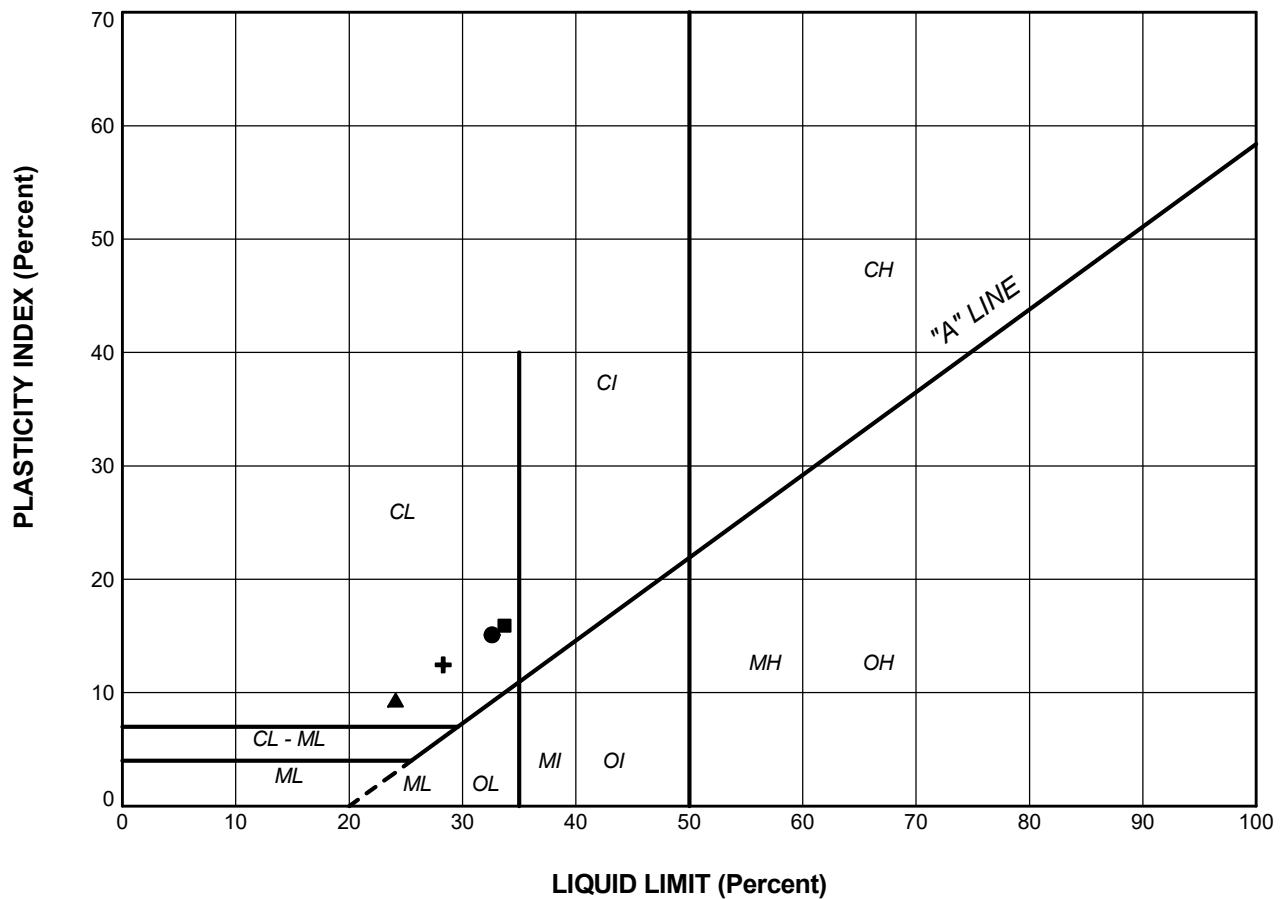


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	2	262.1
■	BH-101	8	257.5
▲	BH-104	1	263.6
+	BH-104	5	260.5

PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00			
TITLE		GRAIN SIZE DISTRIBUTION CLAYEY SILT (CL) - (FILL) to sandy CLAYEY SILT (CL) - (FILL)			
		PROJECT No.		19124560	
		FILE No.		19124560-2001-R010B1	
		SCALE		N/A	
		REV.			
DRAWN		ZJB		May 27/20	
CHECK					
		FIGURE B-1			

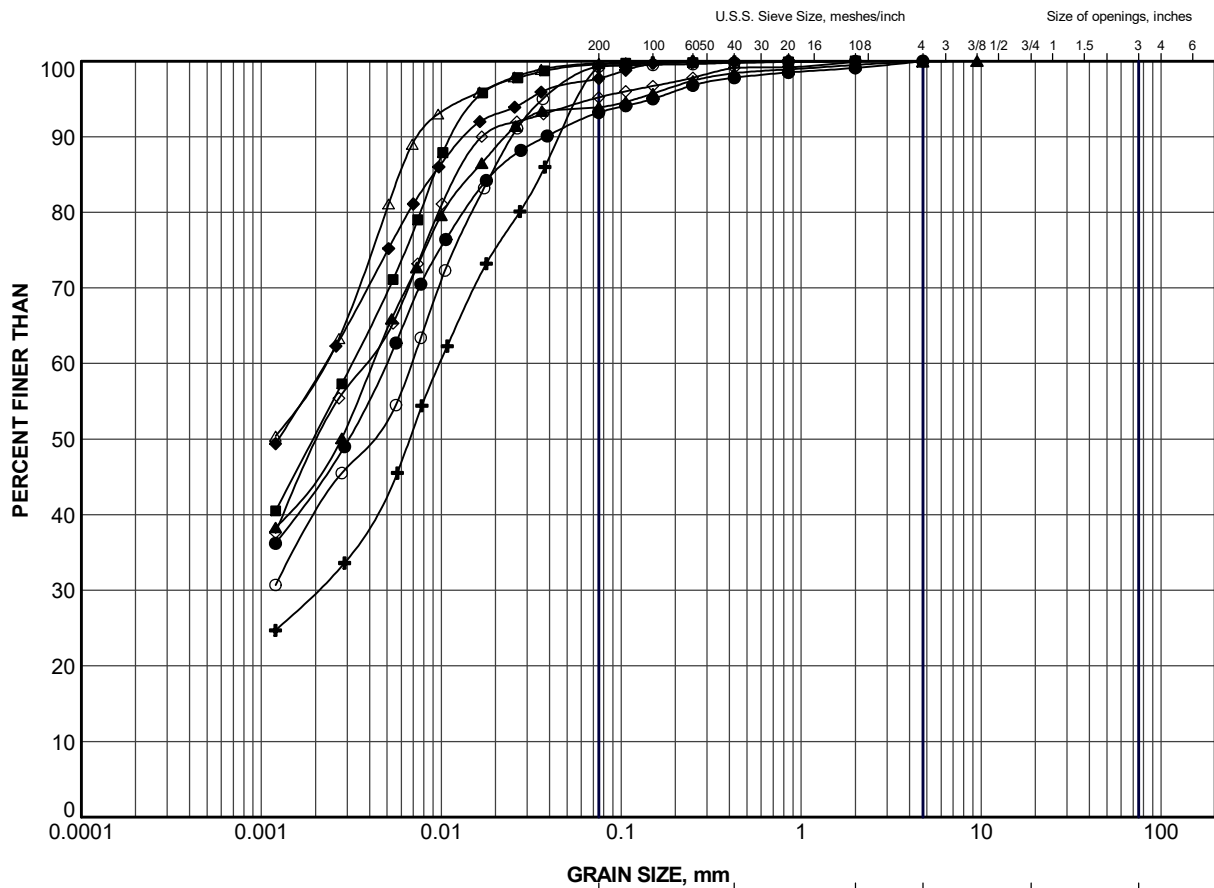


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-101	2	32.6	17.5	15.1
■	BH-101	8	33.7	17.8	15.9
▲	BH-104	1	24.1	14.8	9.4
+	BH-104	5	28.3	15.9	12.5

PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00	
TITLE		PLASTICITY CHART CLAYEY SILT (CL) - (FILL) to sandy CLAYEY SILT (CL) - (FILL)	
PROJECT No. 19124560		FILE No. 19124560-2001-R010B2	
DRAWN	ZJB	May 27/20	SCALE N/A REV.
CHECK			FIGURE B-2

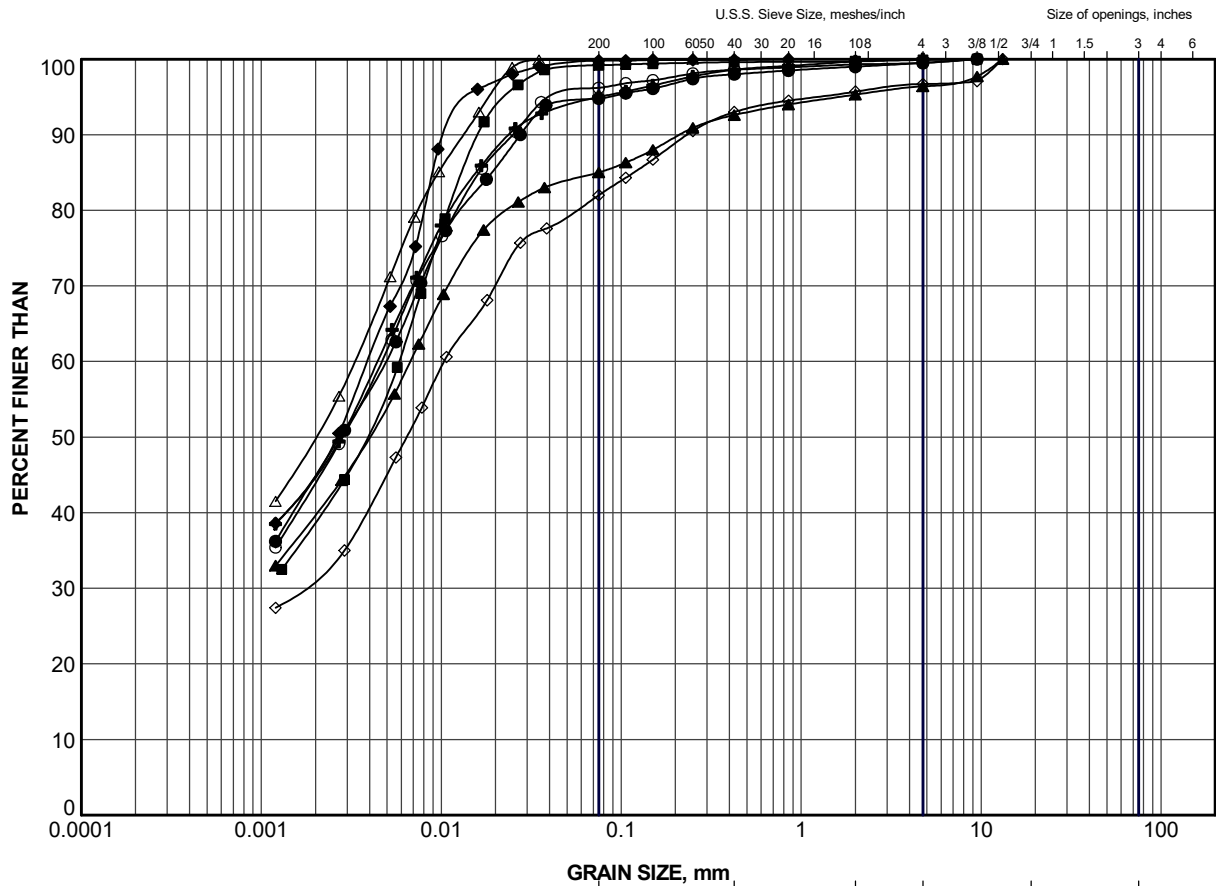




LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	11	252.9
■	BH-101	16	245.3
▲	BH-102	5	254.0
+	BH-102	10	247.2
◆	BH-102	13	242.6
◇	BH-103	6	254.4
○	BH-103	11	249.1
△	BH-103	15	243.0

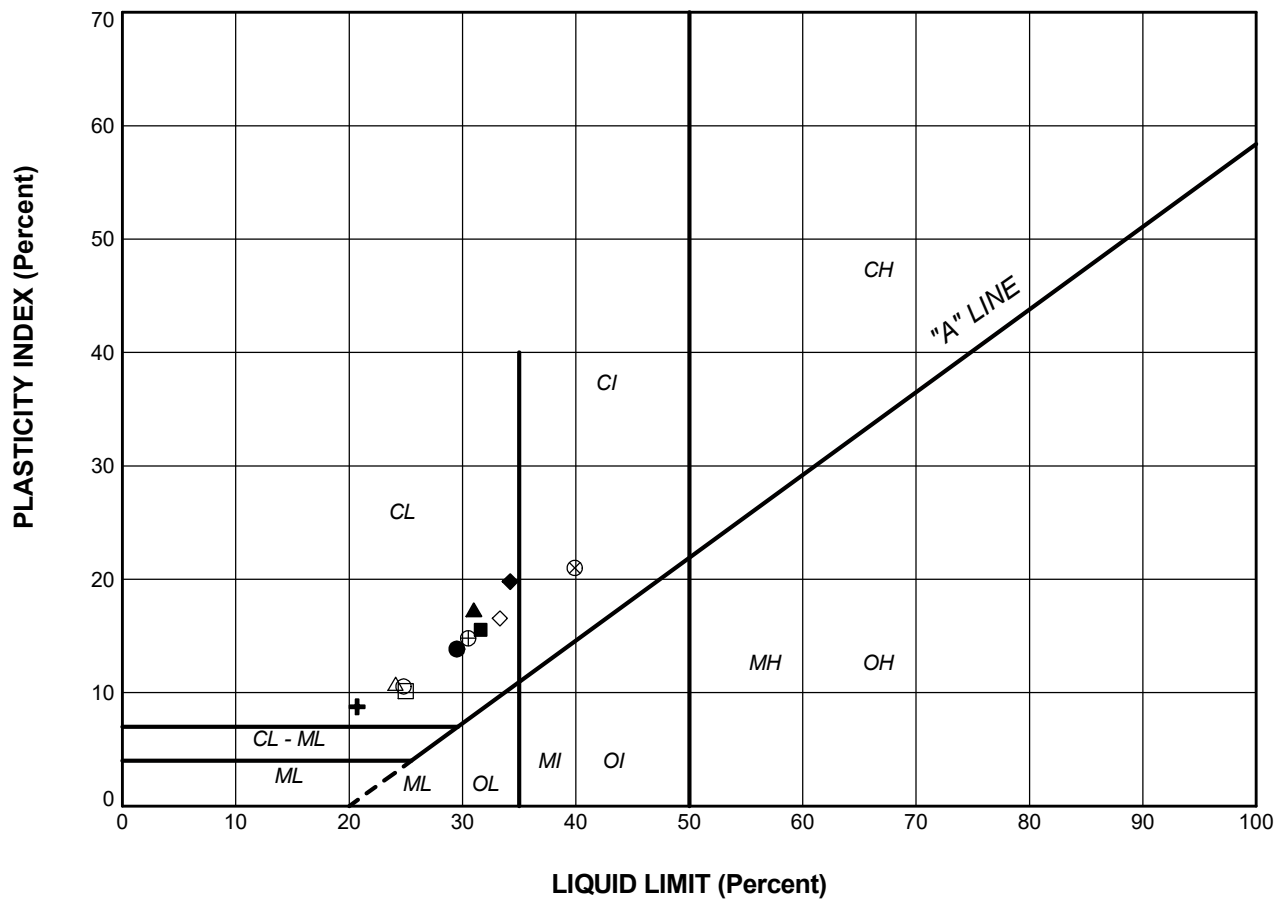
PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00	
TITLE		GRAIN SIZE DISTRIBUTION CLAYEY SILT (CL) to SILTY CLAY (CI)	
PROJECT No.		19124560	FILE No. 19124560-2001-R010B3.1
DRAWN		ZJB	May 27/20
CHECK			
GOLDER		SCALE N/A REV.	
		FIGURE B-3.1	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-104	12	252.1
■	BH-104	16	246.0
▲	BH-105	4	253.0
+	BH-106	6	254.1
◆	BH-106	13	245.0
◇	BH-106	19	235.8
○	BH-107	3	255.0
△	BH-107	10	248.1

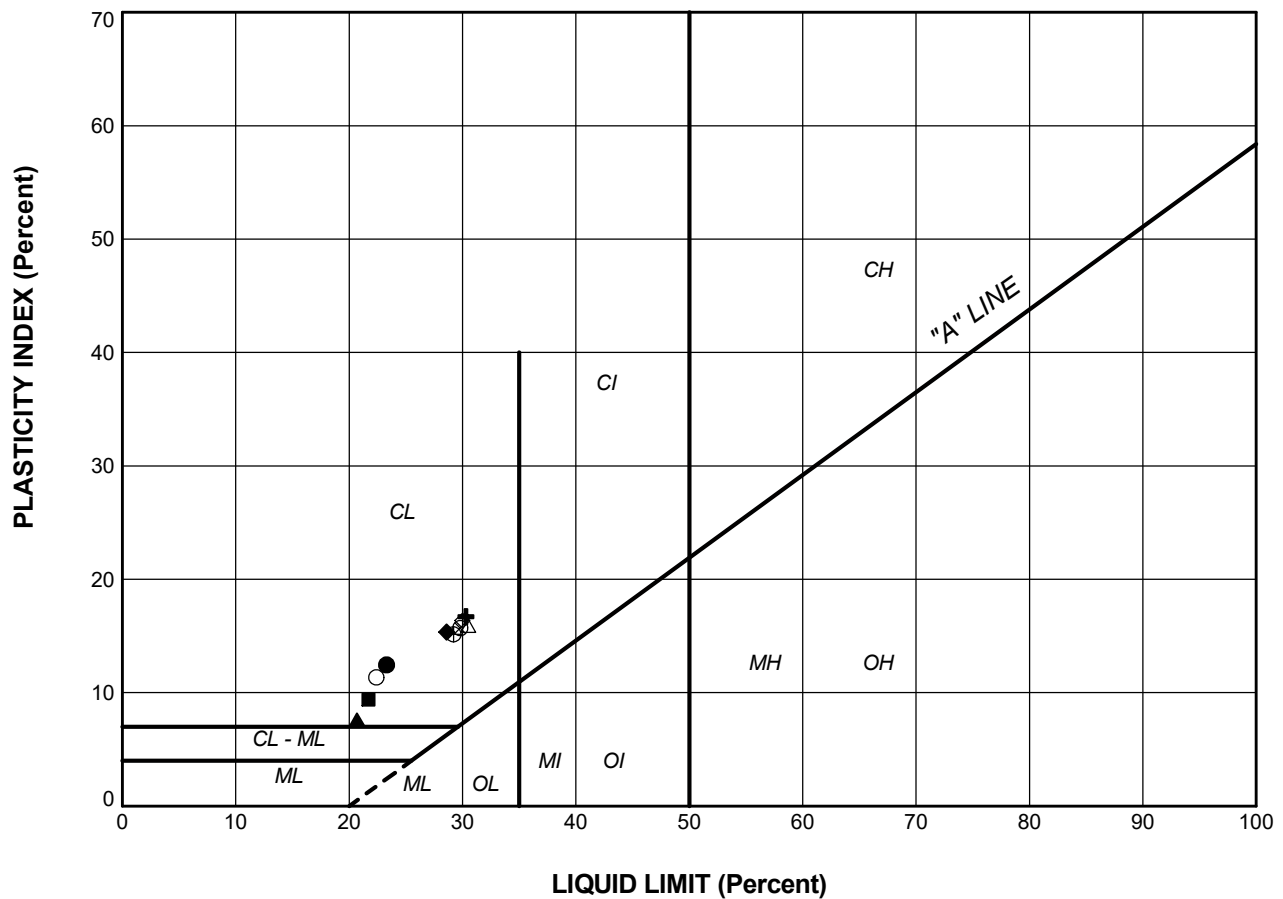
PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00	
TITLE		GRAIN SIZE DISTRIBUTION CLAYEY SILT (CL) to SILTY CLAY (CI)	
PROJECT No.		19124560	FILE No. 19124560-2001-R010B3.2
DRAWN		ZJB	May 27/20
CHECK			
GOLDER		SCALE N/A REV.	
		FIGURE B-3.2	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-101	11	29.5	15.7	13.9
■	BH-101	16	31.6	16.1	15.6
▲	BH-102	5	31.0	13.7	17.3
+	BH-102	10	20.7	12.0	8.8
◆	BH-102	13	34.2	14.4	19.8
◇	BH-103	6	33.3	16.8	16.6
○	BH-103	11	24.8	14.3	10.6
△	BH-103	12	24.1	13.3	10.8
⊗	BH-103	15	39.9	18.9	21.0
⊕	BH-104	12	30.5	15.7	14.8
□	BH-104	16	25.0	14.9	10.2

PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00		
TITLE		PLASTICITY CHART CLAYEY SILT (CL) to SILTY CLAY (CI)		
PROJECT No.		19124560		FILE No. 19124560-2001-R010B4.1
DRAWN	ZJB	May 27/20		SCALE N/A REV.
CHECK				FIGURE B-4.1



SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

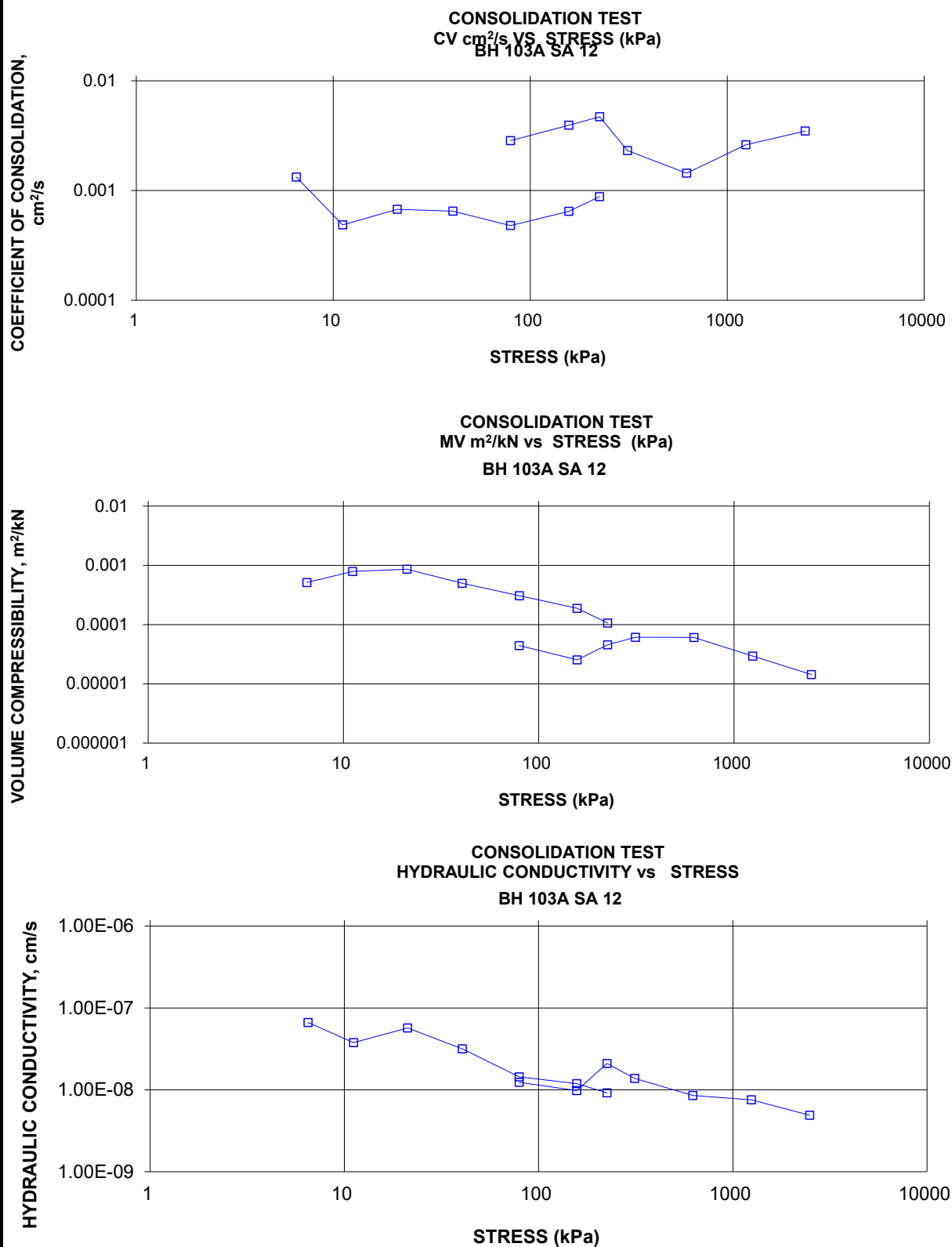
LEGEND

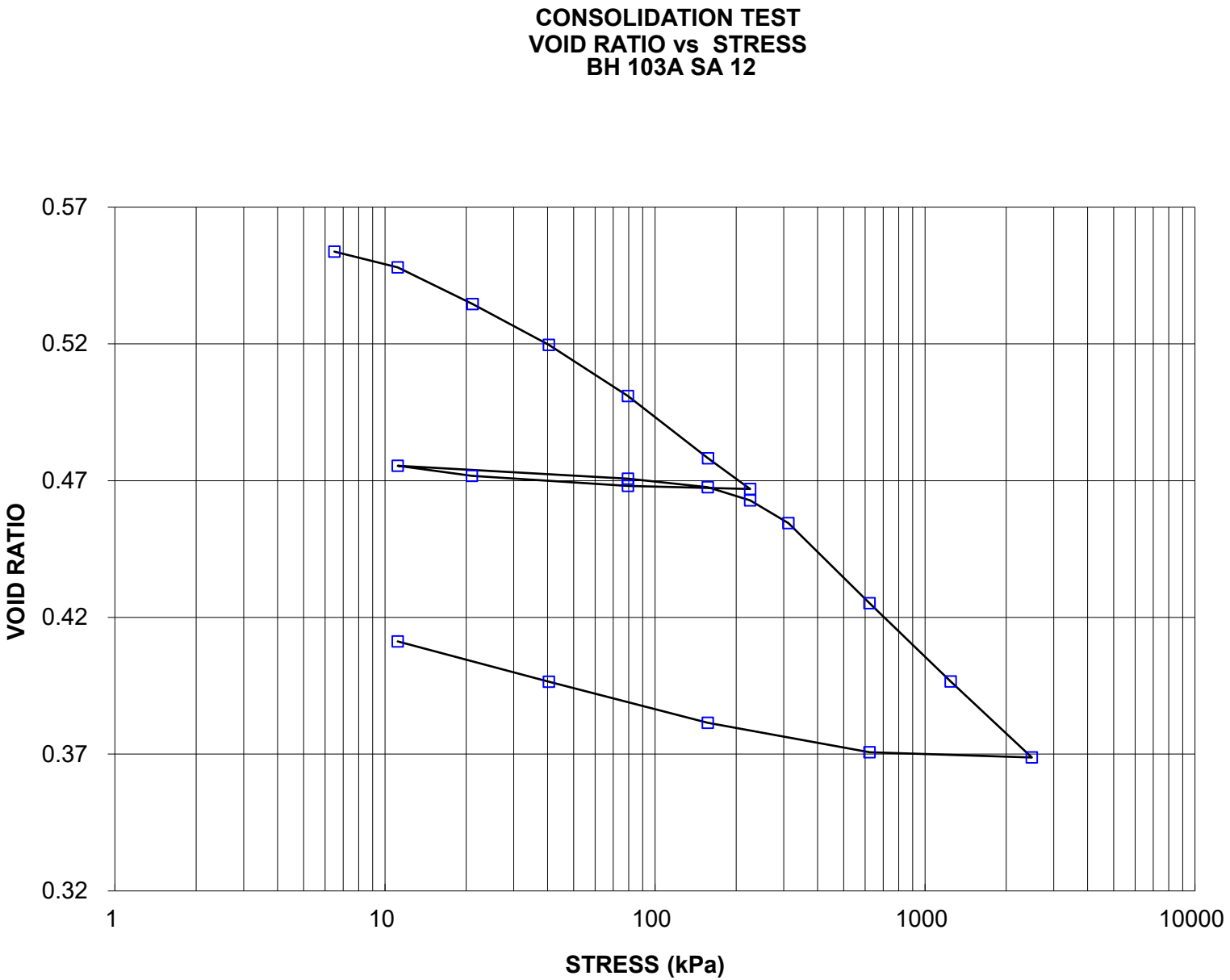
SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-105	4	23.3	10.9	12.5
■	BH-105	9	21.7	12.3	9.4
▲	BH-105	11	20.7	13.1	7.6
+	BH-106	6	30.3	13.6	16.7
◆	BH-106	10	28.6	13.3	15.4
◇	BH-106	13	30.0	13.7	16.4
○	BH-106	19	22.4	11.1	11.4
△	BH-107	3	30.5	14.6	15.9
⊗	BH-107	6	29.8	14.1	15.7
⊕	BH-107	10	29.2	14.1	15.2

PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00	
TITLE		PLASTICITY CHART CLAYEY SILT (CL) to SILTY CLAY (CI)	
PROJECT No. 19124560		FILE No. 19124560-2001-R010B4.2	
DRAWN	ZJB	July 6/20	SCALE N/A REV.
CHECK			FIGURE B-4.2



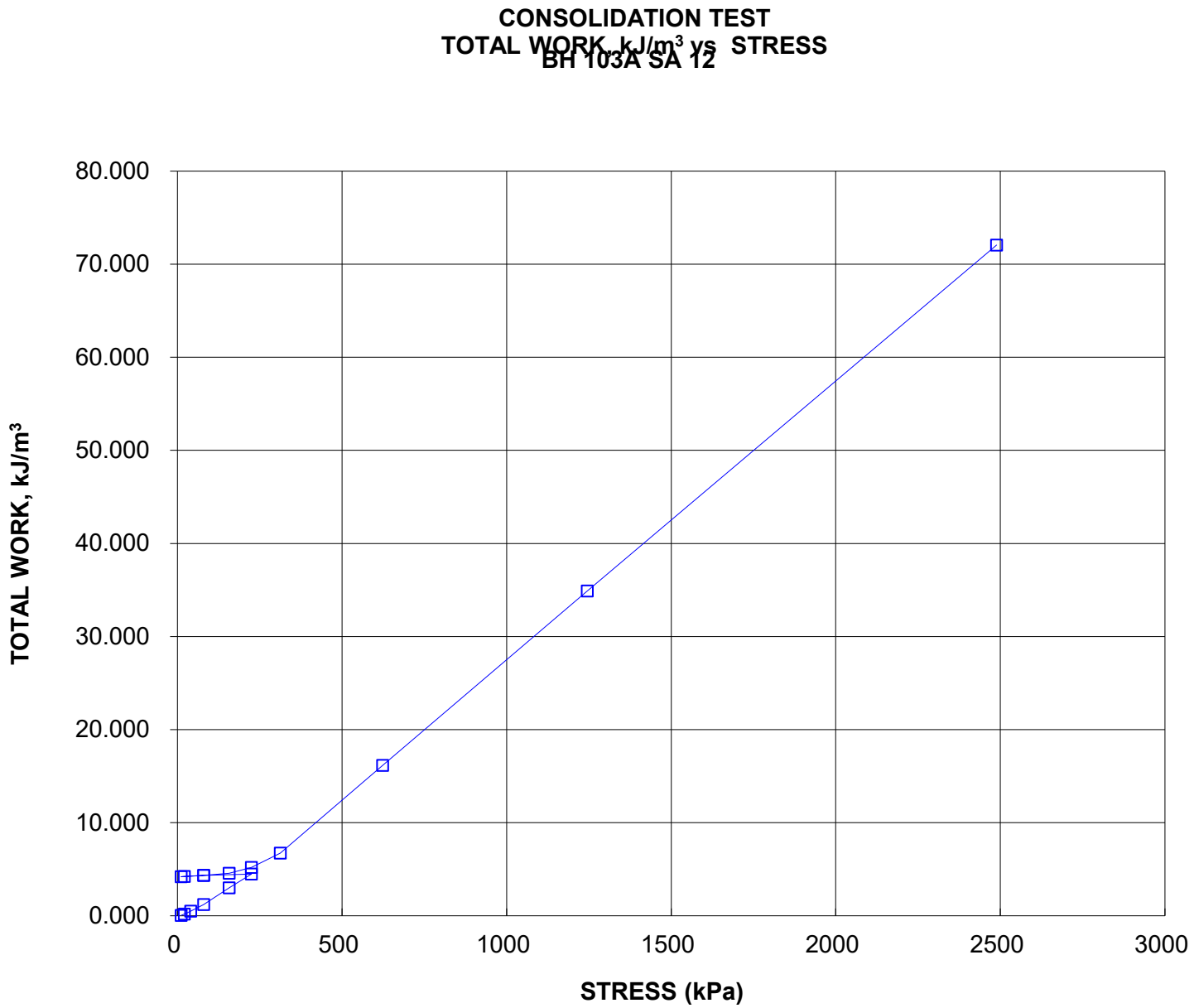
CONSOLIDATION TEST SUMMARY					FIGURE B-5.1		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	19124560			Sample Number	12		
Borehole Number	103A			Sample Depth, m	10.67-11.28		
TEST CONDITIONS							
Test Type	Laboratory Standard			Load Duration, hr	24		
Oedometer Number	9						
Date Started	02/12/2020						
Date Completed	03/01/2020						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.90			Unit Weight, kN/m ³	20.74		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m ³	17.30		
Area, cm ²	31.52			Specific Gravity, measured	2.75		
Volume, cm ³	59.92			Solids Height, cm	1.219		
Water Content, %	19.91			Volume of Solids, cm ³	38.44		
Wet Mass, g	126.74			Volume of Voids, cm ³	21.48		
Dry Mass, g	105.7			Degree of Saturation, %	97.9		
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.901	0.559	1.901				
6.50	1.895	0.554	1.898	577	1.32E-03	5.12E-04	6.64E-08
11.15	1.888	0.548	1.891	1561	4.86E-04	7.92E-04	3.77E-08
21.14	1.871	0.535	1.880	1109	6.75E-04	8.59E-04	5.69E-08
40.47	1.853	0.520	1.862	1135	6.48E-04	4.98E-04	3.16E-08
79.44	1.830	0.501	1.842	1500	4.79E-04	3.07E-04	1.44E-08
157.14	1.803	0.478	1.816	1084	6.45E-04	1.88E-04	1.19E-08
224.98	1.789	0.467	1.796	778	8.79E-04	1.07E-04	9.19E-09
79.40	1.790	0.468	1.789				
21.00	1.795	0.472	1.792				
11.15	1.799	0.475	1.797				
79.40	1.793	0.471	1.796	240	2.85E-03	4.42E-05	1.24E-08
157.01	1.790	0.468	1.792	173	3.93E-03	2.54E-05	9.80E-09
225.00	1.784	0.463	1.787	144	4.70E-03	4.55E-05	2.10E-08
312.12	1.774	0.455	1.779	290	2.31E-03	6.09E-05	1.38E-08
622.97	1.738	0.425	1.756	454	1.44E-03	6.05E-05	8.54E-09
1244.65	1.703	0.397	1.720	240	2.61E-03	2.95E-05	7.56E-09
2488.51	1.669	0.369	1.686	173	3.48E-03	1.43E-05	4.90E-09
622.97	1.672	0.371	1.670				
157.01	1.685	0.381	1.678				
40.47	1.703	0.396	1.694				
11.15	1.721	0.411	1.712				
Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t ₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 0-7cm from bottom of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.72			Unit Weight, kN/m ³	22.01		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m ³	19.11		
Area, cm ²	31.52			Specific Gravity, measured	2.75		
Volume, cm ³	54.24			Solids Height, cm	1.219		
Water Content, %	15.18			Volume of Solids, cm ³	38.44		
Wet Mass, g	121.74			Volume of Voids, cm ³	15.80		
Dry Mass, g	105.7						
Prepared By: LH				Golder Associates		Checked By: MWK	



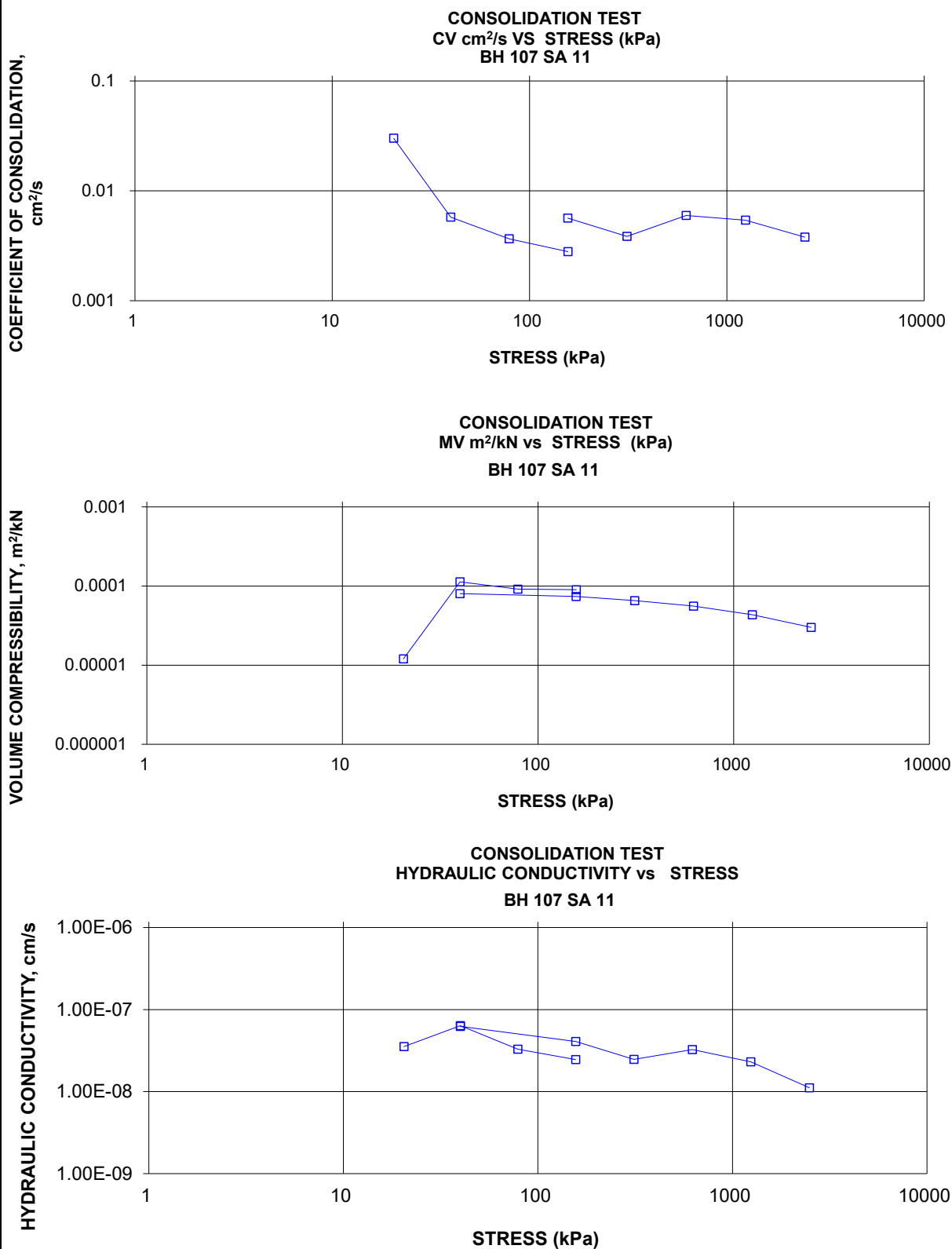


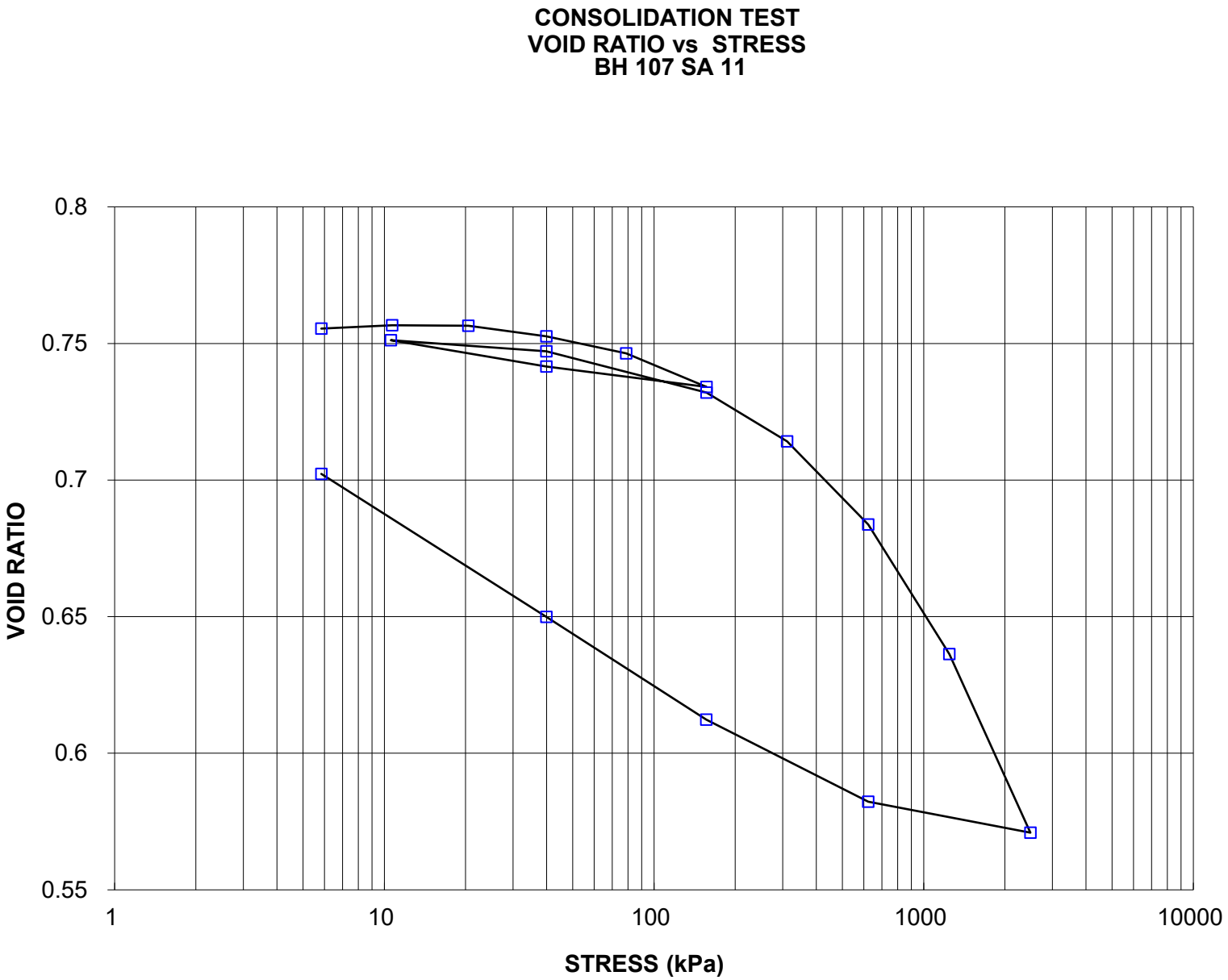
CONSOLIDATION TEST
TOTAL WORK VS STRESS

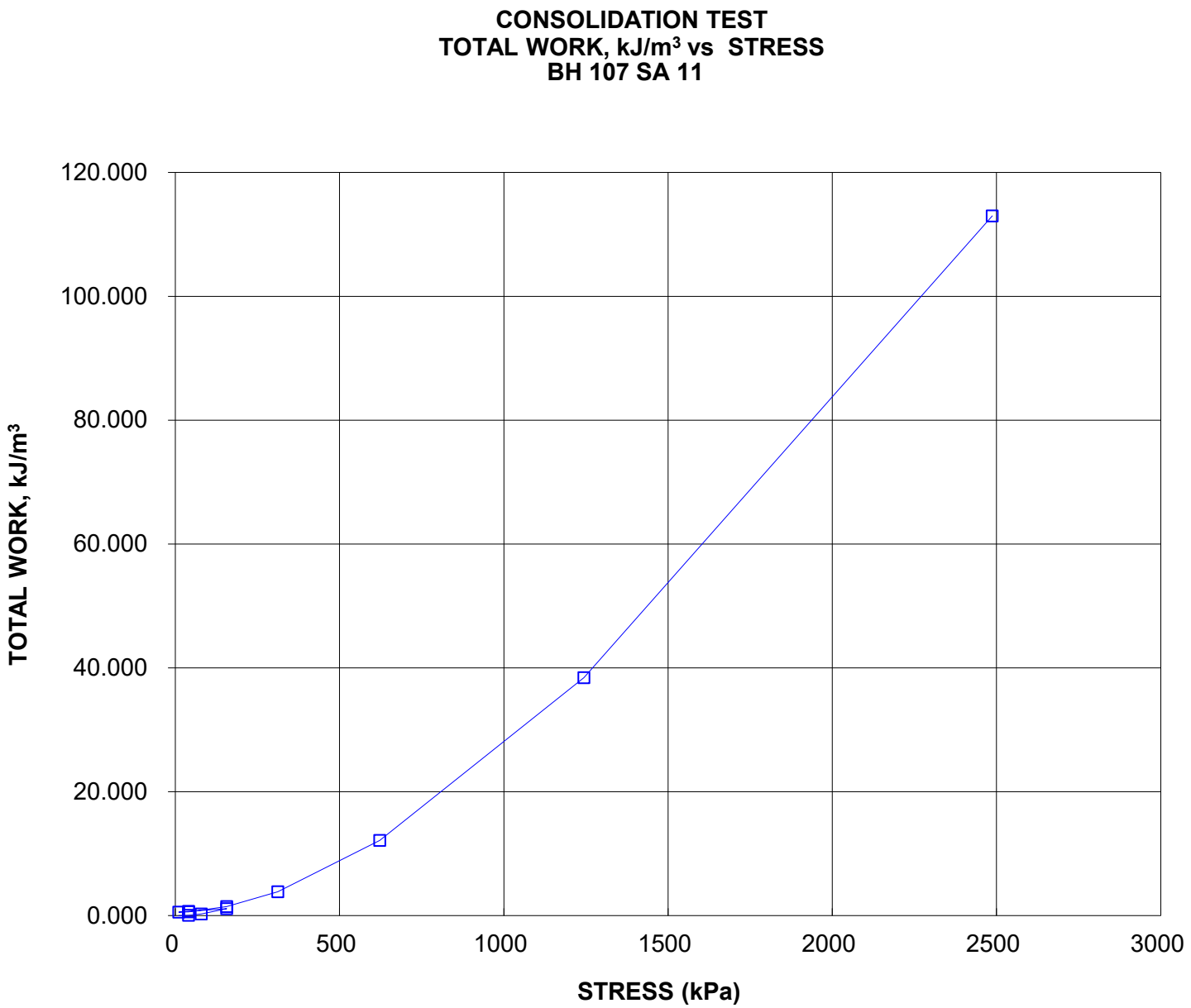
FIGURE B-5.4

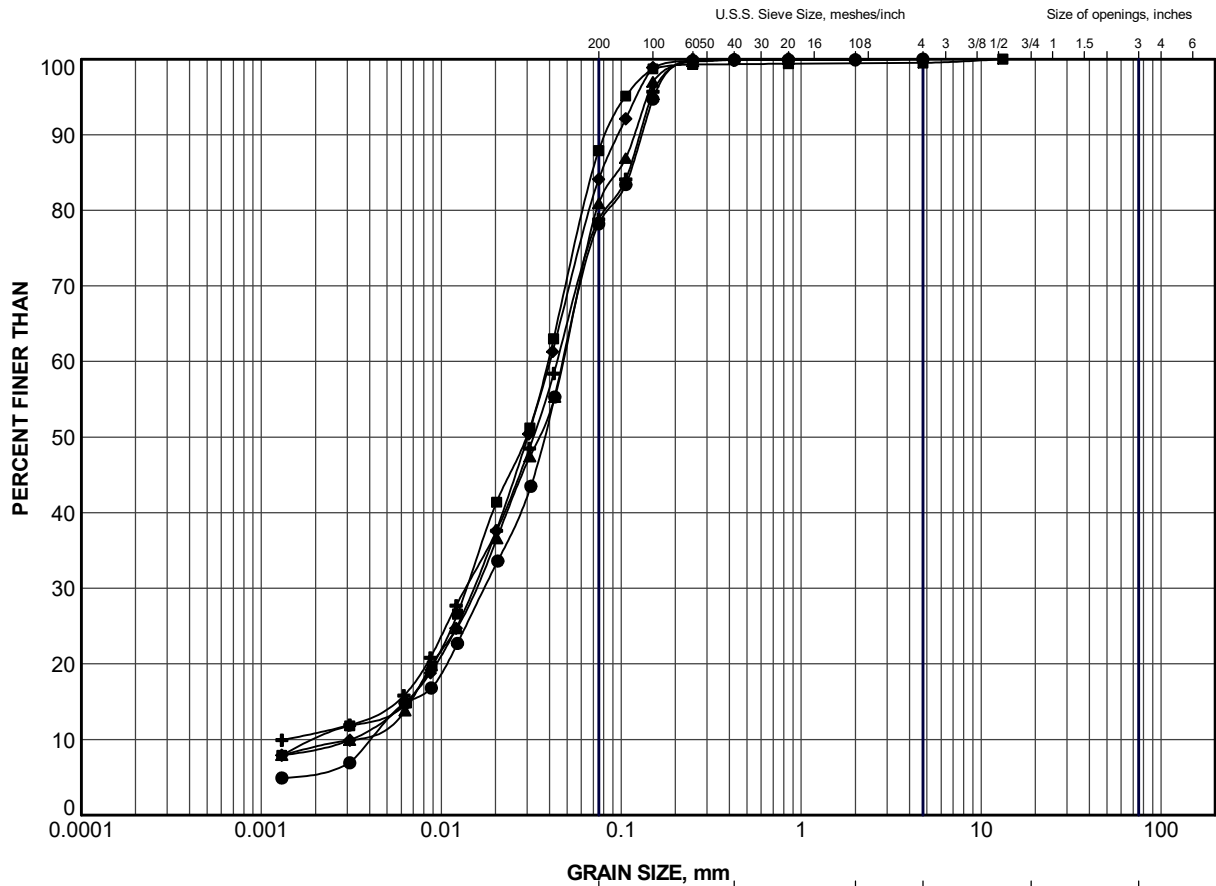


CONSOLIDATION TEST SUMMARY					FIGURE B-6.1			
ASTM D2435/D2435M								
SAMPLE IDENTIFICATION								
Project Number		19124560(2001)			Sample Number		11	
Borehole Number		107			Sample Depth, m		10.67-11.13	
TEST CONDITIONS								
Test Type		Laboratory Standard			Load Duration, hr		24	
Oedometer Number		12						
Date Started		05/05/2020						
Date Completed		05/19/2020						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		2.55			Unit Weight, kN/m ³		19.66	
Sample Diameter, cm		6.34			Dry Unit Weight, kN/m ³		15.47	
Area, cm ²		31.53			Specific Gravity, measured		2.77	
Volume, cm ³		80.46			Solids Height, cm		1.454	
Water Content, %		27.05			Volume of Solids, cm ³		45.84	
Wet Mass, g		161.31			Volume of Voids, cm ³		34.63	
Dry Mass, g		126.97			Degree of Saturation, %		99.2	
TEST COMPUTATIONS								
	Stress	Corr. Height	Void Ratio	Average Height	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
	kPa	cm		cm				
	0.00	2.552	0.755	2.552				
	5.84	2.552	0.755	2.552				
	10.70	2.554	0.757	2.553				
	20.49	2.554	0.756	2.554	46	3.01E-02	1.20E-05	3.54E-08
	39.92	2.548	0.753	2.551	240	5.75E-03	1.13E-04	6.36E-08
	78.88	2.539	0.746	2.543	375	3.66E-03	9.15E-05	3.28E-08
	156.61	2.521	0.734	2.530	485	2.80E-03	8.97E-05	2.46E-08
	39.92	2.532	0.742	2.526				
	10.57	2.546	0.751	2.539				
	39.92	2.540	0.747	2.543	173	7.92E-03	8.01E-05	6.22E-08
	156.43	2.518	0.732	2.529	240	5.65E-03	7.37E-05	4.08E-08
	311.71	2.492	0.714	2.505	346	3.84E-03	6.56E-05	2.47E-08
	622.19	2.448	0.684	2.470	217	5.96E-03	5.58E-05	3.26E-08
	1243.90	2.379	0.636	2.413	228	5.42E-03	4.34E-05	2.30E-08
	2486.98	2.284	0.571	2.331	304	3.79E-03	3.00E-05	1.11E-08
	622.19	2.300	0.582	2.292				
	156.39	2.344	0.612	2.322				
	39.92	2.399	0.650	2.371				
	5.84	2.475	0.702	2.437				
<p>Note:</p> <p>Consolidation loading and unloading schedule assigned by the client.</p> <p>cv and k are approximate only based on t₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M)</p> <p>Specimen taken 41-46cm from top of the tube.</p> <p>Specimen swelled under 10.7kPa.</p>								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		2.47			Unit Weight, kN/m ³		20.02	
Sample Diameter, cm		6.34			Dry Unit Weight, kN/m ³		15.96	
Area, cm ²		31.53			Specific Gravity, measured		2.77	
Volume, cm ³		78.03			Solids Height, cm		1.454	
Water Content, %		25.42			Volume of Solids, cm ³		45.84	
Wet Mass, g		159.25			Volume of Voids, cm ³		32.19	
Dry Mass, g		126.97						
Prepared By: LH					Golder Associates		Checked By: MWK	







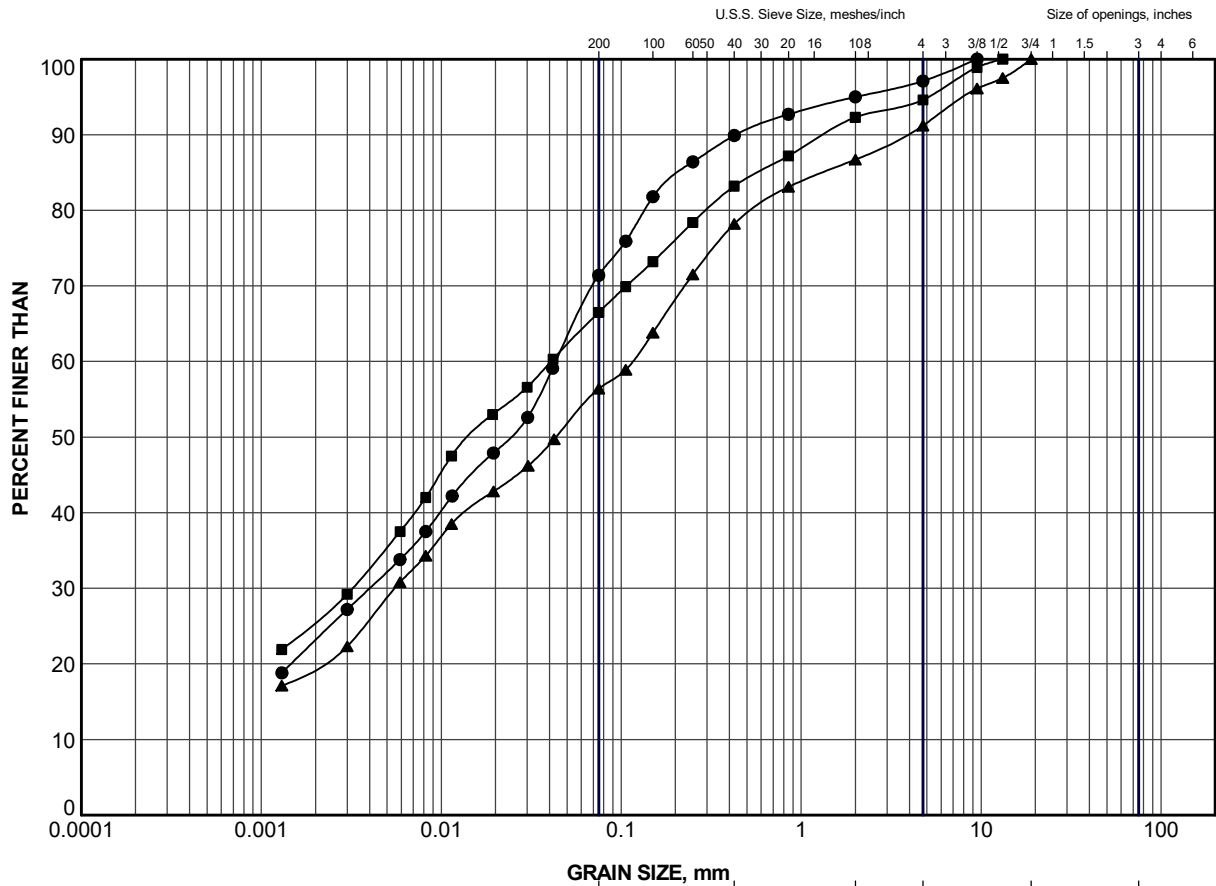


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-102	16	238.0
■	BH-103	18	238.4
▲	BH-105	15	239.3
+	BH-106	17	238.9
◆	BH-107	15	240.5

PROJECT		DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00			
TITLE		GRAIN SIZE DISTRIBUTION SILT (ML) to sandy SILT (ML) Interlayer			
		PROJECT No.	19124560	FILE No.	19124560-2001-R010B7
		SCALE	N/A	REV.	
		DRAWN	ZJB	July 9/20	
		CHECK			
		FIGURE B-7			

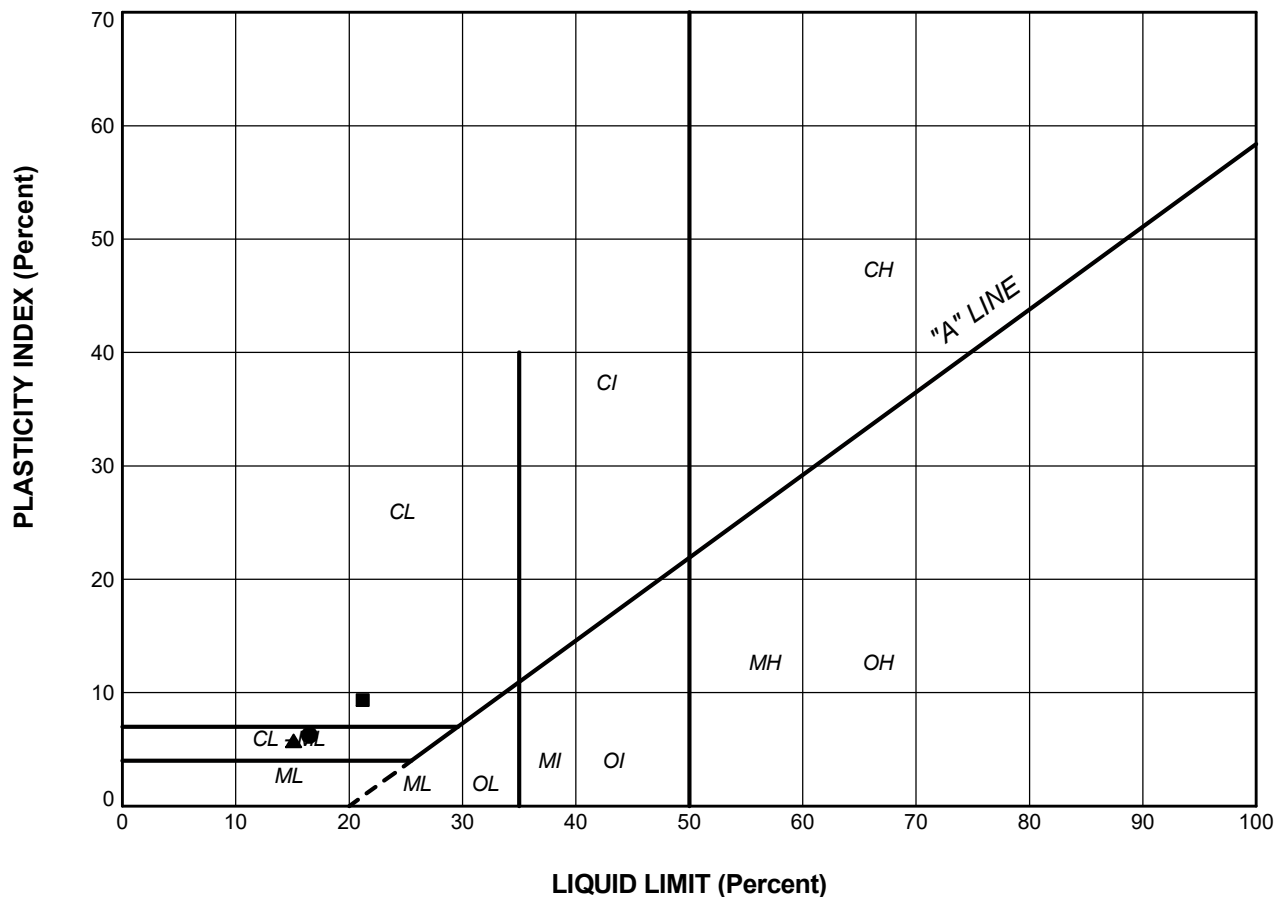


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-102	21	227.4
■	BH-103	23	226.2
▲	BH-106	23	226.7

PROJECT				DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00			
TITLE				GRAIN SIZE DISTRIBUTION sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL/ML) TILL			
PROJECT No.		19124560		FILE No.		19124560-2001-R010B8	
DRAWN		ZJB		SCALE		N/A	
CHECK				REV.			
GOLDER		July 9/20		FIGURE B-8			

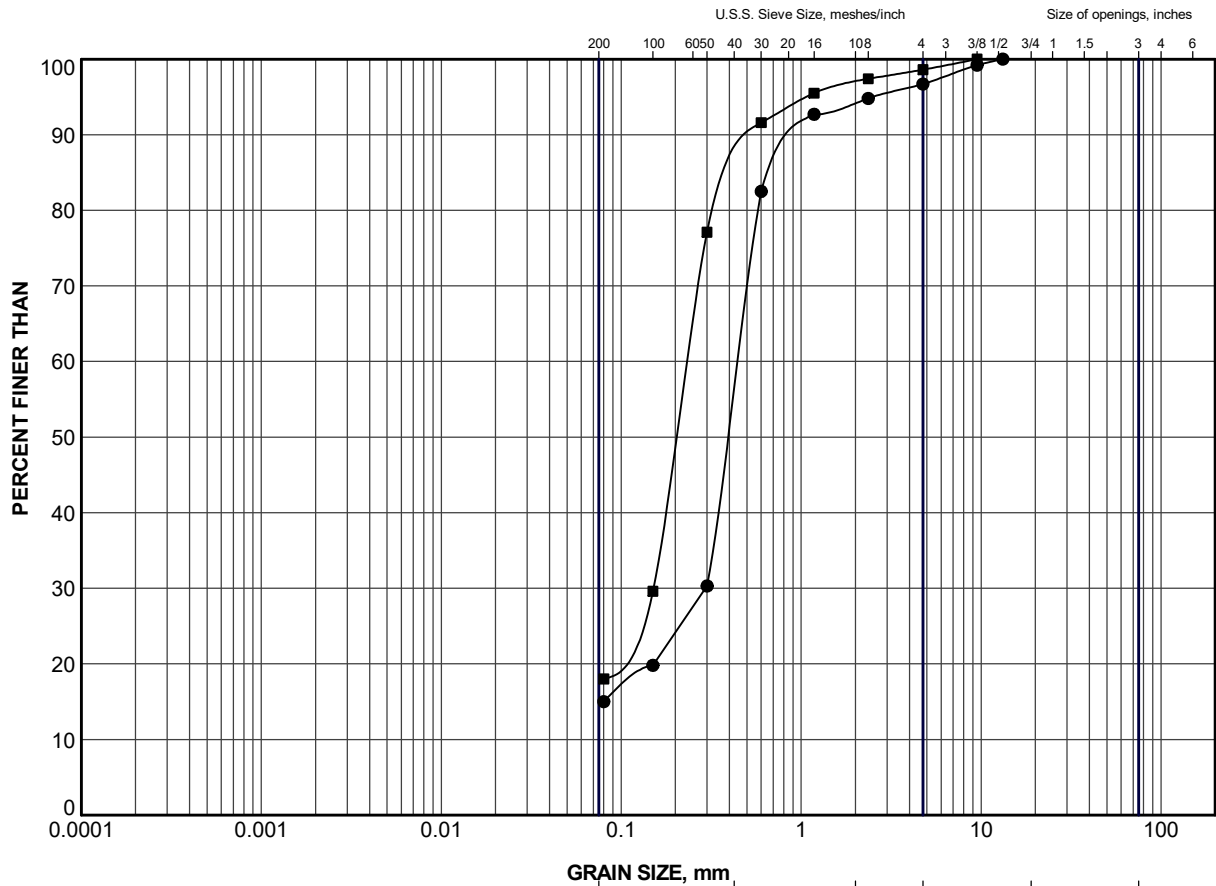


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-102	21	16.5	10.3	6.3
■	BH-103	23	21.2	11.9	9.4
▲	BH-106	23	15.1	9.4	5.8

PROJECT				DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00			
TITLE				PLASTICITY CHART sandy CLAYEY SILT (CL) to CLAYEY SILT (CL/ML) TILL			
PROJECT No.		19124560		FILE No.		19124560-2001-R010B9	
DRAWN	ZJB	July 9/20		SCALE	N/A		REV.
CHECK				FIGURE B-9			





CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-102	24	219.6
■	BH-106	28	219.1

PROJECT				DINGMAN DRIVE UNDERPASS HIGHWAY 401 GWP 3015-18-00			
TITLE				GRAIN SIZE DISTRIBUTION Lower SILTY SAND (SM)			
PROJECT No.		19124560		FILE No.		19124560-2001-R010B10	
DRAWN		ZJB		SCALE		N/A	
CHECK				REV.			
GOLDER		July 9/20		FIGURE B-10			

APPENDIX C

Analytical Testing Laboratory Testing

CLIENT NAME: GOLDER ASSOCIATES LTD.
309 EXETER ROAD, UNIT #1
LONDON, ON N6L1C1
(519) 652-0099

ATTENTION TO: Adam Core

PROJECT: Highway 401

AGAT WORK ORDER: 19L548019

SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer

DATE REPORTED: Dec 04, 2019

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 19L548019

PROJECT: Highway 401

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

SAMPLING SITE: Dingman Drive

ATTENTION TO: Adam Core

SAMPLED BY: J. Kiss

Corrosivity Package

DATE RECEIVED: 2019-11-25

DATE REPORTED: 2019-12-04

SAMPLE DESCRIPTION: BH 103A, Sa#5

SAMPLE TYPE: Soil

DATE SAMPLED: 2019-11-14

Parameter	Unit	G / S	RDL	747046
Chloride (2:1)	µg/g		2	364
Sulphate (2:1)	µg/g		2	8
pH (2:1)	pH Units		NA	8.15
Electrical Conductivity (2:1)	mS/cm		0.005	0.840
Resistivity (2:1) (Calculated)	ohm.cm		1	1190
Redox Potential 1	mV		NA	391
Redox Potential 2	mV		NA	387
Redox Potential 3	mV		NA	389

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

747046 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter. Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:

Divine Basily



Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: Highway 401

SAMPLING SITE: Dingman Drive

AGAT WORK ORDER: 19L548019

ATTENTION TO: Adam Core

SAMPLED BY: J. Kiss

Soil Analysis

RPT Date: Dec 04, 2019			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper

Corrosivity Package

Chloride (2:1)	747440		<2	<2	NA	< 2	91%	80%	120%	102%	80%	120%	102%	70%	130%
Sulphate (2:1)	747440		33	33	0.0%	< 2	99%	80%	120%	103%	80%	120%	105%	70%	130%
pH (2:1)	747419		7.74	7.67	0.9%	NA	101%	90%	110%						
Electrical Conductivity (2:1)	747419		0.277	0.280	1.1%	< 0.005	100%	90%	110%						
Redox Potential 1	1					NA	100%	90%	110%						

Comments: NA signifies Not Applicable.

Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL

pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Certified By: _____

Divine Basily

Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 19L548019

PROJECT: Highway 401

ATTENTION TO: Adam Core

SAMPLING SITE: Dingman Drive

SAMPLED BY: J. Kiss

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B, SSA #5 Part 3	CALCULATION
Redox Potential 1	INOR-93-6066	G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 2	INOR-93-6066	G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 3	INOR-93-6066	G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE

CLIENT NAME: GOLDER ASSOCIATES LTD.
309 EXETER ROAD, UNIT #1
LONDON, ON N6L1C1
(519) 652-0099

ATTENTION TO: Adam Core

PROJECT: 19L548019

AGAT WORK ORDER: 19T550614

SOLID ANALYSIS REVIEWED BY: Sherin Moussa, Senior Technician

DATE REPORTED: Dec 04, 2019

PAGES (INCLUDING COVER): 5

Should you require any information regarding this analysis please contact your client services representative at (905) 501-9998

*NOTES



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 19T550614

PROJECT: 19L548019

5623 McADAM ROAD
MISSISSAUGA, ONTARIO
CANADA L4Z 1N9
TEL (905)501-9998
FAX (905)501-0589
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Adam Core

(201-042) Sulfide

DATE SAMPLED: Dec 01, 2019

DATE RECEIVED: Dec 02, 2019

DATE REPORTED: Dec 04, 2019

SAMPLE TYPE: Other

Analyte: Sulfide

Unit: %

Sample ID (AGAT ID) RDL: 0.05

BH 103A, Sa#5 (762238) <0.05

Comments: RDL - Reported Detection Limit

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:

Sherin Moosaj



AGAT Laboratories

Quality Assurance - Replicate

AGAT WORK ORDER: 19T550614

PROJECT: 19L548019

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FAX (905)501-0589
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Adam Core

(201-042) Sulfide

REPLICATE #1															
Parameter	Sample ID	Original	Replicate	RPD											
S	762238	0.006	0.005	18.2%											
Sulfate	762238	< 0.01	<0.01	0.0%											
Sulfide	762238	< 0.05	<0.05	0.0%											



AGAT Laboratories

Quality Assurance - Certified Reference materials

AGAT WORK ORDER: 19T550614

PROJECT: 19L548019

5623 McADAM ROAD
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FAX (905)501-0589
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Adam Core

(201-042) Sulfide

CRM #1															
Parameter	Expect	Actual	Recovery	Limits											
S	0.8	0.81	101%	90% - 110%											
Sulfate	0.01	0.01	100%	90% - 110%											
Sulfide	0.8	0.8	100%	90% - 110%											



Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 19T550614

PROJECT: 19L548019

ATTENTION TO: Adam Core

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Solid Analysis Sulfide	MIN-200-12037		LECO

CLIENT NAME: GOLDER ASSOCIATES LTD.
309 EXETER ROAD, UNIT #1
LONDON, ON N6L1C1
(519) 652-0099

ATTENTION TO: ADAM CORE

PROJECT: 19124560

AGAT WORK ORDER: 20L596388

SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer

DATE REPORTED: May 05, 2020

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*Notes

Disclaimer:

- All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.
- All samples will be disposed of within 30 days following analysis, unless expressly agreed otherwise in writing. Please contact your Client Project Manager if you require additional sample storage time.
- AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.
- This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
- The test results reported herewith relate only to the samples as received by the laboratory.
- Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.
- All reportable information as specified by ISO/IEC 17025:2017 is available from AGAT Laboratories upon request.



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 20L596388

PROJECT: 19124560

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

SAMPLING SITE: 401/Dingman

ATTENTION TO: ADAM CORE

SAMPLED BY: M. Rhody

Corrosivity Package

DATE RECEIVED: 2020-04-27

DATE REPORTED: 2020-05-05

		SAMPLE DESCRIPTION:		BH102 Sa5	BH106 Sa6
		SAMPLE TYPE:		Soil	Soil
		DATE SAMPLED:		2020-04-23	2020-04-23
Parameter	Unit	G / S	RDL	1097413	1097415
Chloride (2:1)	µg/g		2	164	130
Sulphate (2:1)	µg/g		2	43	54
pH (2:1)	pH Units		NA	7.85	7.98
Electrical Conductivity (2:1)	mS/cm		0.005	0.488	0.452
Resistivity (2:1) (Calculated)	ohm.cm		1	2050	2210
Redox Potential 1	mV		NA	95	98
Redox Potential 2	mV		NA	106	84
Redox Potential 3	mV		NA	122	79

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

1097413-1097415 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter. Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:

Divine Basily



Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 19124560

SAMPLING SITE: 401/Dingman

AGAT WORK ORDER: 20L596388

ATTENTION TO: ADAM CORE

SAMPLED BY: M. Rhody

Soil Analysis

RPT Date: May 05, 2020			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper

Corrosivity Package

Chloride (2:1)	1097413	1097413	164	163	0.6%	< 2	99%	70%	130%	106%	80%	120%	108%	70%	130%
Sulphate (2:1)	1097413	1097413	43	42	2.4%	< 2	98%	70%	130%	102%	80%	120%	100%	70%	130%
pH (2:1)	1097413	1097413	7.85	7.92	0.9%	NA	100%	90%	110%						
Electrical Conductivity (2:1)	1097413	1097413	0.488	0.498	2.0%	< 0.005	110%	80%	120%						
Redox Potential 1	1					NA	101%	90%	110%						

Comments: NA signifies Not Applicable.

pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Certified By: _____

Divine Basily



Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 20L596388

PROJECT: 19124560

ATTENTION TO: ADAM CORE

SAMPLING SITE:401/Dingman

SAMPLED BY:M. Rhody

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	modified from MSA PART 3, CH 14 and SM 2510 B	EC METER
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION
Redox Potential 1	INOR-93-6066	modified G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 2	INOR-93-6066	modified G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 3	INOR-93-6066	modified G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE

CLIENT NAME: GOLDER ASSOCIATES LTD.
309 EXETER ROAD, UNIT #1
LONDON, ON N6L1C1
(519) 652-0099

ATTENTION TO: Adam Core

PROJECT: 20L596388

AGAT WORK ORDER: 20T597407

SOLID ANALYSIS REVIEWED BY: Sherin Moussa, Senior Technician

DATE REPORTED: May 04, 2020

PAGES (INCLUDING COVER): 5

Should you require any information regarding this analysis please contact your client services representative at (905) 501-9998

*NOTES



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 20T597407

PROJECT: 20L596388

5623 McADAM ROAD
MISSISSAUGA, ONTARIO
CANADA L4Z 1N9
TEL (905)501-9998
FAX (905)501-0589
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Adam Core

(201-042) Sulfide

DATE SAMPLED: Apr 30, 2020

DATE RECEIVED: May 01, 2020

DATE REPORTED: May 04, 2020

SAMPLE TYPE: Other

Analyte: Sulfide

Unit: %

Sample ID (AGAT ID) RDL: 0.05

BH102 Sa5-1097413 (1102874) 0.13

BH102 Sa5-1097413-DUP (1102875) 0.13

BH106 Sa6-1097415 (1102876) 0.13

Comments: RDL - Reported Detection Limit

Analysis performed at AGAT 5623 McAdam Rd., Mississauga, ON (unless marked by *)

Certified By:

Sherin Moosaj



AGAT Laboratories

Quality Assurance - Replicate

AGAT WORK ORDER: 20T597407

PROJECT: 20L596388

5623 McADAM ROAD
MISSISSAUGA, ONTARIO
CANADA L4Z 1N9
TEL (905)501-9998
FAX (905)501-0589
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Adam Core

(201-042) Sulfide

Parameter	REPLICATE #1				REPLICATE #2				REPLICATE #3							
	Sample ID	Original	Replicate	RPD	Sample ID	Original	Replicate	RPD	Sample ID	Original	Replicate	RPD				
S	1102874	0.134	0.123	8.6%	1102875	0.131	0.127	3.1%	1102876	0.132	0.131	0.8%				
Sulfate	1102874	< 0.01	< 0.01	0.0%	1102875	< 0.01	< 0.01	0.0%	1102876	< 0.01	< 0.01	0.0%				
Sulfide	1102874	0.13	0.12	8.0%	1102875	0.13	0.13	0.0%	1102876	0.13	0.13	0.0%				



AGAT Laboratories

Quality Assurance - Certified Reference materials

AGAT WORK ORDER: 20T597407

PROJECT: 20L596388

5623 McADAM ROAD
MISSISSAUGA, ONTARIO
CANADA L4Z 1N9
TEL (905)501-9998
FAX (905)501-0589
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Adam Core

(201-042) Sulfide

Parameter	CRM #1				CRM #2				CRM #3							
	Expect	Actual	Recovery	Limits	Expect	Actual	Recovery	Limits	Expect	Actual	Recovery	Limits				
S	0.80	0.81	101%	90% - 110%	0.80	0.80	100%	90% - 110%	0.80	0.81	101%	90% - 110%				
Sulfate	0.01	0.01	100%	90% - 110%	0.01	0.01	100%	90% - 110%	0.01	0.01	100%	90% - 110%				
Sulfide	0.80	0.80	100%	90% - 110%	0.80	0.79	98%	90% - 110%	0.80	0.80	100%	90% - 110%				

Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 20T597407

PROJECT: 20L596388

ATTENTION TO: Adam Core

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Solid Analysis Sulfide	MIN-200-12037		LECO

APPENDIX D

Non-Standard Special Provisions

WORKING SLAB - Item No.

Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab, including mass concrete on the prepared subgrade under foundations where necessary for the Highway 401/Dingman Drive underpass replacement.

2.0 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 Definitions - Not Used

4.0 Design and Submission Requirements - Not Used

5.0 Materials

Concrete for working slabs shall have a minimum 28-day compressive strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement for Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

AMENDMENT TO OPSS 903, APRIL 2016

Special Provision No. 109F57

January 2020

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International**:

D 4945-12 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the deletion of the definitions for Certificate of Conformance and Quality Verification Engineer.

Section 903.03 of OPSS 903 is further amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the quality of deep foundations and/or performance of the drive system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a tested pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02.04.02.01 Milestone Inspections

Clause 903.04.02.04.02.01 of OPSS 903 is deleted in its entirety.

903.04.02.05 Qualifications

Clause 903.04.02.05 of OPSS 903 is deleted in its entirety

903.04.02.06 Review of Splice Test Results and Permission to Proceed

Clause 903.04.02.06 of OPSS 903 is deleted in its entirety.

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.07 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Contractor personnel; however, testing shall be performed under the direction of an Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator and Foundation Engineering Specialist on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (Case Pile Wave Analysis Program - CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

- a) Pile ultimate resistance and integrity.
- b) Calculated driving stresses.
- c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator and Foundation Engineering Specialist within 10 Days of the field testing. The final report shall include the following:

- a) Results of pile ultimate resistance and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.
- b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.
- c) An appendix shall be included containing the following documents:
 - i. Pile installation record
 - ii. Reference subsurface information (borehole record)
 - iii. Pile location drawing
 - iv. Initial calibration check by the test computer unit
 - v. Test set up geometry

The report shall be signed and sealed by two Engineers of the independent testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

903.07 CONSTRUCTION

903.07.02.03.03 H-Piles, Tube Piles, and Sheet Piles

Clause 903.07.02.03.03 of OPSS 903 is amended by deleting the second paragraph in its entirety and replacing it with the following:

Steel H-piles and steel tube piles may be spliced providing that the pieces being spliced are not less than 3 m long, except for piles at integral abutments, the pieces being spliced shall not be less than 7 m long. Splices in piles located into a watercourse shall only be introduced under the low water level, unless the piles are encased in concrete.

903.07.02.07.01 General

Clause 903.07.02.07.01 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile under the direction of the Contractor. Pile driving records shall be submitted to the Contract Administrator.

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the ultimate resistance shall be assessed by the Contractor's independent testing company using the Hiley Dynamic Formula on all piles, and Pile Dynamic Analyzer (PDA) testing on a minimum of 10% of piles or two piles per foundation element (whichever is greater), in each stage, at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved at the end of initial driving, each such pile shall be retapped no less than 48 hours after completion of initial driving, in accordance with Clause 903.07.02.07.06.

The Hiley Dynamic Formula and PDA test results shall be provided to the Contract Administrator and Foundation Engineering Specialist in accordance with Clause 903.04.02.07. A Request to Proceed shall be submitted to the Contract Administrator after the design ultimate resistance is achieved.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.07.02.07.03.03 Driving to Bedrock

Clause 903.07.02.07.03.03 of OPSS 903 is amended by deleting the last sentence in its entirety.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 of OPSS 903 is deleted in its entirety and replaced with the following:

903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing

903.07.02.07.04 .01 Wave Equation Analysis

Prior to mobilizing piling equipment to the site, a Wave Equation Analysis of Piles (WEAP) analysis shall be performed by the Contractor to demonstrate the potential for the proposed piling equipment to activate the specified ultimate resistance specified in the Contract Documents.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

903.07.02.07.04.02 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed by the Contractor to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by an Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing and shall be submitted to the Contract Administrator for information purposes.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group, rounded up, but no fewer than two piles.

Additional high-strain dynamic testing shall be carried out during the retapping of piles as set out in Section 903.07.02.07.06. Such high strain dynamic testing during retapping shall apply to a minimum of 10% of piles in each pile group, rounded up, but no fewer than two piles.

903.07.02.07.06 Retapping Tests on Piles

Section 903.07.02.06 is deleted in its entirety and replaced by the following:

In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles, shall be retapped to confirm that the ultimate axial geotechnical resistance has been achieved and/or sustained. This shall include all piles that have not met the specified ultimate geotechnical resistance on completion of initial driving.

Retapping shall be carried out no sooner than 48 hours after installation of the individual pile. If the hammer needs to be warmed up prior to performing a retap, it shall not be warmed up by striking the intended test pile(s).

903.07.03.07 Concrete

903.07.03.07.01 General

Clause 903.07.03.07.01 of OPSS 903 is deleted in its entirety and replaced with the following:

A Request to Proceed shall be submitted to the Contract Administrator before the concrete placement.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

The placement of concrete shall not proceed until the Contract Administrator has inspected the caisson hole and issued to the Contractor a Notice to Proceed.

Concrete shall be placed immediately after the Notice to Proceed has been received and shall be placed in the caisson according to OPSS 904 and as specified herein.

Arching of concrete during casing withdrawal shall be prevented.

903.07.03.07.05 Founding Elevation

Clause 903.07.03.07.05 of OPSS 903 is amended by deleting the last paragraph in its entirety and replacing it with the following:

Complete access to inspect the bearing area of the caisson pile prior to the placement of concrete shall be given to the Contract Administrator.

903.07.06 Load Test

Subsection 903.07.06 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

When a load test is specified in the Contract Documents, the testing shall be according to ASTM D 1143M for piles under vertical static load, ASTM D 3689 for piles under tensile load, and ASTM D 3966 for piles under lateral loads. The Contract Administrator shall witness the pile load test. All records and results of the pile load test shall be submitted to the Contract Administrator.

903.07.08 Quality Control

903.07.08.01.01 Qualifications of Companies and Individuals

Clause 903.07.08.01.01 of OPSS 903 is deleted in its entirety

903.07.08.01.02 Visual Inspection of Welds

Clause 903.07.08.01.02 of OPSS 903 is deleted in its entirety and replaced with the following:

Complete access to visually inspect the welds shall be given to the Contract Administrator.

All welds shall conform with the requirements of CSA W59 and the Contract Documents. A representative sample of splice welds, not less than 30% of the welds will be selected by the Contract Administrator for visual inspection. The selected splice welds shall be taken from different piles.

If the sample welds do not pass the visual inspection and need to be repaired, the visual inspection by the Contract Administrator may be increased up to 100% of the welds.

903.07.08.01.03 Non-Destructive Testing of Welds

Clause 903.07.08.01.03 of OPSS 903 is deleted in its entirety and replaced with the following:

The Contract Administrator shall be notified in writing, 48 hours in advance of installing piles which will require weld splicing. The Contract Administrator shall be immediately notified in writing, if there are any schedule changes for each pile requiring weld splicing.

A Request to Proceed shall be submitted to the Contract Administrator after the completion of splice welds for each construction stage of work.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

Radiographic or ultrasonic testing shall be carried out by the Contract Administrator using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contract Administrator.

The welds selected for the random ultrasonic or radiographic testing shall be taken from different piles and shall include 10% of the splice welds, rounded to the next highest number, but no fewer than two.

If any welds do not pass the ultrasonic or radiographic-testing and need to be repaired, these non-destructive testing requirements may be increased up to 100% of the welds.

903.07.08.01.04 Repaired Welds

Clause 903.07.08.01.04 of OPSS 903 is deleted in its entirety and replaced with the following:

All welds that have been repaired shall be visually inspected and shall undergo non-destructive testing performed by the Contract Administrator

903.07.08.02 Visual Inspection Reports and Non-Destructive Test Reports

Clause 903.07.08.02 of OPSS 903 is deleted in its entirety and replaced with the following:

Results from completed Visual Inspection Reports and Non-Destructive Test Reports will be provided upon request

903.07.08.03 Certificate of Conformance

Clause 903.07.08.03 of OPSS 903 is deleted in its entirety.

903.07.08.04 Displacement Caisson Piles

Clause 903.07.08.04 of OPSS 903 is amended by deleting the fourth paragraph in its entirety and replacing it with the following:

A Request to Proceed shall be submitted to the Contract Administrator before the installation of displacement caisson piles.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.10 BASIS FOR PAYMENT

903.10.01 Supply Equipment for Installing Driven Piles - Item Supply Equipment for Installing Caisson Piles - Item Supply Equipment for Installing Displacement Caisson Piles - Item

Subsection 903.10.01 of OPSS 903 is amended by deleting the second paragraph in its entirety and replacing it with the following:

For payment purposes, 50% of the work under this item shall be paid when the satisfactory performance of the equipment has been demonstrated to the Contract Administrator by the installation of 1% of piles.

Another 40% shall be paid by progress payments proportional to the work completed. The remaining 10% shall be paid on the satisfactory completion of the installation of piles.

903.10.04 Failed Visual Inspection or Non-Destructive Testing of Welds

Section 903.10 of OPSS 903 is amended by the addition of the following:

Costs associated with any required removals and replacement or repairs of defective welds, following the visual inspection or non-destructive testing, shall be the Contractor's responsibility at no additional cost to the Owner. No additional payment will be made for labour and equipment provided by the Contractor, and the Contractor will pay the Owner \$500 for each weld requiring additional re-testing.

CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

Submission and Design Requirements

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method of preventing soil squeeze and/or base heave during installation of the CSPs;
7. Method for preventing water and debris from entering the CSP prior to placing sand; and
8. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

Material

Corrugated steel pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 μm	#30	80% to 100%
425 μm	#40	40% to 80%
250 μm	#60	5% to 25%
150 μm	#100	0% to 6%

Construction

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Install CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

Basis of Payment

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

END OF SECTION

OBSTRUCTIONS – Item No.

Notice to Contractor

The Contactor shall be alerted to the potential presence of cobbles and boulders in the native glacially-derived soils, particularly within the glacial till deposit(s). Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, driving piles, advancing drilled shafts, and installation of any temporary protection systems that may be required.

Ground and Groundwater Control for Drilled Shaft Installation - Item No.

Notice to Contractor

Drilled shafts will be advanced through existing cohesive and non-cohesive fill materials, clayey silt to silty clay that contains interlayers of silt to silty sand, and glacial till deposits. The non-cohesive/granular soils could slough (if dry) or flow (if water-bearing) into unsupported auger holes during drilled shaft installation. Appropriate construction procedures and equipment will be required to minimize ground loss during drilling, installation of reinforcement and concrete placement.



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