



FINAL REPORT

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 401/Bloomfield Road Underpass Replacement (Site No. 13X-0241/B0)

County of Kent, Ontario

MTO GWP 3078-18-00, Assignment No. 3018-E-0011

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GEOCRES No.: 40J8-84

Latitude: 42.351025°

Longitude: -82.184201°

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Distribution List

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/BLOOMFIELD ROAD UNDERPASS REPLACEMENT
(SITE No. 13X-0241/B0)
COUNTY OF KENT, ONTARIO
MTO GWP 3078-18-00, ASSIGNMENT No. 3018-E-0011**

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now WSP Canada Inc.) 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. The Bloomfield Road underpass, the subject of this report, is part of GWP 3078-18-00, which addresses improvements to the interchanges and underpass structures on Highway 401 from Essex Road 42 westerly to the Chatham-Kent/Elgin County boundary.

This report presents the results of a foundation investigation conducted in support of preliminary design for the replacement of the Bloomfield Road underpass (MTO Structure Site No. 13X-0241/B0) crossing Highway 401. 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 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.

2.0 SITE DESCRIPTION

The topography in the area of the existing underpass at Highway 401/Bloomfield Road consists of flat to undulating land typically used for agriculture. 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 Bloomfield Road to be oriented north-south. The existing bridge is an 11 metre (m) wide by 66 m long, four-span steel girder bridge spanning four lanes of Highway 401.

The natural ground surface in the immediate vicinity of the site is at approximately Elevation 180 m. Highway 401 has been constructed on a low embankment with its grade at approximately Elevation 180 m at the structure site, while the existing Bloomfield Road grade is at approximately Elevation 186 m at the underpass. The existing approach embankments on Bloomfield Road are up to approximately 7 m high relative to the original ground surface. The existing embankment side slopes are well vegetated. The existing pavements and curb and gutter system do not appear to exhibit any notable displacements or distortions nor any areas of cracking that may suggest a global stability issue. As such, the existing embankments appear to be performing well and as intended.

The original design drawings indicate that the existing abutments supported on approximately 14 m long, 390 millimetre (mm) diameter, concrete filled steel tube piles. The front row of piles at the abutments are battered at 1 horizontal to 5 vertical (1H:5V). The piers are supported on approximately 9 m long, 390 mm diameter, concrete-filled steel tube piles. The underside of pier caps is at Elevation 178 m and the underside of abutment pile caps is at Elevation 183.8 m.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation at the proposed Bloomfield Road underpass replacement was carried out between July 13 and 16, 2020, during which time two sampled boreholes (designated as Boreholes BH-401 and BH-402) were advanced as near as practicable to the footprint of the proposed replacement structure abutments. The locations of the boreholes are shown on Drawing 1 following the text of this report.

The borehole investigation was carried out using track-mounted drilling equipment supplied and operated by specialist drilling contractors. The boreholes were advanced through the overburden and bedrock to depths of 23.8 m to 24.3 m, using HW casing and mud rotary techniques in the overburden and HQ size core barrels in the bedrock. Soil samples were obtained at 0.75 m and 1.5 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-402 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 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 Borehole BH-401 was backfilled with bentonite grout upon completion in accordance with Ontario Regulation 903 (as amended).

The field work was observed by a member of WSP 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 WSP 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 a sample of the cohesive deposits. The geotechnical laboratory tests were carried out to MTO and/or ASTM Standards, as applicable, and the results are presented in Appendix B.

Two 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.

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

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 borehole/drillhole records and shown on Drawings 1 and 2 are positioned relative to MTM NAD 83 (Zone 11) 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 11)		Ground Surface Elevation (m)	Borehole Termination Depth (m)	Borehole Termination Elevation (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)			
BH-401	4,690,205.9 (42.351026)	330,781.9 (-82.184632)	180.0	24.3	155.7
BH-402	4,690,170.6 (42.350707)	330,820.5 (-82.184164)	179.9	23.8	156.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This project lies within the physiographic region of southwestern Ontario known as the Bothwell Sand Plain which was the delta of the Thames River in prehistoric glacial Lake Warren. The Bothwell Sand Plain primarily consists of a thin layer of sand, approximately 1 metre thick, over the clay.⁴ Quaternary geology mapping indicates that surficial materials consist primarily of glaciolacustrine deposits of clayey silt and silty sand overlain by glaciolacustrine silty sand and sand.⁵ The mapping also indicates that a “till moraine” is present immediately south of the site. Based on geologic mapping the underlying bedrock surface is estimated to be about 25 m below the ground surface or at about Elevation 160 m.⁶ The rock is described as limestone, dolostone and shale of the Hamilton Group of middle Devonian age.⁷

Although the mapping provides a general indicator of the geologic conditions of the site, these maps only address the most recent phase of the region’s glacial geology based on near-surface materials and may not characterize the geologic complexity of the site at greater depths. In southwestern Ontario, the most significant prehistoric glacial features are associated with the last advance and retreat of ice through the area. As the ice receded from the region, a number of moraines and lakes were formed near the retreating ice front. In some areas, such as the Windsor-Chatham-Wallaceburg area, the clayey silt or silty clay deposits have a grain size distribution consistent with that of a cohesive glacial till although the density and strength of the materials are not consistent with deposition below a grounded ice sheet as commonly assumed for materials described as glacial till. Some of the soils described as glacial till were likely deposited from the underside of floating ice through a shallow water depth as a diamict (broadly graded mud) and, therefore, the soil carried little or no weight of the overlying ice while in

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

⁵ Kelly, R.I., 1991: Quaternary geology of the Chatham-Wheatley area; Ontario Geological Survey, Open File Map 163, scale 1:50 000.

⁶ Sado, E.V. and Faught, R.B. 1981: Drift Thickness of Chatham Area, Southern Ontario; Ontario Geological Survey Preliminary Map P.2453, Drift Thickness Series. Scale 1:50 000.

⁷ Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

other areas, the ice sheet may have been grounded and produced hard cohesive glacial till. Further, there are also likely areas the ice may have been floating or partially floating which has resulted in complex conditions. Near moraines, geologic conditions can be especially complex because of highly localized outwash (sand and gravel) deposits, silt and clay deposited in local ice-proximal lakes and ponds and comparatively short duration re-advances and retreats of the former ice sheets.

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 on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change and moreover, the interpreted stratigraphy shown on Drawing 1 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 topsoil, fill and relatively thin layers of fine-grained granular materials underlain by a thick deposit of very soft to very stiff clayey silt to silty clay and which is subsequently underlain by compact silty sand and gravel and limestone bedrock. 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 Topsoil

Topsoil with a thickness ranging between about 100 mm and 175 mm was encountered at the ground surface.

4.2.2 Fill

A layer of cohesive fill was encountered underlying the topsoil in Boreholes BH-402; this cohesive fill consists of silty clay with traces of sand and topsoil. The total fill thickness is 2.1 m and the base of the fill material is at approximately Elevation 177.7 m as encountered in this borehole.

The Standard Penetration Test (SPT) “N”-values measured within the cohesive fill range from 6 blows to 21 blows per 0.3 m of penetration indicating that the cohesive fill has firm to very stiff consistency. The natural water content measured on samples of the cohesive fill ranges from about 18% to 22%.

4.2.3 Surficial Silt

Silt was encountered underlying the topsoil layer in Borehole BH-401 and underlying the fill materials in Borehole BH-402. These layers are about 1.5 m and 2.8 m thick and the base of these materials are at depths of 3.0 m and 3.7 m below ground surface or at Elevations 177.0 m and 176.2 m.

The SPT “N”-values measured within the silt deposit range between 4 blows and 35 blows per 0.3 m of penetration which indicates that the deposit is loose to dense. The natural water content measured on samples of the silt layer is about 17% to 26%.

The results of grain size distribution tests carried out on two samples of the silt deposit are shown on Figure B-1 in Appendix B.

4.2.4 Clayey Silt

A cohesive deposit of clayey silt with traces to some sand and was encountered below the sandy silt and silt. The top of the deposit was encountered between Elevation 176.2 m and 177.0 m. The thickness of the deposit ranges between 15.6 m and 16.1 m in the boreholes.

The SPT “N”-values measured within the cohesive deposit range between 1 blow and 17 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 42 kPa to 84 kPa, as well as greater than 96 kPa, with a calculated sensitivity between about 1.6 and 1.8. 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 very stiff consistency, but is generally stiff.

The results of grain size distribution tests carried out on four samples of the clayey silt deposit are shown on Figure B-2 in Appendix B. Atterberg limits tests were carried out on four samples of this deposit and measured liquid limits ranging between about 28% and 34%, plastic limits ranging between about 14% and 16%, and plasticity indices ranging between about 14% and 18%. These results, which are plotted on the plasticity chart on Figure B-3 in Appendix B, indicate that the material can be classified as clayey silt of low plasticity. The natural water content measured on samples of the cohesive deposit ranges from about 4% to 51% with an average water content of about 21%.

A laboratory consolidation test was carried out on a sample of the clayey silt obtained from a Shelby tube sample. A preconsolidation stress of 360 kPa was estimated from the void ratio versus logarithmic stress plots, indicating an overconsolidation ratio (OCR) of 1.6. A bulk unit weight of about 20.2 kN/m³ and a specific gravity of 2.69 were measured on the consolidation test specimen. Details of the consolidation test results are included on Figure B-4 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-402 Sample 15	17.0 m / 162.9 m	220	360	140	1.6	0.156	0.019	0.54	3.2×10^{-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 cm ² /s

4.2.5 Silty Sand and Gravel

A non-cohesive layer of silty sand and gravel was encountered beneath the clayey silt deposit in Boreholes BH-401 and BH-402. The surface of the silty sand and gravel was encountered at about Elevation 160.5 m to 161.0 m and the thickness of the layer is about 1.2 m to 1.7 m in the boreholes.

The SPT “N”-values measured within the silty sand and gravel layer are 25 blows and 28 blows per 0.3 m of penetration indicating the layer is compact.

The results of grain size distribution tests carried out on two samples from the silty sand and gravel layer are shown on Figure B-5 in Appendix B. The natural water content measured on samples of the silty sand and gravel ranges from about 10% to 14%.

4.2.6 Limestone Bedrock

Bedrock was encountered underlying the silty sand and gravel layers in BH-401 and BH-402 at Elevation 159.2 m and 159.4 m. The bedrock consisted of dark grey, medium strong to strong limestone. The bedrock was cored for 3.1 m prior to terminating the boreholes. The total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) for each core run are as follows:

Borehole	Core Run	Core Depth/ Elevation (m)	TCR (%)	SCR (%)	RQD (%)
BH-401	1	21.2 to 22.7	95	95	95
BH-401	2	22.7 to 24.3	100	100	100
BH-402	1	20.7 to 22.2	95	51	38
BH-402	2	22.2 to 23.8	95	93	93

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. 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. Based on the colour change in the soils from brown to grey, the long-term groundwater level is estimated to be at about Elevation 176.5 m. In addition, some groundwater may be present within the surficial silt perched atop the clayey silt deposit.

A vibrating wire piezometer (VWP) was installed in Borehole BH-402 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 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

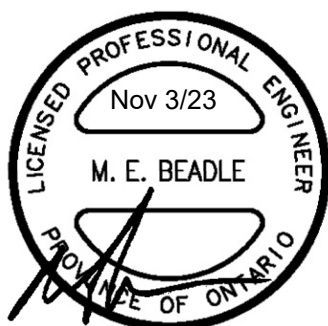
Two soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the following table summarizes the results of the testing:

Parameter	BH-401 Sample 4 (Elev. 176.6 m)	BH-402 Sample 4 (Elev. 176.5 m)
pH	7.72	7.95
Resistivity (ohm-cm)	741	2,280
Electrical Conductivity (mS/cm)	1.35	0.633
Chlorides ($\mu\text{g/g}$)	591	162
Soluble Sulphates ($\mu\text{g/g}$)	164	67
Soil Type	Clayey silt	Silt

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Michael Beadle, P. Eng., a Senior Principal geotechnical engineer with WSP Golder. Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP Golder, conducted an independent quality review of this report.

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[https://golderassociates.sharepoint.com/sites/110354/project/files/6/deliverables/fdns/fidrs to finalize/19124560-bl-rev0 final pfidr bloomfield \(gwp 3078-18-00\) oct 2023.docx](https://golderassociates.sharepoint.com/sites/110354/project/files/6/deliverables/fdns/fidrs%20to%20finalize/19124560-bl-rev0%20final%20pfidr%20bloomfield%20(gwp%203078-18-00)%20oct%202023.docx)

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 401/BLOOMFIELD ROAD UNDERPASS REPLACEMENT
(SITE No. 13X-0241/B0)
COUNTY OF KENT, ONTARIO
MTO GWP 3078-18-00, ASSIGNMENT No. 3018-E-0011**

6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides discussion and recommendations on geotechnical/foundation aspects for the preliminary design of the Bloomfield Road underpass replacement and associated approach embankments. The recommendations are based on WSP 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 Preliminary 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 the Preliminary Foundation Investigation 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 future 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 an 11 m wide by 67 m long, four-span steel girder bridge spanning four lanes of Highway 401. The existing abutments are supported on approximately 14 m long, 390 mm diameter, concrete-filled steel tube piles. The front row of piles at the abutments are battered at 1 horizontal to 5 vertical. The piers are supported on approximately 9 m long, 390 mm diameter, concrete-filled steel tube piles. 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.

It is understood that the existing bridge will be replaced with a new two-span integral abutment bridge on the current alignment. The new bridge will accommodate four lanes of traffic (two through lanes and two speed change lanes). As such, some symmetrical widening of the existing approach embankments will be required.

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

- **Strip or spread footings founded on the stiff to very stiff silty clay:** Shallow footings are feasible at this site. However, this option would result in settlements 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 1.0 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 compact to dense silt 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 approach embankments are technically feasible for this site, although similar to the footing option described above, these foundations would be affected by settlement under the widened 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 on bedrock:** 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 predominantly from end-bearing. Design tip elevations should be expected to vary across the foundation elements and will require piles that are about 20 m in length. 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 (caissons) founded on bedrock:** Drilled shafts (caissons) could be considered for support of the proposed centre pier. While feasible for support of the abutments, they would not permit use of integral abutments. Caissons at the centre pier would be approximately 20 m long and would develop resistance predominantly from end-bearing on/in the bedrock. This option may be advantageous during construction staging in the median, and it may also allow for elimination of a below-grade pile cap and support of structural columns directly on individual caissons. As with driven piles, cobbles and boulders within the native soil deposits should be anticipated which could affect drilling and augering; however, given the anticipated diameters, mechanical equipment can probably break and/or extract such obstructions.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments and new centre pier on driven steel H-piles founded on the bedrock, particularly if integral abutments are preferred. Caisson foundations are also considered to be a good alternative for support of the median pier, and their use may eliminate the need for a below-grade pile cap.

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.⁸ 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 and the measured undrained shear strengths, 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.

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 recommendations in this report have been developed based on the 5th generation seismic hazard maps developed by the GSC, which were made available for public use in December 2015. The values in the following section can be updated based on the 6th generation seismic hazard maps if warranted.

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

⁸ 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.

Site Class C Spectral Values

Seismic Hazard Values	2% Probability of Exceedance in 50 years (2,475-year return period)
PGA (g)	0.069
PGV (m/s)	0.054
$S_a(0.2)$ (g)	0.114
$S_a(0.5)$ (g)	0.070
$S_a(1.0)$ (g)	0.039
$S_a(2.0)$ (g)	0.019
$S_a(5.0)$ (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

Seismic Hazard Values	2% Probability of Exceedance in 50 Years (2,475-year)
PGA (g)	0.089
PGV (m/s)	0.079
$S_a(0.2)$ (g)	0.141
$S_a(0.5)$ (g)	0.103
$S_a(1.0)$ (g)	0.060
$S_a(2.0)$ (g)	0.030
$S_a(5.0)$ (g)	0.008
$S_a(10.0)$ (g)	0.003

6.4 Driven Steel H-Piles

6.4.1 Tip Elevations and Axial Geotechnical Resistances

For preliminary design, a factored ultimate axial geotechnical resistance of 3,000 kilonewtons (kN) per pile may be used for HP 310x110 piles driven to refusal on the limestone bedrock at approximately Elevation 159 m. This factored ultimate geotechnical resistance is greater than the structural capacity of conventional HP 310x110 piles.

The factored serviceability geotechnical resistances for 25 mm of settlement will be higher than the factored ultimate geotechnical resistance value given, so Serviceability Limit States (SLS) will not govern the design. The General Arrangement drawings should note that the piles are to be driven to bedrock.

Pile installation should be in accordance with OPSS.PROV 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. These criteria must therefore be established at the time of construction once the piling equipment is confirmed. The structural engineer should incorporate potential for variability of the bedrock surface in estimates of the lengths of the piles.

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

The clayey silt and silty sand and gravel deposits have the potential to contain cobbles and boulders that may interfere with driving of the piles or cause damage to pile tips. Further, the piles will be driven to the limestone bedrock. As such, all piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles. Driving shoes (such as Titus Standard “H” Bearing Pile points) are preferred over flange plates (OPSD 3000.10: Foundation Piles – Steel H-Pile Driving Shoe).

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 1.0 metres of soil cover or thermal equivalent.

6.4.3 Downdrag Loads at Abutments

The widening of the approach embankments will cause long-term consolidation settlement of the underlying 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 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 pile driven to the design elevations given in the preceding sections has been assessed as approximately 200 kN per pile, based on the Alpha method outlined in the *Canadian Foundation Engineering Manual* (CFEM, 2006) together with engineering judgment. 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

6.4.4 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 percent 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.

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 (American Petroleum Institute (API), 2002) and s_u may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile.

Foundation Element	Soil Layer	Approximate Elevation Range (m)	n_h (kPa/m)	s_u (kPa)
North Abutment	Compact silt	177 to 180	5,500	--
	Firm clayey silt	175 to 177	--	50
	Very stiff clayey silt	167 to 175	--	150
	Stiff clayey silt	161 to 167	--	80
	Compact silty sand and gravel	159 to 161	21,000	--

Foundation Element	Soil Layer	Approximate Elevation Range (m)	n_h (kPa/m)	S_u (kPa)
Centre Pier	Compact to dense silt	Above 176	5,500	--
	Stiff to very stiff clayey silt	176 to 161	--	80 to 125
	Compact silty sand and gravel	159 to 161	21,000	--
South Abutment	Firm to very stiff clayey silt fill	178 - 180	--	75
	Compact to dense sandy silt	176 to 178	5,500	--
	Stiff clayey silt	161 to 176	--	125
	Compact silty sand and gravel	160 to 161	21,000	--

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 the 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 the CHBDC* (2019).

6.5 Drilled Shafts (Caissons)

6.5.1 Tip Elevations and Axial Geotechnical Resistances

It is recommended that drilled shafts (caissons) for support of the centre pier be socketed approximately 1.0 m into the medium strong to strong limestone bedrock. For preliminary design, a founding level at Elevation 158 m is recommended, which will result in caissons lengths of about 20 to 22 m.

The factored ultimate axial geotechnical resistances provided in the table below may be used for preliminary design; these values have been developed based on an estimated average uniaxial compressive strength of 50 MPa. The factored serviceability geotechnical resistances (for 25 mm of settlement) are greater than the factored ultimate geotechnical resistances, which will govern the design. It is noted that the bedrock surface elevation and the weathering, quality, compressive strength and modulus of the upper portion of the bedrock must be confirmed through further investigations in the detail design stage to confirm and potentially refine these preliminary recommendations.

Nominal Rock Socket Diameter (m)	Factored Ultimate Geotechnical Resistance (MN)	Factored Serviceability Geotechnical Resistance (MN)
0.9	15	>15
1.2	25	>25
1.5	40	>40
2.1	75	>75

6.5.2 Resistance to Lateral Loads

At this preliminary design stage, resistance to lateral loads for caissons at the centre pier may be assessed based on the recommendations provided in Section 6.4.4 of this report.

6.6 Lateral Earth Pressures for Design

The lateral earth pressure acting on the abutment walls and associated wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge, the freedom of movement of the structure, and the drainage conditions behind the walls.

Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type I or Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).

For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.0 m (equivalent to the depth of frost penetration at this site as interpreted from OPSD 3090.101) behind the back of the wall, as shown on Figure C6.31(a) of the *Commentary* to CHBDC (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn flatter than 1H:1V extending up and back from the rear face of pile cap as shown on Figure C6.31(b) of the *Commentary* to CHBDC (2019).

The lateral earth pressure coefficients provided the table below have been developed for flat (i.e., non-sloping) ground above the wall, as well as for a 2H:1V slope condition for unrestrained walls. If the inclination of the slope above the wall differs, revised lateral earth pressures parameters will need to be calculated in accordance with CHBDC Clause C6.12.1, Figure C6.28 (active earth pressure), and Clause C6.12.2.2 (at-rest earth pressure).

Wall Movement Condition	Restrained Wall	Unrestrained Wall			
Fill Material	Earth Fill Behind Granular, $\Phi'=30^\circ$	Granular A and B Type II $\Phi'=36^\circ$		Granular B Type I $\Phi'=32^\circ$	
Unit Weight (kN/m ³)	19	22	22	21	21
Ground Surface Above Top of Wall	Horizontal	Horizontal	2H:1V	Horizontal	2H:1V
Active Earth Pressure (K_a)	--	0.26	0.36	0.31	0.46
At-Rest Earth Pressure (K_0)	0.50	--	--	--	--

If the wall support and superstructure allow lateral yielding, 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.

6.7 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 the existing alignment and the existing embankments (which have side slopes oriented at approximately 2H:1V) will be symmetrically widened by about 5 m on each side. A grade raise on the order of 2 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 interpreted from the data from the subsurface investigation; a groundwater level of Elevation 176.5 m was used adjacent to the north and south abutments.

As indicated above, the existing embankments are about 7 m in height with side slopes oriented at about 2H:1V. The side slopes are well vegetated. The existing pavements and curb and gutter system do not appear to exhibit any notable displacements or distortions nor any areas of cracking that may suggest a global stability issue. As such, the existing embankments appear to be performing well and as intended.

6.7.1 Subgrade Preparation and Embankment Construction

Prior to construction of the widened 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.

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). Relatively free draining fill is considered necessary for the widening to reduce the potential for water in the existing fill to become trapped in the embankment which could result in increased porewater pressures and a consequent decrease in overall embankment stability.

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. Following filling, the embankment side slopes should be trimmed to an inclination of 2 horizontal to 1 vertical or flatter. Embankment widenings constructed in this manner will have a long-term factor of safety in excess of 1.3.

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.

6.7.2 Settlement

6.7.2.1 Methods and Parameters

To estimate the magnitude of the expected settlements under the widened approach embankment loading, analyses were carried out 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) 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 , used in the time/rate settlement analysis for the over-consolidated soils at this site is estimated to be $3 \times 10^{-3} \text{ cm}^2/\text{s}$, 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)	OCR	e_o	C_c	C_r	Elastic Modulus (MPa)
Compact silt	20	--	--	--	--	30
Firm clayey silt	19	3	0.6	0.20	0.020	18
Very stiff clayey silt	19	5	0.5	0.15	0.015	53
Stiff clayey silt	19	1	0.65	0.20	0.020	30
Compact silty sand & gravel	20	--	--	--	--	150

6.7.2.2 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 the 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 widening. 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 widening of the approach embankments for the 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.

6.7.2.3 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.

Location	Settlement Criteria (mm)	Estimated Total Settlement (mm)	Estimated Immediate Settlement (mm)	Estimated Post-Construction Settlement (mm)
Abutment	25	50	20-25	25-30
20 m	50	55	25	30
50 m	75	50	20	30
75 m	100	50	15	35

As shown above, the estimated post-construction primary consolidation settlement is less than MTO's settlement performance criteria except at the abutments where it exceeds the criterion by less than 5 mm. Further, based on our experience in these soils, the actual soil response tends to be stiffer than the modelling suggests. As such, the actual settlements are expected to be somewhat less than those shown above.

6.8 Analytical Testing for Construction Materials

The results of an analytical test on two samples of the upper materials are provided in Part A 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.

6.9 Construction Considerations

6.9.1 Excavations and Groundwater Control

Excavations for pile caps and/or abutments will penetrate the existing fill materials (including those of the existing embankment) silt to sandy silt and extend into the clayey silt deposits. The groundwater level is expected to be at about Elevation 176.5 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 foundation's 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.PROV 902, 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 deposit would be classified as Type 2 soils.

6.9.2 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.

6.9.3 Deep Foundations

Cobbles and boulders should be expected in the soils at the site, which may impact installation of protection systems and/or pile driving operations. An NSSP or Notice to Contractor should be added to the future Contract Documents to alert the contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile installation. Further, the piles will be driven to the limestone bedrock. As such, piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles. Driving shoes (such as Titus Standard "H" Bearing Pile points) are preferred over flange plates. 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.

6.10 Recommendations for Additional Work in Detailed Design

The following additional foundation investigation is recommended as part of detailed design, in order to meet the typical requirements in MTO's Guideline for Foundation Engineering Services:

- One or two new boreholes at the median pier, with sufficient rock coring and testing (uniaxial compressive strength and modulus) to allow design of caissons founded on or socketed into the bedrock.
- One borehole at each abutment on the east side of the existing bridge, to assess the soil properties and bedrock elevation (including confirmation of any variation in the bedrock surface elevation for assessment of pile length).

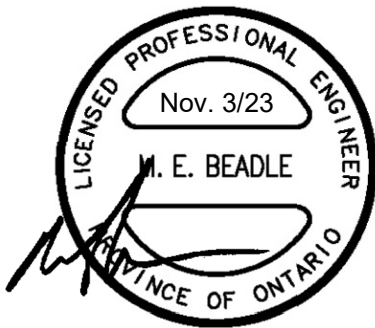
- One borehole through the existing embankment at each approach to supplement the subsurface model for assessment and design of approach embankment widening as well as to provide information for contractor design of protection systems, if these will be required for the bridge replacement.
- Geophysics testing such as Multi-Channel Analysis of Surface Waves (MASW) or vertical seismic profiling to measure the site-specific seismic shear wave velocity for potential refinement of seismic site class and input to seismic design; such testing is generally considered helpful given the likely application of the higher values associated with the 6th generation seismic hazard mapping.
- One or more of the above elements could be replaced by seismic CPT testing, although such testing will not provide information regarding the bedrock for deep foundation design, and it may not adequately characterize the seismic shear wave velocity over a 30 m profile.

The preliminary foundation recommendations provided in this report should be reassessed and updated, as applicable, based on the results of additional investigation in detailed design.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Michael Beadle, P.Eng., a Senior Principal geotechnical engineer with WSP Golder. An independent technical and quality review of this report was carried out by Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP Golder.

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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
------------	---------------------------------------------------------------------------------------------

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 Sheetting
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

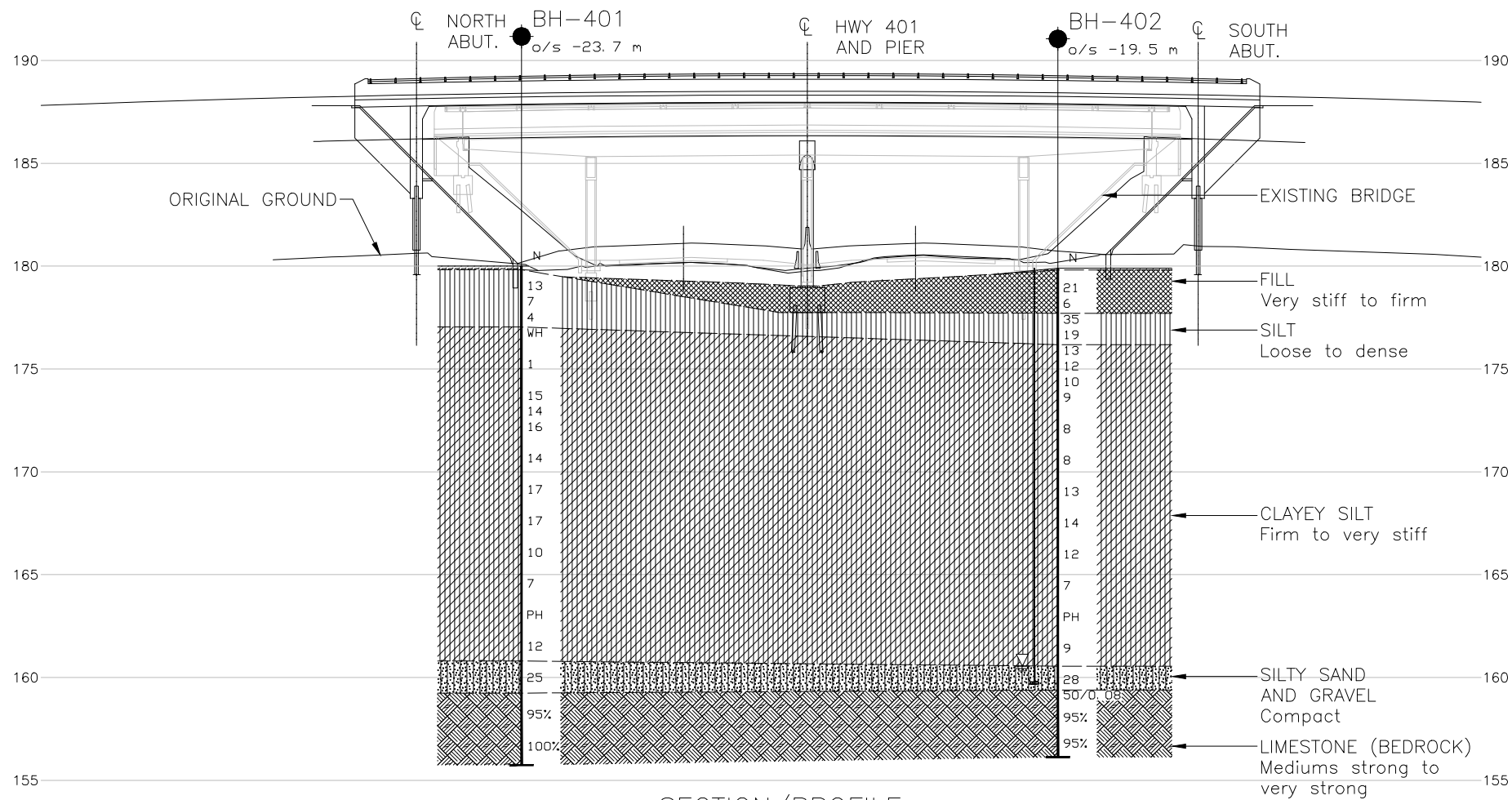
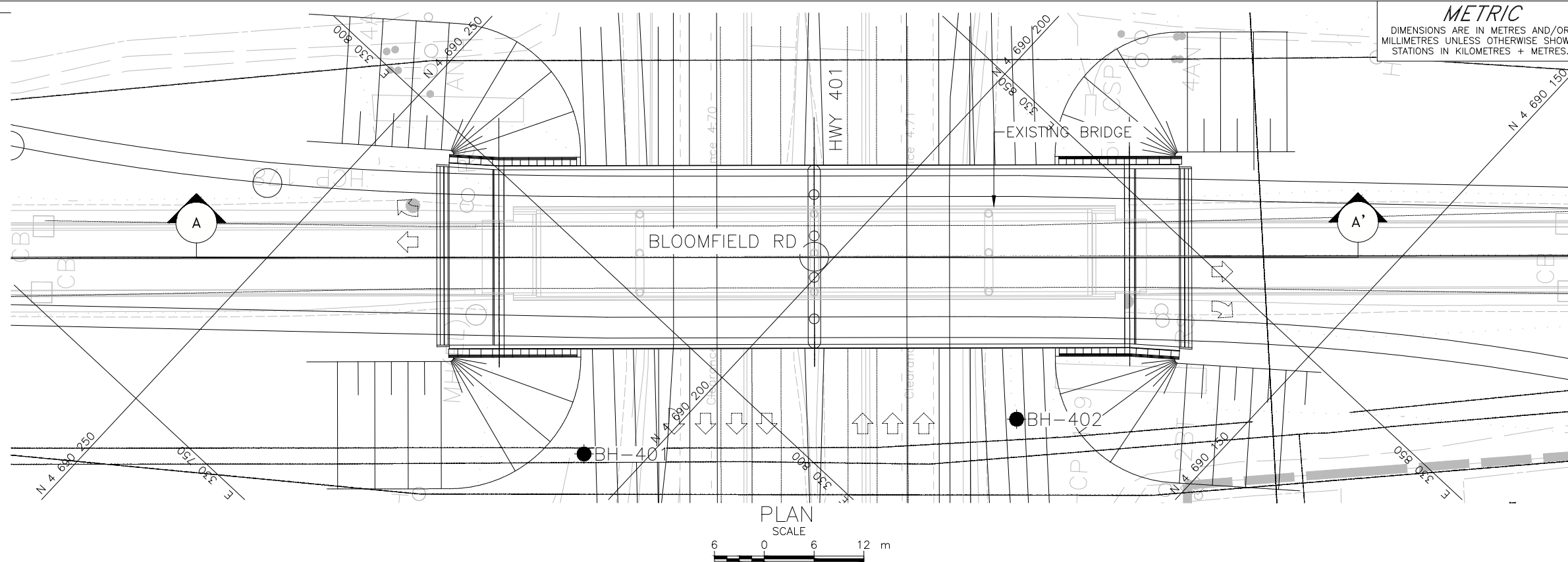
Ontario Water Resources Act

Ontario Regulation 903, Wells (as amended)

Ontario Occupational Health and Safety Act

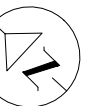
Ontario Regulation 213/91, Construction Projects (as amended)

Drawings

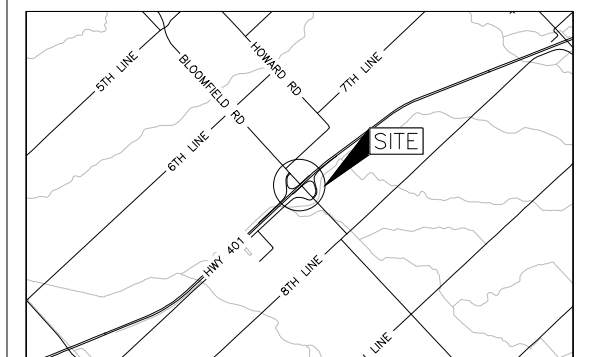


CONT No.
GWP No.3078-18-00

HIGHWAY 401
BLOOMFIELD ROAD
BOREHOLES LOCATION PLAN AND
SOIL STRATA



SHEET



LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BH-401	180.0	4690205.9	330781.9
BH-402	179.0	4690170.6	330820.5



NOTES

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

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

REFERENCE

Base plan provided in digital format by Stantec, drawing file no. 944_HWY_401_XR_EBP.dwg.
GA provided in digital format by Stantec, drawing file no. 165001119-13-241-p1.dwg, received May 16, 2023.

NO.	DATE	BY	REVISION
Geocres No. 40J8-84			
HWY.		PROJECT NO. 19124560	DIST. .
SUBM'D. MEB	CHKD. MEB	DATE: 11/06/2023	SITE: .
DRAWN: DD	CHKD. MEB	APPD. LCC	DWG. 1

Tables

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Option	Feasibility	Advantages	Disadvantages	Geotechnical Risks/Consequences	Constructability	Relative Costs
Strip or spread footings founded on the compact to dense silt	<ul style="list-style-type: none"> Geotechnically feasible for pier and abutment foundations, although would have to design for total and differential settlements 	<ul style="list-style-type: none"> Relatively minor groundwater seepage anticipated from surficial silt, so pumping from filtered sumps expected to provide adequate groundwater control. 	<ul style="list-style-type: none"> Relatively low geotechnical resistances for 25 mm of settlement Construction of approach embankments would result in total settlements of up to about 50 mm and post-construction consolidation settlements on the order of 25 mm to 30 mm that would impact abutments. Temporary protection system required along Bloomfield Road, unless Bloomfield Road is closed during bridge replacement. 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"> Geotechnically 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 total settlements of up to about 50 mm, and long-term post-construction settlements on the order of 25 mm to 30 mm, which would impact serviceability resistance for footings. Temporary protection system required along Bloomfield Road, unless Bloomfield Road is closed during bridge replacement. 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 on strong limestone bedrock	<ul style="list-style-type: none"> Feasible and preferred for support of bridge abutments and centre pier 	<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. Pile caps may be perched in approach embankments, eliminating requirement for groundwater control 	<ul style="list-style-type: none"> Potential of piles “hanging up” on or being damaged by cobbles/boulders 	<ul style="list-style-type: none"> Negligible risk of post-construction settlements 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 on or in strong limestone bedrock	<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 Concrete would have to be placed by tremie methods Not suitable for integral abutment design 	<ul style="list-style-type: none"> Moderate risk of disturbance of water-bearing non-cohesive soils above bedrock, requiring use of temporary liners and tremie concrete 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 	<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.

APPENDIX A

Borehole Records

[illegible]

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTO\HWY 401 CHATAM-LONDON\02 DATA\GIN\THWY 401 CHATAM-LONDON.GPJ GAL-GTA.GDT 11/6/23

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



PROJECT		19124560		RECORD OF BOREHOLE No BH-402		SHEET 1 OF 2		METRIC					
G.W.P.		3078-18-00		LOCATION		N 4690170.6; E 330820.5 MTM NAD 83 ZONE 11 (LAT. 42.350707; LONG. -82.184164)		ORIGINATED BY MR					
DIST		West HWY 401		BOREHOLE TYPE		Mud Rotary		COMPILED BY ZJB					
DATUM		Geodetic		DATE		July 16, 2020		CHECKED BY					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
179.9	GROUND SURFACE												
0.0	TOPSOIL, clayey Dark brown												
	CLAYEY SILT (CL), trace sand, with topsoil and roots (FILL) Very stiff to firm Brown and grey		1	SS	21		179			o			
			2	SS	6		178			o			
177.7	SILT (ML), trace to some clay Compact to dense Brown		3	SS	35		177			o			0 1 83 16
			4	SS	19					o			
176.2	CLAYEY SILT (CL), some sand, trace gravel Stiff Grey		5	SS	13		176			o			
			6	SS	12		175			o			
			7	SS	10		174			o			
			8	SS	9					o			3 33 36 28
			9	SS	8		172			o			
			10	SS	8		171			o			
			11	SS	13		169			o			
			12	SS	14		168			o			
			13	SS	12		166			o			
							165						

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTOWHY_401_CHATAM-LONDON\02_DATA\GINT\HWY_401_CHATAM-LONDON.GPJ GAL-GTA.GDT 11/6/23

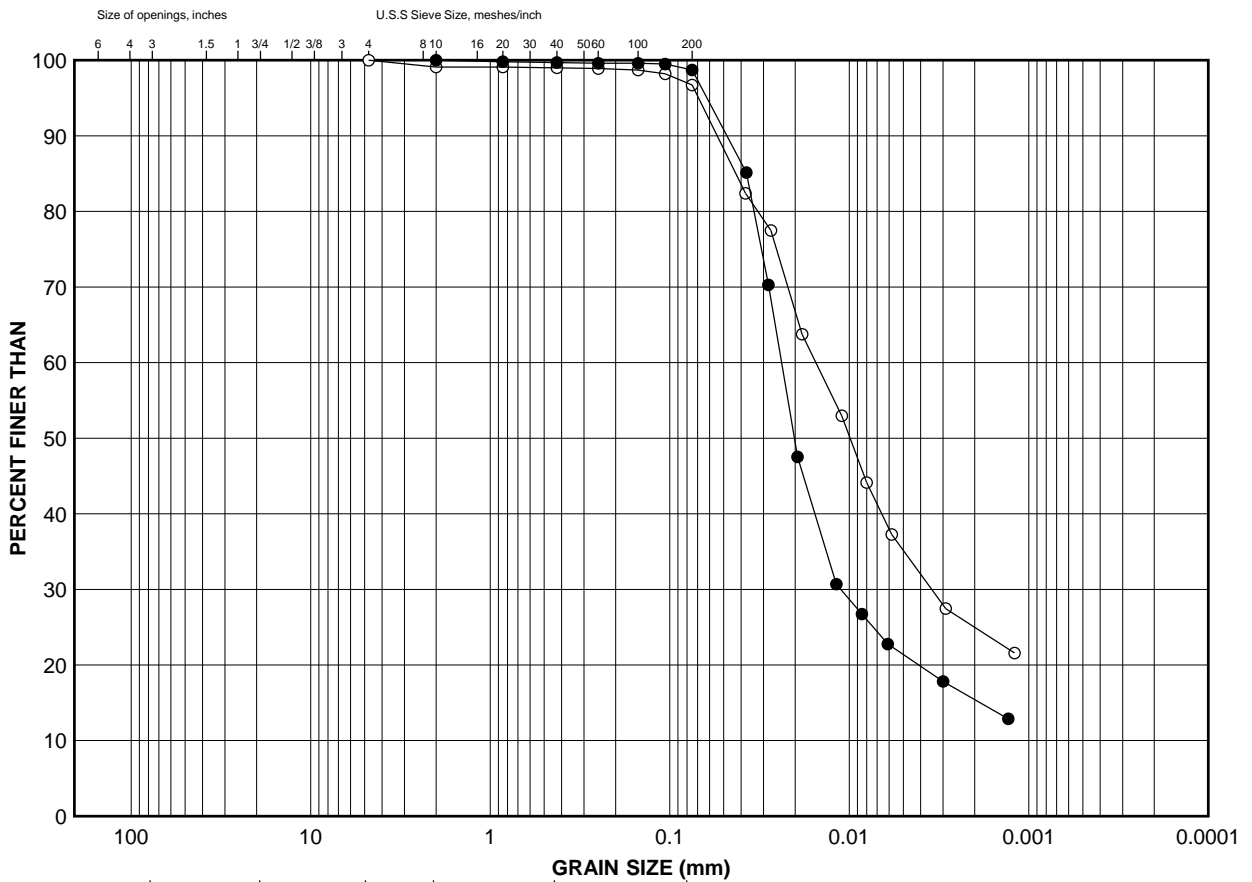


PROJECT		RECORD OF BOREHOLE		No BH-402		SHEET 2 OF 2		METRIC					
G.W.P. 19124560		LOCATION		N 4690170.6; E 330820.5 MTM NAD 83 ZONE 11 (LAT. 42.350707; LONG. -82.184164)		ORIGINATED BY		MR					
DIST West HWY 401		BOREHOLE TYPE		Mud Rotary		COMPILED BY		ZJB					
DATUM Geodetic		DATE		July 16, 2020		CHECKED BY							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
--- CONTINUED FROM PREVIOUS PAGE ---													
160.6	CLAYEY SILT (CL), some sand, trace gravel Stiff Grey		14	SS	7		164						2 20 42 36
19.4	SILTY SAND and GRAVEL Compact Grey		15	SS	PH		163						
159.4			16	SS	9		162						
20.5	LIMESTONE BEDROCK Dark grey		17	SS	28		161						
			18	SS	50/76mm		160						28 45 19 8
			-	RC	REC 95%		159						RQD = 38%
				RC	REC 95%		158						
							157						RQD = 93%
156.1	END OF BOREHOLE												
23.8	NOTE: 1. Groundwater encountered during drilling at 19.35m bgs (Elev. 160.5 m) on July 13, 2020.												

GTA-MTO 001 S:\CLIENTS\MTOWHY_401_CHATAM-LONDON\02_DATA\GINT\HWY_401_CHATAM-LONDON.GPJ GAL-GTA.GDT 11/6/23

APPENDIX B

Geotechnical Laboratory Test Results

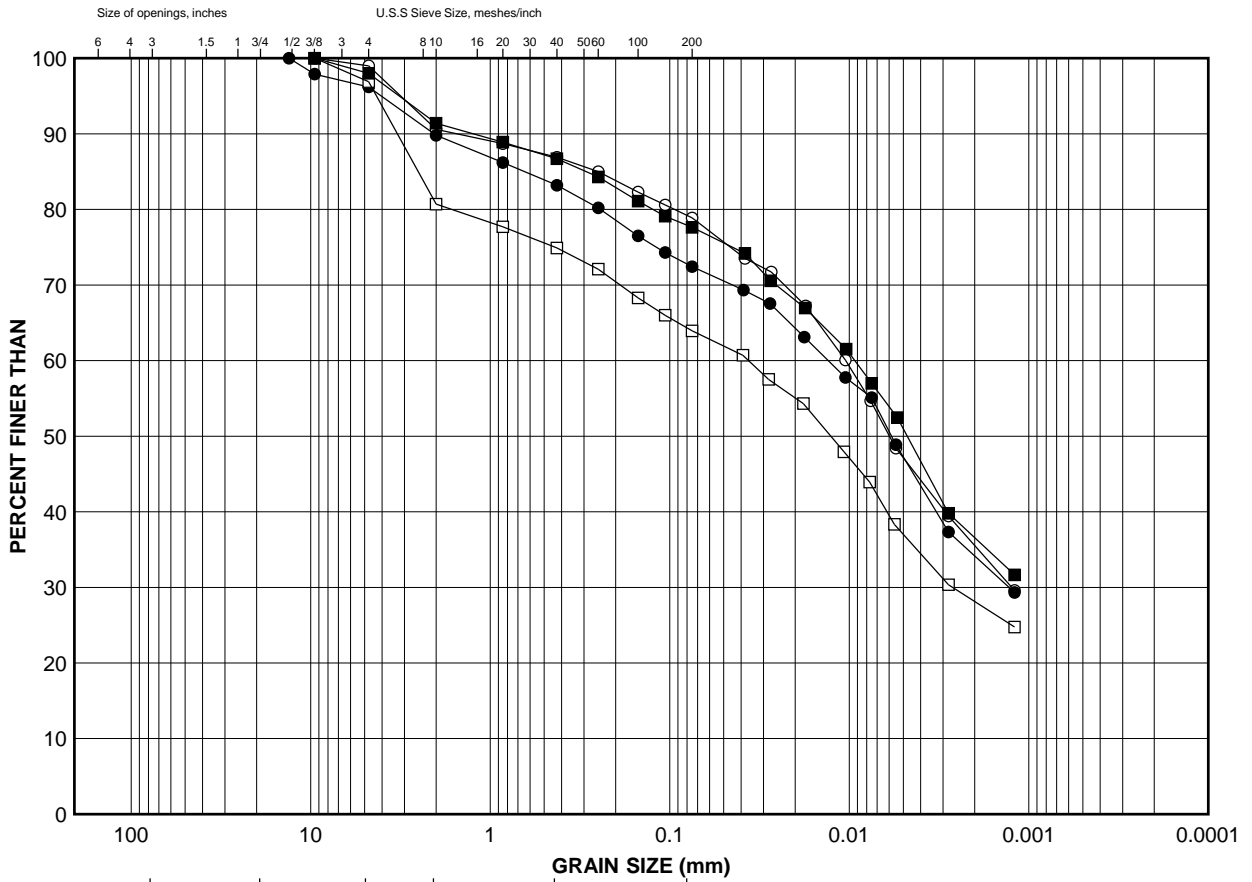


Cobble Size	coarse	fine	coarse	medium	fine	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
○	401	2	178.2
●	402	3	177.4

PROJECT BLOOMFIELD ROAD UNDERPASS REPLACEMENT HIGHWAY 401 RECONSTRUCTION GWP 3024-18-00					
TITLE GRAIN SIZE DISTRIBUTION SILT					
		PROJECT No.		19124560	
		FILE No.		GSD-v2 - SILT	
		DRAWN		MEB NOV 29-22	
		CHECK			
		SCALE		N/A REV. 0	
FIGURE B-1					

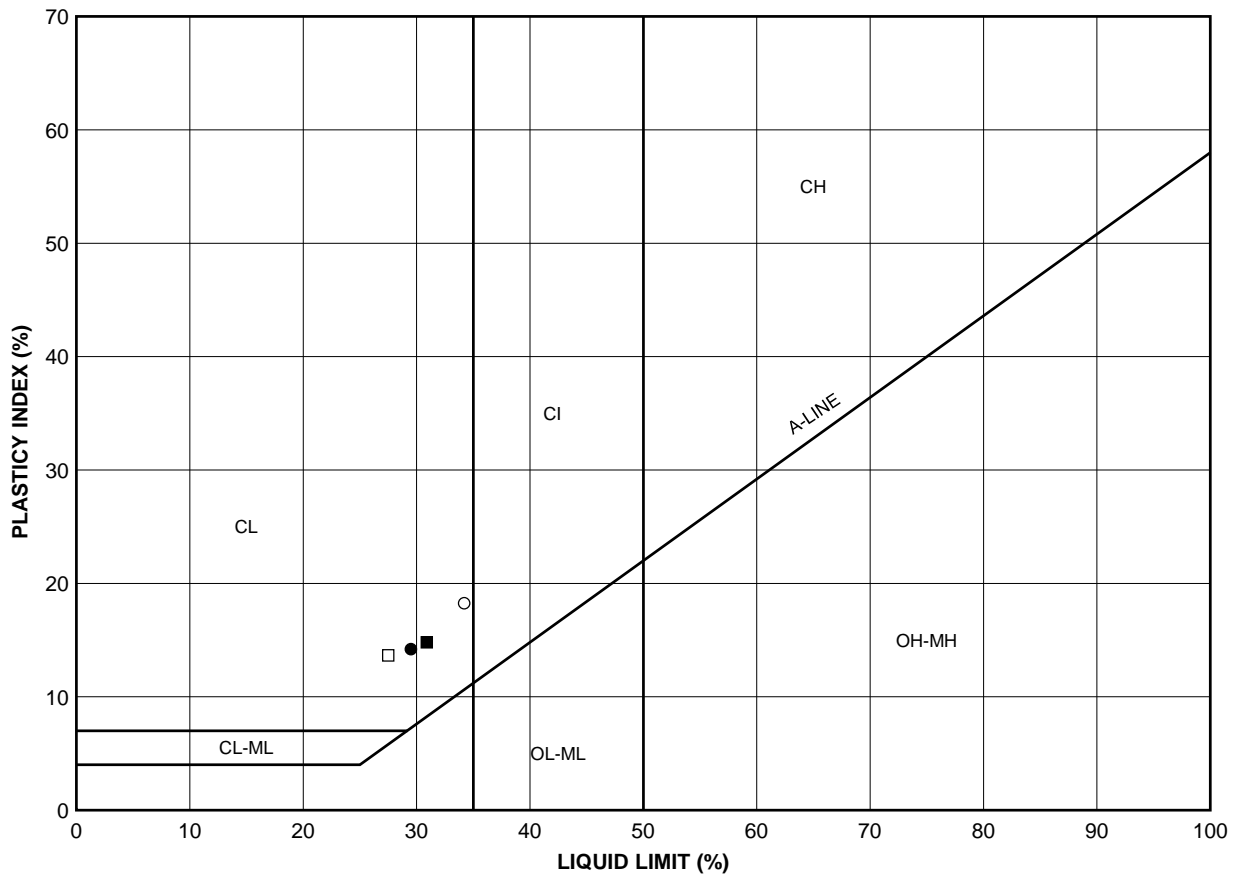


Cobble Size	coarse	fine	coarse	medium	fine	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

LEGEND

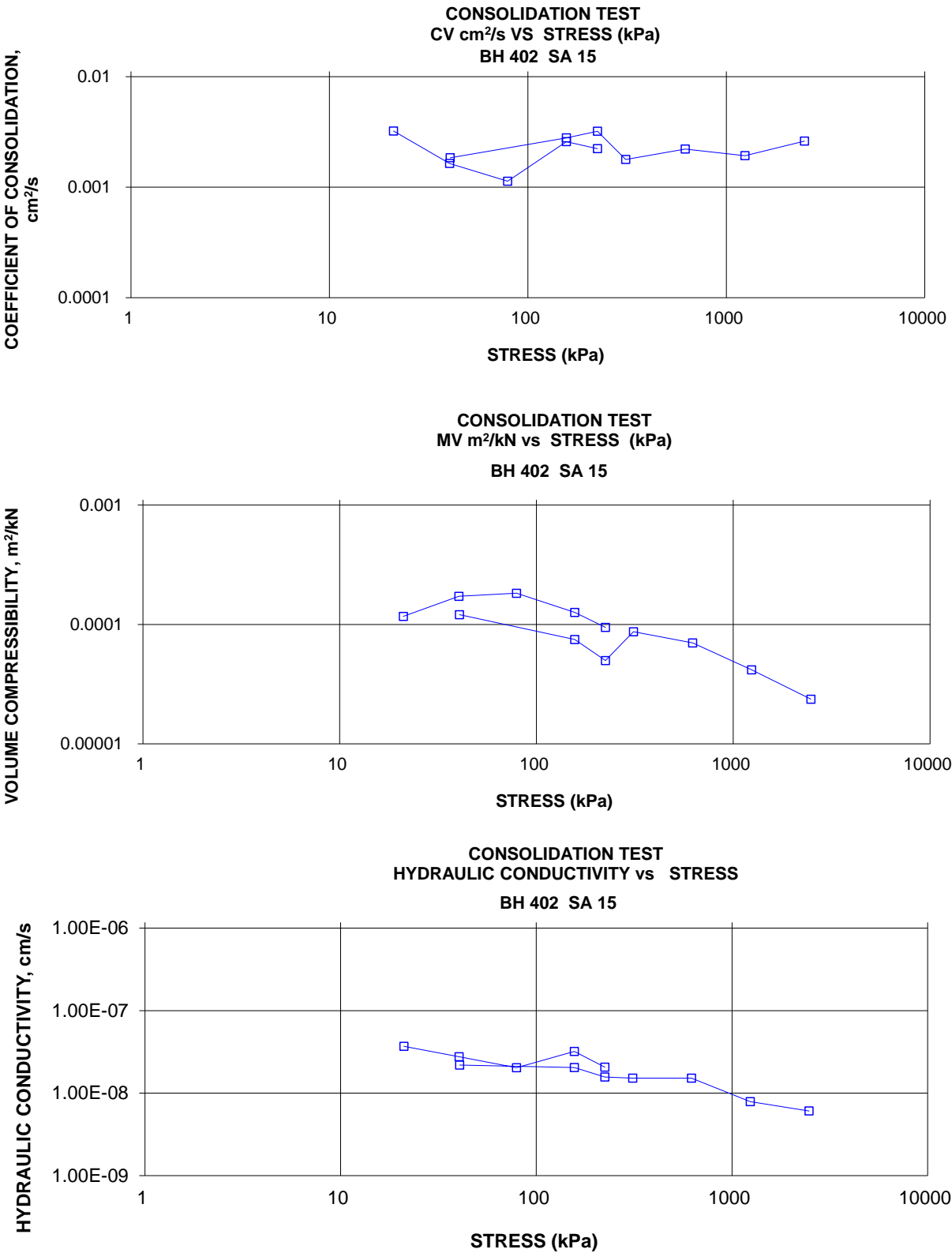
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
○	401	5	175.1
●	401	11	167.5
□	402	8	173.6
■	402	14	164.5

PROJECT BLOOMFIELD ROAD UNDERPASS REPLACEMENT HIGHWAY 401 RECONSTRUCTION GWP 3024-18-00					
TITLE GRAIN SIZE DISTRIBUTION CLAYEY SILT					
		PROJECT No.		19124560	
		FILE No.		GSD-v2 - CLAYEY SILT	
		SCALE		N/A	
DRAWN		MEB		NOV 29-22	
CHECK					
FIGURE B-2					



PROJECT				BLOOMFIELD ROAD UNDERPASS REPLACEMENT HIGHWAY 401 RECONSTRUCTION GWP 3024-18-00			
TITLE				PLASTICITY CHART CLAYEY SILT			
PROJECT No.		19124560		FILE No.		Limits-MTO - clayey silt	
DRAWN		MEB		SCALE		N/A	
CHECK				REV.		0	
wsp		GOLDER		FIGURE B-3			

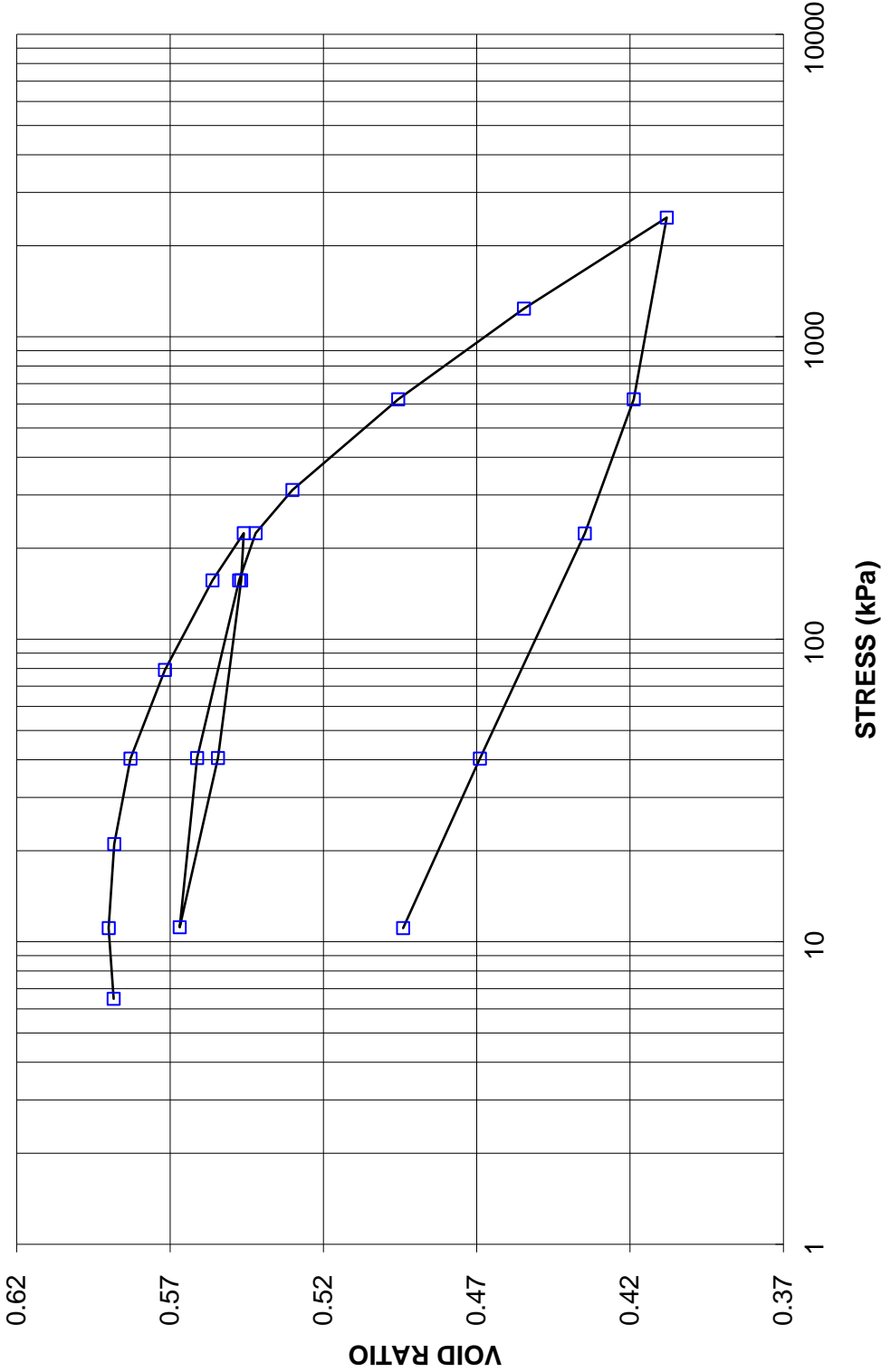
CONSOLIDATION TEST SUMMARY					FIGURE B-4.1			
ASTM D2435/D2435M								
SAMPLE IDENTIFICATION								
Project Number		19124560			Sample Number		15	
Borehole Number		402			Sample Depth, m		16.77-17.23	
TEST CONDITIONS								
Test Type		Laboratory Standard			Load Duration, hr		24	
Oedometer Number		8						
Date Started		10/20/2020						
Date Completed		11/04/2020						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		1.91			Unit Weight, kN/m ³		20.24	
Sample Diameter, cm		6.35			Dry Unit Weight, kN/m ³		16.62	
Area, cm ²		31.63			Specific Gravity, measured		2.69	
Volume, cm ³		60.38			Solids Height, cm		1.202	
Water Content, %		21.80			Volume of Solids, cm ³		38.03	
Wet Mass, g		124.60			Volume of Voids, cm ³		22.35	
Dry Mass, g		102.3			Degree of Saturation, %		99.8	
TEST COMPUTATIONS								
	Stress	Corr. Height	Void Ratio	Average Height	t ₉₀	cv.	mv	k
	kPa	cm		cm	sec	cm ² /s	m ² /kN	cm/s
	0.00	1.909	0.588	1.909				
	6.48	1.910	0.588	1.909				
	11.09	1.912	0.590	1.911				
	21.04	1.910	0.588	1.911	240	3.22E-03	1.17E-04	3.69E-08
	40.30	1.903	0.583	1.906	470	1.64E-03	1.72E-04	2.76E-08
	79.14	1.890	0.572	1.896	673	1.13E-03	1.83E-04	2.03E-08
	156.50	1.871	0.556	1.880	290	2.58E-03	1.26E-04	3.20E-08
	224.09	1.859	0.546	1.865	331	2.23E-03	9.46E-05	2.07E-08
	156.50	1.860	0.547	1.859				
	40.55	1.869	0.554	1.864				
	11.18	1.884	0.567	1.876				
	40.55	1.877	0.561	1.881	406	1.85E-03	1.21E-04	2.19E-08
	156.44	1.861	0.547	1.869	265	2.79E-03	7.48E-05	2.05E-08
	224.13	1.854	0.542	1.857	228	3.21E-03	4.98E-05	1.57E-08
	311.26	1.840	0.530	1.847	406	1.78E-03	8.68E-05	1.52E-08
	620.87	1.798	0.496	1.819	317	2.21E-03	7.01E-05	1.52E-08
	1240.27	1.749	0.455	1.774	346	1.93E-03	4.17E-05	7.88E-09
	2478.46	1.693	0.408	1.721	240	2.62E-03	2.37E-05	6.07E-09
	620.87	1.706	0.419	1.699				
	224.03	1.725	0.435	1.716				
	40.35	1.766	0.469	1.746				
	11.09	1.796	0.494	1.781				
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 12-17cm from bottom of the tube. Specimen swelled under 11.09kPa.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		1.80			Unit Weight, kN/m ³		21.03	
Sample Diameter, cm		6.35			Dry Unit Weight, kN/m ³		17.66	
Area, cm ²		31.63			Specific Gravity, measured		2.69	
Volume, cm ³		56.82			Solids Height, cm		1.202	
Water Content, %		19.12			Volume of Solids, cm ³		38.03	
Wet Mass, g		121.86			Volume of Voids, cm ³		18.79	
Dry Mass, g		102.3						
Prepared By: LH					WSP Golder		Checked By: MM	



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B-4.3

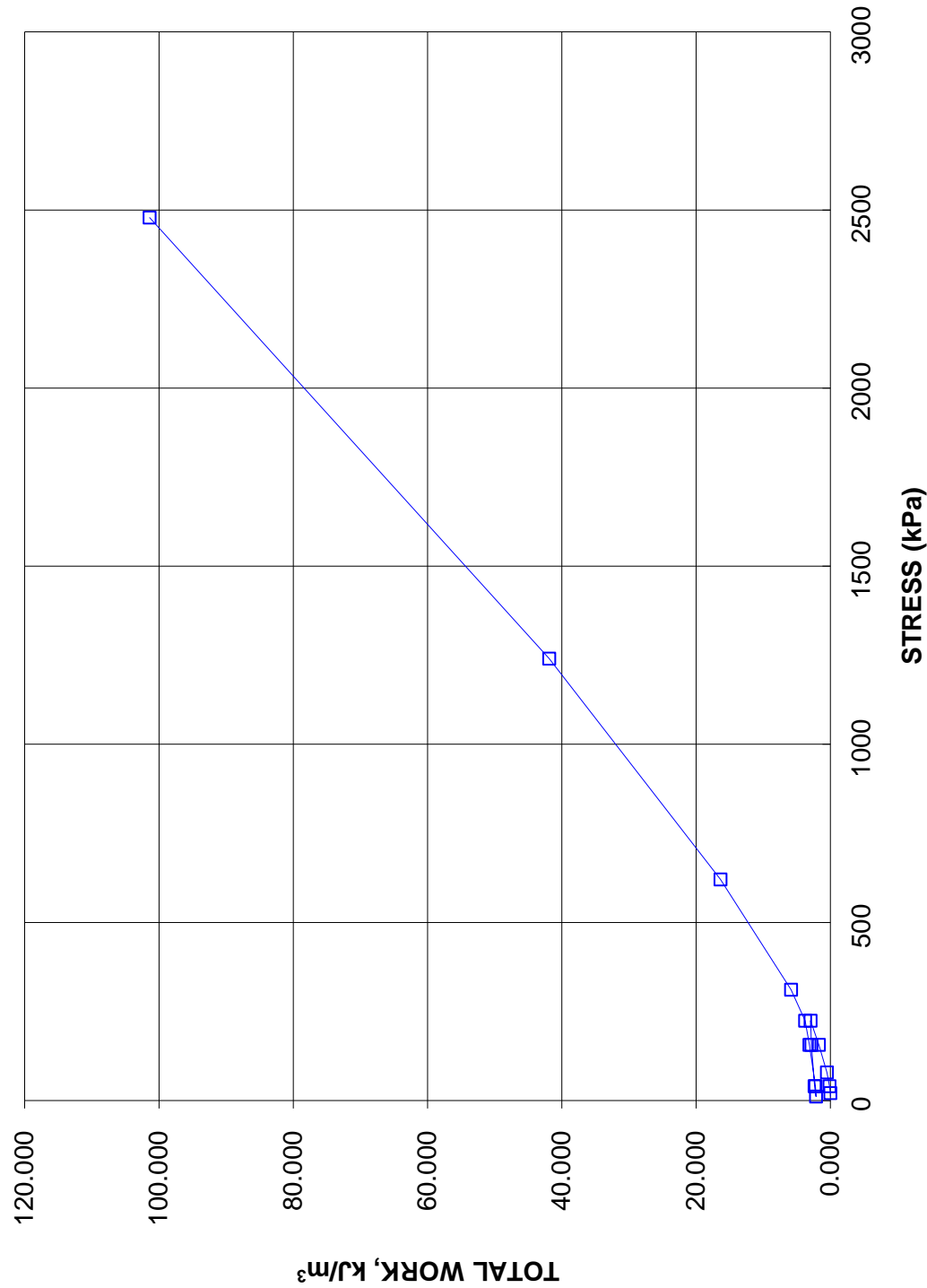
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 402 SA 15

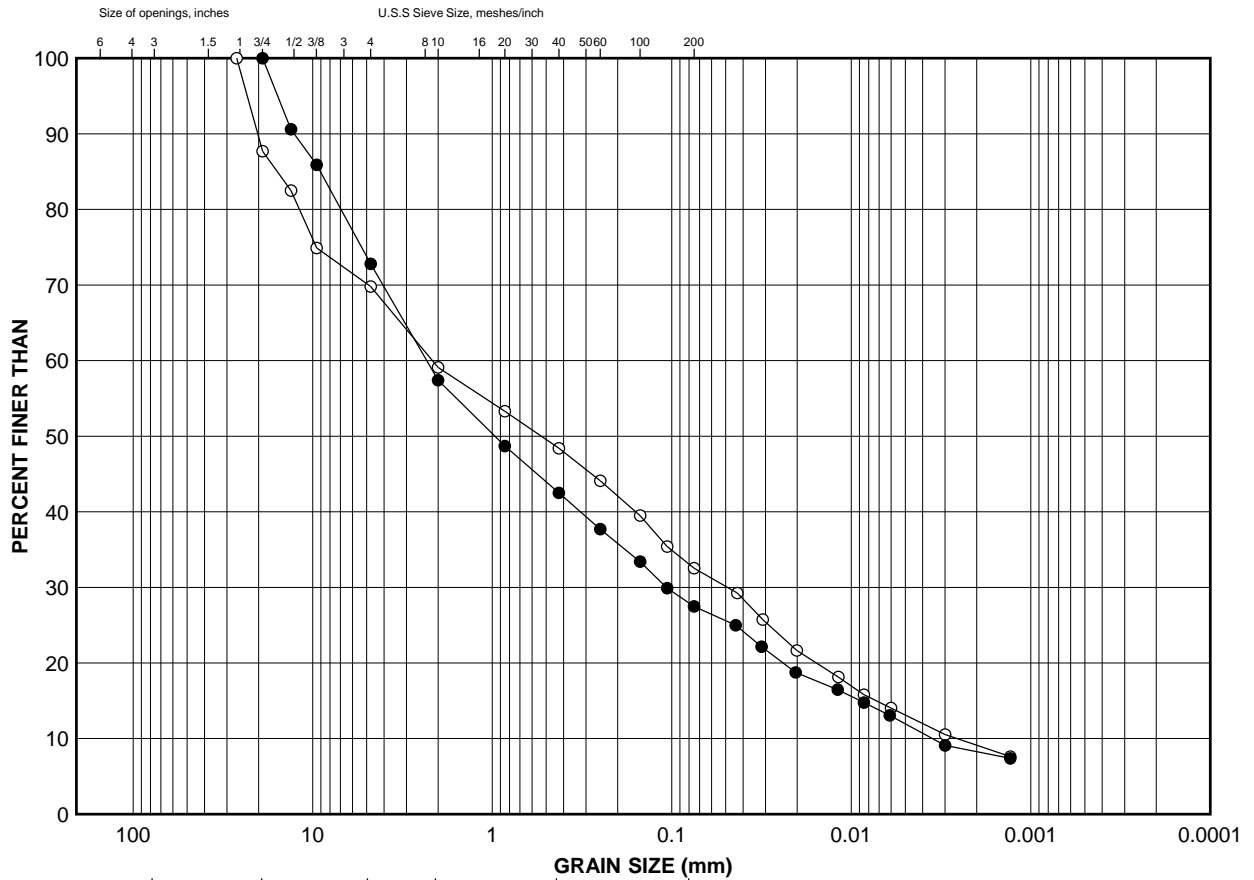


CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B-4.4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH 402 SA 15





APPENDIX C

Analytical Testing Laboratory Testing

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

ATTENTION TO: MIKE BEADLE

PROJECT: 19124560

AGAT WORK ORDER: 20L629663

SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer

DATE REPORTED: Aug 04, 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.



Certificate of Analysis

AGAT WORK ORDER: 20L629663

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 Chatham/ Kent

ATTENTION TO: MIKE BEADLE

SAMPLED BY: M. Rhody

Corrosivity Package

DATE RECEIVED: 2020-07-24

DATE REPORTED: 2020-08-04

		SAMPLE DESCRIPTION:		BH 502 SA 4	BH 501 SA 5	BH 402 SA 4	BH 202 SA 4	BH 401 SA 4	
		SAMPLE TYPE:		Soil	Soil	Soil	Soil	Soil	
		DATE SAMPLED:		2020-07-23	2020-07-22	2020-07-15	2020-07-17	2020-07-13	
Parameter	Unit	G / S	RDL	1297973	1297980	1297981	1297983	RDL	1297984
Chloride (2:1)	µg/g	2	257	221	162	215	4	591	
Sulphate (2:1)	µg/g	2	121	216	67	164	4	164	
pH (2:1)	pH Units	NA	7.47	7.71	7.95	7.81	NA	7.72	
Electrical Conductivity (2:1)	mS/cm	0.005	0.656	0.696	0.438	0.633	0.005	1.35	
Resistivity (2:1) (Calculated)	ohm.cm	1	1520	1440	2280	1580	1	741	
Redox Potential 1	mV	NA	126	72	66	46	NA	55	
Redox Potential 2	mV	NA	122	68	69	38	NA	57	
Redox Potential 3	mV	NA	120	63	65	40	NA	53	

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

1297973-1297983 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.

1297984 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. Dilution required, RDL has been increased accordingly.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:



Mike Beadle

Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 19124560

SAMPLING SITE: 401 Chatham/ Kent

AGAT WORK ORDER: 20L629663

ATTENTION TO: MIKE BEADLE

SAMPLED BY: M. Rhody

Soil Analysis

RPT Date: Aug 04, 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)	1297973	1297973	257	257	0.0%	< 2	91%	70%	130%	104%	80%	120%	NA	70%	130%
Sulphate (2:1)	1297973	1297973	121	123	1.6%	< 2	94%	70%	130%	103%	80%	120%	99%	70%	130%
pH (2:1)	1297973	1297973	7.47	7.48	0.1%	NA	100%	90%	110%						
Electrical Conductivity (2:1)	1297973	1297973	0.656	0.646	1.5%	< 0.005	100%	80%	120%						
Redox Potential 1	1					NA	100%	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.

If the RPD value is NA, the results of the duplicates are under 5X the RDL and will not be calculated.

Matrix spike NA: Spike level < native concentration. Matrix spike acceptance limits do not apply and are not calculated.

Certified By:


Nivine Basily

Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 20L629663

PROJECT: 19124560

ATTENTION TO: MIKE BEADLE

SAMPLING SITE: 401 Chatham/ Kent

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

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Report Information:

Company: GOLDER
Contact: MIKE BEADLE
Address: 309 Exeter Road
519 652 0099 Fax: 519 652 6299
Phone: 519 652 6299
Reports to be sent to:
1. Email: mbeadle@golder.com
2. Email: mrhody@golder.com

Project Information:

Project: 19124560 (2004)
Site Location: 401 Chatham / Kent
Sampled By: M. Rhody
AGAT Quote #: PO:
Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Bill To Same: Yes ☒ No ☐
Company:
Contact:
Address:
Email:

Regulatory Requirements:

(Please check all applicable boxes)

☐ Regulation 153/04 ☐ Sewer Use ☐ Regulation 558
☐ Table ☐ Sanitary ☐ CCME
☐ Ind/Com ☐ Storm ☐ Prov. Water Quality
☐ Res/Park ☐ Agriculture ☐ Objectives (PWQO)
☐ Soil Texture (Check One) ☐ Other
☐ Coarse ☐ MISA ☐ Region ☐ Indicate One
☐ Fine ☐ Indicate One

Is this submission for a Record of Site Condition?

☐ Yes ☐ No

Report Guideline on Certificate of Analysis

☐ Yes ☐ No

Sample Matrix Legend

B Biota
GW Ground Water
O Oil
P Paint
S Soil
SD Sediment
SW Surface Water

Field Filtered - Metals, Hg, CrVI

O. Reg 153

Metals and Inorganics	Full Metals Scan	Regulation/Custom Metals	Nutrients: <input type="checkbox"/> TP <input type="checkbox"/> NH ₃ <input type="checkbox"/> TKN <input type="checkbox"/> NO ₃ <input type="checkbox"/> NO ₂ <input type="checkbox"/> NO ₃ +NO ₂	Volatiles: <input type="checkbox"/> VOC <input type="checkbox"/> BTEX <input type="checkbox"/> THM	PHCs F1 - F4	ABNs	PAHs	PCBs: <input type="checkbox"/> Total <input type="checkbox"/> Aroclors	Organochlorine Pesticides	TCLP: <input type="checkbox"/> M&I <input type="checkbox"/> VOCs <input type="checkbox"/> ABNs <input type="checkbox"/> B(a)P <input type="checkbox"/> PCBs	Sewer Use	Potentially Hazardous or High Concentration (Y/N)
<input type="checkbox"/> All Metals <input type="checkbox"/> 153 Metals (excl. Hydrides)	<input type="checkbox"/> Hydride Metals <input type="checkbox"/> 153 Metals (incl. Hydrides)	<input type="checkbox"/> ORPs: <input type="checkbox"/> B-HWS <input type="checkbox"/> Cl <input type="checkbox"/> CN <input type="checkbox"/> Cr ⁶⁺ <input type="checkbox"/> EC <input type="checkbox"/> FOC <input type="checkbox"/> Hg <input type="checkbox"/> pH <input type="checkbox"/> SAR										

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Y / N
BH 502 SA 4	July 23/20		1	SOIL	Corrosivity Package	
BH 501 SA 5	July 22/20		1	SOIL		
BH 402 SA 4	July 15/20		1	SOIL		
BH 202 SA 4	July 12/20		1	SOIL		
BH 401 SA 4	July 13/20		1	SOIL		

Samples Relinquished By (Print Name and Sign): <u>Math Rhody</u>	Date: <u>July 24</u> Time: <u>1255</u>	Samples Received By (Print Name and Sign): <u>J. Smith</u>	Date: <u>20/7/24</u> Time: <u>1:00</u>
Samples Relinquished By (Print Name and Sign): <u>Brendan G</u>	Date: <u>July 27/20</u> Time: <u>225</u>	Samples Received By (Print Name and Sign): <u> </u>	Date: <u> </u> Time: <u> </u>

Laboratory Use Only

Work Order #: 20L629663
Cooler Quantity: 1 sm
Arrival Temperatures: 9.7 9.3 9.2
53.52 4.8
Custody Seal Intact: ☐ Yes ☐ No ☐ N/A
Notes: ice pack

Turnaround Time (TAT) Required:

Regular TAT ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

OR Date Required (Rush Surcharges May Apply):

Please provide prior notification for rush TAT
*TAT is exclusive of weekends and statutory holidays

For 'Same Day' analysis, please contact your AGAT CPM



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