

REPORT

Foundation Investigation and Design Report

Leslie Street Overpass (Site No.37X-0208/B1)

*Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue,
Toronto, Ontario*

MTO G.W.P. 2130-01-00

Submitted to:

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1786302-LS Rev.0

October 1, 2021



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Records of Drillholes	N/E RS-2, N/E RS-3, N/E RS-4, N/E RS-5, N/E RS-6, N/E RS-8, N/E RS-9, L-3 and L-4
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Certificate of Analysis	Report# R5979847 Report# R6308766
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Geomechanica Inc.	Rock Laboratory Testing Results
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APPENDIX D: Non-Standard Special Provisions

NSSP	Drilled Shafts (Caisson Piles)
NSSP	Rigid Expanded Polystyrene Embankment Fill
NSSP	Obstructions
NSSP	Vibration Monitoring
NSSP	Piezometer Decommissioning
Special Provision	FOUN0003, Amendment to OPSS 902
Special Provision	109F57, Amendment to OPSS.PROV 903

PART A

**FOUNDATION INVESTIGATION REPORT
LESLIE STREET OVERPASS (SITE NO. 37X-0208/B1)
HIGHWAY 401 EASTBOUND COLLECTOR LANES FROM AVENUE ROAD
TO WARDEN AVENUE, TORONTO, ONTARIO
MTO GWP 2130-01-00**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the rehabilitation of the Highway 401 Eastbound Collector lanes between Avenue Road and Warden Avenue (approximately 10 km) in City of Toronto, Ontario (MTO Assignment No. 2016-E-0089).

This report presents the subsurface conditions for the proposed eastbound Highway 401 collector lane bridge over Leslie Street (Site No. 37X-0208/B1) and associated approach embankments. This report was developed based on the results from Golder's current foundation investigation, and 2015/2016 investigations and testing completed by others (MTO's GEOCREs No. 30M14-460 and 30M14-463).

The results of foundation investigations for other works associated with this assignment are presented in separate reports.

2.0 SITE DESCRIPTION

The existing bridges at the Highway 401/Leslie Street interchange consist of 16 spans with a total length of 270 m that carry Highway 401 over the Metrolinx rail corridor, Oriole GO Station and Leslie Street in Toronto, Ontario. In general, the highway grade ranges from about Elevation 150 m to 142 m, generally declining toward the east, while the existing ground surface below the highway varies between about Elevation 142 m and 136 m, also generally declining to the east toward the East Don River valley.

The land use within the four quadrants of the bridge is characterized by a City of Toronto Municipal Yard and a large retail store to the northwest, North York General Hospital to the northeast, a residential neighborhood to the southwest, and industrial complexes and a wooded area associated with East Don River and Betty Sutherland Trail to the southeast.

3.0 INVESTIGATION PROCEDURES

3.1 2015/2016 Investigations (GEOCREs No. 30M14-460 and 30M14-463)

Between April 2015 and November 2016, two foundation investigations were carried out by Thurber Engineering Ltd. (Thurber) during which time a total of 15 boreholes were advanced for the proposed watermain relocation and widening and rehabilitation of the existing overpass structure. The results of the Thurber investigations are contained in the following reports:

- "Foundation Investigation and Design Report, Proposed 600 mm Watermain Relocation, Highway 401 Leslie Street Interchange, Toronto, Ontario, G.W.P. 2061-13-00," dated March 1, 2017 (GEOCREs No. 30M14-460), prepared by Thurber Engineering Ltd.
- "Foundation Investigation and Design Report, Highway 401 Overpass at GO Station Parking Lot and Leslie Street, Highway 401 and Leslie Street Interchange, Toronto, Ontario, W.P. 2061-13-00, Site 37-206/1-4", dated September 11, 2017 (GEOCREs No. 30M14-463), prepared by Thurber Engineering Ltd.

While these reports do not reference the coordinate system of the borehole locations, it is inferred that they are referenced to the MTM NAD 83 (Zone 10) coordinate system based on the plotted position relative to that reference system. The locations of the relevant boreholes are summarized below along with the geographic coordinates, ground surface elevation (referenced to Geodetic Datum), and the drilled depths of the boreholes. The boreholes are shown on Drawing 1 and the borehole records and figures showing the relevant laboratory test results are presented in Appendix A.

Borehole No.	GEOCRES No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Depth of Borehole (m)
		Northing (Latitude, °)	Easting (Longitude, °)		
WM16-07	30M14-460	4,847,321.4 (43.765778)	315,822.2 (-79.363104)	141.5	18.9
M-04	30M14-463	4,847,328.9 (43.765845)	315,910.0 (-79.362013)	140.2	30.7

3.2 Current Investigation

The foundation investigation for the proposed Leslie Street Overpass was carried out between June 19, 2019 and August 17, 2020, during which time a total of thirteen boreholes and one Cone Penetration Test (CPT) were advanced in the vicinity of the proposed foundation elements and approach embankments. The locations of the boreholes / CPTs are shown on Drawing 1 and the borehole/drillhole and CPT records are provided in Appendix B.

The foundation investigation was completed using the following drilling equipment:

- D120 truck-mounted drill rig supplied and operated by Altech-Drilling and Investigative Services of Cambridge, Ontario;
- CME-55 and CME-75 truck and track-mounted drilling rigs supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, Ontario.

Coring through the existing reinforced concrete in the composite pavement structure was necessary at Borehole L-5, which was completed by Canadian Cutting and Coring Limited, of Brampton, Ontario, using a 254 mm outside diameter (O.D.) core bit.

Soil samples from boreholes were generally obtained at 0.75 m, 1.5 m and 3.0 m intervals of depth, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions. Samples of the cohesive soils were obtained at selected locations using 76 mm O.D. thin-walled 'Shelby' tubes for relatively undisturbed samples. Field vane shear tests were carried out in cohesive soils for assessment of undrained shear strengths using MTO Standard 'N' size vanes. Core samples of the bedrock were obtained using an 'HQ'-size rock core barrel and coring techniques in Boreholes L-3, L-4, N/E RS-2 to N/E RS-6, N/E RS-8 and N/E RS-9. A Cone Penetration Test was carried out at N/E RS-11 using the hydraulic ram system on the drill rig.

Groundwater conditions and water levels in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Borehole N/E RS-4 to allow monitoring of the water level. The installed piezometer consists of a 50 mm diameter PVC pipe with a slotted screen sealed within a selected depth within the borehole. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was then backfilled with bentonite to or near the ground surface. Details of the piezometer installation and water level readings are presented on the borehole records in Appendix B. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended), and the ground surface was restored to near original condition as practical using cold-patch asphalt and quick-set concrete, as applicable.

The field work was observed by members of Golder's engineering and technical staff, who marked the borehole/CPT locations, arranged for the clearance of underground utilities, observed the drilling, sampling and in-situ testing operations, and logged the boreholes and cone penetration testing. The soil samples and bedrock cores were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples and cores underwent further visual examination and laboratory testing in accordance with MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples, and a one-dimensional consolidation (oedometer) test was carried out on two samples of the cohesive deposit. Unconfined compression (UC) tests (including assessment of Young's modulus) were carried out on selected specimens of the bedrock core samples by Geomechanica Inc. on behalf of Golder. The results of the laboratory testing on the soil and rock core samples from the current investigation are included in Appendix C.

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

The as-drilled borehole locations and the ground surface elevations were obtained using either a GPS Trimble GEO 7X, having an accuracy of approximately 0.1 m in the vertical and 0.1 m in the horizontal directions, or were measured relative to identifiable site features and superimposed on the base plan. The locations given on the borehole/drillhole and CPT records and shown on Drawing 1 are positioned related to MTM NAD 83 (Zone 10) CSRS CGVD28 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole/CPT locations, geographic coordinates, ground surface elevations and depths advanced prior to termination are summarized below.

Borehole / CPT No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole / CPT Hole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
L-1	4,847,343.6 (43.765978)	315,808.5 (-79.363274)	140.8	11.3
L-3	4,847,333.8 (43.765890)	315,861.9 (-79.362610)	136.4	34.1

Borehole / CPT No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole / CPT Hole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
L-4	4,847,334.1 (43.765892)	315,874.5 (-79.362454)	136.8	35.9
L-5	4,847,342.3 (43.765966)	315,897.7 (-79.362166)	142.6	39.6
L-6	4,847,343.6 (43.765977)	315,924.7 (-79.361830)	141.7	15.9
N/E RS-2	4,847,306.9 (43.765648)	315,815.9 (-79.363182)	142.0	41.3
N/E RS-3	4,847,321.1 (43.765776)	315,824.7 (-79.363072)	141.2	41.3
N/E RS-4	4,847,310.3 (43.765679)	315,832.8 (-79.362973)	139.3	40.0
N/E RS-5	4,847,323.0 (43.765793)	315,832.6 (-79.362975)	140.3	40.3
N/E RS-6	4,847,312.5 (43.765698)	315,841.2 (-79.362868)	138.8	40.0
N/E RS-8	4,847,313.3 (43.765705)	315,874.3 (-79.362457)	136.8	35.1
N/E RS-9	4,847,316.4 (43.765733)	315,884.2 (-79.362333)	137.3	35.0
N/E RS-10	4,847,313.6 (43.765707)	315,912.3 (-79.361985)	137.3	35.1
N/E RS-11 ¹	4,847,322.2 (43.765784)	315,922.6 (-79.361857)	138.4	17.5

NOTE:

1. CPT Location

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The area surrounding the Highway 401 / Leslie Street interchange is within the physiographic region known as the South Slope, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)¹ and *Urban Geology of Canadian Cities* (Menzies and Taylor, 1998)².

The South Slope physiographic region is characterized by a smooth to drumlinized till plain that was formed as a result of glacial action and deposition of till material south of the Oak Ridges Moraine. The South Slope contains a variety of soil deposits that have developed over till and the overburden soils can typically be more than 50 m thick. The underlying bedrock consists of grey shale of the Georgian Bay Formation interbedded with limestone, siltstone and sandstone. Within and adjacent to the East Don River, interglacial and post-glacial flooding in the valley has produced deposits of glaciolacustrine sands, silts, and silty clay.

4.2 Subsurface Conditions

The borehole/drillhole records and laboratory testing summary figures from the previous investigations are presented in Appendix A. The borehole/drillhole records from the current investigation are presented in Appendix B, while the geotechnical and analytical laboratory test results from the current investigation are presented in Appendix C.

The results of in-situ tests (i.e., SPT) as presented in the borehole records and in Section 4.2 are uncorrected. The boundaries between the soil deposits on the borehole records have been inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface soils encountered consist of surficial layers of topsoil or composite pavement structure, underlain by cohesive and non-cohesive fill. The fill is then underlain by a deposit of silt to sand, which is in turn underlain by a deposit of clay to clayey silt-silt. Underlying the cohesive deposit is a deposit of silt to sand, followed by a deposit of glacial till. The till deposit is in turn underlain by another deposit of sand, followed by a deposit of sandy clayey silt to clayey silt-silt. Organic silt underlies the sandy clayey silt to clayey silt-silt deposit, which is in turn underlain by residual soil and shale bedrock. Detailed descriptions of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

An approximately 50 mm to 400 mm thick layer of topsoil was encountered at ground surface in Boreholes M-04, WM16-07, N/E RS-2 to N/E RS-6, and N/E RS-10.

It should be noted that materials designated as topsoil were classified solely based on visual and textural evidence. Testing for organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

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

² Menzies, J., and Taylor, E.M., 1998. *Urban Geology of St. Catharines-Niagara Falls, Region Niagara*. In *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White.

4.2.2 Asphalt and Concrete Pavement Structure

An approximately 25 mm to 180 mm thick layer of asphalt was encountered at the ground surface in Boreholes L-3 to L-6, N/E RS-8 and N/E RS-9. Boreholes L-3, L-4, N/E RS-8 and N/E RS-9 were advanced through Leslie Street, and Boreholes L-5 and L-6 were advanced at Highway 401 grade.

Approximately 25 mm of gravel was encountered underlying the asphalt in Borehole N/E RS-9.

An approximately 180 mm to 320 mm thick layer of concrete was encountered underlying the asphalt in Boreholes L-3 to L-5 and N/E RS-8, and was encountered immediately below the gravel in Borehole N/E RS-9. Photographic log of the concrete core recovered from Borehole L-5 is presented on Figure B1 Appendix B.

4.2.3 Sandy SILT (ML) to SILTY SAND (SM) to SAND (SP-SM) to Gravelly SAND (SP) (FILL)

A 0.4 m to 4.6 m thick layer of non-cohesive fill consisting of sandy silt to gravelly sand, trace gravel and containing rootlets, wood chips, asphalt and brick fragments, was encountered underlying the topsoil in Boreholes N/E RS-3, N/E RS-10, M-04 and WM16-07, and underlying the composite pavement structure in Boreholes N/E RS-8, N/E RS-9, and L-3 to L-6. The top of the granular fill was encountered at depths ranging from 0.1 m to 1.7 m below ground surface (between Elevations 142.3 m and 136.0 m) and extended to depths ranging from 0.7 m to 4.7 m below ground surface (between Elevations 139.8 m and 134.3 m).

The SPT “N”-values measured within the non-cohesive fill ranges from 5 blows to 45 blows per 0.3 m of penetration, indicating a loose to dense state of compactness.

Grain size distribution testing was carried out on three samples of the granular fill. The results of two grain size distribution tests are presented on Figures A1 and A5 in Appendix A, and one grain size distribution test is presented on Figure C1 in Appendix C. The water content measured on samples of the non-cohesive fill ranges from about 4% to 15%.

4.2.4 CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML/SC) and Sand (FILL)

A 0.4 m to 2.3 m thick layer of cohesive fill was encountered at ground surface in Borehole L-1, underlying the topsoil in Borehole N/E RS-2 and N/E RS-3, and underlying the of non-cohesive fill in Boreholes L-5. The cohesive fill consists of clayey silt to sandy clayey silt to clayey silt and sand to clayey silt-silt and sand, trace gravel, and containing rootlets. In Borehole N/E RS-3, a 0.8 m thick layer of silty sand, some gravel fill was encountered within the cohesive fill. The cohesive fill was encountered at depths ranging from 0 m to 3.4 m below ground surface (between Elevations 141.9 m and 139.3 m) and extends to depths ranging from 2.0 m to 4.5 m below ground surface (between Elevations 139.0 m and 138.1 m).

The SPT “N”-values measured within the cohesive fill ranges from 4 blows to 39 blows per 0.3 m of penetration, suggested a firm to hard consistency.

Grain size distribution testing was carried out on samples of the cohesive fill, and the results are presented on Figure C2 in Appendix C. Atterberg limits testing was carried out on two samples of the cohesive fill and measured a liquid limit of about 17% and 19% and a plastic limit of about 11%, corresponding to plastic indices of about 6% and 8%. The Atterberg limit test results are presented on Figure C3 in Appendix C and indicates that the fill ranges from a clayey silt and sand to a clayey silt-silt and sand of low plasticity. The water content measured on samples of the deposit ranges from about 8% to 12%.

4.2.5 SILT (ML) to Sandy SILT (ML) to SILTY SAND (SM) to SAND (SP)

A 3.5 m to 8.6 m thick deposit of silt to sandy silt to silty sand to sand, containing trace to some gravel, rootlets, and pockets of clayey silt, was encountered underlying the topsoil in Boreholes N/E RS-4 to N/E RS-6, and underlying the fill in Boreholes N/E RS-2, N/E RS-3, N/E RS-8, N/E RS-9, L-1, L-3 to L-6, WM16-07 and M-04. The silt to sand deposit was encountered at depths ranging from 0.1 m to 4.7 m below ground surface (between Elevations 140.2 m and 135.4 m) and extends to depths ranging from 4.7 m to 10.2 m below ground surface (between Elevations 133.9 m and 130.9 m).

The SPT “N”-values measured within the silt to sand deposit range from 0 blows (weight of hammer) to 103 blows per 0.3 m of penetration, indicating a very loose to very dense state of compactness.

Grain size distribution testing was carried out on samples of the silt to sand deposit, and the results are presented on Figures A2 and A6 in Appendix A, and on Figure C4A to C4C in Appendix C. Grain size distribution testing was carried out on a 0.8 m thick clayey silt interlayer encountered within the silty sand deposit at Borehole L-3, and the results are presented on Figure C5 in Appendix C. Atterberg limits testing was carried out on two samples of the deposit and determined that the samples are non-plastic. The water content measured on samples of the deposit range from about 4% to 32%.

4.2.6 CLAY (CH) to SILTY CLAY (CI) to Sandy CLAYEY SILT (CL) to CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

An up to 17.5 m thick deposit of clay to silty clay to sandy clayey silt to clayey silt to clayey silt-silt was encountered underlying the fill in Borehole N/E RS-10, and underlying the silt to sand deposit in Boreholes N/E RS-2 to N/E RS-6, N/E RS-8, N/E RS-9, L-1, L-3 to L-6, WM16-07, and M-04. The top of the cohesive deposit was encountered at depths ranging from 3.1 m to 10.2 m below ground surface (between Elevations 134.3 m and 130.9 m) and extends to depths ranging from 17.1 m to 26.2 m below ground surface (between Elevations 119.5 m and 114.1 m). Boreholes L-1, L-6 and WM16-07 were terminated within this deposit.

The SPT “N”-values measured within the cohesive deposit range from 0 blows (i.e. weight of rods and hammer) to 26 blows per 0.3 m of penetration. In-situ field vane tests carried out within the deposit measured undrained shear strengths ranging from about 17 kPa to greater than 96 kPa with sensitivity values generally between about 0.9 and 8.3. In consideration of the SPT “N”-values as well as the field vane test results, this deposit is considered to have a soft to very stiff consistency.

Grain size distribution testing was carried out on samples of the cohesive deposit and the results are shown on Figures A3 and A7 in Appendix A and Figures C6A to C6C in Appendix C. Atterberg limits testing was carried out on samples of the cohesive deposit and the results are presented on Figures A4 and A8 in Appendix A and Figures C7A and C7B in Appendix C. The Atterberg limits tests measured liquid limits ranging from about 18% to 51%, plastic limits ranging from about 11% to 21%, and plasticity indices ranging from about 4% to 30%, indicating that the material is a clay of high plasticity to clayey silt-silt of low plasticity. The water content was measured on samples of the cohesive deposit and ranges from about 14% to 43%.

Laboratory consolidation testing was carried out on two samples of the clay to sandy clayey silt-silt to silt deposit obtained from Shelby tube samples in Boreholes N/E RS-2 and N/E RS-5. Based on the results of the laboratory consolidation testing and the in-situ testing, the cohesive deposit is lightly over-consolidated with an over-consolidation ratio (OCR) between 1.4 and 2.4. Details of the consolidation test results are presented on Figures C8A to C8D and C9A to C9D, in Appendix C and the results are summarized below.

Borehole / Sample No.	Sample Depth / Elevation (m)	γ (kN/m ³)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_{vo}' - \sigma_p'$ (kPa)	OCR	C_c	C_r	e_0	C_v (cm ² /s)
N/E RS-2 / Sample 11	11.7 / 130.3	17	150	205	60	1.4	0.61	0.054	1.27	8.2×10^{-4}
N/E RS-5 / Sample 12	14.0 / 126.3	22	157	380	284	2.4	0.14	0.008	0.39	5.7×10^{-3}

Where:

- γ Is the bulk unit weight in kN/m³
- σ_{vo}' Is the effective overburden stress in kPa
- σ_p' Is the preconsolidation stress in kPa
- OCR Is the overconsolidation ratio
- C_c Is the compression index
- C_r Is the recompression index
- e_0 Is the initial void ratio
- C_v Is the coefficient of consolidation, valid for 150 kPa to 300 kPa vertical effective stress range

One Cone Penetration Test at N/E RS-11 was advanced from the bottom of the silt to sand deposit into the cohesive deposit for measurement of the tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The results of the CPT at N/E RS-11 is presented on the Cone Penetration Test sheet in Appendix B.

As part of the cone penetration testing, one in-situ pore water pressure dissipation test was carried out in the cohesive deposit to assess the coefficient of consolidation at a specific horizon within the clay to clayey silt-silt deposit. The result of the pore water dissipation test is shown on Figure C10, in Appendix C, and the coefficient of consolidation in the horizontal direction (c_h) obtained from the results of the dissipation test is summarized below.

CPT Hole No.	Depth / Elevation (m)	Coefficient of Horizontal Consolidation, c_h (cm ² /s)
N/E RS-11	13.0 / 125.4	5.8×10^{-3}

Silt Interlayer

In 3.2 m thick interlayer of silt trace clay and sand was encountered at the base of the cohesive deposit in Borehole M-04, at a depth of 23.1 m below ground surface (at Elevation 117.1 m); it extends to a depth of 26.3 m (Elevation 113.9 m).

The SPT "N"-values measured within the interlayer range from 21 blows to 57 blows per 0.3 m of penetration, indicating compact to very dense state of compactness.

Grain size distribution testing was carried out on samples of the deposit, and the results are presented on Figure A9 in Appendix A.

4.2.7 Glacial Till

A 2.2 m to 6.1 m thick glacial till deposit was encountered underlying the clay to clayey silt-silt deposit in Boreholes N/E RS-2, N/E RS-3, N/E RS-8, N/E RS-9, N/E RS-10, L-3, L-4, L-5, and M-04. The cohesive portions of the glacial till deposit ranges from silty clay to sandy clayey silt-silt to clayey sand to sandy silt of slight plasticity, trace to some gravel, while the non-cohesive portions of the glacial till deposit consists of silty sand, trace gravel. Although not

encountered in these boreholes, cobbles and boulders are commonly present within glacially derived soils and should be expected within this deposit.

The top of the till deposit was encountered at depths ranging from 17.1 m to 26.3 m below ground surface (between Elevations 119.5 m and 113.9 m) and extends to depths ranging from 22.0 m to 31.0 m below ground surface (between Elevations 116.1 m and 109.5 m). Borehole M-04 was terminated within this deposit.

The SPT “N”-values measured within the cohesive portions of the till deposit range from 27 blows per 0.3 m of penetration to 100 blows per 0.2 m of penetration, suggesting a very stiff to hard consistency. The SPT “N”-value measured within the non-cohesive portion of the till deposit is 27 blows per 0.3 m of penetration, indicating compact state of compactness.

Grain size distribution testing was carried out on samples of the cohesive portions of the till deposit, and the results are presented on Figure A10 in Appendix A and on Figures C11A and C11B in Appendix C. Atterberg limits testing was carried out on samples of the cohesive portions of the till deposit, and the results are presented on Figure A11 in Appendix A and on Figure C12 in Appendix C. The Atterberg limit tests measured liquid limits ranging from about 17% to 26%, plastic limits ranging from about 10% to 18%, and plasticity indices ranging from about 3% to 13%, indicating that the material is a clayey silt of low plasticity to a silt of slight plasticity. The water content measured on samples of the till deposit range from about 9% to 20%.

4.2.8 SILT (ML) to SILTY SAND (SM) to SAND (SP-SM)

A 4.4 m to 12.5 m thick deposit of silt to silty sand to sand was encountered underlying clay to clayey silt-silt deposit in Boreholes N/E RS-4 to N/E RS-6, and below the glacial till in Boreholes N/E RS-3, N/E RS-8 to N/E RS-10, and L-3 to L-5. The top of the silt to sand deposit was encountered at depths ranging from 19.8 m to 31.0 m below ground surface (between Elevations 119.5 m and 111.1 m) and extends to depths ranging from 29.2 m to 35.4 m below ground surface (between Elevations 107.8 m and 105.1 m).

In general, the SPT “N”-values measured within the silt to sand deposit range from 14 blows per 0.3 m of penetration to 100 blows per 0.2 m of penetration, indicating a compact to very dense compactness condition. However, SPT “N”-values ranging 1 blow to 6 blows per 0.3 m penetration were measured in three samples, indicating the presence of limited zones in which the deposit is very loose to compact.

Grain size distribution testing was carried out on samples of the deposit, and the results are presented on Figures C13A and C13B in Appendix C. The water content measured on samples of this deposit range from about 12% to 23%.

4.2.9 Sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

A 1.5 m to 3.6 m thick deposit of sandy clayey silt containing trace sand was encountered underlying the silt to sand deposit in Boreholes N/E RS-2, N/E RS-5 and N/E RS-6. The sandy clayey silt to clayey silt-silt deposit was encountered at depths ranging from 31.0 m to 35.1 m below ground surface (between Elevations 107.8 m and 106.7 m) and extends to depths ranging from 34.6 m to 36.8 m below ground surface (between Elevations 105.3 m and 104.2 m).

The SPT “N”-values measured within the sandy clayey silt to clayey silt-silt deposit ranges from 23 blows to 33 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

Grain size distribution testing was carried out on a sample of the deposit and the results are presented on Figure C14 in Appendix C. Atterberg limits testing was carried out on two samples of the sandy clayey silt to clayey silt-silt deposit and the results are presented on Figure C15 in Appendix C. The Atterberg limit tests measured

liquid limits of about 25% and 31%, plastic limits of about 18% and 19%, and plasticity indices of about 7% and 12%, indicating that the material is clayey silt to clayey silt-silt of low plasticity. The water content measured on samples of this cohesive deposit ranges from about 17% to 23%.

4.2.10 Organic SILT (OH) to Sandy Organic SILT (OH)

A 1.6 m to 2.2 m deposit of organic silt to sandy organic silt was encountered overlying the bedrock in Boreholes N/E RS-3, N/E RS-5 and N/E RS-6. The top of the organic deposit was encountered at depths ranging from 34.6 m to 35.4 m below ground surface (between Elevations 105.8 m and 104.2 m) and extends to depths ranging from 36.2 m to 37.6 m below ground surface (between Elevations 103.6 m and 102.6 m).

The SPT “N”-values measured within the organic deposit range from 39 blows to 71 blows for 0.3 m of penetration, suggesting a hard consistency.

Grain size distribution testing was carried out on a sample of the sandy organic silt and the results are presented on Figure C16 in Appendix C. Atterberg limits testing was carried out on two samples of the organic deposit, and the results are presented on Figure C17 in Appendix C. The Atterberg limit tests measured liquid limits of about 54% and 59%, plastic limits of about 41% and 44%, and plasticity indices of about 14% and 16%, indicating that the material is organic silt of high plasticity. The water content measured on samples of the organic deposit measured about 39% and 50%. The organic content measured on three samples of the organic silt deposit ranges between about 8% to 14%.

4.2.11 Residual Soil

Residual soil was encountered overlying the bedrock in Boreholes N/E RS-2, N/E RS-3, N/E RS-6, N/E RS-8, N/E RS-9, N/E RS-10, and L-3 to L-5.

The thickness of the residual soil ranges between approximately 0.1 m and 3.0 m and consists of gravelly clayey silt to sandy clayey silt, trace gravel and containing shale fragments. The residual soil was encountered at depths ranging from 29.2 m to 37.6 m below ground surface (between Elevations 107.6 m and 102.6 m) and extends to depths ranging from 30.9 m to 38.4 m below ground surface (between Elevations 105.5 m and 102.2 m). Borehole N/E RS-10 was terminated within the residual soil.

The SPT “N”-values measured within the residual soil deposit ranges from 65 blows per 0.3 m of penetration to 100 blows per 0.03 m of penetration, suggesting a hard consistency.

Atterberg limits testing was carried out on a sample of the residual soil deposit, and the results are presented on Figure C18 in Appendix C. The Atterberg limit tests measured a liquid limit of about 27%, a plastic limit of about 15%, and a plastic index of about 12%, indicating that the material is a clayey silt of low plasticity. The water content measured on samples of the residual soil ranges from about 13% to 16%.

4.2.12 Bedrock

Bedrock was encountered in Boreholes L-3, L-4, L-5, N/E RS-2, N/E RS-3, N/E RS-4, N/E RS-5, N/E RS-6, N/E RS-8 and N/E RS-9. Core samples were recovered in all boreholes that encountered bedrock, except for Borehole L-5, where the bedrock surface was inferred from split-spoon sampling. The depth to bedrock below surface, the corresponding surface elevation, and the core depths are summarized below.

Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
N/E RS-2	37.1	104.9	0.1 m of auger penetration. Bedrock cored 4.1 m
N/E RS-3	38.0	103.2	Bedrock cored 3.3 m
N/E RS-4	32.3	107.0	4.4 m of auger penetration. Bedrock cored 3.2 m
N/E RS-5	36.9	103.4	0.4 m of auger penetration. Bedrock cored 3.4 m
N/E RS-6	36.4	102.4	Bedrock cored 3.6 m
N/E RS-8	32.3	104.5	Bedrock cored 2.8 m
N/E RS-9	32.3	105.0	Bedrock cored 2.7 m
L-3	30.9	105.5	Bedrock cored 3.2 m
L-4	32.1	104.7	Bedrock cored 3.8 m
L-5	38.4	104.2	1.2 m of auger penetration

In general, the bedrock surface as encountered or inferred is relatively flat and consists of shale of the Georgian Bay formation. The bedrock samples are described as completely weathered to fresh, thinly bedded, fine grained, non-porous to slightly porous, very weak to medium strong, grey shale with limestone/siltstone interbeds at varying intervals of depth. The limestone/siltstone layers range from about 10 mm to 140 mm in thickness. The bedrock details are presented in on the drillhole records and photographs of the recovered bedrock core samples on Figures B2 to B10 in Appendix B. The degree of weathering of the bedrock samples (i.e. fresh to completely weathered – W1 to W5), and the strength classification of the intact rock mass based on field identification (i.e. very weak to strong – R1 to R4) are described in accordance with the International Society for Rock Mechanics (ISRM)³ standard classification system. The surface and upper portion of the shale bedrock generally had a higher degree of weathering as shown on the drillhole records.

The Rock Quality Designation (RQD) measured on the core samples obtained from the current investigation generally range from about 20% to 100%, indicating a rock mass of very poor to excellent quality, as per Table 3.10 of CFEM (2006)⁴. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) range between 69% and 100% and between 38% and 100%, respectively.

³ International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

⁴ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

Unconfined Compression (UC) Tests were carried out on selected core samples of the shale bedrock. The uniaxial compressive strength (UCS), bulk density and tangent Young's modulus of the intact samples are summarized below for cores retrieved from the current investigation, and the details are presented on the Rock Laboratory Test Results report from Geomechanica in Appendix C.

Borehole	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm ³)	Tangent Young's Modulus (GPa)
L-3	32.50 – 32.71	103.90 – 103.69	9.1	2.552	0.3
N/E RS-2	40.62 – 40.79	101.38 – 101.21	47.4	2.615	7.8
N/E RS-4	39.35 – 39.51	99.95 – 99.79	21.9	2.611	2.6
N/E RS-6	37.59 – 37.69	101.21 – 101.11	24.7	2.618	5.1
N/E RS-9	33.01 – 33.14	104.29 – 104.16	7.7	2.569	0.3

Based on the results from the UCS, the shale bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa) to medium strong (R3, 25 MPa < UCS < 50 MPa) in accordance with Table 3.5 in CFEM (2016). Although not tested in this investigation, the limestone/siltstone interlayers are generally medium strong to strong, with UCS values typically greater than 50 MPa.

4.3 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling are presented on the record of borehole. A total of two standpipe piezometers were installed: one during the previous investigation and one during the current investigation.

Borehole I.D.	Screened Stratigraphy	Water Level		Date of Measurement
		Depth (m)	Elevation (m)	
N/E RS-4	Sand	1.0	138.3	July 16, 2019
		0.9	138.4	August 21, 2019
		0.9	138.4	October 25, 2019
WM16-07	Sand	3.0	138.5	November 9, 2016
		4.5	137.0	December 21, 2016
		2.8	138.7	August 21, 2019
		3.7	137.8	October 25, 2019
		5.4	136.1	September 14, 2020

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

4.4 Analytical Testing

Four samples were collected and submitted to Bureau Veritas Laboratories for analysis of parameters used to assess corrosion potential and sulphate attack. A summary of the results is presented in the following table. The Certificates of Analysis are provided in Appendix C.

Borehole Number	Sample	Sample Depth (Elevation) (m)	Soil Type	Parameters				
				Chloride (µg/g)	Sulphate (µg/g)	pH	Conductivity (µmho/cm)	Resistivity (ohm-cm)
N/E RS-2	4	139.6 – 139.0	Silty Clay to Sand (Fill)	220	<20	7.75	528	1,900
N/E RS-3	5	138.1 – 137.5	Sand	<20	<20	7.86	130	7,700
N/E RS-6	7	132.7 – 132.1	Sand	400	100	7.90	829	1,200
N/E RS-9	4	135.0 – 134.4	Silt	1,700	42	7.92	2,650	380
L-3	3	134.8 – 134.3	Silty Sand	850	62	8.14	1,510	660
L-5	7	136.5 – 135.9	Silty Sand	230	<20	8.07	402	2,500

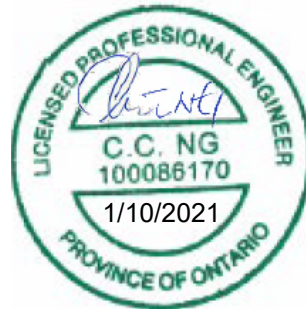
5.0 CLOSURE

The Foundation Investigation Report was prepared by Ms. Katelyn Nero, P.Eng., and reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate with Golder. Ms. Lisa Coyne, P.Eng., a Principal and MTO Foundations Designated Contact with Golder, conducted an independent technical and quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT

LESLIE STREET OVERPASS (SITE NO. 37X-0208/B1)

HIGHWAY 401 EASTBOUND COLLECTOR LANES FROM AVENUE ROAD
TO WARDEN AVENUE, TORONTO, ONTARIO

MTO GWP 2130-01-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for detail design of the proposed Leslie Street Overpass (Site No. 37X-0207/B1) as part of the widening of Highway 401 Eastbound Collector lanes. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the previous and current field investigations. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and carry out the detail design of the bridge foundations. The Foundation Design Report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Contractors must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Based on the General Arrangement drawing provided by AECOM, dated July 29, 2021, the proposed Leslie Street Overpass will be a four-span structure located immediately south of the existing bridges that carry Highway 401 over Leslie Street. Based on extensive design iterations between the structural and geotechnical team, the Leslie Street Overpass is to be designed as an integral abutment, integral pier structural system with piles supporting the abutments and drilled shafts supporting the piers. EPS is to be used behind the abutments and that CSP be installed surround the piles at the east abutment. The proposed Leslie Street Overpass will have a total span length of 63 m and a width of 26.5 m and 22.3 m at the west and east abutments, respectively, which includes three lanes of eastbound collector traffic, plus the speed change lane associated with the N-E Ramp. The proposed grade at the west and east abutments will be at approximately Elevations 145.4 m and 143.6 m, respectively, matching the existing highway grade. The existing ground surface at Leslie Street is at approximately Elevation 138 m.

The existing bridges consist of sixteen spans with a total span length of 270 m, supported on a combination of 762 mm (30 in.) and 1219 mm (48 in.) diameter drilled shafts (caissons), and HP 310x79 (12BP53), and HP 360x108 (14BP73) driven piles. The design, load tests, and issues related to the installation of deep foundations of the existing bridges were documented in various reports, technical memoranda, and drawings in MTO's GEOCRE system. An overview of the geotechnical events was summarized in a report titled "Foundation Engineering Assessment Report, Highway 401 and Leslie Street Interchange, Toronto, Ontario, G.W.P. 2130-01-00" dated August 10, 2011 (GEOCRE No. 30M14-328), and is briefly presented below:

- In 1953, failure occurred during construction of the approach embankments of the Highway 401 Westbound Collector bridge (Site No. 37-206/2) and N-W Ramp bridge (Site No. 37-206/7), to heights greater than 9.8 m and 9.1 m respectively. Remedial design recommendations included the flattening of the embankment slopes to 2 horizontal to 1 vertical (2H:1V) or flatter, the addition of embankment buttresses (berms) and founding the bridge on H-piles.
- In 1965, it was noted that piles near the west abutment of the Eastbound Collector bridge and the W-N/S Ramp bridge (Site No. 37-206/5) were driven to 46 m below ground surface as compared to typical pile lengths of approximately 27 m.

- In 1966, two occurrences of blow-out were recorded during installation of drilled shafts in the area between the Metrolinx rail corridor and Leslie Street. As a result, drilled shafts were replaced by H-piles where blow-out occurred. Upon completion of an additional borehole investigation, subsequent drilled shafts were installed with a larger base diameter (from 762 mm to 1219 mm) and a higher founding elevation.
- In 1973, settlement was observed on caissons at two bents of the Westbound Collector bridge (Site 37-206/2). The caissons were replaced with approximately 22 m long, 220 mm diameter, concrete-filled steel tube piles, associated with underpinning and reconstruction of beams at these bents.
- In 1990, movement of the retaining walls at the west abutment of the Westbound Collector bridge (Site No. 37-206/2) and at the east abutment of the N-W Ramp Overhead (Site No. 37-206/7) were observed during construction and as such, lightweight slag fill was used as embankment fill in lieu of Granular 'A'.

Most recently as part of MTO Contract 2016-2048, it is understood that installation of drilled shafts for the nearby new W-N/S Ramp Overhead experienced issues related to flowing sand during casing advancement, and that polymer slurry was employed to mitigate further issues. This W-N/S Ramp structure, including the overhead structure, embankments and retaining walls, remains under construction at the time of preparation of this report.

6.2 Foundation Options

Based on the proposed structure configuration and the subsurface conditions encountered at this site, together with compatibility of the foundation types for the existing and future replacement bridges at the site, the following shallow and deep foundation options have been considered for support of the new abutments and piers. A summary of the advantages and disadvantages associated with each 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.

- **Spread/strip footings founded on native soils:** Due to the presence of the soft to firm zone of cohesive soils at this site, shallow foundations are not considered feasible as a result of low factored ultimate and serviceability geotechnical resistances, and the expected post-construction settlement of this deposit, in particular the differential between the existing and widened embankment. As such, spread/strip footings are not discussed further herein.
- **Driven steel H-piles:** Steel HP 310x110 piles driven to found on bedrock are suitable and feasible to support the proposed abutments and piers and would allow for integral abutment construction. Heavier piles may also be considered if required from a structural perspective.
- **Drilled shafts (caissons):** Drilled shafts founded within the dense to very dense / hard native soils or bedrock are also feasible for the support of the abutments and piers, although this option does not allow for integral abutment design. Drilled shafts can often accommodate a narrower footprint for construction in constrained working areas as compared with shallow foundations and driven piles (particularly where multiple rows and battered piles are employed), and can be extended directly to the underside of the superstructure at the piers, eliminating the need for foundation excavations to construct below-grade pile caps. If drilled shafts are adopted for support of the abutment and/or piers, temporary or permanent casings will be required and will need to be advanced with a water, bentonite, or polymer drilling slurry inside the casings. Drilled shafts would be more expensive than other foundation types; however, the higher costs per drilled shaft element could be offset by schedule and cost savings associated with minimizing the working footprint for traffic staging, and potentially

minimizing excavation and groundwater control if the below-grade pile caps can be eliminated at the piers. This option would not allow for integral abutment design if the abutments are supported on drilled shafts.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments on driven H-piles and the piers on drilled shafts, founded at the top of bedrock.

6.3 Design Consideration

6.3.1 Consequences and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC, 2019) and its *Commentary*, the proposed bridge and its foundation system are expected to carry high traffic volumes and its performance will have potential impacts on other transportation corridors; hence, the structure is classified as having a “typical consequence level” associated with exceeding limits states design.

In addition, given the project-specific foundation investigation carried out at this site (as presented in the Foundation Investigation Report (Part A of the report), in comparison to the degree of site understanding in Section 6.5 of 2019 CHBDC, the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC, 2019 have been used for design.

6.3.2 Seismic Design

6.3.2.1 Seismic Site Classification and Importance Category

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average penetration resistance, \bar{N}_{60} and/or soil average undrained shear strength, s_u within the upper 30 m of the soil layers 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.

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.

6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the 2019 CHBDC, the peak ground acceleration (PGA), peak ground velocity (PGV) and 5% damped spectral response acceleration ($S_a(T)$) values for Site Class C are presented below.

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.040	0.071	0.133
PGV (m/s)	0.031	0.051	0.089
$S_a(0.2)$ (g)	0.068	0.114	0.207
$S_a(0.5)$ (g)	0.043	0.066	0.112

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$S_a(1.0)$ (g)	0.024	0.036	0.058
$S_a(2.0)$ (g)	0.011	0.018	0.028
$S_a(5.0)$ (g)	0.002	0.004	0.007
$S_a(10.0)$ (g)	0.001	0.002	0.003

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 in Section 6.3.2.1 (Site Class D) to obtain design spectral values. As indicated in Section 4.4.3.3 of the CHBDC, the value of reference PGA , PGA_{ref} , for use with Tables 4.2 to 4.9 shall be taken as 80% of the PGA for Site Class C, where $S_a(0.2)/PGA$ is less than 2.0. Based on this requirement, a PGA_{ref} value of 0.106 was used for the 2,475-year return period. The corresponding site-specific Site Class D seismic hazard values, the peak ground acceleration (PGA), peak ground velocity (PGV) and design spectral response acceleration ($S(T)$), are presented below.

Seismic Hazard Values for Site Class D	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.052	0.092	0.170
PGV (m/s)	0.046	0.075	0.130
$aS(0.2)$ (g)	0.084	0.141	0.255
$S(0.5)$ (g)	0.063	0.097	0.163
$S(1.0)$ (g)	0.037	0.056	0.089
$S(2.0)$ (g)	0.017	0.028	0.044
$S(5.0)$ (g)	0.003	0.006	0.011
$S(10.0)$ (g)	0.001	0.003	0.004

In accordance with Table 4.10 of the 2019 CHBDC, the bridge structure (Importance Category of “Major-Route”), falls within Seismic Performance Category 1.

6.3.2.3 Potential for Liquefaction

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

conditions even catastrophic failure of slopes often referred to as “flow slides”. Lateral spreading and flow slide often accompany liquefaction along rivers and other shorelines.

In general, the soils at this bridge site consist of very loose to very dense silty sand to sand over firm to stiff silty clay to sandy clayey silt, in turn underlain by very stiff to hard glacial till and generally compact to very dense sand. Based on the compactness and consistency of the soils and the site-specific PGA, the soils at this site are considered to have a low potential for liquefaction during a seismic event.

6.4 Driven Steel H-Piles

Driven steel H-piles founded within the shale bedrock are considered feasible for support of the abutments and piers. Consideration must be given to the presence of cobbles and boulders within the glacially derived deposit at the site, as cobbles were inferred by auger grinding encountered in several boreholes at the site within the till deposits. It is recommended that the H-piles be reinforced with driving shoe as per OPSD 3000.100 (Steel H-Pile Driving Shoe) to reduce the potential for damage to the piles during driving into the very dense deposits.

6.4.1 Founding Elevation

Steel H-piles should be driven into the bedrock, the surface of which varies across the site. The following ranges in pile tip elevations may be used for design.

Foundation Element	Founding Stratum	Elevation at Underside of Pile Cap (m) ¹	Estimated Pile Tip Elevation (m)	Estimated Pile Length (m)
West Abutment	Shale Bedrock	138.5 ¹	102.2	36.3
Pier 1	Shale Bedrock	135.3 ²	102.2	33.1
Pier 2	Shale Bedrock	136.0 ²	104.3	31.7
Pier 3	Shale Bedrock	136.4 ²	104.7	31.7
East Abutment	Shale Bedrock	139.0 ¹	102.1	36.9

NOTES:

1. As per AECOM's General Arrangement Drawing dated July 2021
2. Assuming underside of pile cap is 1.2 m below ground surface

It is recommended that provision be made in the Contract for dealing with varying pile lengths due to the variability in the elevation of the bedrock. The depths indicated above should be considered minimum depths.

6.4.2 Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical resistances that may be used for the design of steel HP 310x110 piles are presented below.

Foundation Element	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) (kN)
West Abutment	102.2	Shale Bedrock	1,450	-- ¹
Pier 1	102.2	Shale Bedrock	1,450	-- ¹
Pier 2	104.3	Shale Bedrock	1,450	-- ¹
Pier 3	104.7	Shale Bedrock	1,450	-- ¹
East Abutment	102.1	Shale Bedrock	1,450	-- ¹

NOTE:

1. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and, as such, the SLS condition does not apply.

The estimated factored ultimate geotechnical resistance provided above are based on tip resistance. The following note should be included on the General Arrangement and Foundation Layout drawings:

- Piles to be driven to bedrock.

All pile installation/driving should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*) as amended by Special Provision 109F57.

6.5 Drilled Shafts (Caissons)

6.5.1 Founding Elevation

The following drilled shaft base elevations and strata may be used in the design, based on the lowest encountered bedrock elevation within each foundation element to achieve at least 1 m of penetration into the shale bedrock.

Foundation Element	Founding Stratum	Estimated Drilled Shaft Founding Elevation (m)
West Abutment	Shale Bedrock	101.2
Pier 1	Shale Bedrock	101.2
Pier 2	Shale Bedrock	103.3
Pier 3	Shale Bedrock	103.7
East Abutment	Shale Bedrock	101.1

6.5.2 Geotechnical Axial Resistances

The following provides the recommended factored ultimate geotechnical resistances for drilled shafts socketed approximately 1 m into the shale bedrock at the founding elevations given in Section 6.5.1.

Foundation Element	Drilled Shaft Diameter (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)
All Foundation Elements	0.9	Shale Bedrock	4,400	-- ¹
	1.2	Shale Bedrock	7,900	-- ¹
	1.5	Shale Bedrock	12,300	-- ¹

NOTE:

1. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and, as such, the SLS condition does not apply.

The estimated factored ultimate geotechnical resistances provided above are based on tip resistance. Drilled shaft foundations should be constructed in accordance with OPSS.PROV 903 (*Deep Foundations*), as amended by Special Provision 109F57.

Temporary casings should be used to support the overburden soils during construction to minimize disturbance to the side walls. The casing should be advanced while filled with polymer slurry to minimize the potential for non-cohesive materials ("flowing sands") to migrate into the drillhole, and to control base disturbance / basal heave due to groundwater pressures / seepage. It is expected that the casing would be installed using rotation methods or a vibratory hammer. If a vibratory hammer is used, vibration monitoring of the existing bridge structure, rail and pipelines is recommended. In addition, placement of concrete by tremie methods would be required.

Given that the above drilled shaft capacities have a significant end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the base of the drilled shaft. As such, the base of each drilled shaft excavation must be cleaned to remove all loose cuttings so that the concrete is in intimate contact with the subgrade soils or bedrock. A qualified geotechnical engineer should be retained during construction to inspect the drilled shafts to check that the conditions encountered are consistent with the information obtained from the boreholes and to confirm the base elevation of the drilled shaft and cleanliness. Further to the above discussion regarding the requirement for temporary casings to control the ground and groundwater, such casings are also required to allow for visual remote inspection of the base of the drilled shafts, which can be accomplished by means of a shaft inspection device (SID) such as a video camera or a shaft quantitative inspection device (SQUID). Should the inspection indicate that loosened material is present at the base of the drilled shafts, the base would need to be re-cleaned and re-inspected.

An NSSP has been provided in Appendix D to address the requirements for the use of temporary casings and slurry for the installation drilled shafts, the placement of concrete by tremie methods, and cleaning and inspection of the base of the drilled shafts, for inclusion into the Contract Documents.

6.6 Cladding Walls

It is understood that cladding walls are required along the north and south side of the west abutment of the Leslie Street Overpass. The proposed cladding walls are to be approximately 11.2 m in length, with a maximum height of approximately 7 m.

The footing for the cladding wall should be founded at a depth of 1.2 m below ground surface after the sub-excavation of all underlying topsoil and organic material, or below the depth of excavation for the installation of

EPS, whichever is deeper. Footings with widths of 1.5 m placed on a 500 mm thick compacted Granular 'A' pad could be designed based on factored ultimate and serviceability geotechnical resistances presented below.

Cladding Wall	Founding Stratum	Highest Founding Elevation (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
West Abutment	Loose to Compact Silt to Sand	Elevation 138.5, founded on 500 mm thick Granular 'A' pad compacted to 100% SMPDD	225	125

6.7 Frost Protection

All pile caps should be provided with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically from ground surface and perpendicular from the face of the abutment slope to the edge of the underside of the footing or pile cap. If adequate soil cover cannot be provided for the footing or pile cap, rigid Styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.8 Resistance to Lateral Loads

The design of piles and drilled shafts subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile/drilled shaft to the surrounding soil, the fixity condition at the head of the pile/drilled shaft (i.e., at the pile cap level), the structural capacity of the pile/drilled shaft to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile/drilled shaft and group effects. For longer, more flexible elements, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. Lateral loading could be resisted fully or partially using battered piles, where applicable.

Pressure-lateral displacement relationships (also known as P-y curves) have been developed for the assessment of resistance to lateral loads and are presented in Figures 1 to 5. The P-y curves were generated using the commercially available program RSPile (Version 3.0), developed by Rocscience Inc. The geotechnical engineering parameters and models used in the analysis are presented below.

Soil Unit	Elevation (m)	Unit Weight, γ (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Strain Factor, ϵ_{50}	Effective Angle of Internal Friction, ϕ (°)	Initial Modulus of Subgrade Reaction (kPa/m ³)	Soil Model
East Abutment							
Loose Sand in 1.5 m long CSP	139 – 137.5	19	--	--	28	6,800	API Sand, Above Water Table

Soil Unit	Elevation (m)	Unit Weight, γ (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Strain Factor, ε_{50}	Effective Angle of Internal Friction, ϕ (°)	Initial Modulus of Subgrade Reaction (kPa/m ³)	Soil Model
Loose to Compact Silty Sand to Gravelly Sand Fill	137.5 – 134.3	21	--	--	36	24,400	API Sand, Above Water Table
Firm to Stiff Clayey Silt-Silt to Clayey Silt	134.3 – 117.2	21	50	0.007	33	--	Soft Clay, Below Water Table
Very Stiff to Hard Silty Clay to Sandy Silt Till	117.2 – 111.1	21	--	--	37	34,000	API Sand, Below Water Table
Very Dense Sand (Residual Soil)	111.1 – 202.2	21	--	--	37	34,000	API Sand, Below Water Table
Piers							
Very Loose to Compact Silty Sand	Surface to 131.9	20	--	--	32	16,300	API Sand, Above Water Table
Soft to Stiff Sandy Clayey Silt	131.9 – 119.3	21	35	0.007	32	--	Modified Stiff Clay without Free Water
Very Stiff to Hard Silty Clay to Sandy Silt Till	119.3 – 113.2	21	--	--	36	34,000	API Sand, Below Water Table
Very Loose to Compact Sand	113.2 – 102.3	21	--	--	36	16,300	API Sand, Below Water Table
West Abutment							
Silty Sand to Sand	Surface to 132.1	20	--	--	32	16,300	API Sand, Below Water Table
Soft to Firm Clayey Silt to Silty Clay	132.1 – 119.5	21	35	0.007	32	--	Modified Stiff Clay without Free Water
Dense Silt	119.5 – 116.1	21	--	--	32	34,000	API Sand, Below Water Table
Compact to Very Dense Sand	116.1 – 107.0	21	--	--	36	34,000	API Sand, Below Water Table

Both the structural and geotechnical resistances of the piles or drilled shafts should be evaluated to establish the governing case at Ultimate Limit States (ULS). At Serviceability Limit States (SLS), the horizontal reaction of the piles or drilled shafts will be controlled by deflections, and the horizontal resistance of the piles or drilled shafts should be calculated based on the P-y curves. For this structure, the lateral resistance has been taken as that corresponding to a horizontal deflection of 10 mm at the underside of the abutment wall for units supporting the abutments (*CHBDC (2019) Commentary* Section 6.11.2.2).

For single vertical HP 310x110 piles or drilled shafts installed to the design tip elevation provided in Sections 6.4.1 and 6.5.1, the estimated factored lateral resistance at ULS and the factored lateral resistance at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analyses carried out using the commercially available program RSPile (Version 3.0), developed by Rocscience Inc.

Foundation Element	Deep Foundation Type	Factored Lateral Ultimate Geotechnical Resistance at the Pile Cap / Top of Drilled Shaft (kN)	Factored Lateral Serviceability Geotechnical Resistance for 10 mm of Deflection at the Pile Cap / Top of Drilled Shaft (kN)
West Abutment	Steel HP 310x110 Pile	60	20
Pier 1	1.2 m dia. Drilled Shaft	750	675
Pier 2	1.2 m dia. Drilled Shaft	750	675
Pier 3	1.2 m dia. Drilled Shaft	750	675
East Abutment	Steel HP 310x110 Pile	100	25

Note: The analysis assumes a free-head condition.

Group action for lateral loading should be considered in accordance with Section C6.11.3.4 of the *Commentary to the CHBDC (2019)*.

6.9 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls, or on adjacent 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 including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II should be used beneath the EPS backfill behind the abutment wall and wingwalls. Subdrains should be installed within the granular fill beneath the EPS to provide positive drainage. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting).

6.9.1 Static At-Rest and Active Lateral Earth Pressures

The following guidelines and recommendations are provided regarding the lateral earth pressures for static loading conditions. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

The following parameters (unfactored) may be used assuming the use of EPS as backfill behind the abutment:

Fill Type	Unit Weight of Material (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
EPS	0.5	0.22	0.22 ¹

Note:

1. This represents the maximum value of K_a (which is equal to K_o) and is only applicable when the angle of the EPS/granular fill interface behind the abutment wall from horizontal is less than the internal angle of friction of the granular fill.

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.

6.10 Embankment Design

Design of the approach embankments within 20 m of the west and east abutments for Leslie Street Overpass is discussed the following sections. Recommendations regarding the high fill embankments that transition into these approach embankments, the North-East ramp from Leslie Street and the South-East ramp from Leslie Street, are discussed in Golder's report titled, "Foundation Investigation and Design Report, High Fill Embankments, Highway 401 Eastbound Collector Lanes from Avenue Road to Warden Avenue, Toronto, Ontario, MTO GWP 2130-01-00".

6.10.1 Parameter Selection

The foundation engineering parameters for the soil types encountered in the boreholes at the approach embankments are summarized below. For stability and settlement analysis, the groundwater level behind the abutment stem walls was assumed to be at Elevation 138.4 m and 134.8 m for the west and east abutments, respectively. Figure 6 presents the parameters associated with the cohesive deposit encountered at the site.

Stratigraphic Unit	γ (kN/m ³)	ϕ' (°)	S_u (kPa)	σ_p' (kPa)	e_o	C_c	C_r	E' (MPa)
New Fill (Granular 'A' or 'B' Type II)	21	38	--	--	--	--	--	75
Expanded Polystyrene (EPS) Fill	0.5	--	14	--	--	--	--	--
Loose to Dense Sandy Silt to Gravelly Sand Fill	20	32	--	--	--	--	--	15
Firm to Hard Clayey Silt to Clayey Silt-Silt and Sand Fill	21	28	100	--	--	--	--	30
Very Loose to Very Dense Silt to Sand	20	32	--	--	--	--	--	15
Soft to Stiff Clay to Clayey Silt-Silt	21	33	35	160 – 250	0.8	0.5	0.05	--
Very Stiff to Hard Silty Clay to Sandy Silt Till	22	--	200	--	--	--	--	150
Compact Silty Sand Till	22	37	--	--	--	--	--	150
Compact to Very Dense Silt to Sand	20	--	--	--	--	--	--	150

Stratigraphic Unit	γ (kN/m ³)	ϕ' (°)	S_u (kPa)	σ_p' (kPa)	e_o	C_c	C_r	E' (MPa)
Very Stiff Hard Sandy Clayey Silt to Clayey Silt-Silt	22	--	100	--	--	--	--	150
Hard Organic Silt to Sandy Organic Silt	22	35	100	--	--	--	--	100
Hard Gravelly Clayey Silt to Sandy Clayey Silt (Residual Soil)	21	--	200	--	--	--	--	150

6.10.2 Global Stability

Limit equilibrium global stability analyses were carried out for the proposed abutment walls using the commercially available program Slide2 (version 9.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analysis, the Factors of Safety of numerous potential failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. 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. $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, minimum Factors of Safety of 1.3 and 1.5 have been used for the design of the approach embankment slopes for the temporary and permanent conditions, respectively, as per Table 6.2 of CHBDC (2019).

Based on the profile provided on the GA Drawing, the approach embankment side slopes will be inclined at 2 horizontal to 1 vertical (2H:1V) with an overall height of about 5.0 m and 5.9 m of new fill over the native subgrade materials at the west and east approach embankments, respectively.

The stability analyses for the abutment walls indicate that for the short-term (undrained) conditions, the approach embankments / abutments will have a global Factor of Safety of greater than 1.3, and for the long-term (permanent) conditions, the approach embankments / abutments will have a global Factor of Safety greater than 1.5. The results of the stability analysis are summarized below and are shown on Figures 7 to 12 following the text of this report.

Foundation Element	Relevant Boreholes	Static Global Stability	Slope	Factor of Safety
West Abutment	L-1, N/E RS-2, N/E RS-3, N/E RS-4, N/E RS-5, N/E RS-6	Temporary Condition	North-South	>1.3
		Permanent Condition		>1.5
	L-1, N/E RS-2, N/E RS-3, N/E RS-4, N/E RS-5, N/E RS-6	Temporary Condition	West-East	>1.3
		Permanent Condition		>1.5

Foundation Element	Relevant Boreholes	Static Global Stability	Slope	Factor of Safety
East Abutment	L-5, L-6, N/E RS-10	Temporary Condition	North-South	>1.3
		Permanent Condition		>1.5
	L-5, L-6, N/E RS-10	Temporary Condition	West-East	>1.3
		Permanent Condition		>1.5

6.10.3 Settlement

To estimate the magnitude of expected settlement, analyses were carried out at the critical sections of the proposed approach embankments. The critical sections correspond to the area of the approach embankment near the abutment. The settlement analyses assume that topsoil and any surficial deposits containing organic material, or any other deleterious materials have been removed and replaced with SSM, earth fill or granular fill. The settlement analyses were carried out using the commercially available program Settle3 (Version 5.0), developed by Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on Westergaard's (1938) solution.

The sources of settlement are considered to include the following:

- Immediate settlement of the granular soils (short-term);
- Primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory – long-term); and,
- Secondary time dependent (creep) consolidation of the cohesive deposits (long term).

The thickness of the compressible foundation soils and the height of the approach embankments vary along the approach embankment alignment, and as such the settlements along the length of the alignment will similarly vary; however, the settlements estimated from the settlement analysis represent the maximum anticipated value at the approach embankments.

The settlement performance criteria for design of approach embankments are outlined in MTO's "Embankment Settlement Criteria for Design", dated July 2, 2010. In general, new embankments approaching structural elements such as bridge abutments are to be designed such that total settlement and rate of differential settlement do not exceed 25 mm, over a 20-year period following completion of construction.

A summary of the estimated magnitudes of settlement for each of the approach embankments constructed using conventional granular fill is presented below, assuming the use of conventional earth or granular fill for construction of the new/widened embankment for the Highway 401 EB Collectors.

Foundation Investigation Area Designation	Foundation Investigation Area Limits	Proposed Maximum Embankment Height ¹	Estimated Settlement over a 20-Year Period ² (mm)	
West Approach Embankment	Station 25+689 to 25+709 and	5.5 m	$\delta_{Immediate} =$	85
			$\delta_{Primary} =$	505
			$\delta_{Secondary} =$	45
			$\delta_{Total} =$	635
East Approach Embankment	Station 25+772 to 25+792	5.9 m	$\delta_{Immediate} =$	35
			$\delta_{Primary} =$	65
			$\delta_{Secondary} =$	35
			$\delta_{Total} =$	135

Notes:

1. The proposed maximum embankment heights are based on centreline profiles of the highway alignments and existing ground surface profiles provided in AECOM's General Arrangement Drawing dated September 2020. Embankment heights are approximate and are relative to original ground surface.
2. The total settlement (δ_{Total}) is defined as the sum of the immediate settlement ($\delta_{Immediate}$) due to elastic compression of the non-cohesive deposits as well as primary ($\delta_{Primary}$) and secondary ($\delta_{Secondary}$) settlements due to time dependent consolidation of the cohesive deposits.

Based on the estimated magnitude of settlement above, settlement mitigation options will be required to meet the settlement performance criterion.

6.10.3.1 Mitigation Options

Several settlement mitigation options have been considered to meet the settlement performance criterion and a brief discussion on these alternatives is provided below. Other ground improvement measures such as surcharging, wick drains, rammed aggregate piers, deep soil mixing, and dynamic compaction are not considered suitable or cost effective due to the composition and thickness of the deposit and such options are not discussed further in this report.

- **Preloading:** Where cohesive deposits are thick at the Leslie Street Overpass embankments, preloading is expected to require an extended period of time to reach the settlement performance criterion; there may not be adequate time during construction to induce sufficient secondary (creep) consolidation to occur and this may potentially result in long-term settlement of the embankment. Further, there is a risk that the magnitude of settlement under conventional fill loading will affect the serviceability of adjacent/nearby bridge structures and underground utilities. An instrumentation and monitoring would be required during construction to assess when the settlement performance criterion has been achieved.

- **Full Sub-Excavation (or Other Forms of Full-Depth Ground Improvement):** Although full-depth sub-excavation of the compressible cohesive deposits underlying the embankment would greatly improve post-construction settlement, it is not a feasible alternative given the depth to the bottom of the cohesive deposit (up to about 26 m below ground surface) and the proximity of other bridge structures in the immediate area. This depth also renders rammed aggregate piers or other soil improvement techniques unsuitable for this site.
- **Lightweight Fill (Lightweight Slag or Cellular Concrete):** Various lightweight fill materials are available, from lightweight slag with a unit weight of approximately 14 kN/m³, to cellular concrete with a unit weight between 4 and 7 kN/m³. However, for the volume of fill required for the EB Collector embankment widening, a significant preloading period would still be required to achieve the settlement performance criterion, with similar (although lesser) disadvantages compared to preloading using conventional fill materials.
- **Lightweight Fill (Expanded Polystyrene):** The use of expanded polystyrene (EPS) is another alternative that can be considered to significantly reduce the magnitude of consolidation settlement. Where required, EPS can be used in conjunction with preloading, surcharging or subexcavation for any net increase in embankment loading to achieve the settlement performance criterion sooner and therefore, reduce the length of time for construction. Another advantage with EPS is that it allows for a rapid construction of an embankment and can, therefore, shorten the construction schedule. However, the cost of EPS is higher than that of granular fill.

Based on the above considerations, preloading and lightweight EPS fill is considered the technically preferred alternative to mitigate long-term post-construction settlement at this site. Full sub-excavation (or other forms of full-depth ground improvement) is not discussed further herein due to the inherent disadvantages outweighing the advantages.

6.10.3.2 Preloading

Based on the estimated coefficient of consolidation (c_v) of 5.5×10^{-3} cm²/s and a modified secondary compression index ($c_{\alpha\varepsilon}$) of 1.9×10^{-3} mm per log cycle of time for the cohesive deposits, it is estimated that the following preload periods will be required for each approach embankment area to meet the settlement performance criterion assuming the embankments are constructed of granular fill.

Embankment	Height of Embankment (m)	Estimated Preload Period (days)
West Approach Embankment	5.5	1000
East Approach Embankment	5.9	700

The Contractor should be notified about the magnitude of the estimated settlement at the completion of the preload period so that the embankment is constructed such that the resultant embankment/finished grade after the immediate settlement has occurred meets the geometric requirements. Considering the length of the preload period, if this alternative is to be adopted, the magnitude and time-rate of settlement during and after construction of the preload embankment should be assessed by a monitoring program consisting of settlement plates (SPs) and vibrating wire piezometers (VWPs) to confirm the end of the preload period.

6.10.3.3 Lightweight Fill (Expanded Polystyrene)

It is understood that the pavement structure will consist of approximately 90 mm of asphalt over 250 mm of reinforced concrete and a 250 mm of Granular 'A' subbase. In addition, a 175 mm thick of concrete will be placed over the EPS as a protective cover. Due to the additional load from the cover material and the granular backfill behind the abutment wall, the design includes sub-excavation of the existing fill and/or native silty sand deposit below the ground surface that is to be replaced with EPS, to offset additional loads.

A summary of the estimated magnitudes of settlement for the proposed west and east approach embankments is presented below assuming they are constructed using EPS fill behind the abutment walls. The combined thickness of all cover material above the EPS and on the side slopes is assumed to be about 1.0 m, and where applicable, the sub-excavation and replacement of the existing fill and/or native silty sand deposit is carried out with EPS to offset the loads from the cover material and pavement structure.

Embankment	Height of Approach Embankment (m)	Minimum Thickness of EPS (m)	Depth of Sub-excavation	Estimated Post-Construction Settlement over a 20-Year Period (mm)
West Approach Embankment	6.0	6.5	Minimum 1.5 m below ground surface or from ground surface to Elevation 139.0 m, whichever is greater	$\delta_{Immediate} = 0$ $\delta_{primary} = 10$ $\delta_{secondary} = 5$ $\delta_{Total} = 15$
East Approach Embankment	5.9	Up to 6.4	Minimum 0.5 m below ground surface or from ground surface to Elevation 139.8 m, whichever is greater	$\delta_{Immediate} = 0$ $\delta_{primary} = 20$ $\delta_{secondary} = 0$ $\delta_{Total} = 20$

Based on the estimated magnitude of settlement for the EPS embankment, preloading of the foundation soils will not be required to meet the settlement performance criterion given that the immediate settlement is expected to occur during construction.

6.11 Preferred Foundation Mitigation Option

Given the results of stability and settlement analysis, the preferred foundation mitigation options for the embankments at the west and east and west approach embankments is summarized in the table below. This table summarizes the primary consolidation settlement remaining after completion of construction plus the secondary (creep) settlement associated with the eastbound collector lane embankment loading over a 20-year period.

Embankment	Settlement Criterion 20-years After Construction (mm)	Type of Embankment Fill	Depth of Sub-Excavation	Embankment Side Slope	Preload Period (days)	Estimated Post-construction Settlement over a 20-Year Period (mm)
West Approach Embankment	25	EPS fill with 1.5 m of sub-excavation	Minimum 1.5 m below ground surface or from ground surface to Elevation 139.0 m, whichever is greater, to be replaced with EPS fill	2H:1V	Not Required	$\delta_{Immediate} = 0$ $\delta_{primary} = 10$ $\delta_{secondary} = 5$ $\delta_{Total} = 15$
East Approach Embankment	25	EPS Fill	Minimum 0.5 m below ground surface or from ground surface to Elevation 139.8 m, whichever is greater, to be replaced with EPS fill	2H:1V	Not Required	$\delta_{Immediate} = 0$ $\delta_{primary} = 20$ $\delta_{secondary} = 0$ $\delta_{Total} = 20$

6.12 Analytical Testing of Construction Materials

The results of analytical tests carried out on six soil samples and are presented in Section 4.4 and on the Certificate of Analysis in Appendix C. The analytical test results were compared to CSA A23.1 Table 3 (*Additional requirements for concrete subjected to sulphate attack*) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured on the soil samples range from less than 0.002% to 0.01%, which indicates a less than Moderate degree of exposure (i.e., below the class S3 exposure limits) and may be considered negligible according to Table 7.2 of MTO's Gravity Pipe Design Guidelines (2004). Therefore, based on the soil samples tested, when the designer is selecting the exposure class for the concrete structure, the effects of sulphates from within the site soils in contact with the spread footing or pile cap and any portion of the proposed structure constructed below the ground surface may not need to be considered. However, given that the proposed structure will be exposed to de-icing salt/chemicals, consideration should also be given by the designer to designing the concrete structure for a "C" type exposure class as defined by CSA A23.1 Table 1.

The pH measured on the soil samples range from about 7.8 to about 8.1. According to the MTO Gravity Pipe Design Guidelines (2014), a pH greater than 8.5 is considered strongly alkaline and is indicative of an increased potential for corrosion. The resistivity measured in the six soil samples range from 380 ohm-cm to 7,700 ohm-cm which indicates that the soil corrosiveness is very low ($10,000 > R > 6,000$) to severe ($2,000 > R$) as per Table 3.2 of the MTO Gravity Pipe Design Guideline (2014).

These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing, the potential for corrosion and the corrosion susceptibility of materials to be used in construction

of the structure foundations in Table 7.1 of the MTO Gravity Pipe Design Guideline (2014) into consideration of the ultimate selection of materials. Ultimately, it is the designer's decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) are satisfied.

6.13 Construction Considerations

6.13.1 Open-Cut Excavations

The excavation for construction of the new pile caps at the abutments and piers will extend through the topsoil, cohesive and non-cohesive fill, native sandy silt to sand deposits. The topsoil and any organic / deleterious materials encountered within the footprint of the proposed foundation elements and approach embankments (and below any required sub-excavation and replacement with lightweight fill, where applicable) should be sub-excavated and replaced with OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II material and Select Subgrade Material. All excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended.

The soils to be excavated above the groundwater level can be classified according to OHSA as Type 3 soils. Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes of 1H:1V or flatter. Where sub-excavation is to be carried below the groundwater level without dewatering, the soils can be classified as Type 4, which requires side slopes of 3H:1V or flatter.

6.13.2 Subgrade Preparation and Embankment Construction

The EPS fill should be installed in accordance with the NSSP for Rigid Expanded Polystyrene Embankment Fill provided in Appendix D. It is recommended that a levelling pad comprised of at least 300 mm of OPSS.PROV 1010 (*Aggregates*) Granular 'A' material be placed prior to the installation of the first layer of EPS. The levelling pad should be compacted to at least 95% of the material's Standard Proctor maximum dry density. The EPS should be covered with a 10 mil thick polyethylene sheet and overlain with a minimum 175 mm thick 30 MPa reinforced concrete top slab constructed on top of the EPS, followed by a protective cover/pavement structure over the slab. The EPS on the side slopes of the embankment should be covered with a 1 m thick layer of conventional soil/granular material at an inclination of 2 horizontal to 1 vertical (2H:1V) or flatter.

Placement of granular fill (satisfying OPSS.PROV 1010 Granular 'B' Type I or Type II requirements) above the water table for construction of new embankments should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*). Granular fill should be compacted in accordance with OPSS.PROV 501 (*Compacting*). In addition, benching of the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (*Benching of Earth Slopes*), as appropriate.

6.13.3 Erosion Protection

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding should be carried out as soon as practicable after construction of the embankments. In the short-term, if placement of cover material cannot be carried out soon after the construction of the embankments, erosion control blankets should be installed to minimize erosion of the embankment slopes. The erosion protection should be in accordance with OPSS 802 (*Topsoil*) and OPSS.PROV 804 (*Seed and Cover*).

6.13.4 Control of Groundwater

The groundwater level measured in the piezometers at Leslie Street were at about Elevation 136.1 m in September 2020. However, it is noted that the groundwater level could be higher during periods of heavy/sustained precipitation during the wet seasons.

It is anticipated that foundation excavations for pile caps at the piers will extend below the groundwater table, which will require dewatering measures so that the pile caps can be constructed in dry conditions. Dewatering operations should be carried out / managed in accordance with OPSS PROV 902 (*Excavating and Backfilling - Structures*) as amended by Special Provision FOUN0003 (Dewatering Structure Excavations), a copy of which is included in Appendix D for inclusion into the Contract Documents.

Water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking and a Section 53 approval for discharge of water to the environment. A “Water Taking Plan” and a “Discharge Plan” are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan (to be developed by a qualified professional). The contractor will be responsible for obtaining any required discharge approvals. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day.

Depending on the final foundation option, an EASR or PTTW may be required and a hydrogeological assessment should be conducted to estimate the expected water extraction requirements, assist in registration, and to provide the required documentation.

If drilled shaft foundations are adopted, temporary casings with a balancing head of bentonite/polymer slurry will be required to support the overburden soils and equalize groundwater pressures during construction. In addition, placement of concrete by tremie methods would be required.

6.13.5 Temporary Protection Systems

Temporary protection systems will be required to facilitate the staged construction of the abutments and piers, and depending on the selected foundation option, temporary protection systems may also be required to facilitate the construction of the piers to reduce the size of excavation. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*). The lateral movement of the temporary protection systems shall meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation. The selection and design of the protection system will be the responsibility of the contractor.

6.13.6 Obstructions

Cobbles and/or boulders may be encountered within the glacial till deposits, which may affect the installation of deep foundations. It is recommended that pile tip reinforcement, such as flange reinforcement or driving shoes per OPSD 3000.100, be used on all steel H-piles to minimize damage during pile driving at the site. If drilled shaft foundations are selected, the construction equipment should be capable of advancing the temporary or permanent casings through such obstructions.

An NSSP has been provided in Appendix D to address the presence of obstructions within the till deposit, for inclusion into the Contract Documents.

6.13.7 Vibration Monitoring

Vibration monitoring and condition surveys are recommended during installation of deep foundations and temporary protection systems to confirm that construction techniques and associated vibration levels experienced at nearby structures and utilities are maintained below tolerable levels, and to mitigate potential claims from property owners.

An example NSSP is provided in Appendix D based on the following maximum peak particle velocity (PPV) values:

- Existing overheads, overpasses, and bridges along Highway 401: 50 mm/s;
- Utilities (including the recently installed 0.6 m watermain): 50 mm/s;
- Conventional commercial/industrial buildings: 50 mm/s; and,
- Residential homes and wells: 25 mm/s.

It is considered good practice to conduct vibration monitoring and pre- and post-construction condition surveys at existing structures within an approximately 100 m radius of any installation of deep foundations and/or temporary protection systems. In some cases, agencies may choose to expand the radius beyond that anticipated for attenuation of construction-induced vibrations.

6.13.8 Piezometer Decommissioning

A standpipe piezometer was installed in Boreholes WM16-07 and N/E RS-4 to permit monitoring of the groundwater level at the site. Ontario Regulation (O.Reg.) 903 amended by O.Reg. 128/03 of the Ontario Water Resources Act requires that monitoring wells are properly abandoned/decommissioned by qualified personnel. It is recommended that the decommissioning of the standpipe piezometers be carried out as part of the construction activities at the site so that water level measurements can be taken immediately prior to and during construction as may be appropriate, prior to decommissioning by the Contractor. A Notice to Contractor for this item is included in Appendix D.

7.0 CLOSURE

Foundation Design Report was prepared by Ms. Katelyn Nero, P.Eng., and reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate with Golder. Ms. Lisa Coyne, P.Eng., a Principal and MTO Foundations Designated Contact with Golder, conducted an independent technical and quality control review of this report.

Golder Associates Ltd.



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Principal, MTO Foundations Designated Contact

KN/CN/LCC/ml

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<https://golderassociates.sharepoint.com/sites/20393g/deliverables/foundations/2-leslie-street-overpass/4-final/1786302-fidr-rev.0-leslie-st-overpass-1-october-2021.docx>

REFERENCES

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Commercial Software:

RSPile (Version 3.0) by Rocscience Inc.

Settle3 (Version 5.0) by Rocscience Inc.

Slide (Version 9.0) by Rocscience Inc.

Ontario Regulations:

O. Reg. 213 Construction Projects (as amended)

O. Reg. 903 Wells (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS.PROV 802 Construction Specification for Topsoil

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

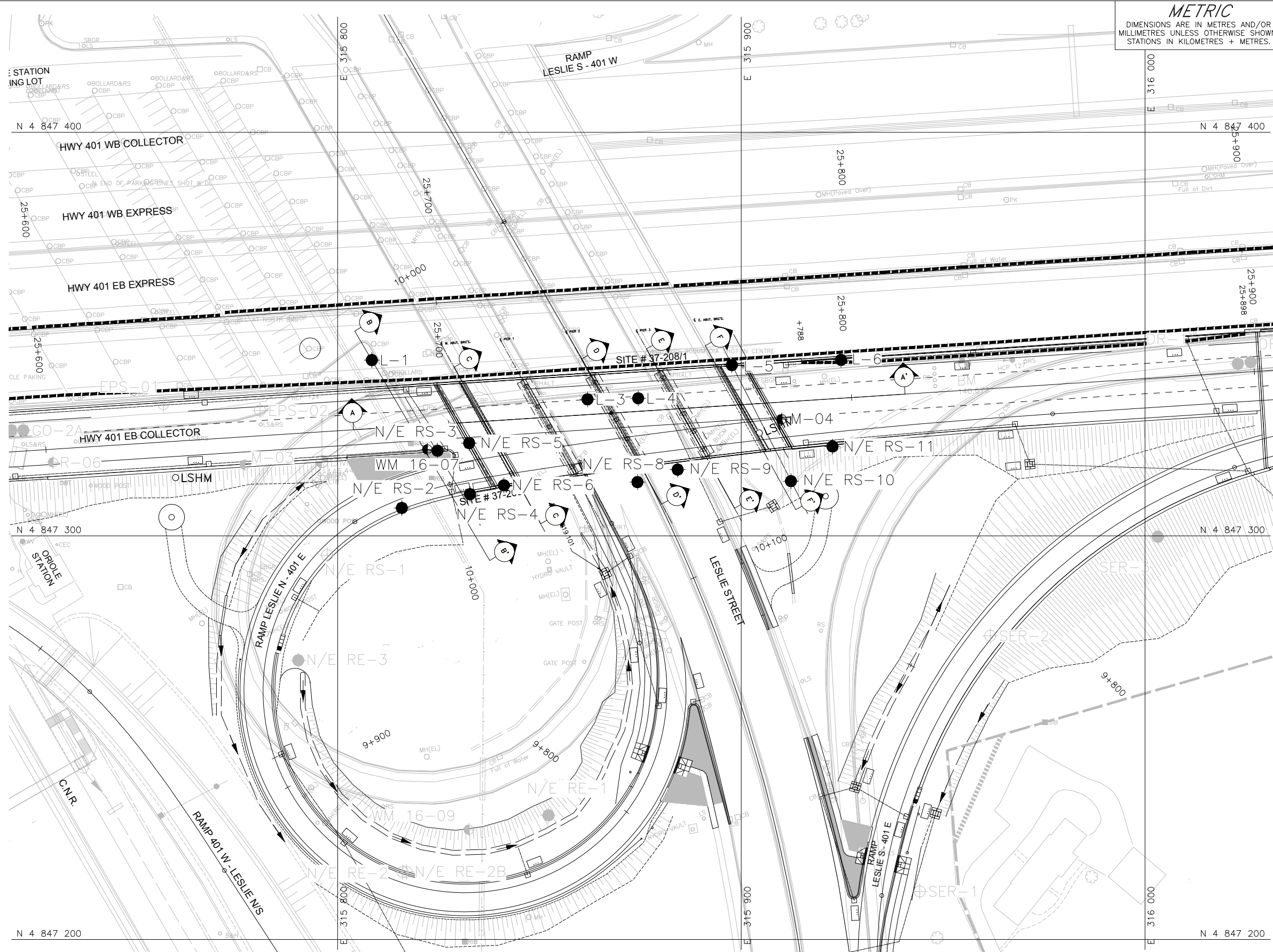
Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundations, Piles, Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements
OPSD 3190.100	Walls, Retaining and Abutments, Wall Drain

Table 1: Comparison of Foundation Alternatives – Leslie Street Overpass

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Driven Steel H-piles (HP 310x110) driven to bedrock	<ul style="list-style-type: none"> Feasible for all foundation elements 	<ul style="list-style-type: none"> Conventional construction methods for H-pile foundations. High axial resistances available Allow for integral abutment design if used to support abutments 	<ul style="list-style-type: none"> Dewatering measures may be required at abutments and piers for the construction of pile caps. Requires driving shoes due to potential presence of cobbles and boulders within the till deposits. 	<ul style="list-style-type: none"> Lower relative cost than drilled shafts (caissons). 	<ul style="list-style-type: none"> Risk of damage to piles due to cobbles and boulders that maybe present within the till deposits.
0.9 m, 1.2 m or 1.5 m Diameter Drilled Shafts (Caissons) Socketed 1 m into Bedrock	<ul style="list-style-type: none"> Feasible for all foundation elements 	<ul style="list-style-type: none"> Conventional construction methods for drilled shaft foundations. Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. Requires a smaller footprint for construction in constrained working areas, as compared with multiple rows of vertical or battered piles. Low energy installation suited in and adjacent to the rail corridor and near gas pipelines. 	<ul style="list-style-type: none"> Casings will be required, plus special measures such as use of slurry to counterbalance groundwater pressures, reduce risk of blow-out, and minimize ground disturbance. Generation of soil and rock cuttings during drilled shaft advancement. Challenges associated with inspection of shaft base. Does not allow for integral abutment design if used at the abutments. 	<ul style="list-style-type: none"> Higher relative cost than driven piles. 	<ul style="list-style-type: none"> Will be difficult to inspect the base of the drilled shaft due to the need for slurry inside the casings.

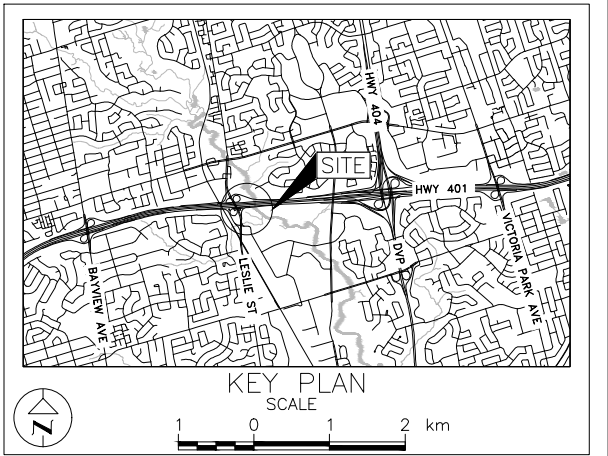
Drawings



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MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2130-01-00

HIGHWAY 401 EB COLLECTORS
LESLIE STREET OVERPASS (SITE NO. 37X-0208/B1)
BOREHOLE LOCATIONS



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (MTO GEOCRETS No. 30M14-460 and 30M14-463)
- Cone Penetration Test (CPT) - Current Investigation
- Borehole - Golder Other Investigation
- Cone Penetration Test (CPT) - Golder Other Investigation
- Borehole - Previous Investigation Other Report

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
L-1	140.8	4847343.6	315808.5
L-3	136.4	4847333.8	315861.9
L-4	136.8	4847334.1	315874.5
L-5	142.6	4847342.3	315897.7
L-6	141.7	4847343.6	315924.7
M-04	140.2	4847328.9	315910.0
N/E RS-2	142.0	4847306.9	315815.9
N/E RS-3	141.2	4847321.1	315824.7
N/E RS-4	139.3	4847310.3	315832.8
N/E RS-5	140.3	4847323.0	315832.6
N/E RS-6	138.8	4847312.5	315841.2
N/E RS-8	136.8	4847313.3	315874.3
N/E RS-9	137.3	4847316.4	315884.2
N/E RS-10	137.3	4847313.6	315912.3
N/E RS-11	138.4	4847322.2	315922.6
WM 16-07	141.5	4847321.4	315822.2

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 Contract Documents.

REFERENCE
Previous investigation data obtained from "Foundation Investigation and Design Report, Proposed 600 mm Watermain Relocation, Highway 401 Leslie Street Interchange, Toronto, Ontario, G.W.P. 2061-13-00" dated March 1, 2017 (GEOCRETS No. 30M14-460), and from "Foundation Investigation and Design Report, Highway 401 Overpass at GO Station Parking Lot and Leslie Street, Highway 401 and Leslie Street Interchange, Toronto, Ontario, W.P. 2061-13-00, Site 37-206/1-4", dated September 11, 2017 (GEOCRETS No. 30M14-463), prepared by Thurber Engineering Ltd.
Base and design plans provided in digital format by AECOM, drawing file no. 401_EBC_Avenue-Warden_base.dwg and 401_EBC_Avenue-Warden_plan.dwg, received September 17, 2019. General Arrangement provided in digital format by AECOM, drawing file no. Leslie Street Overpass_GA.dwg, received July 28, 2021.

NO.	DATE	BY	REVISION
GEOCRETS No. 30M14-330		PROJECT NO. 1786302	DIST. .
HWY. 401			
SUBM'D. KN	CHKD. KN	DATE: 09/17/2021	SITE: 37X-0208/B1
DRAWN: SA/DD/TR	CHKD. CN	APPD. LCC	DWG. 1



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

CONT No. _____
GWP No. 2130 01 00

HIGHWAY 401 EB COLLECTORS
LESLIE STREET OVERPASS (SITE NO. 37X-0208/B1)
SOIL STRATA



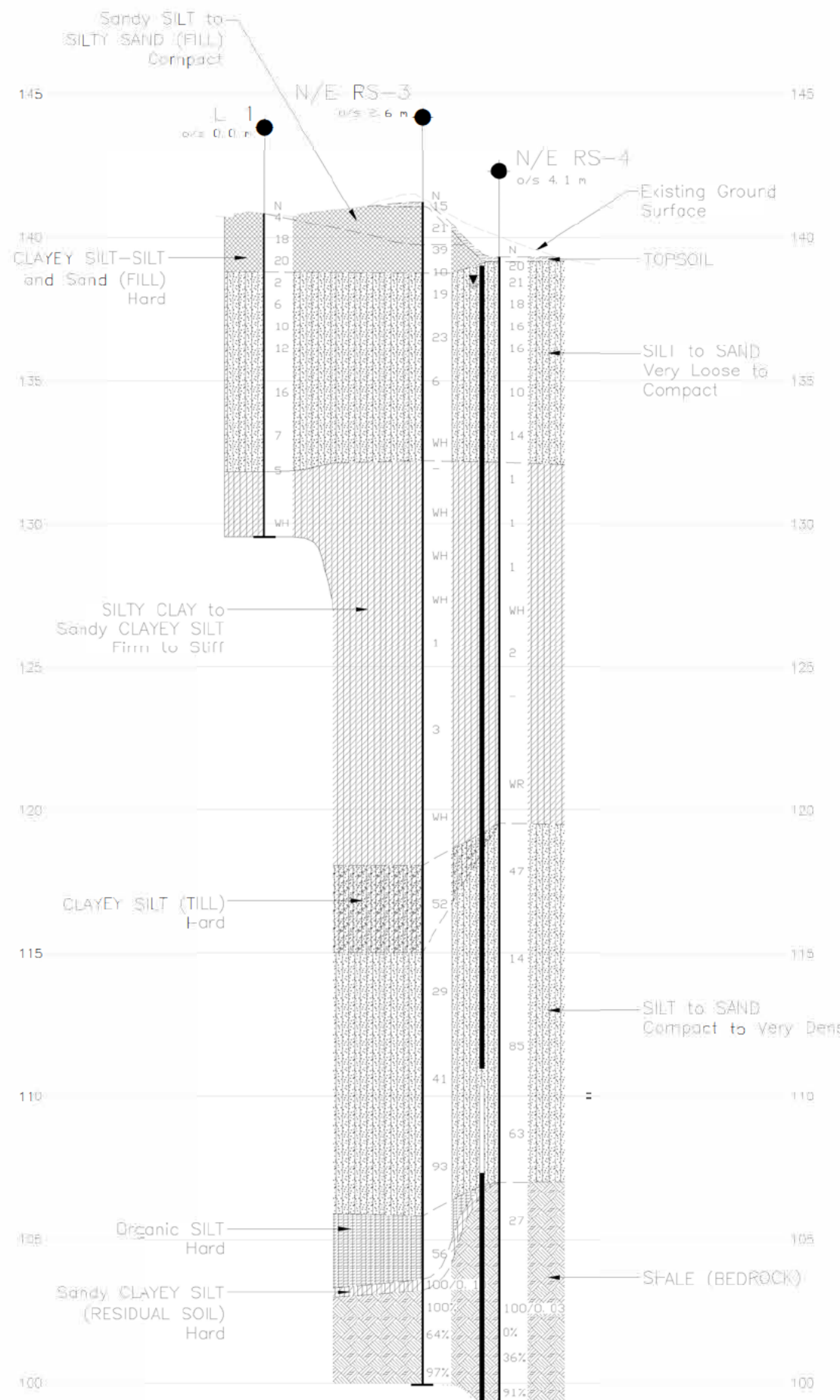
BOREHOLE CO-ORDINATES (NAD 83 M/TM ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
L-1	140.8	4847343.6	315808.5
N/E RS-3	141.2	4847321.1	315824.7
N/E RS-4	139.3	4847310.3	315832.8
N/E RS-5	140.3	4847323.0	315832.6
N/E RS-6	138.8	4847312.5	315841.2



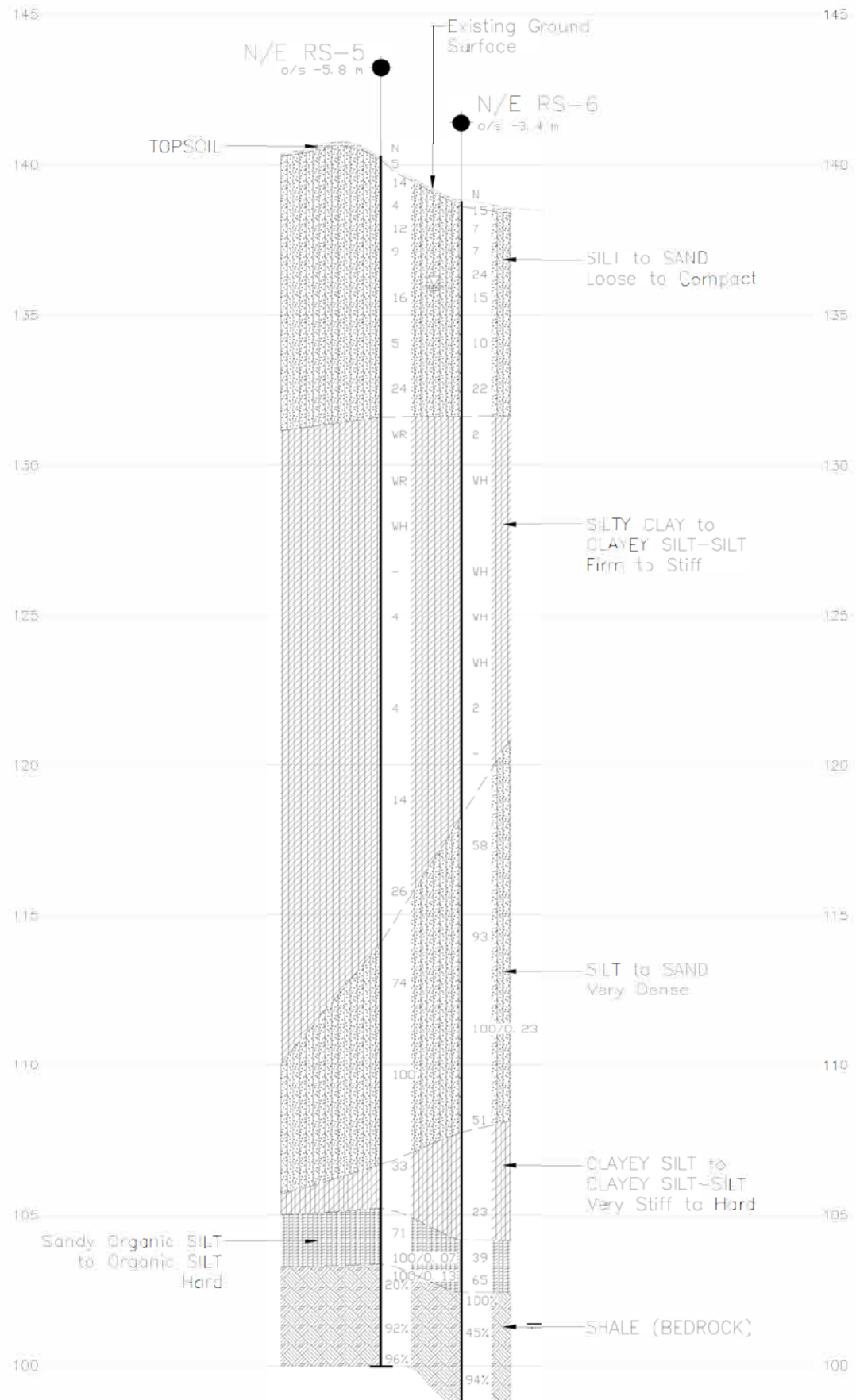
REFERENCE
Existing ground topography provided by, drawing file no. 401NY02, received February 7, 2019.

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

NO.	DATE	BY	REVISION
Geocres No. 30M14-330			
HWY. 401	PROJECT NO. 1786302	DIST.	
SUBM'D. KN	CHKD. KN	DATE: 09/17/2021	SITE: 37X-0208/B1
DRAWN: SA/TR	CHKD. CN	APPD. LCC	DWG. 3



B-B' SECTION B-B' - WEST ABUTMENT

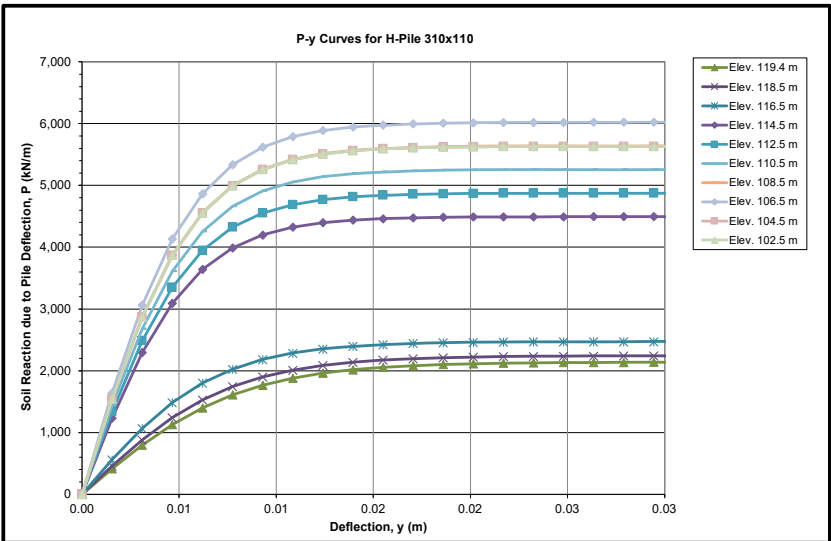
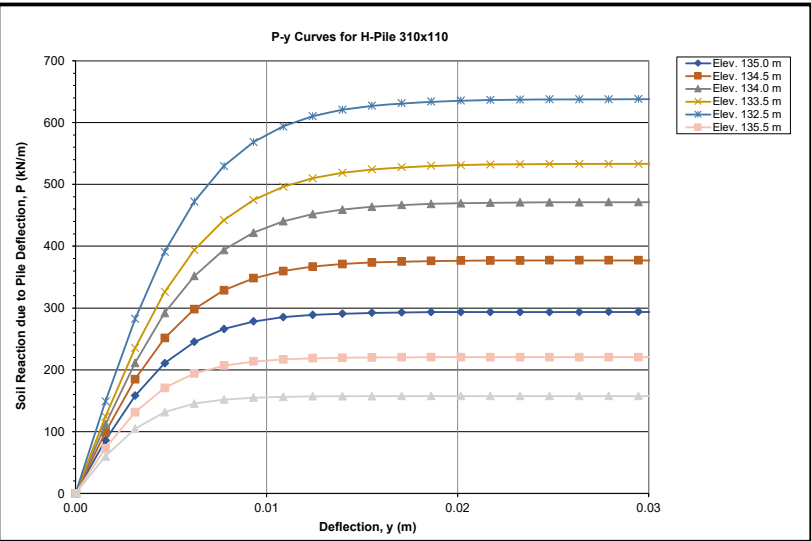
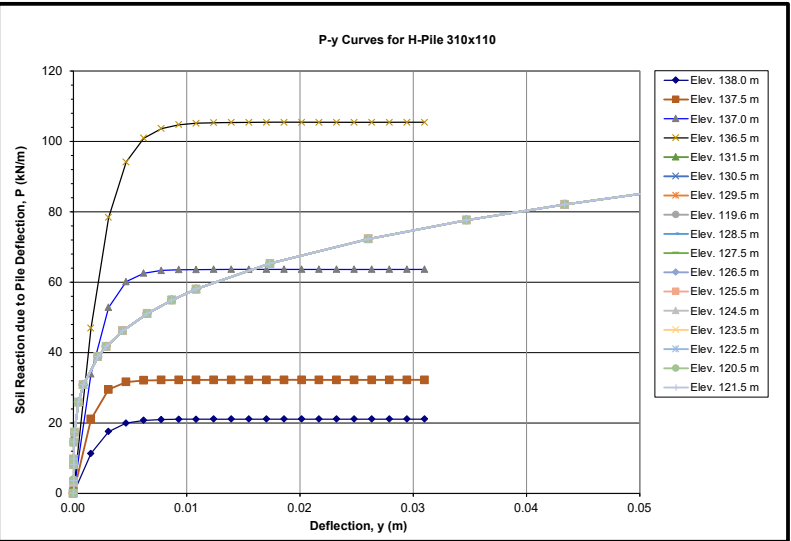


C-C' SECTION C-C' - WEST ABUTMENT

Figures

Figure 1

Description Depth (z) Elevation P-y Curves	Very Loose to Compact Silty Sand to Sand																								Soft to Very Stiff Clayey Silt to Silty Clay																							
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 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													
	Elev. 138.0 m		Elev. 137.5 m		Elev. 137.0 m		Elev. 136.5 m		Elev. 136.0 m		Elev. 135.5 m		Elev. 135.0 m		Elev. 134.5 m		Elev. 134.0 m		Elev. 133.5 m		Elev. 132.5 m		Elev. 131.5 m		Elev. 130.5 m		Elev. 129.5 m		Elev. 128.5 m		Elev. 127.5 m		Elev. 126.5 m		Elev. 125.5 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	11.296	0.00155	21.09307	0.00155	33.95629	0.00155	46.977684	0.00155	59.98338	0.00155	72.938459	0.00155	85.844557	0.00155	98.70796	0.00155	111.532634	0.00155	124.01555	0.00155	148.80174	8.7E-07	2.4159946	8.7E-07	2.532957	8.7E-07	2.6506811	8.7E-07	2.76719577	8.7E-07	2.8845	0.000000868	3.00118889	8.7E-07	3.1188333												
0.0031	17.528	0.0031	29.513646	0.0031	52.83932	0.0031	78.366111	0.0031	104.7823	0.0031	131.46328	0.0031	158.15624	0.0031	184.75683	0.0031	211.239012	0.0031	235.30875	0.0031	282.24964	4.3E-06	8.211	4.3E-06	8.211	4.3E-06	8.211	4.3E-06	8.211	4.3E-06	8.211	4.3E-06	8.211	0.00000434	8.211	4.3E-06	8.211											
0.00465	19.9232	0.00465	31.623924	0.00465	60.12853	0.00465	94.132041	0.00465	131.4785	0.00465	170.69031	0.00465	210.78349	0.00465	251.253748	0.00465	291.809638	0.00465	325.90066	0.00465	390.74289	4.3E-06	9.765	8.7E-06	9.765	8.7E-06	9.765	8.7E-06	9.765	8.7E-06	9.765	0.00000868	9.765	8.7E-06	9.765													
0.0062	20.7153	0.0062	32.084455	0.0062	62.5457	0.0062	100.92931	0.0062	145.3093	0.0062	193.92349	0.0062	245.1533	0.0062	298.00269	0.0062	351.798726	0.0062	393.95348	0.0062	472.11258	4.3E-05	14.6	4.3E-05	14.6	4.3E-05	14.6	4.3E-05	14.6	4.3E-05	14.6	0.00000434	14.6	4.3E-05	14.6													
0.00775	20.9644	0.00775	32.182446	0.00775	63.30538	0.00775	103.66908	0.00775	151.9577	0.00775	206.68567	0.00775	266.04045	0.00775	328.7236	0.00775	393.775575	0.00775	442.05833	0.00775	529.5381	8.7E-05	17.36	8.7E-05	17.36	8.7E-05	17.36	8.7E-05	17.36	8.7E-05	17.36	0.00000868	17.36	8.7E-05	17.36													
0.0093	21.0418	0.0093	32.203015	0.0093	63.45049	0.0093	104.74006	0.0093	155.0335	0.0093	213.41304	0.0093	278.18146	0.0093	348.0394	0.0093	421.90102	0.0093	474.61972	0.0093	568.33563	0.00043	25.97	0.00043	25.97	0.00043	25.97	0.00043	25.97	0.00043	25.97	0.00043	25.97	0.00043	25.97													
0.01085	21.0652	0.01085	32.207443	0.01085	63.61344	0.01085	105.16107	0.01085	156.4407	0.01085	216.87435	0.01085	285.06965	0.01085	359.84482	0.01085	440.199978	0.01085																														

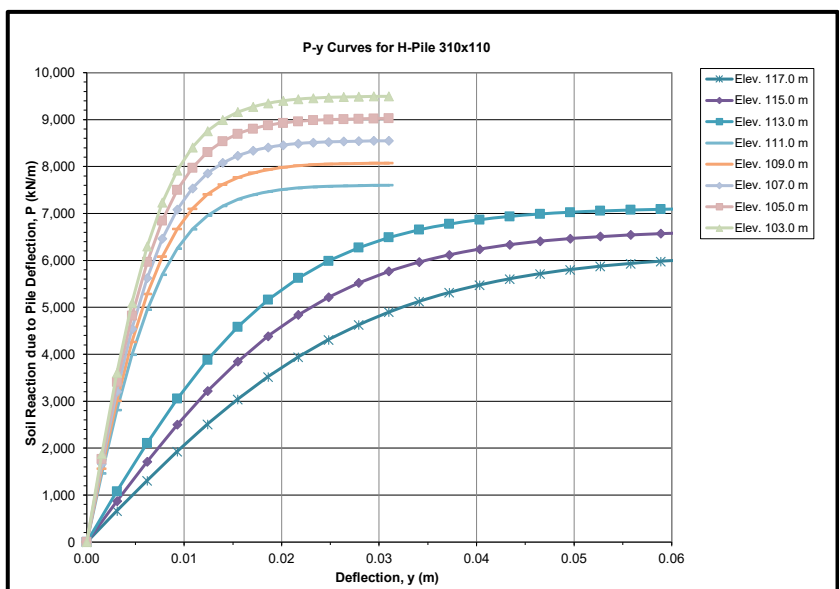
[illegible]

Date: September 2021
Project No: 1786302

Prepared By: ACK
Checked By: CN

Figure 5

Description Depth (z) * Elevation P-y Curves	Firm to Stiff Silty Clay to Clayey Silt-Silt																Hard Silty Clay to Sandy Clayey Silt to Silt Till						Dense to Very Dense Sand										
	z= 13.0 m		z= 14.0 m		z= 15.0 m		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		z= 30.0 m		z= 32.0 m		z= 34.0 m		z= 36.0 m		
	Elev. 126.0 m		Elev. 125.0 m		Elev. 124.0 m		Elev. 123.0 m		Elev. 122.0 m		Elev. 121.0 m		Elev. 120.0 m		Elev. 119.0 m		Elev. 117.0 m		Elev. 115.0 m		Elev. 113.0 m		Elev. 111.0 m		Elev. 109.0 m		Elev. 107.0 m		Elev. 105.0 m		Elev. 103.0 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
	0.00271	36.54	0.00271	36.54	0.00271	36.54	0.00271	36.54	0.00271	36.54	0.00271	36.54	0.00271	36.54	0.0031	663.672632	0.0031	872.00827	0.0031	1079.5221	0.00155	1457.3596	0.00155	1561.1193	0.00155	1664.9032	0.00155	1768.65957	0.00155	1872.4129	0.00155	1976.1702	
0.00543	46.03	0.00543	46.03	0.00543	46.03	0.00543	46.03	0.00543	46.03	0.00543	46.03	0.00543	46.03	0.0062	1312.22842	0.0062	1714.5921	0.0062	2110.6811	0.0031	2811.59362	0.0031	3009.9802	0.0031	3208.3124	0.0031	3406.59362	0.0031	3604.888	0.0031	3803.1792		
0.00814	52.7	0.00814	52.7	0.00814	52.7	0.00814	52.7	0.00814	52.7	0.00814	52.7	0.00814	52.7	0.0093	1931.91947	0.0093	2502.2508	0.0093	3053.4295	0.00465	3986.7298	0.00465	4644.3444	0.00465	4541.9241	0.00465	4819.22979	0.00465	5096.544	0.00465	5373.869		
0.01085	58	0.01085	58	0.01085	58	0.01085	58	0.01085	58	0.01085	58	0.01085	58	0.0124	2513.46842	0.0124	3215.9602	0.0124	3881.5453	0.0062	4947.4681	0.0062	5286.692	0.0062	5625.7487	0.0062	5964.5617	0.0062	6303.1722	0.0062	6641.7827		
0.01356	62.48	0.01356	62.48	0.01356	62.48	0.01356	62.48	0.01356	62.48	0.01356	62.48	0.01356	62.48	0.0155	3844.8283	0.0155	5042.57053	0.0155	6248.3347	0.00775	7595.4021	0.00775	7909.723	0.00775	8263.6877	0.00775	8617.6483	0.00775	8971.6089	0.00775	9325.5695		
0.01628	66.39	0.01628	66.39	0.01628	66.39	0.01628	66.39	0.01628	66.39	0.01628	66.39	0.01628	66.39	0.0186	3520.42105	0.0186	4385.3089	0.0186	5161.3168	0.0093	6255.7447	0.0093	6671.5166	0.0093	7087.0861	0.0093	7502.19149	0.0093	7916.997	0.0093	8331.8022		
0.01899	69.89	0.01899	69.89	0.01899	69.89	0.01899	69.89	0.01899	69.89	0.01899	69.89	0.01899	69.89	0.0217	3943.23421	0.0217	4839.9257	0.0217	5625.0168	0.01085	6663.5085	0.01085	7100.5925	0.01085	7537.3321	0.01085	7973.65532	0.01085	8409.673	0.01085	8845.8905		
0.0217	73.08	0.0217	73.08	0.0217	73.08	0.0217	73.08	0.0217	73.08	0.0217	73.08	0.0217	73.08	0.0248	4311.92421	0.0248	5215.956	0.0248	5989.7726	0.0124	6954.2	0.0124	7495.1497	0.0124	7885.8449	0.0124</							



GOLDER

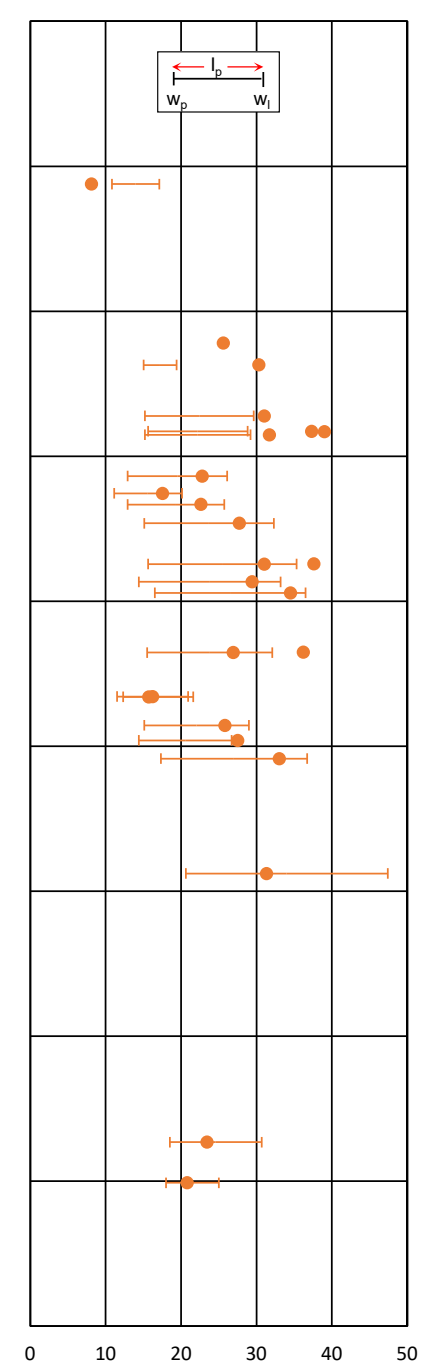
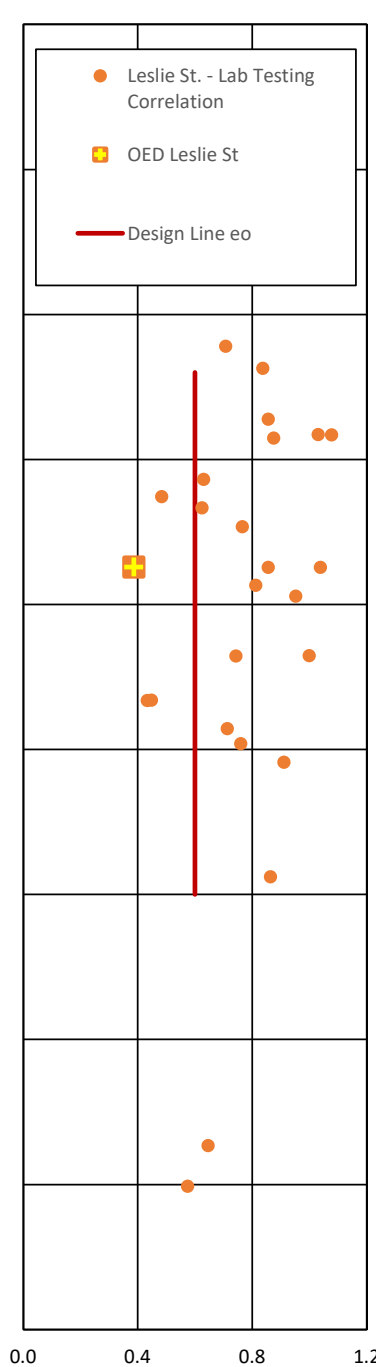
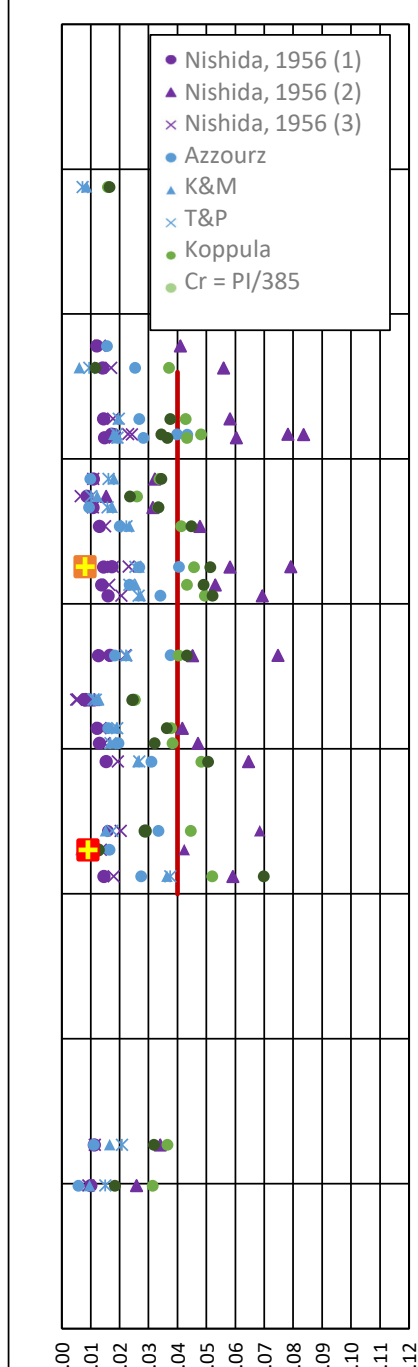
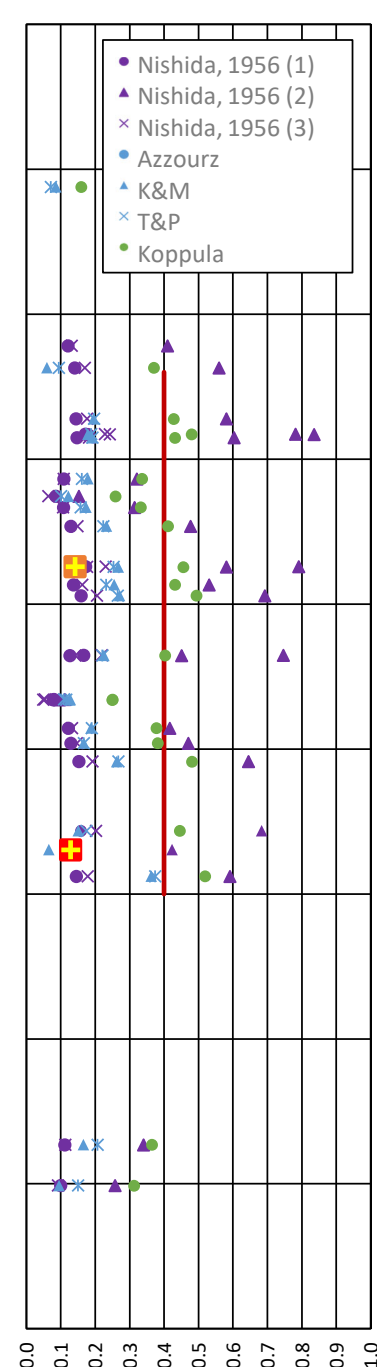
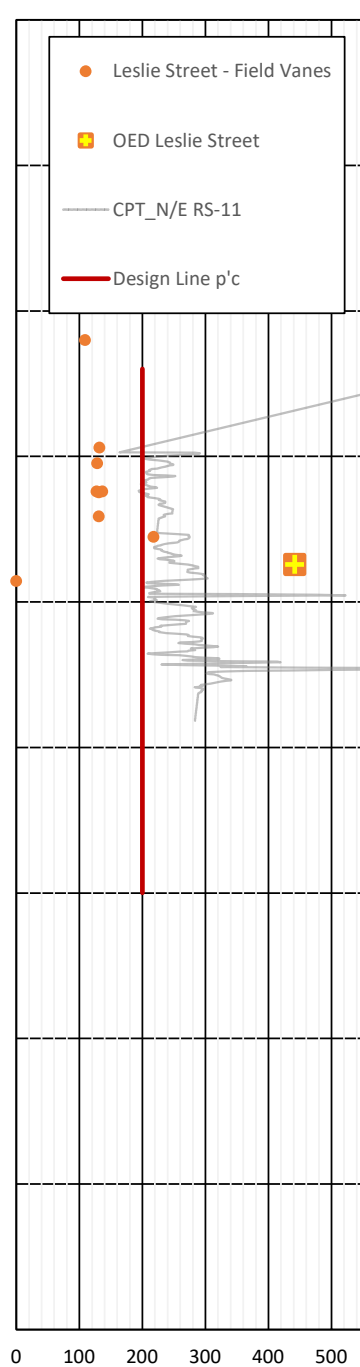
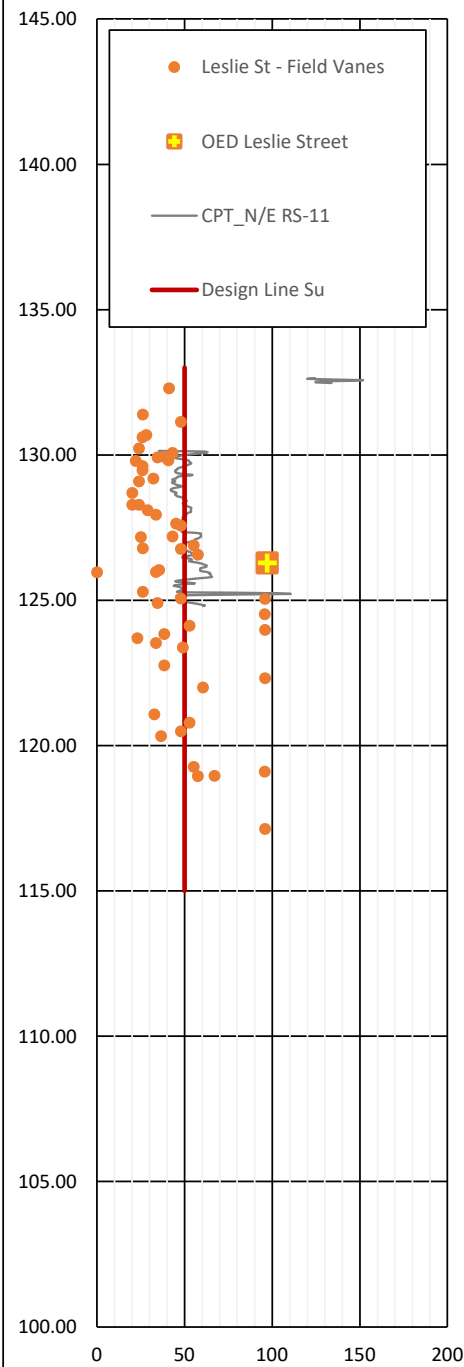
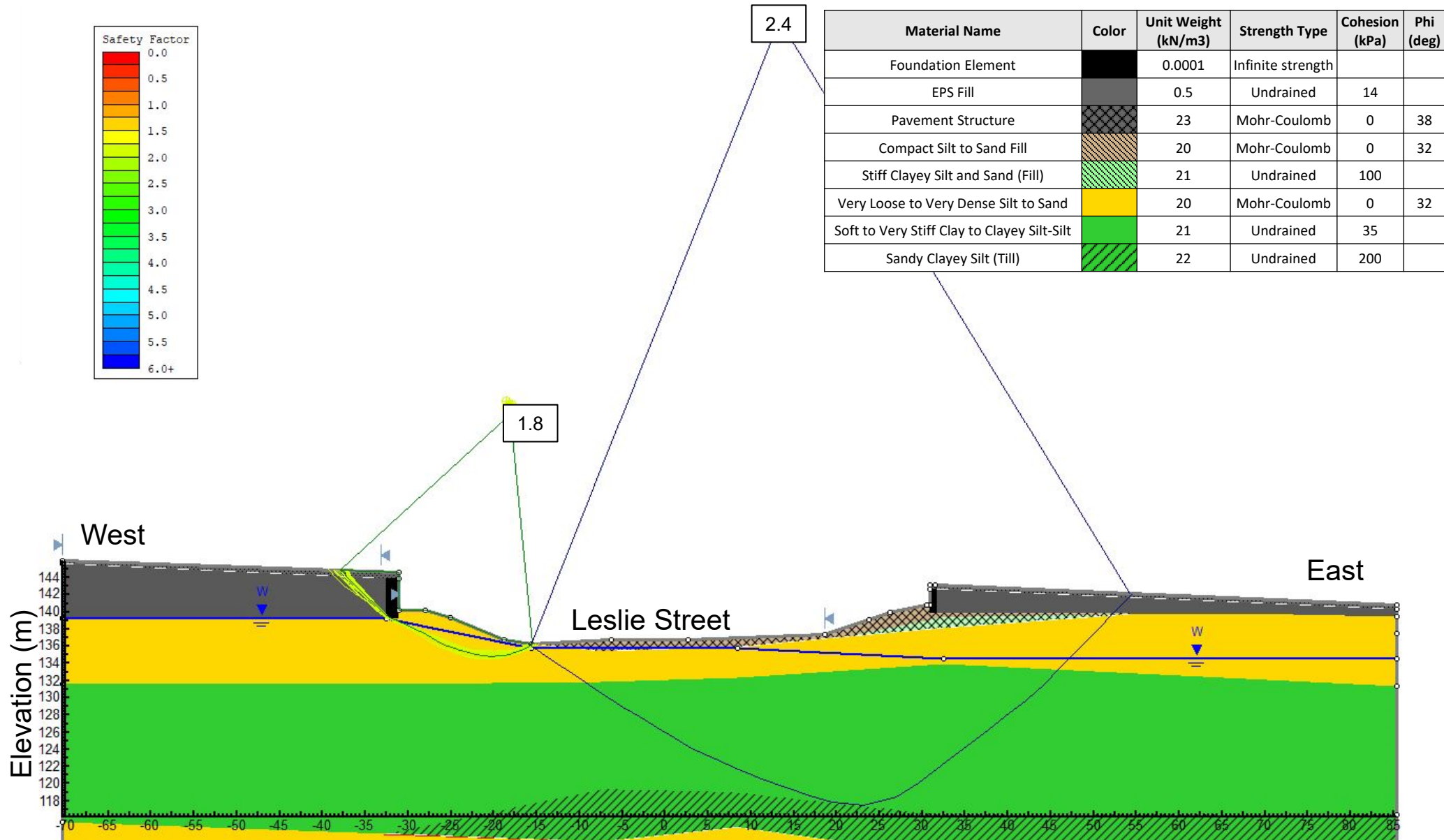


FIGURE 6

Highway 401 EBC, Leslie St. Overpass East and West Embankments

Static Slope Stability Analysis – Temporary Conditions

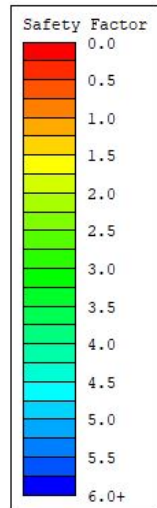
Figure 7



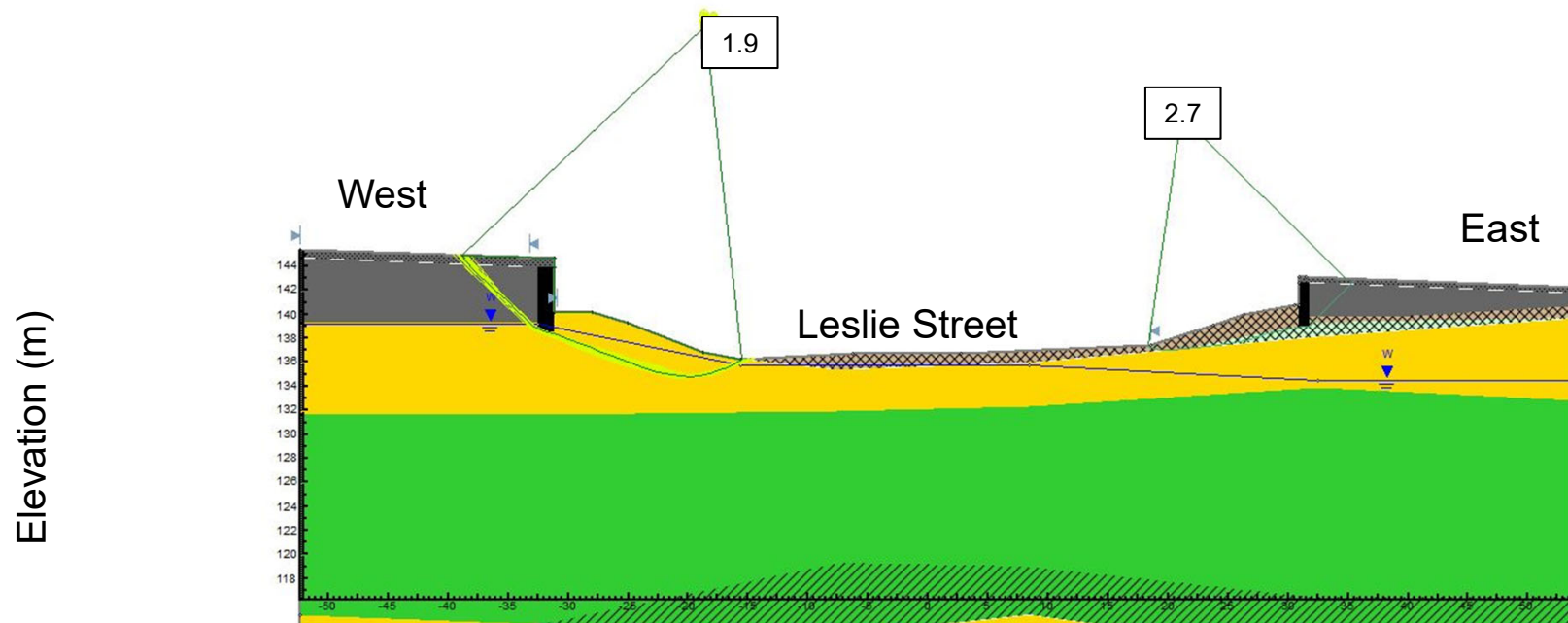
Highway 401 EBC, Leslie St. Overpass East and West Embankments

Static Slope Stability Analysis – Permanent Conditions

Figure 8



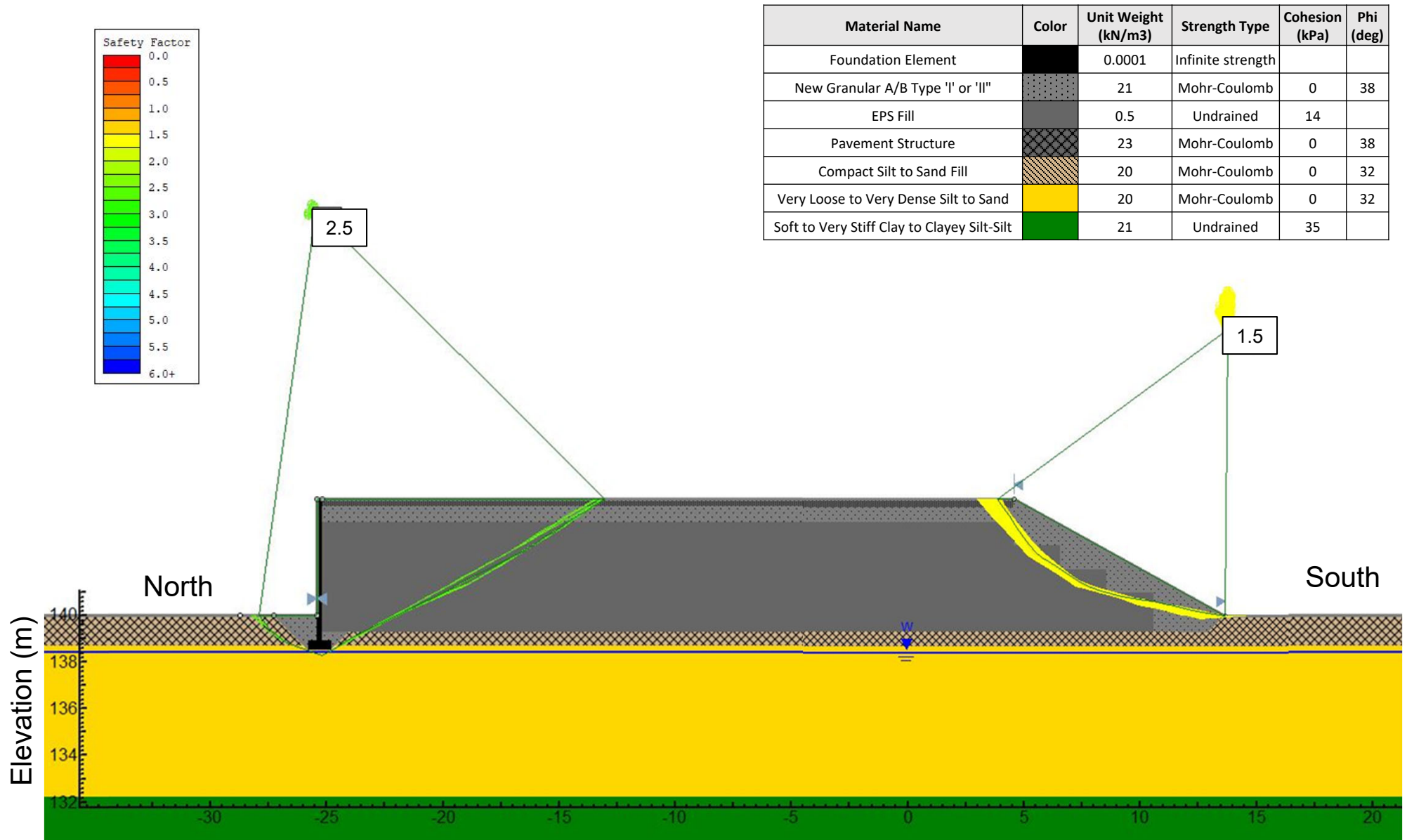
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Foundation Element		0.0001	Infinite strength		
EPS Fill		0.5	Undrained	14	
Pavement Structure		23	Mohr-Coulomb	0	38
Compact Silt to Sand Fill		20	Mohr-Coulomb	0	32
Stiff Clayey Silt and Sand (Fill)		21	Mohr-Coulomb	0	28
Very Loose to Very Dense Silt to Sand		20	Mohr-Coulomb	0	32
Soft to Very Stiff Clay to Clayey Silt-Silt		21	Mohr-Coulomb	0	33
Sandy Clayey Silt (Till)		22	Mohr-Coulomb	0	37



Highway 401 EBC, Leslie St. Overpass West Approach

Static Slope Stability Analysis – Temporary Conditions

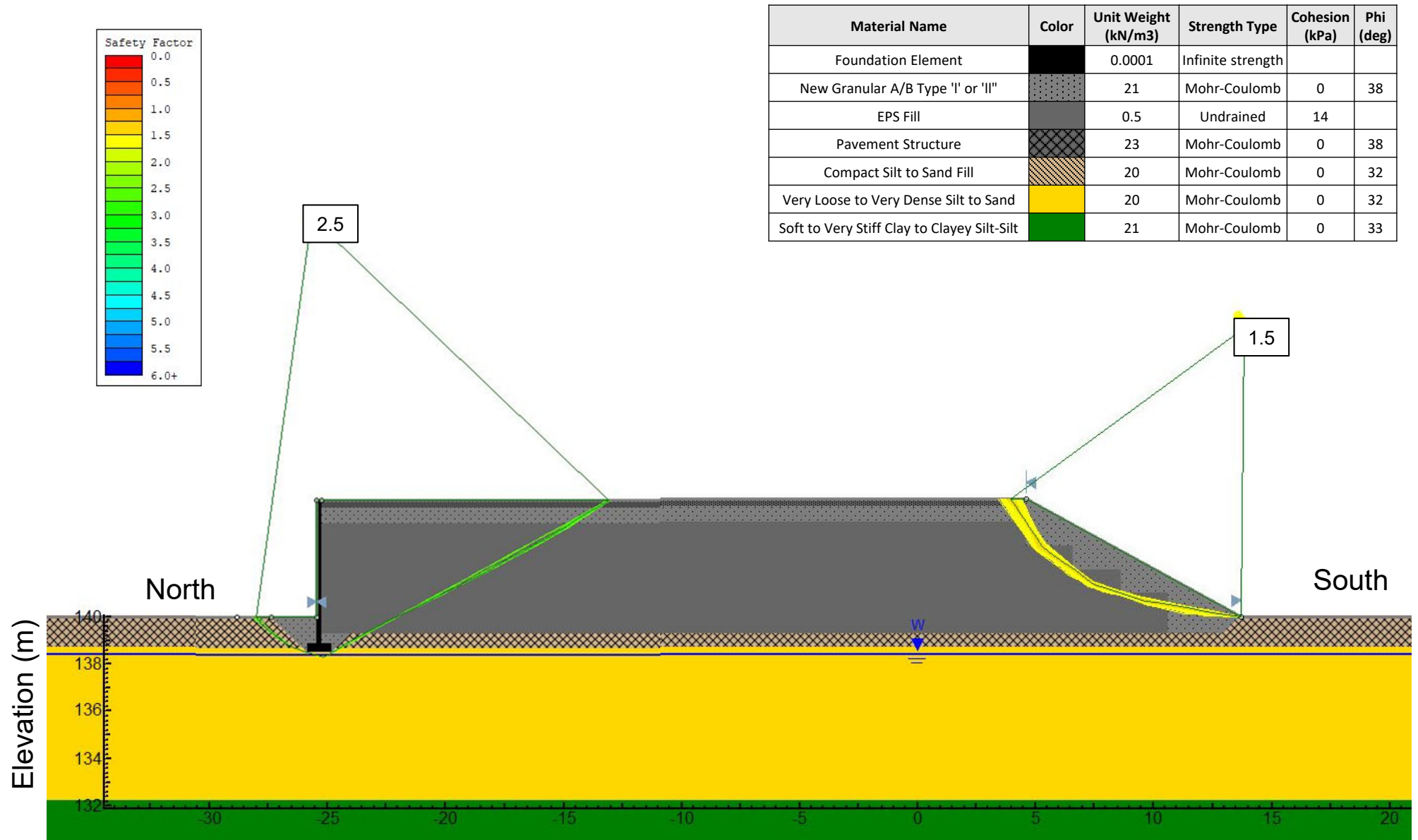
Figure 9



Highway 401 EBC, Leslie St. Overpass West Approach

Static Slope Stability Analysis – Permanent Conditions

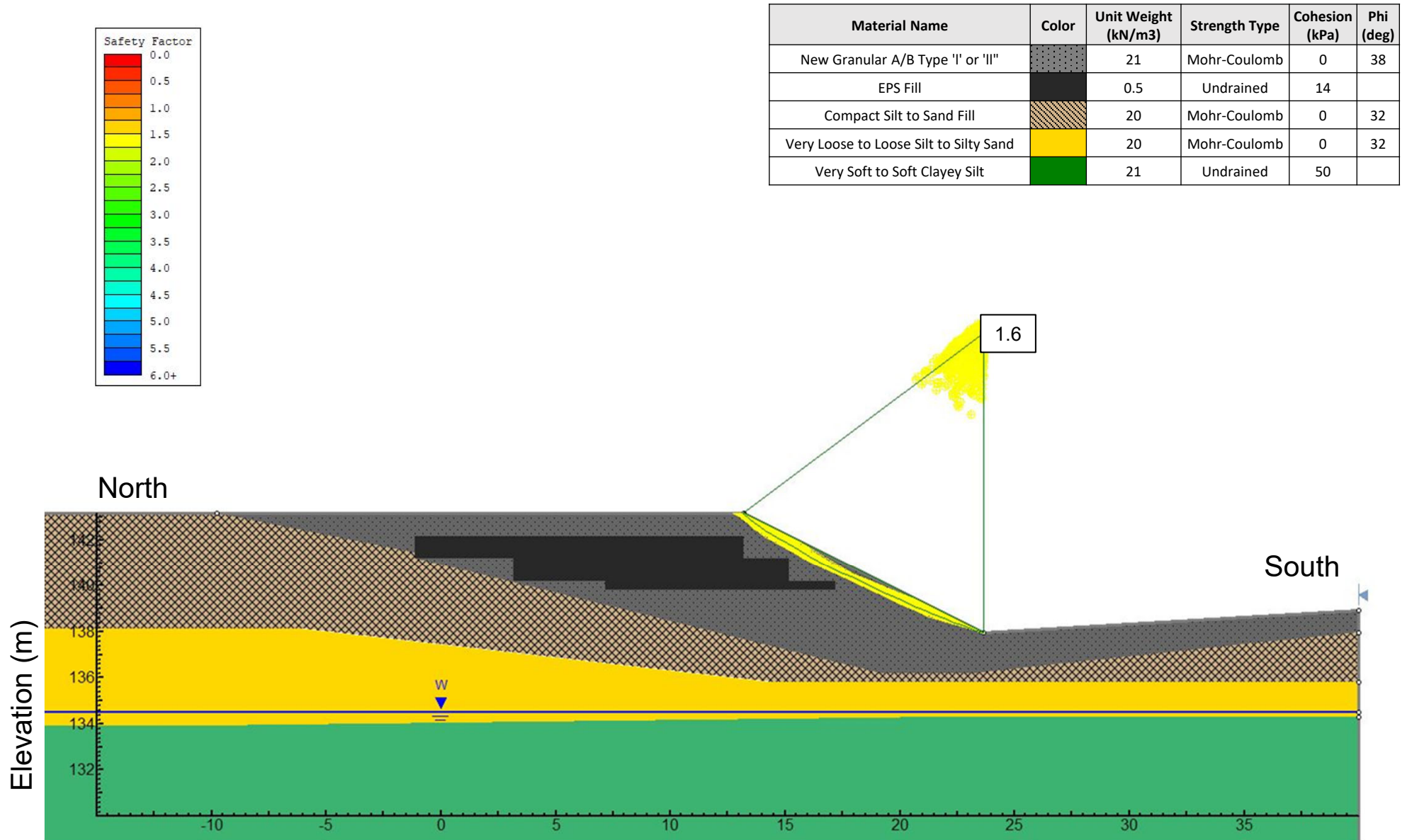
Figure 10



Highway 401 EBC, Leslie St. Overpass East Approach

Static Slope Stability Analysis – Temporary Conditions

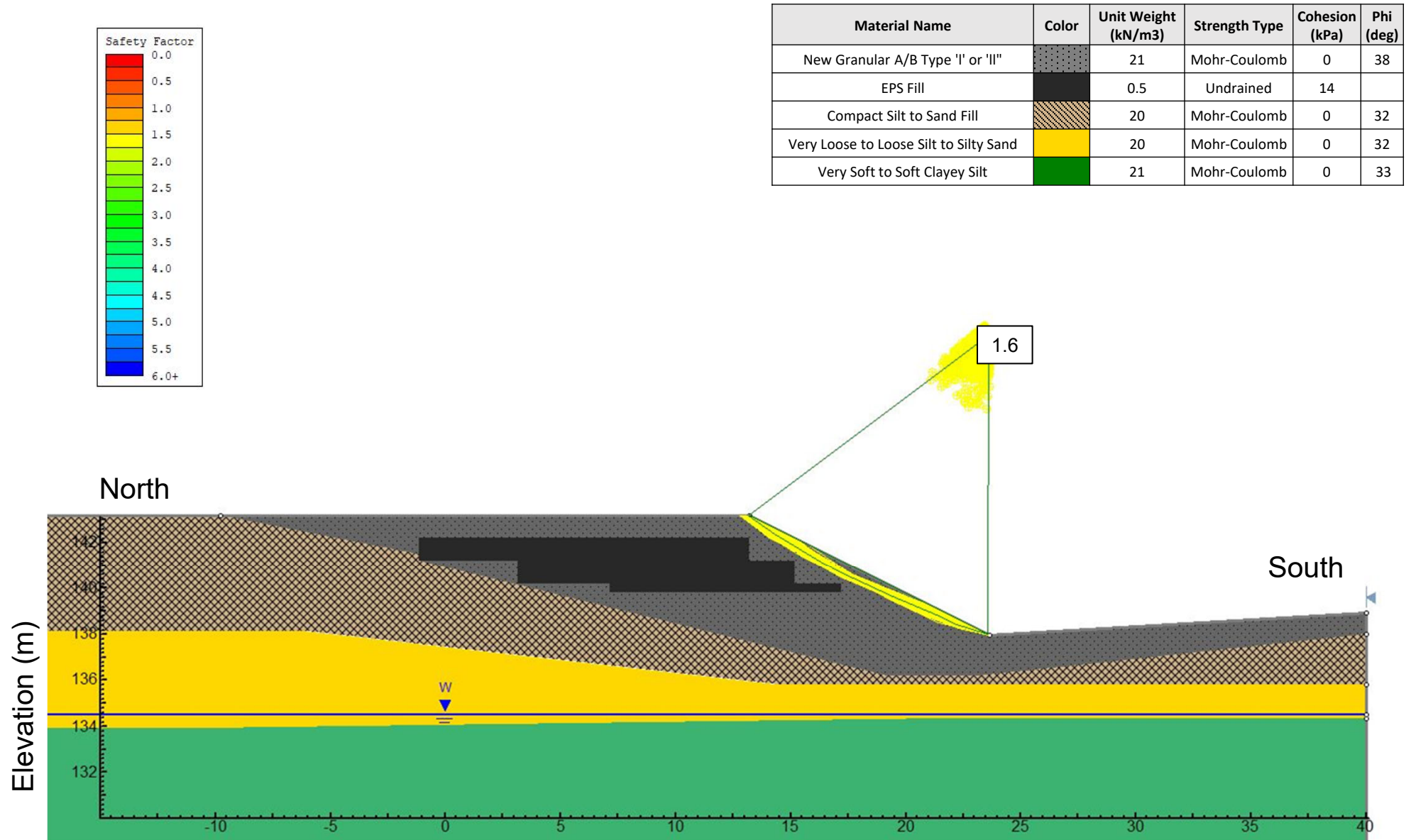
Figure 11



Highway 401 EBC, Leslie St. Overpass East Approach

Static Slope Stability Analysis – Permanent Conditions

Figure 12



APPENDIX A

**Previous Investigations
(MTO GEOCRES NO. 30M14-460 & 30M14-463)**



GEOCRES No. 30M14-460

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

EXPLANATION OF ROCK LOGGING TERMS


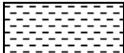



ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	Approximate Uniaxial Compressive Strength (psi)	Field Estimation of Hardness*
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

RECORD OF BOREHOLE No WM 16-07

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION N 4 847 321.4 E 315 822.2 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2016.10.25 - 2016.10.25 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
141.5	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL , some clay, trace gravel, roots and rootlets Compact Dark Brown Moist		1	SS	16		141											
141.1																		
0.4																		
	Silty SAND , some clay, trace gravel Compact to Dense Brown Moist (FILL) Occasional organic seams		2	SS	30		140											6 50 28 16
				3	SS	45												
139.2																		
2.3	SAND , fine grained, some silt to silty, trace gravel Compact Brown Moist to Wet		4	SS	14		139											
				5	SS	26		138										0 80 18 2
				6	SS	21												0 84 14 2
							137											
			7	SS	14													
			8	SS	13		136											
135.5																		
6.0	Loose to Very Loose		9	SS	5		135											
	Saturated		10	SS	2		134											
							133											
131.9			11	SS	4		132											
9.6	Silty CLAY , some sand, trace gravel Soft to Firm Grey																	

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No WM 16-07

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION N 4 847 321.4 E 315 822.2 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2016.10.25 - 2016.10.25 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	W _P	W	W _L					
	Continued From Previous Page							20	40	60	80	100							
122.6 <																			

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No WM 16-07

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION N 4 847 321.4 E 315 822.2 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2016.10.25 - 2016.10.25 CHECKED BY SKP

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page																
	WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2016.11.09 3.0 138.5 2016.12.21 4.5 137.0																

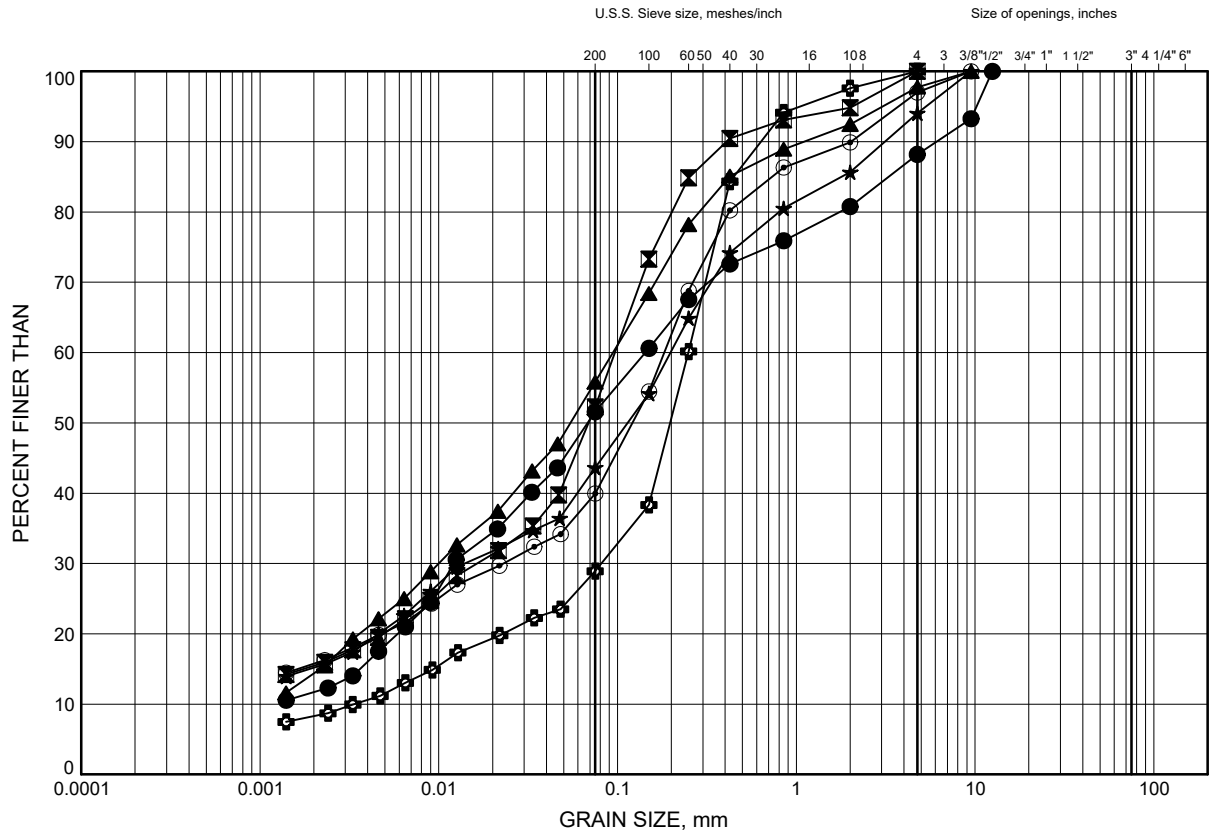


Appendix B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

SAND and SILT / Silty SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	WM 16-02	2.59	142.81
⊠	WM 16-04	1.83	139.07
▲	WM 16-05	2.59	138.91
★	WM 16-07	1.07	140.43
⊙	WM 16-08	0.30	140.20
⊕	WM 16-09	1.83	138.77

Date December 2016

W.P. 2061-13-00



Prep'd AN

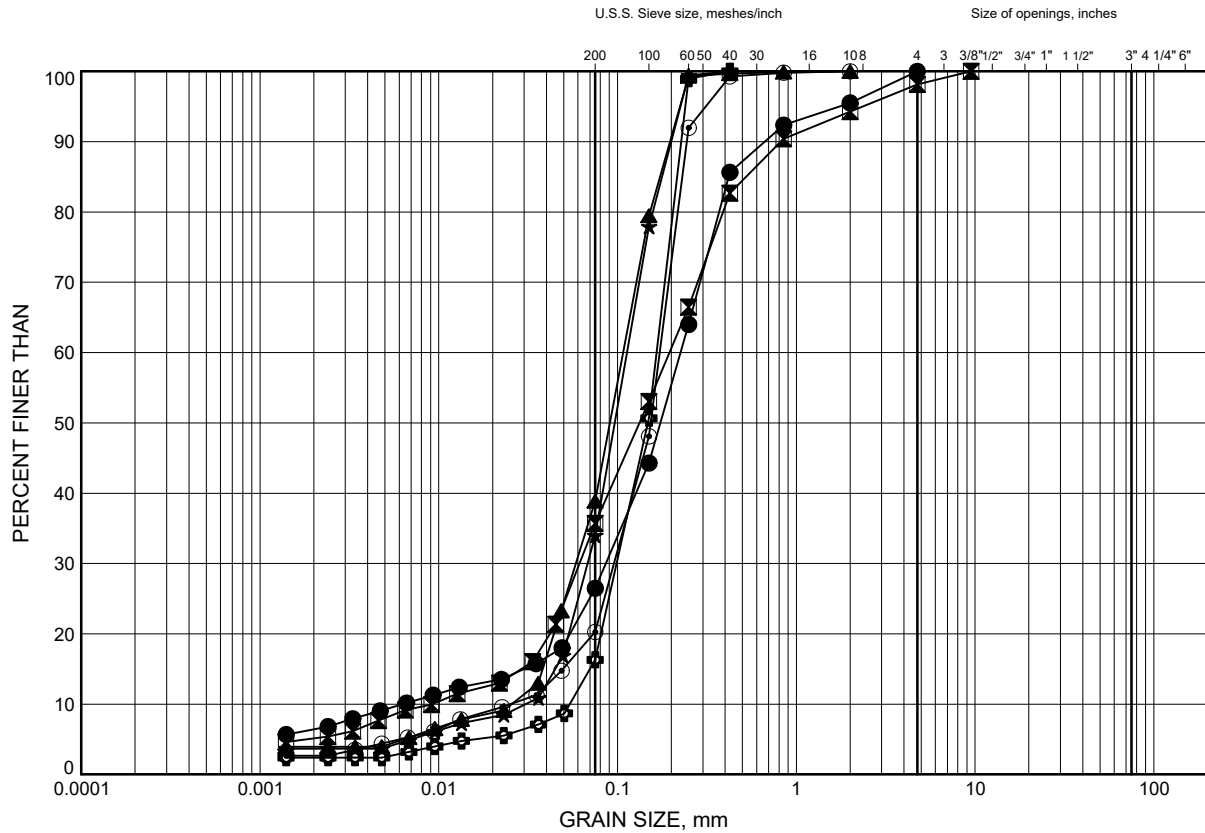
Chkd. SKP

Relocated Watermain Crossing Hwy 401 at Leslie

GRAIN SIZE DISTRIBUTION

FIGURE A2

SAND / Silty SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	WM 16-01	7.92	133.48
⊠	WM 16-04	7.92	132.98
▲	WM 16-05	4.88	136.62
★	WM 16-06	4.11	136.79
⊙	WM 16-07	3.35	138.15
⊕	WM 16-07	4.11	137.39

Date December 2016
W.P. 2061-13-00



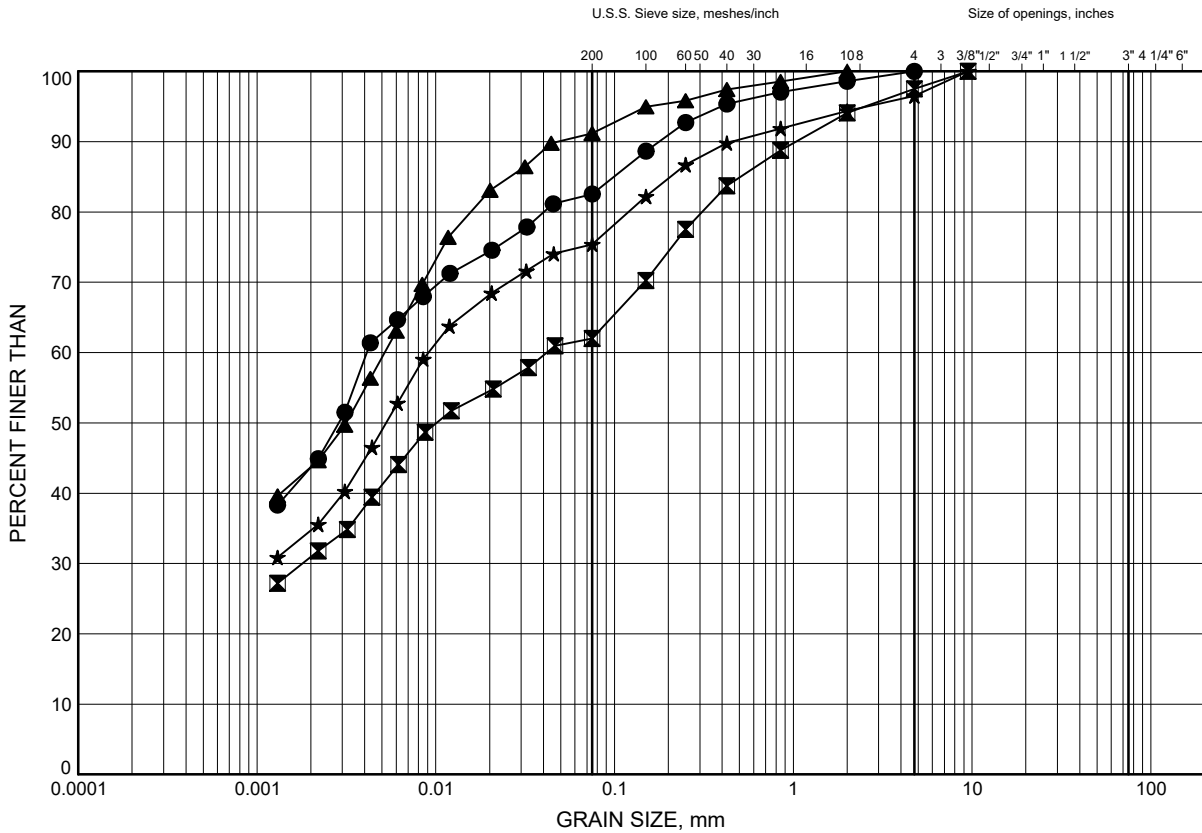
Prep'd AN
Chkd. SKP

Relocated Watermain Crossing Hwy 401 at Leslie

GRAIN SIZE DISTRIBUTION

FIGURE A3

Silty CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	WM 16-01	9.45	131.95
◻	WM 16-04	12.50	128.40
▲	WM 16-06	9.45	131.45
★	WM 16-07	10.97	130.53

Date December 2016
W.P. 2061-13-00



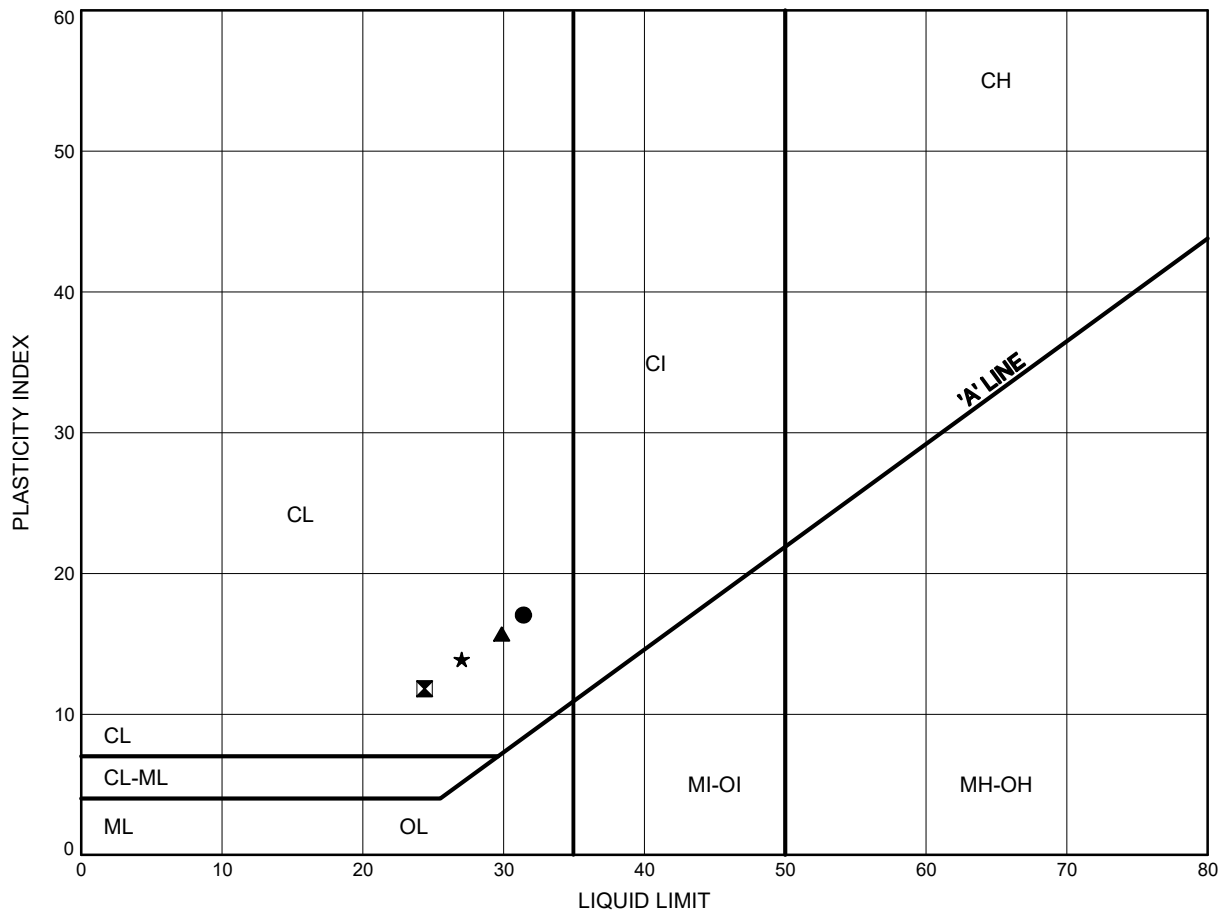
Prep'd AN
Chkd. SKP

Relocated Watermain Crossing Hwy 401 at Leslie

ATTERBERG LIMITS TEST RESULTS

FIGURE A4

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	WM 16-01	9.45	131.95
⊠	WM 16-04	12.50	128.40
▲	WM 16-06	9.45	131.45
★	WM 16-07	10.97	130.53

Date December 2016
W.P. 2061-13-00



Prep'd AN
Chkd. SKP

GEOCRES No. 30M14-463

Appendix A

Record of Borehole Sheets

19-5161-205

RECORD OF BOREHOLE No M-04

1 OF 4

METRIC

W.P. 2061-13-00 LOCATION N 4 847 328.9 E 315 910.0 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.17 - 2015.04.20 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT 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				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE				W _P W W _L				
								● QUICK TRIAXIAL × LAB VANE								
140.2	GROUND SURFACE															
0.0 0.1	TOPSOIL: (75mm)						140									
	SAND and SILT, trace gravel, some clay Compact to Loose Brown Moist (FILL)		1	SS	19		139									
	Occasional roots		2	SS	8		138								5 41 43 11	
	Brown to Grey		3	SS	8		137									
			4	SS	8		136									
135.5							135									
4.7	SAND and SILT, some clay Dense Brown Moist Possible cobbles		5	SS	50		134									
134.4							133									
5.8	SAND, some silt, some clay Compact Brown Wet		6	SS	16		132								0 12 78 10	
133.0							131									
7.2			7	SS	6											
	Loose Wet															
130.9																
9.3	Silty CLAY, trace to some sand Soft Grey Moist		8	SS	1											

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
15
10
5
0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M-04

2 OF 4

METRIC

W.P. 2061-13-00 LOCATION N 4 847 328.9 E 315 910.0 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.17 - 2015.04.20 CHECKED BY RPR

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 (%)							
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						
	Continued From Previous Page						20	40	60	80	100	20	40	60									
	Silty CLAY , trace sand Firm to Stiff Grey Wet								3.0														
			1	TW	PH								○										
									2.9														
			9	SS	2								○										
									3.0														
			10	SS	3									4			0	6	31	63			
									3.1														
			2	TW	PH								○										
	Very Stiff																						
123.4																							
16.8	Layer of sandy silt		11	SS	3									○									
122.8																							
17.4																							
			12	SS	6									○									
									3.3														

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M-04

3 OF 4

METRIC

W.P. 2061-13-00 LOCATION N 4 847 328.9 E 315 910.0 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.04.17 - 2015.04.20 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL 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	
								20 40 60 80 100	W _p W W _L											
	Continued From Previous Page							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
								20 40 60 80 100				20 40 60								

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M-04

4 OF 4

METRIC

W.P. 2061-13-00 LOCATION N 4 847 328.9 E 315 910.0 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.17 - 2015.04.20 CHECKED BY RPR

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									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20 40 60 80 100 20 40 60 80 100					20	40	60				
109.5	Continued From Previous Page						110										
30.7	Silty CLAY , some sand, trace gravel Hard Grey Moist (TILL) Sand layer (50mm) at 30.6m END OF BOREHOLE AT 30.7m. WATER LEVEL AT 6.8m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.		19	SS	100/ 0.200												

Appendix B

Laboratory Test Results

Grain Size Analysis

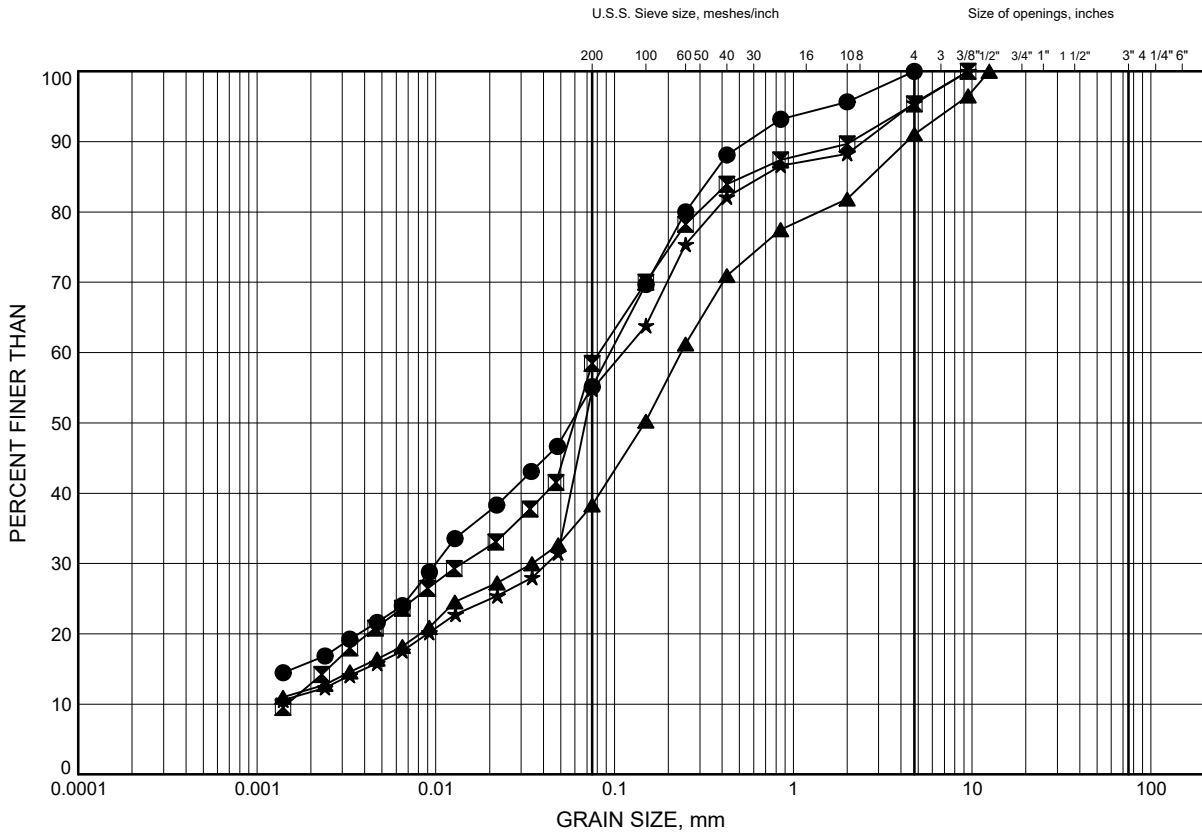
Atterberg Limits Test Results

19-5161-205

Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE **A5**

SAND & SILT / Silty SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-01	3.24	140.46
⊠	M-02	4.88	136.32
▲	M-03	4.88	140.42
★	M-04	1.83	138.37

Date March 2016
W.P. 2061-13-00

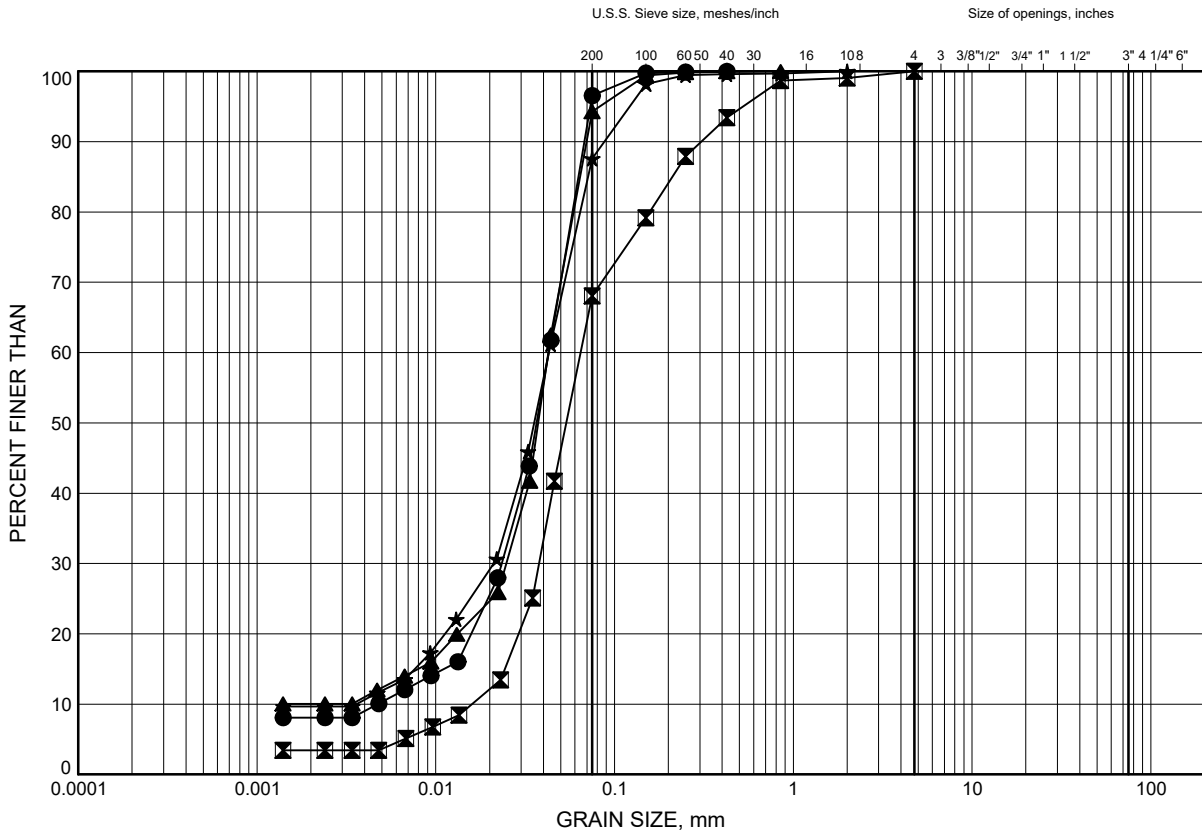


Prep'd AN
Chkd. SKP

Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE **A6**

SANDS & SILTS



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-01	7.92	135.78
⊠	M-02	7.92	133.28
▲	M-03	10.97	134.33
★	M-04	6.40	133.80

Date March 2016
W.P. 2061-13-00

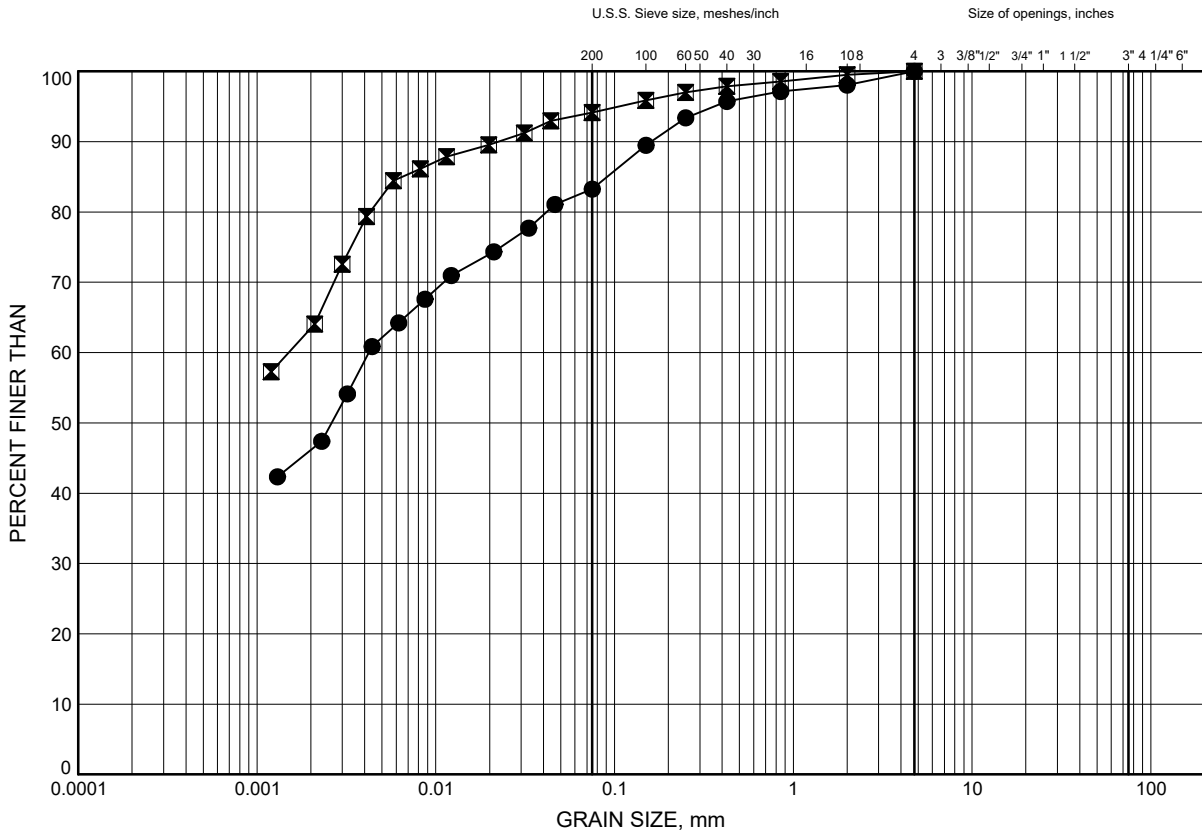


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Chkd. SKP

Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE **A7**

Silty CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-03	15.54	129.76
⊠	M-04	14.02	126.18

Date March 2016
W.P. 2061-13-00



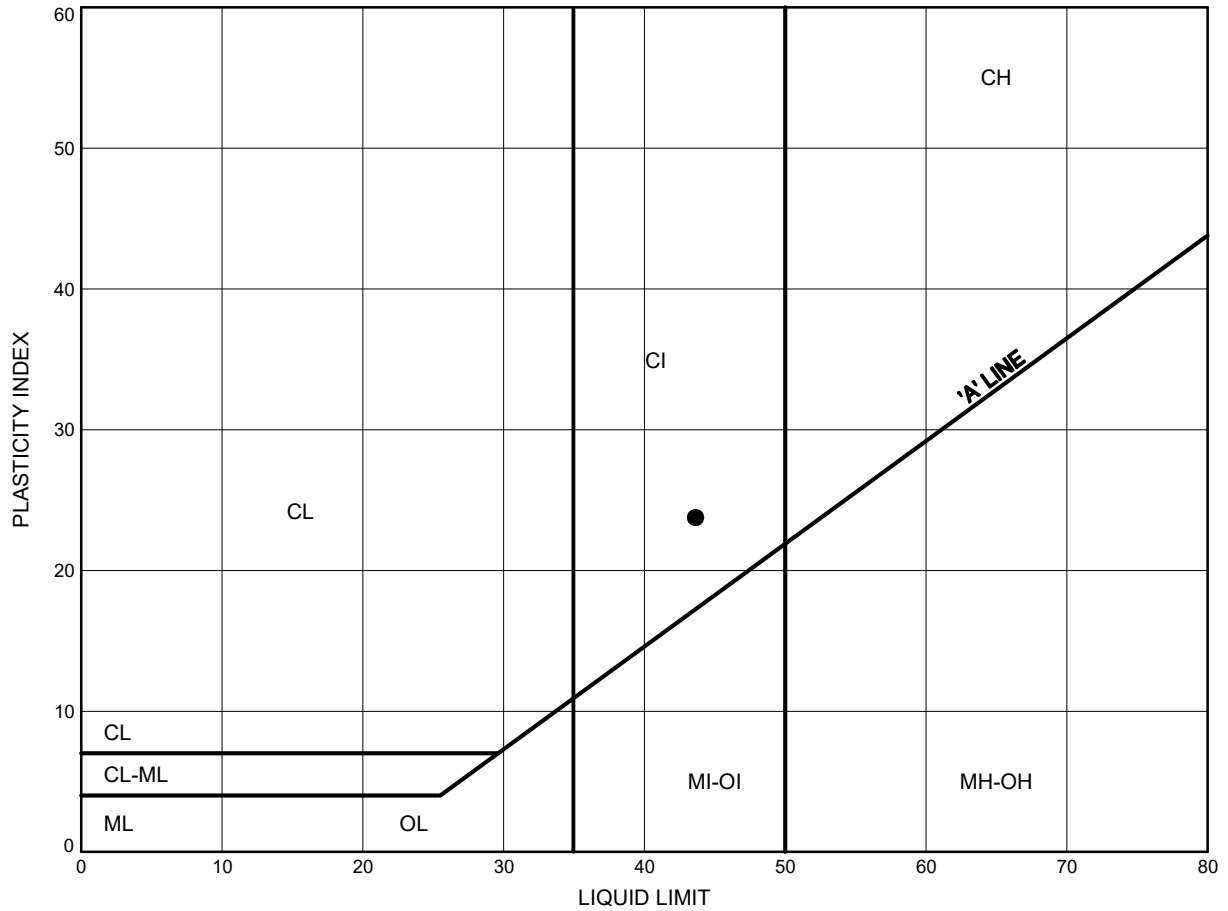
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Chkd. SKP

Hwy 401 Leslie Street 2013-E-0032

ATTERBERG LIMITS TEST RESULTS

FIGURE **A8**

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-04	14.02	126.18

Date March 2016
W.P. 2061-13-00

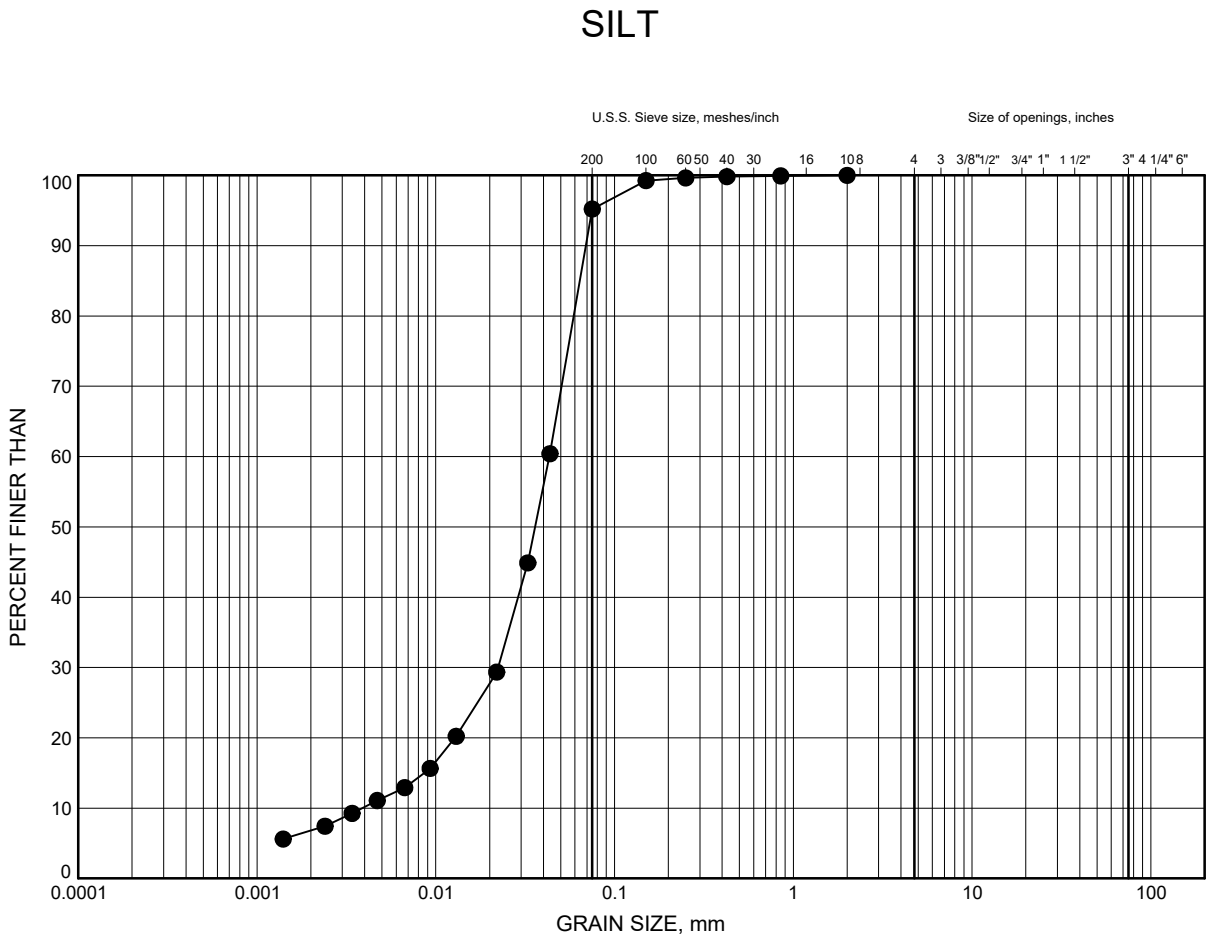


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Hwy 401 Leslie Street 2013-E-0032

GRAIN SIZE DISTRIBUTION

FIGURE **A9**



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-04	24.69	115.51

Date March 2016
W.P. 2061-13-00



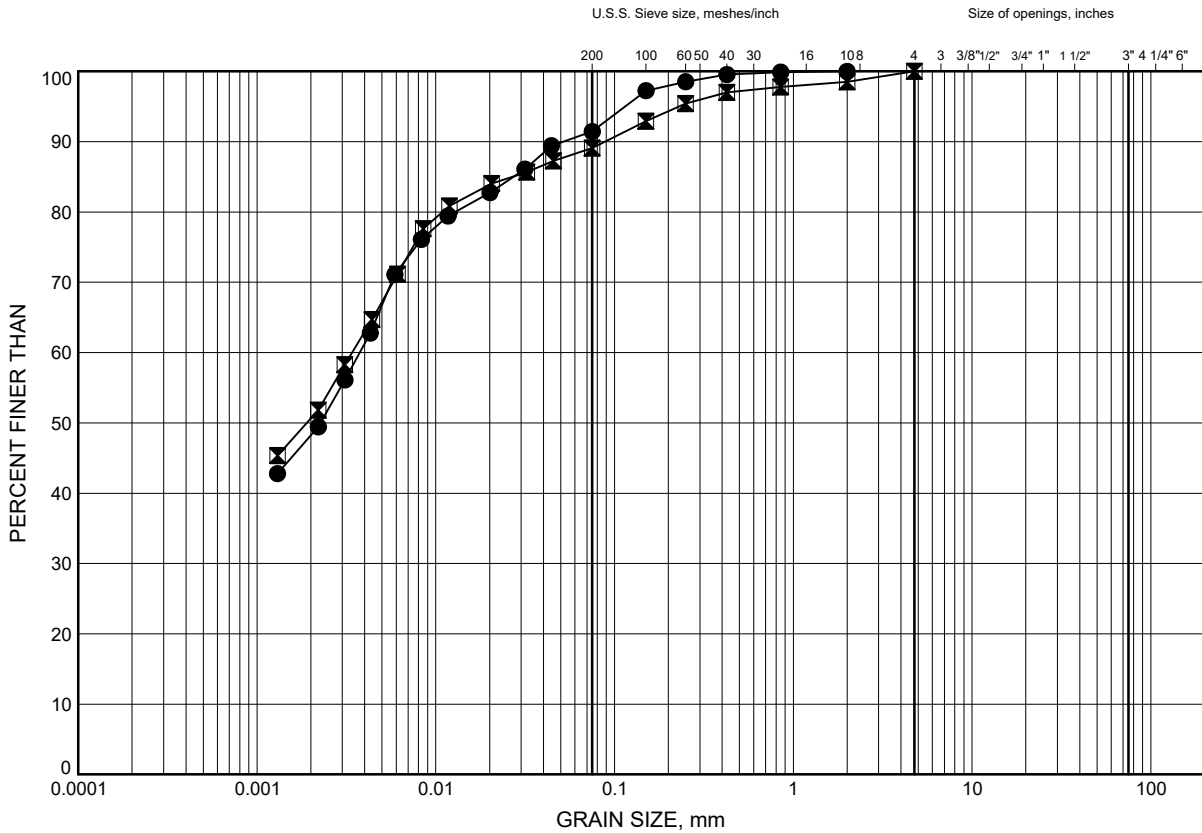
Prep'd AN
Chkd. SKP

Hwy 401 Leslie Street 2013-E-0032

GRAIN SIZE DISTRIBUTION

FIGURE A10

Silty CLAY TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-03	27.74	117.56
⊠	M-04	29.26	110.94

Date March 2016
W.P. 2061-13-00



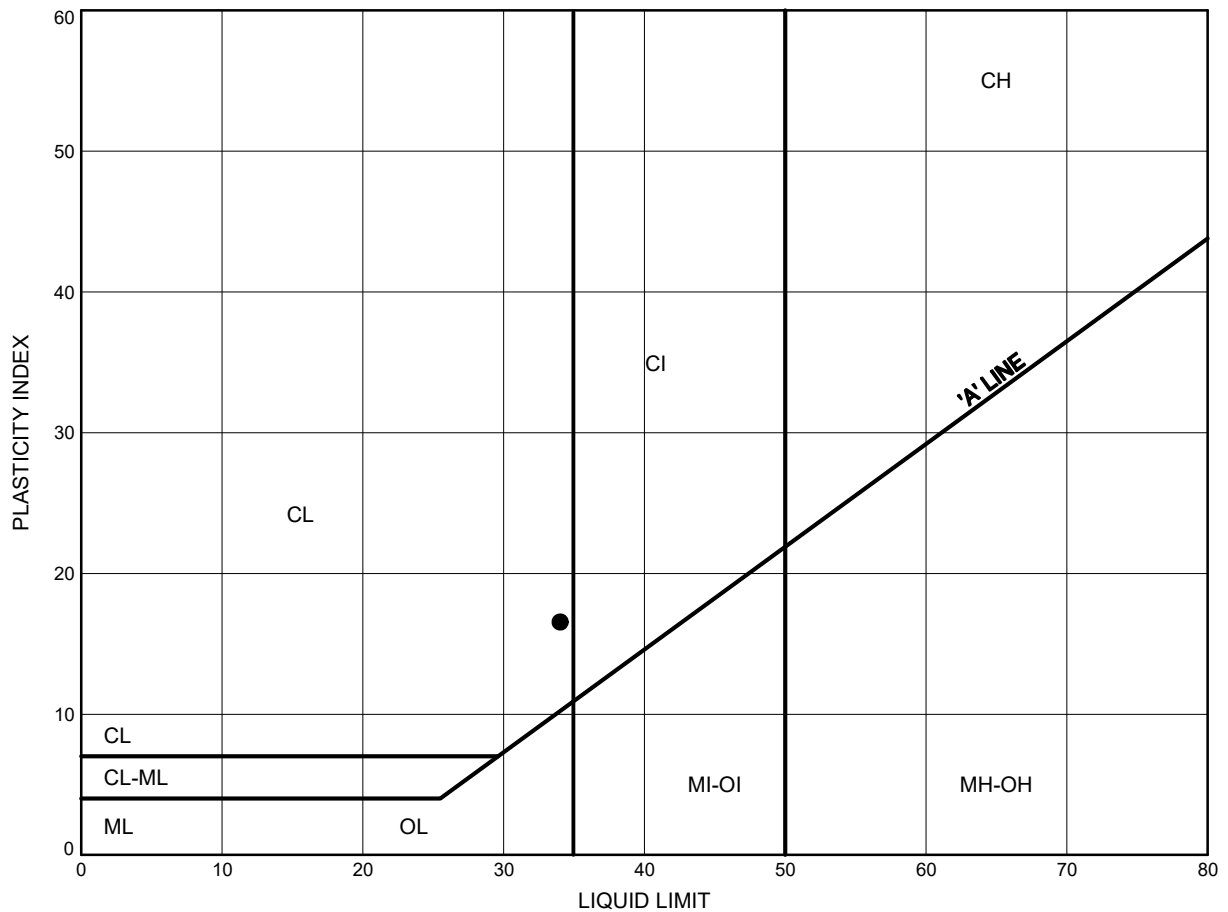
Prep'd AN
Chkd. SKP

Hwy 401 Leslie Street 2013-E-0032

ATTERBERG LIMITS TEST RESULTS

FIGURE **A11**

Silty CLAY TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	M-04	29.26	110.94

Date March 2016
W.P. 2061-13-00



Prep'd AN
Chkd. SKP

APPENDIX B

**Current Investigation –
Record of Boreholes/Drillholes, Cone Penetration Tests,
Pavement Core Photographs
and Bedrock Core Photographs**

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

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

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

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

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

COARSE-GRAINED SOILS

Compactness¹

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

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

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

FINE-GRAINED SOILS

Consistency

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

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

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

Field Moisture Condition

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

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-2					SHEET 1 OF 4		METRIC			
G.W.P. 2130-01-00		LOCATION N 4847306.9; E 315815.9 MTM NAD 83 ZONE 10 (LAT. 43.765648; LONG. -79.363182)					ORIGINATED BY DH					
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)					COMPILED BY RM					
DATUM Geodetic		DATE June 19 to 21, 2019					CHECKED BY DH					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W_p	NATURAL MOISTURE CONTENT W		
142.0	GROUND SURFACE											
0.0	TOPSOIL (130 mm)		1	SS	5							
0.1	CLAYEY SILT (CL), trace gravel, sandy to some sand, trace to some rootlets (FILL) Firm Brown to dark brown Moist		2	SS	7							
140.3												
1.7	SAND (SP-SM), trace gravel, some fines, wood chips and brick fragments (FILL) Compact to dense Brown to grey to black Moist		3A 3B	SS	34							
139.4			4A 4B	SS	22							
139.0	SANDY CLAYEY SILT (CL), trace gravel (FILL) Very stiff Grey Moist		5	SS	16							
3.0												
137.9	SILTY SAND (SM), trace rootlets Compact Dark brown Moist		6	SS	14							
4.1												
	SILT (ML), trace sand Very loose to dense Brown Moist											
	- wet below a depth of 2.6 m below ground surface (Elev. 135.9 m)		7	SS	11							
			8	SS	3							
			9	SS	31							
131.8												
10.2	CLAY (CH) to CLAYEY SILT (CL), trace gravel, trace to some sand Soft to firm Grey Moist		10	SS	WH							
			11	TO	-							
			12	SS	WH							
								</				

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

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

PROJECT				RECORD OF BOREHOLE No N/E RS-2								SHEET 3 OF 4						METRIC					
G.W.P.				LOCATION				N 4847306.9; E 315815.9 MTM NAD 83 ZONE 10 (LAT. 43.765648; LONG. -79.363182)				ORIGINATED BY				DH							
DIST				BOREHOLE TYPE				Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY				RM							
DATUM				DATE				June 19 to 21, 2019				CHECKED BY				DH							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT 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 + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)										
						20 40 60 80 100					10 20 30												
<div>-- CONTINUED FROM PREVIOUS PAGE --</div>																							
SILTY SAND (SM) Very dense Brown Wet						20 SS 81													0 85 13 2				
						21 SS 74																	
107.0 35.1 Sandy CLAYEY SILT (CL) Hard Grey Moist																							
105.3						22A SS 83																	
104.8 37.2 Sandy CLAYEY SILT (CL), some gravel, containing shale fragments (RESIDUAL SOIL) Hard Grey Moist						22B																	
Inferred completely weathered, grey SHALE (Georgian Bay Formation) SHALE (BEDROCK)						1 RC REC 100%													RQD = 38%				
Bedrock cored from a depth of 37.2 m to 41.3 m (between Elev. 104.8 m and 100.8 m).						2 RC REC 100%													RQD = 79%				
For rock coring details refer to Record of Drillhole N/E RS-2.						3 RC REC 100%													RQD = 93%				
100.7 41.3 END OF BOREHOLE																							
NOTE: 1. Borehole dry prior to switching over to casing advancement at a depth of 5.2 m below ground surface (Elev. 136.8 m).																							

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

SHEET 4 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: Geo-environmental Drilling

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CHECKED: DH

PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-3				SHEET 1 OF 4		METRIC								
G.W.P. 2130-01-00		LOCATION N 4847321.1; E 315824.7 MTM NAD 83 ZONE 10 (LAT. 43.765776; LONG. -79.363072)				ORIGINATED BY SEM										
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY RM										
DATUM Geodetic		DATE July 11, 12 and 16, 2019				CHECKED BY DH										
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							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED								
141.2		GROUND SURFACE						20 40 60 80 100								
0.0		TOPSOIL (150 mm)														
0.2		Sandy SILT (ML), some gravel (FILL)		1	SS	15										
140.5		Compact Brown Moist														
0.7		SILTY SAND (SM), some gravel (FILL)		2	SS	21										
139.8		Compact Brown Moist														
1.5		CLAYEY SILT-SILT (CL-ML/SC) and sand, trace gravel (FILL)		3	SS	39										
138.7		Hard Brown Moist														
2.5		SAND (SP-SM), trace fines (FILL)		4A	SS	10										
		Compact Brown Moist		4B												
		- wet below a depth of 4.1 m below ground surface (Elev. 137.1 m)														
				5	SS	19										
				6	SS	23										
135.6		SILT (ML), some sand														
5.6		Loose Brown Wet														
				7	SS	6										
133.6		SILTY SAND (SM)														
7.6		Very loose Grey Wet														
				8	SS	WH										
132.2		CLAYEY SILT (CI) to SILTY CLAY (CL), trace gravel, trace sand to some sand														
9.0		Firm to stiff Grey Moist to wet														
				9	TO	-										
				10	SS	WH										
				11	SS	WH										
				12	SS	WH										

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+³, X³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-3				SHEET 2 OF 4		METRIC							
G.W.P. 2130-01-00		LOCATION N 4847321.1; E 315824.7 MTM NAD 83 ZONE 10 (LAT. 43.765776; LONG. -79.363072)				ORIGINATED BY SEM									
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY RM									
DATUM Geodetic		DATE July 11, 12 and 16, 2019				CHECKED BY DH									
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 (%)
--- CONTINUED FROM PREVIOUS PAGE ---															
118.0 23.2	CLAYEY SILT (CI) to SILTY CLAY (CL), trace gravel, trace sand to some sand Firm to stiff Grey Moist to wet	[Hatched Pattern]	13	SS	1		126								
								125							
								124							
								123							
								122							
								121							
								120							
								119							
								118							
								117							
115.0 26.2	CLAYEY SILT (CL), some sand, some gravel (TILL) Hard Grey Moist	[Dotted Pattern]	16	SS	52		116								
								115							
								114							
								113							
112	SAND (SP), trace gravel, trace fines Compact to very dense Grey Wet	[Dotted Pattern]	17	SS	29		112								

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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-3				SHEET 3 OF 4		METRIC								
G.W.P. 2130-01-00		LOCATION N 4847321.1; E 315824.7 MTM NAD 83 ZONE 10 (LAT. 43.765776; LONG. -79.363072)				ORIGINATED BY SEM										
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY RM										
DATUM Geodetic		DATE July 11, 12 and 16, 2019				CHECKED BY DH										
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								
--- CONTINUED FROM PREVIOUS PAGE ---																
	SAND (SP), trace gravel, trace fines Compact to very dense Grey Wet		18	SS	41		111									
							110									
							109									
							108									
			19	SS	93		107									3 93 4 0
							106									
105.8	Organic SILT (OH), containing sand seams, wood pieces, trace rootlets and peat seams Hard Dark grey to black Moist						105									
35.4			20	SS	56		104									OC=14.1%
103.6	Sandy CLAYEY SILT (CL) (RESIDUAL SOIL) Hard Grey Moist		21	SS	100%		103									RQD = 100%
103.2	SHALE (BEDROCK)						102									RQD = 64%
38.0	Bedrock cored from a depth of 38.0 m to 41.3 m (between Elev. 103.2 m and 99.9 m). For rock coring details refer to Record of Drillhole N/E RS-3.		2	RC	REC 100%		101									RQD = 97%
			3	RC	REC 100%		100									
99.9	END OF BOREHOLE															
41.3	NOTE: 1. Borehole dry prior to switching over to casing advancement at a depth of 3.0 m below ground surface (Elev. 138.2 m).															

SHEET 4 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: Geo-environmental Drilling

[illegible]

DEPTH SCALE

1 : 50

LOGGED: DH

CHECKED: CN





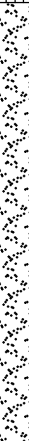

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT 1786302 **RECORD OF BOREHOLE No N/E RS-4** SHEET 2 OF 4 **METRIC**

G.W.P. 2130-01-00 LOCATION N 4847310.3; E 315832.8 MTM NAD 83 ZONE 10 (LAT. 43.765679; LONG. -79.362973) ORIGINATED BY DH

DIST Central HWY 401 BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.) COMPILED BY RM

DATUM Geodetic DATE July 11, 12 and 15, 2019 CHECKED BY DH

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 (%)	
								20 40 60 80 100	20 40 60 80 100						10 20 30	
-- CONTINUED FROM PREVIOUS PAGE --																
119.5 19.8	CLAYEY SILT (CI) to SILTY CLAY (CL), trace gravel, trace sand Firm Grey Moist		TO	SS	-		124									
							123									
							122									
			14	SS	WR		121									
							120									
116.1 23.2	SILT (ML), trace sand Dense Grey Wet						119									
			15	SS	47		118						0 7 87 6			
							117									
	SAND (SP-SM), trace gravel, trace fines Compact to very dense Grey Wet						116									
							115									
							114									
							113									
							112						0 92 8 0			
			17	SS	85		111									
							110									

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

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PROJECT		1786302		RECORD OF BOREHOLE		No N/E RS-4		SHEET 3 OF 4		METRIC							
G.W.P.		2130-01-00		LOCATION		N 4847310.3; E 315832.8 MTM NAD 83 ZONE 10 (LAT. 43.765679; LONG. -79.362973)		ORIGINATED BY		DH							
DIST		Central HWY 401		BOREHOLE TYPE		Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY		RM							
DATUM		Geodetic		DATE		July 11, 12 and 15, 2019		CHECKED BY		DH							
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
107.0	SAND (SP-SM), trace gravel, trace fines Compact to very dense Grey Wet		18	SS	63												
32.3	Inferred highly weathered, grey, SHALE (BEDROCK)																
102.6	Moderately weathered, grey, SHALE (BEDROCK) (Georgian Bay Formation)		19	SS	27												
36.7	Bedrock cored from a depth of 36.8 m to 40.0 m (between Elev. 102.6 m and 99.3 m). For rock coring details refer to Record of Drillhole N/E RS-4.		20A 20B	SS RC	100/0.0 REC 100%												RQD = 0%
			2	RC	REC 100%												RQD = 36%
			3	RC	REC 100%												RQD = 91%
99.3	END OF BOREHOLE																
40.0	NOTES: 1. Groundwater level encountered at a depth of 2.7 m below ground surface (Elev. 136.6 m). 2. Groundwater level measurements in piezometer: Date Depth (m) Elev. (m) 19/07/16 1.0 138.3 19/08/21 0.9 138.4 19/10/25 0.9 138.4 20/09/21 *destroyed*																

PROJECT: 1786302

RECORD OF DRILLHOLE: N/E RS-4

SHEET 4 OF 4

LOCATION: N 4847310.34 ;E 315832.78


DRILLING DATE: July 15, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA				WEATH- ERING INDEX				Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jso	W1	W2	W3	W4			W5			W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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37	Rotary Diamond Drill HQ Size Core Barrel	Continued from Record of Borehole N/E RS-4		102.56																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		

DEPTH SCALE

1 : 50

LOGGED: DH

CHECKED: RM

PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-5		SHEET 1 OF 4	METRIC
G.W.P. 2130-01-00		LOCATION N 4847323.0; E 315832.6 MTM NAD 83 ZONE 10 (LAT. 43.765793; LONG. -79.362975)		ORIGINATED BY SEM	
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY RM	
DATUM Geodetic		DATE July 8, 9 and 10, 2019		CHECKED BY DH	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT 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			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
140.3	GROUND SURFACE													
0.0	TOPSOIL (130 mm)													
0.1	SILTY SAND (SM) to SAND (SP), trace gravel, trace rootlets Loose to compact Brown Moist to wet		1	SS	5									
			2	SS	14									2 56 41 1
			3	SS	4									
			4	SS	12									
			5	SS	9									
			6	SS	16									0 81 19 0
134.7	SILT (ML), trace sand Loose Grey Wet		7	SS	5									0 5 87 8
133.1	SAND (SP-SM), trace fines Compact Grey Wet		8	SS	24									
131.6	SILTY CLAY (CI) to CLAYEY SILT-SILT (CL-ML), trace gravel, trace to some sand Firm to stiff Grey Wet		9	SS	WR									
			10	SS	WR									
			11	SS	WH									
			12	TO	-									

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

GTA-MTO 001 S:\CLIENTS\MTOWHY_401_LESLIE_STREET\02_DATAGIN\HWY_401_LESLIE_STREET.GPJ GAL-GTA.GDT 9/21/21

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

PROJECT		RECORD OF BOREHOLE				No N/E RS-5		SHEET 3 OF 4		METRIC							
G.W.P. 2130-01-00		LOCATION				N 4847323.0; E 315832.6 MTM NAD 83 ZONE 10 (LAT. 43.765793; LONG. -79.362975)				ORIGINATED BY SEM							
DIST Central HWY 401		BOREHOLE TYPE				Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY RM							
DATUM Geodetic		DATE				July 8, 9 and 10, 2019				CHECKED BY DH							
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
106.7	SAND (SP-SM), trace gravel, trace fines Very dense Grey Wet		18	SS	100												
33.6	CLAYEY SILT (CL), trace sand Hard Grey Moist		19A 19B	SS	33												0 2 80 18
105.3	Organic SILT (OH), contains wood pieces and peat seams Hard Dark grey to black Moist		20	SS	71												OC = 8%
103.4	Inferred completely to moderately weathered, grey SHALE (BEDROCK)		21	SS	100/0.07												
36.9	SHALE (BEDROCK)		22A 22B	SS	100/0.13												RQD = 20%
103.0	Bedrock cored from a depth of 36.9 m to 40.3 m (between Elev. 103.4 m and 100.0 m) For rock coring details refer to Record of Drillhole N/E RS-5.		1	RC	REC 69%												RQD = 92%
37.3			2	RC	REC 100%												RQD = 96%
100.0			3	RC	REC 100%												
40.3	END OF BOREHOLE																
	NOTE: 1. Borehole was dry before switching to mud rotary drilling at a depth of 5.2 m below ground surface (Elev. 135.1 m).																

PROJECT: 1786302

RECORD OF DRILLHOLE: N/E RS-5

SHEET 4 OF 4

LOCATION: N 4847322.98 ;E 315832.58

DRILLING DATE: July 10, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP W/L CORE AXIS	DISCONTINUITY DATA				WEATH- ERING INDEX		Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcom	W1 W2 W3 W4 W5 W6 W7 W8	W9 W10 W11 W12																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
37	Rotary Diamond Drill HQ Size Core Barrel	Continued from Record of Borehole N/E RS-5		103.38	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							

DEPTH SCALE


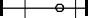



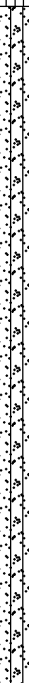

1 : 50

LOGGED: SEM

CHECKED: DH

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

PROJECT <u>1786302</u>		RECORD OF BOREHOLE No N/E RS-6		SHEET 2 OF 4	METRIC
G.W.P. <u>2130-01-00</u>		LOCATION <u>N 4847312.5; E 315841.2 MTM NAD 83 ZONE 10 (LAT. 43.765698; LONG. -79.362868)</u>		ORIGINATED BY <u>DH</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)</u>		COMPILED BY <u>RM</u>	
DATUM <u>Geodetic</u>		DATE <u>July 8, 9 and 10, 2019</u>		CHECKED BY <u>DH</u>	

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							WATER CONTENT (%)	
									20 40 60 80 100						10 20 30	
-- CONTINUED FROM PREVIOUS PAGE --																
	SILTY CLAY (CI) to CLAYEY SILT (CL), trace to some gravel, sandy to some sand Firm Grey Moist		12	SS	WH		123							18 29 36 17		
							122									
							121									
							120									
							119									
							118									
119.0	SILT (ML), some sand Very dense Grey Wet						117							0 11 87 2		
19.8			15	SS	58		116									
115.6	SILTY SAND (SM) Very dense Grey Wet						115							0 76 22 2		
23.2							114									
							113									
							112									
							111									
			17	SS	100/0.23		110									
							109									

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

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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-6					SHEET 3 OF 4		METRIC							
G.W.P. 2130-01-00		LOCATION N 4847312.5; E 315841.2 MTM NAD 83 ZONE 10 (LAT. 43.765698; LONG. -79.362868)					ORIGINATED BY DH									
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)					COMPILED BY RM									
DATUM Geodetic		DATE July 8, 9 and 10, 2019					CHECKED BY DH									
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
-- CONTINUED FROM PREVIOUS PAGE --																
107.8	SILTY SAND (SM) Very dense Grey Wet		18A 18B	SS	51		108									
31.0	CLAYEY SILT-SILT (CL-ML), trace sand Very stiff Grey Moist						107									
							106									
							105									
	- Trace rootlets encountered at a depth of 33.8 m below ground surface (Elev. 105.0 m)		19	SS	23											
104.2							104									
34.6	Sandy Organic SILT (OH), contains rootlets, wood chips and peat seams Hard Brown and grey Moist		20	SS	39											
			21A 21B	SS	65		103									
102.6																
36.4	Gravelly CLAYEY SILT (CL) (RESIDUAL SOIL) Hard Grey Moist SHALES (BEDROCK)		1	RC	REC 100%		102									
	Bedrock cored from a depth of 36.3 m to 40.0 m (between Elev. 102.5 m and 98.8 m)		2	RC	REC 100%		101									
	For rock coring details refer to Record of Drillhole N/E RS-6.		3	RC	REC 100%		100									
98.8							99									
40.0	END OF BOREHOLE															
	NOTE: 1. Groundwater level measured at a depth of 2.8 m below ground surface (Elev. 136.0 m) before switching to casing advancement at a depth of 3.0 m below ground surface (Elev. 135.8 m).															

PROJECT: 1786302

RECORD OF DRILLHOLE: N/E RS-6

SHEET 4 OF 4

LOCATION: N 4847312.51 ;E 315841.21

DRILLING DATE: July 10, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Truck-Mounted Drilling

DRILLING CONTRACTOR: Geo-environmental Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °/°/°	DISCONTINUITY DATA				WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jso																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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		Continued from Record of Borehole N/E RS-6		102.42																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
	Rotary Diamond Drill HQ Size Core Barrel	Slightly to highly weathered, thinly laminated, grey, fine grained, slightly porous, very weak to weak SHALE (Georgian Bay Formation) with interbeds of medium strong to strong LIMESTONE/SILTSTONE		36.35	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															</

DEPTH SCALE

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LOGGED: DH/RM

CHECKED: CN




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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-8					SHEET 1 OF 4		METRIC			
G.W.P. 2130-01-00		LOCATION N 4847313.3; E 315874.3 MTM NAD 83 ZONE 10 (LAT. 43.765705; LONG. -79.362457)					ORIGINATED BY KN					
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)					COMPILED BY SK					
DATUM Geodetic		DATE August 12 to 17, 2020					CHECKED BY KN					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
136.8	GROUND SURFACE											
0.0	ASPHALT (80 mm)											
136.5	CONCRETE (200 mm)											
136.1	SAND (SP-SM), some fines (FILL) Brown Moist		1	GS	-							
0.7	SILTY SAND (SM) Very loose to dense Brown Wet		2	SS	40		136					
			3	SS	31		135					0 62 38 0
			4	SS	17		134					
			5	SS	3		133					
			6	SS	14		132					0 52 46 2
			7	SS	12		131					
131.2	CLAYEY SILT (CL), trace to some sand, trace gravel Firm to stiff Grey Wet		8	SS	1		130					
5.6			9	SS	WH		129					
			10	SS	WR		128	2.7				
			11	SS	3		127	3.0				
			12	TO	-		126					5 18 36 41
			13	SS	1		125					
			14	SS	WH		124	2.8				
							123					2 16 42 40
							122	1.6				

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

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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-8				SHEET 2 OF 4		METRIC							
G.W.P. 2130-01-00		LOCATION N 4847313.3; E 315874.3 MTM NAD 83 ZONE 10 (LAT. 43.765705; LONG. -79.362457)				ORIGINATED BY KN									
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY SK									
DATUM Geodetic		DATE August 12 to 17, 2020				CHECKED BY KN									
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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
--- CONTINUED FROM PREVIOUS PAGE ---															
119.0	CLAYEY SILT (CL), trace to some sand, trace gravel Firm to stiff Grey Wet		15	SS	WH		121								
17.8	SANDY CLAYEY SILT-SILT (CL-ML), trace gravel (TILL) Hard Grey Moist to wet		16	SS	WH		120								
			17	SS	47		119								
							118								
							117								
							116								
			18A	SS	37		115								
			18B												
	- 0.3 m Sandy SILT interlayer from 21.6 m to 21.9 m below ground surface (between Elev. 115.2 m and 114.9 m)						114								
113.6	SAND (SP-SM), trace to some fines Loose to compact Grey Wet						113								
23.2															
			19	SS	24		112								
							111								
							110								
			20	SS	16		109								
							108								
							107								

Continued Next Page

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

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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-8				SHEET 3 OF 4		METRIC								
G.W.P. 2130-01-00		LOCATION N 4847313.3; E 315874.3 MTM NAD 83 ZONE 10 (LAT. 43.765705; LONG. -79.362457)				ORIGINATED BY KN										
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY SK										
DATUM Geodetic		DATE August 12 to 17, 2020				CHECKED BY KN										
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								
	--- CONTINUED FROM PREVIOUS PAGE ---															
105.1	SAND (SP-SM), trace to some fines Loose to compact Grey Wet		21	SS	6											
31.7	Sandy CLAYEY SILT (CL), containing SHALE fragments (RESIDUAL SOIL) Grey Moist		22	SS	100/0.18											
104.5	SHALE (BEDROCK)		1	RC	REC 100%											RQD = 0%
32.3	Bedrock cored from a depth of 32.3 m to 35.1 m (between Elev. 104.5 m and 101.7 m) For rock coring details refer to Record of Drillhole N/E RS-8.		2	RC	REC 100%											RQD = 85%
			3	RC	REC 80%											RQD = 24%
101.7	END OF BOREHOLE															
35.1	NOTE: 1. Borehole dry prior to switching over to casing advancement at a depth of 3.7 m below ground surface (Elev. 133.1 m).															

PROJECT: 1786302

LOCATION: N 4847313.32 ;E 315874.27

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: N/E RS-8

DRILLING DATE: August 18, 2020

DRILL RIG: CME 55- Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

SHEET 4 OF 4

DATUM: Geodetic

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY															FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS	DISCONTINUITY DATA				WEATH- ERING INDEX					Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jso	W1	W2	W3	W4	W5				W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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DEPTH SCALE

1 : 50

LOGGED: KN

CHECKED: CN

PROJECT <u>1786302</u>		RECORD OF BOREHOLE No N/E RS-9				SHEET 1 OF 4		METRIC	
G.W.P. <u>2130-01-00</u>		LOCATION <u>N 4847316.4; E 315884.2 MTM NAD 83 ZONE 10 (LAT. 43.765733; LONG. -79.362333)</u>				ORIGINATED BY <u>KN</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)</u>				COMPILED BY <u>SK</u>			
DATUM <u>Geodetic</u>		DATE <u>August 9 to 12, 2020</u>				CHECKED BY <u>KN</u>			

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
137.3	GROUND SURFACE															
0.0	ASPHALT (25 mm)															
	GRAVEL (25 mm)															
136.6	CONCRETE (230 mm)		1	GS	-											
0.7	Gravelly SAND (SP-SM), some fines (FILL)		2	SS	43											
	Grey Dry															
	SILTY SAND (SM)															
	Dense to very dense		3	SS	103											
	Brown Dry															
135.1	SILT (ML), trace sand															
2.2	Very loose to compact		4	SS	16											
	Brown Wet															
			5	SS	3											
			6	SS	7											
			7	SS	12											
131.7	SILTY CLAY (CI) to CLAYEY SILT (CL), trace gravel, trace to some sand		8	SS	WH											
5.6	Firm to stiff		9	TO	-											
	Grey Wet															
			10	SS	1											
			11	SS	WH											
			12	SS	WH											
			13	SS	WH											
			14	SS	1											

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

PROJECT <u>1786302</u>		RECORD OF BOREHOLE No N/E RS-9				SHEET 2 OF 4		METRIC						
G.W.P. <u>2130-01-00</u>		LOCATION <u>N 4847316.4; E 315884.2 MTM NAD 83 ZONE 10 (LAT. 43.765733; LONG. -79.362333)</u>				ORIGINATED BY <u>KN</u>								
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)</u>				COMPILED BY <u>SK</u>								
DATUM <u>Geodetic</u>		DATE <u>August 9 to 12, 2020</u>				CHECKED BY <u>KN</u>								
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						
	--- CONTINUED FROM PREVIOUS PAGE ---													
	SILTY CLAY (CI) to CLAYEY SILT (CL), trace gravel, trace to some sand Firm to stiff Grey Wet		15	SS	4		122							
							121							
			16	SS	14		120							
119.5														
17.8	SILTY SAND (SM), trace gravel (TILL) Compact Grey Wet		17	SS	27		119							
							118							
117.2														
20.1	CLAYEY SAND (SC), some gravel (TILL) Hard Grey Wet		18	SS	32		116							
							115							
114.1														
23.2	SAND (SP-SM), trace to some fines, trace gravel Dense to very dense Grey Wet		19	SS	75		114							
							113							
							112							
							111							
							110							
			20	SS	47		109							
							108							
107.3														

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PROJECT 1786302		RECORD OF BOREHOLE No N/E RS-9				SHEET 3 OF 4		METRIC									
G.W.P. 2130-01-00		LOCATION N 4847316.4; E 315884.2 MTM NAD 83 ZONE 10 (LAT. 43.765733; LONG. -79.362333)				ORIGINATED BY KN											
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY SK											
DATUM Geodetic		DATE August 9 to 12, 2020				CHECKED BY KN											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100					10 20 30				
30.0	Sandy CLAYEY SILT (CL), containing SHALE fragments (RESIDUAL SOIL) Hard Grey Moist - Grinding at a depth of 31.4 m below ground surface (Elev. 105.9 m)		21	SS	80/0.03		107										
								106									
105.0				22A	SS	103/0.28		105									
32.3	SHALE (BEDROCK) Bedrock cored from a depth of 32.3 m to 35.0 m (between Elev. 105.0 m and 102.3 m) For rock coring details refer to Record of Drillhole N/E RS-9.			1	RC	REC 100%		104									RQD = 42%
			2	RC	REC 70%		103									RQD = 27%	
102.4	END OF BOREHOLE																
35.0	NOTE: 1. Groundwater measured at a depth of 1.4 m below ground surface (Elev. 135.9 m) prior to switching over to casing advancement at a depth of 3.8 m below ground surface (Elev. 133.5 m).																

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

PROJECT <u>1786302</u>				RECORD OF BOREHOLE No N/E RS-10				SHEET 2 OF 3				METRIC					
G.W.P. <u>2130-01-00</u>				LOCATION <u>N 4847313.6; E 315912.3 MTM NAD 83 ZONE 10 (LAT. 43.765707; LONG. -79.361985)</u>				ORIGINATED BY <u>SEM</u>									
DIST <u>Central</u> HWY <u>401</u>				BOREHOLE TYPE <u>Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)</u>				COMPILED BY <u>DH</u>									
DATUM <u>Geodetic</u>				DATE <u>September 4 and 5, 2019</u>				CHECKED BY <u>RM</u>									
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 (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L —————				
								20	40	60	80	100	10	20	30		
117.2	CLAYEY SILT (CL), trace gravel, sandy to trace sand Firm to stiff Grey Wet		14	SS	3		122										
							121			2.2 + 1.8 +							
			15	SS	4		120										2 24 42 32
							119			2.3 + 2.8 +							
			16	SS	WH		118			2.0 +							
20.1	SANDY CLAYEY SILT (CL), trace gravel (TILL) Hard Grey Wet						117										
							116										2 27 52 19
			17	SS	39		115										
							114										
							113										
			18	SS	47		112										
							111										
111.1	SAND (SP-SM), trace fines Very dense Grey Wet						110										0 93 6 1
26.2			19	SS	100		109										
							108										
			20	SS	68												

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

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PROJECT <u>1786302</u>		RECORD OF BOREHOLE No N/E RS-10				SHEET 3 OF 3		METRIC										
G.W.P. <u>2130-01-00</u>		LOCATION <u>N 4847313.6; E 315912.3 MTM NAD 83 ZONE 10 (LAT. 43.765707; LONG. -79.361985)</u>				ORIGINATED BY <u>SEM</u>												
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)</u>				COMPILED BY <u>DH</u>												
DATUM <u>Geodetic</u>		DATE <u>September 4 and 5, 2019</u>				CHECKED BY <u>RM</u>												
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										
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60 80 100 WATER CONTENT (%)						
105.2	SAND (SP-SM), trace fines Very dense Grey Wet						107											
105.2	SANDY CLAYEY SILT (CL), trace gravel (RESIDUAL SOIL) Hard Grey Moist		21A 21B 21C	SS	100		106											
104.2			22	SS	100/0.00		105											
103.2			23	SS	100/0.00		104											
102.2	END OF BOREHOLE		24	SS	100/0.00		103											
101.2	NOTE: 1. Borehole open and dry before switching to mud rotary method at a depth of 3.7 m below ground surface (Elev. 133.6 m).																	

PROJECT		1786302		RECORD OF BOREHOLE No L-1		SHEET 1 OF 1		METRIC											
G.W.P.		2130-01-00		LOCATION		N 4847343.6; E 315808.5 MTM NAD 83 ZONE 10 (LAT. 43.765978; LONG. -79.363274)		ORIGINATED BY AK											
DIST		Central HWY 401		BOREHOLE TYPE		Power Auger; 140 mm Solid Stem Augers		COMPILED BY DH											
DATUM		Geodetic		DATE		February 28, 2020		CHECKED BY KN											
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40
140.8	GROUND SURFACE																		
0.0	Sandy CLAYEY SILT (CL), trace gravel (FILL) Firm to very stiff Brown Moist		1	SS	4														
			2	SS	18														
	- Containing asphalt fragments at a depth of 1.5 m (Elev. 139.3 m)		3A	SS	20														
138.8			3B																
2.0	SAND (SP-SM), trace fines Very loose to compact Brown Moist		4	SS	2														
			5	SS	6														
			6	SS	10														
	- Wet below a depth of 4.6 m below ground surface (Elev. 136.2 m)		7	SS	12														
			8A	SS	16														
134.3			8B																
6.5	SILT (ML), trace sand Compact Brown Wet																		
133.6																			
7.2	SAND (SP-SM), trace fines Loose Grey Wet		9	SS	7														
131.8																			
9.0	Sandy CLAYEY SILT (CL), trace gravel Firm Grey Wet		10	SS	5														
129.5			11	SS	WH														
11.3	END OF BOREHOLE																		
	NOTES: 1. Borehole dry upon completion of drilling. 2. Borehole caved to a depth of 4.3 m below ground surface (Elev. 136.5 m) upon removal of augers.																		



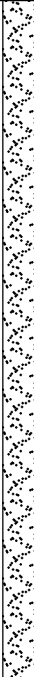
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PROJECT 1786302		RECORD OF BOREHOLE No L-3		SHEET 1 OF 4		METRIC											
G.W.P. 2130-01-00		LOCATION N 4847333.8; E 315861.9 MTM NAD 83 ZONE 10 (LAT. 43.765890; LONG. -79.362610)		ORIGINATED BY LM													
DIST Central HWY 401		BOREHOLE TYPE Power auger, 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SK													
DATUM Geodetic		DATE August 12, 2020		CHECKED BY KN													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	γ	GR	SA	SI	CL	
136.4	GROUND SURFACE																
0.0	ASPHALT (50 mm)																
	CONCRETE (300 mm)																
0.4	SAND (SP-SM), some fines, trace gravel, containing asphalt pieces (FILL)		1	SS	38		136										
135.4	Compact Brown Moist		2	SS	28		135										
1.0	SILTY SAND (SM), trace gravel		3	SS	20												
134.2	Compact Brown Moist		4	SS	3		134										
2.2	CLAYEY SILT (CL) trace sand		5	SS	3		133										
133.4	Soft Brown Moist		6	SS	6		132										
3.0	SILTY SAND (SM), trace gravel		7	SS	1		131	1.1									
131.9	Very loose Brown Moist		8	SS	WH		130										
4.5	Sandy CLAYEY SILT (CL), trace gravel		9	SS	1		129	2.6									
	Firm to stiff		10	SS	WH		128	2.2									
	Grey Moist		11	SS	1		127	3.3									
			12	SS	WH		126	3.3									
			13	SS	WH		125	2.0									
							124	2.8									
							123										
							122										

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


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PROJECT 1786302		RECORD OF BOREHOLE No L-3		SHEET 2 OF 4		METRIC																
G.W.P. 2130-01-00		LOCATION N 4847333.8; E 315861.9 MTM NAD 83 ZONE 10 (LAT. 43.765890; LONG. -79.362610)		ORIGINATED BY LM																		
DIST Central HWY 401		BOREHOLE TYPE Power auger, 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SK																		
DATUM Geodetic		DATE August 12, 2020		CHECKED BY KN																		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p — W — W _L			γ kN/m ³			GR	SA	SI	CL
--- CONTINUED FROM PREVIOUS PAGE ---																						
119.3 17.1	Sandy CLAYEY SILT (CL), trace gravel Firm to stiff Grey Moist		14	SS	1		121															
								120														
			15	SS	2			119														
								118														
			16	SS	63																	
113.2 23.2	SANDY CLAYEY SILT-SILT (CL-ML), trace gravel (TILL) Very stiff to hard Grey Moist						117															
								116														
								115														
			17	SS	27																	
								114														
								113														
	SAND (SP-SM), trace to some fines Very loose to compact Grey Moist						112															
								111														
								110														
			18	SS	2			109														
								108														
			19	SS	22			107														

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

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PROJECT 1786302		RECORD OF BOREHOLE No L-3				SHEET 3 OF 4		METRIC									
G.W.P. 2130-01-00		LOCATION N 4847333.8; E 315861.9 MTM NAD 83 ZONE 10 (LAT. 43.765890; LONG. -79.362610)				ORIGINATED BY LM											
DIST Central HWY 401		BOREHOLE TYPE Power auger, 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)				COMPILED BY SK											
DATUM Geodetic		DATE August 12, 2020				CHECKED BY KN											
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
105.9	Sandy CLAYEY SILT (CL), containing SHALE fragments (RESIDUAL SOIL) Hard Grey Wet SHALE (BEDROCK) Bedrock cored from a depth of 31.0 m to 34.1 m (between Elev. 105.4 m and 102.3 m) For rock coring details refer to Record of Drillhole L3.		20A	SS	100/0.13		106										RQD = 53%
30.5			20B				105										
105.5			1	RC	REC 80%		104										
30.9			2	RC	REC 100%		103										
102.3	END OF BOREHOLE																
34.1	NOTE: 1. Groundwater measured at a depth of 2.1 m below ground surface (Elev. 134.3 m) prior to switching over to casing advancement at a depth of 4.6 m below ground surface (Elev. 131.8 m).																

PROJECT: 1786302

RECORD OF DRILLHOLE: L-3

SHEET 4 OF 4

LOCATION: N 4847333.83 ;E 315861.91

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55- Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA					WEATH- ERING INDEX							Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
						TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jso	W1	W2	W3	W4	W5				W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
		Continued from Record of Borehole L3		105.49																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					

DEPTH SCALE

1 : 50

LOGGED: LM



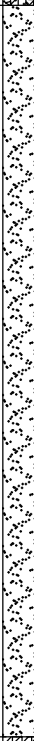
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PROJECT		1786302		RECORD OF BOREHOLE No L-4		SHEET 1 OF 4		METRIC												
G.W.P.		2130-01-00		LOCATION		N 4847334.1; E 315874.5 MTM NAD 83 ZONE 10 (LAT. 43.765892; LONG. -79.362454)		ORIGINATED BY LM												
DIST		Central HWY 401		BOREHOLE TYPE		Power auger, 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SK												
DATUM		Geodetic		DATE		August 9, 2020		CHECKED BY KN												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	20	40	60	80	100	W _p	W	W _L	γ	GR	SA	SI	CL
136.8		GROUND SURFACE																		
0.0		ASPHALT (50 mm)																		
0.2		CONCRETE (180 mm)																		
135.9		SAND (SP-SM), some fines, trace gravel (FILL)		1	SS	42														
0.9		Dense Brown Wet		2	SS	42														
		Sandy SILT (ML)		3	SS	38														
		Very loose to dense Brown Moist to wet		4	SS	18														
				5	SS	3														
				6	SS	2														
132.3		SILTY CLAY (CI) to CLAYEY SILT (CL), trace sand		7	SS	WH														
4.5		Firm Grey Moist		8	TO	-														
				9	SS	WH														
				10	SS	WH														
				11	SS	WH														
				12	TO	-														
				13	SS	2														
				14	SS	4														

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

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PROJECT 1786302		RECORD OF BOREHOLE No L-4		SHEET 2 OF 4		METRIC							
G.W.P. 2130-01-00		LOCATION N 4847334.1; E 315874.5 MTM NAD 83 ZONE 10 (LAT. 43.765892; LONG. -79.362454)		ORIGINATED BY LM									
DIST Central HWY 401		BOREHOLE TYPE Power auger, 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SK									
DATUM Geodetic		DATE August 9, 2020		CHECKED BY KN									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
--- CONTINUED FROM PREVIOUS PAGE ---													
119.0 17.8	SILTY CLAY (CI) to CLAYEY SILT (CL), trace sand Firm Grey Moist		15	SS	WR		121						
			16	SS	2		120						
			17	SS	15		119						
114.9 22.0	SANDY CLAYEY SILT-SILT (CL-ML), trace gravel (TILL) Very stiff to hard Grey Moist		18	SS	42		118						
			19	SS	1		117						
			20	SS	70		116						
107.6 29.2	SAND (SP-SM), trace fines, trace gravel Very loose to very dense Grey Moist to wet		21A	SS	35		115						
			21B				114						
							113						
							112						
							111						
							110						
							109						
							108						
							107						

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

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PROJECT		1786302		RECORD OF BOREHOLE No L-4		SHEET 3 OF 4		METRIC							
G.W.P.		2130-01-00		LOCATION		N 4847334.1; E 315874.5 MTM NAD 83 ZONE 10 (LAT. 43.765892; LONG. -79.362454)		ORIGINATED BY LM							
DIST		Central HWY 401		BOREHOLE TYPE		Power auger, 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SK							
DATUM		Geodetic		DATE		August 9, 2020		CHECKED BY KN							
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							
	--- CONTINUED FROM PREVIOUS PAGE ---														
104.7	Sandy CLAYEY SILT (CL), trace gravel, containing SHALE fragments (RESIDUAL SOIL) Hard Grey Moist		22	SS	50/0/10										
32.1	SHALE (BEDROCK) Bedrock cored from a depth of 32.1 m to 35.9 m below (between Elev. 104.7 m and 100.9 m) For rock coring details refer to Record of Drillhole L4.		1	RC	REC 100%										RQD = 57%
			2	RC	REC 100%										RQD = 87%
			3	RC	REC 100%										RQD = 88%
101.0	END OF BOREHOLE														
35.9	NOTE: 1. Groundwater measured at a depth of 3.4 m below ground surface (Elev. 133.4 m) prior to switching over to casing advancement at a depth of 4.6 m below ground surface (Elev. 132.2 m).														

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SHEET 4 OF 4

DATUM: Geodetic

DRILL RIG: CME 55- Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

[illegible]

LOGGED: LM


CHECKED: CN

PROJECT 1786302		RECORD OF BOREHOLE No L-5		SHEET 1 OF 3		METRIC											
G.W.P. 2130-01-00		LOCATION N 4847342.3; E 315897.7 MTM NAD 83 ZONE 10 (LAT. 43.765966; LONG. -79.362166)		ORIGINATED BY RM													
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SE													
DATUM Geodetic		DATE November 5 to 18, 2019		CHECKED BY KN													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	W _p	W	W _L	γ	GR	SA	SI	CL	
142.6	GROUND SURFACE																
0.0	ASPHALT (90 mm)																
0.3	CONCRETE (320 mm)																
	SAND (SP-SM) to SILTY SAND (SM), trace gravel (FILL) Compact Brown Moist		1A 1B	SS	16		142										
			2	SS	22		141										
			3	SS	26		140										
139.3	CLAYEY SILT (CL/SC) and sand, trace gravel (FILL) Stiff Brown Moist		4A 4B	SS	14		139										
3.4			5	SS	13												
138.1	SILTY SAND (SM) Compact to dense Brown to light brown Moist		6	SS	17		138										
4.5			7	SS	43		137										
			8	SS	40		136										
			9	SS	6		135										
133.9	SILTY CLAY (CI) to CLAYEY SILT (CL), sandy to trace sand, trace gravel Firm Grey Wet		10	SS	7		134										
8.7			11	TO	-		133										
			12	SS	2		132										
			13	SS	WH		131										
							130										
							129										
							128										

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

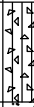
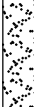


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PROJECT 1786302		RECORD OF BOREHOLE No L-5		SHEET 2 OF 3		METRIC											
G.W.P. 2130-01-00		LOCATION N 4847342.3; E 315897.7 MTM NAD 83 ZONE 10 (LAT. 43.765966; LONG. -79.362166)		ORIGINATED BY RM													
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SE													
DATUM Geodetic		DATE November 5 to 18, 2019		CHECKED BY KN													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR SA SI CL			
--- CONTINUED FROM PREVIOUS PAGE ---																	
116.4 26.2	SILTY CLAY (CI) to CLAYEY SILT (CL), sandy to trace sand, trace gravel Firm Grey Wet		14	SS	WH		127						45	0 4 36 60			
			15	SS	TO		126										
			16	SS	WH		125	3.4									
			17	SS	1		124										
							123	3.2									
							122										
							121										
							120	4.2									
							119										
							118								40.8		
			19A 19B	SS	WH		117										
							116										
							115						1 21 72 6				
			20	SS	47		114										
							113										

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PROJECT 1786302		RECORD OF BOREHOLE No L-5		SHEET 3 OF 3		METRIC															
G.W.P. 2130-01-00		LOCATION N 4847342.3; E 315897.7 MTM NAD 83 ZONE 10 (LAT. 43.765966; LONG. -79.362166)		ORIGINATED BY RM																	
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)		COMPILED BY SE																	
DATUM Geodetic		DATE November 5 to 18, 2019		CHECKED BY KN																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kn/m ³							
111.6	SILT (ML), trace gravel, sandy to trace sand (TILL) Hard Grey Wet		21A	SS	38		112														
31.0	SAND (SP-SM), trace gravel, trace to some fines Very dense Grey Wet		21B				111														
							110														
							109														
			22	SS	60		108														
							107														
107.2	Sandy CLAYEY SILT (CL), some gravel (RESIDUAL SOIL) Hard Grey Moist						106														
35.4			23	SS	100/0.28		105														
							104														
104.2	Inferred highly weathered, SHALE (BEDROCK) (Georgian Bay Formation)						103														
38.4																					
103.0	END OF BOREHOLE		24	SS	100/0.08																
39.6	NOTE: 1. Borehole dry prior to switching to casing advancement at a depth of 7.6 m below ground surface (Elev. 135.0 m).																				

PROJECT 1786302		RECORD OF BOREHOLE No L-6		SHEET 1 OF 2		METRIC							
G.W.P. 2130-01-00		LOCATION N 4847343.6; E 315924.7 MTM NAD 83 ZONE 10 (LAT. 43.765977; LONG. -79.361830)		ORIGINATED BY DH									
DIST Central HWY 401		BOREHOLE TYPE Power Auger; 70 mm I.D., 150 mm O.D. Hollow Stem Augers		COMPILED BY RM									
DATUM Geodetic		DATE June 25 and 26, 2019		CHECKED BY DH									
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
141.7	GROUND SURFACE												
0.0	ASPHALT (180 mm)												
0.2	Gravelly SILTY SAND (SM), trace fines (FILL) Compact to dense Brown Moist		1	SS	26		141						
			2	SS	38								
	- Layer of Sandy CLAYEY SILT (CL), some gravel (FILL) between a depth of 1.5 m and 1.8 m below ground surface (between Elev. 140.2 m and 139.9 m)		3A	SS	27		140						
			3B										
			4	SS	34		139						21 42 31 6
138.7													
3.0	SILTY SAND (SM) to SAND (SP), trace fines, trace to some gravel, pockets of sandy clayey silt Loose to very dense Brown Moist		5	SS	32		138						
			6	SS	25		137						
							136						
			7	SS	49		135						
	- Wet below a depth of 7.2 m below ground surface (Elev. 134.5 m)		8	SS	78		134						0 59 41 0
							133						
	- Grey below a depth of 8.7 m below ground surface (Elev. 133.0 m)		9	SS	9		132						
131.5													
10.2	CLAYEY SILT (CL), some sand, trace gravel Firm Grey Moist		10	SS	WH		131						
							130						
			11	SS	2		129						
							128						
			12	SS	WH		127						2 13 44 41

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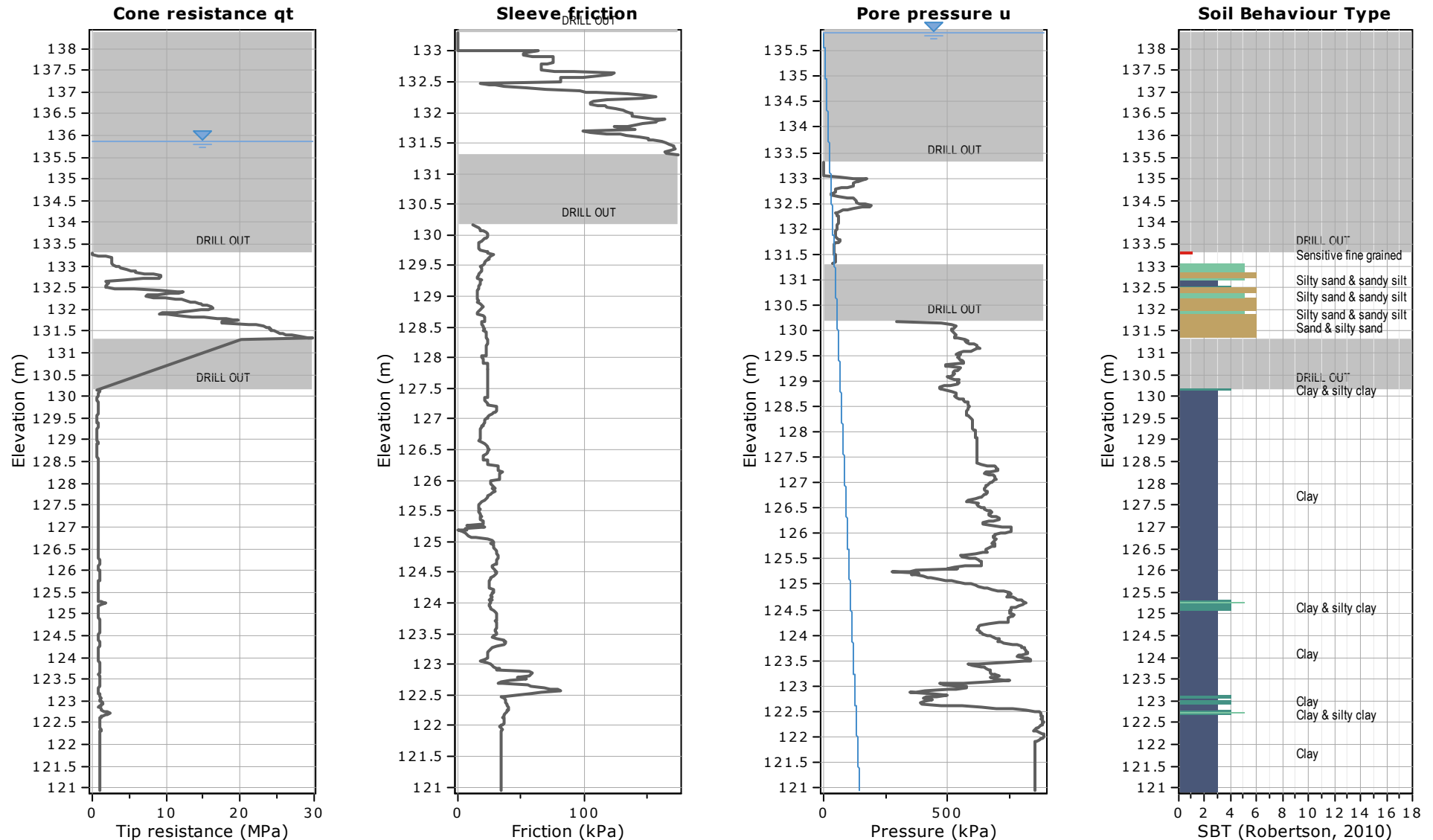
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PROJECT <u>1786302</u>		RECORD OF BOREHOLE No L-6		SHEET 2 OF 2		METRIC														
G.W.P. <u>2130-01-00</u>		LOCATION <u>N 4847343.6; E 315924.7 MTM NAD 83 ZONE 10 (LAT. 43.765977; LONG. -79.361830)</u>		ORIGINATED BY <u>DH</u>																
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger; 70 mm I.D., 150 mm O.D. Hollow Stem Augers</u>		COMPILED BY <u>RM</u>																
DATUM <u>Geodetic</u>		DATE <u>June 25 and 26, 2019</u>		CHECKED BY <u>DH</u>																
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												
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> 10 20 30 </div>								
125.9 15.9	END OF BOREHOLE NOTES: 1. Borehole dry upon completion of drilling. 2. Borehole caved to a depth of 8.2 m below ground surface (Elev. 133.5 m) after removing augers.		13	SS	WH	126														


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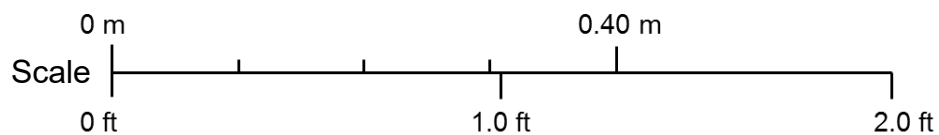
Project: Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue


Location: North York, Ontario (MTO Assignment No. 2016-E-0089)

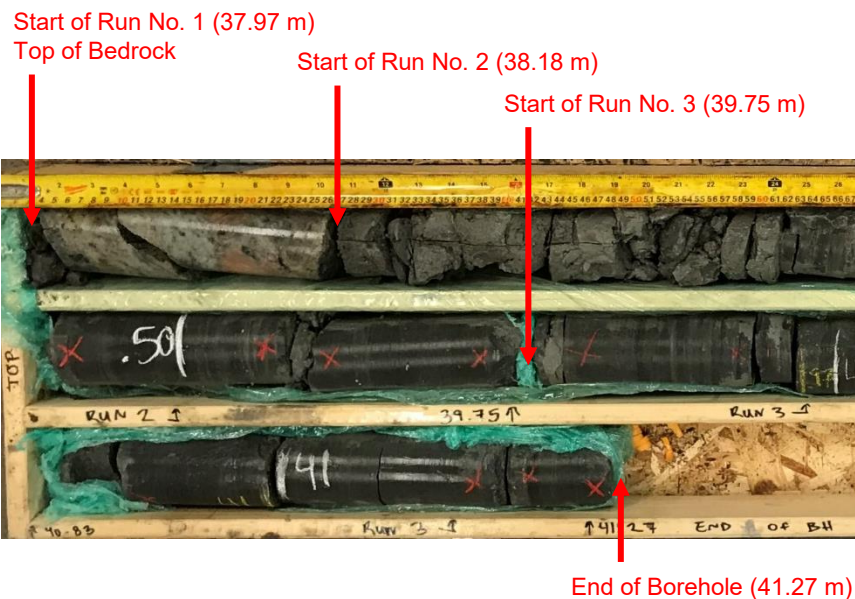




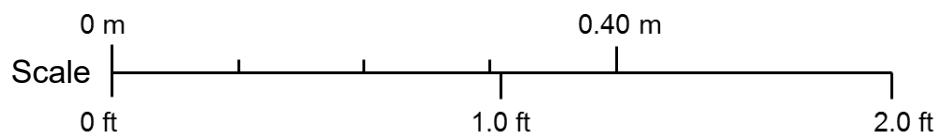
PROJECT					
Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue, Toronto, Ontario					
TITLE					
PAVEMENT CORE PHOTOGRAPH BOREHOLE L5 0 mm to 410 mm					
 GOLDER <small>MEMBER OF WSP</small>	PROJECT No. 1786302			FILE No. ----	
	DESIGN	KNN	20200304	SCALE	NTS
	CADD	--	--	FIGURE B1	
	CHECK	CN	20200924		
	REVIEW	CN	20200924		




PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-2			
 GOLDER <small>MEMBER OF WSP</small>		PROJECT No. 1786302		FILE No. ----	
		DESIGN	KN	20190724	SCALE NTS
		CADD	--	--	VER. 1.
		CHECK	CN	20210213	FIGURE B2
		REVIEW	CN	20210213	



Borehole N/E RS-3: Bedrock cored between depths of about 37.97 m to 41.27 m



PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-3			
 GOLDER MEMBER OF WSP	PROJECT No. 1786302		FILE No. ----		
	DESIGN	KN	20190724	SCALE	NTS
	CADD	--	--	FIGURE B3	
	CHECK	CN	20210213		
	REVIEW	CN	20210213		

Start of Run No. 1 (36.75 m)
Top of Bedrock

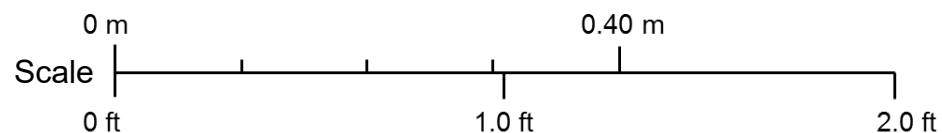
Start of Run No. 2 (36.94 m)




Start of Run No. 3 (38.51 m)

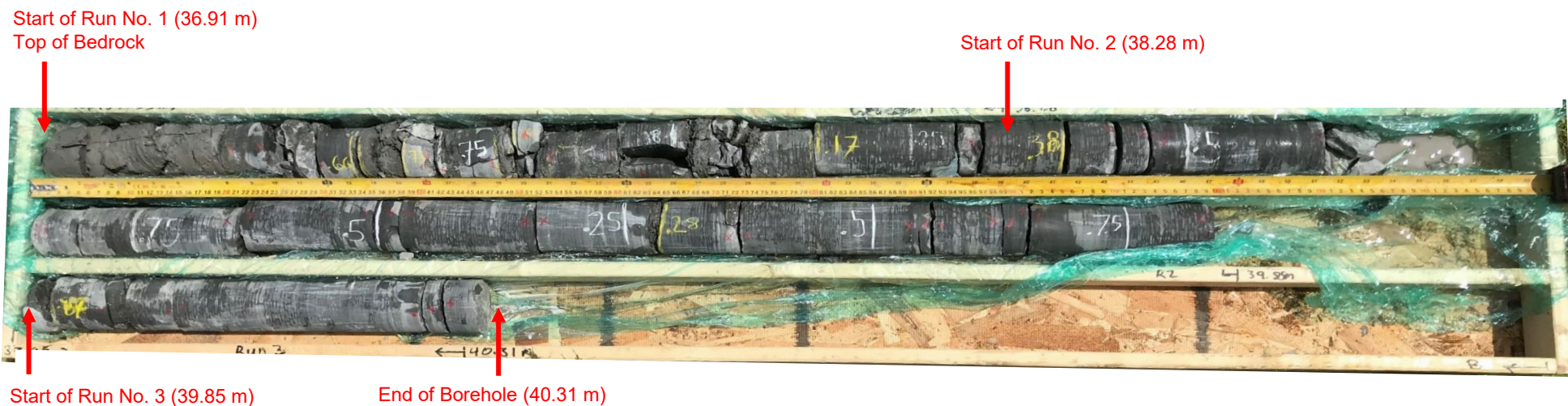
End of Borehole (40.03 m)

Borehole N/E RS-4: Bedrock cored between depths of about 36.75 m to 40.03 m

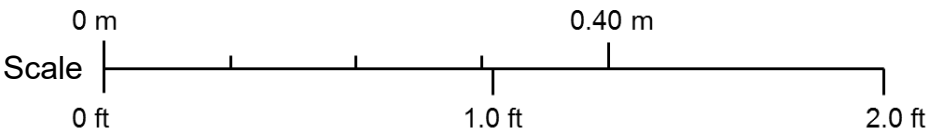



PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-4			
 GOLDER <small>MEMBER OF WSP</small>	PROJECT No. 1786302			FILE No. ----	
	DESIGN	KN	20190724	SCALE	NTS
	CADD	--	--	FIGURE B4	
	CHECK	CN	20210213		
	REVIEW	CN	20210213		
		VER. 1.			

REVISION DATE: 20200924 BY: KNN Project: 1786302

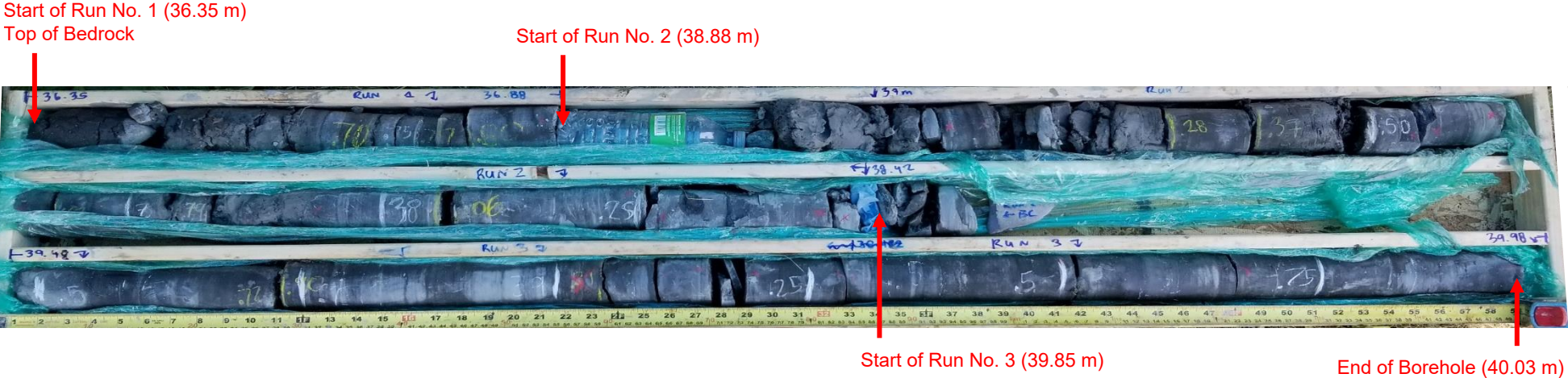


Borehole N/E RS-5: Bedrock cored between depths of about 36.91 m to 40.31 m

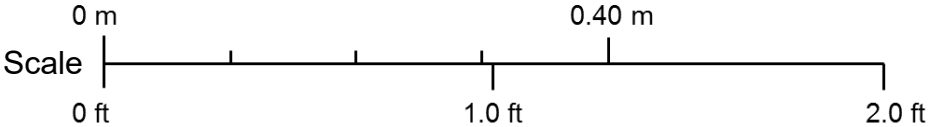



PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-5			
 GOLDER MEMBER OF WSP		PROJECT No. 1786302		FILE No. ----	
		DESIGN	KN	20190724	SCALE NTS
		CADD	--	--	VER. 1.
		CHECK	CN	20210213	FIGURE B5
		REVIEW	CN	20210213	

REVISION DATE: 20200924 BY: KNN Project: 1786302



Borehole N/E RS-6: Bedrock cored between depths of about 36.35 m to 40.03 m



PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-6			
 GOLDER MEMBER OF WSP		PROJECT No. 1786302		FILE No. ----	
		DESIGN	KN	20190724	SCALE NTS
		CADD	--	--	VER. 1.
		CHECK	CN	20210213	FIGURE B6
		REVIEW	CN	20210213	

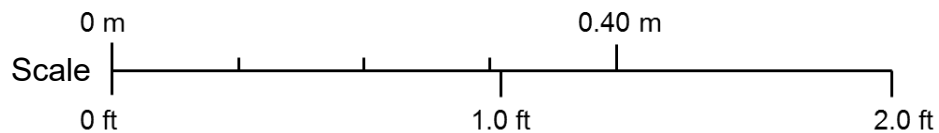
Start of Run No. 1 (32.33 m)
Top of Bedrock


Start of Run No. 2 (32.43 m)

Start of Run No. 3 (34.05 m)

End of Borehole (35.07 m)

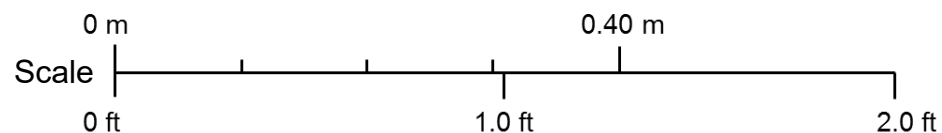
Borehole N/E RS-8: Bedrock cored between depths of about 32.33 m to 35.07 m




PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-8			
 GOLDER MEMBER OF WSP		PROJECT No. 1786302		FILE No. ----	
		DESIGN	KN	20190724	SCALE NTS
		CADD	--	--	VER. 1.
		CHECK	CN	20210213	FIGURE B7
		REVIEW	CN	20210213	



Borehole N/E RS-9: Bedrock cored between depths of about 32.28 m to 34.95 m

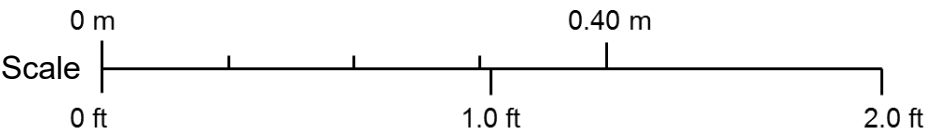



PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE N/E RS-9			
 GOLDER MEMBER OF WSP	PROJECT No. 1786302		FILE No. ----		
	DESIGN	KN	20190724	SCALE	NTS
	CADD	--	--	FIGURE B8	
	CHECK	CN	20210213		
	REVIEW	CN	20210213		

REVISION DATE: 20200924 BY: KNN Project: 1786302



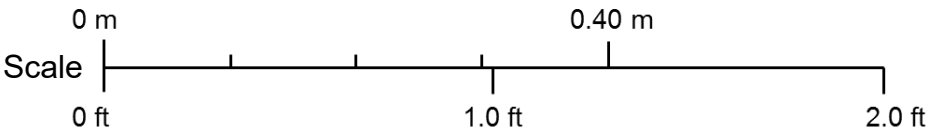
Borehole L-3: Bedrock cored between depths of about 30.91 m to 34.06 m




PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE L-3			
 GOLDER MEMBER OF WSP		PROJECT No. 1786302		FILE No. ----	
		DESIGN	KN	20190724	SCALE NTS
		CADD	--	--	VER. 1.
		CHECK	CN	20210213	FIGURE B9
		REVIEW	CN	20210213	



Borehole L-4: Bedrock cored between depths of about 32.10 m to 35.85 m



PROJECT		Highway 401 EB Collectors Avenue to Warden, (Site No. 37-208/1) Toronto, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPH BOREHOLE L-4			
 GOLDER MEMBER OF WSP		PROJECT No. 1786302		FILE No. ----	
		DESIGN	KN	20190724	SCALE NTS
		CADD	--	--	VER. 1.
		CHECK	CN	20210213	FIGURE B10
		REVIEW	CN	20210213	

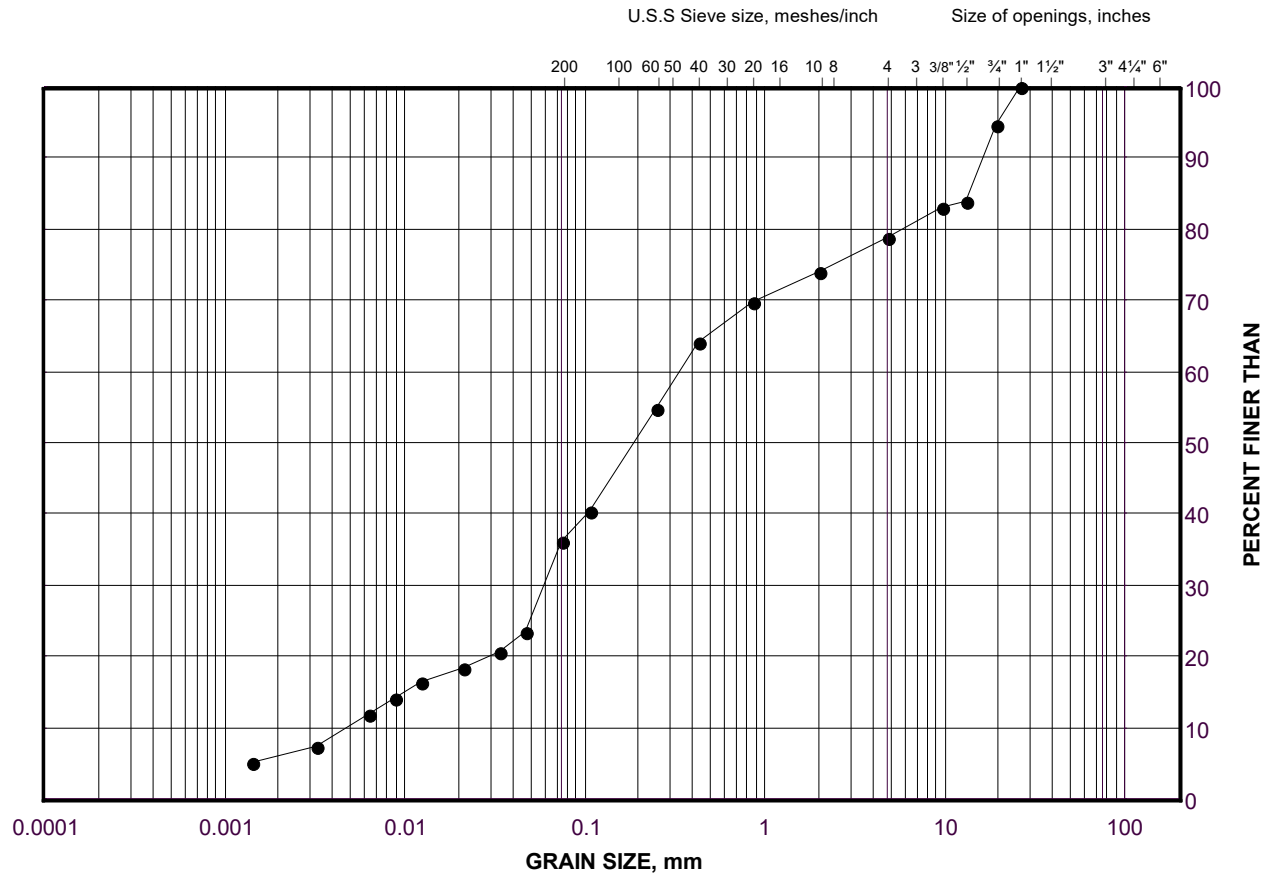
APPENDIX C

Geotechnical And Analytical Test Results

GRAIN SIZE DISTRIBUTION

Gravelly SILTY SAND (SM) (FILL)

FIGURE C1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	L-6	4	139.1

Project Number: 1786302

Checked By: CN

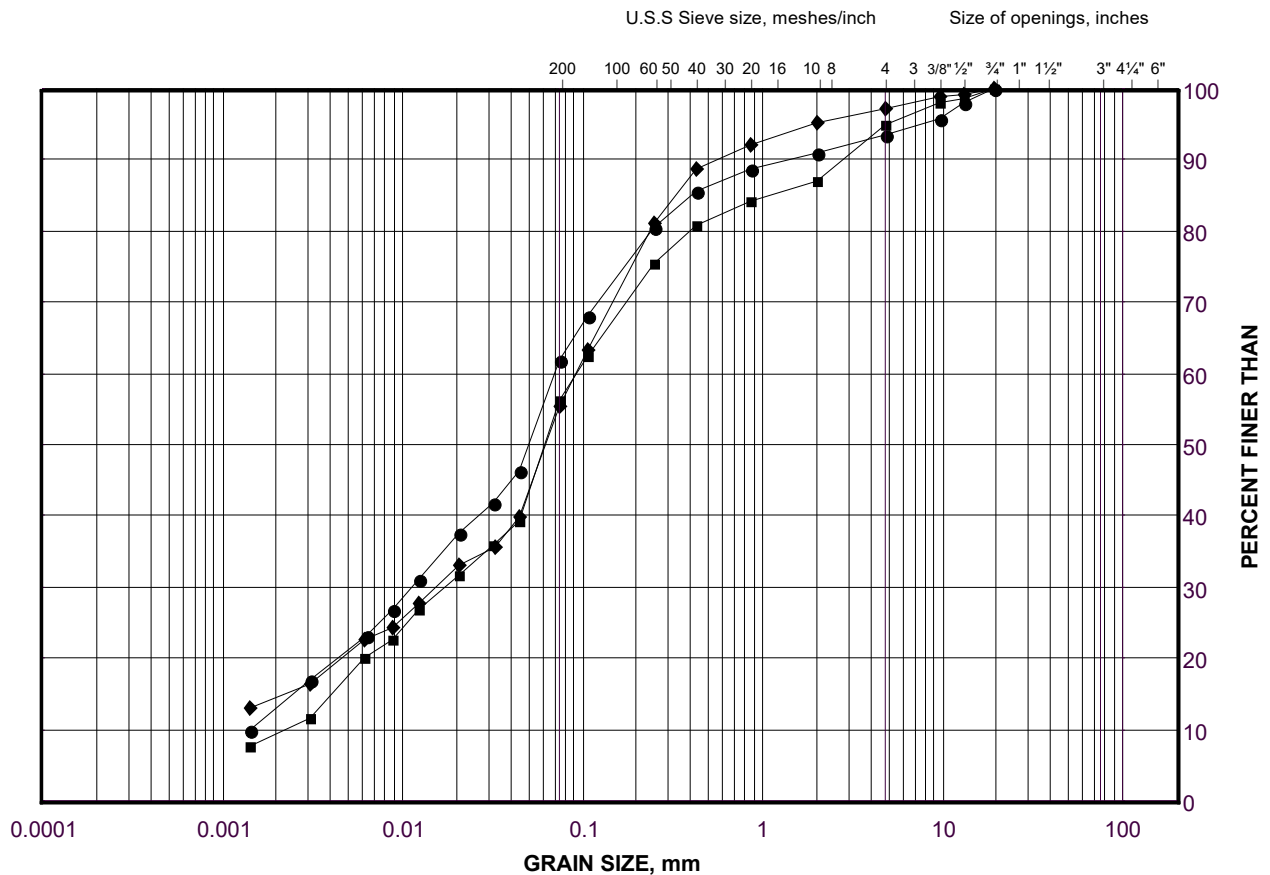
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT (CL) to CLAYEY SILT (CL/SC)
and Sand to CLAYEY SILT-SILT (CL-ML/SC) and Sand (FILL)

FIGURE C2



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

LEGEND

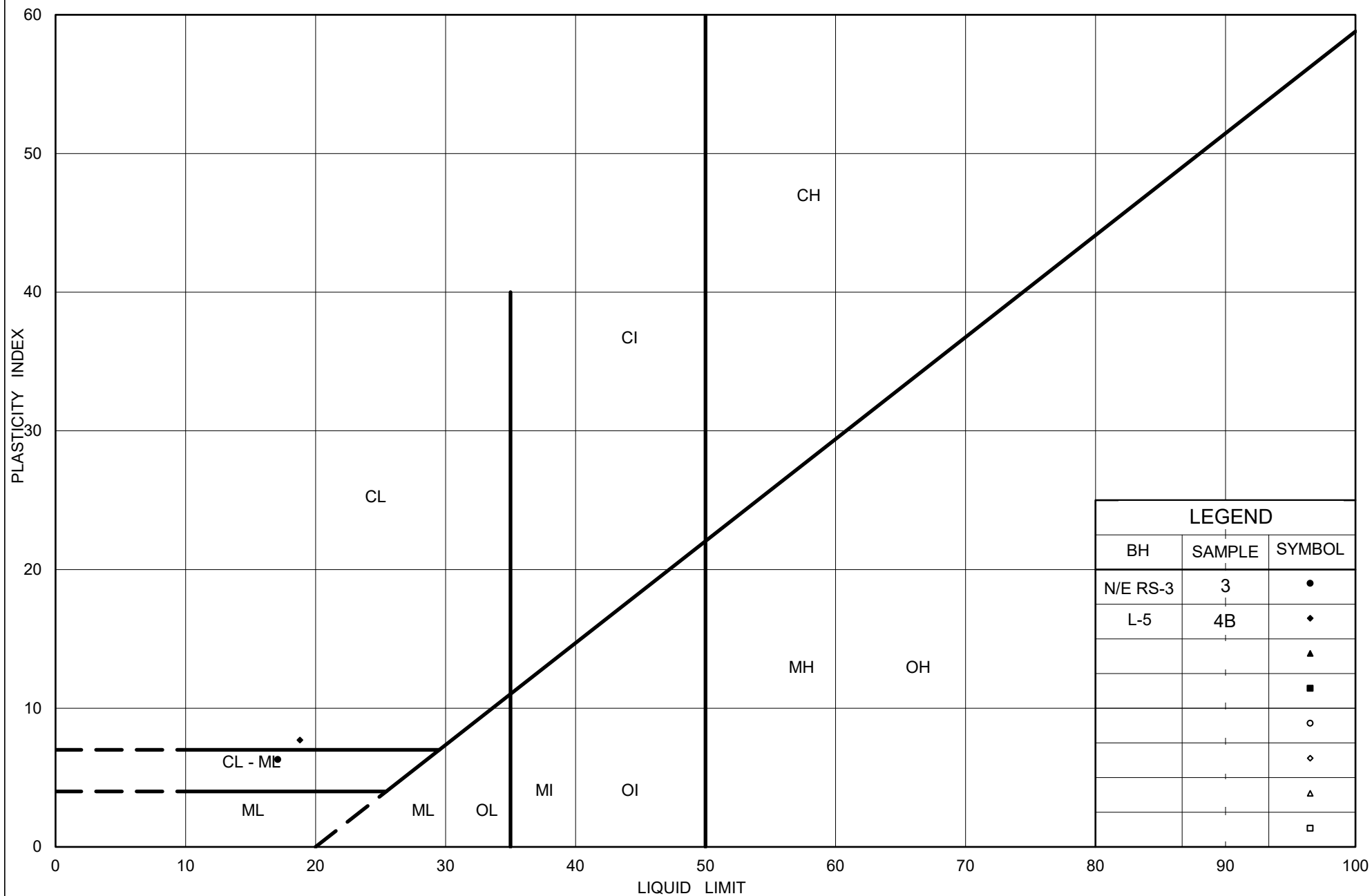
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	L-1	2	139.7
■	N/E RS-3	3	139.4
◆	L-5	4B	139.0

Project Number: 1786302

Checked By: CN

Golder Associates

Date: 22-Oct-20



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PLASTICITY CHART CLAYEY SILT (CL/SC) and Sand to CLAYEY SILT-SILT (CL-ML/SC) and Sand (FILL)

Figure No. C3

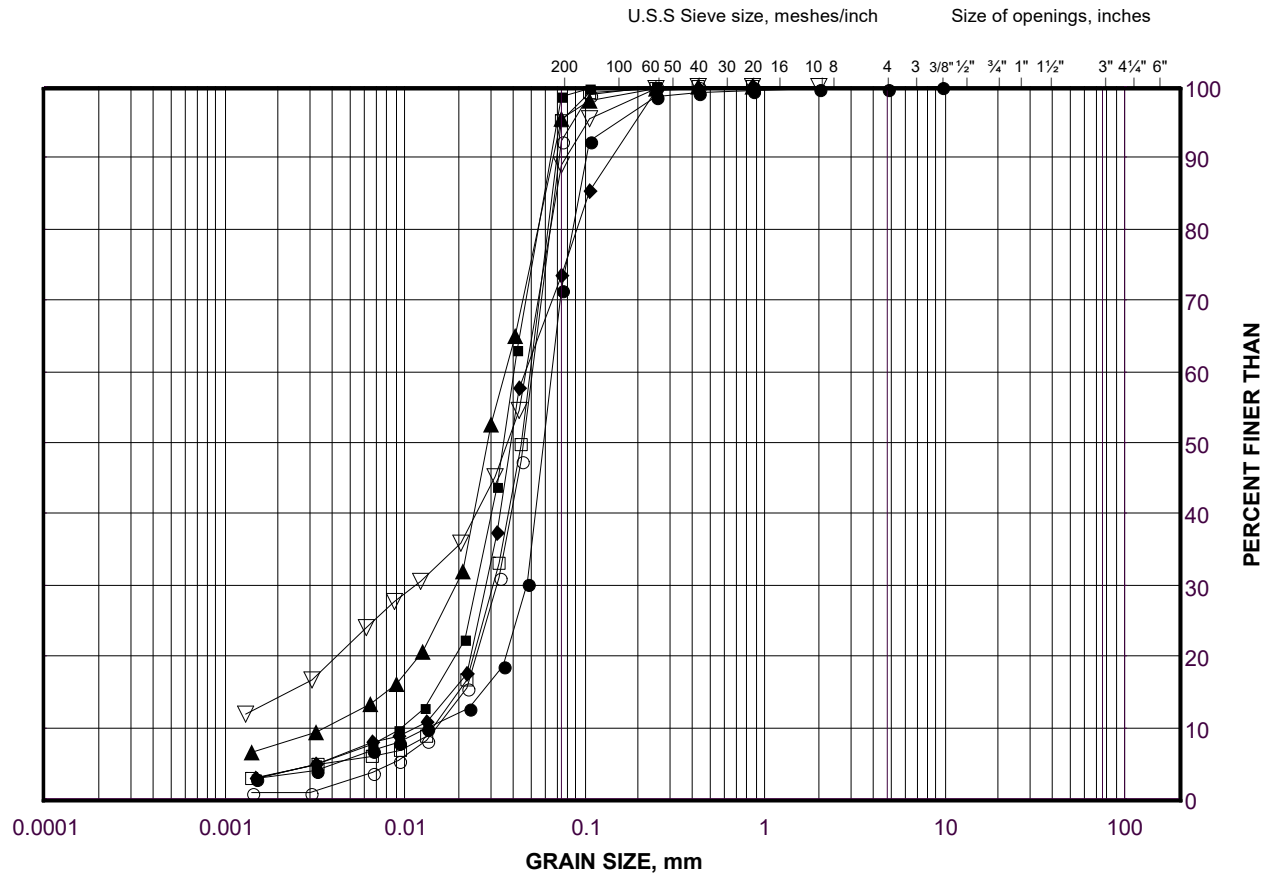
Project No. 1786302

Checked By: CN

GRAIN SIZE DISTRIBUTION

SILT (ML) to Sandy SILT (ML)

FIGURE C4A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	L-4	3	135.0
■	N/E RS-9	5	133.9
◆	L-4	6	132.7
▲	N/E RS-5	7	133.9
▽	N/E RS-3	7	134.7
○	N/E RS-2	8	134.1
□	L-1	8B	134.2

Project Number: 1786302

Checked By: CN

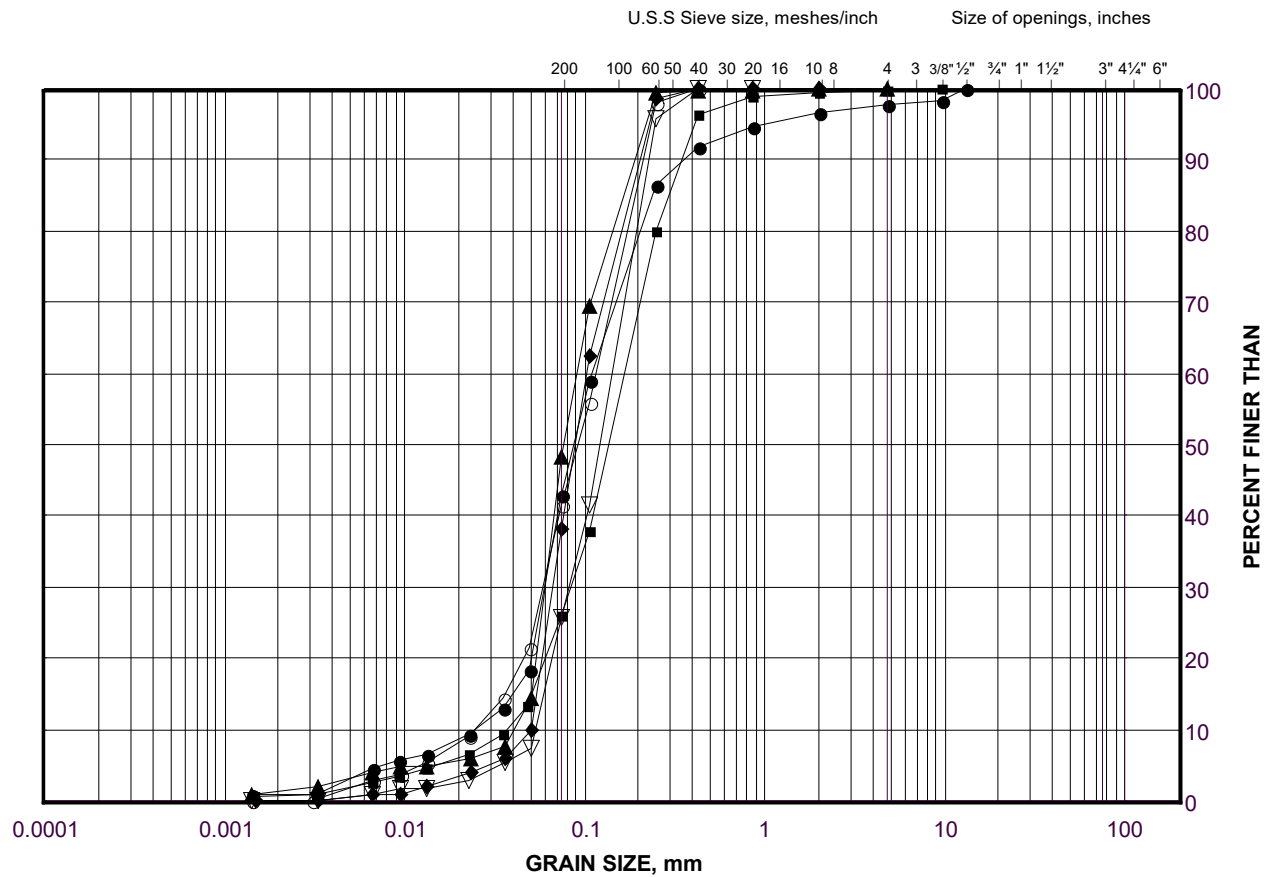
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

SILTY SAND (SM)

FIGURE C4B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	N/E RS-5	2	139.2
■	N/E RS-4	2	138.2
◆	N/E RS-8	3	135.0
▲	N/E RS-8	7	131.9
▽	L-5	7	136.2
○	L-6	8	133.8

Project Number: 1786302

Checked By: CN

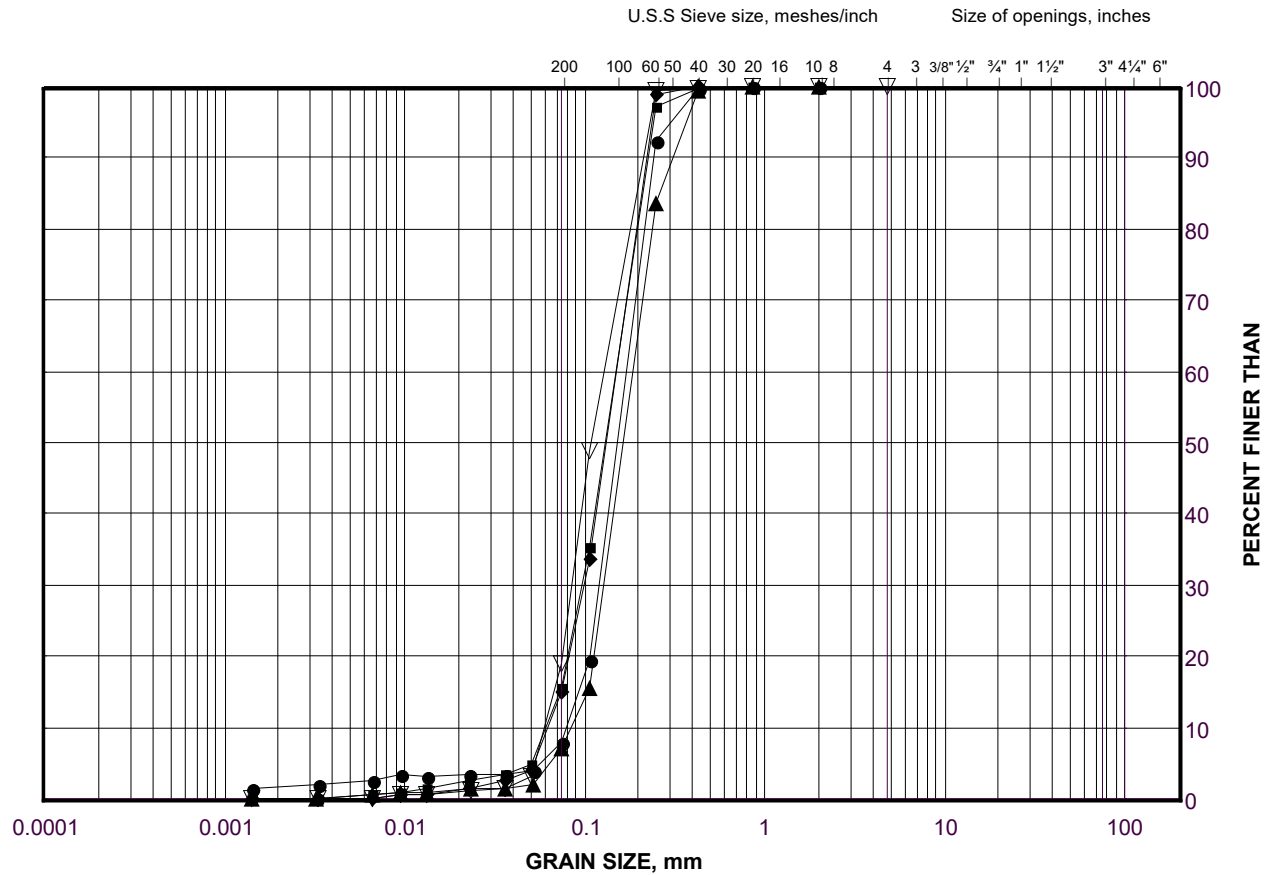
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

SILTY SAND (SM) to SAND (SP-SM)

FIGURE C4C



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	L-1	5	137.4
■	N/E RS-6	5	135.4
◆	N/E RS-4	5	135.9
▲	N/E RS-3	5	137.8
▽	N/E RS-5	6	135.4

Project Number: 1786302

Checked By: CN

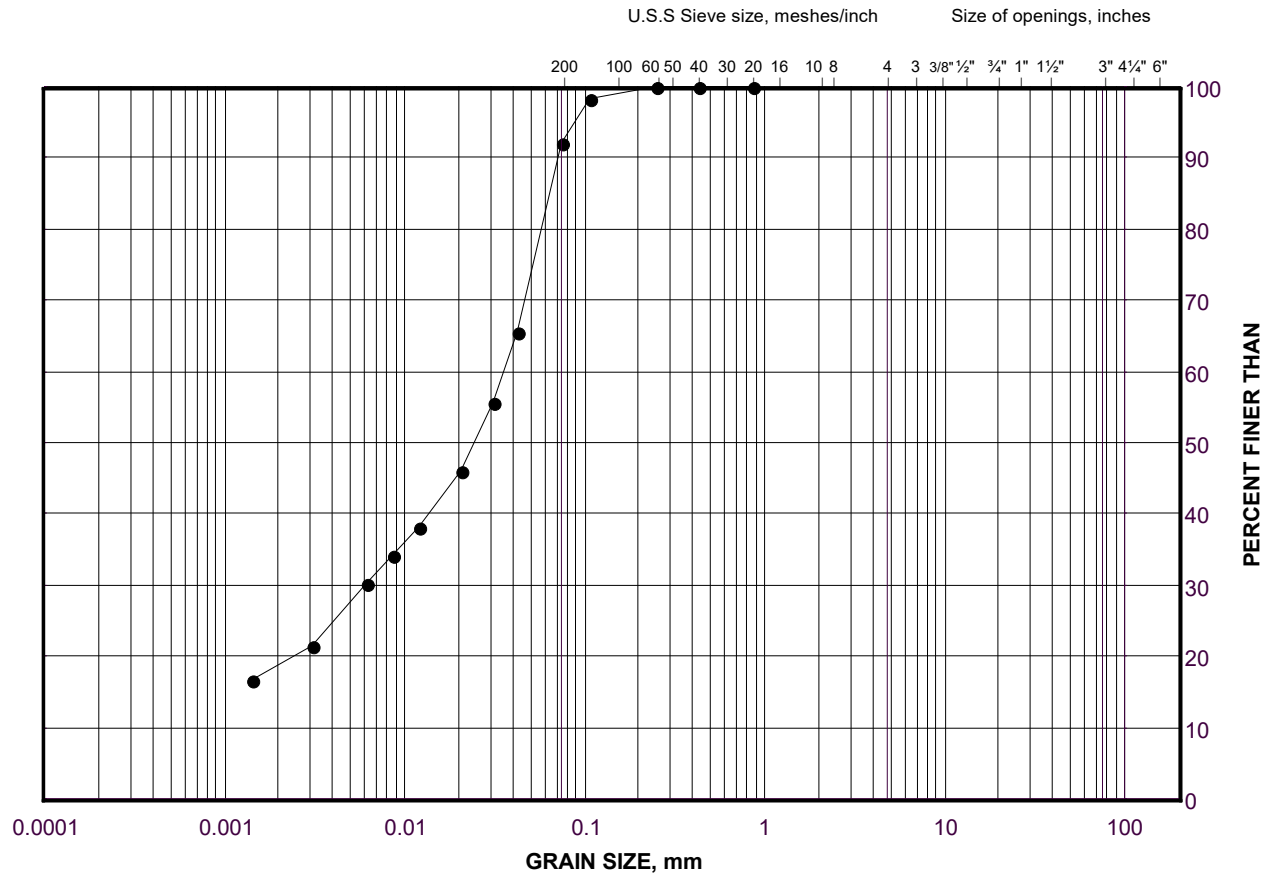
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

CLAYEY SILT (CL) - Interlayer

FIGURE C5



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	L-3	4	133.8

Project Number: 1786302

Checked By: CN

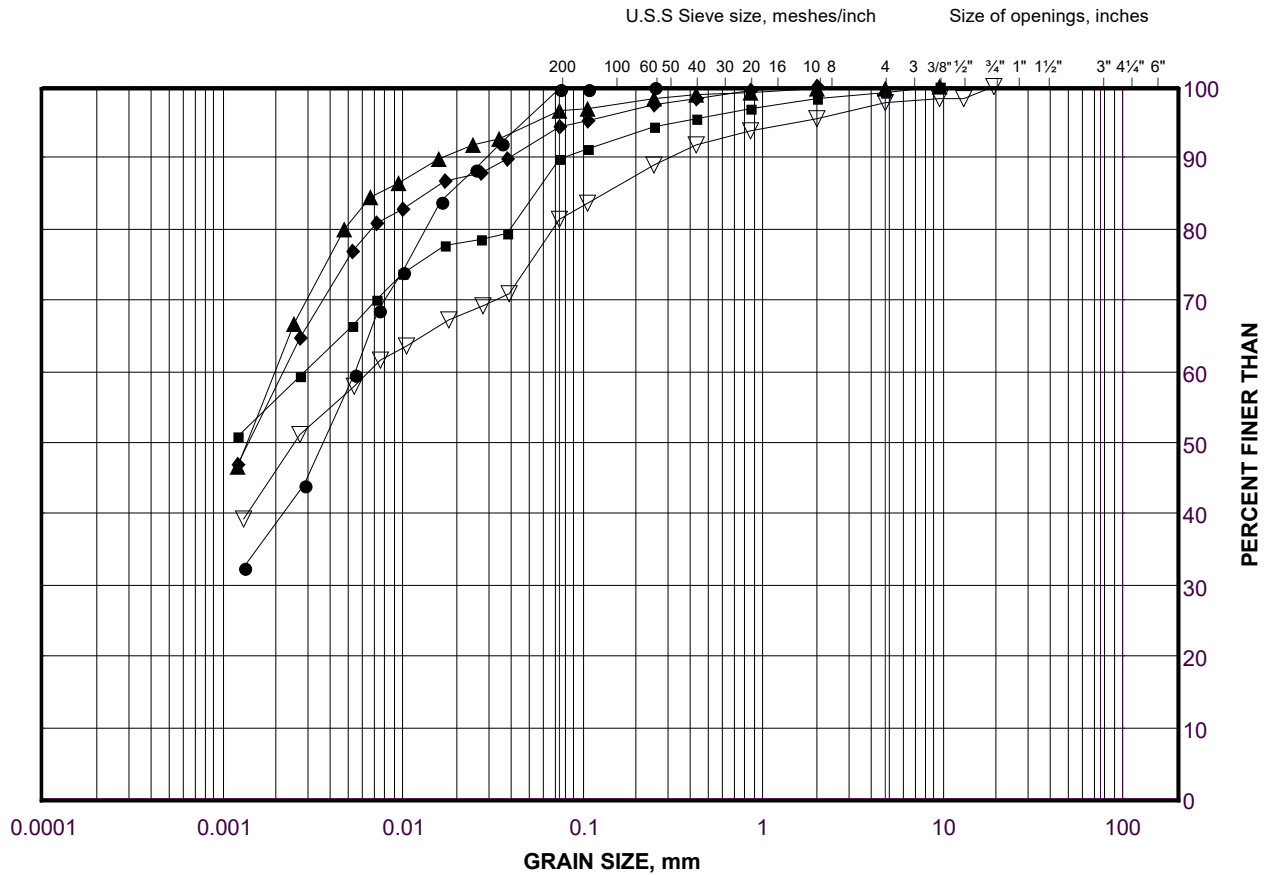
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

SILTY CLAY (CI) to CLAYEY SILT (CL)

FIGURE C6A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	L-5	10	131.6
■	N/E RS-2	10	131.0
◆	L-4	11	127.3
▲	L-5	14	127.0
▽	N/E RS-2	14	126.5

Project Number: 1786302

Checked By: CN

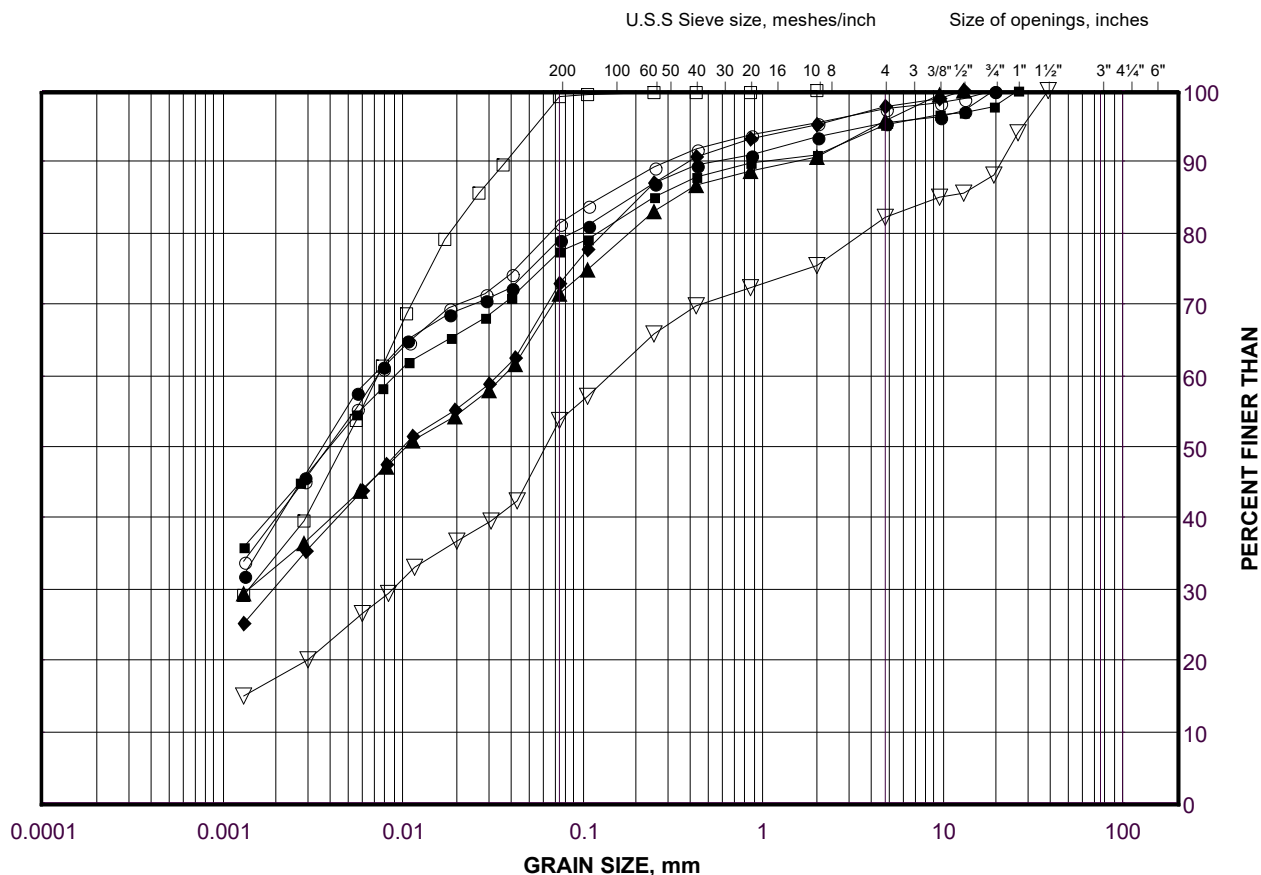
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

CLAYEY SILT (CL) to Sandy CLAYEY SILT (CL)

FIGURE C6B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	N/E RS-9	10	129.4
■	N/E RS-8	11	125.8
◆	L-1	11	129.9
▲	N/E RS-9	13	124.8
▽	N/E RS-6	13	121.7
○	N/E RS-8	14	122.8
□	N/E RS-4	8	131.4

Project Number: 1786302

Checked By: CN

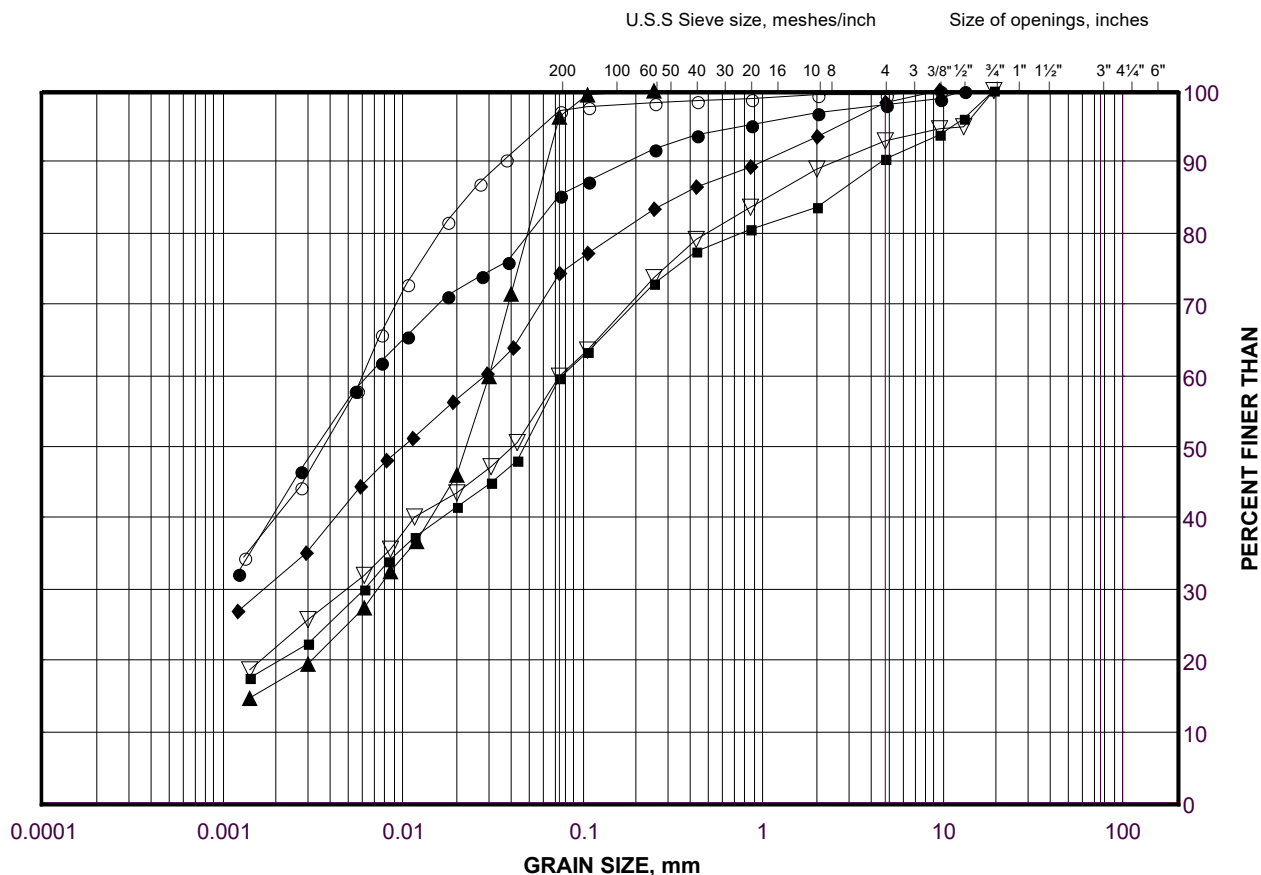
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

CLAYEY SILT (CL) to Sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

FIGURE C6C



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

LEGEND

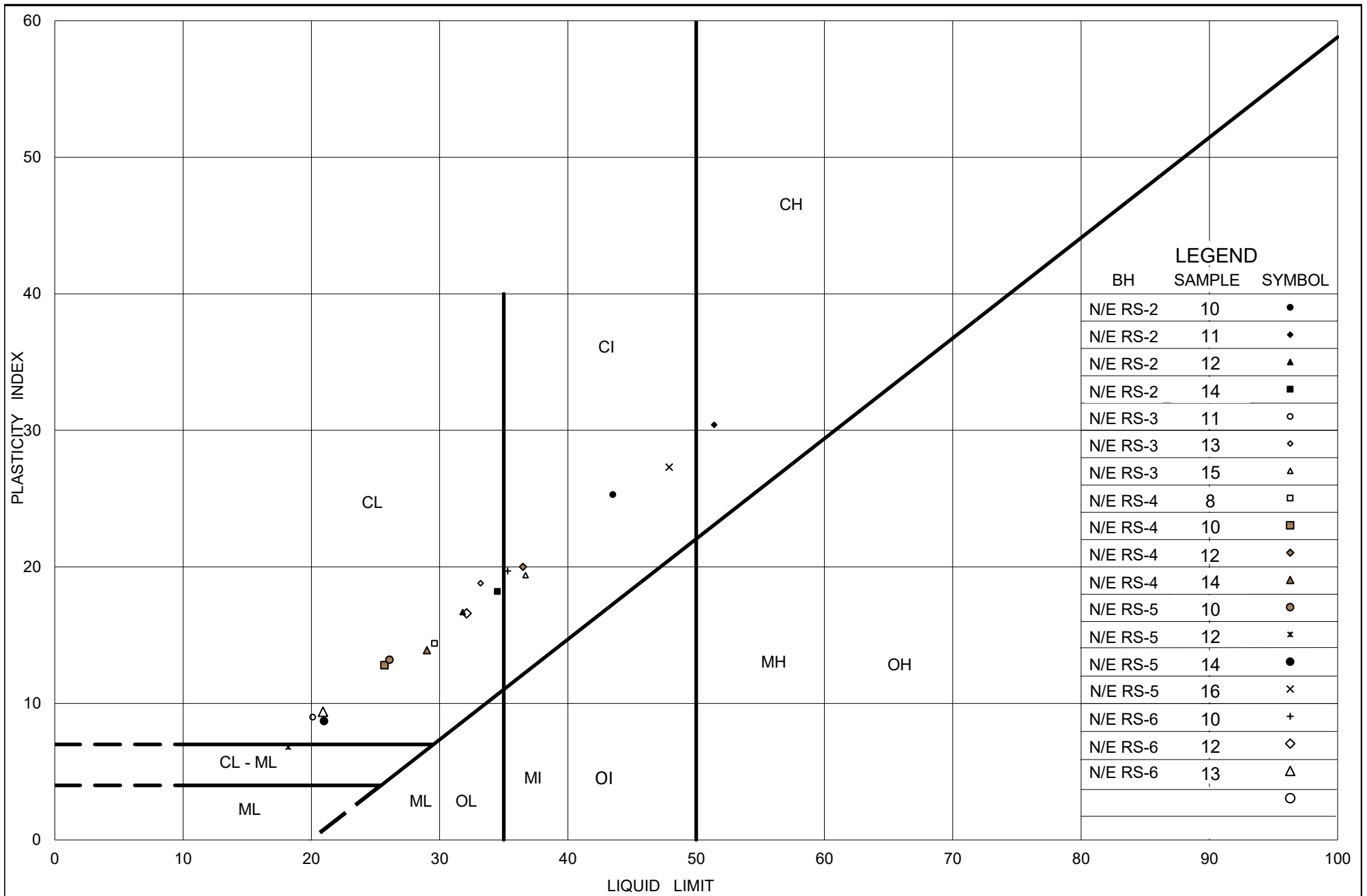
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	L-6	12	127.7
■	L-3	13	122.4
◆	N/E RS-10	15	121.3
▲	N/E RS-10	6	132.7
▽	L-3	8	130.0
○	N/E RS-10	8	127.4

Project Number: 1786302

Checked By: CN

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Date: 22-Oct-20



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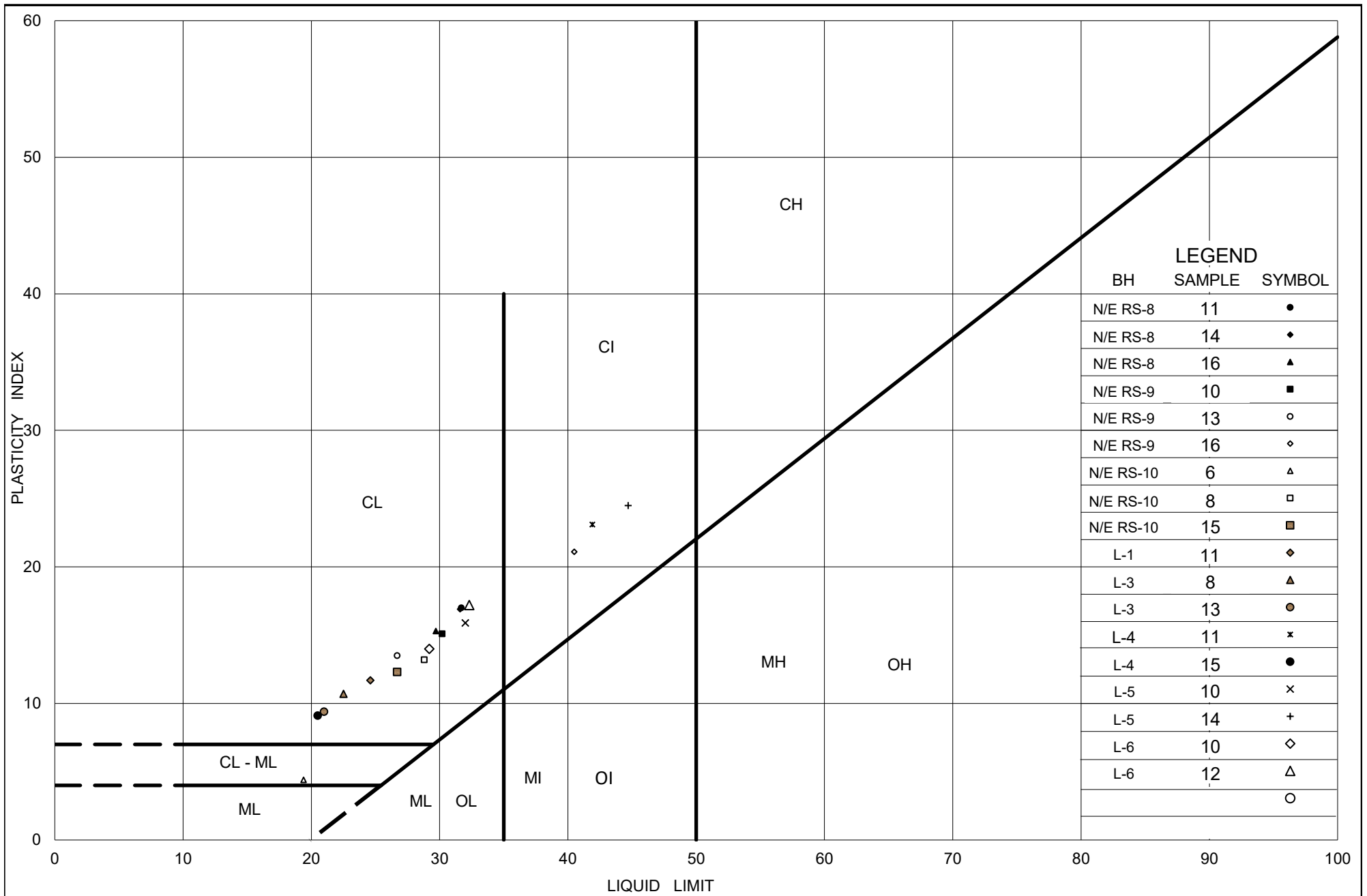
Ontario

PLASTICITY CHART CLAY (CH) to SILTY CLAY (CI) to CLAYEY SILT (CL)

Figure No. C7A

Project No. 1786302

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Ontario

PLASTICITY CHART
 SILTY CLAY (CI) to CLAYEY SILT (CL) to Sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

Figure No. C7B

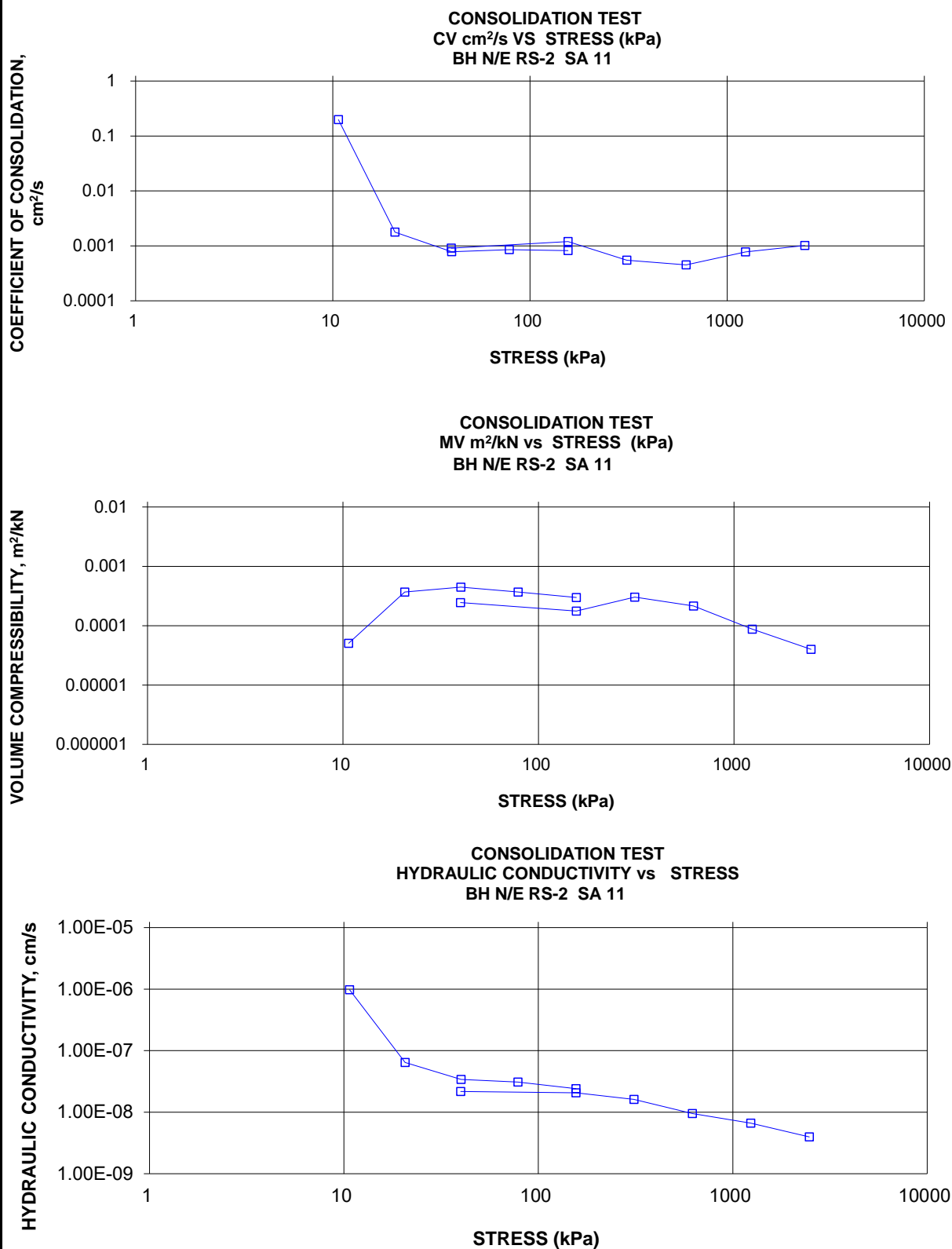
Project No. 1786302

Checked By: CN

CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE C8A CLAY (CH)		
SAMPLE IDENTIFICATION							
Project Number	1786302 (1000)	Sample Number	11				
Borehole Number	N/E RS-2	Sample Depth, m	11.43-12.01				
TEST CONDITIONS							
Test Type	Laboratory Standard	Load Duration, hr	24				
Oedometer Number	1						
Date Started	10/31/2019						
Date Completed	11/14/2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.56	Unit Weight, kN/m ³	17.23				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	11.85				
Area, cm ²	31.67	Specific Gravity, measured	2.74				
Volume, cm ³	80.91	Solids Height, cm	1.126				
Water Content, %	45.49	Volume of Solids, cm ³	35.67				
Wet Mass, g	142.20	Volume of Voids, cm ³	45.24				
Dry Mass, g	97.74	Degree of Saturation, %	98.3				
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	2.555	1.268	2.555				
6.03	2.557	1.270	2.556				
10.68	2.556	1.270	2.557	7	1.98E-01	5.05E-05	9.80E-07
20.66	2.547	1.261	2.552	778	1.77E-03	3.69E-04	6.41E-08
40.05	2.525	1.242	2.536	1750	7.79E-04	4.46E-04	3.41E-08
78.56	2.489	1.209	2.507	1561	8.53E-04	3.69E-04	3.09E-08
156.00	2.429	1.157	2.459	1561	8.21E-04	3.00E-04	2.41E-08
40.05	2.455	1.179	2.442				
10.63	2.489	1.210	2.472				
39.86	2.471	1.193	2.480	1441	9.05E-04	2.45E-04	2.17E-08
155.99	2.418	1.147	2.444	1058	1.20E-03	1.76E-04	2.06E-08
310.18	2.299	1.041	2.359	2160	5.46E-04	3.03E-04	1.62E-08
619.43	2.129	0.890	2.214	2306	4.51E-04	2.15E-04	9.49E-09
1237.69	1.991	0.767	2.060	1162	7.74E-04	8.77E-05	6.65E-09
2475.11	1.865	0.655	1.928	778	1.01E-03	3.99E-05	3.96E-09
619.43	1.911	0.696	1.888				
156.00	1.989	0.766	1.950				
39.80	2.075	0.842	2.032				
10.63	2.150	0.908	2.112				
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 34-40cm from top of the tube. Specimen swelled under 6.03 kPa							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	2.15	Unit Weight, kN/m ³	18.75				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.08				
Area, cm ²	31.67	Specific Gravity, measured	2.74				
Volume, cm ³	68.07	Solids Height, cm	1.126				
Water Content, %	33.13	Volume of Solids, cm ³	35.67				
Wet Mass, g	130.12	Volume of Voids, cm ³	32.40				
Dry Mass, g	97.74						
<div style="display: flex; justify-content: space-between;"> Prepared By: SJ Golder Associates Checked By: </div>							

CONSOLIDATION TEST SUMMARY

FIGURE C8B
CLAY (CH)



Project No. 1786302 (1000)

Prepared By: SJ

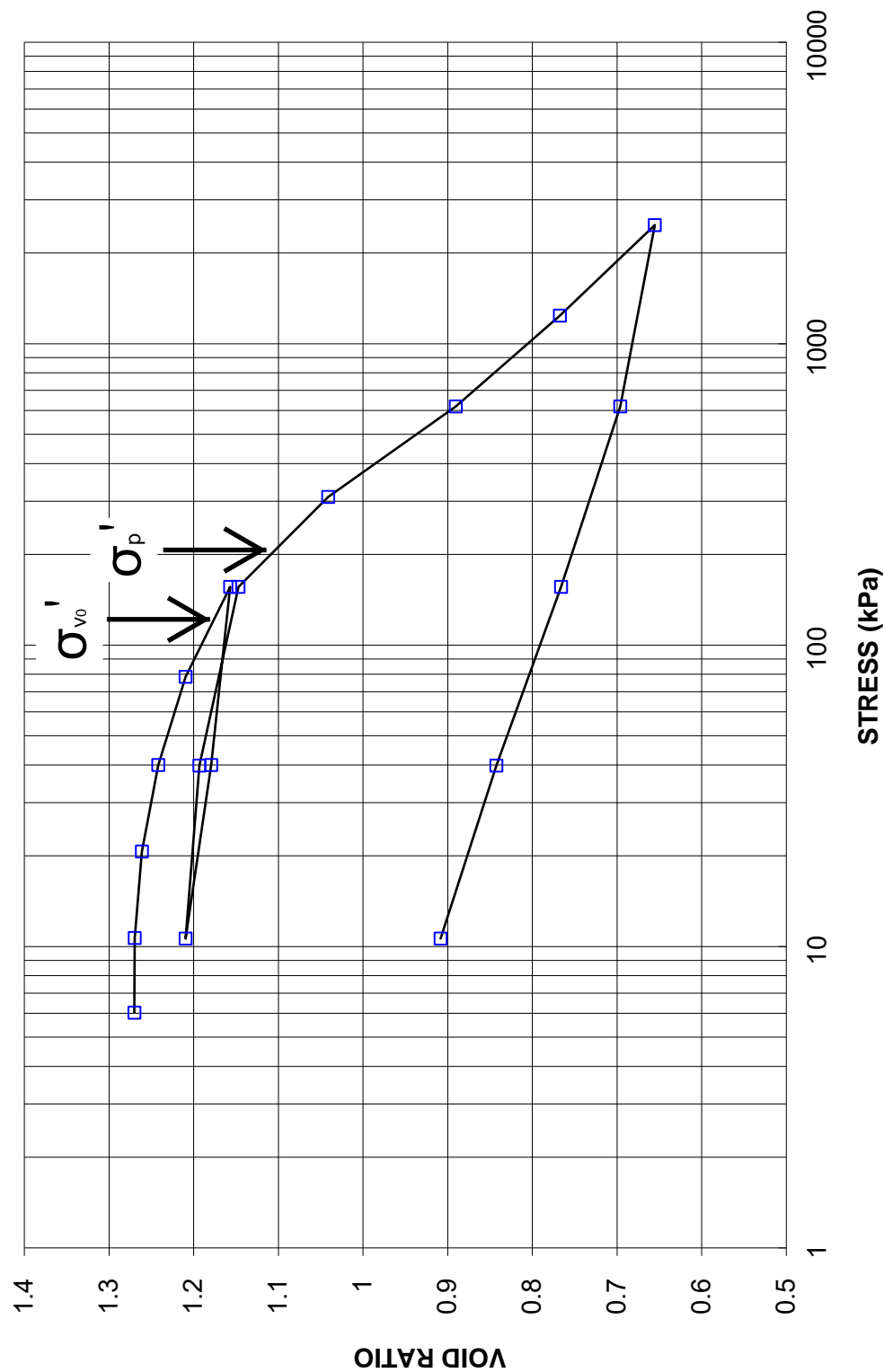
Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C8C
CLAY (CH)

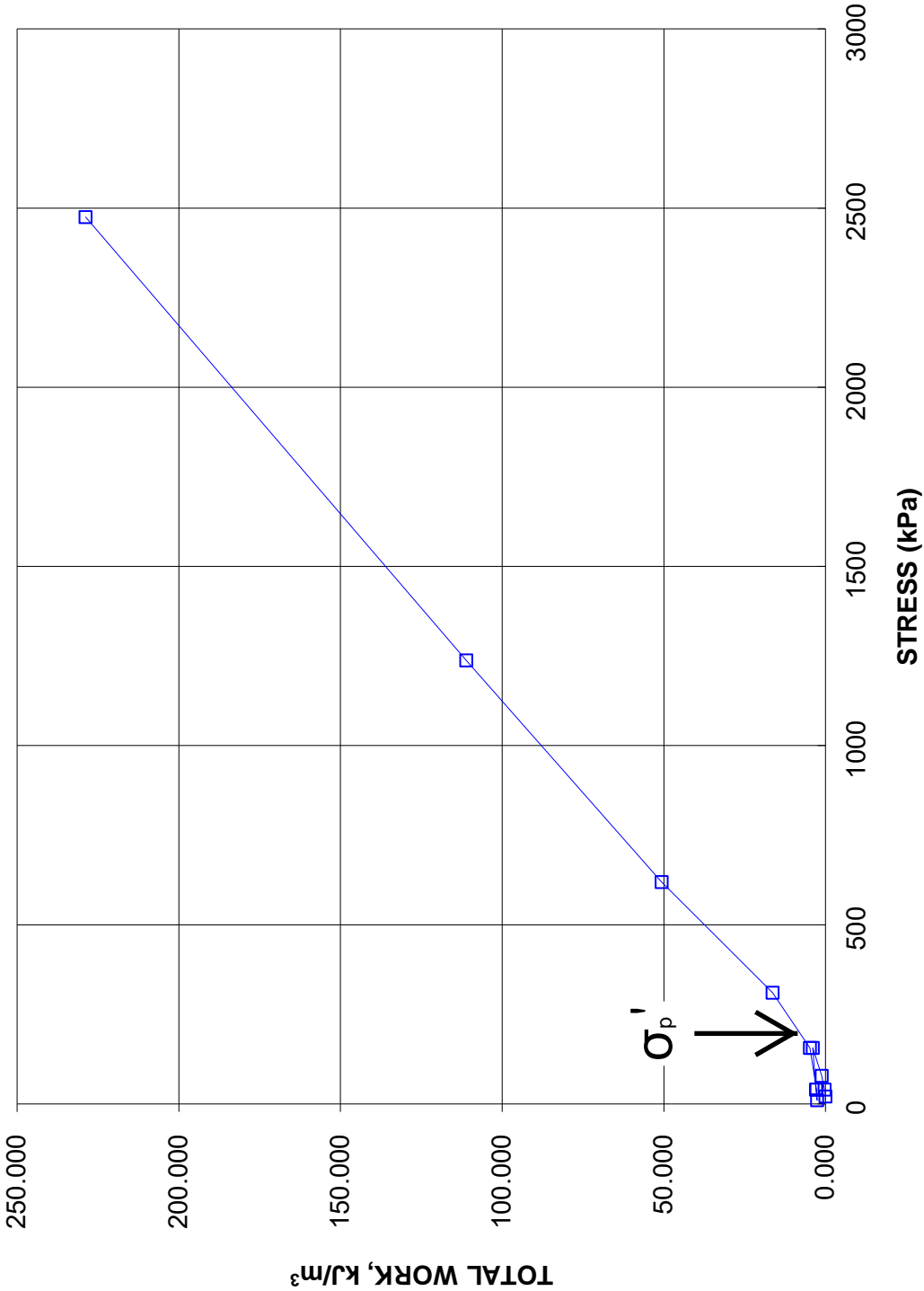
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH N/E RS-2 SA 11



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C8D
CLAY (CH)

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH N/E RS-2 SA 11



CONSOLIDATION TEST SUMMARY**ASTM D2435/D2435M****FIGURE C9A****CLAYEY SILT-SILT (CL-ML)****SAMPLE IDENTIFICATION**

Project Number	1786302 (2000)	Sample Number	SA12
Borehole Number	NE RS-5	Sample Depth, m	13.72-14.33

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	12/12/2019		
Date Completed	1/02/2020		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	22.17
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	19.25
Area, cm ²	31.52	Specific Gravity, measured	2.72
Volume, cm ³	59.92	Solids Height, cm	1.372
Water Content, %	15.14	Volume of Solids, cm ³	43.24
Wet Mass, g	135.43	Volume of Voids, cm ³	16.68
Dry Mass, g	117.62	Degree of Saturation, %	106.8

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.901	0.386	1.901				
6.53	1.887	0.376	1.894	2667	2.85E-04	1.10E-03	3.08E-08
11.15	1.882	0.372	1.885	3197	2.36E-04	6.07E-04	1.40E-08
21.00	1.872	0.365	1.877	2492	3.00E-04	5.34E-04	1.57E-08
40.52	1.860	0.356	1.866	1058	6.98E-04	3.23E-04	2.21E-08
79.44	1.845	0.345	1.852	735	9.90E-04	2.08E-04	2.02E-08
147.99	1.829	0.333	1.837	505	1.42E-03	1.18E-04	1.64E-08
40.47	1.832	0.335	1.830				
11.15	1.837	0.339	1.834				
40.52	1.834	0.337	1.835	60	1.19E-02	5.55E-05	6.48E-08
147.99	1.825	0.330	1.829	86	8.25E-03	4.28E-05	3.46E-08
316.31	1.804	0.315	1.815	217	3.22E-03	6.45E-05	2.03E-08
626.89	1.771	0.291	1.788	217	3.12E-03	5.72E-05	1.75E-08
1248.01	1.739	0.267	1.755	135	4.83E-03	2.72E-05	1.29E-08
2491.00	1.697	0.237	1.718	60	1.04E-02	1.77E-05	1.81E-08
623.00	1.704	0.242	1.701				
147.98	1.713	0.249	1.709				
40.49	1.726	0.258	1.720				
11.15	1.735	0.265	1.731				

Note:

Consolidation loading and unloading schedule assigned by the client.

cv and k are approximate only based on t₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M)

Specimen taken 15-23cm from top of the tube.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

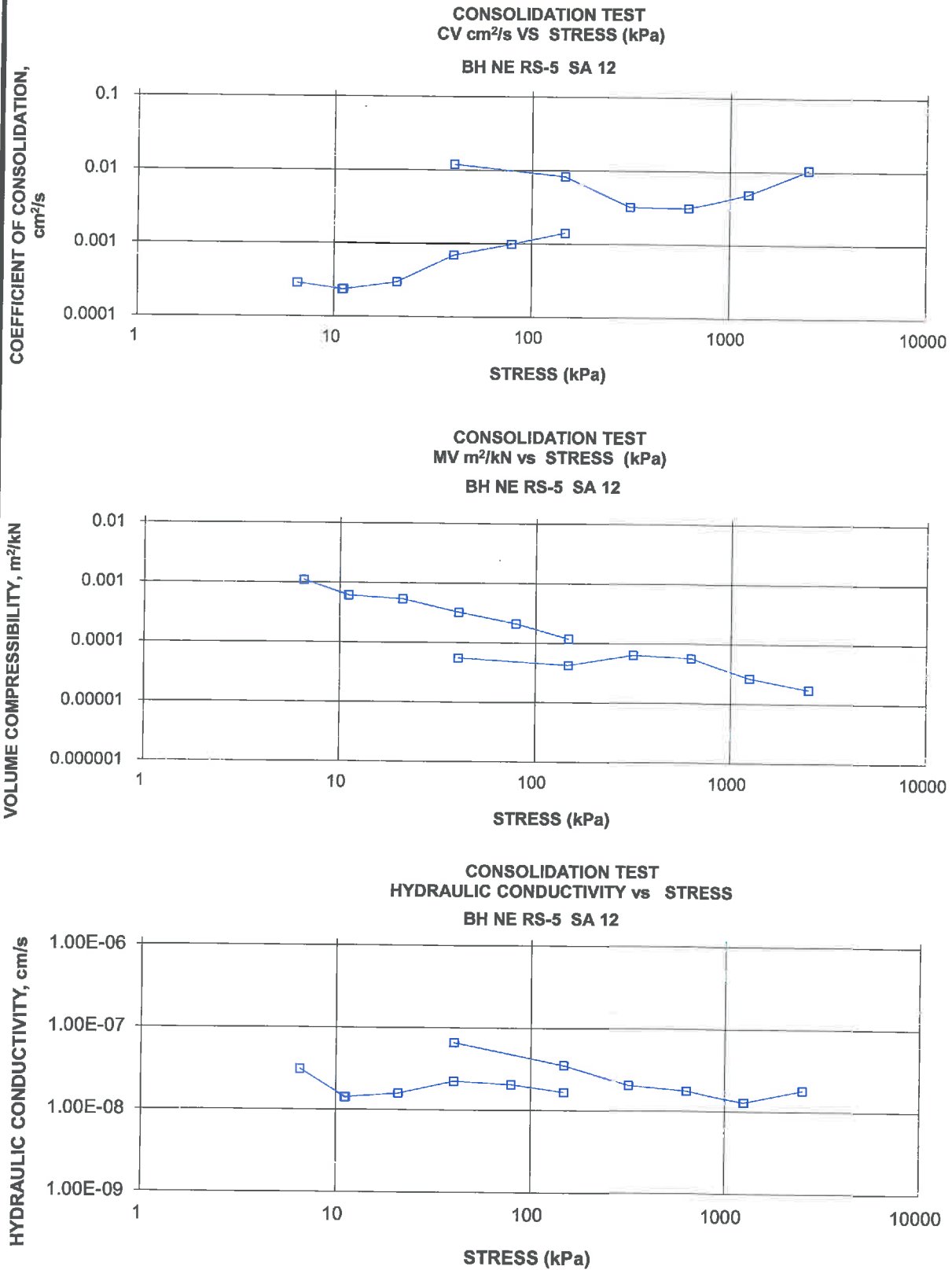
Sample Height, cm	1.74	Unit Weight, kN/m ³	23.45
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	21.09
Area, cm ²	31.52	Specific Gravity, measured	2.72
Volume, cm ³	54.70	Solids Height, cm	1.372
Water Content, %	11.21	Volume of Solids, cm ³	43.24
Wet Mass, g	130.80	Volume of Voids, cm ³	11.46
Dry Mass, g	117.62		

Prepared By: SJ

Golder AssociatesChecked By: 

CONSOLIDATION TEST SUMMARY

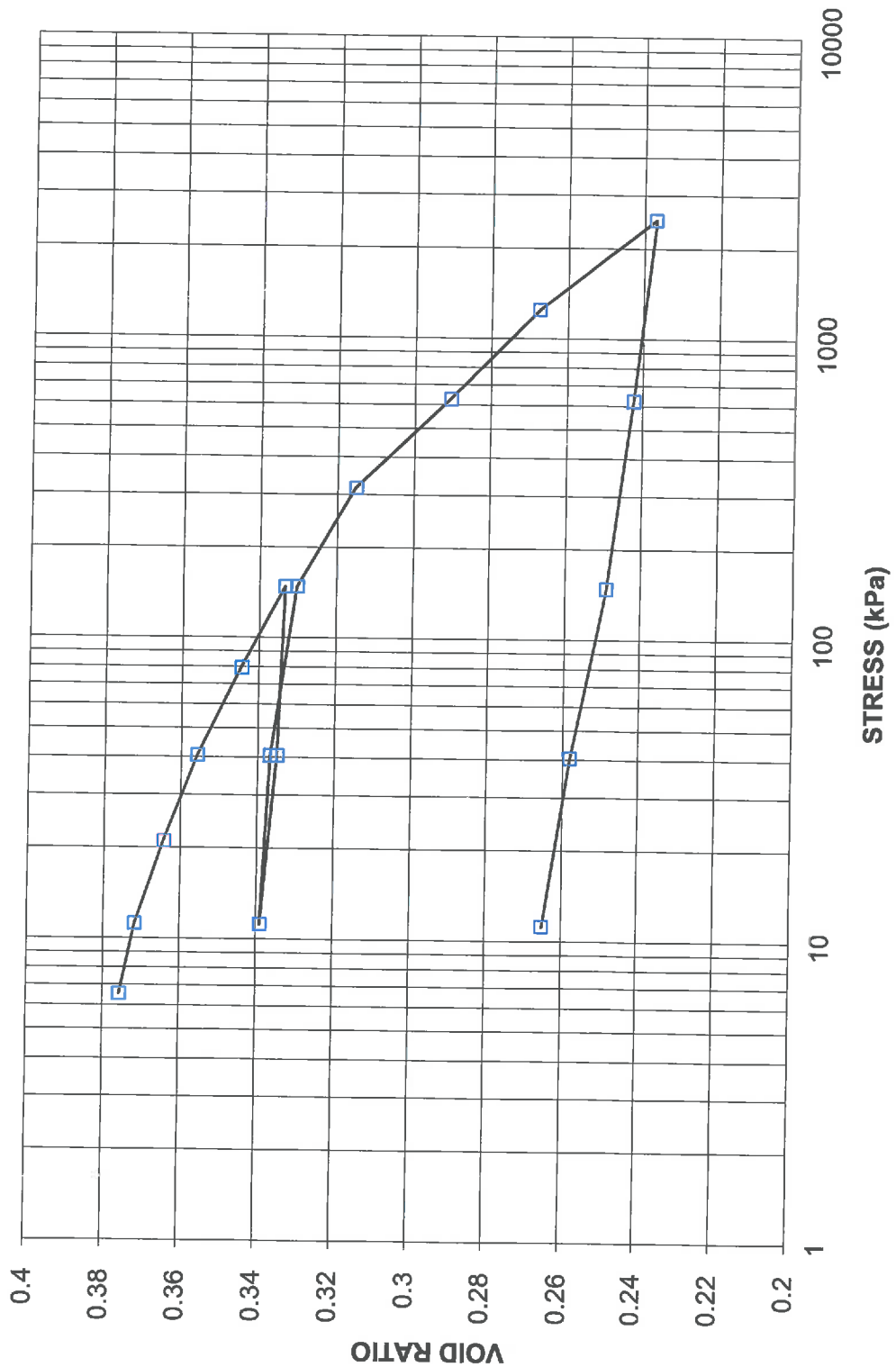
FIGURE C9B



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C9C

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH NE RS-5 SA 12



Project No. 1786302(2000)

Prepared By: SJ

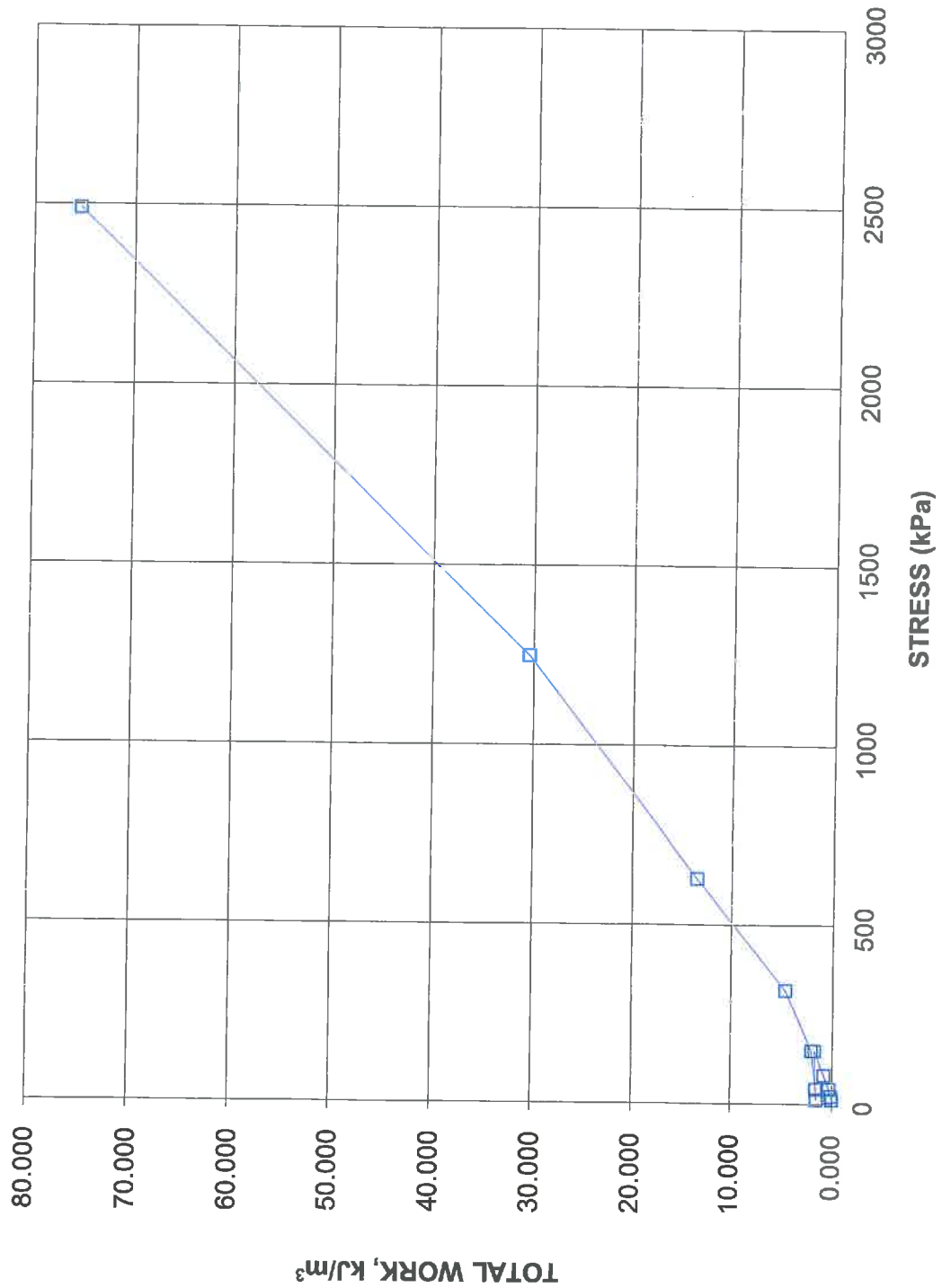
Golder Associates

Checked By: *[Signature]*

**CONSOLIDATION TEST
TOTAL WORK VS STRESS**

FIGURE C9D

**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH NE RS-5 SA 12**



Project No. 1786302(2000)

Prepared By: SJ

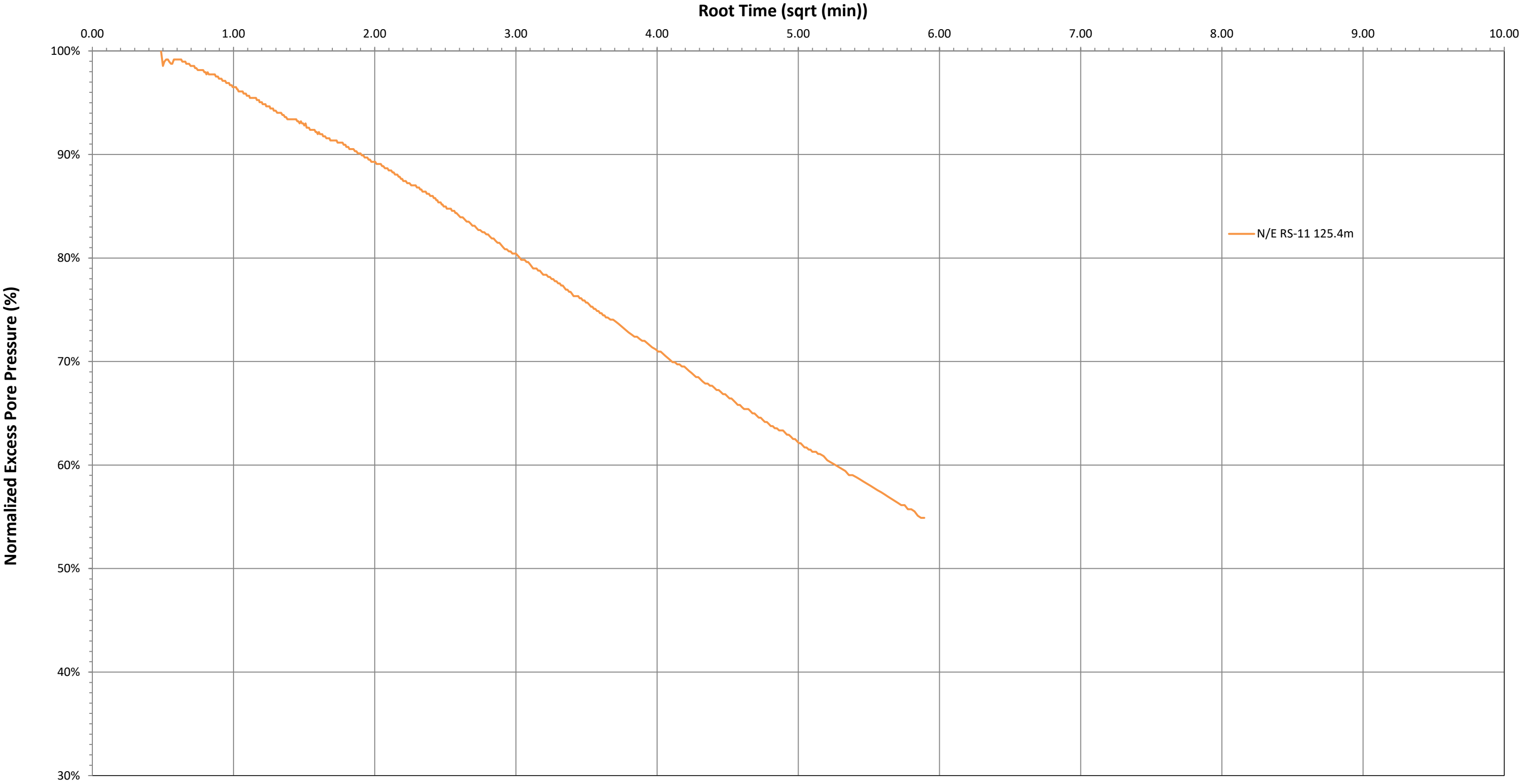
Golder Associates

Checked By: *[Signature]*

https://golderassociates.sharepoint.com/sites/20393g/Technical Work/Foundations/06 - Analysis/0.0 Material Parameters/[1786302 CPT Dissipation Tests 2020Feb26.xlsx] Leslie St

SUMMARY PLOT OF PORE WATER DISSIPATION TESTING
Leslie Street Bridge at Highway 401 (Site No. 37X-0208/B1)

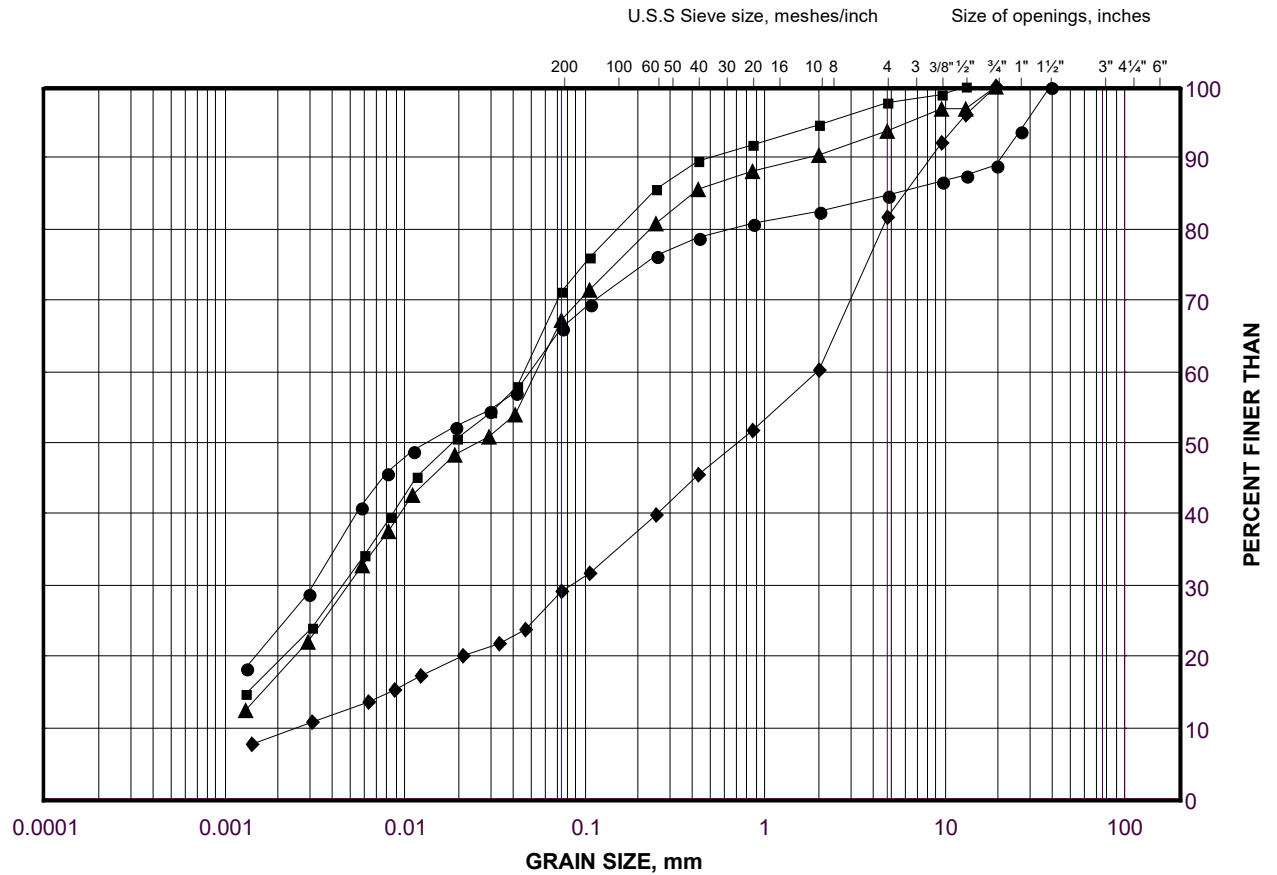
FIGURE C10



GRAIN SIZE DISTRIBUTION

CLAYEY SILT (CL) to Sandy CLAYEY SILT (CL) to CLAYEY SAND (SC)
(TILL)

FIGURE C11A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	N/E RS-3	16	116.5
■	N/E RS-10	17	115.7
◆	N/E RS-9	18	115.7
▲	N/E RS-2	18	117.3

Project Number: 1786302

Checked By: CN

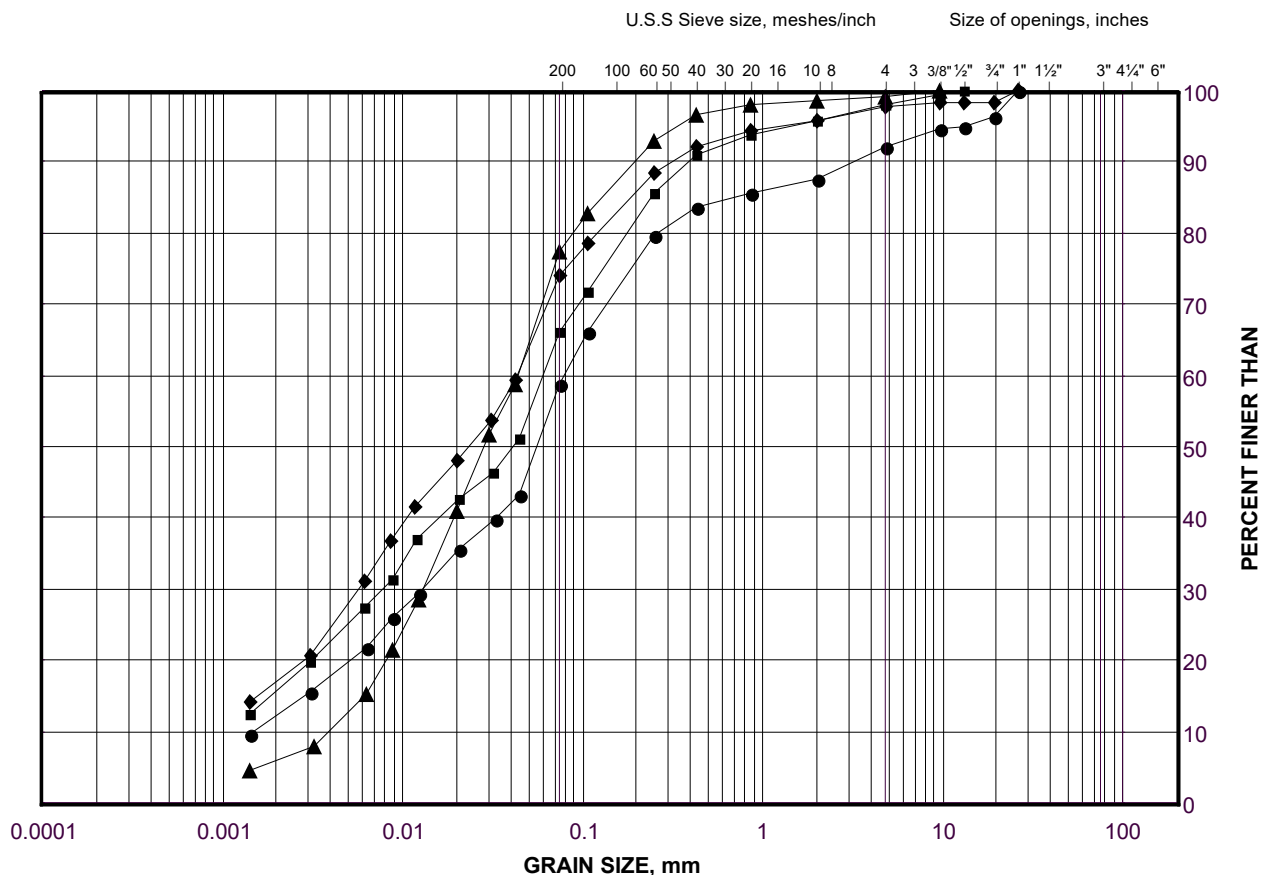
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT-SILT (CL-ML) to Sandy SILT (ML)
of slight plasticity (TILL)

FIGURE C11B



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

LEGEND

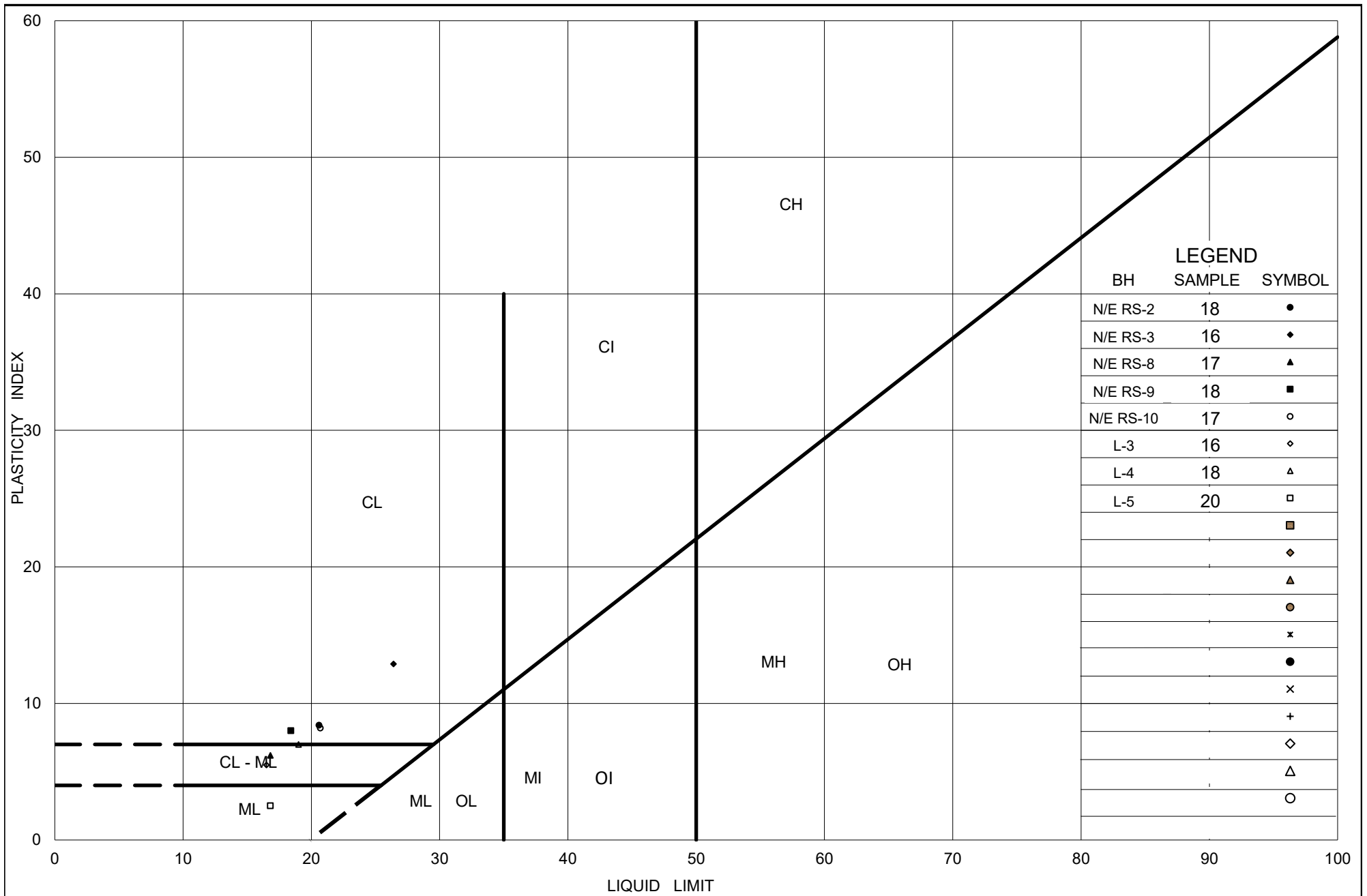
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	L-3	16	117.8
■	N/E RS-8	17	118.2
◆	L-4	18	116.7
▲	L-5	20	114.9

Project Number: 1786302

Checked By: CN

Golder Associates

Date: 22-Oct-20



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy CLAYEY SILT (CL) to Sandy CLAYEY SILT-SILT (CL-ML) to CLAYEY SAND (TILL)

Figure No. C12

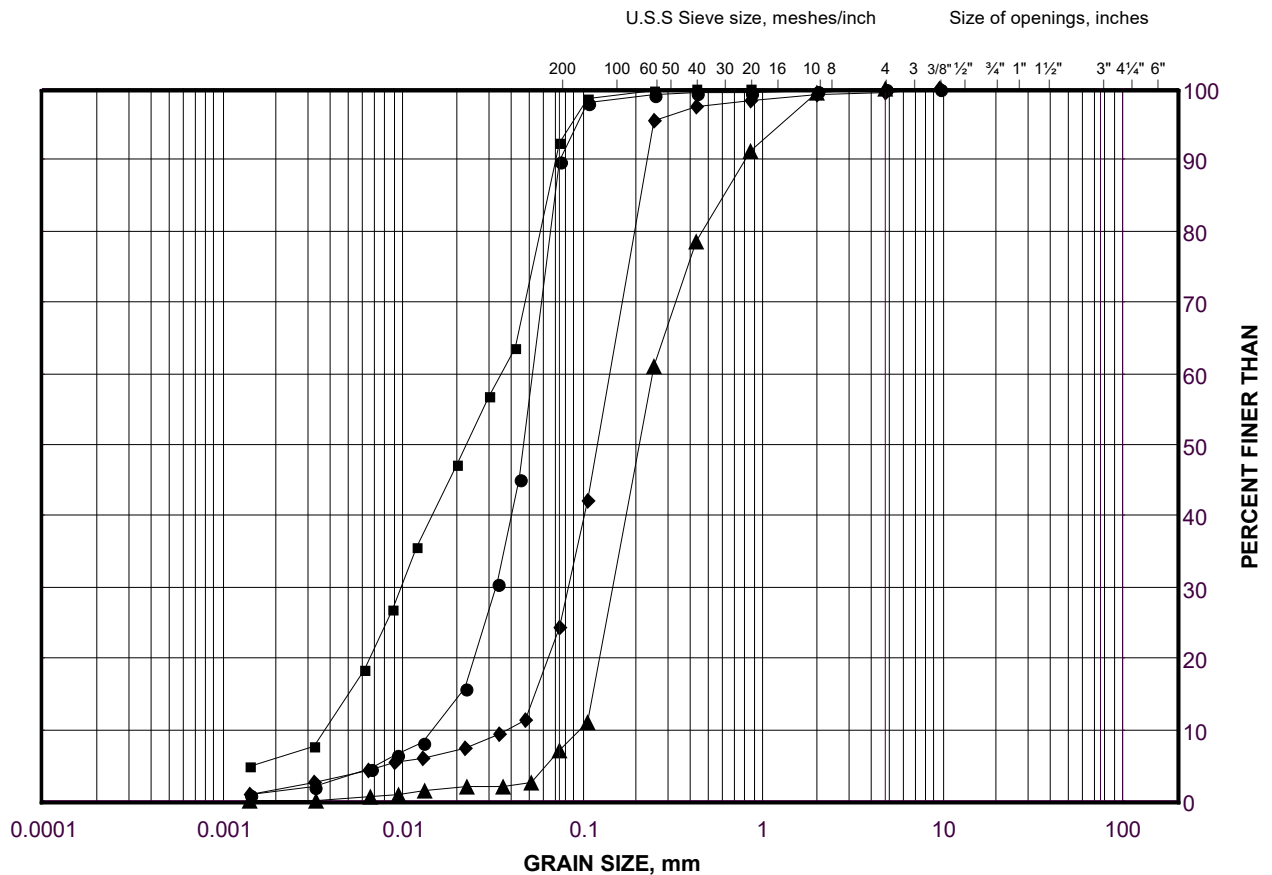
Project No. 1786302

Checked By: CN

GRAIN SIZE DISTRIBUTION

SILT (ML) to SILTY SAND (SM) to SAND (SP-SM)

FIGURE C13A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	N/E RS-6	15	117.2
■	N/E RS-4	15	117.7
◆	N/E RS-6	17	111.2
▲	N/E RS-4	17	111.6

Project Number: 1786302

Checked By: CN

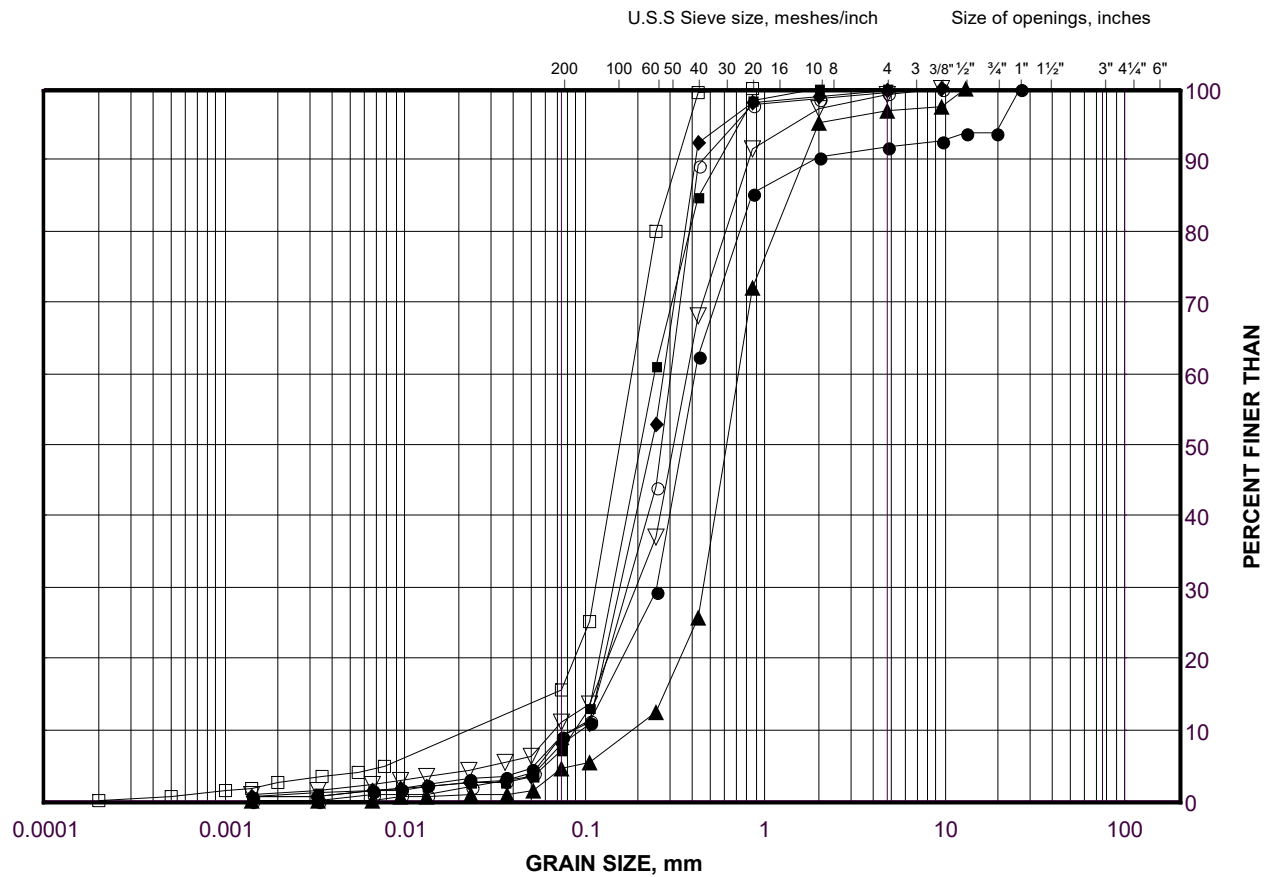
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

SILTY SAND (SM) to SAND (SP-SM) to SAND (SP)

FIGURE C13B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	N/E RS-5	17	112.6
■	L-3	18	111.7
◆	N/E RS-10	19	109.6
▲	N/E RS-3	19	107.4
▽	N/E RS-9	20	109.6
○	L-4	20	110.6
□	N/E RS-2	20	111.2

Project Number: 1786302

Checked By: CN

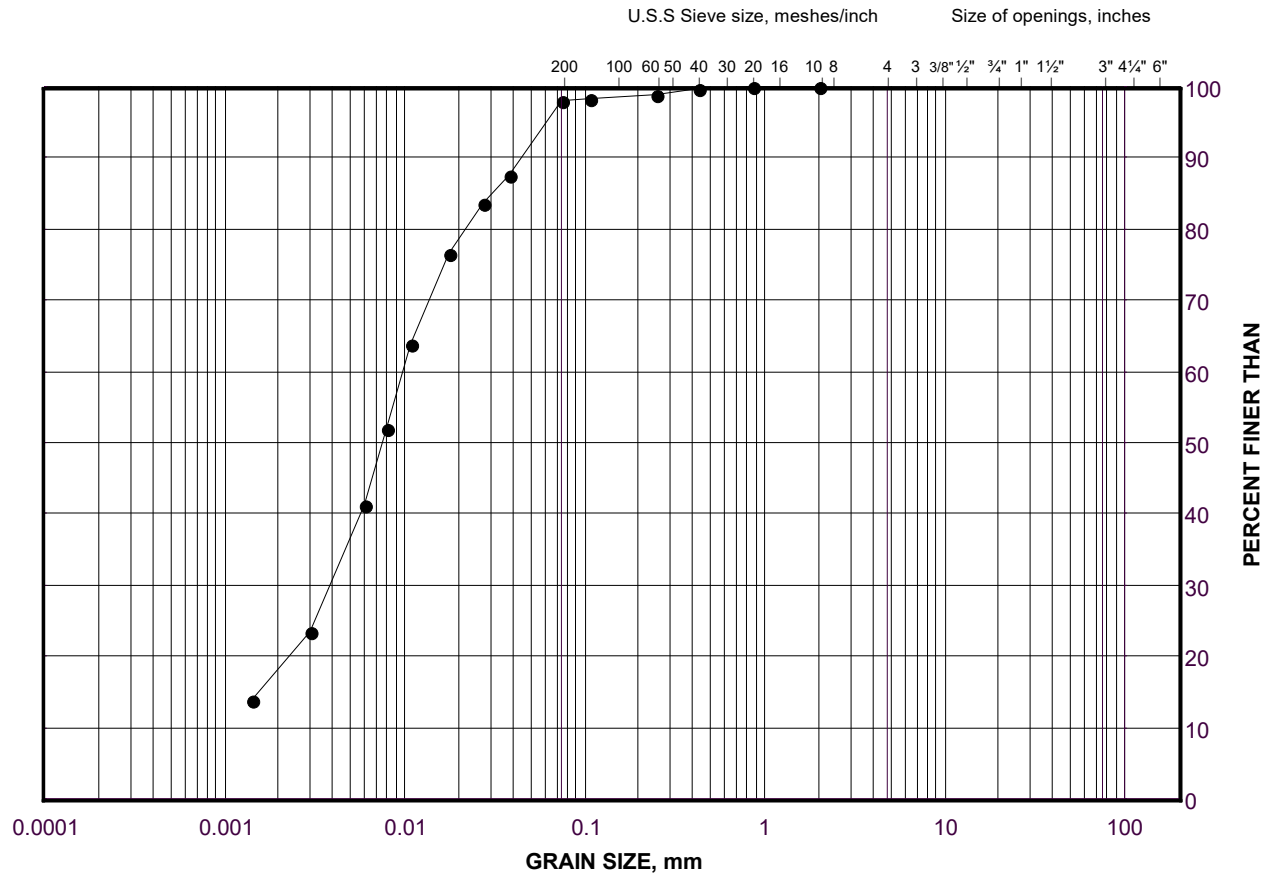
Golder Associates

Date: 22-Oct-20

GRAIN SIZE DISTRIBUTION

CLAYEY SILT (CL)

FIGURE C14



LEGEND

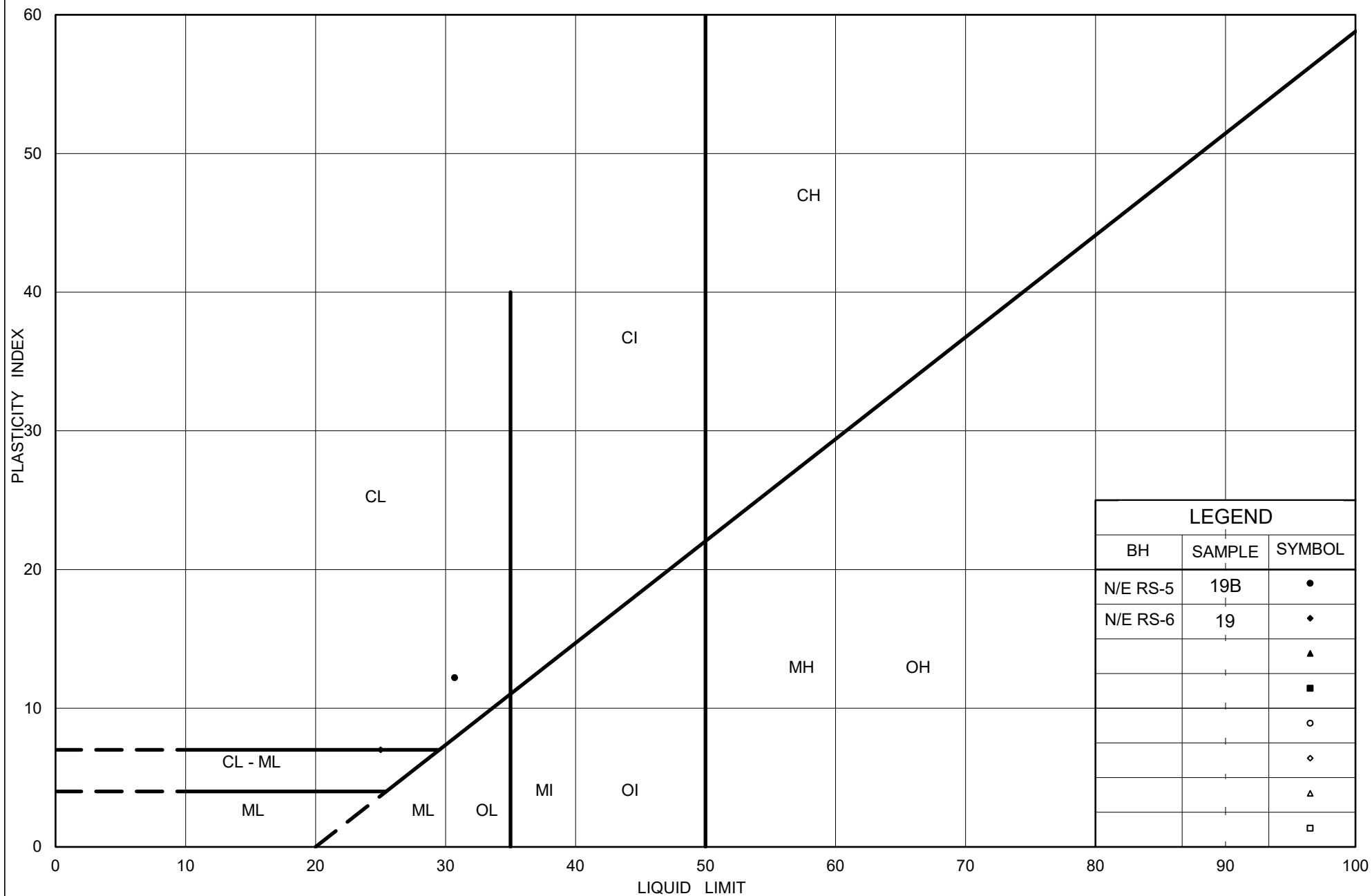
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	N/E RS-5	19B	106.4

Project Number: 1786302

Checked By: CN

Golder Associates

Date: 22-Oct-20



Ministry of Transportation

Ontario

PLASTICITY CHART CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

Figure No. C15

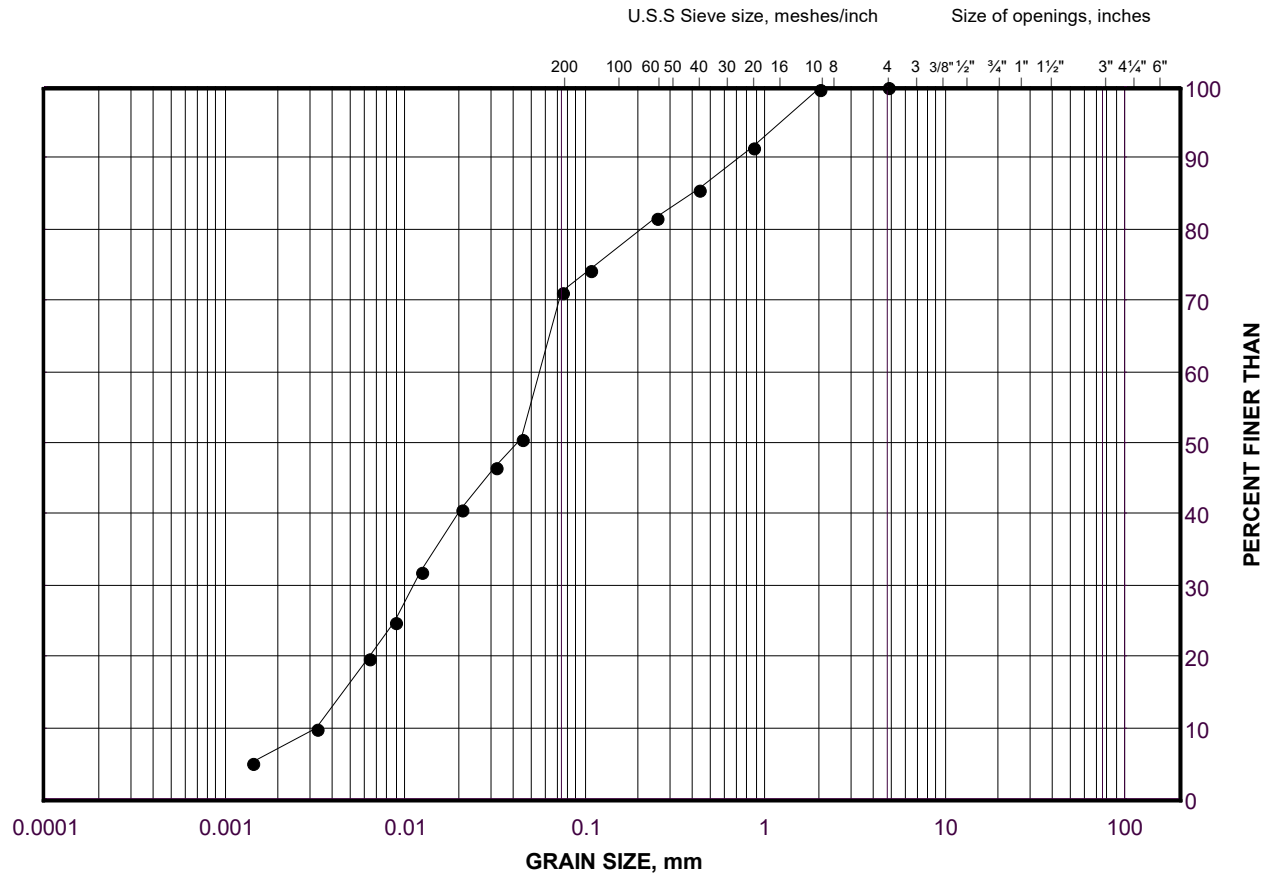
Project No. 1786302

Checked By: CN

GRAIN SIZE DISTRIBUTION

Sandy Organic SILT (OH)

FIGURE C16



LEGEND

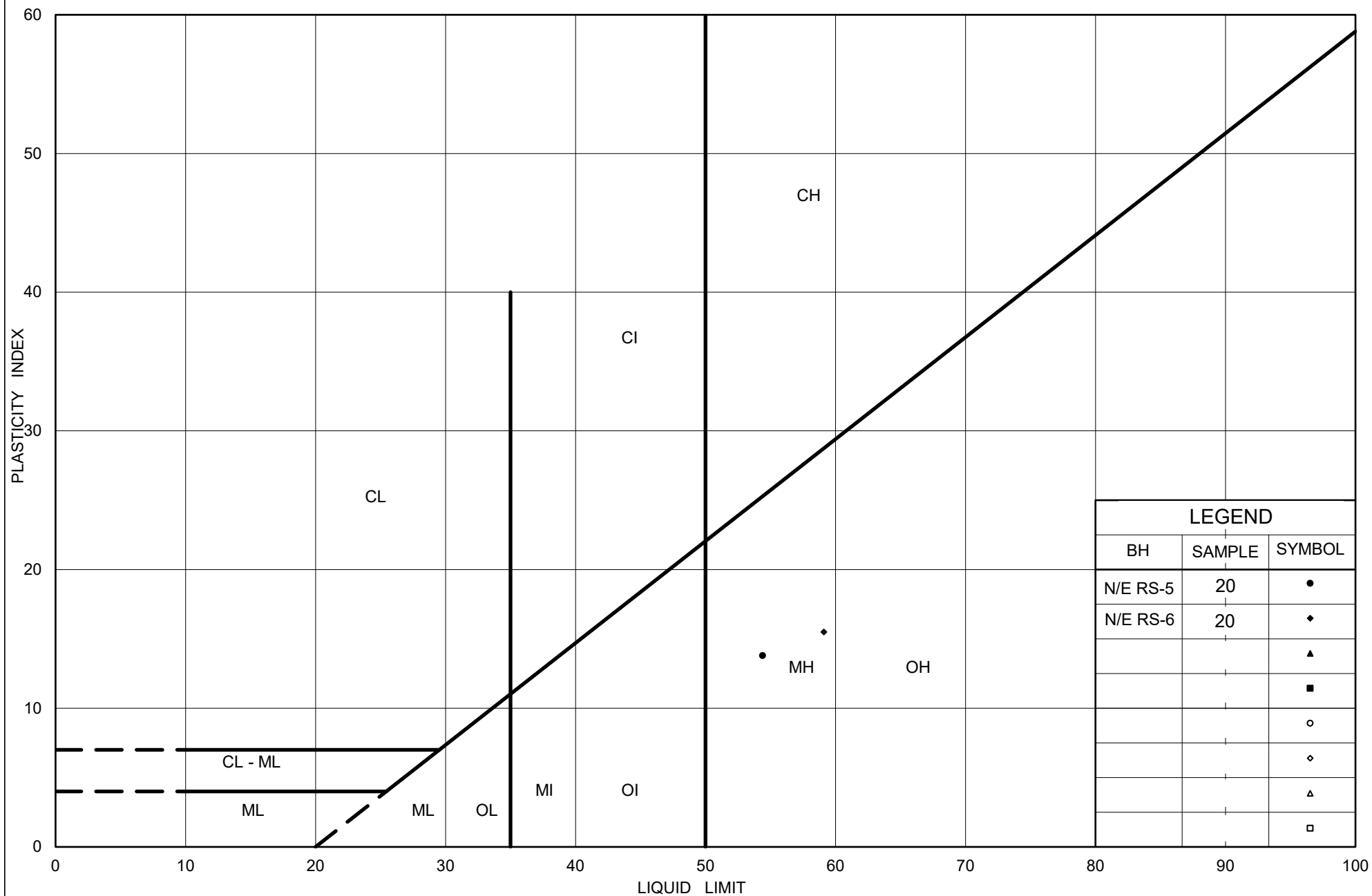
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	N/E RS-6	20	103.4

Project Number: 1786302

Checked By: CN

Golder Associates

Date: 22-Oct-20



Ministry of Transportation

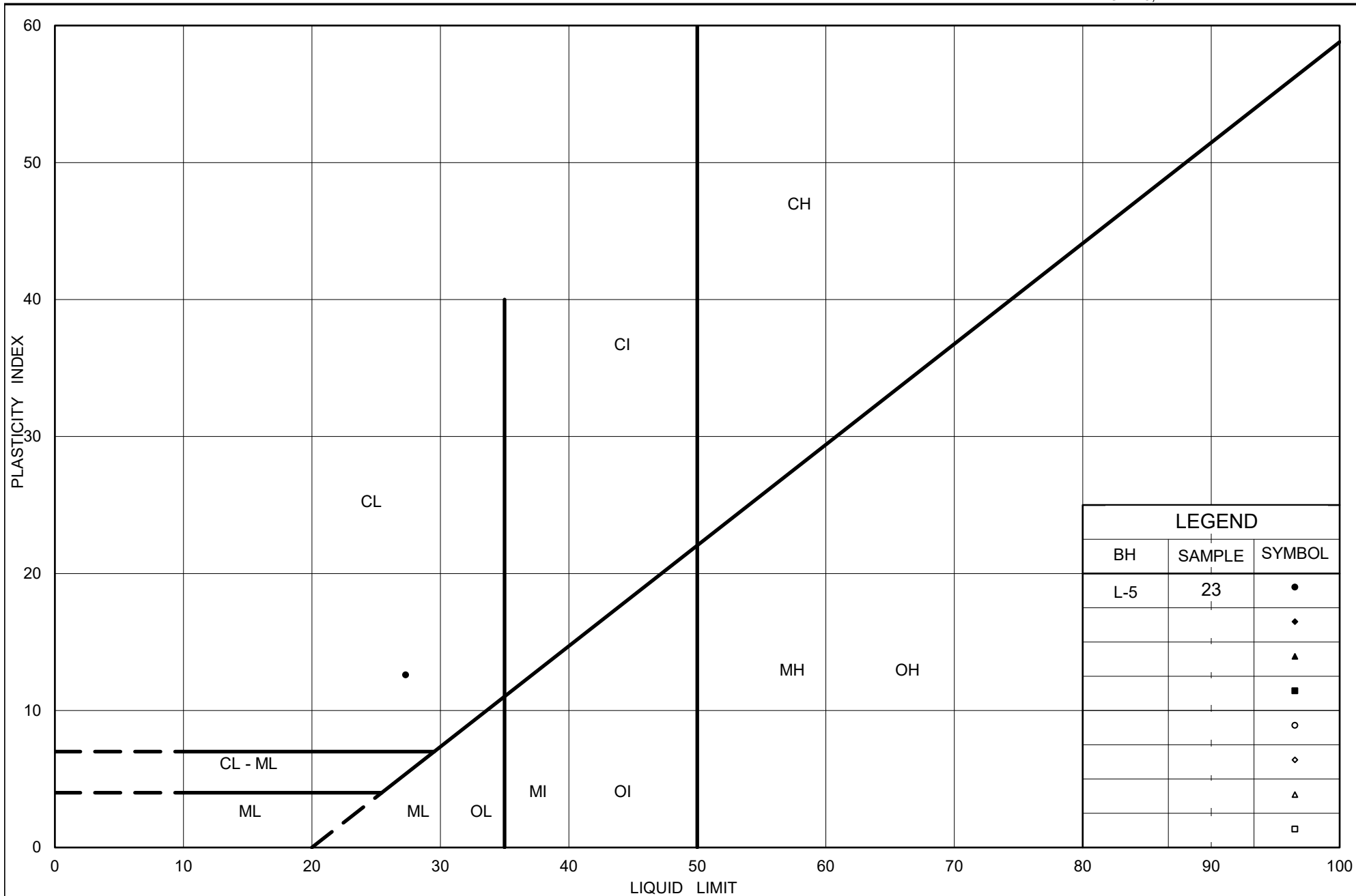
Ontario

PLASTICITY CHART Sandy Organic SILT (OH) to Organic SILT (OH)

Figure No. C17

Project No. 1786302

Checked By: CN



Ministry of Transportation

PLASTICITY CHART Sandy SILTY CLAY (CL) (RESIDUAL SOIL)

Ontario

Figure No. C18

Project No. 1786302

Checked By: CN



Your Project #: 1786302
Site Location: EHWY 401 / LESLIE
Your C.O.C. #: 729512-02-01

Attention: Katelyn Nero

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

Report Date: 2019/11/25
Report #: R5979847
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9W7428

Received: 2019/11/20, 16:33

Sample Matrix: Soil
Samples Received: 5

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	5	2019/11/25	2019/11/25	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	5	2019/11/25	2019/11/25	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	5	2019/11/23	2019/11/25	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	5	2019/11/21	2019/11/25	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	5	2019/11/25	2019/11/25	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

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

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

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

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



Your Project #: 1786302
Site Location: EHWY 401 / LESLIE
Your C.O.C. #: 729512-02-01

Attention: Katelyn Nero

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

Report Date: 2019/11/25
Report #: R5979847
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9W7428
Received: 2019/11/20, 16:33

Encryption Key

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

=====

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BUREAU
VERITAS

BV Labs Job #: B9W7428
Report Date: 2019/11/25

Golder Associates Ltd
Client Project #: 1786302
Site Location: EHWY 401 / LESLIE
Sampler Initials: RM

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		LJB251			LJB251			LJB252	LJB253		
Sampling Date		2019/07/25			2019/07/25			2019/08/28	2019/07/11		
COC Number		729512-02-01			729512-02-01			729512-02-01	729512-02-01		
	UNITS	DR6_SA5	RDL	QC Batch	DR6_SA5 Lab-Dup	RDL	QC Batch	DR5_SA6	NERS3_SA5	RDL	QC Batch

Calculated Parameters

Resistivity	ohm-cm	1800		6455226				1200	7700		6455226
-------------	--------	------	--	---------	--	--	--	------	------	--	---------

Inorganics

Soluble (20:1) Chloride (Cl-)	ug/g	63	20	6460811				400	<20	20	6460811
Conductivity	umho/cm	541	2	6460675				823	130	2	6460675
Available (CaCl2) pH	pH	7.64		6460218				7.48	7.86		6460218
Soluble (20:1) Sulphate (SO4)	ug/g	350	20	6460812	360	20	6460812	<20	<20	20	6460812

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

Lab-Dup = Laboratory Initiated Duplicate

BV Labs ID		LJB254	LJB255			LJB255		
Sampling Date		2019/07/08	2019/11/05			2019/11/05		
COC Number		729512-02-01	729512-02-01			729512-02-01		
	UNITS	NERS6_SA7	L5_SA7	RDL	QC Batch	L5_SA7 Lab-Dup	RDL	QC Batch

Calculated Parameters

Resistivity	ohm-cm	1200	2500		6455226			
-------------	--------	------	------	--	---------	--	--	--

Inorganics

Soluble (20:1) Chloride (Cl-)	ug/g	400	230	20	6460811	230	20	6460811
Conductivity	umho/cm	829	402	2	6460675			
Available (CaCl2) pH	pH	7.90	8.07		6460218			
Soluble (20:1) Sulphate (SO4)	ug/g	100	<20	20	6460812			

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

Lab-Dup = Laboratory Initiated Duplicate



BUREAU
VERITAS

BV Labs Job #: B9W7428

Report Date: 2019/11/25

Golder Associates Ltd

Client Project #: 1786302

Site Location: EHWY 401 / LESLIE

Sampler Initials: RM

TEST SUMMARY

BV Labs ID: LJB251
Sample ID: DR6_SA5
Matrix: Soil

Collected: 2019/07/25
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6460811	2019/11/25	2019/11/25	Deonarine Ramnarine
Conductivity	AT	6460675	2019/11/25	2019/11/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6460218	2019/11/23	2019/11/25	Surinder Rai
Resistivity of Soil		6455226	2019/11/25	2019/11/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6460812	2019/11/25	2019/11/25	Alina Dobreanu

BV Labs ID: LJB251 Dup
Sample ID: DR6_SA5
Matrix: Soil

Collected: 2019/07/25
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	6460812	2019/11/25	2019/11/25	Alina Dobreanu

BV Labs ID: LJB252
Sample ID: DR5_SA6
Matrix: Soil

Collected: 2019/08/28
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6460811	2019/11/25	2019/11/25	Deonarine Ramnarine
Conductivity	AT	6460675	2019/11/25	2019/11/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6460218	2019/11/23	2019/11/25	Surinder Rai
Resistivity of Soil		6455226	2019/11/25	2019/11/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6460812	2019/11/25	2019/11/25	Alina Dobreanu

BV Labs ID: LJB253
Sample ID: NERS3_SA5
Matrix: Soil

Collected: 2019/07/11
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6460811	2019/11/25	2019/11/25	Deonarine Ramnarine
Conductivity	AT	6460675	2019/11/25	2019/11/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6460218	2019/11/23	2019/11/25	Surinder Rai
Resistivity of Soil		6455226	2019/11/25	2019/11/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6460812	2019/11/25	2019/11/25	Alina Dobreanu

BV Labs ID: LJB254
Sample ID: NERS6_SA7
Matrix: Soil

Collected: 2019/07/08
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6460811	2019/11/25	2019/11/25	Deonarine Ramnarine
Conductivity	AT	6460675	2019/11/25	2019/11/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6460218	2019/11/23	2019/11/25	Surinder Rai
Resistivity of Soil		6455226	2019/11/25	2019/11/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6460812	2019/11/25	2019/11/25	Alina Dobreanu



BUREAU
VERITAS

BV Labs Job #: B9W7428
Report Date: 2019/11/25

Golder Associates Ltd
Client Project #: 1786302
Site Location: EHWY 401 / LESLIE
Sampler Initials: RM

TEST SUMMARY

BV Labs ID: LJB255
Sample ID: L5_SA7
Matrix: Soil

Collected: 2019/11/05
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6460811	2019/11/25	2019/11/25	Deonarine Ramnarine
Conductivity	AT	6460675	2019/11/25	2019/11/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6460218	2019/11/23	2019/11/25	Surinder Rai
Resistivity of Soil		6455226	2019/11/25	2019/11/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6460812	2019/11/25	2019/11/25	Alina Dobreanu

BV Labs ID: LJB255 Dup
Sample ID: L5_SA7
Matrix: Soil

Collected: 2019/11/05
Shipped:
Received: 2019/11/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6460811	2019/11/25	2019/11/25	Deonarine Ramnarine



BUREAU
VERITAS

BV Labs Job #: B9W7428

Report Date: 2019/11/25

Golder Associates Ltd

Client Project #: 1786302

Site Location: EHWY 401 / LESLIE

Sampler Initials: RM

GENERAL COMMENTS

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

Package 1	9.3°C
-----------	-------

Results relate only to the items tested.



BUREAU
VERITAS

BV Labs Job #: B9W7428

Report Date: 2019/11/25

QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 1786302

Site Location: EHWWY 401 / LESLIE

Sampler Initials: RM

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6460218	Available (CaCl ₂) pH	2019/11/25			98	97 - 103			2.0	N/A
6460675	Conductivity	2019/11/25			101	90 - 110	<2	umho/cm	0.50	10
6460811	Soluble (20:1) Chloride (Cl ⁻)	2019/11/25	NC	70 - 130	102	70 - 130	<20	ug/g	1.8	35
6460812	Soluble (20:1) Sulphate (SO ₄)	2019/11/25	NC	70 - 130	105	70 - 130	<20	ug/g	2.0	35

N/A = Not Applicable

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

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

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

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

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



BUREAU
VERITAS

BV Labs Job #: B9W7428

Report Date: 2019/11/25

Golder Associates Ltd

Client Project #: 1786302

Site Location: EHWY 401 / LESLIE

Sampler Initials: RM

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

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

CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name: #2292 Golder Associates Ltd	Company Name: <u>Golder</u>	Quotation #: B80683	BV Labs Job #:	Bottle Order #:			
Attention: Accounts Payable	Attention: <u>Katelyn Nerd</u>	P.O. #: <u>1796302</u>					
Address: 100 Scotia Crt	Address:	Project: <u>HWY401/LESLIE</u>	COC #:		Project Manager:		
Whitby ON L1N 8Y6		Site #:			Ema Gitej		
Tel: (905) 723-2727 Fax: (905) 723-2182	Tel: <u>(905) 817-5700</u> Fax: <u>(905) 817-5777</u>	Sampled By: <u>KM/DH</u>	C#729512-02-01				
Email: AP_CustomerService@golder.com	Email: <u>knerd@golder.com</u>						
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY							
Regulation 153 (2011)		Other Regulations		Special Instructions		Turnaround Time (TAT) Required: Please provide advance notice for rush projects	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Medium/Fine	<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw					<input type="checkbox"/> Regular (Standard) TAT: (will be applied if Rush TAT is not specified): Standard TAT = 5-7 Working days for most tests.	
<input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse	<input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw					Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.	
<input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC	<input type="checkbox"/> MISA Municipality					<input type="checkbox"/> Job Specific Rush TAT (if applies to entire submission)	
<input type="checkbox"/> Table	<input type="checkbox"/> PWQO					Date Required: Time Required:	
Include Criteria on Certificate of Analysis (Y/N)?						Rush Confirmation Number: (call lab for #)	
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr-VI	O Reg 153 VOCs by HS	# of Bottles
1	DR6-SA5	July 25/19	AM	SOIL		X	1
2	DR5-SA6	Aug 28/19	AM	"		X	1
3	NERS3-SA5	July 11/19	AM	"		X	1
4	NERS6-SA7	July 8/19	AM	"		X	1
5	L5-SA7	Nov 5/19	AM	"		X	1
6							
7							
8							
9							
10							
* RELINQUISHED BY: (Signature/Print)				RECEIVED BY: (Signature/Print)		Laboratory Use Only	
Kate Nerd / Kate Nerd				Juni - COLENE CURTIS		Time Sensitive	
Date: (YY/MM/DD) 19/11/20				Date: (YY/MM/DD) 20/11/20		Temperature (°C) on Recv	
Time				Time 10:53		9/9/10 15	
# jars used and not submitted						Custody Seal	
						Present	
						Intact	
						Yes	
						No	
* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.							
* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.							
** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.							
SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS							
White: BV Labs Yellow: Client							

Bureau Veritas Canada (2019) Inc.



Your Project #: 1786302
Site Location: HWY 401/LESLIE
Your C.O.C. #: 778726-02-02, 778726-02-01

Attention: Katelyn Nero

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

Report Date: 2020/08/27
Report #: R6308766
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: COL5529

Received: 2020/08/21, 16:00

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2020/08/26	2020/08/26	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2020/08/26	2020/08/26	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2020/08/26	2020/08/26	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2020/08/24	2020/08/26	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2020/08/26	2020/08/26	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

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



Your Project #: 1786302
Site Location: HWY 401/LESLIE
Your C.O.C. #: 778726-02-02, 778726-02-01

Attention: Katelyn Nero

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

Report Date: 2020/08/27
Report #: R6308766
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C0L5529
Received: 2020/08/21, 16:00

Encryption Key

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

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BUREAU
VERITAS

BV Labs Job #: COL5529
Report Date: 2020/08/27

Golder Associates Ltd
Client Project #: 1786302
Site Location: HWY 401/LESLIE
Sampler Initials: LM

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		NKZ915		NKZ915			NKZ916		
Sampling Date		2020/08/12 23:00		2020/08/12 23:00			2020/08/09 23:00		
COC Number		778726-02-01		778726-02-01			778726-02-01		
	UNITS	L3_SA3	QC Batch	L3_SA3 Lab-Dup	RDL	QC Batch	NERS9-SA4	RDL	QC Batch
Calculated Parameters									
Resistivity	ohm-cm	660	6905731				380		6905731
Inorganics									
Soluble (20:1) Chloride (Cl-)	ug/g	850	6910820	800	20	6910820	1700	60	6910820
Conductivity	umho/cm	1510	6910878	1460	2	6910878	2650	2	6910878
Available (CaCl2) pH	pH	8.14	6910280				7.92		6910280
Soluble (20:1) Sulphate (SO4)	ug/g	62	6910823	58	20	6910823	42	20	6910823
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate									



BUREAU
VERITAS

BV Labs Job #: COL5529
Report Date: 2020/08/27

Golder Associates Ltd
Client Project #: 1786302
Site Location: HWY 401/LESLIE
Sampler Initials: LM

TEST SUMMARY

BV Labs ID: NKZ915
Sample ID: L3_SA3
Matrix: Soil

Collected: 2020/08/12
Shipped:
Received: 2020/08/21

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6910820	2020/08/26	2020/08/26	Kazzandra Adeva
Conductivity	AT	6910878	2020/08/26	2020/08/26	Neil Dassanayake
pH CaCl2 EXTRACT	AT	6910280	2020/08/26	2020/08/26	Surinder Rai
Resistivity of Soil		6905731	2020/08/26	2020/08/26	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6910823	2020/08/26	2020/08/26	Kazzandra Adeva

BV Labs ID: NKZ915 Dup
Sample ID: L3_SA3
Matrix: Soil

Collected: 2020/08/12
Shipped:
Received: 2020/08/21

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6910820	2020/08/26	2020/08/26	Kazzandra Adeva
Conductivity	AT	6910878	2020/08/26	2020/08/26	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	6910823	2020/08/26	2020/08/26	Kazzandra Adeva

BV Labs ID: NKZ916
Sample ID: NERS9-SA4
Matrix: Soil

Collected: 2020/08/09
Shipped:
Received: 2020/08/21

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6910820	2020/08/26	2020/08/26	Kazzandra Adeva
Conductivity	AT	6910878	2020/08/26	2020/08/26	Neil Dassanayake
pH CaCl2 EXTRACT	AT	6910280	2020/08/26	2020/08/26	Surinder Rai
Resistivity of Soil		6905731	2020/08/26	2020/08/26	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6910823	2020/08/26	2020/08/26	Kazzandra Adeva



BUREAU
VERITAS

BV Labs Job #: COL5529

Report Date: 2020/08/27

Golder Associates Ltd

Client Project #: 1786302

Site Location: HWY 401/LESLIE

Sampler Initials: LM

GENERAL COMMENTS

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

Package 1	14.0°C
-----------	--------

Results relate only to the items tested.



BUREAU
VERITAS

BV Labs Job #: C0L5529

Report Date: 2020/08/27

QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 1786302

Site Location: HWY 401/LESLIE

Sampler Initials: LM

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6910280	Available (CaCl ₂) pH	2020/08/26			100	97 - 103			0.19	N/A
6910820	Soluble (20:1) Chloride (Cl ⁻)	2020/08/26	NC	70 - 130	105	70 - 130	<20	ug/g	6.1	35
6910823	Soluble (20:1) Sulphate (SO ₄)	2020/08/26	NC	70 - 130	102	70 - 130	<20	ug/g	6.9	35
6910878	Conductivity	2020/08/26			103	90 - 110	<2	umho/cm	2.9	10

N/A = Not Applicable

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

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

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

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

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



BUREAU
VERITAS

BV Labs Job #: COL5529

Report Date: 2020/08/27

Golder Associates Ltd

Client Project #: 1786302

Site Location: HWY 401/LESLIE

Sampler Initials: LM

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

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



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

CHAIN OF CUSTODY RECORD

Page 1 of 1

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name:	#1326 Golder Associates Ltd	Company Name:	Golder Associates	Quotation #:	C01600	BV Labs Job #:	Bottle Order #:
Attention:	Accounts Payable	Attention:	Adam Affleck Kate Nero	P.O. #:			
Address:	6925 Century Ave Suite 100	Address:		Project:	20143381 1786302		
	Mississauga ON L5N 7K2			Project Name:	HWY 401/LESLIE	COC #:	Project Manager:
Tel:	(905) 567-4444 Fax: (905) 567-6561	Tel:	(905) 567-6100 Ext: 1243 Fax: knero@golder.com	Site #:	HWY 401/LESLIE		
Email:	CanadaAccountsPayableInvoices@golder.com	Email:	adam.affleck@golder.com, Avisek.Chatterjee@golder.com	Sampled By:	LM / KN		Erna Gitej

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY				ANALYSIS REQUESTED (PLEASE BE SPECIFIC)												Turnaround Time (TAT) Required: Please provide advance notice for rush projects			
Regulation 153 (2011)		Other Regulations		Special Instructions		Field Filtered (please circle): Metals / Hg / Cr VI soil corrosivity package												Regular (Standard) TAT: (will be applied if Rush TAT is not specified): Standard TAT = 5-7 Working days for most tests. Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Table 2 <input type="checkbox"/> Table 3 <input type="checkbox"/> Table	<input type="checkbox"/> Res/Park <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Agri/Other	<input type="checkbox"/> Medium/Fine <input type="checkbox"/> Coarse <input type="checkbox"/> For RSC	<input type="checkbox"/> CCME <input type="checkbox"/> Reg 558 <input type="checkbox"/> MISA <input type="checkbox"/> PWQO <input type="checkbox"/> Other	<input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> Storm Sewer Bylaw Municipality	Include Criteria on Certificate of Analysis (Y/N)?													Job Specific Rush TAT (if applies to entire submission) Date Required: Time Required: <input type="checkbox"/> Rush Confirmation Number: (call lab for #)	
Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix													# of Bottles	Comments	
1	L3-SA3	Aug 12 2020	11 PM	SOIL													1		
2	NEQS9-SA4	Aug 9 2020	11 PM	SOIL													1		
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			

21-Aug-20 16:00

Erna Gitej



C0L5529

URE ENV-858

* RELINQUISHED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)		Date: (YY/MM/DD)	Time	# jars used and not submitted	Laboratory Use Only				
Lukas MacDonnell		20/08/21		[Signature]		20/08/21	1600		Time Sensitive	Temperature (°C) on Receipt	Custody Seal	Yes	No
										14/14/14	Present		
											Intact		

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.

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

White: BV Labs Yellow: Client

November 26, 2019

Ms. Katelyn Nero
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: UCS testing
(Golder Project No. 1786302)

Dear Ms. Nero:

On November 12, 2019 six (6) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder Personnel. These samples were identified as being from Golder project 1786302 / 2000 / 2610. From these samples, six (6) UCS tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results and photographs of the test specimens before and after testing are presented in the accompanying laboratory report and summary spreadsheet(s).

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

Katelyn Nero
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Prepared by:

Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#900-390 Bay St.
Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

November 26, 2019
Project number: 1786302

Abstract

This document summarizes the results of rock laboratory testing, including 6 uniaxial compressive strength (UCS) tests. The results including UCS and tangent Young's modulus along with photographs of test specimens before and after testing are presented.

In this document:

1 Uniaxial Compressive Strength Tests	1
Appendices	4

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.150 mm/min (Figure 1). The preparation and testing of each test specimen included the following:

1. Unwrapping of the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting of core sample to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within 0.25°).
4. Placing of the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS) and tangent Young's modulus.



Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-08. The side straightness criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 with the following exceptions:

- Tests included measurement of the UCS and elastic modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

1.2 Results

The results of the tests are summarized in Table 1. The corresponding stress-strain curves for the tests are presented in Figure 2. The Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength. Please note that additional specimen and testing details are available in the summary spreadsheet that accompanies this report.

Table 1: Summary of UCS test results.

Sample	Depth (m)	Bulk density ρ (g/cm ³)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
DR9	37.99 - 38.21	2.616	45.2	8.3	Georgian Bay Formation - Shale	1
DR6	30.23 - 30.42	2.627	37.2	8.6	Georgian Bay Formation - Shale	1
DR3	40.84 - 41.02	2.619	34.6	10.1	Georgian Bay Formation - Shale and Siltstone	1
N/E RS-2	40.62 - 40.79	2.615	47.4	7.8	Georgian Bay Formation - Shale and Siltstone	1
N/E RS-6	39.35 - 39.51	2.618	24.7	5.1	Georgian Bay Formation - Shale and Siltstone	1
N/E RS-4	37.59 - 37.69	2.611	21.9	2.6	Georgian Bay Formation - Shale and Siltstone	1, 2
Average		2.618	35.2	7.1		
Standard deviation		0.005	9.5	2.5		

¹ Axial splitting failure

² Length:Diameter ratio less than 2

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.

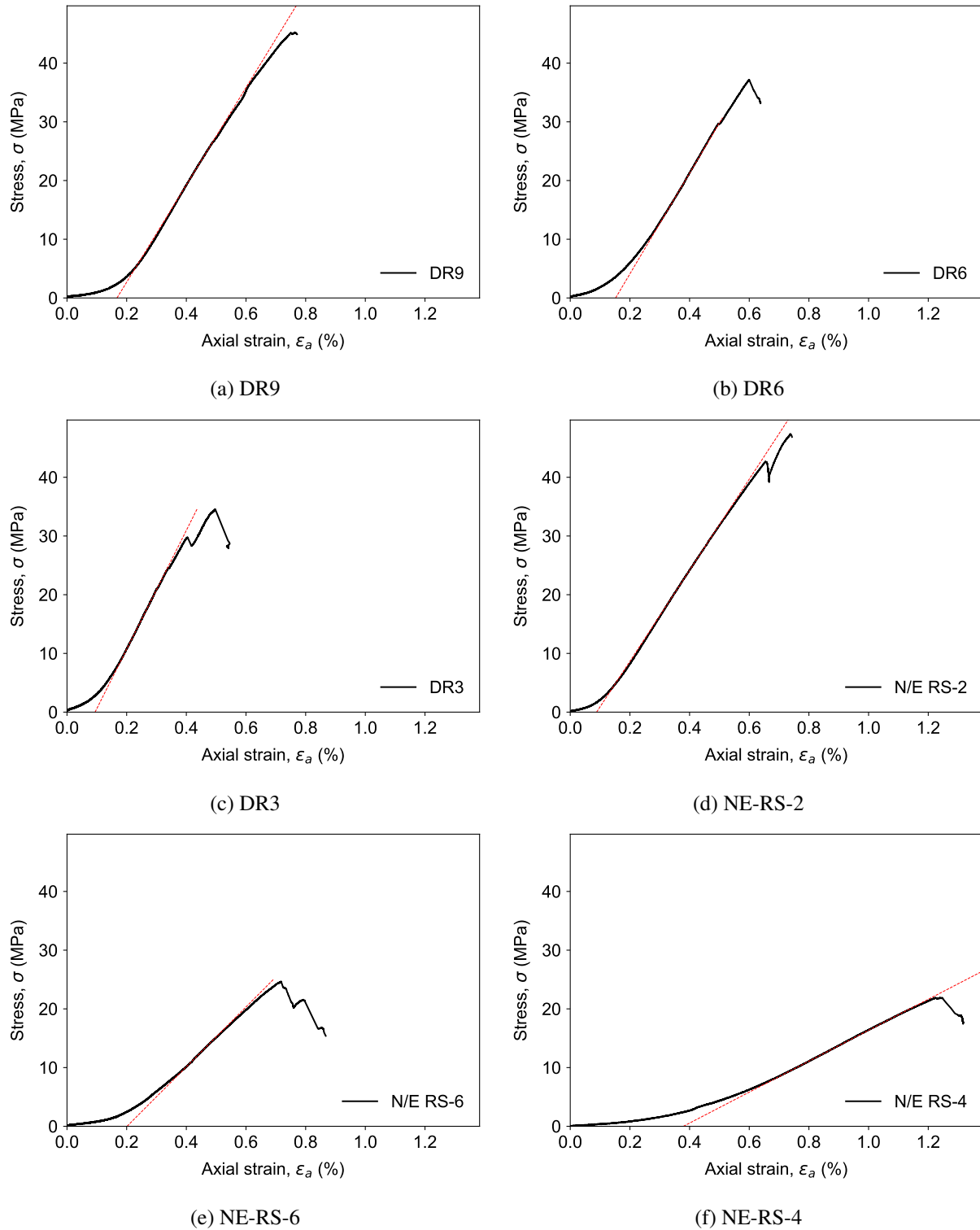




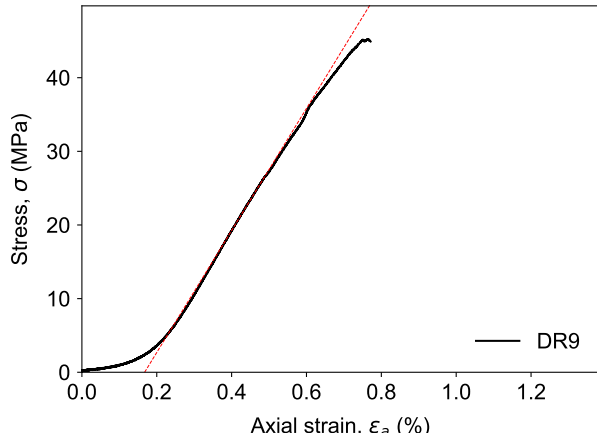
Figure 2: Measured stress-strain curves.

Appendices



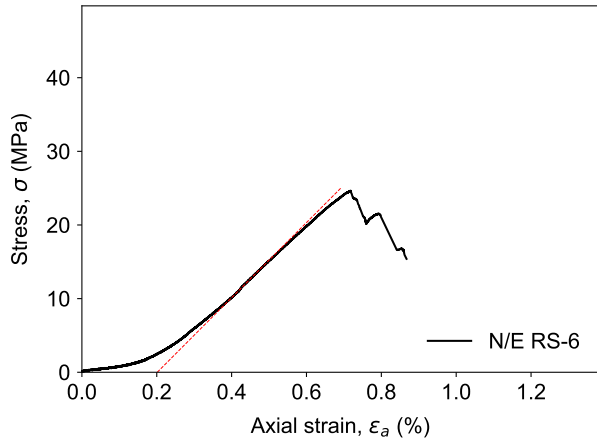
Specimen sheets

- DR9
- DR6
- DR3
- N/E RS-2
- N/E RS-6
- N/E RS-4



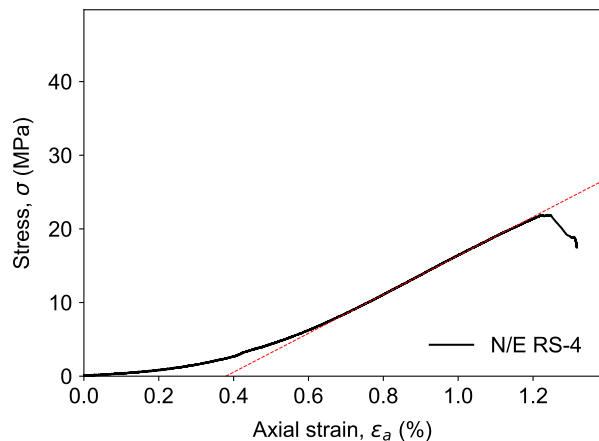
Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1786302														
Sample	DR9	Depth	37.99 - 38.21														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)^a</td><td>62.02</td></tr><tr><td>Length (mm)^a</td><td>125.90</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.616</td></tr><tr><td>UCS (MPa)</td><td>45.2</td></tr><tr><td>Young's modulus E (GPa)^b</td><td>8.3</td></tr><tr><td>Lithology</td><td>Georgian Bay Formation Shale</td></tr><tr><td>Failure description^c</td><td>1</td></tr></table>		Diameter (mm) ^a	62.02	Length (mm) ^a	125.90	Bulk density ρ (g/cm ³)	2.616	UCS (MPa)	45.2	Young's modulus E (GPa) ^b	8.3	Lithology	Georgian Bay Formation Shale	Failure description ^c	1	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	62.02																
Length (mm) ^a	125.90																
Bulk density ρ (g/cm ³)	2.616																
UCS (MPa)	45.2																
Young's modulus E (GPa) ^b	8.3																
Lithology	Georgian Bay Formation Shale																
Failure description ^c	1																
<div><div><div><div><div><div>^a Additional specimen measurement/details provides in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ¹ Axial splitting failure;</div></div></div><div></div></div></div></div>																	
Remarks:																	
Performed by	BSAT	Date	2019-11-25														

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1786302														
Sample	N/E RS-6	Depth	39.35 - 39.51														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)^a</td><td>62.77</td></tr><tr><td>Length (mm)^a</td><td>125.99</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.618</td></tr><tr><td>UCS (MPa)</td><td>24.7</td></tr><tr><td>Young's modulus E (GPa)^b</td><td>5.1</td></tr><tr><td>Lithology</td><td>Georgian Bay Formation Shale and Siltstone</td></tr><tr><td>Failure description^c</td><td>1</td></tr></table>		Diameter (mm) ^a	62.77	Length (mm) ^a	125.99	Bulk density ρ (g/cm ³)	2.618	UCS (MPa)	24.7	Young's modulus E (GPa) ^b	5.1	Lithology	Georgian Bay Formation Shale and Siltstone	Failure description ^c	1	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	62.77																
Length (mm) ^a	125.99																
Bulk density ρ (g/cm ³)	2.618																
UCS (MPa)	24.7																
Young's modulus E (GPa) ^b	5.1																
Lithology	Georgian Bay Formation Shale and Siltstone																
Failure description ^c	1																
<div><div><div><div><div><div>^a Additional specimen measurement/details provides in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ¹ Axial splitting failure;</div></div></div><div></div></div></div></div>																	
Remarks:																	
Performed by	BSAT	Date	2019-11-25														

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1786302														
Sample	N/E RS-4	Depth	37.59 - 37.69														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)^a</td><td>62.68</td></tr><tr><td>Length (mm)^a</td><td>96.30</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.611</td></tr><tr><td>UCS (MPa)</td><td>21.9</td></tr><tr><td>Young's modulus E (GPa)^b</td><td>2.6</td></tr><tr><td>Lithology</td><td>Georgian Bay Formation Shale and Siltstone</td></tr><tr><td>Failure description^c</td><td>1, 2</td></tr></table>		Diameter (mm) ^a	62.68	Length (mm) ^a	96.30	Bulk density ρ (g/cm ³)	2.611	UCS (MPa)	21.9	Young's modulus E (GPa) ^b	2.6	Lithology	Georgian Bay Formation Shale and Siltstone	Failure description ^c	1, 2	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	62.68																
Length (mm) ^a	96.30																
Bulk density ρ (g/cm ³)	2.611																
UCS (MPa)	21.9																
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Lithology	Georgian Bay Formation Shale and Siltstone																
Failure description ^c	1, 2																
<div><div><div><div><div>^a Additional specimen measurement/details provides in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ¹ Axial splitting failure; ² Length:Diameter ratio less than 2;</div></div></div><div></div></div></div>																	
Remarks:																	
Performed by	BSAT	Date	2019-11-25														

September 2, 2020

Ms. Katelyn Nero
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: UCS testing
(Golder Project 1786302)

Dear Ms. Nero:

On August 21st, 2020 three (3) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder personnel. These samples were identified as being from Golder Project No. 1786302. From these samples, two (2) UCS specimens were prepared and tested.

Details regarding the steps of specimen preparation and testing along with the results and photographs of the test specimens before and after testing are presented in the accompanying laboratory report and summary spreadsheet(s).

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

Katie Nero
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Prepared by:

Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#900-390 Bay St.
Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

September 2, 2020

Project number: 1786302-2000

Abstract

This document summarizes the results of rock laboratory testing, including 2 Uniaxial Compressive Strength tests. The UCS values and Young's modulus along with photographs of specimens before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests	1
Appendices	4

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.150 mm/min (Figure 1). The preparation and testing procedure of each specimen included the following:

1. Unwrapping of the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting of the core sample to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of the specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within 0.25°).
4. Placing the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS) and tangent Young's modulus.



Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 with the following exceptions:

- The tests herein included the measurement of the UCS and elastic modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

1.2 Results

The results of UCS testing are summarized in Table 1. The corresponding stress-strain curves are presented in Figure 2. The Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength. Additional specimen details and measurements are including in the summary spreadsheet that accompanies this report.

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm ³)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
NERS9	33.01 - 33.14	2.569	7.7	0.3	Shale	1, 2, 3
L3	32.50 - 32.71	2.552	9.1	0.3	Shale	4
Average		2.561	8.4	0.3		
Standard deviation		0.008	0.7	0.0		

¹ Axial splitting failure

² Localized crushing

³ Length:Diameter ratio less than 2

⁴ Inclined shear fracture and axial splitting failure

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.

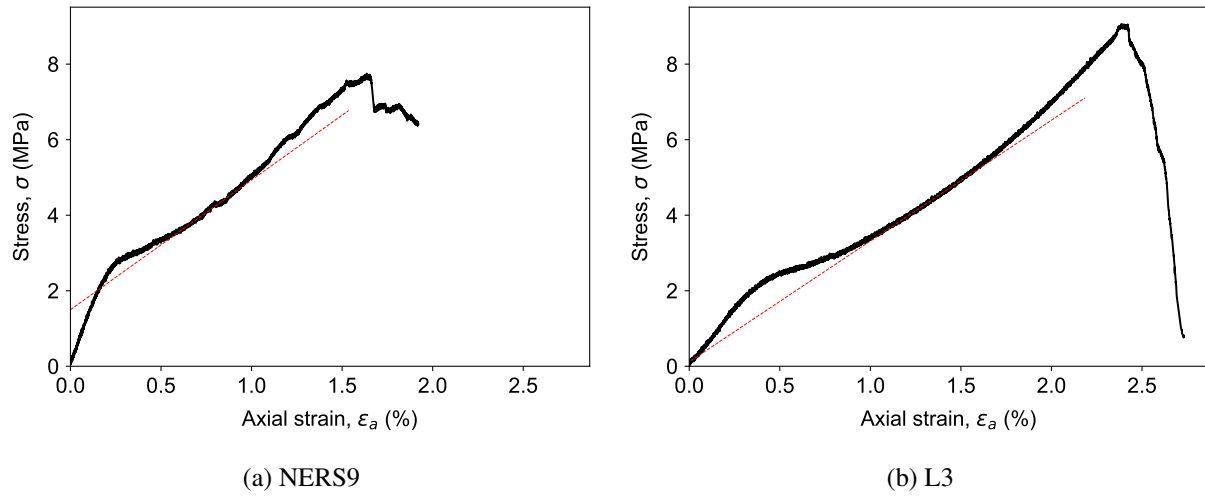




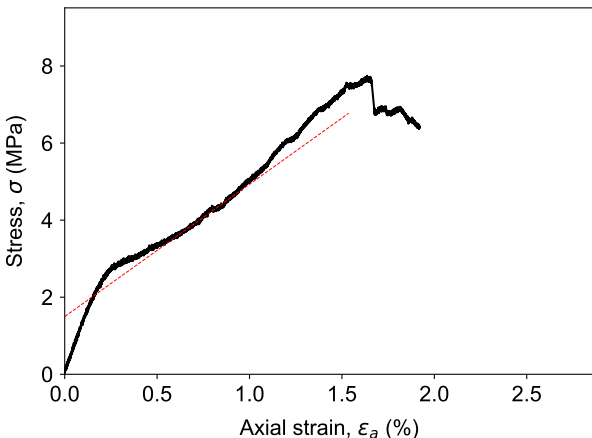
Figure 2: Measured stress-strain curves.

Appendices

Specimen sheets



- NERS9
- L3

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1786302-2000														
Sample	NERS9	Depth	33.01 - 33.14														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)^a</td><td>63.11</td></tr><tr><td>Length (mm)^a</td><td>120.13</td></tr><tr><td>Bulk density ρ (g/cm³)</td><td>2.569</td></tr><tr><td>UCS (MPa)</td><td>7.7</td></tr><tr><td>Young's modulus E (GPa)^b</td><td>0.3</td></tr><tr><td>Lithology</td><td>Shale</td></tr><tr><td>Failure description^c</td><td>1, 2, 3</td></tr></table>		Diameter (mm) ^a	63.11	Length (mm) ^a	120.13	Bulk density ρ (g/cm ³)	2.569	UCS (MPa)	7.7	Young's modulus E (GPa) ^b	0.3	Lithology	Shale	Failure description ^c	1, 2, 3	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) ^a	63.11																
Length (mm) ^a	120.13																
Bulk density ρ (g/cm ³)	2.569																
UCS (MPa)	7.7																
Young's modulus E (GPa) ^b	0.3																
Lithology	Shale																
Failure description ^c	1, 2, 3																
<div><div><div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div><div>^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.</div><div>^c Failure description: ¹ Axial splitting failure; ² Localized crushing; ³ Length:Diameter ratio less than 2;</div></div><div></div></div>																	
Remarks:																	
Performed by	BSAT	Date	2020-09-01														

Uniaxial Compression Test

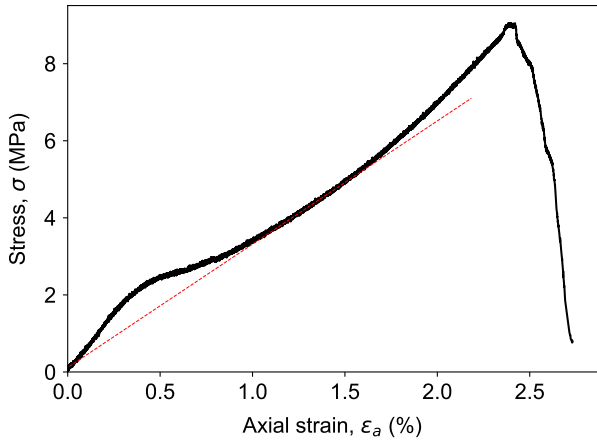
Client	Golder Associates Ltd.	Project	1786302-2000
Sample	L3	Depth	32.50 - 32.71

Specimen parameters		Prior to testing	After testing
Diameter (mm) ^a	62.80		
Length (mm) ^a	126.45		
Bulk density ρ (g/cm ³)	2.552		
UCS (MPa)	9.1		
Young's modulus E (GPa) ^b	0.3		
Lithology	Shale		
Failure description ^c	4		

^a Additional specimen measurement/details provided in accompanying summary spreadsheet.

^b Tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.

^c Failure description: ⁴ Inclined shear fracture and axial splitting failure;



Remarks:

Performed by	BSAT	Date	2020-09-01
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APPENDIX D

Non-Standard Special Provisions (NSSP)

DRILLED SHAFTS (CAISSON PILES) - Item No.
SUPPLY EQUIPMENT FOR INSTALLING DRILLED SHAFTS - Item No.
DRILLED SHAFTS – 900 mm and 1200mm DIAMETER - Item No.
SHAFT INSPECTION - Item No.
CROSS-HOLE SONIC LOGGING (CSL) TESTING - Item No.

Non-Standard Special Provision

1.0 SCOPE

This specification covers the requirements for the supply and installation of cast-in-place concrete drilled shaft (caisson pile) deep foundation units for the following structures:

- GO Transit / Metrolinx Overhead (37X-0206/B1);
- Leslie Street Overpass (37X-0208/B1); and,
- Don River East Branch Bridge (37X-0207/B1).

1.01 Specification Significance and Use

This specification is written as a provincial-oriented specification. Provincial-oriented specifications are developed to reflect the administration, testing, and payment policies, procedures, and practices of the Ontario Ministry of Transportation.

Use of this specification or any other specification shall be according to the Contract Documents.

2.0 REFERENCES

When the Contract Documents indicate that provincial-oriented specifications are to be used and there is a provincial-oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal-oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be to the OPSS listed, unless use of a municipal-oriented specification is specified in the Contract Documents.

This specification refers to the following specifications, standards, or publications:

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 904	Concrete Structures
OPSS.PROV 905	Steel Reinforcement for Concrete
OPSS.PROV 909	Prestressed Concrete - Precast members
OPSS.PROV 911	Coating Structural Steel Systems

Ontario Provincial Standard Specifications, Material

OPSS.PROV 1350	Concrete - Materials and Production
OPSS.PROV 1440	Steel Reinforcement for Concrete

CSA Standards

G40.20-04/G40.21-04 (R2009)	General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel
W47.1-03 (R2008)	Certification of Companies for Fusion Welding of Steel
W48-06	Filler Materials and Allied Materials for Shielded Metal Arc Welding
W59-03(R2008)	Welded Steel Construction (Metal Arc Welding)
W178.1-08	Certification of Welding Inspection Organizations
W178.2-08	Certification of Welding Inspectors

Canadian General Standards Board (CGSB)

48.9712-2006	Non-destructive Testing, Qualification and Certification of Personnel
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ASTM International

A 252-98(2007)	Welded and Seamless Steel Pipe Piles
A 328/A 328M-07	Steel Sheet Piling

American Petroleum Institute (API)

API 13A	Drilling Fluid Materials, 19 th Edition, 10.00.08
RP 13B-1	Standard Procedure for Field Testing Water Based Drilling Fluids, 5 th Edition,

Steel Structures Painting Council (SSPC)

SP10/NACE No.2-Jan. 1, 2001	Near-White Blast Cleaning
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International Organization for Standardization/International Electrotechnical Commission (ISO/IEC)

17025	General Requirements for the Competence of the Testing and Calibration Laboratories
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3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Bedrock means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin that may or may not be weathered.

Casing means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground. Casings are structurally required and can be used to stabilize an excavated hole.

Crosshole Sonic Logging (CSL) is a non-destructive testing method to measure the structural integrity of drilled shafts and other concrete piles by means of measuring energy and waveform generated by a signal emitter. The method is used to determine the structural soundness of concrete within the steel reinforcement cage, facilitated by the installation of hollow tubes bundled to the interior of the rebar cage.

Deep Foundation Unit means a structural member, driven or otherwise, installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

Drilled Shaft or Caisson Pile means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

Drilled Shaft or Caisson Pile Cap means a footing or some other structural component used to transfer the load to the caisson piles as well as maintaining them in position.

Liner means open ended enclosing steel tubing or pipe temporarily installed to facilitate the construction of drilled shafts or caisson piles.

Obstruction means a material and/or objects that cannot be removed from a shaft during normal excavation operations with the drilling equipment adequate to excavate earth materials found on the project, and which necessitate the use of other method and/or equipment to remove. Such obstructions may be rock fragments, boulders, waterlogged timbers, or any material, natural or man-made which requires use of special tools or procedures not otherwise required for excavation of rock or earth materials on the project.

Pile Integrity Test (PIT) or Low Strain Impact Integrity Test is a non-destructive testing method to measure the structural integrity of drilled shafts and other concrete piles by means of transient dynamic response. It is a simple and rapid test method to determine the uniformity of concrete within the drilled shaft but is less accurate than other types of testing for drilled shafts.

Pumped Concrete means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

Slurry means a drilling fluid, consisting of water or water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

Tremie means a hopper with a vertical pipe used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete in the pipe is always above water level.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions are described elsewhere in the contract.

1. The installation method and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the native soils and/or drilling through hard interbeds in bedrock as per Contract Documents.
2. Drilled shafts excavation will extend through water-bearing non-cohesive sand deposits, soft to hard cohesive soils, till materials containing cobbles and boulders, residual soil and highly weathered bedrock prior to encountering less weathered bedrock. The bedrock consists of shale with medium strong to strong limestone layers. Equipment supplied to advance the drilled shafts must be able to penetrate these materials to advance each drilled shaft into bedrock and form the required socket. Details of the bedrock are provided elsewhere in the Contract.

3. Drilled shafts will extend through soft to firm cohesive soils. Equipment supplied must be able to support the excavation walls in the cohesive overburden soils.
4. Drilled shafts will extend through non-cohesive overburden soils below the groundwater level. The selected installation methods and equipment must be able to support the excavation walls in the non-cohesive overburden soils and highly weathered portion of the bedrock and prevent materials from falling into the socket.

5.0 DESIGN AND SUBMISSION REQUIREMENTS

5.01 Design Requirements

5.01.01 Concrete

The Contractor is responsible for providing concrete with suitable characteristics for installation. The concrete shall be flow able, non-segregating concrete that does not exhibit rapid slump loss. The concrete mix shall satisfy the requirements specified herein.

5.02 Submission Requirements

5.02.01 General

All submissions shall bear the seal and signature of an Engineer experienced in the field of deep foundations. All submissions shall be submitted to the Contract Administrator as specified in the Contract Documents. In lieu of any specified timeline in the Contract Documents, all submissions shall be submitted 30 days prior to construction.

When welded field splices are used, welding procedures according to the Canadian Welding Bureau shall be submitted to the Contract Administrator.

5.02.01.01 Casing

If the use of casing is applicable to the project, the Contractor is responsible for providing casing of sufficient size and strength to facilitate the excavation whilst maintaining sidewall stability.

5.02.02 Preconstruction Survey

If required by the Contract Documents, a condition survey of property and structures that may be affected by the work shall be submitted to the Contract Administrator prior to commencing the work. The survey shall be conducted in accordance with the Contract Documents as specified and include the locations and conditions of adjacent properties; buildings; underground structures; above ground and underground utilities; and structures, such as walls abutting the site and along rail corridor.

5.02.03 Materials

5.02.03.01 Mill Certificates

One copy of the mill certificates, indicating that the steel meets the requirements for the appropriate standards for casings shall be submitted to the Contract Administrator at the time of delivery.

Where mill test certificates originate from a mill outside Canada or the United States of America, the information on the mill certificates shall be verified by testing by a Canadian laboratory. The laboratory shall be certified by an organization accredited by the Standards Council of Canada to comply with the requirements of ISO/IEC 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date (i.e., yyyy-mm-dd), and the signature of an authorized officer of the Canadian testing laboratory.

5.02.03.02 Concrete

Submissions of concrete mix shall follow OPSS.PROV 1350 requirement.

5.02.04 Installation

5.02.04.01 Drilled Shaft Pre-Construction

The drilled shaft pre-construction submittal shall be comprised of the following four components:

- a) construction experience
- b) shaft installation work plan
- c) shaft slurry technical assistance (if applicable) and
- d) non-destructive QC testing personnel.

5.02.04.01.01 Construction Experience

The Contractor's experience and qualifications in the construction of drilled shaft shall include at least three separate drilled shaft projects with:

- Ground conditions similar to those as specified in the Contract Documents.
- Drilled shaft diameters and depths similar or larger to those as specified in the Contract Documents.

The on-site drilled shaft supervisors shall have a minimum 10 years experience in supervising construction of drilled shafts of similar size (diameter and depth), scope and subsurface conditions to those as specified in the Contract Documents. Work experience shall be direct supervisory responsibility for the on-site drilled shaft construction operations. Project management level positions indirectly supervising on-site shaft construction operations are not acceptable for this experience requirement.

The drill rig operators shall have a minimum of five years experience in construction of drilled shaft foundations.

A Request to Proceed with the work of drilled shaft pre-construction shall be submitted to the Contract Administrator with:

- A project reference list for the Contractor's experience and qualifications, and
- Individual's experience lists for the on-site supervisors and drill rig operators assigned to the work.

The project reference list shall contain a description of each listed project with the name and current phone number of the projects' owner(s) or the owner's Contractor(s).

The individual's experience lists shall be limited to a single page for each supervisor or operator and contain a description of the on-site experience in drilled shaft excavation operations and placement of assembled steel reinforcing bar cages and concrete in shafts.

The drilled shaft installation shall not proceed until a Notice of Proceed has been received from the Contract Administrator.

5.02.04.01.02 Drilled Shaft Installation Work Plan

The Contractor shall submit a drilled shaft installation Work Plan to the Contract Administrator at least 4 weeks prior to the start of drilled shaft installation. In preparing the Work Plan, the Contractor shall reference the available subsurface information presented elsewhere in the contract. This Work Plan shall provide at least the following information:

- a) Proposed overall construction operation schedule and sequence.
- b) Means of access to the drilling site and details of concrete delivery to site. Description, size, and capacities of proposed equipment, including but not limited to, cranes, drills, auger, coring equipment to get through obstructions or hard rock, bailing buckets, final cleaning equipment, and drilling unit. The Work Plan shall describe why the equipment was selected and describe equipment suitability to the anticipated site conditions and work methods. The Work Plan shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar soil/rock conditions. The Work Plan shall also include details of shaft excavation and cleanout methods.
- c) Details of the method(s) to be used to ensure shaft stability (i.e., prevention of caving, bottom heave, using temporary casing, slurry, or other means) during excavation (including pauses and stoppages during excavation) and concrete placement, placement of temporary and permanent casings and removal of temporary casings. If casings are required, casing dimensions and detailed procedures for installation shall be provided.
- d) Details of casings to be used, including calculations showing that the casing can withstand stresses due to installation without undue deformation. Details shall include methods for casing handling, splicing, straightening and out-of-round correction.
- e) A slurry mix design, including all additives and their specific purpose in the slurry mix, with a discussion of its suitability to the anticipated subsurface conditions, shall be submitted and include the procedures for mixing, using, and maintaining the slurry.
- f) A detailed plan for quality control of the selected slurry, including tests to be performed, test methods to be used, and minimum and/or maximum property requirements which must be met to ensure the slurry functions as intended, considering the anticipated subsurface conditions and shaft construction methods, in accordance with the slurry manufacturer's recommendations and this project special provision shall be included. As a minimum, the slurry quality control plan shall include the tests specified in Sections 6.07.01, 6.07.02 and 6.07.03.
- g) Description of an emergency construction joint method.
- h) Methods for dewatering of the site as necessary.
- i) Description of the method used to fill or eliminate all voids below the top of shaft between the plan shaft diameter and excavated shaft diameter when permanent casing is specified.
- j) The proposed concrete mix to be used.
- k) Details of concrete placement, including proposed operational procedures for pumping methods,

and a sample uniform yield form to be used by the Contractor for plotting the approximate volume of concrete placed versus the depth of shaft for all shaft concrete placement (except concrete placement in the dry).

- l) Methods to prevent and handle delays in concrete batching and delivery to site.
- m) When shafts are constructed in water, the submittal shall include seal thickness calculations, seal placement procedure, and descriptions of provisions for casing, shoring, and dewatering.
- n) Description and details of the containment, storage and disposal plan for excavated material and drilling slurry (if applicable).
- o) A contingency plan for containment and clean-up of any spill or discharge of material which might contaminate public waters. The plan shall address the plan for regular day-to-day operations and for unplanned emergency situations.
- p) Reinforcing steel shop drawings with details of reinforcement placement, including bracing, centering, and lifting methods, and the method to ensure the reinforcing cage position is maintained during construction, including use of bar boots and/or rebar cage base plates, and including placement of rock backfill below the bottom of shaft elevation.
- q) Contingency plan to remedy sinking of the reinforcing cage into concrete.

The reinforcing steel shop drawings and shaft installation plan shall include, at a minimum:

- a) Procedure and sequence of steel reinforcing bar cage assembly.
- b) The tie pattern, tie types, and tie wire gages for all ties on permanent reinforcing and temporary bracing.
- c) Number and location of primary handling steel reinforcing bars used during lifting operations.
- d) Type and location of all steel reinforcing bar splices.
- e) Details and orientation of all internal cross-bracing, including a description of connections to the steel reinforcing bar cage.
- f) Description of how temporary bracing is to be removed.
- g) Location of support points during transportation.
- h) Cage weight and location of the center of gravity.
- i) Number and location of pick points used for lifting for installation and for transport (if assembled off-site).
- j) Crane charts and a description and/or catalog cuts for all spreaders, blocks, sheaves, and chockers used to equalize or control lifting loads.
- k) The sequence and minimum inclination angle at which intermediate belly rigging lines (if used) are released.
- l) Pick point loads at 0, 45, 60, and 90 degrees and at all intermediate stages of inclination where rigging lines are engaged or slackened.
- m) Methods and temporary supports required for cage splicing.
- n) For picks involving multiple cranes, the relative locations of the boom tips at various stages of lifting, along with corresponding net horizontal forces imposed on each crane.
- o) A description of spacers and supports to be used for the reinforcement.

The Contract Administrator will evaluate the shaft installation Work Plan for conformance with the Drawings, Specifications, and project special provisions, within the review time specified. If deemed necessary by the Contract Administrator, a Shaft Installation Work Plan Submittal Meeting will be scheduled by the Contract Administrator.

5.02.04.01.03**Slurry Methodology**

If slurry other than water slurry is used to construct the shafts, the Contractor shall provide or arrange for technical assistance in the use of the slurry. The Contractor shall submit the following to the Contract Administrator:

- a) The name and current phone number of the slurry manufacturer's technical representative assigned to the project, and the frequency of scheduled visits to the project site by the slurry manufacturer's representative.
- b) The name(s) of the Contractor's personnel assigned to the project and trained by the slurry manufacturer in the proper use of the slurry. The submittal shall include a signed training letter from the slurry manufacturer for each trained Contractor's employee listed, including the date of the training.

The following shall be submitted:

- a) The type, source, and physical and chemical properties of the bentonite (mineral) or polymer (synthetic) slurry.
- b) The source of water.
- c) Method of mixing slurry.
- d) The water solids ratio and the mass and volumes of the constituent parts, including any chemical admixtures or physical treatment employed to produce slurry with the required physical properties.
- e) Details of procedure to be used for monitoring the quality of the slurry.
- f) A test report showing the properties of the slurry and certifying that the slurry meets the requirements of API RP 13B-1.
- g) Method of disposal of the slurry.

5.02.04.01.04**Cage Lift**

The Contractor is responsible for providing proper lift procedure for rebar cage. Contractor shall submit a proposed procedure the Contract Administrator at least 4 weeks prior to the start of drilled shaft installation.

5.03**Drilled Shaft Pre-Construction Meeting**

A shaft preconstruction meeting shall be held at least 14 working days prior to the Contractor beginning any shaft construction work at the site to discuss construction procedures, personnel, and equipment to be used, and other elements of the approved shaft installation narrative. As a minimum the following shall represent the Contractor at the meeting:

- a) Project Manager
- b) Project Engineer
- c) Project Superintendent
- d) On site supervisors, and all foremen in charge of excavating the shaft, placing the casing and slurry as applicable, placing the steel reinforcing bars and placing the concrete.
- e) If slurry is used to construct the shafts, the slurry manufacturer's representative or approved Contractor's employees trained in the use of the slurry shall also attend.

5.04 Acceptance of Submissions

The Contract Administrator will review the Submissions for the purpose of verifying compliance to contract requirements, within 7 calendar days after the Pre-Construction Meeting and provide written comments if changes are necessary to meet Contract requirements. The Contractor shall submit to the Contract Administrator a final installation plan which meets all Contract requirements within 7 calendar days.

If revisions in the previously reviewed Work Plans are required to accommodate site conditions, or for other reasons, the Contractor shall submit the revised Work Plans to the Contract Administrator prior to implementation. The proposed final shaft installation work plan shall be submitted to the Contract Administrator with a Request to Proceed. The Contractor shall not proceed with the shaft installation work plan until a Notice to Proceed is given by the Contract Administrator.

The Contract Administrator's approval of the installation plan does not relieve the Contractor of full responsibility for the safe and successful completion of construction of the drilled shafts.

6.0 MATERIALS

6.01 Casing or Liner for Drilled Shafts

6.01.01 General

Casings shall be according to ASTM A36, ASTM A 252, Grade 2 or 3, ASTM A572, or ASTM A588.

Casings shall be continuous wherever possible or practical. Casings shall be installed as per the Contract requirements. Casing shall be installed to stabilize the shaft excavation against collapse.

If welded, casing shall be welded by the electric arc method according to CSA W59.

The casing wall thickness specified is the minimum that shall be supplied.

Steel casings and liners shall conform to a straightness tolerance of 1.5 mm maximum per meter of length.

The casings must be of ample strength to withstand handling stresses, driving (installation) stresses, internal pressure of fluid concrete, external pressure of surrounding earth and water, and be watertight.

Where drilled shafts are located in open water areas, casings shall be extended with due consideration of risk from fluctuating water levels and flood events to the specified bottom of casing elevation to protect the shaft concrete from water action during placement and curing of concrete unless otherwise specified in the contract documents.

6.01.02 Permanent Casing

For permanent casing, the outside surface of the casing shall be smooth to not over cut soil during casing advancement (i.e., driving shoe should not be installed on the outside).

Casings shall be non-corrugated, smooth, clean, and watertight and free of hardened concrete. Casings shall be protected from corrosion during construction.

Inspection of welds will be of a visual nature on 30% of the welds. 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.

If evidence indicating poor welding is found, radiographic or ultrasonic testing shall be carried out by the Contract Administrator using procedures according to CSA W59 on 10% of the welds.

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

6.01.03 Temporary Casing or Liner

Temporary casing or liner is defined as casing installed to facilitate shaft construction only, which is not designed as part of the shaft structure, and which shall be completely removed after shaft construction is complete unless otherwise shown on the Contract Drawings. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft and without disturbing the surrounding soil.

6.02 Steel Reinforcement

Steel reinforcement shall be according to OPSS.PROV 1440 unless otherwise specified in the project specifications or drawings.

6.03 Concrete

6.03.01 General

Concrete shall be according to OPSS.PROV 1350 and CSA A23.1-19. Concrete shall also comply with the additional requirements specified in Tables 1.1.1 and 1.1.2 below:

Table 1.1.1
Concrete for Tremie Placement Method (Wet Excavation)

Property	Test	Test Procedure	Specified Value
Workability	Slump	CSA A23.1-19C	190+/-40 mm Stability of concrete shall be assessed, as mixes with such high slump would be prone to segregation and bleeding
Workability Retention	-	-	Minimum of slump flow of 350 mm at the end of concrete placement (including removal of temporary casing, if necessity)
Maximum Coarse Aggregate Size	-	-	19 mm or not more than one quarter of the reinforcement clear spacing, whichever is smaller
Maximum Water/Cement Ratio	-	-	0.40

Table 1.1.2.
Concrete for Free Fall Placement Method (Dry Excavation)

Property	Test	Test Procedure	Specified Value
Workability	Slump	CSA A23.2-C	150 to 190 mm
Workability Retention	-	-	Minimum slump of 130 mm at the end of temporary casing removal (if applicable)
Maximum Water/Cement Ratio	-	-	0.45

6.03.02 Concrete Making Materials:

Concrete making materials shall be according to Section 1350.05 of OPSS.PROV 1350, CSA A3000 and CSA A23.1-19.

6.04 Reinforcing bar Spacers and Support Devices

Rebar spacers, centralizers and other support devices shall be according to OPSS.PROV 905.

6.05 Crosshole Sonic Logging (CSL) Access Tubes and Caps

Crosshole Sonic Logging (CSL) access tubes shall be round steel pipe with a minimum inside diameter of 38 mm (the inside diameter should be enough to allow the easy passage of the ultrasonic probes over the entire length of the access tube). The access tube shall be watertight with clean internal and external faces to ensure good bond between the concrete and the access tube. PVC access tubes are not allowed, unless approved by the Contract Administrator.

The access tubes shall be fitted with watertight threaded steel or PVC caps on the bottom and top. The access tubes shall be filled with water prior to the start of concrete placement.

6.06 Grout for filling CSL Access Tubes

Grout for filling CSL Access Tubes at the completion of the cross sonic logging shall be a homogeneous mixture of neat cement and potable water with the maximum water/cement ratio of 0.45. The grout mix design shall be approved by the Contract Administrator.

6.07 Slurry

Bentonite (mineral) slurry shall be according to API Spec 13A.

Polymer (synthetic) slurry shall be according to Guide to Support Fluids for Deep Foundations, First Edition EFFF and DFIEFFC/DFI Support Fluids Task Group.

The slurry shall consist of a stable colloidal suspension of pulverized solids or polymers thoroughly mixed with water.

Drilling slurry will be defined as water, bentonite, polymer slurry formed during the drilling process, or other fluids used to maintain stability of the drilled shaft excavation to aid in the drilling process or to maintain the quality of the shaft excavation. In addition, the term polymer slurry will be defined as the final mixed composite of all additives, including polymer slurry additives required to produce the acceptable drilling slurry.

Bentonite drilling or other mineral slurry shall not be used in shaft excavation at the Metrolinx Overhead. Bentonite drilling slurry shall not be used in shaft excavations at the Leslie Street Overpass and Don River bridge, unless approved by the Contract Administrator.

A slurry manufacturer representative shall be onsite for the first application of slurry and can be onsite as requested by the Contractor on subsequent applications. Drilling slurry, when used, will be non-compensable and effect on time of performance due to the use of the slurry will be non-excusable.

The material used to make the slurry shall not be detrimental to the concrete or surrounding ground strata. Polymer slurries shall have appropriate viscosity and gel characteristics to transport excavated material to suitable screening systems or settling tanks. The percentage and specific gravity of the material used to make the slurry shall be sufficient to maintain the stability of the excavation and to allow proper concrete placement. The entire fluid column shall be replaced with fresh slurry after drilling and during final clean-out with an airlift or other approved method; a clean-out bucket is not sufficient for final cleanout.

Prior to introduction into the shaft excavation, the manufactured polymer slurry admixture shall be pre-mixed thoroughly with clean, fresh water and for adequate time in accordance with the slurry admixture manufacturer's recommendations allotted for hydration. Water used for mixing shall be potable. Slurry tanks of adequate capacity will be required for slurry mixing, circulation, storage and treatment. No excavated slurry pits will be allowed in lieu of slurry tanks. Adequate equipment will be required as necessary to control slurry properties during the drilled shaft excavation in accordance with the values provided in the table below.

6.07.01 Water Slurry

Water without site soils or soil additive can be used as slurry when casing is used for the entire length of hole. Clean water may be used as a drilling fluid when entire length of the shaft excavation is cased. Water slurry shall conform to the following requirements:

Property	Test Procedure	Specified Value
Density	Mud Weight (Density) API 13B-1, Section 1	1040 (kg/m ³) Maximum
Sand Content	Sand API 13B-1, Section 5	1.0 (%) Maximum
Temperature (prior to concrete placement)	-	5.0 (°C) Minimum

6.07.02 Polymer (Synthetic) Slurry

Polymer slurry shall be used as per manufacturers recommendations and shall conform to the following requirements:

Property	Test Procedure	Specified Value
Density	Mud Weight (Density) API 13B-1, Section 1	1040 (kg/m ³) Maximum
Sand Content	Sand API 13B-1, Section 5	1.0 (%) Maximum
Temperature (prior to concrete placement)	-	5.0 (°C) Minimum
Viscosity	Marsh Funnel and Cup API 13b-1, Section 2.2	32 to 135
pH	Glass Electrode, pH Meter, or pH Paper	8 to 10

6.07.03 Bentonite (Mineral) Slurry

The use of bentonite slurry is not permitted for shaft excavation at the Metrolinx Overhead. Use of bentonite slurry for the shaft excavation for the Leslie Street Overpass and Don River bridge is not permitted unless approved by the Contract Administrator. If approved for use, bentonite slurry shall conform to the following requirements:

Property	Test Procedure	Specified Value
Density	Mud Weight (Density) API 13B-1, Section 1	1010 (kg/m ³) to 1200 (kg/m ³)
Sand Content (prior to final cleaning and immediately prior to placing concrete)	Sand API 13B-1, Section 5	4.0 (%) Maximum

Temperature (prior to concrete placement)	-	5.0 (°C) Minimum
Viscosity	Marsh Funnel and Cup API 13b-1, Section 2.2	26 to 50
pH	Glass Electrode, pH Meter, or pH Paper	8 to 11

7.0 EQUIPMENT

7.01 Drilling and Excavation Equipment

Drilling equipment used to perform the drilled shaft work shall have the capability of providing sufficient torque and down-thrust for drilling and excavating shafts. Appropriate drilling and coring equipment must be available to drill through obstructions and bedrock and harder interbeds in bedrock.

The excavation equipment shall be capable of excavating the drilled shaft to the dimensions required in the plan with a level bottom. The cutting edges of the excavation tools used to form the base of the drilled shaft must be normal to the vertical axis of the equipment within a tolerance of (± 13 mm) per (305 mm) of shaft diameter.

7.02 Concrete Placement Equipment

Tremie pipe to place concrete underwater shall be completely watertight and of sufficient length, weight, and diameter to discharge concrete at the shaft base elevation. The tremie must not contain aluminum parts that will have contact with the concrete. The tremie inside diameter must not be less than 250 mm for an open system or 125 mm for a closed system. The inside and outside surfaces of the tremie must be clean and smooth to permit both flow of concrete and unimpeded withdrawal during concrete placement. The discharge end of the tremie must be constructed to permit the free radial flow of concrete. Wall thickness of the tremie must be adequate to prevent crimping or sharp bends that may restrict concrete placement.

A plug shall be placed at the top of the tremie or pump line to separate the concrete from the water/slurry until the concrete is flowing through the orifice. Plugs, if left in the shaft concrete, must be of a material approved by the Contract Administrator. Tremie pipe sections must have connections that will not loosen and separate and remain watertight if a portion of the tremie becomes stuck.

8.0 CONSTRUCTION

8.01 Transporting, Storing, and Handling Piles, Casings, Liners, and Reinforcing Steel Reinforcement Cages

8.01.01 General

Casings, liners, and steel reinforcement shall be transported, stored, and handled in such a manner that damage is prevented and the strength of the components is not affected by deterioration or deformation.

Components shall be lifted and placed using appropriate lifting equipment, temporary bracing, guys, or stiffening devices so that the components are at no time overloaded, unstable, or unsafe.

Material shall be supported to prevent unequal settlement when stacked.

8.01.02 Drilled Shaft Casings and Liners

Casings and liners shall be handled and stored in such a manner to avoid damage or distortion to them. The casings and liners shall be maintained circular within $\pm 2\%$ of the casing or liner diameter.

8.02 Shaft Excavation

8.02.01 General

The Contractor shall submit Requests to Proceed prior to construction and at the milestones specified. Construction of drilled shafts shall commence only after Notices to Proceed have been given by the Contract Administrator.

Shafts shall be excavated to the required depth as shown on the Contract Drawings. Shaft excavation operations shall conform to this section and the shaft installation Work Plan.

8.02.02 Continuity of Shaft Excavation Operations

Once the excavation operation has been started, the excavation shall be conducted in a continuous operation until the excavation of the shaft is completed, except for pauses and stops as noted, using approved equipment capable of excavating through the type of material expected. Pauses during the excavation operation, except for casing splicing, tooling changes, slurry maintenance, and removal of obstructions, are not allowed.

Pauses, defined as momentary interruptions of the excavation operation, will be allowed only for casing splicing, tooling changes, slurry maintenance, and removal of obstructions. Shaft excavation operation interruptions not conforming to this definition shall be considered stops. Stops for uncased excavations (including partially cased excavations) shall not exceed 16 hours duration. Stops for fully cased excavations, excavations in rock, and excavations with casing seated into rock, shall not exceed 48 hours duration unless approved by the Contract Administrator.

For stops exceeding the time durations specified above, the Contractor shall stabilize the excavation using the following method:

For both a cased and uncased excavation, backfill the hole with either Lean Concrete or granular material. The Contractor shall backfill the hole to the ground surface, if the excavation is not cased, or to a minimum of 1.5 m above the bottom of casing (temporary or permanent), if the excavation is cased. Backfilling of shafts with casing fully seated into rock, as determined by the Contract Administrator, will not be required.

During stops, the Contractor shall protect the base of the shaft from weathering and stabilize the shaft excavation to prevent bottom heave, caving, head loss, and loss of ground. The Contractor bears full responsibility for selection and execution of the method(s) of stabilizing and maintaining the shaft excavation. Shaft stabilization shall conform to the shaft installation Work Plan.

If slurry is present in the shaft excavation, the Contractor shall conform to the requirements of OPSS.PROV 903.07.05. regarding the maintenance of the slurry and the minimum level of drilling slurry

throughout the stoppage of the shaft excavation operation and shall recondition the slurry to the required slurry properties prior to recommencing shaft excavation operations.

Once the excavation of the rock socket reaches the target depth, over-ream the shaft side walls, prior to placement of the reinforcing cage. The duration from the time of base inspection to the start of concreting shall not exceed 6 hours.

Rock socket side walls shall be roughened if specified on the Contract Drawings.

8.02.03 General Shaft Casing or Liner Requirements

8.02.03.01 General

Shaft casing or liner shall be watertight and clean prior to placement in the excavation. The outside diameter of the casing shall not be less than the specified diameter of the shaft. The diameter of the casing shall not be greater than the specified diameter of the shaft plus 150 mm.

The Contractor shall conduct casing installation and removal operations and shaft excavation operations such that the adjacent soil outside the casing and shaft excavation for the full height of the shaft is not disturbed. Disturbed soil is defined as soil whose geotechnical properties have been changed from those of the original in situ soil, and whose altered condition adversely affects the capacity and structural integrity of the shaft foundation.

8.02.03.02 Permanent Shaft Casing

Permanent casing is defined as casing designed as part of the shaft structure and installed to remain in place after construction is complete. All permanent casing shall be of ample strength to resist damage and deformation from transportation and handling, installation stresses, and all pressures and forces acting on the casing. Where the minimum thickness of permanent casing is specified in the Contract Drawings, it is specified to satisfy structural design requirements only. The Contractor shall increase the casing thickness as necessary to satisfy the requirements of this section.

The outside surface of the casing should be smooth, so it does not overcut soil during advancement (creating a void behind casing). Should the void between casing and a wall of shaft excavation occur, the void shall be filled with a material which approximates the geotechnical properties of the in-situ soils, in accordance with the shaft installation work plan.

The cutting tools and driving shoes of permanent casing shall not overcut the ground and the cutting tools and driving shoes shall be flush with the outside diameter of the casing.

8.02.03.03 Temporary Shaft Casing or Liner

Temporary casing or liner is defined as casing installed to facilitate shaft construction only, which is not designed as part of the shaft structure, and which shall be completely removed after shaft construction is complete unless otherwise shown on the Contract Drawings. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft and without disturbing the surrounding soil.

To maintain stable excavations and to facilitate construction, the Contractor may furnish and install temporary casing in addition to the required casing specified on the Contract Drawings. The Contractor

shall provide temporary casing at the site in sufficient quantities to meet the needs of the anticipated construction method.

The Contractor shall use the temporary casing method at all sites where it is inappropriate to use the dry or wet construction methods without the use of temporary casings other than surface casings. In this method, the casing is advanced prior to excavation and withdrawn after concrete placement. In the event seepage conditions prevent use of the dry method, the excavation and concrete placement shall be carried out using wet methods. Wet non-plastic soil shall not be considered as impervious, regardless of permeability.

Where drilling through materials that are susceptible to sloughing, the Contractor shall use appropriate means and method to prevent sidewall and basal instability including but not limited to or a combination of slurry and temporary casing. The Contractor shall take the necessary steps as required to prevent caving during shaft excavation. Should the Contractor select to remove a casing and replace it with a longer casing through caving soils, the excavation shall be backfilled. The Contractor may use soil previously excavated or soil from the site to backfill the excavation. Contractor may use other acceptable methods which will control the size of the excavation and protect the integrity of the foundation soils to excavate through caving layers.

Temporary casing must not be withdrawn until the head of concrete inside the casing is at a sufficient level that the concrete pressure at the bottom of casing exceeds the fluid pressure (e.g., groundwater pressure) on the outside of the casing at all times.

When conditions warrant, the Contractor may pull the casing in partial stages. Before withdrawing the casing, ensure that the level of fresh concrete is at such a level that the fluid trapped behind the casing is displaced upward. As the casing is withdrawn, maintain the level of concrete within the casing so that fluid trapped behind the casing is displaced upward out of the shaft excavation without mixing with or displacing the shaft concrete.

All temporary casing shall be removed. The Contractor shall ensure that permanent casings installed below the shaft cut-off elevation remains in position as a permanent part of the drilled shaft. When casings that are to be removed become bound in the shaft excavation and cannot be practically removed, a proposal shall be submitted to the Contract Administrator for review and acceptance.

If temporary casing is advanced deeper than the minimum top of rock socket elevation shown on the Contract Drawings or actual top of rock elevation if deeper, the Contractor shall withdraw the casing from the rock socket and overream the shaft. If the temporary casing cannot be withdrawn from the rock socket before final cleaning, the rock socket shall be extended below the design tip to maintain a full socket depth. When the shaft extends above ground or through a body of water, the Contractor may form the exposed portion with removable casing except when the Permanent Casing Method is specified. For permanent casings, the Contractor shall remove the portion of metal casings in accordance with the Contract Drawings. The Contractor shall dismantle casings removed to expose the concrete as required above in a manner which will not damage the drilled shaft concrete.

Temporary casing shall be removed gradually as concrete is placed in the shaft. The proposed method of extraction shall be submitted to the Contract Administrator with a Request to Proceed. The Contractor shall not proceed with the extraction until a Notice to Proceed is given by the Contract Administrator.

Contract Administrator may permit movement of the casing by rotating, oscillating or extraction with a vibratory hammer. The extraction method should be coordinated with the Contract Administrator. The Contractor shall extract casing at a slow, uniform rate while the concrete remains fluid.

Expandable or split casings that are removable are not permitted for use below water.

8.02.03.04 Temporary Telescopic Casing

If permitted by the Contract Administrator, the Contractor shall submit a temporary telescoping casing proposal for drilled shafts with a Request to Proceed to the Contract Administrator, subject to the following conditions:

- a) A maximum of two telescoping casing diameter changes will be allowed.
- b) The maximum diameter change at each casing diameter transition shall be 300 mm.

The Contractor shall not proceed until a Notice to Proceed is given by the Contract Administrator.

8.02.04 Cleaning of Bottom of Shaft Excavation and Inspection

8.02.04.01 Cleaning

The Contractor is responsible for cleaning the base of the drilled shafts to comply with the requirements of the specification. Shaft and base cleanliness will be verified by the Contract Administrator.

The Contractor shall use appropriate means such as a cleanout bucket (bailing bucket) and air lift or other devices to clean the bottom of the excavation of all shafts to achieve direct contact between the concrete and undisturbed end bearing formation. The entire slurry column shall be exchanged during final clean-out for wet excavations. A clean-out bucket alone is not sufficient for final clean-out for wet excavations.

The following cleaning criteria must be followed for thickness of sediments at the time of concrete placement:

- a) End Bearing Drilled shafts in Soil: The average thickness of the sediments shall be less than 13 mm. At least 50 percent of the base of each shaft shall have less than 13 mm of sediment. The maximum thickness of sediment at any place on the base of the shaft shall not exceed 25 mm.
- b) End Bearing Drilled shafts in rock: The average thickness of the sediments shall be less than 8 mm. At least 50 percent of the base of each shaft shall have less than 8 mm of sediment. The maximum thickness of sediment at any place on the base of the shaft shall not exceed 15 mm.
- c) Friction shaft without any end bearing: The maximum thickness of sediment at any place on the base of the shaft shall not exceed 50 mm.

8.02.04.02 Inspection

Each excavated shaft shall be inspected and accepted by the Contract Administrator prior to proceeding with construction. The bottom of each excavated shaft shall be inspected using both Shaft Inspection Device (SID) and Shaft Quantitative Inspection Device (SQUID) (or-an approved alternate down-hole equipment) to verify shaft bottom cleanliness and thickness of debris/sediment prior to concreting as specified in the Contract Documents.

After installation for the rebar cage and immediately before placement of the concrete, the bottom of the shaft shall be sounded with an airlift pipe, a tape with a heavy weight attached to the end of the tape, or other means acceptable by Contract Administrator to determine that the shaft bottom meets the requirements.

The Contractor shall cooperate with the Contract Administrator in using this inspection device, including placing the inspection device in position for inspection and removing it after the inspection. If any of the SID inspections indicate the cleanliness or bearing material requirements are not achieved, reinspection after additional cleaning or drilling will be required at no additional cost.

The Contractor shall submit a request to proceed before placing reinforcing cage and concreting and shall not proceed until a Notice to Proceed is received from the Contract Administrator.

After completion of the inspection of a shaft, the Contract Administrator will direct the Contractor as to whether additional clean-out is necessary.

Both SID and SQUID method of base inspection shall be used for each drilled shaft.

8.02.04.02.01 Shaft Inspection Device (SID)

The SID shall be provided and operated by the Contract Administrator. The Contractor shall cooperate with the Contract Administrator in conducting the SID.

The Contractor shall provide a means to position and lower the SID into the shaft excavation to enable the bell housing to rest vertically on the bottom of the excavation. The inspection of each drilled shaft excavation after final cleaning shall be continuously videotaped.

For Contractor's information, the Contract Administrator will furnish a SID device satisfying the following requirements:

- a) A remotely operated, high resolution, color video camera sealed inside a watertight bell housing.
- b) Provides a clear view of the bottom inspection on a video monitor at the surface in real time.
- c) Provides a permanent record of the entire inspection with voice annotation with a resolution of not less than 720 x 480.
- d) Provides a minimum field of vision of 710 cm², with at least two graduated measuring devices to record the thickness of debris/sediment on the bottom of the shaft excavation to a minimum accuracy of 12 mm and a length greater than 37 mm.
- e) Provides sufficient lighting to illuminate the entire field of vision at the bottom of the shaft for the operator and inspector to clearly see the depth measurement scale on the video monitor and to produce a clear recording of the inspection.
- f) Provides a compressed air or gas system to displace drilling fluids from the bell housing and a pressurized water system to assist in determination of bottom sedimentation depth.

For shafts with diameter of up to 2 m, the thickness of debris/sediment will be measured at least in five locations, one in the center of the shaft as well as in the four quadrants surrounding the shaft center. If the diameter of the shaft is between 2 m to 3 m, five measurement of the thickness shall be performed on the middle 2 m diameter of the shaft (similar to the shafts with 2 m diameter) and at least six thickness measurements shall be performed on the perimeter beyond the middle portion.

8.02.04.02.02

Shaft Quantitative Inspection Device (SQUID)

The SQUID shall be provided and operated by the Contract Administrator. The Contractor shall cooperate with the Contract Administrator to supply and install the Kelly bar adapter and to execute the test.

For Contractor's information, the device shall include the following components:

SQUID Unit – Unless updated by the equipment manufacturer, the SQUID Unit shall be a hexagonal shaped device with a height of approximately 630 mm, a diagonal of approximately 650 mm, and a weight of approximately 188 kg. The unit shall include three penetrometers each having a surface area of 10 cm² to measure force and three displacement plates each having a diameter of 152 mm and a weight of 7.75 kg to determine displacements. The unit shall also be supplied with two downhole data transmission cables and two transmitter boxes for signal conditioning.

Kelly Bar Adapter – Drill rig Kelly bar dimensions vary depending upon the manufacturer and require an adapter to attach to the SQUID unit. For each drilling rig on the project, the Contractor shall submit to the Contract Administrator a completed adaptor detail to the SQUID equipment supplier two weeks prior to installing the initial drilled shaft with that drill rig.

The SQUID Unit shall be pin-connected to the Kelly bar using a properly sized adapter provided by the SQUID equipment supplier or Contractor. After the pin-connection and prior to testing, the verticality of the SQUID Unit shall be checked and confirmed. The signal transmission from the SQUID Unit to the SQUID Tablet shall also be confirmed prior to commencing the test. Signal transmission shall be checked by manually lifting each displacement plate and observing the increasing displacement on the SQUID Tablet. After verticality and signal transmission checks are completed, the SQUID Unit shall be moved over the open shaft excavation and lowered without rotation until the unit is approximately 0.6 m above the shaft base.

The test shall proceed by slowly lowering the Kelly bar without rotation until the entire weight of the Kelly bar is transferred to, and is resting on, the SQUID Unit. Penetrometer force and plate displacement measurements shall be continuously acquired, displayed, and stored on the SQUID Tablet during the test process. A test run shall be terminated once two of the three penetrometers have registered a force greater than 2.2 kN or the maximum penetrometer travel of 152 mm is reached for any one of the penetrometers.

Sediment, loose material, or debris at the base of the shaft is defined as a material that has a minimum resistance to penetrometer force of 0.089 kN. Natural soils are defined as materials that have a resistance to penetrometer force greater than 0.71 kN. The thickness of sediment, loose material, or debris at the base of the drilled shaft is defined as the difference in the displacement plate measurements that occurs between a penetrometer force of 0.089 kN to 0.71 kN.

If the shaft base diameter is 0.9 m or less, a single SQUID run shall be performed at the shaft center. At least five SQUID runs shall be performed for the shafts with diameter of up to 2 m, one in the center of the shaft as well as in the four quadrants surrounding the shaft center. If the diameter of the shaft is between 2 m to 3 m, at least five SQUID runs shall be performed on the middle 2 m diameter of the shaft (similar to the shafts with 2 m diameter) and at least six SQUID runs shall be performed on the perimeter beyond the middle portion.

Following the testing at the center, the SQUID Unit shall be repositioned in one of the four perimeter quadrants (North, South, East, or West) around the shaft center and the process described above repeated. For each SQUID run, the average debris thickness determined using the force versus displacement results

from a minimum of two penetrometers shall be used to determine if the drilled shaft base condition meets the specified base cleanliness criteria or whether additional cleaning and retesting is required.

A drilled shaft base often contains irregularities from a level surface due to pilot holes or grooves from cutting teeth on drilling tools. Therefore, a SQUID run shall be considered complete provided the debris thickness can be determined from a minimum of two force versus displacement plots. Interpretation of reading for determination of the thickness of debris/sediment and reporting shall be based on the manufacture's recommended procedure.

8.02.05 Shaft Obstruction

When obstructions are encountered, the Contractor shall notify the Contract Administrator promptly. An obstruction is defined as a specific object encountered during the shaft excavation operation which prevents or hinders the advance of the shaft excavation.

An obstruction will be classified as material and/or objects that cannot be efficiently removed from a shaft during normal excavation operations with the drilling equipment adequate to excavate earth materials found on the project, and which necessitate the use of other methods and/or equipment to remove not otherwise required for excavation of rock or earth materials on the project. Such obstructions may be rock fragments or layers, boulders, waterlogged timbers, or any material, natural or man-made, which requires use of special tools or procedures.

For this project, the following are *not* classified as obstructions and, if present, must be removed by the Contractor with no additional compensation.

1. Material present which is:
 - a. required to be removed by the Contract; or
 - b. known to the Contractor or readily visible upon site investigation and which can be removed by conventional surface excavation methods.
2. Boulders that are one-fourth, or less, of the casing shaft diameter

When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being significantly reduced relative to the rate of advance for the portion of the shaft excavation in the geological unit that contains the obstruction, then the Contractor shall remove, break up, or push aside the obstruction.

Subsurface obstructions at drilled shaft locations shall be removed, broken or pushed aside by the Contractor. The Contractor shall employ special procedures or tools when the hole cannot be advanced using conventional equipment. Blasting will not be permitted. Except as provided in this section, all cost and time effects, direct, indirect and cumulative of subsurface obstruction of whatever nature, will be conclusively deemed fully compensated under the pay items in accordance with the contract. Encountering unexpected obstructions will be considered inherent risks in this work, both as to type and extent as is variability in material encountered in the work as to effort required to drill through or excavate the material. In the event the Contractor encounters at the site of a drilled shaft location a subsurface or latent physical condition that differs materially from that indicated in the contract documents, the Contractor shall strictly follow the procedure provided for a differing site condition set forth in Contract Documents. Any adjustment to the contract amount or time will only be those expressly permitted by the Contract Documents and only to the extent expressly provided in the Contract Documents. Drilling tools lost in the excavation will not be considered obstructions and shall be promptly removed by the Contractor. All work required to

remove lost tools or to perform associated corrective work, including but not limited to repair of hole degradation due to removal operations and any effect on time, will be non-compensable.

8.02.06 Use of Slurry in Shaft Excavation

The Contractor shall use slurry to maintain a stable excavation during excavation and concrete placement operations once water begins to enter the shaft excavation at an infiltration rate of 300 mm of depth or more in an hour. If concrete is to be placed in the dry, the Contractor shall pump all accumulated water in the shaft excavation down to a 75 mm maximum depth prior to beginning concrete placement operations. The concrete shall not be placed in the dry for wet non-plastic soils.

Use of specially designed polymer slurry may be permitted to stabilize uncased excavations, if approved by the Contract Administrator.

8.02.06.01 Slurry Technical Assistance

If slurry other than water is used, the slurry manufacturer's representative, shall:

- a) Provide technical assistance for the use of the slurry,
- b) Be at the site prior to introduction of the slurry into the first drilled hole requiring slurry, and,
- c) Remain at the site during the construction of at least the first shaft excavated to adjust the slurry mix to the specific site conditions.

After the manufacturer's representative is no longer present at the site, the Contractor's employee trained in the use of the slurry, as identified to the Contract Administrator shall be present at the site throughout the remainder of shaft slurry operations for this project to perform the duties specified in items a) through c) above.

8.02.06.02 Minimum Level of Slurry in the Excavation

When slurry is used in a shaft excavation the following is required:

- a) The height of the slurry shall be as required to provide and maintain a stable excavation to prevent bottom heave, caving or sloughing of all unstable zones.
- b) The slurry level in the shaft while excavating shall be maintained above the groundwater level the greater of the following dimensions:
 - i. Not less than 1.5 m for bentonite (mineral) slurries.
 - ii. Not less than 1.5 m for water slurries.
 - iii. Not less than 1.5 m for polymer (synthetic) slurries.
- c) The slurry level in the shaft throughout all stops and during concrete placement shall be no lower than the water level elevation outside the shaft.

8.02.06.03 Slurry Sampling and Testing

Bentonite slurry and polymer slurry shall be mixed and thoroughly hydrated in slurry tanks, ponds, or storage areas. Mixing in the shaft excavation is not permitted.

The Contractor shall draw sample sets from the slurry storage facility and test the samples for conformance with the specified viscosity and pH properties before beginning slurry placement in the drilled hole. A sample set shall be composed of samples taken at mid-height and within 600 mm of the bottom of the storage area. The Contractor shall keep a written record of all additives and concentrations of the additives in the polymer slurry. These records shall be submitted to the Contract Administrator once the slurry system has been established in the first drilled shaft on the project. The Contractor shall provide revised data to the Contract Administrator if changes are made to the type or concentration of additives during construction.

The date, time, names of the persons sampling and testing the slurry, and the results of the tests shall be recorded. A copy of the recorded slurry test results shall be submitted to the Contract Administrator at the completion of each shaft, and during construction of each shaft when requested by the Contract Administrator. Sample sets of all slurry, composed of samples taken at mid-height and within 600 mm of the bottom of the shaft and the storage area, shall be taken and tested once every 4 hours minimum at the beginning and during drilling shifts and prior to cleaning the bottom of the hole to verify the control of the viscosity and pH properties of the slurry. Sample sets of all slurry shall be taken and tested at least once every 2 hours if the previous sample set did not have consistent viscosity and pH properties. All slurry shall be recirculated, or agitated with the drilling equipment, when tests show that the sample sets do not have consistent viscosity and pH properties. Cleaning of the bottom of the hole shall not begin until tests show that the samples taken at mid-height and within 600 mm of the bottom of the hole have consistent viscosity and pH properties. Sample sets of all slurry, as specified, shall be taken and tested to verify control of the viscosity, pH, density, and sand content properties after final cleaning of the bottom of the hole just prior to placing concrete. Placement of the concrete shall not start until tests show that the samples taken at mid-height and within 600 mm of the bottom of the hole have consistent specified properties.

8.02.06.04 Maintenance of a Stable Excavation

The Contractor shall demonstrate to the satisfaction of the Contract Administrator that stable conditions are being maintained. If the Contract Administrator determines that stable conditions are not being maintained, the Contractor shall immediately take action to stabilize the shaft. The Contractor shall submit to the Contract Administrator a revised shaft installation plan that addresses the problem and prevents future instability. The Contractor shall not continue with the shaft construction until the damage that has already occurred is repaired in accordance with the specifications, and until receiving the Contract Administrator's review of the revised shaft installation Work Plan.

8.02.06.05 Disposal of Slurry and Drill Cuttings

Disposal of the soil/rock cutting, slurry, and slurry contacted spoils shall be in accordance with all applicable regulatory requirements.

8.03 Assembly and Placement of Reinforcing Steel

8.03.01 Reinforcing Bar Cage Assembly

The Contractor shall assemble the drilled shaft reinforcement cage and place as a unit in accordance with the installation plan. The drilled shaft reinforcement shall be placed immediately after the shaft excavation is inspected and accepted, and just prior to shaft concrete placement.

All reinforcing steel in the shaft shall be double-wire tied and supported such that the steel remains within the allowable tolerances specified herein during placement of concrete. Splices shall be located in

accordance with and as shown on the Contract Drawings. Mechanical bar splices meeting the requirements specified in the contract documents shall be used. Mechanical bar splices in adjacent bars shall be staggered not less than 3'-6" (1067 mm) apart. Welding of reinforcing steel will not be permitted.

The reinforcing cage shall be rigidly braced to retain its configuration during handling and construction. The Contractor shall show bracing and any extra reinforcing steel required for fabrication of the cage on the shop drawings. Shaft reinforcing bar cages shall be supported on a continuous surface to the extent possible. All rigging connections shall be located at primary handling bars, as identified in the reinforcing steel assembly and installation plan. Internal bracing is required at each support and lift point. When lifting the cage for placement in the shaft, the Contractor shall provide sufficient pick points to prevent bending of the cage that will cause deformation of the reinforcement bars and damage to inspection cables.

Damaged bars and inspection cables must be replaced at the Contractor's expense.

The reinforcement shall be carefully positioned and securely fastened to provide the minimum clearances listed below, and to ensure no displacement of the reinforcing steel bars occurs during placement of the concrete.

8.03.02 Reinforcing Bar Cage Centralizers and Template

Rolling centralizers for reinforcing steel shall be used to minimize disturbance of the shaft sidewalls. The reinforcing steel centralizers at each longitudinal space plane shall be placed in accordance with the following minimum criteria:

- a) A plane of centralizers shall be provided within 0.5 m of bottom of the shaft.
- b) A plane of centralizers shall be provided within 1.5 m of top of the shaft.
- c) Planes of centralizers shall be provided at a maximum longitudinal spacing of either 2.5 times the shaft diameter or 4.5 m, whichever is less.
- d) Each plane of centralizers shall consist of either one centralizer per 0.3 m diameter of the shaft or four centralizers whichever is more.

The Contractor shall furnish and install additional centralizers as required to maintain the specified concrete cover throughout the length of the shaft.

The Contractor shall provide a template at the top of each shaft to locate and align vertical shaft reinforcement bars to match that shown on the Contract Drawings.

8.03.03 Reinforcing Bar Cage Installation and Support

Reinforcing bar cage should be securely held in the position immediately before, during and after the concrete placement. The reinforcing cage bottom supports shall be positioned such that the reinforcing steel is not allowed to come into contact with the soil or rock and to ensure that the bottom of the cage is maintained at the proper distance above the base as identified in the contract documents.

The Contractor shall laterally support the reinforcement cage at the top during placement of the concrete. The support system must be concentric to prevent racking and displacement of the cage. Temporary internal cage stiffeners shall be removed as the cage is placed in the shaft such that interference with the placement of concrete does not occur.

The rebar cage can be released only when the concrete achieved sufficient strength to support the weight of the cage. For smaller diameter drilled shafts the entire weight of the cage may be supported by bar boots. Information about the type and number of bar boots along with shop drawings shall be submitted to the Contract Administrator.

The elevation of the top of the reinforcing cage shall be checked before and after the concrete is placed. The reinforcing cage shall be maintained within the specified tolerances, and the Contractor shall make corrections to those tolerances, as required, to the satisfaction of the Contract Administrator.

No additional shafts shall be constructed until the Contractor has modified the reinforcing cage support to obtain the required tolerances.

If after placement of the reinforcement the Contract Administrator determines that the condition of the shaft is unsuitable or if concrete placement does not immediately follow the reinforcing steel placement, the Contractor shall remove the cage from the shaft as directed by the Contract Administrator so that the integrity of the excavation, including accumulation of loose material in the bottom of the shaft and the condition of the sides of the shaft, can be determined by inspection. If the reinforcement cage moves up or down from its original position by more than 75 mm, the Contractor shall submit a proposal to the Contract Administrator for approval to address the out of tolerance reinforcement installation.

8.04 Concrete Placement

8.04.01 General

Concrete should be placed as soon as possible but not to exceed 6 hours after completing cleaning of the shaft excavation, inspecting and finding it satisfactory, and immediately after placing reinforcing steel.

The full-depth drilled shaft shall be open no more than 96 hours prior to receiving concrete, including all the necessary time to clean the base, exchange the slurry, inspect the base, and place the cage

The concrete shall be placed continuously at a rate to prevent cold joints within the drilled shaft. An unplanned stoppage of work may require an emergency construction joint during the shaft construction. A detailed plan for an emergency construction joint shall be included in the installation plan.

During concrete placement, the Contractor shall monitor, and minimize, the difference in the level of concrete inside and outside of the steel reinforcing bar cage.

If temporary casing is used, it is important to establish sufficient head of concrete prior to breaking the casing seal, so that the concrete pressure exceeds the fluid pressure on the outside of the casing. The concrete level should always be maintained a minimum of 2.0 m and 5.0 m above the bottom of the casing during a concrete placement for dry and wet method, respectively.

Upward and downward movement of the reinforcing cage should be monitored during the pour.

8.04.02 Concrete Placement in Dry Excavations (Free Fall Method)

If not more than 50 mm of water is present in the shaft excavation and the water inflow into the excavation is less than 0.3 m per hour (or 5 mm per minute), the concrete placement in dry excavation method can be

used. Concrete placement in dry excavation method is not permitted in non-plastic soil below groundwater levels.

The concrete shall be deposited through the center of the reinforcement cage using the free fall method. The concrete shall be placed using drop chute or any acceptable device such that the free-fall is vertical down the center of the shaft without hitting the sides, the steel reinforcing bars or the reinforcing bar cage bracing. The height of concrete free fall should be limited to 25 m. Use of a flexible hose is not permitted.

Continuously place concrete in the shaft to the target elevation. If the top of the shaft is near the ground surface, upper contaminated concrete should be removed until clean fresh concrete is revealed. Upper 1.5 m of concrete should be consolidated using vibrators (after complete removal of temporary casing, if temporary casing is used).

The theoretical volume of concrete required to fill the shaft excavation should be computed prior to the concrete placement. If the actual volume installed (based on delivery tickets) is considerably less than the theoretical volume, the Contract Administrator should be informed immediately, as immediate concrete removal (before concrete sets) and reinstallation may be necessary.

For this project, all drilled shafts shall be not concreted using free fall method.

8.04.03 Concrete Placement in Wet Excavation (Tremie Method)

When more than 50 mm of water is present in the excavation or water inflow rate exceeds 0.3 m per hour (or 5 mm per minute) or for shaft within non-plastic soils below groundwater table concrete should be placed using tremie method. Concrete used for tremie placement method should have the ability to achieve sufficient compaction by gravity when placed by a tremie pipe and should have the ability to displace the drilling fluid inside the shaft excavation without intermixing and segregation.

Drilling fluid level should be maintained constant during the concrete placement.

The tremie pipe should be pressure fed by a pump; gravity tremie pipe (open tremie pipe) should not be used, unless approved by the Contract Administrator. Tremie pipe used for concrete placement should comply with the following requirements:

- a) Should be watertight;
- b) Should have a minimum inside diameter of:
 - i. Pressurized tremie pipe – 100 mm or three times of maximum coarse aggregate size, whichever is larger;
 - ii. Gravity tremie pipe – 200 mm or eight times of maximum coarse aggregate size, whichever is larger;
- c) Should be sufficiently robust (not flexible);
- d) Should be made of steel (aluminum or PVC should not be used);
- e) Should have clean inner surface to minimize drag on the concrete flow.

The tremie pipe should be embedded into previously placed concrete at all times during the concrete placement. The tremie pipe embedment should be within the range of 3 m to 4m.

The discharge end of the tremie pipe shall be sealed using a sacrificial plate prior to lowering the tremie pipe into the excavation. Alternatively, the Contractor may use a plug that is inserted from the top end of the tremie pipe and travels through the tremie to keep the concrete separated from the slurry in the shaft excavation. The concrete should only get into contact with the slurry once it flows out of the tremie pipe.

During the start of the placement operations, ensure that the discharge end of the tremie pipe is within 150 ± 50 mm of the bottom of the shaft excavation until at least 3.0 m of concrete embedment has been established (tremie pipe should first be placed to rest on the bottom of the shaft and then raised approximately 150 mm).

Volume of concrete sufficient to fill at least 5 m of the shaft length should be available on site before the pour can start. The concrete pour shall be continuous.

A minimum of 5 m of the shaft length, should be place prior to the first spilt of the tremie.

Depths of top of the concrete, discharge end of the tremie pipe, bottom of the casing should be continuously monitored during concrete placement. These depths should be plotted against concrete volume and compared to theoretical values computed prior to concrete placement. These graphs should later be provided to the Contract Administrator.

At the completion of the concrete placement there is usually up to a meter of contaminated laitance concrete at the upper portion of the shaft, which should later be removed. Therefore, it is often advised to over-pour the shaft by approximately 1 m above the target cut-off elevation.

Slurry should be kept above the top of the concrete for at least 24 hours after the pour completion.

If tremie concrete placement operation is interrupted, the Contract Administrator may require the Contractor to prove that the quality of the final product was not affected. The methodology of the investigation shall be specified by the Contract Administrator. All costs related to such investigation shall be responsibility of the Contractor.

If at any time during the concrete pour the tremie line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete surface, the entire drilled shaft will be considered defective. In such a case, the Contractor shall either: 1) remove the reinforcing cage and concrete, complete any necessary sidewall cleaning or overreaming and repair the shaft; or 2) construct an emergency construction joint if the level of the concrete is high enough in the permanent casing to allow entry into the shaft after the concrete cures.

If Option 2 is performed, the emergency cold joint shall be properly prepared by chipping away the surface of the concrete until sound, competent concrete is exposed and accepted by the Contract Administrator. The remainder of the shaft shall then be poured in the dry by methods approved by the Contract Administrator. All costs related to such investigation shall be responsibility of the Contractor.

For this Contract, all drilled shafts shall be concreted using tremie method.

8.04.04 Protection of Fresh and Immature Concrete

No construction operations capable of producing excessive ground vibrations or ground loss (e.g. drilling operations) should be performed in the radius of 10 meters or three shaft diameters, whichever is larger, from the freshly place concrete for first 48 hours or until concrete reaches a compressive strength of 14.0 MPa, whichever happens first. Construction equipment capable of producing excessive ground vibration

includes vibratory hammers, pile drivers (hydraulic hammers and vibratory pile drivers), machine mounted impact tools, large drilling rigs, roller compactors and other large pieces of equipment.

Cold and hot weather concreting practices should be as per OPSS.PROV 904.

8.04.05 Concrete Quality Control Testing

Concrete Quality Control testing should be performed as per requirements specified in OPSS.PROV 1350.

8.05 Tolerances

During excavation of the shaft, the Contractor shall perform plumbness, alignment and dimensional checks of the shaft at 1500 mm increments. Any deviation exceeding the allowable construction tolerances specified herein shall be corrected by the Contractor.

Drilled shaft excavations constructed in such a manner that the concrete shaft cannot be completed within the required tolerances will not be accepted.

When a shaft excavation is completed with unacceptable tolerances, the Contractor shall propose, develop and, submit a plan to the Contract Administrator describing the procedure for the corrective work. The Contractor shall submit a Request to Proceed and shall not continue with the work until a Notice to Proceed is given.

When a shaft excavation is completed with unacceptable tolerances, the Contractor shall propose, develop and, submit a plan to the Contract Administrator describing the procedure for the corrective work.

The following construction tolerances will apply to drilled shafts unless stated otherwise in the contract documents:

- a) Shafts shall be constructed such that the center of the top of the shaft is within 75mm of plan position in the horizontal plane at the plan elevation for the top of the shaft.
- b) The vertical alignment of a vertical shaft excavation shall not vary from the plan alignment by more 6 mm per 305 mm of depth. The overall plumbness, including the drilled shaft and column, shall be within 75 mm of the vertical alignment of the shaft and column.
- c) Shaft steel reinforcing bar concrete cover tolerance shall be 13 mm. Ensure that the reinforcing cage is concentric with the shaft within a tolerance of 25 mm.
- d) After placing all the concrete, ensure that the top of the reinforcing steel cage is no more than 75 mm above or below the plan position.
- e) All casing diameters shall conform to the Plan dimensions. The Contractor may use different casing diameter if it can be proved the diameter of the drilled shaft meets the design and it must be preapproved by the Contract Administrator. When conditions are such that a series of telescoping casings are used, provide the casing sized to maintain the minimum shaft diameters.
- f) Use excavation equipment and methods designed so that the completed shaft excavation will have a flat bottom. Ensure that the cutting edges of excavation equipment are normal to the vertical axis of the equipment within a tolerance of plus or minus 100 mm.

8.06 Repair of Welds

Any section of weld that does not meet the requirements of the Contract Documents shall be removed and rewelded.

8.07 Quality Control

8.07.01 Inspection and Testing of Welds

8.07.01.01 Qualifications of Companies and Individuals

An independent testing company with no corporate affiliation with the Contractor shall be employed to carry out the non-destructive testing of welds. The independent testing company shall be certified by the Canadian Welding Bureau to the requirements of CSA W178.1 for bridge structures by radiographic or ultrasonic test methods.

Testing shall be done by a non-destructive testing technician employed by an independent testing company. The non-destructive testing technician shall have documented evidence of training and professional knowledge, skill, and experience in non-destructive testing of structural steel welds and material and have a valid certificate showing qualification to a Level II or III according to CAN/CGSB-48.9712 and the Canadian Welding Bureau for the non-destructive testing specified.

Visual inspections shall be performed by a welding inspector employed by an independent testing company. The welding inspector shall have documented evidence of training, professional knowledge, skill and experience in the visual inspection of structural steel welds and material, and have a valid certificate showing qualification to Level II or III according to CSA W178.2.

8.07.01.02 Visual Inspection of Welds

A representative sample of not less than 30% of the welds, as determined by the Contract Administrator, shall be visually inspected for conformance to the requirements of CSA W59, the Contract Documents, and the Working Drawings.

8.07.01.03 Non-Destructive Testing of Welds

Radiographic or ultrasonic testing shall be carried out 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 or the Welding Inspector assigned to carry out visual inspection.

8.07.01.04 Repaired Welds

All welds that have been repaired shall be visually inspected and shall undergo non-destructive testing.

8.08 Non-Destructive Post Construction Testing

Non-destructive QC concrete integrity testing of shafts will include Pile Integrity Testing (PIT) in accordance with ASTM D5882 and Crosshole Sonic Logging (CSL) in accordance with ASTM D6760.

The Contractor is responsible for the supply and assembly of access tubes for the testing, as well as the decommissioning of the access tubes upon completion of testing. The Contractor shall coordinate this work with the Contract Administrator, who will carry out the testing. Coordination efforts associated with testing are considered part of the work and no additional payment will be made by the Owner.

For this assignment, Pile Integrity Testing (PIT) shall be carried out on all drilled shaft and that Crosshole Sonic Logging (CSL) shall be carried out on at least one drilled shaft per foundation element.

8.08.01 Pile Integrity Testing (PIT) or Low Strain Impact Integrity Testing

PIT shall be performed in accordance with ASTM D5882.

The PIT shall be carried by the Contract Administrator on all production drilled shafts. The Contractor shall coordinate this work with the Contract Administrator. Coordination efforts associated with PIT are considered part of the work and no additional payment will be made by the Owner.

8.08.01.01 Preparation of the Surface of the Drilled Shaft

The Contractor shall ensure that the pile head surface is accessible, above water, and clean of loose concrete, soil or other foreign materials resulting from construction. The Contractor shall remove sufficient pile section to reach sound concrete, and to prepare a smooth surface for sensor attachment and impact.

8.08.01.02 Procedure

The PIT testing shall be carried at least 7 days after shaft concrete placement or after the concrete has achieved 75% of the design strength, whichever occurs earlier.

8.08.02 Cross-Hole Sonic Logging (CSL)

CSL shall be performed in accordance with ASTM D6760.

The CSL shall be carried by the Contract Administrator on the following number production drilled shafts:

Structure	Foundation Element	Number of Test(s)
GO Transit / Metrolinx Overhead	West Pier	1
	East Pier	1
Leslie Street Overpass	Pier 1	1
	Pier 2	1
	Pier 3	1
Don River East Branch Bridge	West Pier	1
	East Pier	1

The Contractor shall coordinate this work with the Contract Administrator. Coordination efforts associated with CSL are considered part of the work and no additional payment will be made by the Owner.

When a shaft contains three or four tubes, test shall be carried out at every possible tube combination. For shafts with five or more tubes, test all pairs of adjacent tubes around the perimeter, and one-half of the remaining number of tube combinations, chosen randomly but always including the diametrically opposite tube.

8.08.02.01 Access Tubes Supply, Assembly and Decommissioning

The Contractor shall securely attach the access tubes to the interior of the reinforcement cage of the shaft. The following number of access tubes shall be furnished and installed for each test:

Diameter of Drilled Shaft	Number of Access Tubes
Less than 1000 mm	3
1000 mm to less than 1500 mm	4
1500 mm to 2100 mm	6

The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement, and bundled with the vertical reinforcement. Where circumferential components of the rebar cage bracing system prevent bundling the access tubes directly to the vertical reinforcement, the access tubes shall be placed inside the circumferential components of the rebar cage bracing system as close as possible to the nearest vertical steel reinforcement bar.

The access tubes shall be installed in straight alignment and as near to parallel to the vertical axis of the reinforcement cage as possible. The access tubes shall extend from the bottom of the reinforcement cage to at least 600 mm above the top of the shaft. Splice joints in the access tubes, if required to achieve full length access tubes, shall be watertight. The Contractor shall clear the access tubes of all debris and extraneous materials before installing the access tubes. The tops of access tubes shall be deburred. Care shall be taken to prevent damaging the access tubes during reinforcement cage installation and concrete placement operations in the shaft excavation.

The access tubes shall be filled with potable water before concrete placement, and the top watertight caps shall be reinstalled and secured. The Contractor shall keep all access tubes full of water through the completion of non-destructive QA testing of that shaft. When temperatures below freezing are possible, the Contractor shall protect the access tubes against freezing by wrapping the exposed tubes with insulating material, adding antifreeze to the water in the tubes, or other methods acceptable to the Contract Administrator.

After acceptance of production shafts by the Contract Administrator, the Contractor shall remove all water from the access tubes, fill the tubes with a structural non-shrinkable grout from the bottom via tremie tube. Place the grout utilizing enough pressure to fill the tubes completely.

8.08.02.01 Procedure

The CSL testing shall be carried at least 5 days after shaft concrete placement and after the concrete has achieved 65% of the design strength. Additional curing time prior to testing may sometimes be required. The Contractor shall furnish information regarding the shaft, tube lengths and depths, construction dates, and other pertinent shaft installation observations and details to the Contract Administrator prior to testing. The Contractor shall verify access tube lengths and their condition prior to CSL testing. If the access tubes

do not provide access over the full length of the shaft, the Contractor shall repair the existing tube(s) or core additional hole(s), as directed by the Contract Administrator.

8.09 Non-Destructive Quality Control Test Results Submittals

The Contract Administrator will evaluate the PIT and CSL results to determine if the shaft is acceptable. If the Contract Administrator determines additional evaluation is necessary, the Contract Administrator will specify the requirements. If repair is necessary, the Contractor is responsible for developing and submitting a repair plan to the Contract Administrator for approval as well as executing the approved plan.

8.10 Milestone Inspections

The Contract Administrator shall witness the following interim inspections of the work for drilled shaft:

- a) Excavation
- b) Steel reinforcement installation
- c) Placing of concrete

A Request to Proceed shall be submitted to the Contract Administrator after the excavation and prior to steel reinforcement installation and after the steel reinforcement installation and prior to concreting.

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

9.0 QUALITY ASSURANCE - Not Used

10.0 MEASUREMENT FOR PAYMENT

10.01 Actual Measurement

10.01.01 Supply Equipment for Installing Drilled Shaft – Item

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

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 5% of drilled shafts.

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 drilled shafts.

10.02 Drilled Shafts – Item **Cross Hole Sonic Logging Access Tubes and Caps – Item**

Drilled Shafts – Item

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

Cross Hole Sonic Logging Access Tubes and Caps – Item

Payment at the Contract Price for the above tender items shall be full compensation for all Labour, Equipment and Material to do the work.

RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL– Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and installation of the Rigid Expanded Polystyrene embankment fill and associated works as shown on the Contract Drawings.

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 212	Earth Borrow
OPSS.PROV 501	Compacting
OPSS.PROV 517	Dewatering
OPSS.PROV 904	Concrete Structures

Ontario Provincial Standard Specifications, Materials

OPSS.PROV 1010	Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
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National Standards of Canada

CAN/ULC-S102-10	Standard Method of Test for Surface Burning Characteristics of Building Materials and Assemblies
CAN/ULC-S701-97	Thermal Insulation, Polystyrene, Boards and Pipe Covering

ASTM International

ASTM C177	Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of Guarded-Hot-Plate Apparatus
ASTM C203	Standard Test Method for Breaking Load and Flexural Properties of Block-Type Thermal Insulation
ASTM C518	Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus
ASTM D1621	Standard Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM D2842	Standard Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Standard Test Method for Measuring the Minimum Oxygen Content
ASTM D6817	Standard Specification for Rigid Cellular Polystyrene Geofoam

Ontario Ministry of Transportation Publications

Designated Sources for Materials (DSM)

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Production Lot: means the quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Manufacturer: means the firm who supplies the Rigid Expanded Polystyrene

Rigid Expanded Polystyrene: means moulded rigid blocks listed on the DSM and produced by a process of pre-expansion, aging and forming of petroleum based raw material.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 Foundation Investigation Report

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

4.02 Submissions

4.02.01 Working Drawings

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the Working Drawings and method statement signed and sealed by the Contractors Engineer that provides full details of materials and construction procedure.

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) The method of construction of the levelling pad.
- c) The method of placement of Rigid Expanded Polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of reinforced concrete top slab.
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

4.02.02 Delivery, Storage, Handling, and Protection Procedure

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the Rigid Expanded Polystyrene manufacturers requirement.

4.02.06 Rigid Expanded Polystyrene

At least two (2) weeks prior to commencement of the installation of the Rigid Expanded Polystyrene blocks, the following details shall be submitted in writing to the Contract Administrator:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the Rigid Expanded Polystyrene.
3. An identification of the laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the Rigid Expanded Polystyrene.
4. The physical and mechanical properties of the Rigid Expanded Polystyrene including:
 - a) Geometry
 - b) Nominal Density
 - c) Compressive Strength
 - d) Flexural Strength
 - e) Thermal Resistance
 - f) Flammability
 - g) Water Absorption
5. Aging and durability characteristics of the Rigid Expanded Polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
6. A sample of the Rigid Expanded Polystyrene material.

4.02.07 Quality Test Certificates

Prior to installation of the Rigid Expanded Polystyrene, the Contractor shall submit Quality test certification for each production lot supplied from a laboratory accredited by the Standards Council. The Quality test certificates shall demonstrate compliance with all requirements of this special provision.

4.02.08 Rigid Expanded Polystyrene embankment

For each Rigid Expanded Polystyrene embankment, a Request to Proceed shall be submitted to the Contract Administrator at each of the following milestones:

- a) Following submission of the Quality Test Certificate and prior to construction.
- b) Following foundation excavation and preparation and prior to installation of the leveling pad;
- c) Following placement of Rigid Expanded Polystyrene blocks and prior to construction of the polyethylene sheeting and concrete top slab;

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

4.02.09 As-Built Drawings

As-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to final acceptance of work.

The as-built drawings shall be signed and sealed by the design Engineer and design check Engineer

5.0 MATERIALS

5.01 Granular Levelling Pad

The levelling pad shall be as specified elsewhere in the contract documents and consist of a Granular “A” material with gradation and physical requirements as specified in OPSS 1010.

5.02 Rigid Expanded Polystyrene

5.02.01 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The Rigid Expanded Polystyrene shall be free from defects affecting serviceability.

5.02.02 Detail Requirements

The Rigid Expanded Polystyrene shall be listed on the DSM and meet the physical and mechanical properties requirements shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear Dimensions - Flatness - Squareness	mm (min)	1200 x 600 x 300 ± 1%10 mm in 3 m ± 0.5%	--
Compressive Strength at 5% Deformation	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203 (Method 1, Procedure B.2.7.4)

Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

5.03 Polyethylene Sheeting

The protective sheeting shall be at a minimum 6 mil polyethylene sheeting or better if specified elsewhere in the Contract Package.

5.04 Concrete Top Slab

The reinforced concrete top slab shall be as specified elsewhere in the contract documents.

6.0 EQUIPMENT

All cutting of Rigid Expanded Polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the Rigid Expanded Polystyrene as per the manufacturer's requirement.

7.0 CONSTRUCTION

7.01 General

7.01.01 Rigid Expanded Polystyrene Installation

The installation of the Rigid Expanded Polystyrene shall be undertaken under the supervision of the Contractor's Engineer.

The Contractor inspection of the Rigid Expanded Polystyrene full-time.

The Contractor's manufacturer representative shall be on site to oversee installation of the Rigid Expanded Polystyrene blocks at the commencement of the installation.

7.02 Delivery, Storage and Handling

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the Rigid Expanded Polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

Rigid Expanded Polystyrene shall not be exposed to open flame or other ignition source. The contractor shall protect the Rigid Expanded Polystyrene blocks from petroleum-based products such as gasoline and diesel fuel and organic solvents such as acetone, benzene, and paint thinner.

7.02 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be sub-excavated and replaced with Granular 'A' or Granular 'B' material.

7.03 Leveling Pad

The Contractor shall place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS PROV 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The levelling pad shall not be placed on standing water, accumulated snow or ice or frozen ground. The levelling pad must be placed in-the-dry.

7.04 Installation of Blocks

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the Rigid Expanded Polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

The Rigid Expanded Polystyrene embankment shall be installed to ensure that:

1. The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
2. Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
3. A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer shall not exceed 5 mm.
4. Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
5. Temporary ballast shall be provided as necessary to prevent movement of Rigid Expanded Polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
6. The Rigid Expanded Polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the Rigid Expanded Polystyrene.

7. The Rigid Expanded Polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.
8. Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
9. Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
10. The top surface and side surfaces of the Rigid Expanded Polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

7.05 Side Slope Cover

The side slopes of the Rigid Expanded Polystyrene embankment shall be covered with granular fill as detailed elsewhere in the Contract drawings.

8.0 MEASUREMENT FOR PAYMENT

Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

9.0 BASIS OF PAYMENT

The Concrete top slab and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

OBSTRUCTIONS – Item No.

Non-Standard Special Provision

The Contactor shall be alerted to the potential presence of cobbles and boulders within the glacial till deposits at this site as inferred from auger grinding during advancement of Boreholes N/E RS-9 and M-04. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, protection system installation, pile installation, and installation of drilled shafts (caissons).

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

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10.0	BASIS OF PAYMENT

1.0 SCOPE

This special provision describes requirements for vibration monitoring for the following components of the Contract:

- Installation of deep foundations at Metrolinx Overhead (Site No. 37X-0206/B1).
- Installation of deep foundations at Don River East Branch Bridge (Site No. 37X-0207/B1).
- Installation of deep foundations at Leslie Street Overpass (Site No. 37X-0208/B1).

2.0 REFERENCES

The subsurface conditions at the site are described in the following Foundation Investigation Report:

1. Foundation Investigation Report; Metrolinx Overhead (Site No. 37X-0206/B1), Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue, Toronto, Ontario, MTO G.W.P. 2103-01-00.
2. Foundation Investigation Report; Don River Bridge East Branch Bridge (Site No. 37X-0207/B1), Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue, Toronto, Ontario, MTO G.W.P. 2103-01-00.
3. Foundation Investigation Report; Leslie Street Overpass (Site No. 37X-0208/B1), Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue, Toronto, Ontario, MTO G.W.P. 2103-01-00.

3.0 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second (mm/sec) that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory or vibration-inducing construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory or vibration-inducing construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor or the Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- b) Qualifications of vibration monitoring specialist.
- c) Details regarding proposed instrumentation.
- d) Proposed location of instruments adjacent to the on the residences, utilities, wells, or other potentially vibration-sensitive structures within a 100 m radius from the proposed structures.
- e) Proposed frequency of readings.
- f) Action plan to be taken to adjust deep foundation installation methods or if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within a 100 m radius from each structure.

7.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure within a 100 m radius from each structure, shall be completed a minimum of two (2) weeks prior to commencement of installation of the deep foundations. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

- a) Type of structure, including type of construction and if possible, the date when built.
- b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
- c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

7.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey within a 100 m radius from each structure, is required within two (2) months of completion of the installation of deep foundations.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

- a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
- b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
- c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that

residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface at radial distances of 25 m, 50 m, and 100 m from the bridge structure toward the receptors (e.g., buildings, sensitive utilities). The Contractor shall take readings continuously during installation of deep foundation elements (i.e. driving of piles, drilling of drilled shafts), and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on private/commercial structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities, if applicable, shall not exceed 50 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

7.3 Records

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

- a) The time/duration of each reading.
- b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.
- c) Details of exceedances and modifications to operations.
- d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

10.0 BASIS OF PAYMENT

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

WELL DECOMMISSIONING - Item No.

Special Provision

1.0 SCOPE

This specification covers the requirements for the decommissioning of wells/piezometers identified in Table 1 below, for which the registered owner is the Ministry of Transportation, Ontario.

Table 1 – Well/Piezometer Information

Well/Piezometer Identification	Location (Northing / Easting)	PVC Pipe and Screen Diameter / Borehole Diameter	Depth (Below Ground Surface) to Tip of Screen / Borehole Depth
GO-2A	(4,847,324.1 / 315,622.5)	50 mm / 200 mm	5.5 m / 44.2 m
DR-5	(4,847,350.9 / 316,071.6)	50 mm / 200 mm	5.2 m / 27.9 m
DR-7	(4,847,347.1 / 316,136.0)	50 mm / 150 mm	9.1 m / 30.8 m
WM16-07	(4,847,321.4 / 315,822.2)	50 mm / N/A	9.1 m / 18.9 m

2.0 REFERENCES

This specification refers to the following standards, specifications, or publications:

Ontario Water Resources Act, R.S.O. 1990; Regulation 903

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS – Not Used

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

Each well must be decommissioned/abandoned (sealed) by a licensed well contractor in accordance with regulations of the Ontario Water Resources Act (O.Reg. 903). The Contractor shall obtain said information from the Ministry of Environment, Conservation and Parks and bear liability for compliance to the regulation. As a minimum, the existing casing shall be removed to a depth of 2.0 m below the original ground surface and the remaining well casing backfilled in accordance with regulations.

As part of the abandonment process, in accordance with regulations of the Ontario Water Resources Act (O.Reg. 903), if the well has a well tag, it must be removed and returned to the MECP Director within 30 days after its removal. If the well has a well tag attached to the well casing or near the well, the well tag must

be removed at the beginning of the plugging operation and safeguarded throughout the process. The well tag must be returned within 30 days after completion of abandonment and removing the tag. The well tag must be returned to Wells Help Desk, Environmental Monitoring and Reporting Branch Ministry of the Environment, Conservation and Parks, 125 Resources Road, Toronto, Ontario, M9P 3V6 (1-888-396-9355).

Licensed well contractors shall forward the water well record (abandonment report), with an accompanying transmittal letter to the Ministry of Environment, Conservation and Parks. A copy of the above record and letter shall be sent to the Contract Administrator. This shall be provided to the Contract Administrator before payment of the abandoned wells is approved.

The Contractor must; obtain a blank well record form from the Ministry of the Environment, Conservation and Parks. On completion of the abandonment of a well, the Contractor must:

- Within 14 days after the date on which the well construction equipment is removed from the site, deliver a copy of the well record to the owner of the land on which the well is situated; and
- Within 30 days after the date on which the well construction equipment is removed from the site, forward a copy of the well record and any well tag that was removed from the well, to the Ministry of Environment, Conservation and Parks.

7.01 Removal and Disposal

Any effluent pumped during well decommissioning shall be managed in accordance with the requirements of O.Reg 347. Further, all material resulting from the abandonment of the wells shall become property of the Contractor and shall be disposed of in accordance with OPSS 180.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement for the above tender item shall be for each well decommissioned.

10.0 BASIS OF PAYMENT

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

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling - Structures is amended as follows:

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 2 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 150 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

Designer Fill-Ins

- * Fill in the design storm return period according to MTO Drainage Design Standard TW-1.
- ** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

DEEP FOUNDATIONS – Item No.

Special Provision

Amendment to OPSS.PROV 903, April 2016

903.01 SCOPE

Section 903.01 of OPSS.PROV 903 is amended by the addition of the following:

Under the above tender items, the Contractor shall:

- a) Supply and install H-Piles
- b) Coordinate with the Contractor Administrator or an independent testing company retained by the Contract Administrator for high strain dynamic testing.

All as shown on the Contract Drawings.

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

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.PROV 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be determined using the high-strain dynamic testing at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved, retap/restrike shall be conducted after initial driving as specified in the Contract Documents.

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.

903.07.02.07.04 Wave Equation Analysis

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

High-strain dynamic testing shall be performed by the Contract Administrator or an independent testing company retained by the Contract Administrator 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. The final piles to be tested will be decided by the Contract Administrator.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 25% of piles in each pile group, rounded up, but no fewer than 3 piles, or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 25% of piles in each pile group, rounded up, but no fewer than 3 piles, or as specified in the Contract Documents.

Restrike testing shall be carried out no sooner than 72 hours after installation of the individual pile and at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

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, 25% of the piles rounded up to the next whole number, but no fewer than 3 piles, shall be retapped no sooner than 72 hours after installation of the individual pile to confirm that the ultimate axial geotechnical resistance has been achieved and/or sustained.

Retapping of piles driven to bedrock is not required.



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