REPORT

Foundation Investigation and Design Report

Don River East Branch Bridge (Site No. 37X-0207/B1)
Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue, Toronto, Ontario
MTO G.W.P. 2130-01-00

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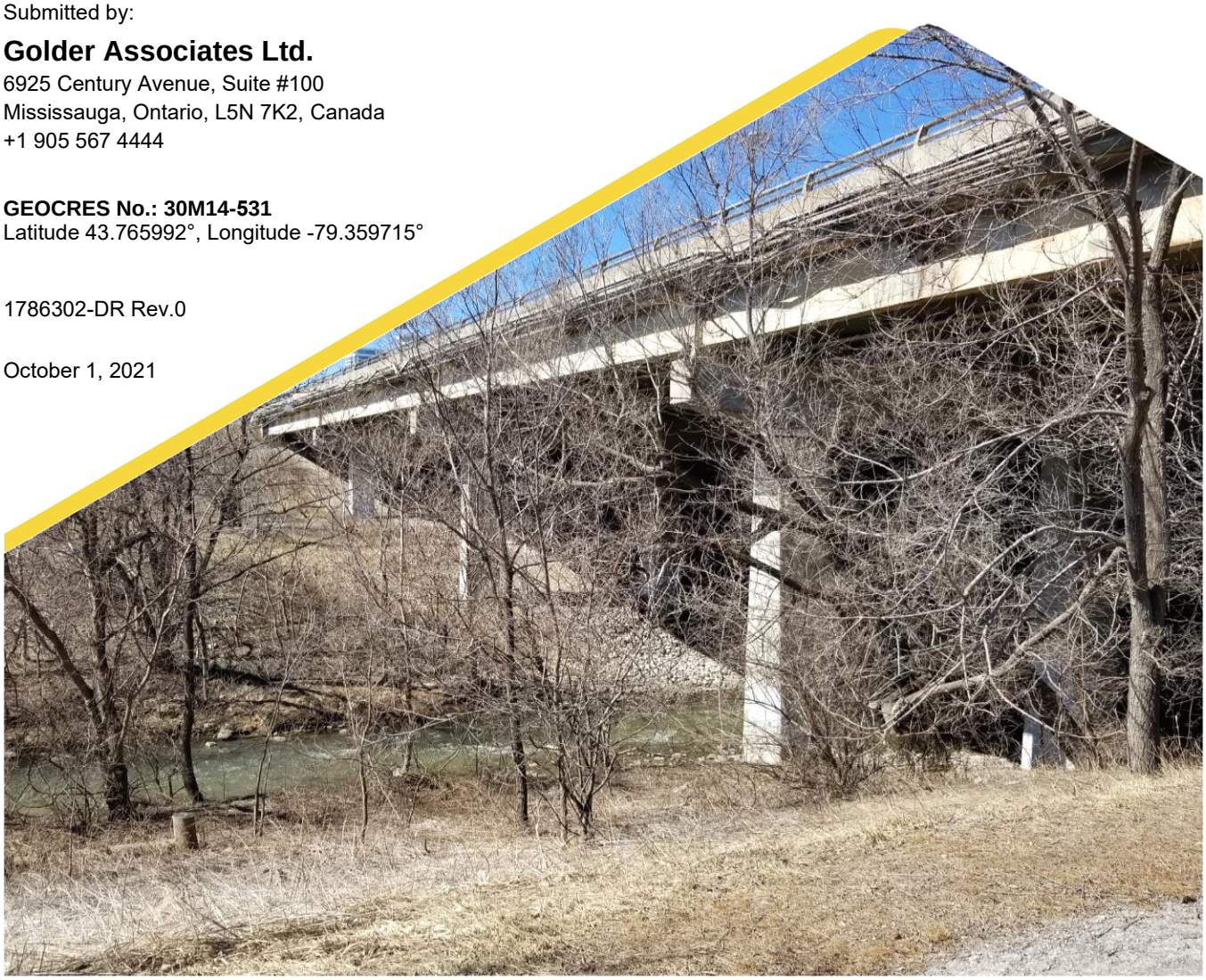

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Certificate of Analysis	Report # R5790260 Report # R5979847
Geomechanica Inc.	Rock Laboratory Testing Results

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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
DON RIVER EAST BRANCH BRIDGE (SITE NO. 37X-0207/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 the City of Toronto (MTO Assignment No. 2016-E-0089).

This report presents the subsurface conditions at the site for the proposed eastbound Highway 401 collector lane bridge over the Don River East Branch (Site No. 37X-0207/B1) and associated approach embankments. This report was developed based on the results from Golder's current foundation investigation and a 2015 investigation and testing completed by others (MTO GEOCRES Report No. 30M14-462).

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

2.0 SITE DESCRIPTION

The existing core and collector bridges carry Highway 401 over the East Don River, which flows from north to south. The natural ground surface at the crest of the valley slope along the proposed Highway 401 eastbound collector lane bridge alignment is at approximately Elevation 135 m on the west side of the river, and approximately Elevation 125 m to 128 m on the east side of the river in the vicinity of the existing and proposed abutments. The Highway 401 grade declines from west to east, from approximately Elevation 139 m near the west abutment to approximately Elevation 135 m near the east abutment. The river channel at the base of the valley is at approximately Elevation 122 m, and the design high water level in the East Don River is at approximately Elevation 126.5 m. The existing valley slopes are orientated at approximately 4H:1V.

The existing Highway 401 bridges over Don River are five-span structures with a total length of about 120 m. The existing piers are supported on battered Franki piles or drilled shafts, and the existing abutments are supported on steel H-piles.

The site, including the river valley slopes, is generally covered by vegetation ranging from low-lying grass areas to heavily treed areas. Betty Sutherland Trail, a paved walking/cycling trail that traverses under Highway 401, is located near the east bank of Don River. Urban development to the east of Leslie Street includes North York General Hospital and a residential neighbourhood north of Highway 401, and industrial complexes to the south.

3.0 INVESTIGATION PROCEDURES

3.1 2015 Investigation (GEOCRES No. 30M14-462)

From February to May 2015, a foundation investigation was carried out by Thurber Engineering Ltd. (Thurber) during which time a total of five boreholes were advanced, designated as Boreholes DRB-01 to DRB-05. The results of the Thurber investigation are contained in their report titled, "Foundation Investigation Report, Don River Bridge at Highway 401, East of Leslie Street Interchange, Toronto, Ontario, W.P. 2061-13-00, Site 37-207", dated September 5, 2017 (GEOCRES No. 30M14-462).

While this report does 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 location of boreholes that are relevant to the subsurface conditions at or near the proposed new eastbound collector structure are summarized in the table below along with the geographic coordinates, ground surface elevations (referenced to Geodetic Datum), and the depths of the boreholes. These 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.	Location	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Depth of Borehole (m)
		Northing (Latitude, °)	Easting (Longitude, °)		
DRB-03	West Abutment	4,847,311.3 (43.765684)	316,033.9 (-79.360475)	130.2	31.8
DRB-04	West Pier	4,847,327.5 (43.765830)	316,072.7 (-79.359992)	129.2	31.8
DRB-05	East Abutment	4,847,331.7 (43.765866)	316,157.9 (-79.358934)	129.4	30.6

3.2 Current Investigation

The foundation investigation for the Highway 401 eastbound collector bridge over East Don River was carried out between June 3 and August 30, 2019, during which time total of seven boreholes (designated as Boreholes DR-1, DR-3, DR-5, DR-6, DR-7, DR-9, and DR-10), and five CPTs (designated as CPT DR-2a, DR-2b, DR-4, DR-8a and DR-8b) 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 records of the boreholes, drillholes and CPTs are provided in Appendix B. Lists of abbreviations and symbols and lithological and geotechnical rock description terminology are also provided in Appendix B to assist in the interpretation of the borehole and drillhole records.

Borehole DR-1 was advanced using hollow stem augers, by a D120 truck-mounted drill rig supplied and operated by Altech Drilling and Investigative Services Ltd. of Cambridge, Ontario. Boreholes / CPTs DR-2a to DR-10 were advanced using hollow stem augers by truck-mounted and track-mounted CME-55 and CME-75 drilling rigs, supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, Ontario. The boreholes and CPT holes were advanced through the overburden using hollow-stem augers and casing with drilling mud.

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. Cone Penetration Tests were carried out using the hydraulic ram system on the drill rigs, after pre-augering through the upper non-cohesive deposits. Core samples of the bedrock were obtained using an ‘HQ’ size rock core barrel and coring techniques in Boreholes DR-3, DR-6, DR-7 and DR-9.

The groundwater conditions and water levels in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Boreholes DR-5 and DR-7 to allow monitoring of the water level at these locations. The installed piezometers consist of a 50 mm diameter PVC pipe with a slotted screen sealed within a selected depth in 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 as near original condition as practicable 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/test locations, arranged for the clearance of underground services, 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. All the soil laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. A one-dimensional consolidation (oedometer) test was carried out on a sample 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 geotechnical laboratory testing 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 a GPS Trimble GEO 7X, having an accuracy of approximately 0.1 m in the vertical and horizontal directions. 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 locations, geographic coordinates, ground surface elevations and drilled depths are summarized below.

Borehole / CPT Hole No.	Location	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole / CPT Hole Depth (m)
		Northing (Latitude, °)	Easting (Longitude, °)		
DR-1	West Approach	4,847,342.6 (43.765966)	316,023.0 (-79.360609)	138.5	12.8
DR-2a	West Abutment	4,847,305.5 (43.765632)	316,039.1 (-79.360410)	130.1	13.6
DR-2b		4,847,305.5 (43.765632)	316,039.1 (-79.360410)	130.1	13.7
DR-3		4,847,343.0 (43.765970)	316,026.2 (-79.360569)	138.5	41.0

Borehole / CPT Hole No.	Location	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole / CPT Hole Depth (m)
		Northing (Latitude, °)	Easting (Longitude, °)		
DR-4	West Pier	4,847,328.0 (43.765834)	316,067.6 (-79.360055)	129.3	13.4
DR-5		4,847,350.9 (43.766040)	316,071.6 (-79.360005)	129.1	27.9
DR-6	East Pier	4,847,329.4 (43.765846)	316,137.5 (-79.359188)	128.0	30.8
DR-7		4,847,347.1 (43.766005)	316,136.0 (-79.359205)	129.3	30.8
DR-8a	East Abutment	4,847,336.7 (43.765911)	316,160.8 (-79.358898)	129.4	15.0
DR-8b		4,847,337.4 (43.765917)	316,158.6 (-79.358925)	129.4	6.2
DR-9		4,847,356.9 (43.766093)	316,164.9 (-79.358846)	134.5	39.3
DR-10	East Approach	4,847,357.5 (43.766098)	316,171.2 (-79.358769)	134.4	15.9

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 General Overview of Subsurface Conditions

The borehole/drillhole records and laboratory testing summary figures from the 2015 investigation (GEOCRES No. 30M14-462) are presented in Appendix A. The soil, bedrock and groundwater conditions as encountered in the boreholes / CPTs advanced during the current investigation are presented on the borehole, drillhole and CPT

[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.

records in Appendix B. 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 and field vanes) as presented in the borehole records and in Section 4.2 are uncorrected. The boundaries between the soil deposits on the borehole records and on the interpreted stratigraphic profile and sections on Drawings 2 to 4 have been inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and Cone 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 boreholes encountered surficial layers of topsoil and asphalt, underlain by non-cohesive and cohesive fill. The fill is underlain by a sequence of non-cohesive and cohesive deposits, commencing with a surficial granular deposit consisting of sandy silt to silty sand, which is in turn underlain by a cohesive deposit consisting of clay to sandy clayey silt-silt, which contains an interlayer of sandy silt to silty sand. Underlying this cohesive deposit is a granular deposit consisting of silt to sand, followed by another cohesive deposit consisting of silty clay to clayey silt, then a cohesive to non-cohesive till deposit, overlying residual soil and shale bedrock. More detailed descriptions of the subsurface conditions encountered in the boreholes is provided in the following sub-sections.

4.2.1 Topsoil

An approximately 50 mm to 175 mm thick layer of topsoil was encountered at ground surface in Boreholes DRB-03 to DRB-05, and DR-7. These materials were classified solely based on visual and textural evidence. Testing of 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.

4.2.2 Asphalt and Granular Subbase

An approximately 100 mm to 280 mm thick layer of asphalt pavement was encountered at ground surface in Boreholes DR-1, DR-3, DR-9 and DR-10, which were advanced from the Highway 401 grade. An 80 mm thick layer of granular subbase was encountered in Borehole DR-10 underlying the asphalt.

An approximately 38 mm thick layer of asphalt pavement was encountered at ground surface in Borehole DR-6, advanced through the walking/cycling path of Betty Sutherland Trail.

4.2.3 Concrete

An approximately 70 mm thick layer of concrete, a part of the composite pavement structure, was encountered underlying the asphalt and granular subbase in Borehole DR-10.

Attempts at advancing boreholes using conventional augers near the existing west and east abutments were unsuccessful due to the presence of a reinforced composite pavement structure underlying the asphalt layer and as such, the as-drilled locations for Boreholes DR-3 and DR-9 were located further away from the abutments to avoid the composite pavement structure.

4.2.4 SILTY SAND (SM) to SAND (SP) and Gravel (FILL)

A 0.1 m to 2.9 m thick layer of non-cohesive fill consisting of silty sand, trace to some gravel, to sand and gravel trace to some silt was encountered at the surface of Borehole DR-5 and underlying the asphalt or topsoil in Boreholes DR-1, DR-3, DR-6, DR-7, DR-9 and DR-10. The top of the non-cohesive fill was encountered at depths ranging from 0.0 m to 0.3 m below ground surface (between Elevations 138.3 m and 128.0 m) and extended to depths ranging from 0.1 m to 3.0 m below ground surface (between Elevations 136.3 m and 127.9 m).

The Standard Penetration Test (SPT) “N”-values measured within the silty sand to sand and gravel fill range from about 11 to 30 blows per 0.3 m of penetration, indicating a compact state of compactness.

The water content measured on samples of the non-cohesive fill ranges from about 4% to 10%.

4.2.5 SILTY CLAY (CI) to CLAYEY SAND (SC) (FILL)

A 0.5 m to 10.3 m thick layer of cohesive fill was encountered underlying the topsoil in Borehole DRB-05, and underlying the non-cohesive fill in Boreholes DR-1, DR-3, DR-5, DR-6, DR-7, DR-9 and DR-10. The cohesive fill ranges from silty clay, trace to some gravel to clayey sand, trace gravel. The top of the cohesive fill was encountered at depths ranging from 0.1 m to 3.0 m below ground surface (between Elevation 136.3 m and 128.4 m) and it extends to depths ranging from 3.0 m to 11.7 m below ground surface (between Elevations 134.0 m and 122.8 m). An approximately 130 mm thick layer of asphalt was encountered within the cohesive fill in Borehole DR-9 at a depth of 9.4 m below ground surface (Elevation 125.1 m), and pockets of sand were encountered within the cohesive fill at depths of 1.7 m and 6.2 m in Boreholes DR-10 and DR-9, respectively.

The SPT “N”-values measured within the cohesive fill range from 2 to 33 blows per 0.3 m of penetration, indicating a variable, soft to very stiff consistency.

Grain size distribution testing was carried out on five samples of the cohesive fill, and the results are presented on Figures C1 and C2 in Appendix C. Atterberg limits testing on three samples of the fill measured liquid limits ranging from about 18% to 23%, plastic limits of about 12%, and plasticity indices ranging from about 7% to 10%. These Atterberg limits test results are presented on Figure C3 in Appendix C and indicate the cohesive fill is a clayey silt to clayey silt-silt of low plasticity. Atterberg limits testing was also carried out on a sample of the clayey sand and measured a liquid limit of about 25%, a plastic limit of about 14%, and a plasticity index of about 11%; this result is presented on Figure C4 in Appendix C and indicates that the fines portion of the deposit is a clayey silt of low plasticity. The water content measured on samples of the cohesive fill range from about 9% to 22%.

4.2.6 ORGANIC SILT (OL)

A 0.8 m thick deposit of organic silt, trace sand was encountered underlying the fill material in Borehole DR-9. This organic silt was encountered at a depth of 11.7 m below ground surface (Elevation 122.8 m); the thicker fill encountered in this borehole may be indicative of previous sub-excavation or partial sub-excavation of organic materials, which may suggest that this organic layer could be thicker beyond the borehole location.

One SPT “N”-value of 7 blows per 0.3 m of penetration was measured within the organic silt deposit, suggesting a firm consistency.

The water content measured on the single sample of this layer was about 126% .

4.2.7 SANDY SILT (ML) to SILTY SAND (SM)

A 1.4 m to 13.3 m thick deposit of sandy silt, trace to some gravel, to silty sand, some gravel was encountered underlying the fill material in Boreholes DRB-05, DR-1, DR-3 and DR-5, and underlying the topsoil in Boreholes DBR-03 and DRB-04. The top of the sandy silt to silty sand deposit was encountered at depths ranging from 0.1 m to 5.6 m below ground surface (between Elevations 134.7 m and 123.5 m) and the deposit extends to depths ranging from 7.0 m to 10.2 m below ground surface (between Elevations 128.9 m and 120.7 m).

In general, the SPT “N”-values measured within the deposit range from 2 to 63 blows per 0.3 m of penetration, indicating a variable, very loose to very dense state of compactness. An SPT “N”-value as high as 50 blows per 0.1 m of penetration was recorded in Borehole DRB-04, and is attributed to the presence of cobbles.

Grain size distribution testing was carried out on six samples of the non-cohesive deposit. Three of the results are shown on Figure A1 in Appendix A, and three results are shown on Figure C5 in Appendix C. An Atterberg limits test was carried out on a sample of the deposit and measured a liquid limit of about 15%, a plastic limit of about 12%, and a plastic index of about 3%. The Atterberg limits test result is presented on Figure C6 in Appendix C and indicates that the fine portion of the soil is a silt of low plasticity. The water content measured on samples of this deposit range from about 6% to 41%.

4.2.7.1 Sandy CLAYEY SILT (CL) – Interlayer

An interlayer of sandy clayey silt, trace gravel, up to about 3.8 m thick, was encountered within the sandy silt to silty sand deposit in Boreholes DR-1 and DR-3. The top of the sandy clayey silt interlayer was encountered at depths of 9.7 m and 10.2 m below ground surface (at Elevations 128.9 m and 128.3 m) in Boreholes DR-1 and DR-3, respectively. The interlayer extended to a depth of 14.0 m below ground surface (Elevation 124.5 m) in Borehole DR-3, and Borehole DR-1 was terminated within the sandy clayey silt interlayer at a depth of 12.8 m below ground surface (Elevation 125.7 m). Rootlets were present within samples obtained from Boreholes DR-1 and DR-3.

The SPT “N”-values measured within the sandy clayey silt interlayer range from 9 to 21 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency.

Grain size distribution testing was carried out on three samples of the sandy clayey silt interlayer, and the results are shown on Figure C7 in Appendix C. Atterberg limits testing on three samples of the cohesive interlayer measured liquid limits ranging from about 23% to 28%, plastic limits ranging from about 13% to 16%, and plasticity indices ranging from about 10% to 13%. The Atterberg limit test results are presented on Figure C8 in Appendix C and indicate that the soil is a clayey silt of low plasticity. The water contents measured on samples of this interlayer range from about 10% to 21%.

4.2.8 CLAY (CH) to Sandy CLAYEY SILT-SILT (CL-ML)

A 4.3 m to 12.6 m thick deposit of clay to sandy clayey silt-silt, trace sand to sandy, trace gravel to gravelly, was encountered underlying the fill in Boreholes DR-6, DR-7, DR-9 and DR-10 on the east side of the river, and underlying the sandy silt to silty sand deposit in Boreholes DRB-03, DRB-04, DRB-05, DR-3 and DR-5.

The cohesive deposit was encountered at depths ranging from 2.2 m to 17.1 m in the boreholes (between Elevations 125.8 m and 120.7 m). Within the boreholes in which the deposit was fully penetrated, the cohesive deposit extends to depths of 12.5 m to 22.9 m (between Elevations 116.6 m and 113.2 m).

In general, the SPT “N”-values measured within the clay to sandy clayey silt-silt deposit range from 2 to 43 blows per 0.3 m of penetration, suggesting a very soft to hard consistency. An SPT “N”-value of 90 blows per 0.3 m of penetration was measured in Borehole DR-7 immediately below the silt and sand interlayer. In-situ field vane tests carried out within the deposit measured undrained shear strengths ranging from about 38 kPa to greater than 96 kPa with a calculated sensitivity between about 2 and 3.5. The field vane test results indicate that the clay to clayey silt-silt deposit has a firm to stiff consistency.

Grain size distribution testing was carried out on fifteen samples of the cohesive deposit. The results of five grain size distribution tests associated with the applicable boreholes from the 2015 investigation are shown on Figures A2 and A3 in Appendix A, and ten results from the current investigation are shown on Figures C9A and C9B in Appendix C. Atterberg limits testing on sixteen samples of the cohesive deposit measured liquid limits ranging from about 19% to 56%, plastic limits ranging from about 11% to 21%, and plasticity indices ranging from about 7% to 35%, indicating that the cohesive deposit ranges from clayey silt-silt of low plasticity to clay of high plasticity. Five of the results from boreholes associated with the 2015 investigation are presented on Figures A8 and A9 in Appendix A, and eleven results from the current investigation are presented on Figure C10 in Appendix C. The water content measured on samples of this deposit range from about 10% to 39%.

Laboratory consolidation testing was carried out on a sample of the clay to sandy clayey silt-silt deposit obtained from a Shelby tube sample in Borehole DR-7. Based on the results of the laboratory consolidation tests and the in-situ testing, the cohesive deposit is lightly over-consolidated with an over-consolidation ratio (OCR) of about 2.2. Details of the consolidation test results are shown on Figures C11A to C11D 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)
DR-7 / SA 8	7.9 / 121.4	21	150	335	185	2.2	0.14	0.009	0.51	1.0 x 10^{-2}

Where:

γ	Is the bulk unit weight in kN/m^3
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_o	Is the initial void ratio
C_v	Is the coefficient of consolidation, valid for 150 kPa to 300 kPa vertical effective stress range.

Three Cone Penetration Tests (DR-2, DR-4 and DR-8) were advanced from the bottom of the upper sandy silt to silty sand deposit into the cohesive deposit for measurement of the tip resistance (qt), sleeve friction (fs) and pore water pressure (u). The results of the CPTs are presented on the Cone Penetration Test sheets in Appendix B.

As part of the cone penetration testing, four in-situ pore water pressure dissipation tests were carried out in the cohesive deposit to assess the coefficient of consolidation at a specific horizon within the clay to sandy clayey silt-silt deposit. The results of the pore water dissipation tests are shown on Figure C12 in Appendix C, and the coefficient of consolidation in the horizontal direction (ch) obtained from the results of the dissipation tests is summarized below.

CPT Hole No.	Depth / Elevation (m)	c_h (cm^2/s)
DR-2	11.3 / 118.8	9.6 x 10^{-3}
DR-2	12.4 / 117.7	1.9 x 10^{-2}
DR-4	9.0 / 120.3	7.2 x 10^{-3}

CPT Hole No.	Depth / Elevation (m)	c_h (cm^2/s)
DR-4	10.0 / 119.3	4.9×10^{-3}

One multi-stage consolidated, isotropic, undrained (CIU) triaxial test was carried out on one undisturbed sample of the clayey silt. The results are presented on Figures C13A to C13D in Appendix C and are summarized in the table below.

Borehole / Sample No.	Depth / Elevation (m)	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, Φ' (Degrees)
DR-4 / TO 12	13.95 / 114.9	0	33

4.2.8.1 Sandy SILT (ML) to SILTY SAND (SM) – Interlayer

Within the clay to sandy clayey silt-silt deposit, a 0.5 m to 3.0 m thick interlayer of sandy silt to silt and sand to silty sand, trace to some gravel was encountered in Boreholes DRB-05, DR-6, DR-7, DR-9 and DR-10. The non-cohesive interlayer was encountered at depths ranging from 5.3 m to 15.4 m below ground surface (between Elevations 123.2 m and 119.0 m) and it extends to depths ranging from 7.2 m to 18.4 m below ground surface (between Elevations 122.7 m and 116.1 m).

The SPT "N"-values measured within the sandy silt to silty sand interlayer range from 3 blows to 42 blows per 0.3 m of penetration, suggesting a variable, very loose to dense state of compactness.

Grain size distribution testing was carried out on three samples of this deposit and the results are shown on Figure C14 in Appendix C. The water content measured on four samples of this non-cohesive layer range from about 10% to 43%.

4.2.9 SILT (ML) to SAND (SW)

A 6.4 m to 12.5 m thick deposit of silt to sandy silt to silt and sand to silty sand to sand was encountered underlying the clay to sandy clayey-silt cohesive deposit in Boreholes DRB-03, DRB-04, DRB-05, DR-3, DR-5, DR-6, DR-7 and DR-9. The silt to sand deposit was encountered at depths ranging from 12.5 m to 22.9 m below ground surface (between Elevations 116.6 m and 113.2 m) and it extended to depths of 21.6 m to 32.0 m below ground surface (between Elevations 109.2 m and 102.5 m).

The SPT "N"-values measured within this lower silt to sand deposit range from 3 blows to 79 blows per 0.3 m of penetration, indicating a variable, very loose to very dense compactness condition.

Grain size distribution testing was carried out on eight samples of the deposit. The results of four grain size distribution tests associated with the relevant boreholes from the 2015 investigation are presented on Figures A4 and A5 in Appendix A, and four grain size distribution tests from the current investigation are presented on Figure C15 in Appendix C. Atterberg limit tests were carried out on two samples of this silt to sand deposit and were both non-plastic. The water contents measured on samples of the deposit range from about 8% to 25%.

4.2.9.1 SILTY CLAY (CI) to CLAYEY SILT (CL) – Interlayer

A 2.8 m to 4.3 m thick interlayer of silty clay to clayey silt, trace to some sand, trace gravel, was encountered in Boreholes DR-3, DR-7 and DR-9 at depths ranging from about 21.6 m to 29.3 m below ground surface (between Elevations 109.8 m and 107.7 m) and extends to depths ranging from 24.4 m to 32.3 m below ground surface (between Elevations 106.2 m and 104.9 m).

In general, the SPT "N"-values measured within the silty clay to clayey silt interlayer range from 31 to 45 blows per 0.3 m of penetration, indicating a hard consistency.

Grain size distribution testing was carried out on two samples of this interlayer and the results are presented in Figure C16 in Appendix C. Atterberg limits testing on three samples of the deposit measured liquid limits ranging from about 25% to 42%, plastic limits ranging from about 17% to 20%, and plasticity indices ranging from about 8% to 22%. The Atterberg limit test results, as presented on Figure C17 in Appendix C, indicate that the deposit is comprised of a clayey silt of low plasticity to a silty clay of medium plasticity. The water content measured on samples of the deposit range from about 21% to 24%.

4.2.10 SILTY CLAY (CI) to Sandy GRAVEL (GW) Till

A 3 m to 5.2 m thick glacial till deposit was encountered underlying the lower silt to sand deposit in Boreholes DRB-03, DRB-04, DRB-05, DR-5 and DR-6 and underlying the silty clay to clayey silt interlayer in Boreholes DR-3 and DR-7. The cohesive portion of the glacial till deposit varies from silty clay, trace sand, trace gravel, to sandy gravelly clayey silt. The non-cohesive portion of the till deposit grades from silty sand to sandy gravel, trace fines. The till deposit was encountered at depths ranging from 22.9 m to 32.3 m below ground surface (between Elevations 106.2 m and 103.9 m) and extends to depths ranging from 25.9 m to 36.9 m below ground surface (between Elevations 102.1 m and 99.8 m). Although not encountered in these boreholes aside from auger grinding in Borehole DR-6, cobbles and boulders are commonly present within glacially derived soils and should be expected within this deposit.

The SPT "N"-values measured within the cohesive portions of the till deposit range from 19 blows per 0.3 m of penetration to 165 blows per 0.2 m of penetration, suggesting a very stiff to hard consistency. The SPT "N"-values measured within the non-cohesive portion of the till deposit range from 23 blows per 0.3 m of penetration to 100 blows to 0.1 m of penetration, indicating a compact to very dense state of compactness.

Grain size distribution testing was carried out on two samples of the non-cohesive till and one sample of the cohesive till, and the results are presented on Figures A6 and A7 respectively in Appendix A. Atterberg limits testing was carried out on a sample of the gravelly sand till deposit and measured a liquid limit of about 17%, a plastic limit of about 12%, and a plasticity index of about 5%; this result, as presented on Figure C18 in Appendix C, indicates that the fines portion of the gravelly sand till is a clayey silt-silt of low plasticity. The water contents measured on samples of the till deposit range from about 6% to 18%.

4.2.11 Residual Soil

Approximately 0.2 m to 1.8 m of residual soil was encountered underlying the glacial till deposit in Boreholes DR-5, DR-6 and DR-7. The residual soil consists of silty clay to clayey silt, some gravel to gravelly, some sand to sandy, and contains shale fragments. The residual soil was encountered at depths ranging from 25.9 m to 29.1 m below ground surface (between Elevations 102.1 m and 100.2 m) and extends to depths ranging from 27.7 m to 29.3 m below ground surface where fully penetrated (between Elevations 100.2 m and 100 m).

The SPT "N"-values measured within the residual soil range from 100 blows per 0.3 m of penetration to 102 blows per 0.2 m of penetration, suggesting a hard consistency.

4.2.12 Shale Bedrock

Bedrock was encountered in Boreholes DRB-03 to DRB-05, DR-3, DR-6, DR-7 and DR-9. Core samples were recovered in all boreholes that encountered bedrock, except for Borehole DRB-05 from the 2015 investigation, where the bedrock surface was inferred from split-spoon sampling. The depths to bedrock, the corresponding surface elevation, and the cored depths are summarized below.

Foundation Element	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
West Abutment	DR-3	36.9	101.6	0.1 m of auger penetration; bedrock cored 4.0 m
	DRB-03	28.9	101.3	0.1 m of auger penetration; bedrock cored 3.0 m
West Pier	DRB-04	28.8	100.4	Bedrock cored 2.5 m
East Pier	DR-6	27.7	100.3	0.1 m of auger penetration; bedrock cored 3.0 m
	DR-7	29.3	100.0	Bedrock cored 1.5 m
East Abutment	DR-9	32.0	102.5	3.4 m of auger penetration; bedrock cored 4.1 m
	DRB-05	29.6	99.8	1.0 m of auger penetration

In general, the bedrock surface as encountered or inferred in the boreholes is relatively flat on the west and east sides of the Don River valley; there appears to be greater variation in the bedrock surface in the vicinity of the east abutment, and variation in the bedrock surface should be expected between borehole locations at the other foundation elements.

The bedrock consists of grey shale of the Georgian Bay formation, which contains limestone and siltstone interbeds at various intervals. The bedrock samples are generally described as moderately weathered to fresh, although a zone of completely weathered shale was logged in Borehole DR-7; the surface and upper portion of the shale bedrock generally exhibit a higher degree of weathering, becoming less weathered then fresh with depth. The recovered shale core samples are thinly bedded, fine grained, non-porous to slightly porous, and very weak to medium strong. The stronger limestone/siltstone layers range from about 10 mm to 30 mm in thickness. The details of the bedrock descriptions are presented in the borehole records from the 2015 investigation in Appendix A, and on the drillhole records from the current investigation in Appendix B, together with photographs of the recovered bedrock core samples on Figures B1 to B4. 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 Rock Quality Designation (RQD) measured on the core samples obtained from the current investigation range from about 50% to 93%, indicating a rock mass of fair to excellent quality, as per Table 3.10 of CFEM (2006)[4]. The RQD measured on the core samples obtained from the previous investigation range from about 0% to 97%, indicating a rock mass of very poor to excellent quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered from the current and previous investigation are between 87% and 100% and between 27% and 100%, respectively.

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. Results from Borehole DRB-04 from the 2015 investigation are also included in the table below, and the associated point load test sheet can be found in Appendix A.

Borehole	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm^3)	Tangent Young's Modulus (GPa)
DR-3	40.84 – 41.02	97.66 – 97.48	34.6	2.619	10.1
DR-6	30.23 – 30.42	97.77 – 97.58	37.2	2.627	8.6
DR-9	37.99 – 38.21	96.51 – 96.29	45.2	2.616	8.3
DRB-04	29.0	100.2	6.8	-	-
	29.4	99.8	24.0	-	-
	30.3	98.9	18.1	-	-
	31.0	98.2	17.3	-	-
	31.6	97.6	15.6	-	-

Based on the laboratory UCS, in accordance with Table 3.5 in CFEM (2006), the shale bedrock is generally classified as weak (R2, 5 MPa < UCS < 25 MPa) to medium strong (R3, 25 MPa < UCS < 50 MPa).

4.3 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling are summarized on the borehole records. Standpipe piezometers were installed in two boreholes to monitor the groundwater level at the site, as shown on the borehole records and summarized below. One standpipe piezometer was also installed as part of

[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.

the 2015 investigation. 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.

Foundation Element	Borehole	Screened Stratigraphy	Water Level		Date of Measurement
			Depth (m)	Elevation (m)	
West Abutment	DRB-03	Silty Sand to Sand Till	-0.5[1]	130.7	June 30, 2015
			3.5	126.7	June 17, 2015
West Pier	DR-5	Sandy Silt to Silty Sand	6.0	123.1	August 30, 2019
			5.6	123.5	October 25, 2019
			5.8	123.3	September 14, 2020
East Pier	DR-7	Silt Interlayer of Clay to Clayey Silt-Silt	6.4	122.9	August 20, 2019
			4.5	124.8	May 1, 2020
			4.8	124.5	September 14, 2020

Note:

1. Negative value indicates the groundwater level is above ground surface.

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 No.	Sample No.	Sample Depth (Elevation) (m)	Soil Type	Parameters				
				Chloride (µg/g)	Sulphate (µg/g)	pH	Conductivity (µmho/cm)	Resistivity (ohm-cm)
DR-3	11	126.3 – 125.7	Sandy Clayey Silt	210	<20	7.61	480	2,100
DR-5	6	125.3 – 124.7	Sandy Clayey Silt Fill	400	<20	7.48	823	1,200
DR-6	5	123.4 – 122.8	Clayey Silt	63	350	7.64	541	1,800
DR-9	17	107.1 – 106.5	Silty Clay	25	140	7.84	313	3,200

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., an MTO Foundations Designated Contact and Principal with Golder, conducted an independent quality control review of this report.

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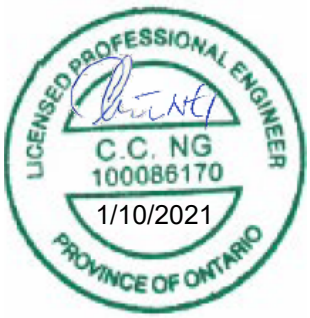

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PART B

FOUNDATION DESIGN REPORT
DON RIVER EAST BRANCH BRIDGE (SITE NO. 37X-0207/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 the proposed Don River East Branch Bridge (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. 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 in the Foundation Investigation Report (Part A of this report) as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Based on the General Arrangement (GA) drawing provided by AECOM, dated July 29, 2021, the proposed Don River East Branch Bridge will be a three-span structure located immediately south of the existing Eastbound Collector bridge over the Don River. Based on extensive design iterations between the structural and geotechnical team, the new Don River Bridge is an integral abutment, integral pier structural system, with piles supporting the abutments and drilled shafts supporting the piers. EPS is to be used behind each abutment. The proposed bridge will have a total length of 126 m and a width of about 34 m and 26 m at the west and east abutments, respectively, and will accommodate 3 lanes of eastbound traffic, plus the speed change lane associated with the S-E on-ramp. The proposed grade at the west and east abutments will be at approximately Elevations 139.2 m and 135.3 m, respectively, matching the existing highway grade.

The base of the river channel is at approximately Elevation 122 m at the proposed bridge location, and the high-water level at this location in the Don River East Branch, as provided by AECOM, is at about Elevation 126.5 m. A bench (level area) is present on the existing valley slopes at approximately Elevation 129 m, in the vicinity of the existing Piers #1 and #4.

From the 1954/1955 Department of Highways Ontario drawings, the original bridges over the East Don River were constructed as part of the Toronto Bypass project. The original bridges are five-span structures, in which the abutments are supported on H-piles (denoted as 14BP73) and the piers are supported on 0.53 m inner diameter Franki piles. The original bridges, which now carry the core lanes, have a total width of about 35.7 m, supporting three lanes of traffic and two shoulders in each of the westbound and eastbound directions. Based on the available drawings, the H-piles at the abutments were driven into the very dense till to about Elevation 106 m while the Franki piles at the piers were installed in compact to dense sand to about Elevation 114 m.

In 1964, drawings from the Department of Highways Ontario show the Don River Bridge was widened to the north (westbound collector) and south (eastbound collector) of the original bridge structure, with five-span structures matching the design of the original bridges. The eastbound collector bridge has a width of 29 m and was supported on H-piles and steel-encased drilled shafts (caissons). The existing foundations for the west and east abutments appear to have been designed to be supported in very dense till at approximately Elevation 106.7 m. Piers #1 to #4 appear to have been designed to be supported in very dense silty sand, around Elevation 116.7 m.

6.2 Foundation Options

Based on the proposed structure configuration and the subsurface conditions encountered at this 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. Consideration was given to the increased structural loads due to longer span lengths as compared to the existing bridges, and the compatibility of foundation types for the existing and future replacement bridges over East Don River. Based on these considerations and the risks associated with this option, no further discussion and recommendations are provided on spread/strip footings on native soils.
- **Driven steel H-piles:** Steel HP 310x110 piles driven to found with silty clay to sandy gravel till or 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. 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.

A summary of the advantages, disadvantages and risks for each foundation option, from a foundations perspective, is presented in Table 1 following the text of this report.

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 on the surface of or slightly into the bedrock.

6.3 Design Considerations

6.3.1 Consequence 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 typical 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, 2014* 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, $\overline{N}_{60}$ 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
$S_a(1.0)$ (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 the 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
a$S(0.2)$ (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.

The founding soils predominantly consist of firm to very stiff clay to sandy clayey silt-silt, compact to very dense silt to sand, and very dense/hard glacial till. 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 glacial till or 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 shoes 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 piles may also be terminated higher within the "100-blow" till deposit, although lower geotechnical resistances would apply. Based on the relatively limited thickness of 100-blow till in some boreholes, it is anticipated that the majority of piles should reach the shale bedrock. The following ranges in pile tip elevations may be used for design.

Foundation Element	Founding Stratum	Elevation at Underside of Pile Cap (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Length (m)
West Abutment	Silty Sand to Sand Till	134.6[1]	104.2	30.4
	Shale bedrock	134.6[1]	101.3	33.3
West Pier	Silty Sand to Sandy Gravel Till	127.8[2]	102.9	24.9
	Shale bedrock	127.8[2]	100.4	27.4
East Pier	Sandy Gravelly Clayey Silt Till	127.8[2]	102.9	24.9
	Shale bedrock	125.7[2]	99.8	25.9
East Abutment	Silty Clay Till	130.8[1]	102.0	28.8
	Shale bedrock	130.8[1]	99.8	31.0

Notes:
1. As per AECOM's General Arrangement drawing dated July 2021.
2. Located at least 1.2 m below finished 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	Pile Tip Elevation (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) (kN)
West Abutment	104.2	Silty Sand to Sand Till	1,700	--[1]
	101.3	Shale bedrock	2,250	--[1]
West Pier	101.9	Silty Sand to Sandy Gravel Till	1,700	--[1]

Foundation Element	Pile Tip Elevation (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) (kN)
	100.4	Shale bedrock	2,250	--[1]
East Pier	102.9	Sandy Gravelly Clayey Silt Till	1,700	--[1]
	99.8	Shale bedrock	2,250	--[1]
East Abutment	103.0	Silty Clay Till	1,700	--[1]
	99.8	Shale bedrock	2,250	--[1]

Notes:

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.

All pile installation/driving should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*) as amended by Special Provision 109F57. As the selected design option incorporates piles driven to bedrock, the following note should be included on the General Arrangement drawing:

- "Piles to be driven to bedrock."

However, if piles were to be founded in glacial till, the pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. The pile capacity should then be verified in the field using the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or wave equation analysis during pile installation on selected piles to confirm the geotechnical resistance of the piles.

The GA drawing shows the proposed abutment piles in relatively close proximity to the existing piles supporting the existing abutments. Based on the distance between the new abutments and the existing piles, there appears to be a low risk of interference between the new and existing piles.

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 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	100.3
West Pier	Shale bedrock	99.4
East Pier	Shale bedrock	98.8
East Abutment	Shale bedrock	98.8

6.5.2 Geotechnical Axial Resistances

The following provides the recommended factored ultimate geotechnical resistances for drilled shafts socketed 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 (kN)
All foundation elements	1.2	Shale bedrock	9,300	--[1]
	1.5		14,500	

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 shaft and tip resistances. 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 to ensure 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 or permanent 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 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 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.

6.7 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 4. 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' (º)	Initial Modulus of Subgrade Reaction (kPa/m^3)	Soil Model
West Abutment							
Loose Sand in CSP	134.6 - 129.6	19	--	--	28	6,800	API Sand, Above Water Table
Loose to Very Dense Sandy Silt to Silty Sand	129.6 - 128.3	20	--	--	32	24,400	API Sand, Below Water Table
Firm to Stiff Sandy Clayey Silt	128.3 - 124.5	17	55	0.007	--	--	Modified Stiff Clay Without Free Water
Loose to Very Dense Sandy Silt to Silty Sand	124.5 - 121.4	20	--	--	32	24,400	API Sand, Below Water Table
Soft to Hard Clayey Silt to Sandy Clayey Silt	121.4 - 115.6	17	70	0.007	--	--	Modified Stiff Clay Without Free Water

Soil Unit	Elevation (m)	Unit Weight, γ (kN/m³)	Undrained Shear Strength, s_u (kPa)	Strain Factor, ε_{50}	Effective Angle of Internal Friction, φ' (°)	Initial Modulus of Subgrade Reaction (kPa/m³)	Soil Model
Compact to Very Dense Sandy Silt to Silty Sand	115.6 - 106.2	20	--	--	36	34,000	API Sand, Below Water Table
Compact to Very Dense Sand to Silty Sand Till	106.2 - 101.3	21	--	--	37	37,000	API Sand, Below Water Table
West Pier							
Loose to Dense Sandy Silt to Silty Sand	128.4 - 127.7	20	--	--	35	61,000	API Sand, Above Water Table
	127.7 - 122.1					34,000	API Sand, Below Water Table
Soft to Stiff Silty Clay with Sand to Sandy Silty Clay	122.1 - 116.4	17	55	0.007	--	--	Modified Stiff Clay Without Free Water
Compact to Very Dense Sand	116.4 - 103.9	20	--	--	36	34,000	API Sand, Below Water Table
Non-Cohesive Till	103.9 - 100.4	21	--	--	37	37,000	API Sand, Below Water Table
East Pier							
Very Soft to Stiff Clayey Silt to Clay	124.6 - 118	17	20	0.01	--	--	Soft Clay
Firm to Hard Clayey Silt to Clay	118 - 113.2	17	150	0.005	--	--	Modified Stiff Clay Without Free Water
Compact to Dense Silt to Sandy Silt to Silty Sand	113.2 - 105.1	20	--	--	35	34,000	API Sand, Below Water Table
Very Dense/Hard Till and Residual Soil	105.1 - 100.3	20	150	0.005	--	--	Modified Stiff Clay Without Free Water
East Abutment							
Loose Sand in CSP	130.8 - 127.8	19	--	--	28	6,800	API Sand, Above Water Table
Cohesive Fill	127.8 - 122.8	18	30	0.007	--	--	Modified Stiff Clay Without Free Water
Very Soft to Stiff Organic silt / Silty Clay and Sand to Clayey Silt	122.8 - 119.1	17	20	0.01	--	--	Soft Clay, Below Water Table
Loose Silty Sand	119.1 - 116	20	--	--	30	5,400	API Sand, Below Water Table
Hard Sandy Clayey Silt-Silt	116 - 114.4	17	150	0.005	--	--	Modified Stiff Clay Without Free Water

Soil Unit	Elevation (m)	Unit Weight, γ (kN/m³)	Undrained Shear Strength, s_u (kPa)	Strain Factor, ε_{50}	Effective Angle of Internal Friction, φ' (º)	Initial Modulus of Subgrade Reaction (kPa/m³)	Soil Model
Compact Silty Sand	114.4 - 109.8	20	--	--	32	16,300	API Sand, Below Water Table
Clayey Silt to Silty Clay	109.8 - 105.5	17	150	0.005	--	--	Modified Stiff Clay Without Free Water
Sand	105.5 - 99.8	20	--	--	35	16,300	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 P-y curves provided. For this structure, the lateral resistance has been taken as that corresponding to a horizonal 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 Ultimate Lateral Geotechnical Resistance (kN)	Factored Serviceability Lateral Geotechnical Resistance for 10 mm of Deflection (kN)
West Abutment	Steel HP 310x110 Pile	100	125
West Pier	1.5 m dia. Drilled Shaft	3250	2700
East Pier	1.5 m dia. Drilled Shaft	575	700
East Abutment	Steel HP 310x110 Pile	75	100

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.*

6.8 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 walls 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).

6.8.1 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 abutments:

Fill Type	Unit Weight of Material (kN/m³)	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 appliable 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.

6.9 Embankment Design

Design of the immediate approach embankments within 20 m of the west and east abutments for the Don River East Branch Bridge (i.e., from approximately Station 25+880 to 25+900 and Station 26+040 to 26+060, respectively) is discussed in this section. Recommendations regarding the high fill embankments that transition into these approach embankments, the South-East Ramp from Leslie Street and the embankment east of the Don River East Branch bridge which includes a proposed temporary access road, are discussed under a separate 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".

6.9.1 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 136 m and 126.5 m for the west and east abutments, respectively. Figure 5 presents the parameters associated with the cohesive deposit encountered at the site.

Stratigraphic Unit	γ (kN/m³)	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	--	--	--	--	--
Very Loose to Dense Clayey Sand to Sand and Gravel Fill	21	36	--	--	--	--	--	15
Soft to Hard Silty Clay to Sandy Clayey Silt Fill	21	28	50	--	--	--	--	30
Very Loose to Very Dense Sandy Silt to Silty Sand	20	35	--	--	--	--	--	15
Stiff to Very Stiff Sandy Clayey Silt Interlayer	21	--	75	--	--	--	--	30
Soft to Hard Clay to Sandy Clayey Silt	21	33	55	220	0.5	0.3	0.03	--
Very Loose to Very Dense Silt to Sand	20	36	--	--	--	--	--	20
Stiff to Hard Silty Clay to Clayey Silt Interlayer	21	--	200	--	--	--	--	100
Hard Sandy Gravelly Clayey Silt Till	22	--	200	--	--	--	--	150
Compact to Very Dense Silty Sand to Sandy Gravel Till	22	37	--	--	--	--	--	150
Hard Silty Clay to Sandy Gravelly Clayey Silt (Residual Soil)	21	--	200	--	--	--	--	150

6.9.2 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 analyses, 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 drawings, the approach embankment side slopes will be inclined at 2 horizontal to 1 vertical (2H:1V) with an overall height of about 8.5 m and 5.2 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 6 to 13 following the text of this report.

Foundation Element	Relevant Boreholes	Static Global Stability	Slope	Factor of Safety
West Abutment	DR-3, DRB-03	Short-term (undrained)	North-South	> 1.3
		Long-term (drained)		> 1.5
	DR-3, DR-5	Short-term (undrained)	West-East	> 1.3
		Long-term (drained)		> 1.5
East Abutment	DR-9, DRB-05	Short-term (undrained)	North-South	> 1.3
		Long-term (drained)		> 1.5
	DR-7, DR-9, DR-10	Short-term (undrained)	West-East	> 1.3
		Long-term (drained)		> 1.5

6.9.3 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 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 granular fill is presented below.

Foundation Investigation Area Designation	Foundation Investigation Area Limits	Proposed Maximum Embankment Height [1]	Settlement, δ[2]	
West Approach Embankment	Station 25+880 to Station 25+908	8.5 m	$\delta_{Immediate}$ =	140 mm
			$\delta_{Primary}$ =	120 mm
			$\delta_{Secondary}$ =	140 mm
			δ_{Total} =	400 mm
East Approach Embankment	Station 26+034 to Station 26+060	5.5 m	$\delta_{Immediate}$ =	180 mm
			$\delta_{Primary}$ =	20 mm
			$\delta_{Secondary}$ =	30 mm
			δ_{Total} =	230 mm

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 May 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.

6.9.3.1 Mitigation Options

Several settlement mitigation options have been considered to meet requirement of the settlement performance criterion and a brief discussion on these alternatives are 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 are not discussed further.

- **Preloading:** Preloading is the base option for mitigated post-construction and is feasible for embankments at the Don River bridge where relatively thin compressible cohesive deposits are present prior to final grading

and paving. Thinner cohesive deposits promote a higher rate of consolidation due to the short drainage paths. However, where cohesive deposits are thick, preloading may require an extended period of time to reach the settlement performance criterion. Another disadvantage with preloading is that there may not be adequate time during construction to induce sufficient secondary (creep) consolidation to occur and may potentially result in long-term settlement of the embankment. If large magnitudes of settlement are expected, there will be risk that the settlement will affect the serviceability of adjacent/nearby bridge structures and underground utilities. In addition, an instrumentation and monitoring will likely be required during construction to assess when the settlement performance criterion has been achieved.

- **Full Sub-Excavation:** Although full sub-excavation of the compressible cohesive deposits underlying the embankment will greatly improve post-construction settlement, it is not considered a feasible alternative given the depth to the bottom of the cohesive deposit (up to about 23 m below ground surface) and that an extensive temporary protection system will be required along existing highway embankments and structures to maintain the safe operation of Highway 401 during the sub-excavation operation.
- **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 preload, surcharge, or subexcavation to compensate for embankment loading added 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 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 fill (EPS) is considered feasible alternatives to mitigate long-term post-construction settlement at this site. Full sub-excavation is not discussed further herein due to their inherent disadvantages outweighing the advantages of the alternative offers.

6.9.3.2 Preloading

Based on the estimated coefficient of consolidation (c_v) of about 7.6 x 10^{-3} cm^2/s and a modified secondary compression index ($c_{\alpha\varepsilon}$) of 1.1x10^{-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	Critical Section	Height of Embankment (m)	Estimated Preload Period (days)
West Approach Embankment	Station 25+880 to Station 25+908	5.9	700
East Approach Embankment	Station 26+034 to Station 26+060	9.7	30

6.9.3.3 Lightweight Fill (Expanded Polystyrene)

It is understood that the cover materials will consist of approximately 90 mm of asphalt over 250 mm of reinforced concrete and 250 mm of Granular 'A' subbase. In addition, a 175 mm thick layer 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 up to a 1.0 m deep 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 load.

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 of thicknesses designed to meet settlement criteria. The combined thickness of all cover material above the EPS and on the side slopes is approximately 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.

Embankment	Critical Section	Height of Approach Embankment (m)	Minimum Thickness of EPS (m)	Depth of Sub-excavation	Estimated Magnitude of Settlement over a 20-Year Period (mm)
West Approach Embankment	Station 25+880 to 25+908	5.9	Up to 6.4	Minimum 0.5 m below ground surface or from ground surface to Elevation 134.9 m, whichever is greater	$\delta_{Immediate}$ = 0 $\delta_{primary}$ = 20 $\delta_{secondary}$ = 0 δ_{Total} = 20
East Approach Embankment	Station 26+034 to 26+060	9.7	Up to 9.7	Minimum 1.0 m below ground surface or from ground surface to Elevation 130.4 m, whichever is greater	$\delta_{Immediate}$ = 0 $\delta_{primary}$ = 20 $\delta_{secondary}$ = 0 δ_{Total} = 20

Based on the estimated magnitude of settlement for EPS embankments, 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.

6.10 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 approach embankments are 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	Minimum 0.5 m below existing ground surface or from ground surface to Elevation 134.9 m, whichever is greater, to be replaced with EPS fill	2H:1V	Not Required	$\delta_{Immediate}$ = 0 $\delta_{primary}$ = 10 $\delta_{secondary}$ = 5 δ_{Total} = 15
East Approach Embankment	25	EPS Fill	Minimum 1.0 m below ground surface or from ground surface to Elevation 130.4 m, whichever is greater, to be replaced with EPS fill	2H:1V	Not Required	$\delta_{Immediate}$ = 0 $\delta_{primary}$ = 10 $\delta_{secondary}$ = 5 δ_{Total} = 15

6.11 Analytical Testing of Construction Materials

The results of analytical tests carried out on four soil samples 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 0.04%, 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 7.5 to about 7.8. 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 four soil samples ranges from 1,200 ohm-cm to 3,200 ohm-cm which indicates that the soil corrosiveness is moderate (4,500 > R > 2,000) 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.

6.12 Construction Considerations

6.12.1 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 silty sand and native clay to clayey silt deposits. The topsoil and any organic / deleterious materials encountered within the footprint of the proposed foundation elements and approach embankments 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.

Temporary excavations should be observed and reviewed during construction to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, a geotechnical engineer should review the excavation plan considering the conditions at that time.

6.12.2 Subgrade Preparation and Embankment Construction

The EPS fill should be installed in accordance with the Non-Standard Special Provision (NSSP) for Rigid Expanded Polystyrene Embankment Fill is 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 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 125 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. Along the longitudinal length of the N-E Ramp and the Highway 401 EBC embankment widening between Leslie Street Overpass and Don River East Branch Bridge, it is recommended that lightweight fill material be placed at a stepped slope of 5 horizontal to 1 vertical (5H:1V) at the transition between the EPS and the granular fill embankment so that any resulting settlement would be gradual.

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). Side slopes should be 2 horizontal to 1 vertical (2H:1V) or flatter. 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.

6.12.3 Erosion Protection

Provision should be made for erosion protection on the embankment fore slopes and riverbanks in the vicinity of the proposed pier locations. The requirements for the extent and design of erosion protection measures along the

riverbanks in the area surrounding the proposed piers should be assessed by the hydraulics design engineer. As a minimum, rip rap (or rock protection) treatment along the riverbanks and fore slopes should be consistent with the requirements of OPSS.PROV 1004 and the gradation (i.e., R-10, R-50, or Rock Protection) and minimum thickness should be determined by the hydraulics engineer. It is recommended that a non-woven geotextile, properly lapped in the direction of river flow, be placed below the rip-rap or rock protection to act as a separator to prevent the migration of fine-grained materials through the coarser grained 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)*.

6.12.4 Control of Groundwater

The groundwater level measured in the piezometers west and east of Don River were at about Elevations 123.5 m and 124.8 m in October 2019 and May 2020, respectively. However, it is noted that the groundwater level could be higher during periods during heavy/sustained precipitation during the wet seasons.

It is anticipated that foundation excavations for pile caps at the piers and east abutment 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 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.

6.12.5 Temporary Protection Systems

Temporary protection systems are expected to be required to facilitate the staged construction of the abutments, 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.

6.12.6 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.

6.12.7 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 Enbridge pipeline and 1.4 m diameter sanitary sewer): 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.

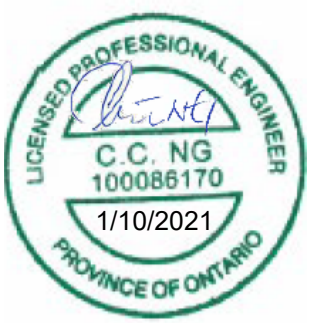
6.13 Piezometer Decommissioning

A standpipe piezometer was installed in Boreholes DR-5 and DR-7 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. The standpipe piezometer in Boreholes DR-5 and DR-7 should then be abandoned under the Construction Contract work; a NSSP for this item is included in Appendix D.

7.0 CLOSURE

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

Golder Associates Ltd.



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KN/CN/LCC/ml/ack

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https://golderassociates.sharepoint.com/sites/20393g/deliverables/foundations/3 - don river bridge/4. final/1786302-fidr-highway 401 ebc-don river east branch bridge_october 1, 2021.docx

REFERENCES

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

Canadian Standard Association (CSA) Group. *Canadian Highway Bridge Design Code* (CHBDC (2019)) and Commentary on CAN/CSA-S6-14.

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.

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Ministry of Transportation, Ontario. 2004. *Gravity Pipe Design Guidelines*.

Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction. *Geotechnique*, Vol. 5, No. 4, pp. 297-326. Discussion in Vol. 6, No. 2, pp. 94-98.

Unified Facilities Criteria, U.S. Navy. 1986. *NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures.* Alexandria, Virginia.

Westergaard, H.M., 1938. A problem of elasticity suggested by a problem in soil mechanics: A soft material reinforced by numerous strong horizontal sheets. In *Contributions to the Mechanics of Solids*, Stephen Timoshenko 60th Anniversary Volume, MacMillan, New York, pp. 260-277.

Commercial Software:

Settle3 (Version 9.0) by Rocscience Inc.

Slide2 (Version 9.0) by Rocscience Inc.

RS2 (Version 10.0) by Rocscience Inc.

RSPile (Version 3.0) by Rocscience Inc.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213	Construction Projects (as amended)
Ontario Regulation 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 – Don River East Branch Bridge

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Driven Steel H-piles (HP 310x110) driven into silty clay to sandy gravel till or to bedrock	■ Feasible for all foundation elements	■ Conventional construction methods for H-pile foundations. ■ Allow for integral abutment design	■ Dewatering measures may be required at abutments and pier for the construction of pile caps. ■ Requires driving shoes due to potential presence of cobbles and boulders within the till deposits.	■ Lower relative cost than drilled shafts (caissons).	■ Risk of damage to piles due to cobbles and boulders that maybe present within the till deposits.
1.2 m or 1.5 m Diameter Drilled Shafts (Caissons) Socketed 1 m into bedrock	■ Feasible for all foundation elements	■ 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.	■ Casings will be required, plus special measures such as use of slurry to counterbalance groundwater pressures and minimize ground disturbance and use of tremie methods for concrete placement. ■ Generation of soil and rock cuttings during drilled shaft advancement. ■ Challenges associated with inspection of shaft base due to presence of slurry. ■ Does not allow for integral abutment design if used at the abutments.	■ Higher relative cost than driven piles.	■ 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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METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

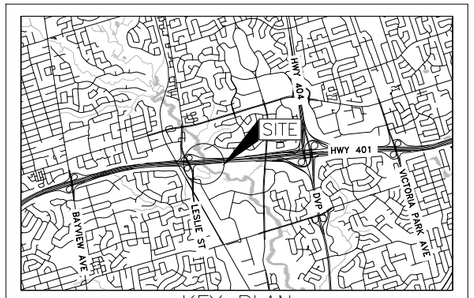
CONT No.
GWP No. 2130-01-00

SHEET

HIGHWAY 401 EB COLLECTORS
DON RIVER EAST BRANCH BRIDGE
(SITE NO. 37X-0207/B1)
BOREHOLE LOCATIONS

GOLDER
MEMBER OF WSP

PLAN
SCALE
10 0 10 20 m



KEY PLAN
SCALE
1 0 1 2 km

LEGEND
- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 30M14-462)
- Cone Penetration Test (CPT)

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)

No.	ELEVATION	NORTHING	EASTING
DR-1	138.5	4847342.6	316023.0
DR-2a/b	130.1	4847305.5	316039.1
DR-3	138.5	4847343.0	316026.2
DR-4	129.3	4847328.0	316067.6
DR-5	129.1	4847350.9	316071.6
DR-6	128.0	4847329.4	316137.5
DR-7	129.3	4847347.1	316136.0
DR-8a	129.4	4847336.7	316160.8
DR-8b	129.4	4847337.4	316158.6
DR-9	134.5	4847356.9	316164.9
DR-10	134.4	4847357.5	316171.2
DRB-03	130.2	4847311.3	316033.9
DRB-04	129.2	4847327.5	316072.7
DRB-05	129.4	4847331.7	316157.9

REFERENCE
Base 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 plan and profile, as well as proposed and existing ground profiles provided in digital format by AECOM, drawing file no. Don River Bridge_GA.dwg, received July 28, 2021. DRB-03 to DRB-05 boreholes from Foundation Investigation Report Don River Bridge at Highway 401 East of Leslie Street Interchange Toronto, Ontario, by Thurber Engineering Ltd., dated September 5, 2017.

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.

LICENSED PROFESSIONAL ENGINEER
L.C. COYNE
Sept. 17, 2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
C.C. NG
100086170
Sept. 17, 2021
PROVINCE OF ONTARIO

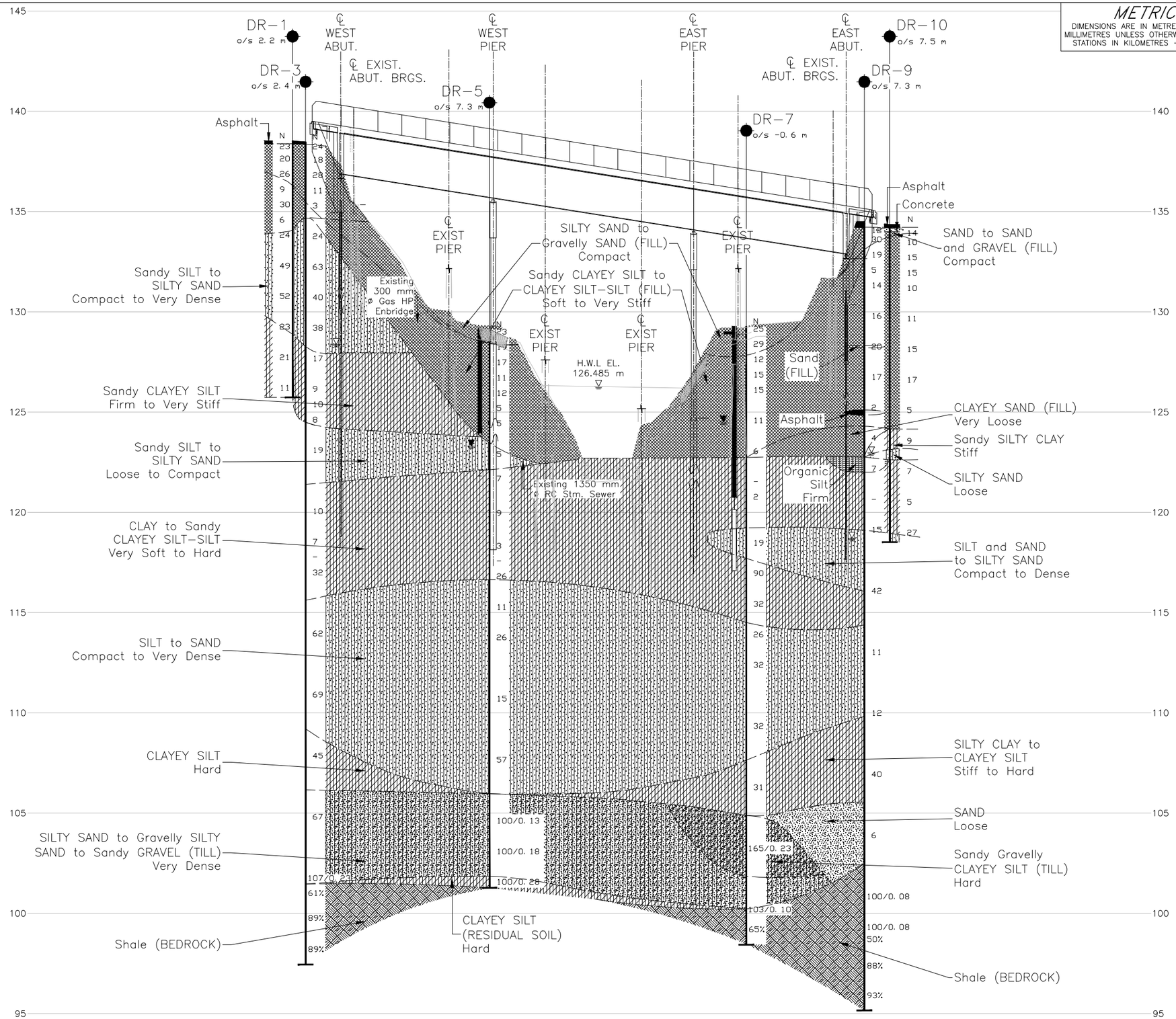
NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 30M14-331

HWY. 401		PROJECT NO. 1786302	DIST. CENTRAL
SUBM'D. DH	CHKD. KN	DATE: 09/17/2021	SITE: 37-307/1
DRAWN: DD/SA/TR	CHKD. CN	APPD. LCC	DWG. 1

PLOT DATE: September 17, 2021
FILENAME: S:\Clients\MTO\Hwy_401_Leslie_Street\99_PROJ\1786302\40_PROD\0004_EB_Collector_Bridge_Don_River\1786302-0004-BG-0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO



A-A' 1 SECTION A-A' – DON RIVER OVERPASS PROFILE

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
DON RIVER EAST BRANCH BRIDGE
(SITE NO. 37X-0207/B1)
SOIL STRATA

SHEET

GOLDER
MEMBER OF WSP

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)

No.	ELEVATION	NORTHING	EASTING
DR-1	138.5	4847342.6	316023.0
DR-3	138.5	4847343.0	316026.2
DR-5	129.1	4847350.9	316071.6
DR-7	129.3	4847347.1	316136.0
DR-9	134.5	4847356.9	316164.9
DR-10	134.4	4847357.5	316171.2

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Sept. 17, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.C. NG 100086170 Sep. 17, 2021 PROVINCE OF ONTARIO

REFERENCE

Base 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 profile provided in digital format by AECOM, drawing file no. X_60572506_Don River_PLAN ELEVATION SECTION.dwg, received December 2, 2020. General arrangement plan, as well as proposed and existing ground profiles provided in digital format by AECOM, drawing file no. Don River Bridge_GA.dwg, received July 28, 2021. DRB-03 to DRB-05 boreholes from Foundation Investigation Report Don River Bridge at Highway 401 East of Leslie Street Interchange Toronto, Ontario, by Thurber Engineering Ltd., dated September 5, 2017.

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NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 30M14-331

HWY. 401	PROJECT NO. 1786302		DIST. CENTRAL
SUBM'D. DH	CHKD. KN	DATE: 09/17/2021	SITE: 37-307/1
DRAWN: TR/SA	CHKD. CN	APPD. LCC	DWG. 2

PLOT DATE: September 17, 2021
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MINISTRY OF TRANSPORTATION, ONTARIO

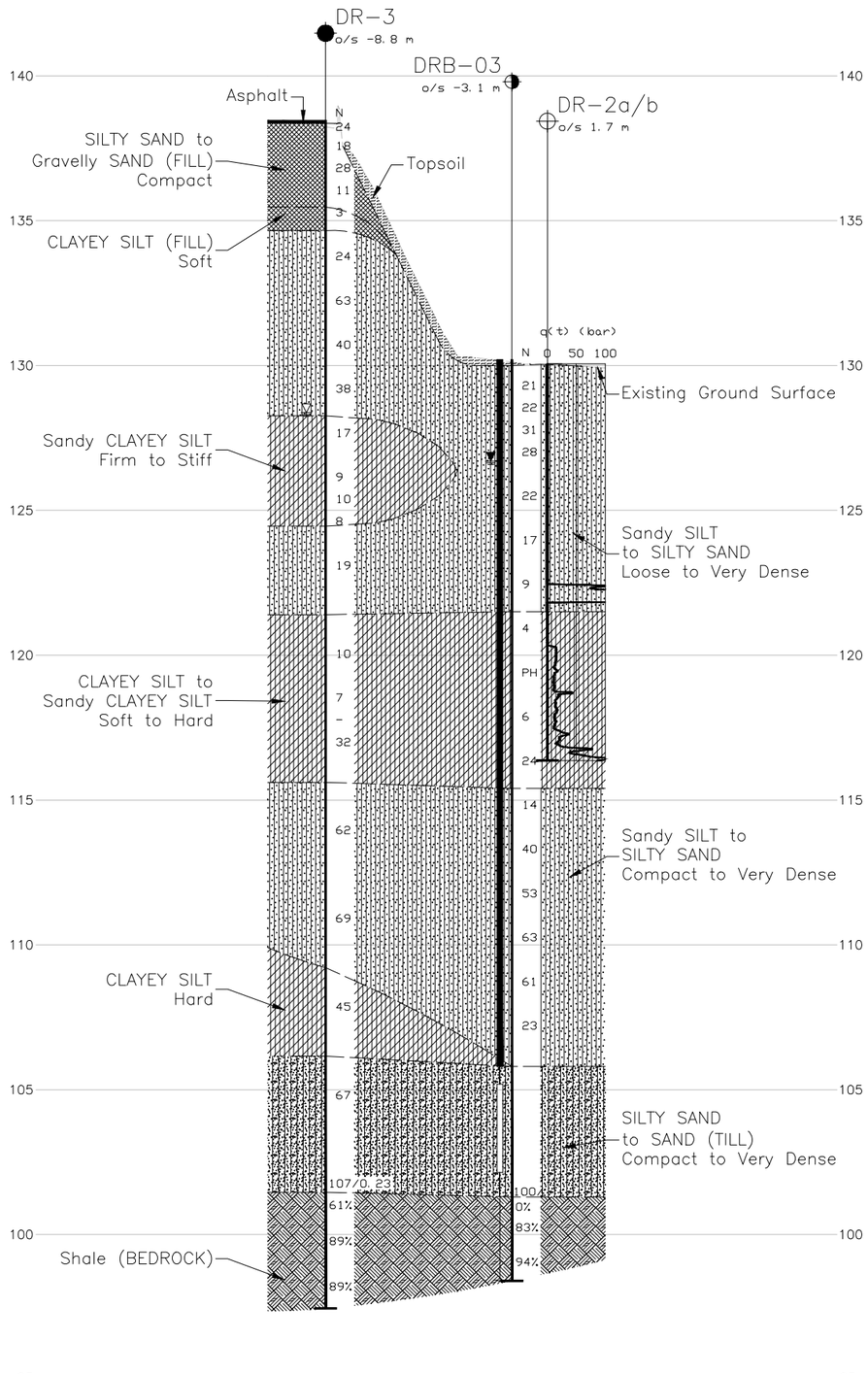
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2130-01-00

HIGHWAY 401 EB COLLECTORS
DON RIVER EAST BRANCH BRIDGE
(SITE NO. 37X-0207/B1)
SOIL STRATA

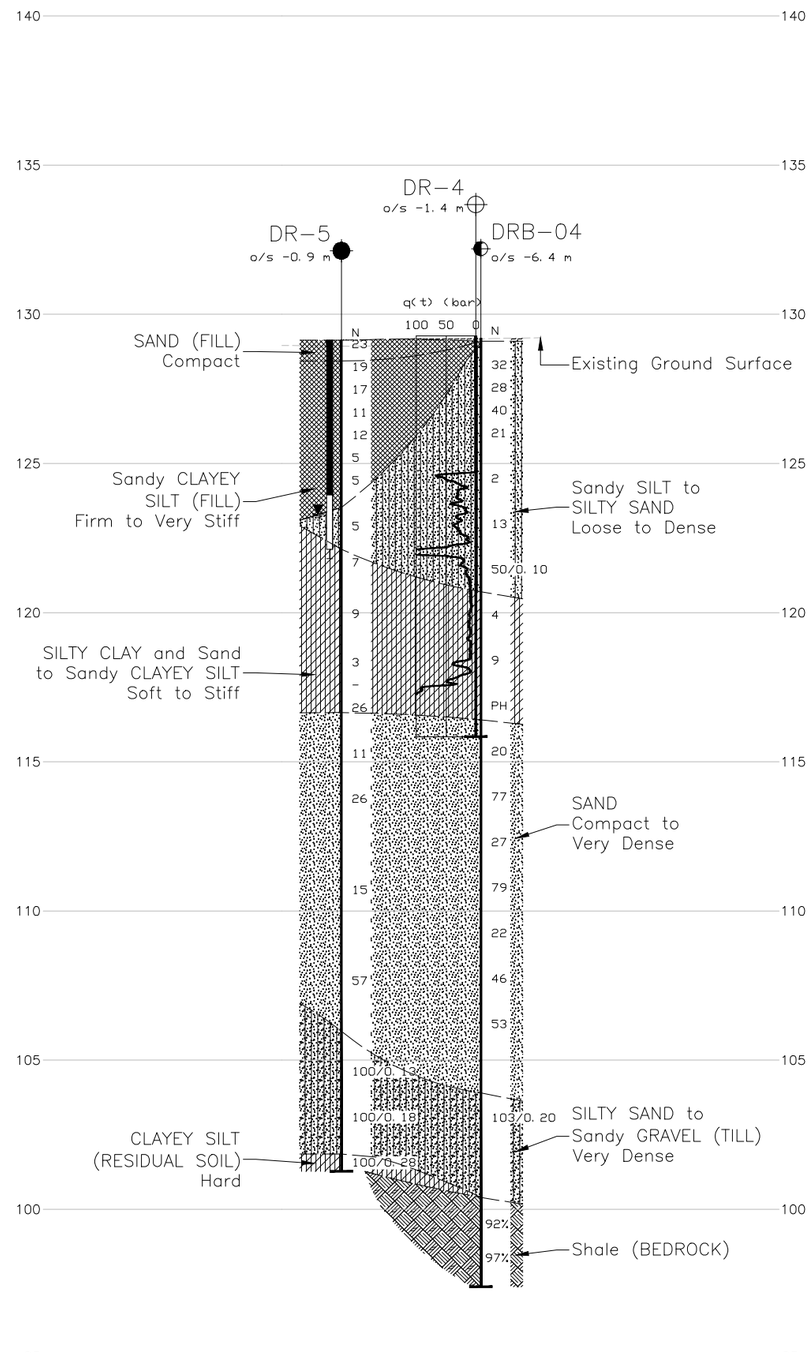
SHEET

GOLDER
MEMBER OF WSP



B-B' 1 SECTION B-B' – WEST ABUTMENT SECTION

HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 2 0 2 4 m



C-C' 1 SECTION C-C' – WEST PIER SECTION

HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 30M14-462)
- Dynamic Cone Penetration Test
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- q(t) — CPT Corrected Tip Resistance (bar)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)

No.	ELEVATION	NORTHING	EASTING
DR-2a/b	130.1	4847305.5	316039.1
DR-3	138.5	4847343.0	316026.2
DR-4	129.3	4847328.0	316067.6
DR-5	129.1	4847350.9	316071.6
DRB-03	130.2	4847311.3	316033.9
DRB-04	129.2	4847327.5	316072.7

LICENSED PROFESSIONAL ENGINEER L.L.C. COYNE Sept. 17, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.C. NG 100086170 Sep. 17, 2021 PROVINCE OF ONTARIO

REFERENCE

Base 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 plan and profile, as well as proposed and existing ground profiles provided in digital format by AECOM, drawing file no. S13 GENERAL ARRANGEMENT.dwg, received March 24, 2020. DRB-03 to DRB-05 boreholes from Foundation Investigation Report Don River Bridge at Highway 401 East of Leslie Street Interchange Toronto, Ontario, by Thurber Engineering Ltd., dated September 5, 2017.

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. 30M14-331

HWY. 401		PROJECT NO. 1786302	DIST. CENTRAL
SUBM'D. DH	CHKD. KN	DATE: 09/17/2021	SITE: 37-307/1
DRAWN: TR	CHKD. CN	APPD. LCC	DWG. 3

PLOT DATE: September 17, 2021
FILENAME: S:\Clients\MTO\Hwy_401_Leslie_Street\99_PROJ\1786302\40_PROD\0004_EB_Collector_Bridge_Don_River\1786302-0004-BG-0003.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

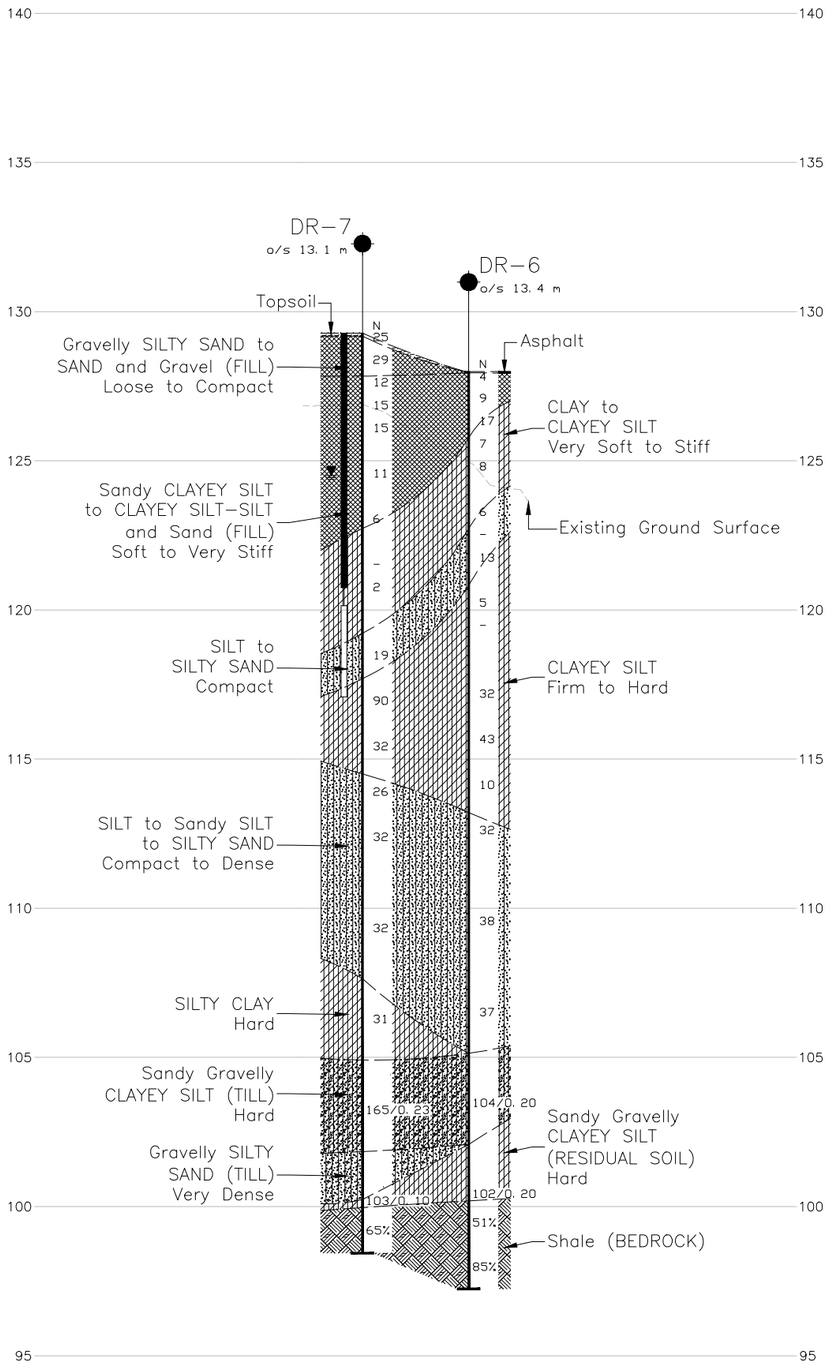
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2130-01-00

HIGHWAY 401 EB COLLECTORS
DON RIVER EAST BRANCH BRIDGE
(SITE NO. 37X-0207/B1)
SOIL STRATA

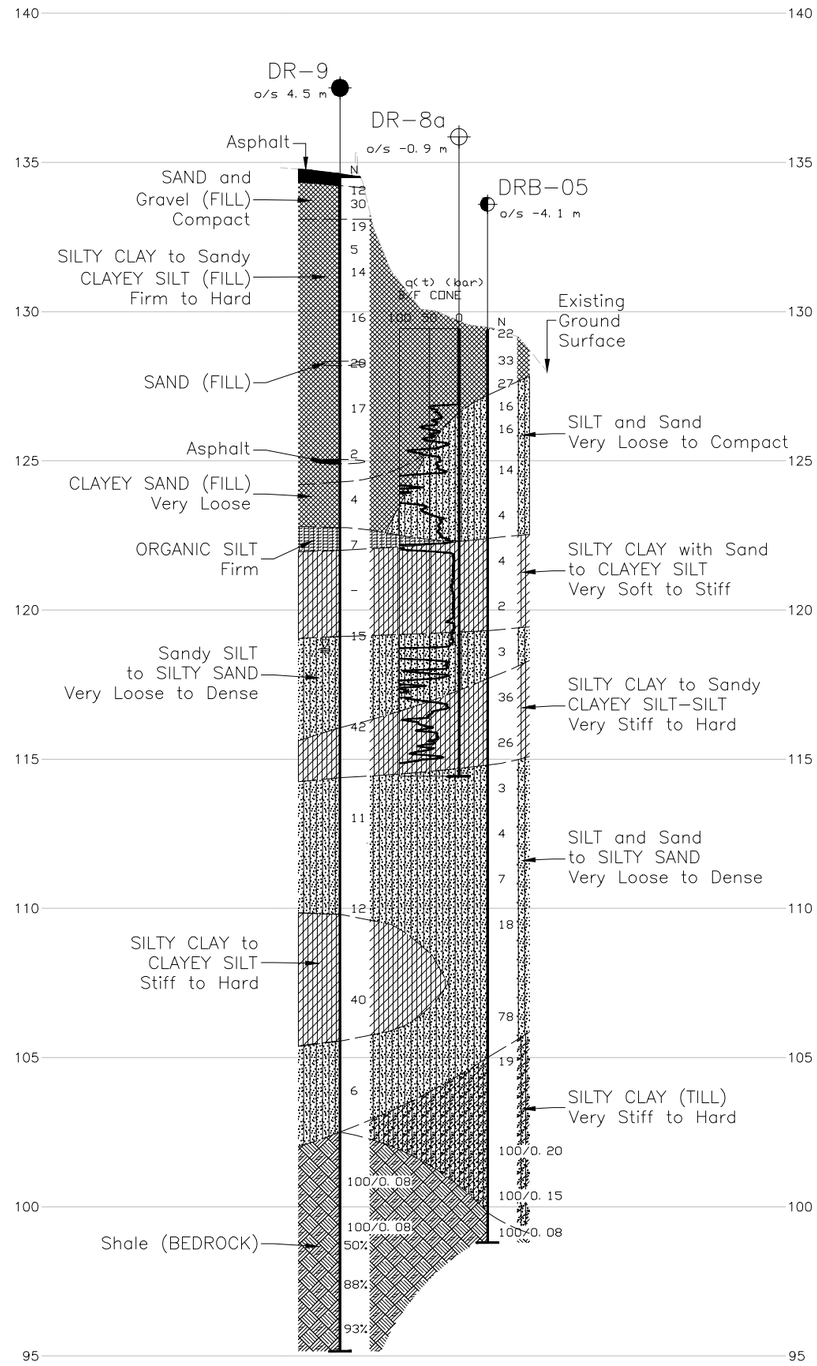
SHEET

GOLDER
MEMBER OF WSP



D-D' 1 SECTION D-D' – EAST PIER SECTION

HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 2 0 2 4 m



E-E' 1 SECTION E-E' – EAST ABUTMENT SECTION

HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 2 0 2 4 m

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 30M14-462)
- Dynamic Cone Penetration Test
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- q(t) — CPT Corrected Tip Resistance (bar)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)

No.	ELEVATION	NORTHING	EASTING
DR-6	128.0	4847329.4	316137.5
DR-7	129.3	4847347.1	316136.0
DR-8a	129.4	4847336.7	316160.8
DR-9	134.5	4847356.9	316164.9
DRB-05	129.4	4847331.7	316157.9

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Sept. 17, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.C. NG 100086170 Sep. 17, 2021 PROVINCE OF ONTARIO

REFERENCE

Base 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 plan and profile, as well as proposed and existing ground profiles provided in digital format by AECOM, drawing file no. S13 GENERAL ARRANGEMENT.dwg, received March 24, 2020. DRB-03 to DRB-05 boreholes from Foundation Investigation Report Don River Bridge at Highway 401 East of Leslie Street Interchange Toronto, Ontario, by Thurber Engineering Ltd., dated September 5, 2017.

NOTES

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NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 30M14-331

HWY. 401		PROJECT NO. 1786302	DIST. CENTRAL
SUBM'D. DH	CHKD. KN	DATE: 09/17/2021	SITE: 37-307/1
DRAWN: TR	CHKD. CN	APPD. LCC	DWG. 4

PLOT DATE: September 17, 2021
FILENAME: S:\Clients\MTO\Hwy_401_Leslie_Street\99_PROJ\1786302\45_PROD\0004_EB_Collector_Bridge_Don_River\1786302-0004-BD-0004.dwg

Figures

Summary of Load-Deflection (P-y Curves) for HP 310x110 at West Abutment

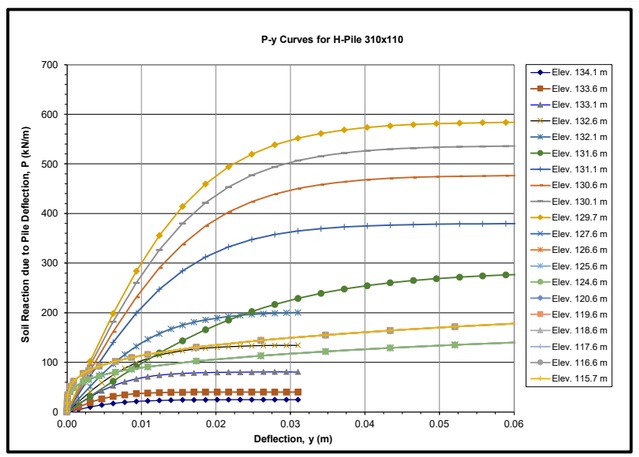
Figure 1

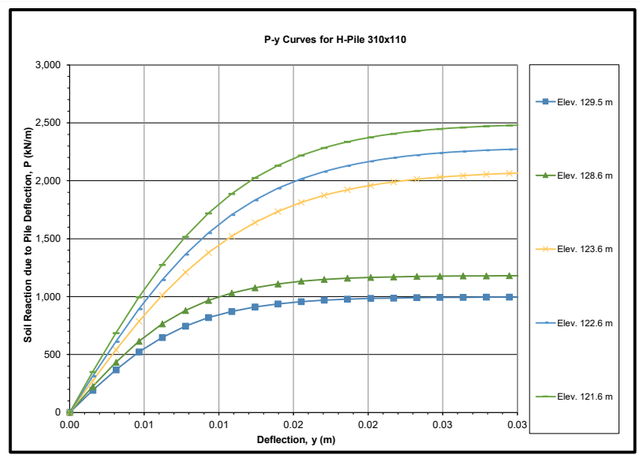
Description	Loose Sand in CSP																				Loose to Very Dense Sandy Silt to Silty Sand			
Depth (z) *	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 4.9 m		z= 5.1 m		z= 6.0 m	
Elevation	Elev. 134.1 m		Elev. 133.6 m		Elev. 133.1 m		Elev. 132.6 m		Elev. 132.1 m		Elev. 131.6 m		Elev. 131.1 m		Elev. 130.6 m		Elev. 130.1 m		Elev. 129.7 m		Elev. 129.5 m		Elev. 128.6 m	
P-y Curves	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
	0.00155	5.1919	0.00155	10.303688	0.00155	15.61059	0.00155	20.909576	0.00155	26.203	0.00155	31.489977	0.0031	72.865175	0.0031	83.454583	0.0031	93.8851657	0.0031	102.23293	0.00155	190.51154	0.00155	224.16538
	0.0031	9.94716	0.0031	19.33337	0.0031	30.09437	0.0031	40.838542	0.0031	51.546	0.0031	62.217147	0.0062	140.55513	0.0062	161.96119	0.0062	182.204383	0.0062	198.40391	0.0031	367.57308	0.0031	432.75077
	0.00465	13.9652	0.00465	26.380157	0.00465	42.63035	0.00465	58.981694	0.00465	75.278	0.00465	91.491506	0.0093	199.26667	0.0093	231.67884	0.0093	260.637684	0.0093	283.80615	0.00465	521.30615	0.00465	614.21154
	0.0062	17.1366	0.0062	31.401366	0.0062	52.84473	0.0062	74.824038	0.0062	96.839	0.0062	118.75425	0.0124	247.2402	0.0124	290.50855	0.0124	326.817095	0.0124	355.86367	0.0062	647.07231	0.0062	763.07308
	0.00775	19.5088	0.00775	34.753409	0.00775	60.76852	0.00775	88.16079	0.00775	115.9	0.00775	143.60579	0.0155	284.56376	0.0155	338.01831	0.0155	380.274619	0.0155	414.07289	0.00775	745.05538	0.00775	879.42231
	0.0093	21.2126	0.0093	36.896891	0.0093	66.68354	0.0093	99.049367	0.0093	132.33	0.0093	165.82932	0.0186	312.50736	0.0186	375.07812	0.0186	421.956251	0.0186	459.4646	0.0093	818.51846	0.0093	966.96154
	0.01085	22.4012	0.01085	38.229806	0.01085	70.97639	0.01085	107.71798	0.01085	146.22	0.01085	185.36086	0.0217	332.83902	0.0217	403.18798	0.0217	453.577972	0.0217	493.89677	0.01085	872.03154	0.01085	1030.9154
	0.0124	23.2138	0.0124	39.044747	0.0124	74.02667	0.0124	114.47464	0.0124	157.74	0.0124	202.26041	0.0248	347.32475	0.0248	424.06388	0.0248	477.071759	0.0248	519.47821	0.0124	910.20077	0.0124	1076.7308
	0.01395	23.7616	0.01395	39.538307	0.01395	76.16278	0.01395	119.67036	0.01395	167.15	0.01395	216.69598	0.0279	357.49053	0.0279	439.3338	0.0279	494.251606	0.0279	538.18408	0.01395	937.02	0.01395	1109.0692
	0.0155	24.1276	0.0155	39.835281	0.0155	77.64371	0.0155	123.61212	0.0155	174.76	0.0155	228.89558	0.031	364.54637	0.031	450.37175	0.031	506.667498	0.031	551.7088	0.0155	955.66077	0.0155	1131.5923
	0.01705	24.371	0.01705	40.013064	0.01705	78.66286	0.01705	126.57792	0.01705	180.84	0.01705	239.09321	0.0341	369.41224	0.0341	458.28771	0.0341	515.575419	0.0341	561.40393	0.01705	968.52615	0.01705	1147.1692
	0.0186	24.5318	0.0186	40.120053	0.0186	79.36102	0.0186	128.78777	0.0186	185.66	0.0186	247.56088	0.0372	372.75015	0.0372	463.92968	0.0372	521.921363	0.0372	568.31309	0.0186	977.35769	0.0186	1157.9154
	0.02015	24.6378	0.02015	40.183446	0.02015	79.83759	0.02015	130.43764	0.02015	189.47	0.02015	254.53259	0.0403	375.03608	0.0403	467.93566	0.0403	526.429322	0.0403	573.21706	0.02015	983.39923	0.02015	1165.2615
	0.0217	24.708	0.0217	40.222042	0.0217	80.16237	0.0217	131.65154	0.0217	192.46	0.0217	260.25633	0.0434	376.59203	0.0434	470.77165	0.0434	529.609296	0.0434	576.68663	0.0217	987.51154	0.0217	1170.3769
	0.02325	24.754	0.02325	40.244839	0.02325	80.38336	0.02325	132.55347	0.02325	194.79	0.02325	264.9341	0.0465	377.64799	0.0465	472.76964	0.0465	531.861273	0.0465	579.14182	0.02325	990.35	0.02325	1173.7923
	0.0248	24.784	0.0248	40.258437	0.0248	80.53355	0.0248	133.21541	0.0248	196.61	0.0248	268.7299	0.0496	378.36797	0.0496	474.17963	0.0496	533.45126	0.0496	580.867	0.0248	992.23462	0.0248	1176.1077
	0.02635	24.8044	0.02635	40.266436	0.02635	80.63534	0.02635	133.70136	0.02635	198.02	0.02635	271.81173	0.0527	378.85795	0.0527	475.17362	0.0527	534.56325	0.0527	582.0878	0.02635	993.52308	0.02635	1177.7231
	0.0279	24.8174	0.0279	40.271435	0.0279	80.70493	0.0279	134.06333	0.0279	199.12	0.0279	274.30558	0.0558	379.19193	0.0558	475.87162	0.0558	535.349245	0.0558	582.93859	0.0279	994.40769	0.0279	1178.8231
	0.02945	24.8258	0.02945	40.274635	0.02945	80.75193	0.02945	134.3233	0.02945	199.97	0.02945	276.31346	0.0589	379.41992	0.0589	476.36362	0.0589	535.903237	0.0589	583.54579	0.02945	995	0.02945	1179.6231
	0.031	24.8312	0.031	40.276434	0.031	80.78393	0.031	134.51728	0.031	200.63	0.031	277.93535	0.062	379.56992	0.062	476.70962	0.062	536.291235	0.062	583.96939	0.031	995.46154	0.031	1180.1231

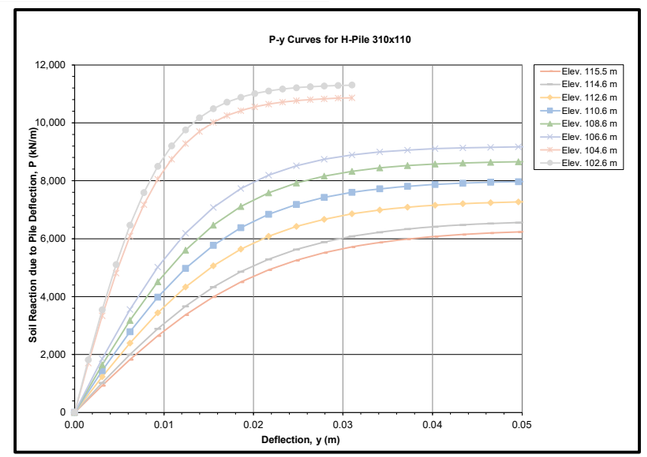
Description	Firm to Stiff Sandy Clayey Silt								Loose to Very Dense Sandy Silt to Silty Sand			
Depth (z) *	z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m	
Elevation	Elev. 127.6 m		Elev. 126.6 m		Elev. 125.6 m		Elev. 124.6 m		Elev. 123.6 m		Elev. 122.6 m	
P-y Curves	y (m)	P (kN/m)	y (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
	8.7E-07	2.241125	8.7E-07	0.9374197	8.7E-07	1.055	8.7E-07	1.171898	0.00155	274.2825	0.00155	311.7637
	4.3E-06	11.20375	4.3E-06	4.6875987	4.3E-06	5.273	4.3E-06	5.859401	0.0031	539.280174	0.0031	612.24526
	8.7E-06	15.34	8.7E-06	9.3741975	8.7E-06	10.55	8.7E-06	11.71898	0.00465	786.945	0.00465	891.73497
	4.3E-05	22.95	4.3E-05	22.95	4.3E-05	22.95	4.3E-05	22.95	0.0062	1011.26977	0.0062	1143.2838
	8.7E-05	27.29	8.7E-05	27.29	8.7E-05	27.29	8.7E-05	27.29	0.00775	1208.9314	0.00775	1362.9156
	0.00043	40.8	0.00043	40.8	0.00043	40.8	0.00043	40.8	0.0093	1378.68895	0.0093	1549.8197
	0.00087	48.53	0.00087	48.53	0.00087	48.53	0.00087	48.53	0.01085	1521.41047	0.01085	1705.3925
	0.00217	61.02	0.00217	61.02	0.00217	61.02	0.00217	61.02	0.0124	1639.27326	0.0124	1832.448
	0.00289	65.57	0.00289	65.57	0.00289	65.57	0.00289	65.57	0.01395	1735.12267	0.01395	1934.6861
	0.00434	72.56	0.00434	72.56	0.00434	72.56	0.00434	72.56	0.0155	1812.20407	0.0155	2015.9104
	0.00651	80.3	0.00651	80.3	0.00651	80.3	0.00651	80.3	0.01705	1873.49477	0.01705	2079.8277
	0.00868	86.29	0.00868	86.29	0.00868	86.29	0.00868	86.29	0.0186	1921.96279	0.0186	2129.8347
	0.01085	91.24	0.01085	91.24	0.01085	91.24	0.01085	91.24	0.02015	1959.98547	0.02015	2168.6382
	0.01736	102.6	0.01736	102.6	0.01736	102.6	0.01736	102.6	0.0217	1989.68547	0.0217	2198.5416
	0.02604	113.6	0.02604	113.6	0.02604	113.6	0.02604	113.6	0.02325	2012.78547	0.02325	2221.6416
	0.03472	122	0.03472	122	0.03472	122	0.03472	122	0.0248	2030.68547	0.0248	2239.3416
	0.0434	129	0.0434	129	0.0434	129	0.0434	129	0.02635	2044.56279	0.02635	2252.9416
	0.05208	135.1	0.05208	135.1	0.05208	135.1	0.05208	135.1	0.0279	2055.34012	0.0279	2263.3382
	0.06076	140.4	0.06076	140.4	0.06076	140.4	0.06076	140.4	0.02945	2063.61744	0.02945	2271.2382
	0.06944	145.1	0.06944	145.1	0.06944	145.1	0.06944	145.1	0.031	2069.99477	0.031	2277.3347
	0.07812	149.5	0.07812	149.5	0.07812	149.5	0.07812	149.5				
	0.0868	153.4	0.0868	153.4	0.0868	153.4	0.0868	153.4				
	0.08897	153.4	0.08897	153.4	0.08897	153.4	0.08897	153.4				

Description	Loose to Very Dense Sandy Silt to Silty Sand		Soft to Hard Clayey Silt to Sandy Clayey Silt											
Depth (z) *	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= 18.9 m	
Elevation	Elev. 121.6 m		Elev. 120.6 m		Elev. 119.6 m		Elev. 118.6 m		Elev. 117.6 m		Elev. 116.6 m		Elev. 115.7 m	
P-y Curves	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
	0.00155	349.2	8.7E-07	6.3091176	8.7E-07	6.425588	8.7E-07	6.5432353	8.7E-07	6.660882	8.7E-07	6.7773529	8.7E-07	6.8832353
	0.0031	685.049	4.3E-06	16.42	4.3E-06	16.42	4.3E-06	16.42	4.3E-06	16.42	4.3E-06	16.42	4.3E-06	16.42
	0.00465	996.183	8.7E-06	19.53	8.7E-06	19.53	8.7E-06	19.53	8.7E-06	19.53	8.7E-06	19.53	8.7E-06	19.53
	0.0062	1274.52	4.3E-05	29.2	4.3E-05	29.2	4.3E-05	29.2	4.3E-05	29.2	4.3E-05	29.2	4.3E-05	29.2
	0.00775	1516	8.7E-05	34.73	8.7E-05	34.73	8.7E-05	34.73	8.7E-05	34.73	8.7E-05	34.73	8.7E-05	34.73
	0.0093	1719.75	0.00043	51.93	0.00043	51.93	0.00043	51.93	0.00043	51.93	0.00043	51.93	0.00043	51.93
	0.01085	1887.8	0.00087	61.76	0.00087	61.76	0.00087	61.76	0.00087	61.76	0.00087	61.76	0.00087	61.76
	0.0124	2023.86	0.00217	77.66	0.00217	77.66	0.00217	77.66	0.00217	77.66	0.00217	77.66	0.00217	77.66
	0.01395	2132.37	0.00289	83.45	0.00289	83.45	0.00289	83.45	0.00289	83.45	0.00289	83.45	0.00289	83.45
	0.0155	2217.75	0.00434	92.35	0.00434	92.35	0.00434	92.35	0.00434	92.35	0.00434	92.35	0.00434	92.35
	0.01705	2284.37	0.00651	102.2	0.00651	102.2	0.00651	102.2	0.00651	102.2	0.00651	102.2	0.00651	102.2
	0.0186	2335.84	0.00868	109.8	0.00868	109.8	0.00868	109.8	0.00868	109.8	0.00868	109.8	0.00868	109.8
	0.02015	2375.52	0.01085	116.1	0.01085	116.1	0.01085	116.1	0.01085	116.1	0.01085	116.1	0.01085	116.1
	0.0217	2405.9	0.01736	130.6	0.01736	130.6	0.01736	130.6	0.01736	130.6	0.01736	130.6	0.01736	130.6
	0.02325	2429.09	0.02604	144.5	0.02604	144.5	0.02604	144.5	0.02604	144.5	0.02604	144.5	0.02604	144.5
	0.0248	2446.79	0.03472	155.3	0.03472	155.3	0.03472	155.3	0.03472	155.3	0.03472	155.3	0.03472	155.3
	0.02635	2460.19	0.0434	164.2	0.0434	164.2	0.0434	164.2	0.0434	164.2	0.0434	164.2	0.0434	164.2
	0.0279	2470.39	0.05208	171.9	0.05208	171.9	0.05208	171.9	0.05208	171.9	0.05208	171.9	0.05208	171.9
	0.02945	2478.1	0.06076	178.6	0.06076	178.6	0.06076	178.6	0.06076	178.6	0.06076	178.6	0.06076	178.6
	0.031	2483.9	0.06944	184.7	0.06944	184.7	0.06944	184.7	0.06944	184.7	0.06944	184.7	0.06944	184.7
			0.07812	190.2	0.07812	190.2	0.07812	190.2	0.07812	190.2	0.07812	190.2	0.07812	190.2
			0.0868	195.3	0.0868	195.3	0.0868	195.3	0.0868	195.3	0.0868	195.3	0.0868	195.3
			0.08897	195.3	0.08897	195.3	0.08897	195.3	0.08897	195.3	0.08897	195.3	0.08897	195.3

Description	Compact to Very Dense Sandy Silt to Silty Sand														Compact to Very Dense Sand to Silty Sand (Till)			
Depth (z) *	z= 19.1 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			
Elevation	Elev. 115.5 m		Elev. 114.6 m		Elev. 112.6 m		Elev. 110.6 m		Elev. 108.6 m		Elev. 106.6 m		Elev. 104.6 m		Elev. 102.6 m			
P-y Curves	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		
	0.0031	931.11095	0.0031	1024.575	0.0031	1231.812	0.0031	1438.5714	0.0031	1645.1214	0.0031	1850.3857	0.00155	1706.4	0.00155	1819.35714		
	0.0062	1822.881	0.0062	2001.6125	0.0062	2395.9503	0.0062	2787.3714	0.0062	3176.5976	0.0062	3556.8143	0.0031	3331.2	0.0031	3547.4		
	0.0093	2642.4976	0.0093	2891.97083	0.0093	3438.9784	0.0093	3977.6571	0.0093	4510.156	0.0093	5016.7952	0.00465	4807.7857	0.00465	5110.34286		
	0.0124	3367.569	0.0124	3670.55833	0.0124	4329.6216	0.0124	4973.2357	0.0124	5605.5155	0.0124	6188.0429	0.0062	6093.8286	0.0062	6462.54286		
	0.0155	3987.7357	0.0155	4326.9875	0.0155	5059.4922	0.0155	5769.2786	0.0155	6462.8571	0.0155	7079.9238	0.00775	7173.1143	0.00775	7588.28571		
	0.0186	4503.0952	0.0186	4863.825	0.0186	5637.6898	0.0186	6383.1786	0.0186	7108.9536	0.0186	7732.681	0.0093	8050.9857	0.0093	8495.77143		
	0.0217	4921.0857	0.0217	5291.85417	0.0217	6083.6263	0.0217	6843.4429	0.0217	7581.9857	0.0217	8196.4571	0.01085	8747	0.01085	9208.57143		
	0.0248	5253.6238	0.0248	5626.4375	0.0248	6420.5012	0.0248	7181.4214	0.0248	7921.1262	0.0248	8519.2095	0.0124	9287.9	0.0124	9757.01429		
	0.0279	5514.0262	0.0279	5883.76667	0.0279	6670.8766	0.0279	7425.8071	0.0279	8160.6452	0.0279	8740.5238	0.01395	9701.5143	0.01395	10172.4286		
	0.031	5715.4714	0.031	6079.40833	0.031	6854.8898	0.031	7600.4571	0.031	8327.9238	0.031	8890.8238	0.0155	10014	0.0155	10483.4286		
	0.0341	5869.8381	0.0341	6226.6625	0.0341	6989.003	0.0341	7724.3286	0.0341	8443.9726	0.0341	8992.2333	0.01705	10247.857	0.01705	10713.8571		
	0.0372	5987.2452	0.0372	6336.81667	0.0372	7086.0287	0.0372	7811.6	0.0372	8523.9214	0.0372	9060.2286	0.0186	10421.857	0.0186	10883.8571		
	0.0403	6076.0714	0.0403	6418.7625	0.0403	7155.9419	0.0403	7872.9286	0.0403	8578.9702	0.0403	9105.7476	0.02015	10550.714	0.02015	11008.8571		
	0.0434	6142.9976	0.0434	6479.4125	0.0434	7206.2299	0.0434	7915.9143	0.0434	8616.6595	0.0434	9136.1524	0.0217	10645.714	0.0217	11099.8571		
	0.0465	6193.2429	0.0465	6524.34583	0.0465	7242.2054	0.0465	7945.9214	0.0465	8642.4488	0.0465	9156.4429	0.02325	10715.714	0.02325	11166.2857		
	0.0496	6230.9071	0.0496	6557.38333	0.0496	7267.9683	0.0496	7966.8071	0.0496	8660.0381	0.0496	9169.8952	0.0248	10766.571	0.0248	11214.2857		
	0.0527	6259.0714	0.0527	6581.70833	0.0527	7286.3311	0.0527	7981.45	0.0527	8672.0976	0.0527	9178.9095	0.02635	10804.571	0.02635	11249.2857		
	0.0558	6280.1143	0.0558	6599.62917	0.0558	7299.4689	0.0558	7991.5929	0.0558	8680.3274	0.0558	9184.8857	0.0279	10831.571	0.0279	11274.7143		
	0.0589	6295.8167	0.0589	6612.84583	0.0589	7308.894	0.0589	7998.7143	0.0589	8685.9571	0.0589	9188.9238	0.02945	10851.571	0.02945	11293.2857		
	0.062	6307.519	0.062	6622.4625	0.062	7315.5192	0.062	8003.6357	0.062	8689.7869	0.062	9191.5619	0.031	10866.429	0.031	11306.2857		







NOTES: * Depth (z) is measured to be positive below the underside of the pile cap (Elevation 134.6 m).
The P-y curves have been generated based on the following assumptions:
1 P-y curves are generated for vertical piles (i.e. no inclination)
2 Static loading condition is considered.
3 There are no pile group effects (i.e. analysis is based on a single pile).

Date: September 2021
Project No: 1786302

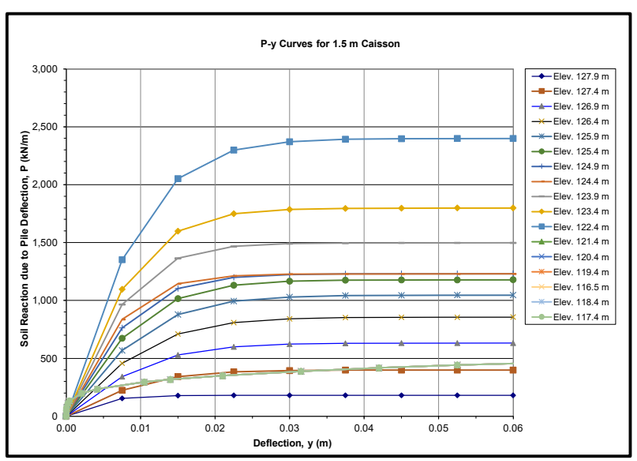
Prepared By: CC/AK
Checked By: CN

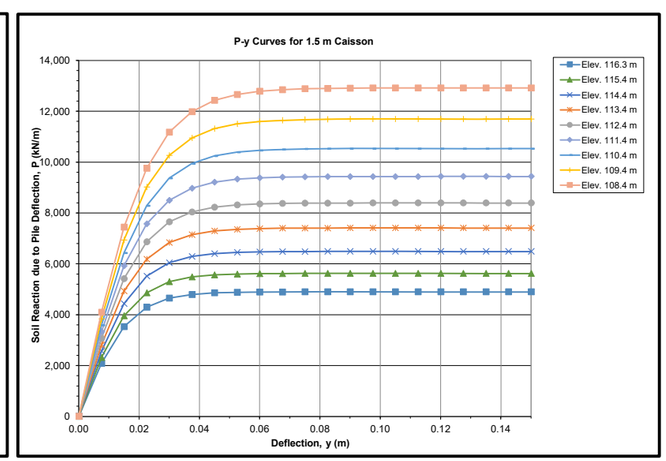
GOLDER

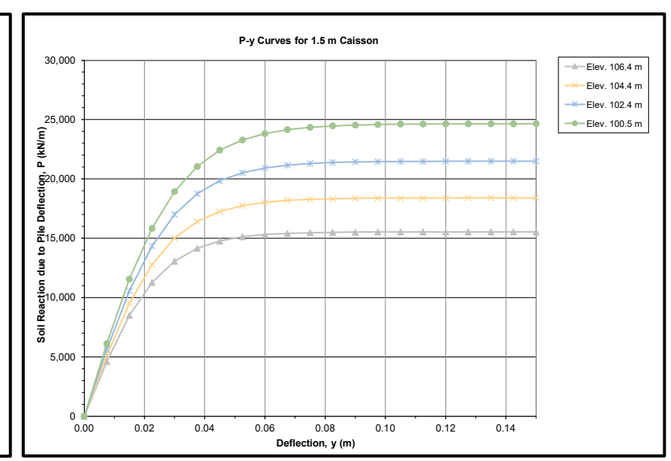
Summary of Load-Deflection (P-y Curves) for 1.5 m diameter Drilled Shaft at East Pier	Figure 2

Description	Loose to Dense Sandy Silt to Silty Sand		Soft to Stiff Silty Clay with Sand to Sandy Clayey Silt																						Compact to Very Dense Silty Sand to Sandy Gravel (Till) to Sand												Compact to Very Dense Silty Sand to Sandy Gravel (Till) to Sand	
Depth (z) *	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= 11.9 m		z= 12.1 m	
Elevation	Elev. 127.9 m		Elev. 127.4 m		Elev. 126.9 m		Elev. 126.4 m		Elev. 125.9 m		Elev. 125.4 m		Elev. 124.9 m		Elev. 124.4 m		Elev. 123.9 m		Elev. 123.4 m		Elev. 122.4 m		Elev. 121.4 m		Elev. 120.4 m		Elev. 119.4 m		Elev. 118.4 m		Elev. 117.4 m		Elev. 116.5 m		Elev. 116.3 m	
P-y Curves	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)	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
	0.0075	154.203	0.0075	225.22525	0.0075	341.8098	0.0075	457.12764	0.0075	568.7417	0.0075	673.00966	0.0075	763.33418	0.0075	836.42586	0.0075	965.65917	0.0075	1097.0571	0.0075	1351.6	4.2E-06	7.7452394	4.2E-06	8.3123357	4.2E-06	8.880007	4.2E-06	9.4434898	4.2E-06	10.01	0.0000042	10.5228571	0.0075	2087.44286	
	0.015	178.663	0.015	341.8451	0.015	529.1016	0.015	711.30827	0.015	877.8939	0.015	1015.0158	0.015	1102.9816	0.015	1143.745	0.015	1364.47237	0.015	1599.1143	0.015	2052.2429	2.1E-05	38.726197	2.1E-05	41.562028	2.1E-05	44.399789	2.1E-05	47.2221088	2.1E-05	50.07	0.000021	52.6142857	0.015	3533.08571	
	0.0225	180.778	0.0225	382.59099	0.0225	599.776	0.0225	809.19063	0.0225	993.3113	0.0225	1131.5255	0.0225	1200.1636	0.0225	1212.9047	0.0225	1468.33644	0.0225	1748.0714	0.0225	2297.3429	4.2E-05	74.25	4.2E-05	74.25	4.2E-05	74.25	4.2E-05	74.25	4.2E-05	74.25	0.000042	74.25	0.0225	4298.84286	
	0.03	180.948	0.03	394.76929	0.03	622.5521	0.03	841.3778	0.03	1030.032	0.03	1165.5729	0.03	1224.144	0.03	1226.5502	0.03	1491.72606	0.03	1786.1286	0.03	2370.7	0.00021	111	0.00021	111	0.00021	111	0.00021	111	0.00021	111	0.00021	111	0.03	4647.67143	
	0.0375	180.96	0.0375	398.23951	0.0375	629.5185	0.0375	851.39953	0.0375	1041.225	0.0375	1175.0163	0.0375	1229.87	0.0375	1229.117	0.0375	1496.84846	0.0375	1795.5	0.0375	2391.6143	0.00042	132	0.00042	132	0.00042	132	0.00042	132	0.00042	132	0.00042	132	0.0375	4795.62857	
	0.045	180.96	0.045	399.21241	0.045	631.6086	0.045	854.46969	0.045	1044.525	0.045	1177.5975	0.045	1231.1794	0.045	1229.6283	0.045	1497.98926	0.045	1797.7857	0.045	2397.5571	0.0021	197.4	0.0021	197.4	0.0021	197.4	0.0021	197.4	0.0021	197.4	0.0021	197.4	0.045	4856.45714	
	0.0525	180.96	0.0525	399.48851	0.0525	632.2308	0.0525	855.40386	0.0525	1045.525	0.0525	1178.3579	0.0525	1231.4818	0.0525	1229.7283	0.0525	1498.20286	0.0525	1798.3571	0.0525	2399.1429	0.0042	234.8	0.0042	234.8	0.0042	234.8	0.0042	234.8	0.0042	234.8	0.0042	234.8	0.0525	4881.08571	
	0.06	180.96	0.06	399.5608	0.06	632.4205	0.06	855.68622	0.06	1045.743	0.06	1178.5183	0.06	1231.5842	0.06	1229.7283	0.06	1498.28926	0.06	1798.4571	0.06	2399.6286	0.0105	295.2	0.0105	295.2	0.0105	295.2	0.0105	295.2	0.0105	295.2	0.0105	295.2	0.06	4891.11429	
	0.0675	180.96	0.0675	399.58157	0.0675	632.477	0.0675	855.76858	0.0675	1045.843	0.0675	1178.6183	0.0675	1231.5842	0.0675	1229.7283	0.0675	1498.28926	0.0675	1798.4571	0.0675	2399.7286	0.014	317.3	0.014	317.3	0.014	317.3	0.014	317.3	0.014	317.3	0.014	317.3	0.0675	4895.12857	
	0.075	180.96	0.075	399.59157	0.075	632.4936	0.075	855.79858	0.075	1045.925	0.075	1178.6183	0.075	1231.5842	0.075	1229.7283	0.075	1498.28926	0.075	1798.5286	0.075	2399.7429	0.021	351.1	0.021	351.1	0.021	351.1	0.021	351.1	0.021	351.1	0.021	351.1	0.075	4896.7	
	0.0825	180.96	0.0825	399.59157	0.0825	632.5036	0.0825	855.80858	0.0825	1045.925	0.0825	1178.6183	0.0825	1231.5842	0.0825	1229.7283	0.0825	1498.28926	0.0825	1798.5286	0.0825	2399.8286	0.0315	388.6	0.0315	388.6	0.0315	388.6	0.0315	388.6	0.0315	388.6	0.0315	388.6	0.0825	4897.4	
	0.09	180.96	0.09	399.59157	0.09	632.5036	0.09	855.80858	0.09	1045.925	0.09	1178.6183	0.09	1231.5842	0.09	1229.7283	0.09	1498.28926	0.09	1798.5286	0.09	2399.8286	0.042	417.5	0.042	417.5	0.042	417.5	0.042	417.5	0.042	417.5	0.042	417.5	0.09	4897.6	
	0.0975	180.96	0.0975	399.59157	0.0975	632.5036	0.0975	855.80858	0.0975	1045.925	0.0975	1178.6183	0.0975	1231.5842	0.0975	1229.7283	0.0975	1498.28926	0.0975	1798.5286	0.0975	2399.8286	0.0525	441.5	0.0525	441.5	0.0525	441.5	0.0525	441.5	0.0525	441.5	0.0525	441.5	0.0975	4897.77143	
	0.105	180.96	0.105	399.59157	0.105	632.5036	0.105	855.80858	0.105	1045.925	0.105	1178.6183	0.105	1231.5842	0.105	1229.7283	0.105	1498.28926	0.105	1798.5286	0.105	2399.8286	0.084	496.5	0.084	496.5	0.084	496.5	0.084	496.5	0.084	496.5	0.084	496.5	0.105	4897.77143	
	0.1125	180.96	0.1125	399.59157	0.1125	632.5036	0.1125	855.80858	0.1125	1045.925	0.1125	1178.6183	0.1125	1231.5842	0.1125	1229.7283	0.1125	1498.28926	0.1125	1798.5286	0.1125	2399.8286	0.126	549.5	0.126	549.5	0.126	549.5	0.126	549.5	0.126	549.5	0.126	549.5	0.1125	4897.8	
	0.12	180.96	0.12	399.59157	0.12	632.5036	0.12	855.80858	0.12	1045.925	0.12	1178.6183	0.12	1231.5842	0.12	1229.7283	0.12	1498.28926	0.12	1798.5286	0.12	2399.8286	0.168	590.5	0.168	590.5	0.168	590.5	0.168	590.5	0.168	590.5	0.168	590.5	0.12	4897.8	
	0.1275	180.96	0.1275	399.59157	0.1275	632.5036	0.1275	855.80858	0.1275	1045.925	0.1275	1178.6183	0.1275	1231.5842	0.1275	1229.7283	0.1275	1498.28926	0.1275	1798.5286	0.1275	2399.8286	0.21	624.4	0.21	624.4	0.21	624.4	0.21	624.4	0.21	624.4	0.21	624.4	0.1275	4897.8	
	0.135	180.96	0.135	399.59157	0.135	632.5036	0.135	855.80858	0.135	1045.925	0.135	1178.6183	0.135	1231.5842	0.135	1229.7283	0.135	1498.28926	0.135	1798.5286	0.135	2399.8286	0.252	653.5	0.252	653.5	0.252	653.5	0.252	653.5	0.252	653.5	0.252	653.5	0.135	4897.8	
	0.1425	180.96	0.1425	399.59157	0.1425	632.5036	0.1425	855.80858	0.1425	1045.925	0.1425	1178.6183	0.1425	1231.5842	0.1425	1229.7283	0.1425	1498.28926	0.1425	1798.5286	0.1425	2399.8286	0.294	679.2	0.294	679.2	0.294	679.2	0.294	679.2	0.294	679.2	0.294	679.2	0.1425	4897.8	
	0.15	180.96	0.15	399.59157	0.15	632.5036	0.15	855.80858	0.15	1045.925	0.15	1178.6183	0.15	1231.5842	0.15	1229.7283	0.15	1498.28926	0.15	1798.5286	0.15	2399.8286	0.336	702.2	0.336	702.2	0.336	702.2	0.336	702.2	0.336	702.2	0.336	702.2	0.15	4897.8	
																							0.378	723.2	0.378	723.2	0.378	723.2	0.378	723.2	0.378	723.2	0.378	723.2			
																							0.42	742.5	0.42	742.5	0.42	742.5	0.42	742.5	0.42	742.5	0.42	742.5			
																							0.4305	742.5	0.4305	742.5	0.4305	742.5	0.4305	742.5	0.4305	742.5	0.4305	742.5			

Description	Compact to Very Dense Silty Sand to Sandy Gravel (Till) to Sand																							
Depth (z) *	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= 27.9 m	
Elevation	Elev. 115.4 m		Elev. 114.4 m		Elev. 113.4 m		Elev. 112.4 m		Elev. 111.4 m		Elev. 110.4 m		Elev. 109.4 m		Elev. 108.4 m		Elev. 106.4 m		Elev. 104.4 m		Elev. 102.4 m		Elev. 100.5 m	
P-y Curves	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
	0.0075	2313.62	0.0075	2566.1	0.0075	2819.819	0.0075	3074.35	0.0075	3329.506	0.0075	3585.225	0.0075	3841.217	0.0075	4097.7	0.0075	4611.1	0.0075	5124.95	0.0075	5639.075	0.0075	6127.4143
	0.015	3957.53	0.015	4437.825	0.015	4926.374	0.015	5421.25	0.015	5921.562	0.015	6426.425	0.015	6934.6596	0.015	7446.2	0.015	8475.725	0.015	9511.6	0.015	10551.75	0.015	11541.857
	0.0225	4863.35	0.0225	5512.375	0.0225	6182.36	0.0225	6870.15	0.0225	7573.624	0.0225	8290.95	0.0225	9019.7511	0.0225	9759.4	0.0225	11263.5	0.0225	12793.5	0.0225	14342.75	0.0225	15827.429
	0.03	5293.8	0.03	6046.35	0.03	6833.062	0.03	7650.5	0.03	8496.306	0.03	9367.925	0.03	10261.915	0.03	11178	0.03	13063.75	0.03	15010	0.03	17003.75	0.03	18933.143
	0.0375	5484.02	0.0375	6292.625	0.0375	7145.759	0.0375	8040.3	0.0375	8973.982	0.0375	9944.575	0.0375	10948.745	0.0375	11986	0.0375	14145.75	0.0375	16405.5	0.0375	18750.25	0.0375	21043.143
	0.045	5565.29	0.045	6402.35	0.045	7290.589	0.045	8227.55	0.045	9211.525	0.045	10241	0.045	11312.468	0.045	12426	0.045	14766.5	0.045	17245.5	0.045	19845.5	0.045	22414.429
	0.0525	5599.51	0.0525	6450.4	0.0525	7356.479	0.0525	8315.85	0.0525	9327.409	0.0525	10390	0.0525	11500.702	0.0525	12661	0.0525	15114.75	0.0525	17737.5	0.0525	20514	0.0525	23280.714
	0.06	5613.91	0.06	6471.375	0.06	7386.305	0.06	8357.1	0.06	9383.282	0.06	10464.25	0.06	11597.319	0.06	12784	0.06	15307	0.06	18021.5	0.06	20914.75	0.06	23816.857
	0.0675	5619.86	0.0675	6480.5	0.0675	7399.668	0.0675	8376.35	0.0675	9410.193	0.0675	10500.75	0.0675	11646.936	0.0675	12848	0.0675	15412	0.0675	18183	0.0675	21152.25	0.0675	24145.857
	0.075	5622.37	0.075	6484.45	0.075	7405.681	0.075	8385.3	0.075	9423.067	0.075	10518.75	0.075	11671.809	0.075	12882	0.075	15469.25	0.075	18275	0.075	21292.25	0.075	24345.571
	0.0825	5623.4	0.0825	6486.175	0.0825	7408.356	0.0825	8389.4	0.0825	9429.217	0.0825	10527.5	0.0825	11683.809	0.0825	12899	0.0825	15500.75	0.0825	18327	0.0825	21374.5	0.0825	24467.143
	0.09	5623.89	0.09	6486.9	0.09	7409.594	0.09	8391.35	0.09	9432.179	0.09	10532.5	0.09	11690.681	0.09	12908	0.09	15517.75	0.09	18356.5	0.09	21422.25	0.09	24540
	0.0975	5624	0.0975	6487.2	0.0975	7410.194	0.0975	8392.2	0.0975	9433.542	0.0975	10534.5	0.0975	11693.681	0.0975	12913	0.0975	15527	0.0975	18373	0.0975	21451	0.0975	24584.286
	0.105	5624.1	0.105	6487.325	0.105	7410.394	0.105	8392.65	0.105	9434.242	0.105	10535.5	0.105	11695.681	0.105	12915	0.105	15532	0.105	18383	0.105	21467	0.105	24610.571
	0.1125	5624.19	0.1125	6487.4	0.1125	7410.531	0.1125	8392.85	0.1125	9434.542	0.1125	10536.25	0.1125	11695.809	0.1125	12917	0.1125	15534.25	0.1125	18388	0.1125	21476.75	0.1125	24626.857
	0.12	5624.19	0.12	6487.425	0.12	7410.531	0.12	8392.95	0.12	9434.704	0.12	10536.25	0.12	11696.681	0.12	12917	0.12	15536	0.12	18391	0.12	21482.75	0.12	24636.857
	0.1275	5624.2	0.1275	6487.425	0.1275	7410.594	0.1275	8392.95	0.1275	9434.804	0.1275	10536.5	0.1275	11696.681	0.1275	12918	0.1275	15536.25	0.1275	18392.5	0.1275	21485.75	0.1275	24642.143
	0.135	5624.2	0.135	6487.425	0.135	7410.594	0.135	8393	0.135	9434.804	0.135	10536.5	0.135	11696.809	0.135	12918	0.135	15537	0.135	18393.5	0.135	21487.75	0.135	24645.857
	0.1425	5624.2	0.1425	6487.425	0.1425	7410.594	0.1425	8393	0.1425	9434.842	0.1425	10536.5	0.1425	11696.809	0.1425	12918	0.1425	15537.25	0.1425	18394.5	0.1425	21488.75	0.1425	24648.143
	0.15	5624.2	0.15	6487.425	0.15	7410.594	0.15	8393	0.15	9434.842	0.15	10536.5	0.15	11696.809	0.15	12918	0.15	15537.25	0.15	18394.5	0.15	21489.75	0.15	24649.143







NOTES:

* Depth (z) is measured to be positive below the top of the caisson (Elevation 128.4 m).

The P-y curves have been generated based on the following assumptions:

1 P-y curves are generated for vertical piles (i.e. no inclination)

2 Static loading condition is considered.

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

Date: September 2021 **Prepared By:** CC

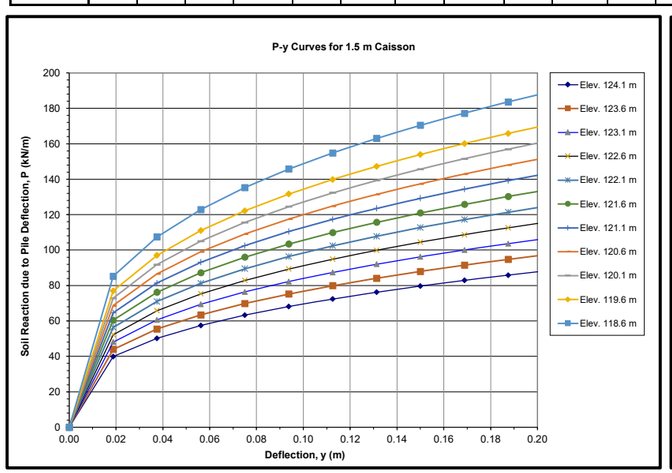
Project No: 1786302 **Checked By:** CN

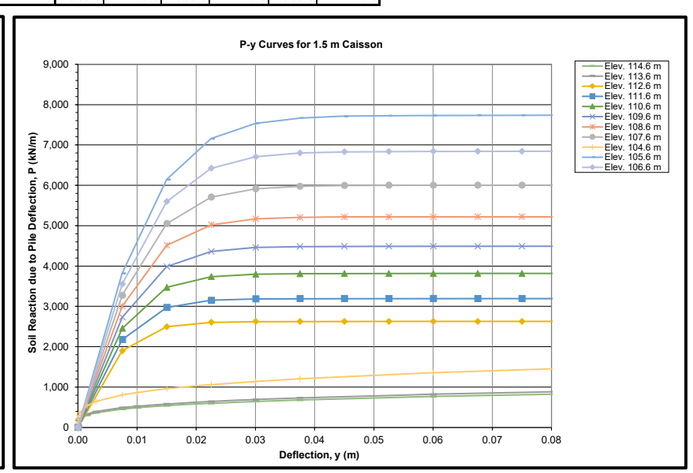
GOLDER

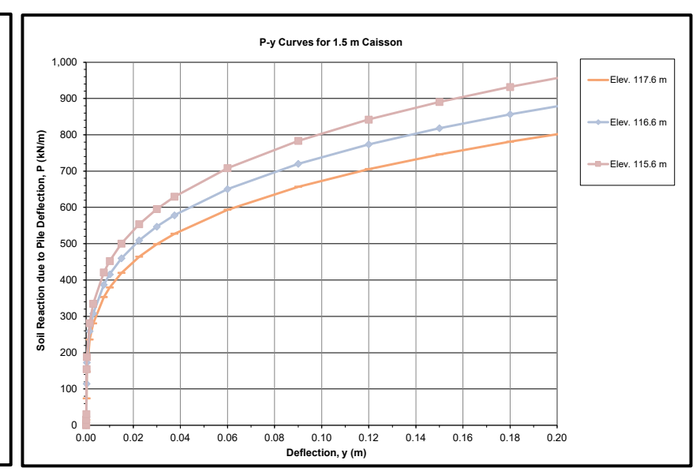
Summary of Load-Deflection (P-y Curves) for 1.5 m diameter Drilled Shaft at East Pier	Figure 3

Description	Very Soft to Stiff Clay to Clayey Silt																								Firm to Hard Clayey Silt										Compact to Dense Silt to Silty Sand			
Depth (z) *	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	
Elevation	Elev. 124.1 m		Elev. 123.6 m		Elev. 123.1 m		Elev. 122.6 m		Elev. 122.1 m		Elev. 121.6 m		Elev. 121.1 m		Elev. 120.6 m		Elev. 120.1 m		Elev. 119.6 m		Elev. 118.6 m		Elev. 117.6 m		Elev. 116.6 m		Elev. 115.6 m		Elev. 114.6 m		Elev. 113.6 m		Elev. 112.6 m		Elev. 111.6 m	
P-y Curves	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)	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
	0.01875	39.8453	0.01875	43.963819	0.01875	48.08695	0.01875	52.207119	0.01875	56.33551	0.01875	60.463898	0.01875	64.584068	0.01875	68.712458	0.01875	72.8341026	0.01875	76.961017	0.01875	85.208644	3E-06	1.4756667	3E-06	2.2856667	3E-06	3.096	3E-06	3.90566667	3E-06	4.715667	0.0075	1900.44146	0.0075	2177.4821
	0.0375	50.1959	0.0375	55.389856	0.0375	60.58915	0.0375	65.778644	0.0375	70.98059	0.0375	76.182881	0.0375	81.367797	0.0375	86.567627	0.0375	91.7607692	0.0375	96.961186	0.0375	107.37881	1.5E-05	7.379	1.5E-05	11.43	1.5E-05	15.48	1.5E-05	19.53	1.5E-05	23.58	0.015	2494.6122	0.015	2972.6472
	0.05625	57.4596	0.05625	63.406884	0.05625	69.36034	0.05625	75.302542	0.05625	81.25661	0.05625	87.205254	0.05625	93.142881	0.05625	99.096949	0.05625	105.051282	0.05625	110.99322	0.05625	122.86949	0.00003	14.756667	0.00003	22.856667	0.00003	30.96	0.00003	39.0566667	0.00003	47.15667	0.0225	2604.62683	0.0225	3150.2415
	0.075	63.2436	0.075	69.789407	0.075	76.33966	0.075	82.877797	0.075	89.42839	0.075	95.984746	0.075	102.51186	0.075	109.10508	0.075	115.58718	0.075	122.17797	0.075	135.26017	0.00015	73.79	0.00015	114.3	0.00015	154.8	0.00015	171.133333	0.00015	183.9667	0.03	2622.74472	0.03	3185.0431
	0.09375	68.1251	0.09375	75.177426	0.09375	82.22983	0.09375	89.273729	0.09375	96.33381	0.09375	103.37797	0.09375	110.41186	0.09375	117.49915	0.09375	124.505128	0.09375	131.62034	0.09375	145.74153	0.0003	147.56667	0.0003	173	0.0003	188.3	0.0003	203.5	0.0003	218.8	0.0375	2625.61057	0.0375	3191.6675
	0.1125	72.3969	0.1125	79.885775	0.1125	87.39	0.1125	94.870339	0.1125	102.3814	0.1125	109.87797	0.1125	117.35085	0.1125	124.89322	0.1125	132.323077	0.1125	139.86271	0.1125	154.8322	0.0015	235.9	0.0015	258.7	0.0015	281.5	0.0015	304.333333	0.0015	327.1667	0.045	2626.11057	0.045	3192.8756
	0.1313	76.2112	0.1313	84.099289	0.1313	91.99559	0.1313	99.876949	0.1313	107.7814	0.1313	115.66949	0.1313	123.55085	0.1313	131.48729	0.1313	139.323077	0.1313	147.20508	0.1313	163.02288	0.003	280.56667	0.003	307.7	0.003	334.8	0.003	361.9	0.003	389.0333	0.0525	2626.19919	0.0525	3193.0797
	0.15	79.6806	0.15	87.927968	0.15	96.18119	0.15	104.4322	0.15	112.7025	0.15	120.91525	0.15	129.18983	0.15	137.38729	0.15	145.658974	0.15	153.90508	0.15	170.41356	0.0075	352.76667	0.0075	386.83333	0.0075	421	0.0075	455.066667	0.0075	489.2333	0.06	2626.19919	0.06	3193.1756
	0.1688	82.87	0.1688	91.446647	0.1688	100.0468	0.1688	108.62881	0.1688	117.2237	0.1688	125.76102	0.1688	134.35932	0.1688	142.88136	0.1688	151.476923	0.1688	160.04746	0.1688	177.21356	0.01	379.06667	0.01	415.73333	0.01	452.4	0.01	489.066667	0.01	525.7333	0.0675	2626.19919	0.0675	3193.1797
	0.1875	85.837	0.1875	94.715326	0.1875	103.6305	0.1875	112.52881	0.1875	121.3449	0.1875	130.26102	0.1875	139.15932	0.1875	148.07542	0.1875	156.876923	0.1875	165.78983	0.1875	183.60424	0.015	419.53333	0.015	460.06667	0.015	500.6	0.015	541.2	0.015	581.7667	0.075	2626.19919	0.075	3193.1797
	0.2063	88.604	0.2063	97.774335	0.2063	106.9305	0.2063	116.12542	0.2063	125.2661	0.2063	134.46102	0.2063	143.62881	0.2063	152.77542	0.2063	161.994872	0.2063	171.1322	0.2063	189.49492	0.0225	464.3	0.0225	509.13333	0.0225	554	0.0225	598.9	0.0225	643.8	0.0825	2626.19919	0.0825	3193.1797
	0.225	91.2109	0.225	100.65334	0.225	110.1034	0.225	119.52542	0.225	128.9661	0.225	138.40678	0.225	147.82881	0.225	157.26949	0.225	166.712821	0.225	176.17458	0.225	195.09492	0.03	498.9	0.03	547.16667	0.03	595.4	0.03	643.633333	0.03	691.8667	0.09	2626.19919	0.09	3193.1797
	0.2438	93.6755	0.2438	103.41202	0.2438	113.0763	0.2438	122.72542	0.2438	132.4873	0.2438	142.15254	0.2438	151.89831	0.2438	161.56356	0.2438	171.212821	0.2438	180.97458	0.2438	200.38559	0.0375	527.53333	0.0375	578.53333	0.0375	629.5	0.0375	680.533333	0.0375	731.5667	0.0975	2626.19919	0.0975	3193.1797
	0.2625	96.0201	0.2625	105.96037	0.2625	115.8763	0.2625	125.82203	0.2625	135.7873	0.2625	145.75254	0.2625	155.6678	0.2625	165.56356	0.2625	175.530769	0.2625	185.47458	0.2625	205.37627	0.06	593.3	0.06	650.63333	0.06	708	0.06	765.4	0.06	822.7333	0.105	2626.19919	0.105	3193.1797
	0.2813	98.2595	0.2813	108.46037	0.2813	118.6492	0.2813	128.71864	0.2813	138.9085	0.2813	149.09831	0.2813	159.2678	0.2813	169.45763	0.2813	179.630769	0.2813	189.81695	0.2813	210.17627	0.09	656.53333	0.09	720.06667	0.09	783.5	0.09	847.033333	0.09	910.5333	0.1125	2626.19919	0.1125	3193.1797
	0.3	100.382	0.3	110.76037	0.3	121.1492	0.3	131.51864	0.3	141.9297	0.3	152.34407	0.3	162.73729	0.3	173.15169	0.3	183.530769	0.3	193.91695	0.3	214.67627	0.12	705.53333	0.12	773.76667	0.12	842	0.12	910.2	0.12	978.4	0.12	2626.19919	0.12	3193.1797
	0.45	100.382	0.45	110.76037	0.45	121.1492	0.45	131.51864	0.45	141.9297	0.45	152.34407	0.45	162.73729	0.45	173.15169	0.45	183.530769	0.45	193.91695	0.45	214.67627	0.15	746	0.15	818.13333	0.15	890.3	0.15	962.4	0.15	1034.333	0.1275	2626.19919	0.1275	3193.1797
																							0.18	780.8	0.18	856.3	0.18	931.8	0.18	1007	0.18	1083	0.135	2626.19919	0.135	3193.1797
																							0.21	811.43333	0.21	889.96667	0.21	968.4	0.21	1047	0.21	1125	0.1425	2626.19919	0.1425	3193.1797
																							0.24	839.03333	0.24	920.16667	0.24	1001	0.24	1082.33333	0.24	1163.667	0.15	2626.19919	0.15	3193.1797
																							0.27	864.06667	0.27	947.66667	0.27	1031	0.27	1114.33333	0.27	1198.667				
																							0.3	887.13333	0.3	972.96667	0.3	1059	0.3	1144.33333	0.3	1230.333				
																							0.3075	887.13333	0.3075	972.96667	0.3075	1059	0.3075	1144.33333	0.3075	1230.333				

Description	Compact to Dense Silt to Silty Sand												Very Dense/Hard Glacial Till					
Depth (z) *	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	
Elevation	Elev. 110.6 m		Elev. 109.6 m		Elev. 108.6 m		Elev. 107.6 m		Elev. 106.6 m		Elev. 105.6 m		Elev. 104.6 m		Elev. 102.6 m		Elev. 100.6 m	
P-y Curves	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
	0.0075	2454.04	0.0075	2729.8667	0.0075	3004.665	0.0075	3278.5434	0.0075	3551.03	0.0075	3822.4634	3E-06	23.082308	3E-06	24.704615	3E-06	26.3215385
	0.015	3471.15	0.015	3985.9667	0.015	4513.571	0.015	5051.0082	0.015	5595.35	0.015	6144.8463	1.5E-05	115.36154	1.5E-05	123.48462	1.5E-05	131.607692
	0.0225	3736.95	0.0225	4361.4	0.0225	5019.421	0.0225	5707.6787	0.0225	6421.703	0.0225	7158.2049	0.00003	202.5	0.00003	202.5	0.00003	202.5
	0.03	3797.26	0.03	4457.9333	0.03	5164.783	0.03	5915.6066	0.03	6706.724	0.03	7535.7122	0.00015	302.8	0.00015	302.8	0.00015	302.8
	0.0375	3810.47	0.0375	4481.8667	0.0375	5204.634	0.0375	5978.0262	0.0375	6799.594	0.0375	7668.0171	0.0003	360.1	0.0003	360.1	0.0003	360.1
	0.045	3813.32	0.045	4487.6667	0.045	5215.418	0.045	5996.5205	0.045	6829.318	0.045	7713.2561	0.0015	538.5	0.0015	538.5	0.0015	538.5
	0.0525	3813.94	0.0525	4489.1	0.0525	5218.314	0.0525	6001.973	0.0525	6838.707	0.0525	7728.6902	0.003	640.4	0.003	640.4	0.003	640.4
	0.06	3814.12	0.06	4489.4667	0.06	5219.162	0.06	6003.5361	0.06	6841.741	0.06	7733.8683	0.0075	805.2	0.0075	805.2	0.0075	805.2
	0.0675	3814.12	0.0675	4489.5	0.0675	5219.362	0.0675	6003.9992	0.0675	6842.641	0.0675	7735.661	0.01	865.3	0.01	865.3	0.01	865.3
	0.075	3814.12	0.075	4489.6	0.075	5219.362	0.075	6004.1992	0.075	6843.019	0.075	7736.261	0.015	957.6	0.015	957.6	0.015	957.6
	0.0825	3814.12	0.0825	4489.6	0.0825	5219.41	0.0825	6004.1992	0.0825	6843.119	0.0825	7736.4537	0.0225	1060	0.0225	1060	0.0225	1060
	0.09	3814.12	0.09	4489.6	0.09	5219.41	0.09	6004.1992	0.09	6843.119	0.09	7736.5537	0.03	1139	0.03	1139	0.03	1139
	0.0975	3814.12	0.0975	4489.6	0.0975	5219.41	0.0975	6004.1992	0.0975	6843.119	0.0975	7736.5537	0.0375	1204	0.0375	1204	0.0375	1204
	0.105	3814.12	0.105	4489.6	0.105	5219.41	0.105	6004.1992	0.105	6843.119	0.105	7736.5537	0.06	1354	0.06	1354	0.06	1354
	0.1125	3814.12	0.1125	4489.6	0.1125	5219.41	0.1125	6004.1992	0.1125	6843.119	0.1125	7736.5537	0.09	1499	0.09	1499	0.09	1499
	0.12	3814.12	0.12	4489.6	0.12	5219.41	0.12	6004.1992	0.12	6843.119	0.12	7736.5537	0.12	1610	0.12	1610	0.12	1610
	0.1275	3814.12	0.1275	4489.6	0.1275	5219.41	0.1275	6004.1992	0.1275	6843.119	0.1275	7736.5537	0.15	1703	0.15	1703	0.15	1703
	0.135	3814.12	0.135	4489.6	0.135	5219.41	0.135	6004.1992	0.135	6843.119	0.135	7736.5537	0.18	1782	0.18	1782	0.18	1782
	0.1425	3814.12	0.1425	4489.6	0.1425	5219.41	0.1425	6004.1992	0.1425	6843.119	0.1425	7736.5537	0.21	1852	0.21	1852	0.21	1852
	0.15	3814.12	0.15	4489.6	0.15	5219.41	0.15	6004.1992	0.15	6843.119	0.15	7736.5537	0.24	1915	0.24	1915	0.24	1915
													0.27	1972	0.27	1972	0.27	1972
													0.3	2025	0.3	2025	0.3	2025
													0.3075	2025	0.3075	2025	0.3075	2025







NOTES:

* Depth (z) is measured to be positive below the top of the caisson (Elevation 124.6 m).

The P-y curves have been generated based on the following assumptions:

1 P-y curves are generated for vertical piles (i.e. no inclination)

2 Static loading condition is considered.

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

Date: September 2021 **Prepared By:** CC/ACK

Project No: 1786302 **Checked By:** CN

GOLDER

Summary of Load-Deflection (P-y Curves) for HP 310x110 at East Abutment	Figure 4

Summary of Load-Deflection (P-y Curves) for HP 310x110 at East Abutment

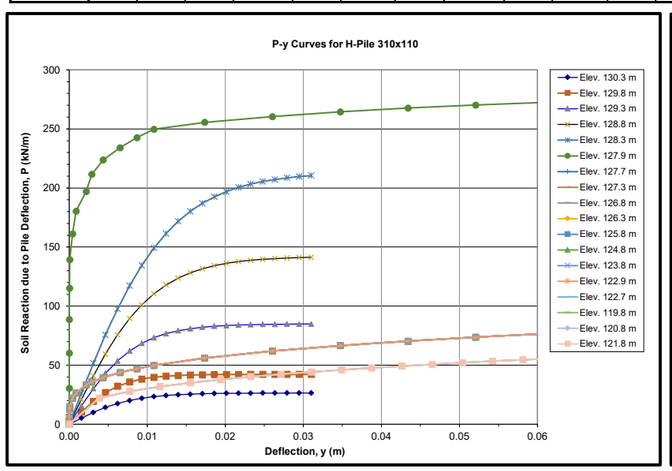
Description	Loose Sand in CSP											
Depth (z) *	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 2.9 m	
Elevation	Elev. 130.3 m		Elev. 129.8 m		Elev. 129.3 m		Elev. 128.8 m		Elev. 128.3 m		Elev. 127.9 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0	0	0	0	0	0	0	0	0	0	0	0
	0.00155	5.2013	0.00155	10.326	0.00155	15.63	0.00155	20.927	0.00155	26.217	0.00155	30.446011
	0.0031	10.015	0.0031	19.488	0.0031	30.236	0.0031	40.964	0.0031	51.655	0.0031	60.186426
	0.00465	14.161	0.00465	26.791	0.00465	43.046	0.00465	59.368	0.00465	75.627	0.00465	88.578046
	0.0062	17.517	0.0062	32.133	0.0062	53.671	0.0062	75.637	0.0062	97.6	0.0062	115.09887
	0.00775	20.098	0.00775	35.799	0.00775	62.083	0.00775	89.54	0.00775	117.24	0.00775	139.3611
	0.0093	22.009	0.0093	38.206	0.0093	68.5	0.0093	101.08	0.0093	134.4	0.0093	161.15734
	0.01085	23.381	0.01085	39.741	0.01085	73.258	0.01085	110.43	0.01085	149.09	0.01085	180.39957
	0.0124	24.347	0.0124	40.702	0.0124	76.713	0.0124	117.86	0.0124	161.47	0.0124	197.14779
	0.01395	25.016	0.01395	41.296	0.01395	79.184	0.01395	123.67	0.01395	171.74	0.01395	211.53401
	0.0155	25.476	0.0155	41.661	0.0155	80.931	0.0155	128.16	0.0155	180.17	0.0155	223.75221
	0.01705	25.789	0.01705	41.884	0.01705	82.156	0.01705	131.59	0.01705	187.02	0.01705	234.03839
	0.0186	26.001	0.0186	42.02	0.0186	83.012	0.0186	134.2	0.0186	192.53	0.0186	242.62256
	0.02015	26.144	0.02015	42.103	0.02015	83.606	0.02015	136.17	0.02015	196.95	0.02015	249.74271
	0.0217	26.241	0.0217	42.154	0.0217	84.019	0.0217	137.66	0.0217	200.47	0.0217	255.61684
	0.02325	26.306	0.02325	42.184	0.02325	84.304	0.02325	138.77	0.02325	203.26	0.02325	260.43896
	0.0248	26.35	0.0248	42.203	0.0248	84.501	0.0248	139.6	0.0248	205.46	0.0248	264.38506
	0.02635	26.379	0.02635	42.214	0.02635	84.637	0.02635	140.23	0.02635	207.2	0.02635	267.60115
	0.0279	26.399	0.0279	42.221	0.0279	84.731	0.0279	140.69	0.0279	208.57	0.0279	270.21923
	0.02945	26.412	0.02945	42.225	0.02945	84.796	0.02945	141.04	0.02945	209.65	0.02945	272.34529
	0.031	26.421	0.031	42.228	0.031	84.841	0.031	141.29	0.031	210.5	0.031	274.06534

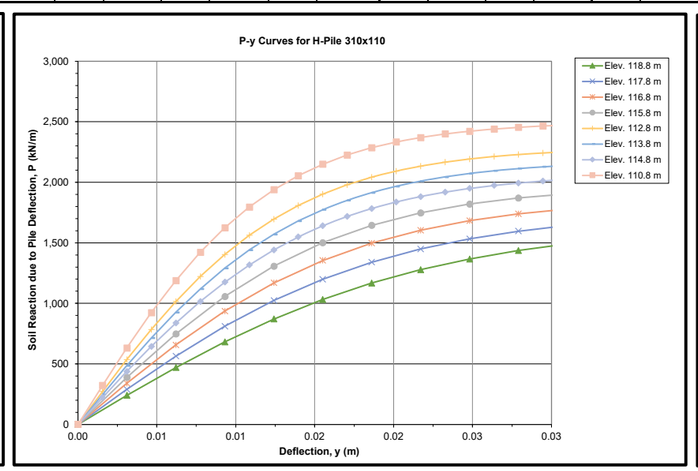
Description	Firm to Hard Silty Clay to Sandy Clayey Silt (Fill)															
Depth (z) *	z= 3.1 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 6.0 m		z= 7.0 m		z= 7.9 m	
Elevation	Elev. 127.7 m		Elev. 127.3 m		Elev. 126.8 m		Elev. 126.3 m		Elev. 125.8 m		Elev. 124.8 m		Elev. 123.8 m		Elev. 122.9 m	
P-y Curves	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
	8.7E-07	0.5713	8.7E-07	0.41015	8.7E-07	0.4687	8.7E-07	0.5273375	8.7E-07	0.5859	8.7E-07	0.7031	8.7E-07	0.820225	8.7E-07	0.9257125
	4.3E-06	2.857	4.3E-06	2.050375	4.3E-06	2.344	4.3E-06	2.63675	4.3E-06	2.9295	4.3E-06	3.5155	4.3E-06	4.1015	4.3E-06	4.62825
	8.7E-06	5.713	8.7E-06	4.1015	8.7E-06	4.687	8.7E-06	5.273375	8.7E-06	5.859	8.7E-06	7.031	8.7E-06	8.20225	8.7E-06	8.37
	4.3E-05	12.52	4.3E-05	12.52	4.3E-05	12.52	4.3E-05	12.52	4.3E-05	12.52	4.3E-05	12.52	4.3E-05	12.52	4.3E-05	12.52
	8.7E-05	14.88	8.7E-05	14.88	8.7E-05	14.88	8.7E-05	14.88	8.7E-05	14.88	8.7E-05	14.88	8.7E-05	14.88	8.7E-05	14.88
	0.00043	22.26	0.00043	22.26	0.00043	22.26	0.00043	22.26	0.00043	22.26	0.00043	22.26	0.00043	22.26	0.00043	22.26
	0.00087	26.47	0.00087	26.47	0.00087	26.47	0.00087	26.47	0.00087	26.47	0.00087	26.47	0.00087	26.47	0.00087	26.47
	0.00217	33.28	0.00217	33.28	0.00217	33.28	0.00217	33.28	0.00217	33.28	0.00217	33.28	0.00217	33.28	0.00217	33.28
	0.00289	35.76	0.00289	35.76	0.00289	35.76	0.00289	35.76	0.00289	35.76	0.00289	35.76	0.00289	35.76	0.00289	35.76
	0.00434	39.58	0.00434	39.58	0.00434	39.58	0.00434	39.58	0.00434	39.58	0.00434	39.58	0.00434	39.58	0.00434	39.58
	0.00651	43.8	0.00651	43.8	0.00651	43.8	0.00651	43.8	0.00651	43.8	0.00651	43.8	0.00651	43.8	0.00651	43.8
	0.00868	47.07	0.00868	47.07	0.00868	47.07	0.00868	47.07	0.00868	47.07	0.00868	47.07	0.00868	47.07	0.00868	47.07
	0.01085	49.77	0.01085	49.77	0.01085	49.77	0.01085	49.77	0.01085	49.77	0.01085	49.77	0.01085	49.77	0.01085	49.77
	0.01736	55.97	0.01736	55.97	0.01736	55.97	0.01736	55.97	0.01736	55.97	0.01736	55.97	0.01736	55.97	0.01736	55.97
	0.02604	61.94	0.02604	61.94	0.02604	61.94	0.02604	61.94	0.02604	61.94	0.02604	61.94	0.02604	61.94	0.02604	61.94
	0.03472	66.56	0.03472	66.56	0.03472	66.56	0.03472	66.56	0.03472	66.56	0.03472	66.56	0.03472	66.56	0.03472	66.56
	0.0434	70.38	0.0434	70.38	0.0434	70.38	0.0434	70.38	0.0434	70.38	0.0434	70.38	0.0434	70.38	0.0434	70.38
	0.05208	73.67	0.05208	73.67	0.05208	73.67	0.05208	73.67	0.05208	73.67	0.05208	73.67	0.05208	73.67	0.05208	73.67
	0.06076	76.56	0.06076	76.56	0.06076	76.56	0.06076	76.56	0.06076	76.56	0.06076	76.56	0.06076	76.56	0.06076	76.56
	0.06944	79.16	0.06944	79.16	0.06944	79.16	0.06944	79.16	0.06944	79.16	0.06944	79.16	0.06944	79.16	0.06944	79.16
	0.07812	81.52	0.07812	81.52	0.07812	81.52	0.07812	81.52	0.07812	81.52	0.07812	81.52	0.07812	81.52	0.07812	81.52
	0.0868	83.7	0.0868	83.7	0.0868	83.7	0.0868	83.7	0.0868	83.7	0.0868	83.7	0.0868	83.7	0.0868	83.7
	0.08897	83.7	0.08897	83.7	0.08897	83.7	0.08897	83.7	0.08897	83.7	0.08897	83.7	0.08897	83.7	0.08897	83.7

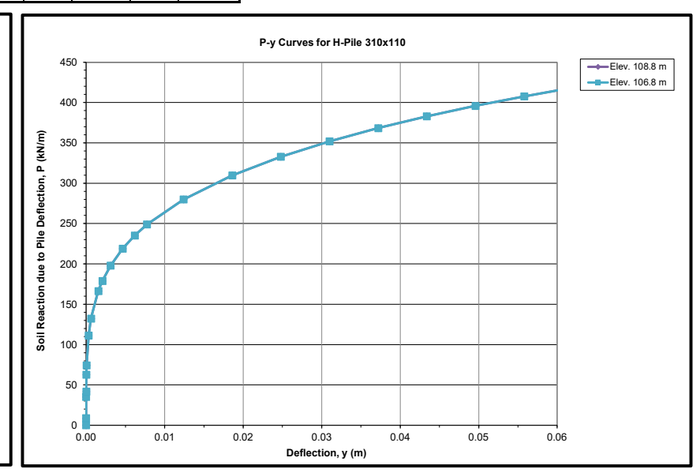
Description	Very Soft to Stiff Silty Clay with Sand to Clayey Silt							
Depth (z) *	z= 8.1 m		z= 9.0 m		z= 10.0 m		z= 11.0 m	
Elevation	Elev. 122.7 m		Elev. 121.8 m		Elev. 120.8 m		Elev. 119.8 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0	0	0	0	0	0	0	0
	0.00388	22.14	0.00388	22.14	0.003875	22.14	0.00388	22.14
	0.00775	27.9	0.00775	27.9	0.00775	27.9	0.00775	27.9
	0.01163	31.94	0.01163	31.94	0.01163	31.94	0.01163	31.94
	0.0155	35.15	0.0155	35.15	0.0155	35.15	0.0155	35.15
	0.01938	37.87	0.01938	37.87	0.01938	37.87	0.01938	37.87
	0.02325	40.24	0.02325	40.24	0.02325	40.24	0.02325	40.24
	0.02713	42.36	0.02713	42.36	0.02713	42.36	0.02713	42.36
	0.031	44.29	0.031	44.29	0.031	44.29	0.031	44.29
	0.03488	46.06	0.03488	46.06	0.03488	46.06	0.03488	46.06
	0.03875	47.71	0.03875	47.71	0.03875	47.71	0.03875	47.71
	0.04262	49.25	0.04262	49.25	0.04262	49.25	0.04262	49.25
	0.0465	50.7	0.0465	50.7	0.0465	50.7	0.0465	50.7
	0.05038	52.07	0.05038	52.07	0.05038	52.07	0.05038	52.07
	0.05425	53.37	0.05425	53.37	0.05425	53.37	0.05425	53.37
	0.05812	54.61	0.05812	54.61	0.05812	54.61	0.05812	54.61
	0.062	55.8	0.062	55.8	0.062	55.8	0.062	55.8
	0.093	55.8	0.093	55.8	0.093	55.8	0.093	55.8

Description	Very Loose to Dense Silt and Sand to Silty Sand																	
Depth (z) *	z= 12.0 m		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	
Elevation	Elev. 118.8 m		Elev. 117.8 m		Elev. 116.8 m		Elev. 115.8 m		Elev. 114.8 m		Elev. 113.8 m		Elev. 112.8 m		Elev. 111.8 m		Elev. 110.8 m	
P-y Curves	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
	0.0031	239.613	0.0031	289.19903	0.0031	338.5877	0.0031	387.79226	0.00155	220.8687	0.00155	245.9229	0.00155	270.96323	0.00155	295.98452	0.00155	320.987742
	0.0062	469.493	0.0062	563.33581	0.0062	655.7548	0.0062	746.87323	0.0031	436.8313	0.0031	485.71323	0.0031	534.45452	0.0031	583.06871	0.0031	631.568387
	0.0093	681.415	0.0093	810.32258	0.0093	935.1694	0.0093	1056.471	0.00465	643.3842	0.00465	713.81581	0.00465	783.81065	0.00465	853.41032	0.00465	922.653226
	0.0124	869.729	0.0124	1022.8419	0.0124	1168.335	0.0124	1307.3258	0.0062	836.8406	0.0062	925.77161	0.0062	1013.7742	0.0062	1100.9968	0.0062	1187.44839
	0.0155	1031.66	0.0155	1198.7387	0.0155	1354.226	0.0155	1500.6161	0.00775	1014.494	0.00775	1118.4968	0.00775	1220.8935	0.00775	1321.9097	0.00775	1421.79032
	0.0186	1167.07	0.0186	1339.4323	0.0186	1497.194	0.0186	1643.8871	0.0093	1174.629	0.0093	1290.2258	0.0093	1403.4258	0.0093	1514.7548	0.0093	1624.26774
	0.0217	1277.66	0.0217	1449.1161	0.0217	1604.074	0.0217	1747.0129	0.01085	1316.768	0.01085	1440.5774	0.01085	1561.4	0.01085	1679.6516	0.01085	1795.66129
	0.0248	1366.2	0.0248	1532.9677	0.0248	1682.219	0.0248	1819.6806	0.0124	1441.035	0.0124	1570.2129	0.0124	1695.7871	0.0124	1818.2097	0.0124	1938.09032
	0.0279	1435.97	0.0279	1595.9806	0.0279	1738.665	0.0279	1870.1194	0.01395	1548.206	0.01395	1680.4903	0.01395	1808.5452	0.01395	1933.0484	0.01395	2054.75484
	0.031	1490.44	0.031	1642.7548	0.031	1778.826	0.031	1904.8161	0.0155	1639.855	0.0155	1773.229	0.0155	1902.0452	0.0155	2027.1484	0.0155	2149.15484
	0.0341	1532.33	0.0341	1677.1677	0.0341	1807.255	0.0341	1928.5129	0.01705	1717.255	0.01705	1850.5097	0.01705	1978.8452	0.01705	2103.3484	0.01705	2224.8
	0.0372	1564.5	0.0372	1702.3806	0.0372	1827.184	0.0372	1944.5677	0.0186	1782.281	0.0186	1914.2903	0.0186	2041.4161	0.0186	2164.7194	0.0186	2284.99032
	0.0403	1588.98	0.0403	1720.771	0.0403	1841.181	0.0403	1955.4516	0.02015	1836.458	0.02015	1966.7129	0.02015	2092.0871	0.02015	2213.7	0.02015	2332.48065
	0.0434	1607.55	0.0434	1734.0387	0.0434	1850.945	0.0434	1962.7645	0.0217	1881.361	0.0217	2009.4548	0.0217	2132.8935	0.0217	2252.771	0.0217	2369.87097
	0.0465	1621.5	0.0465	1743.7065	0.0465	1857.71	0.0465	1967.7065	0.02325	1918.439	0.02325	2044.1968	0.02325	2165.5355	0.02325	2283.6419	0.02325	2399.16129
	0.0496	1632.15	0.0496	1750.6516	0.0496	1862.458	0.0496	1971.0484	0.0248	1948.942	0.0248	2072.4194	0.0248	2191.7065	0.0248	2308.0226	0.0248	2421.99677
	0.0527	1640.09	0.0527	1755.6968	0.0527	1865.79	0.0527	1973.3194	0.02635	1973.845	0.02635	2095.0806	0.02635	2212.5129	0.02635	2327.2032	0.02635	2439.7871
	0.0558	1646.02	0.0558	1759.3194	0.0558	1868.106	0.0558	1974.7903	0.0279	1994.248	0.0279	2113.4226	0.0279	2229.0839	0.0279	2342.2839	0.0279	2453.63226
	0.0589	1650.55	0.0589	1761.9419	0.0589	1869.639	0.0589	1975.8613	0.02945	2010.926	0.02945	2128.1645	0.02945	2242.1903	0.02945	2354.0645	0.02945	2464.42258
	0.062	1653.87	0.062	1763.7645	0.062	1870.739	0.062	1976.5323	0.031	2024.503	0.031	2139.9258	0.031	2252.6613	0.031	2363.3548	0.031	2472.7129

Description	Stiff to Hard Silty Clay to Clayey Silt				Loose to Dense Silt and Sand to Silty Sand, Bedrock					
Depth (z) *	z= 22.0 m		z= 24.0 m		z= 26.0 m		z= 28.0 m		z= 30.0 m	
Elevation	Elev. 108.8 m		Elev. 106.8 m		Elev. 104.8 m		Elev. 102.8 m		Elev. 100.8 m	
P-y Curves	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)	P (kN/m)
	0	0	0	0	0	0	0	0	0	0
	6.2E-07	8.6276471	6.2E-07	8.962375	0.0031	861.24522	0.0031	959.87494	0.00155	535.29367
	3.1E-06	35.19	3.1E-06	35.19	0.0062	1658.3522	0.0062	1842.4342	0.0031	1058.4316
	6.2E-06	41.85	6.2E-06	41.85	0.0093	2344.9013	0.0093	2593.2924	0.00465	1558.2241
	3.1E-05	62.58	3.1E-05	62.58	0.0124	2900.6761	0.0124	3191.2544	0.0062	2025.7114
	6.2E-05	74.42	6.2E-05	74.42	0.0155	3328.2767	0.0155	3642.6506	0.00775	2454.157
	0.00031	111.3	0.00031	111.3	0.0186	3644.656	0.0186	3969.9962	0.0093	2839.6608
	0.00062	132.3	0.00062	132.3	0.0217	3872.0403	0.0217	4200.5063	0.01085	3180.8595
	0.00155	166.4	0.00155	166.4	0.0248	4032.0403	0.0248	4359.5013	0.0124	3478.3481
	0.00207	178.8	0.00207	178.8	0.0279	4143.0138	0.0279	4467.4911	0.01395	3734.4316
	0.0031	197.9	0.0031	197.9	0.031	4219.0874	0.031	4540.2759	0.0155	3952.4785
	0.00465	219	0.00465	219	0.0341	4270.961	0.0341	4588.8658	0.01705	4136.4203
	0.0062	235.3	0.0062	235.3	0.0372	4306.1767	0.0372	4621.2557	0.0186	4290.3253
	0.00775	248.8	0.00775	248.8	0.0403	4329.9503	0.0403	4642.7456	0.02015	4418.1937
	0.0124	279.9	0.0124	279.9	0.0434	4345.966	0.0434	4656.9456	0.0217	4523.862
	0.0186	309.7	0.0186	309.7	0.0465	4356.766	0.0465	4666.3405	0.02325	4610.862
	0.0248	332.8	0.0248	332.8	0.0496	4364.0239	0.0496	4672.6304	0.0248	4682.162
	0.031	351.9	0.031	351.9	0.0527	4368.8818	0.0527	4676.7304	0.02635	4740.3937
	0.0372	368.3	0.0372	368.3	0.0558	4372.1818	0.0558	4679.4304	0.0279	4787.9253
	0.0434	382.8	0.0434	382.8	0.0589	4374.3396	0.0589	4681.2304	0.02945	4826.557
	0.0496	395.8	0.0496	395.8	0.062	4375.8396	0.062	4682.4253	0.031	4857.8886
	0.0558	407.6	0.0558	407.6						
	0.062	418.5	0.062	418.5						
	0.06355	418.5	0.06355	418.5						







NOTES:

* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 130.8 m).

The P-y curves have been generated based on the following assumptions:

1 P-y curves are generated for vertical piles (i.e. no inclination)

2 Static loading condition is considered.

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

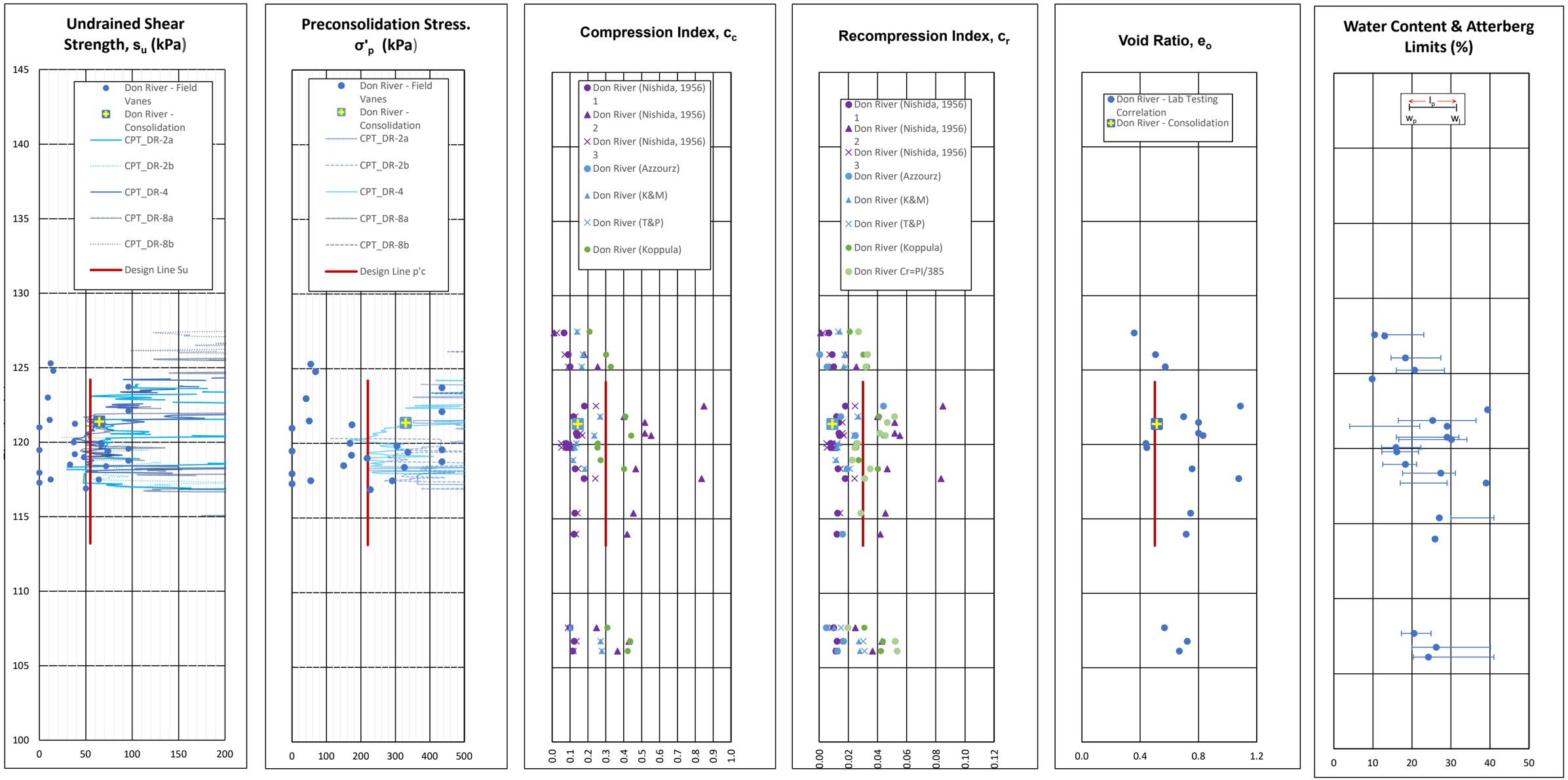
Date: September 2021 **Prepared By:** CC/ACK

Project No: 1786302 **Checked By:** CN

GOLDER

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR SOFT TO HARD CLAY (CH) to SANDY CLAYEY SILT-SILT (CL-ML)
Highway 401 Widening at Leslie Street
Station 25+880 to 26+550 & 9+980 to 10+000

FIGURE 5



Date: 2021-10-01
Project No: 1786302 - 2000

Prepared By: ACK
Checked By: CN

GOLDER MEMBER OF WSP

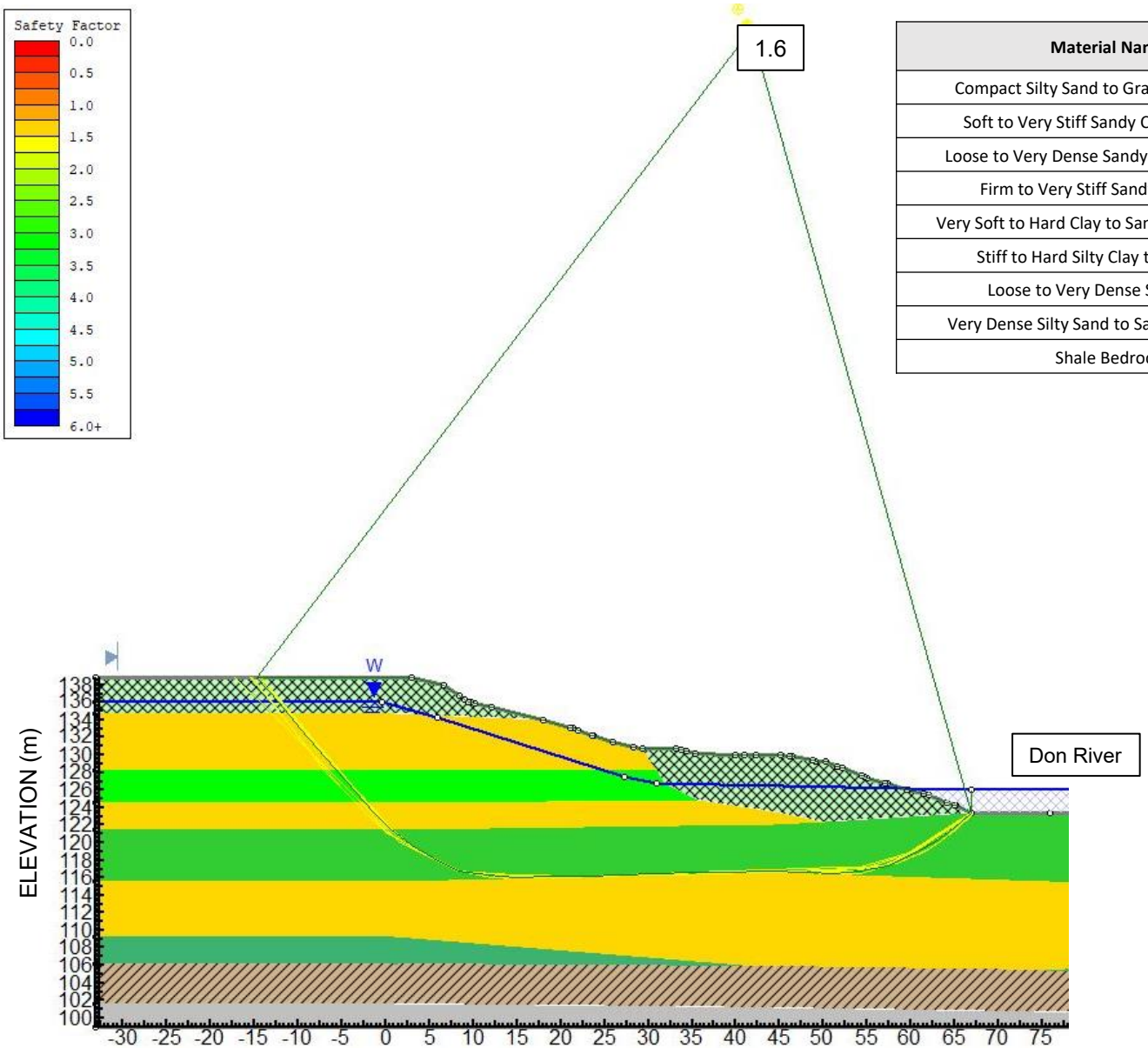
Golder Associates Ltd.

https://golderassociates.sharepoint.com/sites/20393g/Technical Work/Foundations/06 - Analysis/0.0 Material Parameters/[1786302 - Material Parameters Grouped 2020Jul08.xlsx]PLOTS - AREA 3 (H. Fill-Don Ri)

West Approach Embankment (East-West Direction)

Static Slope Stability Analysis – Temporary Conditions

Figure 6



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Compact Silty Sand to Gravelly Sand (Fill)		21	Mohr-Coulomb	0	36
Soft to Very Stiff Sandy Clayey Silt (Fill)		21	Undrained	50	
Loose to Very Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	35
Firm to Very Stiff Sandy Clayey Silt		21	Undrained	75	
Very Soft to Hard Clay to Sandy Clayey Silt-Silt		21	Undrained	55	
Stiff to Hard Silty Clay to Clayey Silt		21	Undrained	150	
Loose to Very Dense Silt to Sand		20	Mohr-Coulomb	0	36
Very Dense Silty Sand to Sandy Gravel (Till)		22	Mohr-Coulomb	0	37
Shale Bedrock		20	Infinite strength		

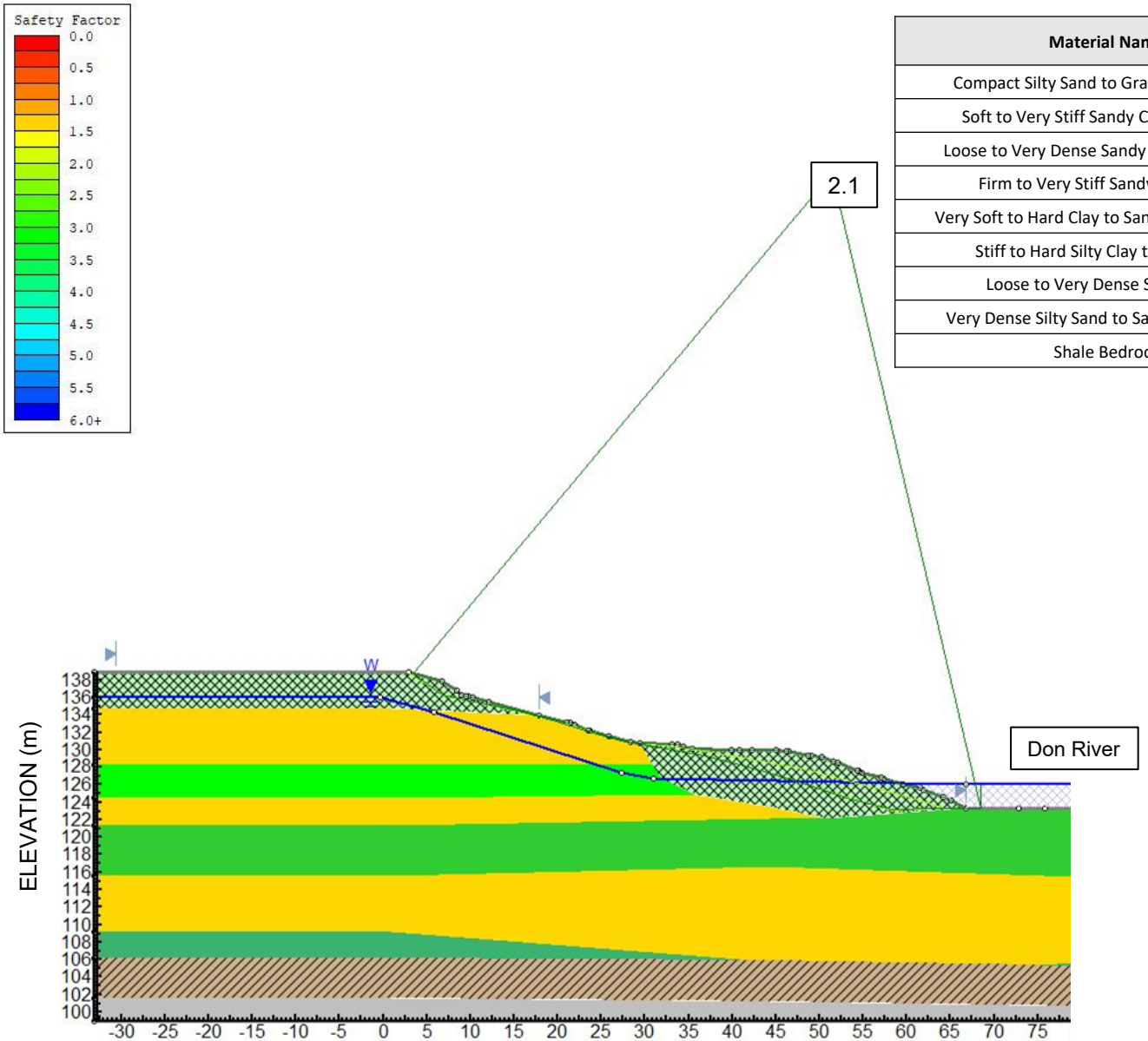
Date: August 2021
Project No: 1786302

Prepared By: KNN
Reviewed By: CN

West Approach Embankment (East-West Direction)

Static Slope Stability Analysis – Permanent Conditions

Figure 7



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Compact Silty Sand to Gravelly Sand (Fill)		21	Mohr-Coulomb	0	36
Soft to Very Stiff Sandy Clayey Silt (Fill)		21	Mohr-Coulomb	0	28
Loose to Very Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	35
Firm to Very Stiff Sandy Clayey Silt		21	Mohr-Coulomb	0	32
Very Soft to Hard Clay to Sandy Clayey Silt-Silt		21	Mohr-Coulomb	0	33
Stiff to Hard Silty Clay to Clayey Silt		21	Mohr-Coulomb	0	33
Loose to Very Dense Silt to Sand		20	Mohr-Coulomb	0	36
Very Dense Silty Sand to Sandy Gravel (Till)		22	Mohr-Coulomb	0	37
Shale Bedrock		20	Infinite strength		

NOTE: FOS ≥ 1.5 for shallow slip surfaces and deep-seated global slip surfaces

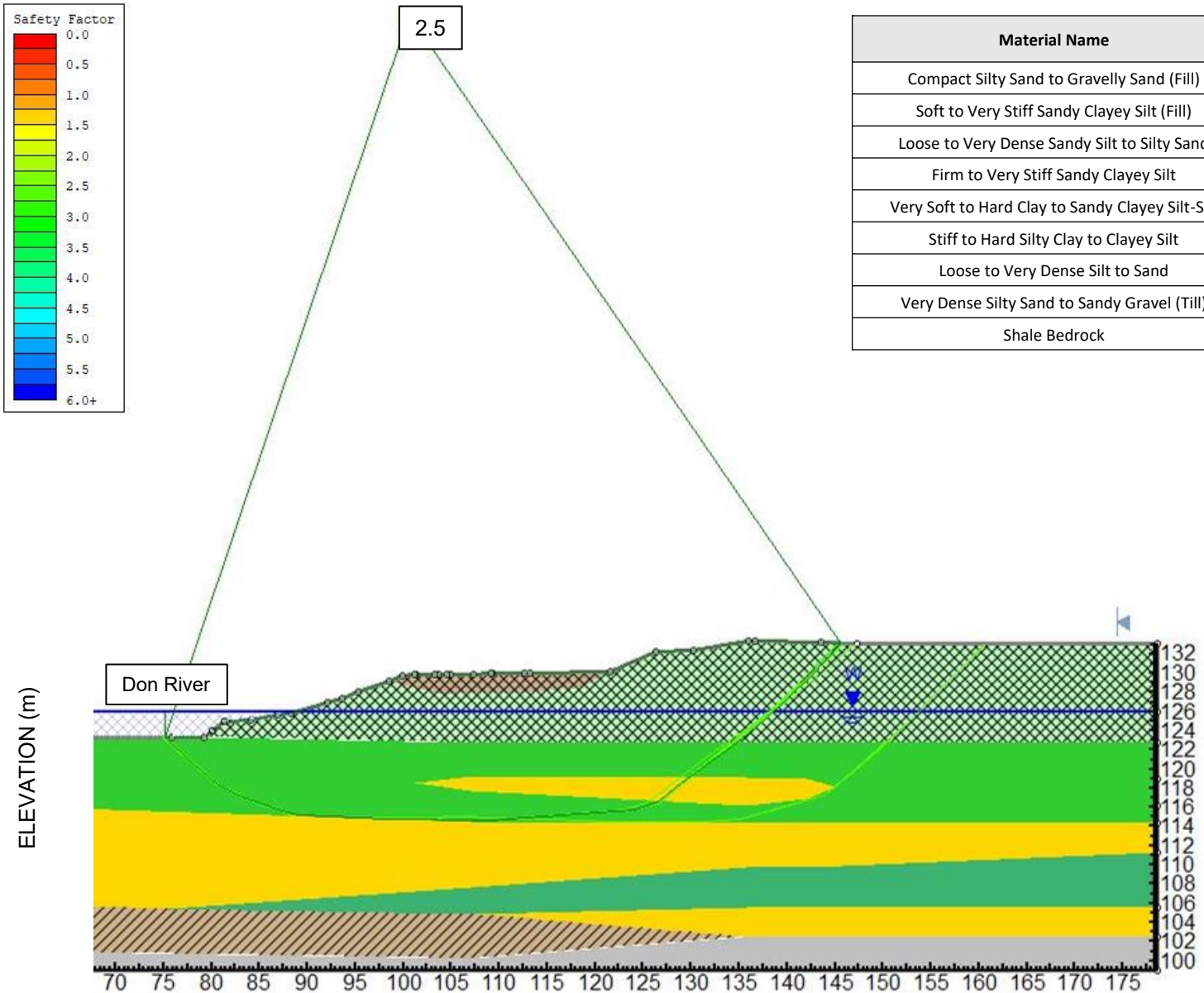
Date: August 2021
Project No: 1786302

Prepared By: KNN
Reviewed By: CN

East Approach Embankment (East-West Direction)

Static Slope Stability Analysis – Temporary Conditions

Figure 8



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Compact Silty Sand to Gravelly Sand (Fill)		21	Mohr-Coulomb	0	36
Soft to Very Stiff Sandy Clayey Silt (Fill)		21	Undrained	50	
Loose to Very Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	35
Firm to Very Stiff Sandy Clayey Silt		21	Undrained	75	
Very Soft to Hard Clay to Sandy Clayey Silt-Silt		21	Undrained	55	
Stiff to Hard Silty Clay to Clayey Silt		21	Undrained	150	
Loose to Very Dense Silt to Sand		20	Mohr-Coulomb	0	36
Very Dense Silty Sand to Sandy Gravel (Till)		22	Mohr-Coulomb	0	37
Shale Bedrock		20	Infinite strength		

Date: August 2021
Project No: 1786302

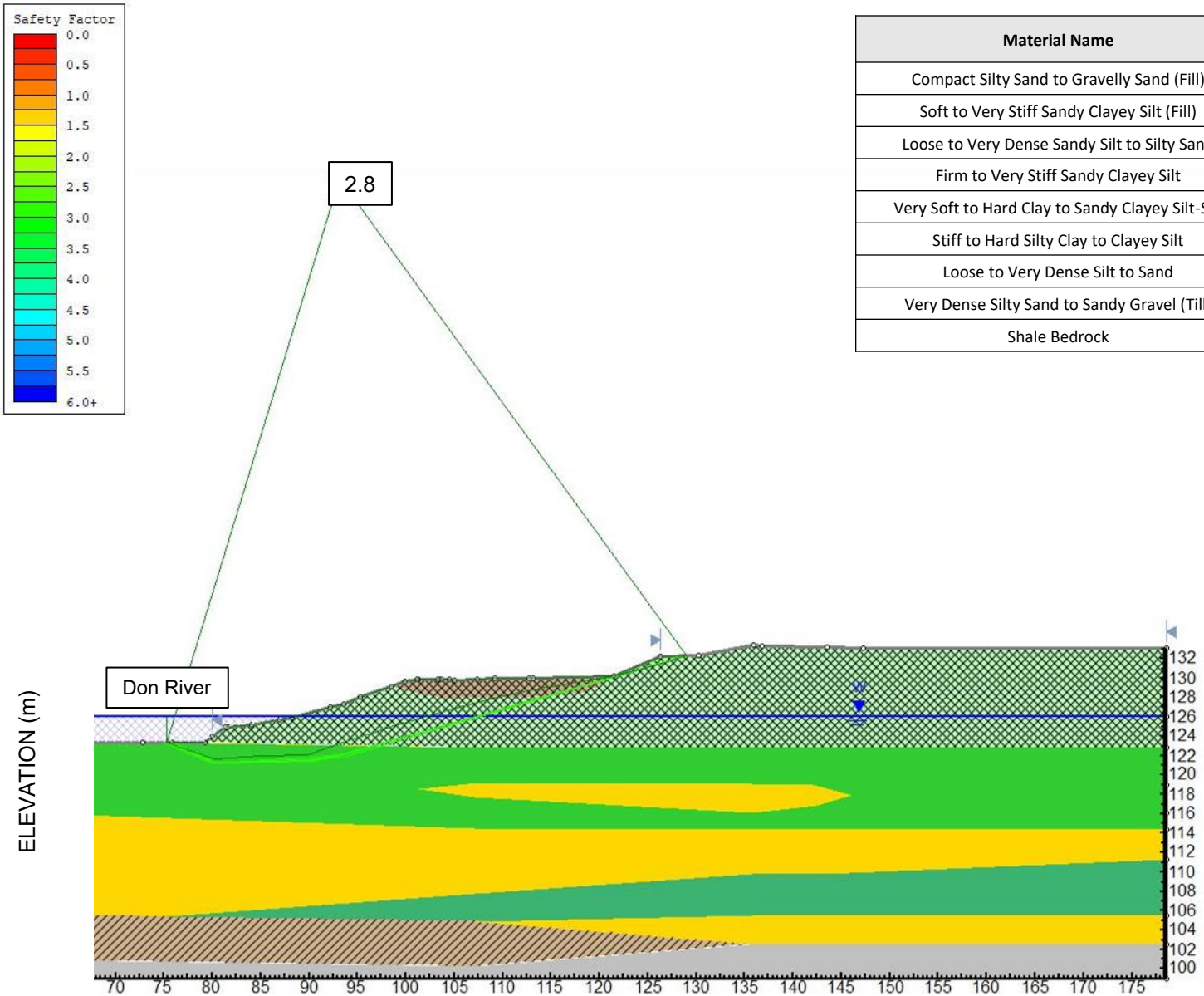
Prepared By: KNN
Reviewed By: CN

East Approach Embankment (East-West Direction)

Static Slope Stability Analysis – Permanent Conditions

Figure 9

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Compact Silty Sand to Gravelly Sand (Fill)		21	Mohr-Coulomb	0	36
Soft to Very Stiff Sandy Clayey Silt (Fill)		21	Mohr-Coulomb	0	28
Loose to Very Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	35
Firm to Very Stiff Sandy Clayey Silt		21	Mohr-Coulomb	0	32
Very Soft to Hard Clay to Sandy Clayey Silt-Silt		21	Mohr-Coulomb	0	33
Stiff to Hard Silty Clay to Clayey Silt		21	Mohr-Coulomb	0	33
Loose to Very Dense Silt to Sand		20	Mohr-Coulomb	0	36
Very Dense Silty Sand to Sandy Gravel (Till)		22	Mohr-Coulomb	0	37
Shale Bedrock		20	Infinite strength		



NOTE: FOS ≥ 1.5 for shallow slip surfaces and deep-seated global slip surfaces

Date: August 2021
Project No: 1786302

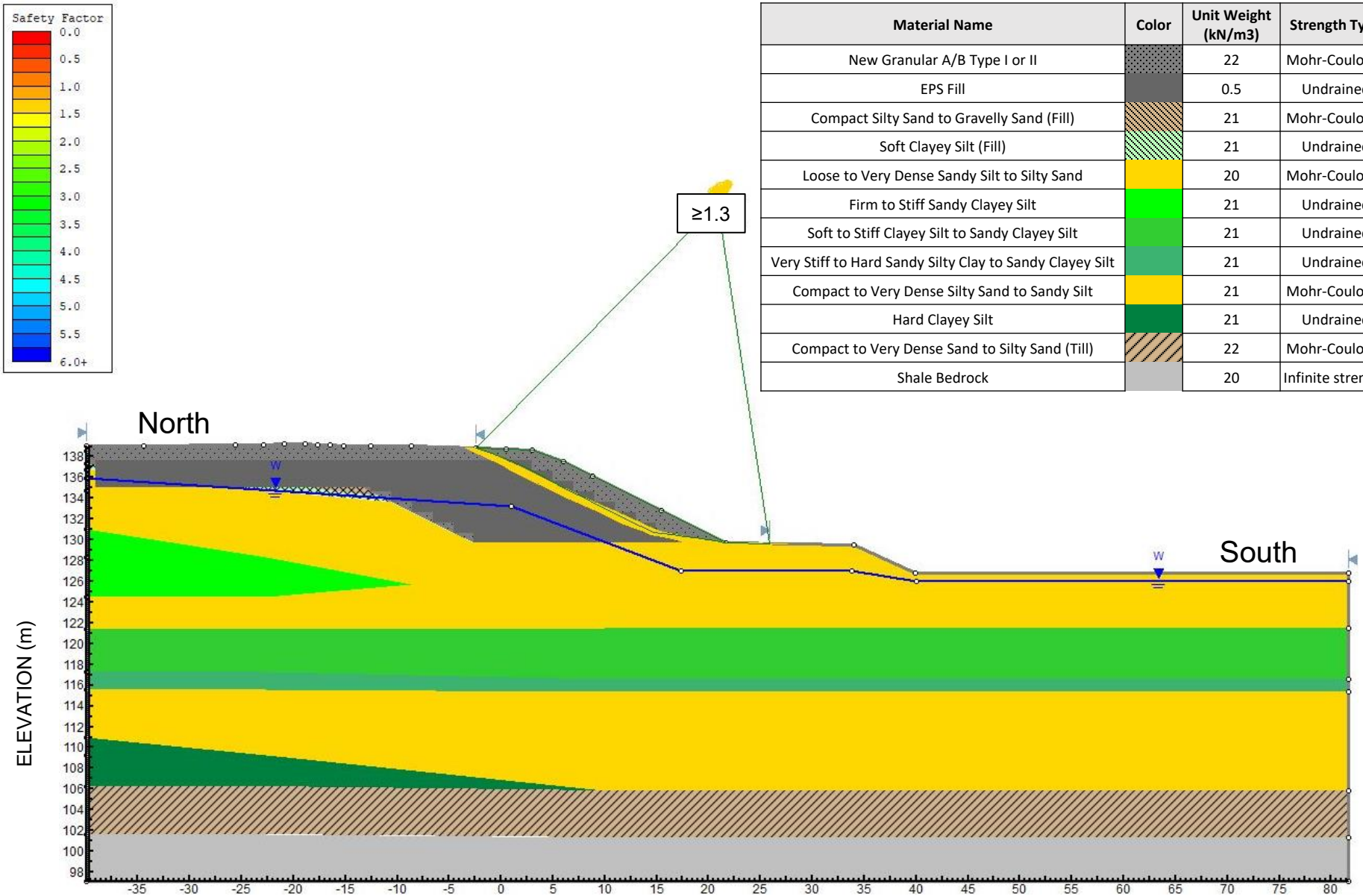
Prepared By: KNN
Reviewed By: CN

West Approach Embankment (North-South Direction)

Static Slope Stability Analysis – Temporary Conditions

Figure 10

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Granular A/B Type I or II		22	Mohr-Coulomb	0	38
EPS Fill		0.5	Undrained	14	
Compact Silty Sand to Gravelly Sand (Fill)		21	Mohr-Coulomb	0	36
Soft Clayey Silt (Fill)		21	Undrained	50	
Loose to Very Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	35
Firm to Stiff Sandy Clayey Silt		21	Undrained	75	
Soft to Stiff Clayey Silt to Sandy Clayey Silt		21	Undrained	55	
Very Stiff to Hard Sandy Silty Clay to Sandy Clayey Silt		21	Undrained	150	
Compact to Very Dense Silty Sand to Sandy Silt		21	Mohr-Coulomb	0	36
Hard Clayey Silt		21	Undrained	200	
Compact to Very Dense Sand to Silty Sand (Till)		22	Mohr-Coulomb	0	37
Shale Bedrock		20	Infinite strength		



Date: August 2021
Project No: 1786302

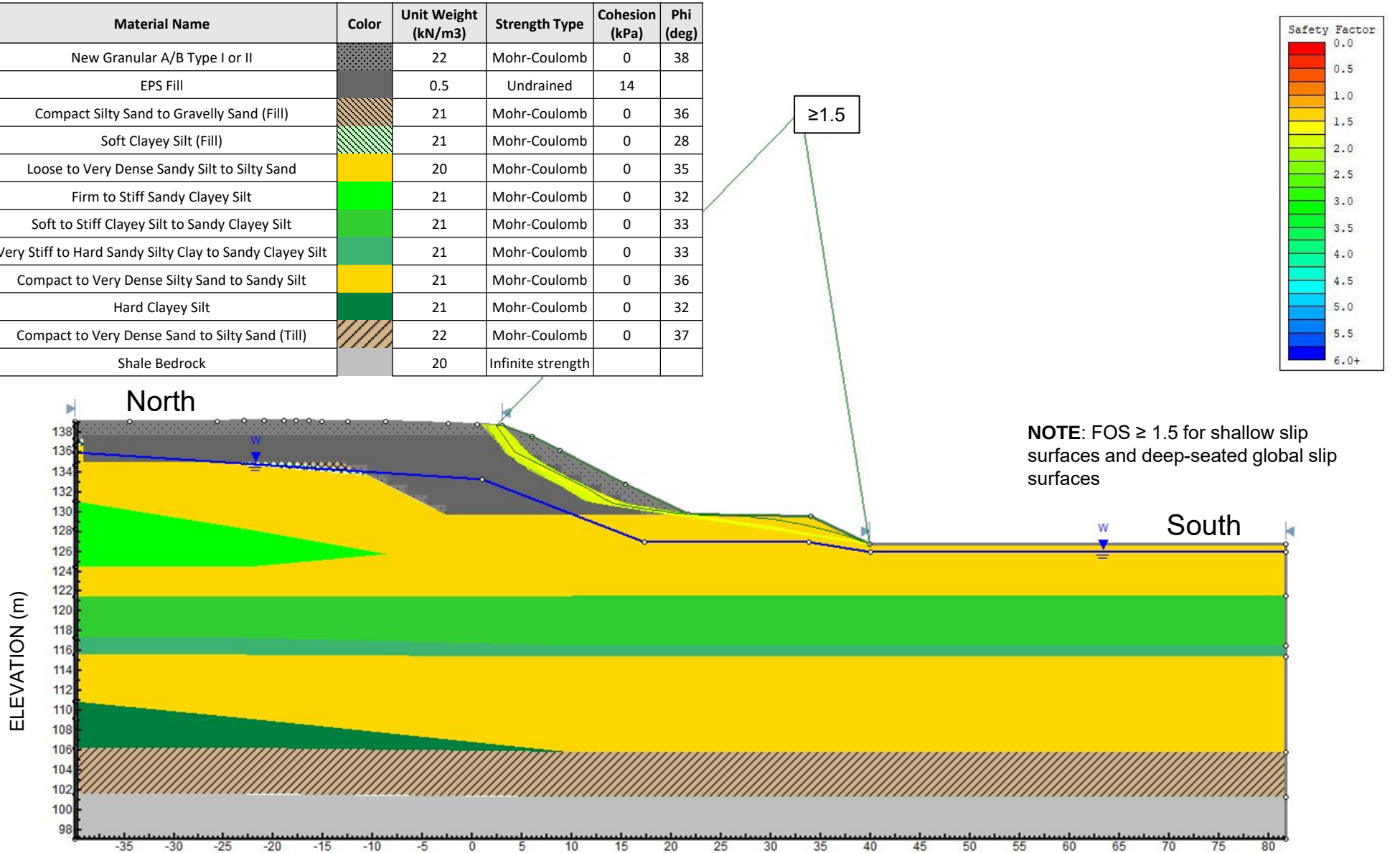
Prepared By: KNN
Reviewed By: CN

West Approach Embankment (North-South Direction)

Static Slope Stability Analysis – Permanent Conditions

Figure 11

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Granular A/B Type I or II		22	Mohr-Coulomb	0	38
EPS Fill		0.5	Undrained	14	
Compact Silty Sand to Gravelly Sand (Fill)		21	Mohr-Coulomb	0	36
Soft Clayey Silt (Fill)		21	Mohr-Coulomb	0	28
Loose to Very Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	35
Firm to Stiff Sandy Clayey Silt		21	Mohr-Coulomb	0	32
Soft to Stiff Clayey Silt to Sandy Clayey Silt		21	Mohr-Coulomb	0	33
Very Stiff to Hard Sandy Silty Clay to Sandy Clayey Silt		21	Mohr-Coulomb	0	33
Compact to Very Dense Silty Sand to Sandy Silt		21	Mohr-Coulomb	0	36
Hard Clayey Silt		21	Mohr-Coulomb	0	32
Compact to Very Dense Sand to Silty Sand (Till)		22	Mohr-Coulomb	0	37
Shale Bedrock		20	Infinite strength		



NOTE: FOS ≥ 1.5 for shallow slip surfaces and deep-seated global slip surfaces

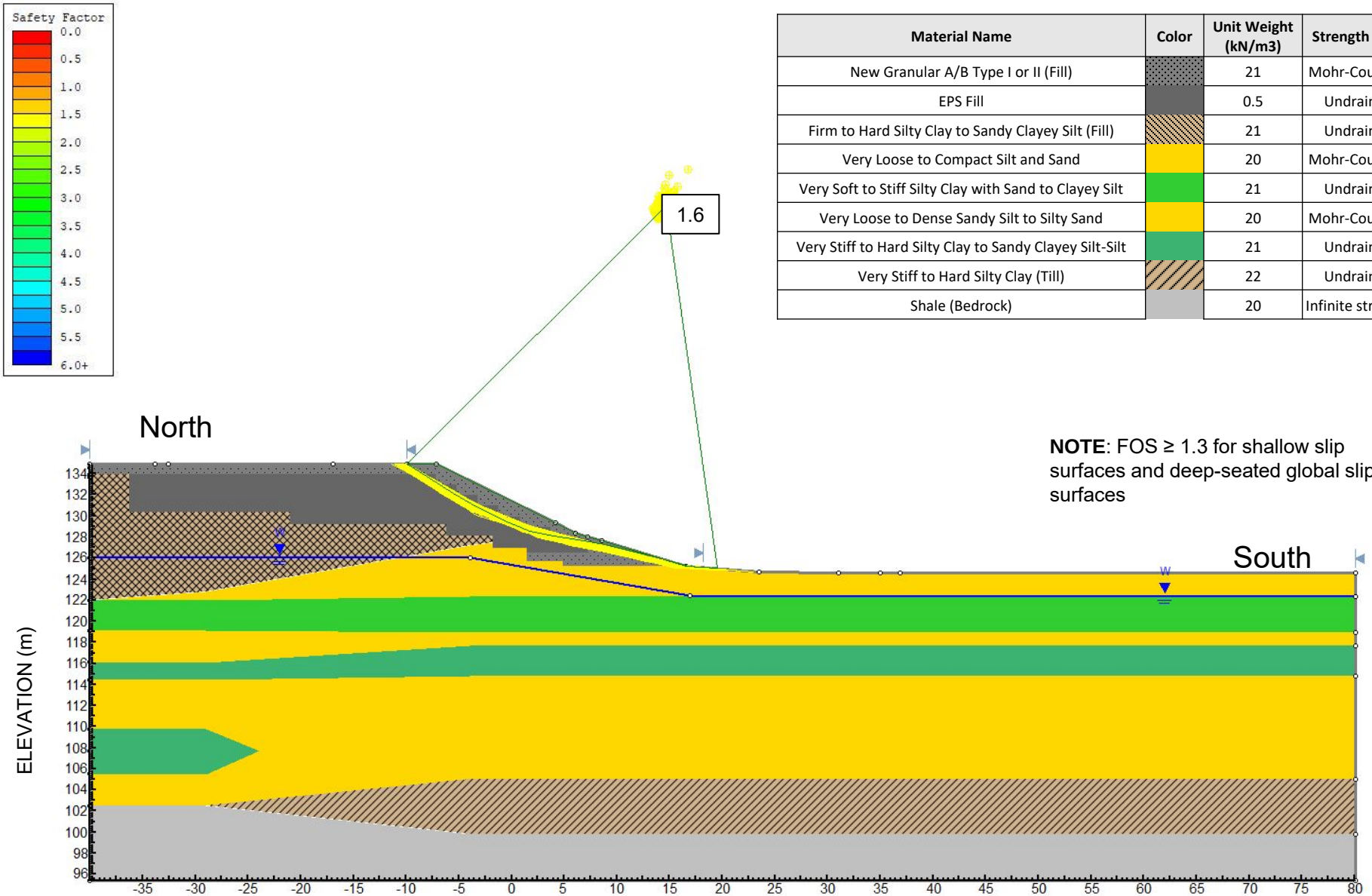
Date: August 2021
Project No: 1786302

Prepared By: KNN
Reviewed By: CN

East Approach Embankment (North-South Direction)

Static Slope Stability Analysis – Temporary Conditions

Figure 12



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Granular A/B Type I or II (Fill)		21	Mohr-Coulomb	0	38
EPS Fill		0.5	Undrained	14	
Firm to Hard Silty Clay to Sandy Clayey Silt (Fill)		21	Undrained	50	
Very Loose to Compact Silt and Sand		20	Mohr-Coulomb	0	35
Very Soft to Stiff Silty Clay with Sand to Clayey Silt		21	Undrained	55	
Very Loose to Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	36
Very Stiff to Hard Silty Clay to Sandy Clayey Silt-Silt		21	Undrained	200	
Very Stiff to Hard Silty Clay (Till)		22	Undrained	200	
Shale (Bedrock)		20	Infinite strength		

NOTE: FOS ≥ 1.3 for shallow slip surfaces and deep-seated global slip surfaces

Date: August 2021
Project No: 1786302

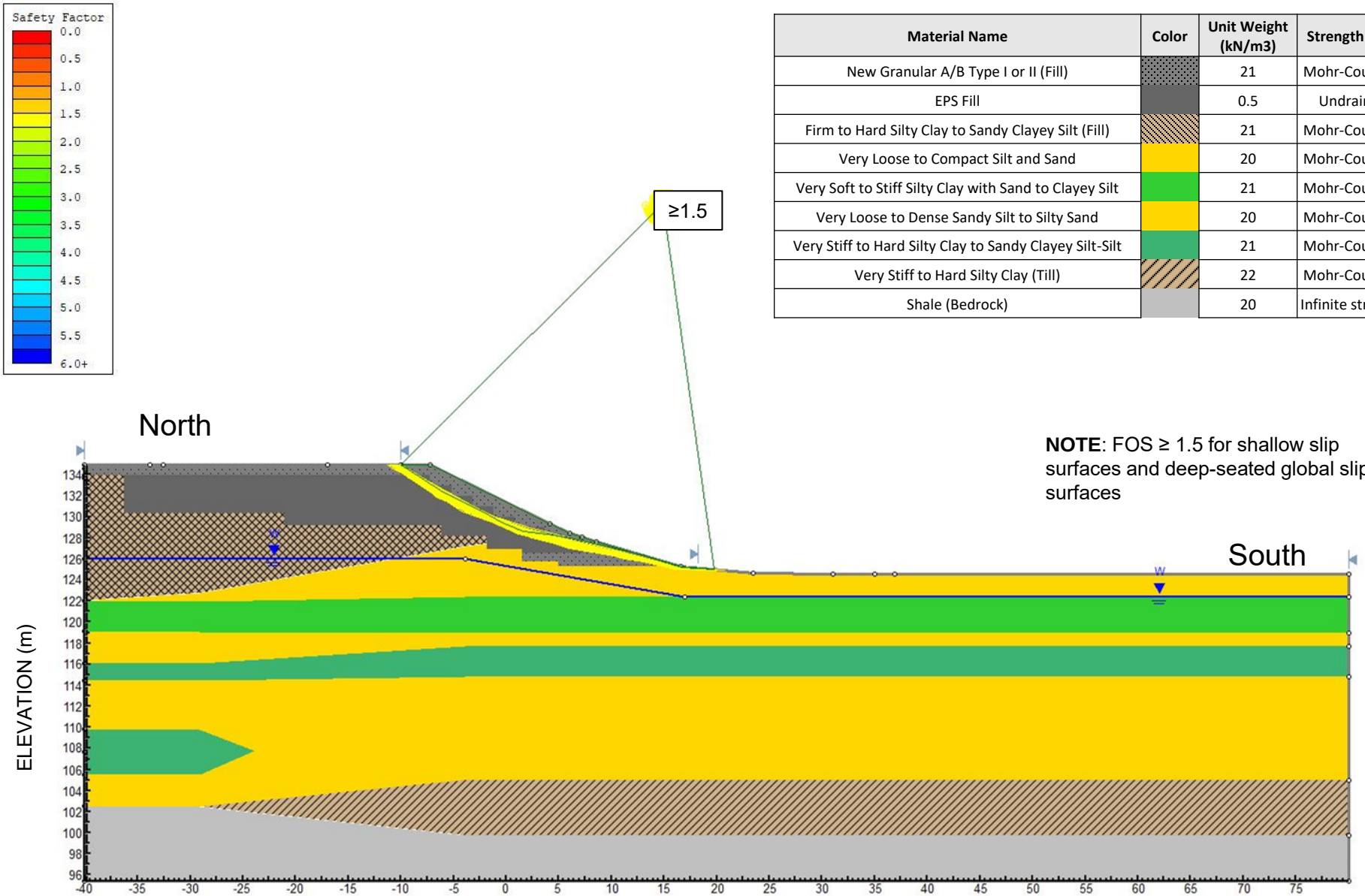
Prepared By: KNN
Reviewed By: CN

East Approach Embankment (North-South Direction)

Static Slope Stability Analysis – Permanent Conditions

Figure 13

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Granular A/B Type I or II (Fill)		21	Mohr-Coulomb	0	38
EPS Fill		0.5	Undrained	14	
Firm to Hard Silty Clay to Sandy Clayey Silt (Fill)		21	Mohr-Coulomb	0	28
Very Loose to Compact Silt and Sand		20	Mohr-Coulomb	0	35
Very Soft to Stiff Silty Clay with Sand to Clayey Silt		21	Mohr-Coulomb	0	33
Very Loose to Dense Sandy Silt to Silty Sand		20	Mohr-Coulomb	0	36
Very Stiff to Hard Silty Clay to Sandy Clayey Silt-Silt		21	Mohr-Coulomb	0	32
Very Stiff to Hard Silty Clay (Till)		22	Mohr-Coulomb	0	31
Shale (Bedrock)		20	Infinite strength		



NOTE: FOS ≥ 1.5 for shallow slip surfaces and deep-seated global slip surfaces

Date: August 2021
Project No: 1786302

Prepared By: KNN
Reviewed By: CN

APPENDIX A

Previous Investigation (MTO GEOCRES No. 30M14-462)

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

C_{pen} 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

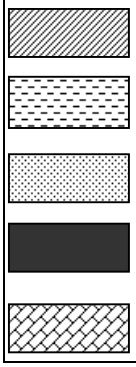
ROCK WEATHERING CLASSIFICATION

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)**	**(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			

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-03

METRIC

W.P.	2061-13-00	LOCATION	Don River Bridge N 4 847 311.3 E 316 033.9	ORIGINATED BY	ES
HWY	401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.04.27 - 2015.04.29	CHECKED BY	RPR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
130.2	GROUND SURFACE
0.0	**TOPSOIL:** (175mm)
0.2	**SAND** and **SILT**, some clay, trace gravel, occasional sand and gravelly sand lenses, occasional cobbles Compact Brown Moist
128.0	
2.2	Dense
127.2	
3.0	Loose
121.5	
8.7	Silty **CLAY**, with sand, trace gravel Firm Grey Wet

STRAT PLOT

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	21
2	SS	22
3	SS	31
4	SS	28
5	SS	22
6	SS	17
7	SS	9
8	SS	4

GROUND WATER CONDITIONS

ELEVATION SCALE: 130, 129, 128, 127, 126, 125, 124, 123, 122, 121

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR	SA	SI	CL
	5	46	36	13
	2	28	27	43

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 15 10 5 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-03

METRIC

W.P.	2061-13-00	LOCATION	Don River Bridge N 4 847 311.3 E 316 033.9	ORIGINATED BY	ES
HWY	401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.04.27 - 2015.04.29	CHECKED BY	RPR

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | SHEAR STRENGTH kPa | WATER CONTENT (%) | UNIT WEIGHT | REMARKS & GRAIN SIZE DISTRIBUTION (%)

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

WATER CONTENT (%): PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page								
	Silty **CLAY**, with sand, trace gravel Firm to Hard Grey Wet								
		1	TW	PH					
		9	SS	6		0	15	44	41
		10	SS	24					
	Becoming sandy, very stiff below 13m depth								
115.4 / 14.8	Sandy **SILT** to Silty **SAND**, trace clay, trace gravel, occasional clay lense, occasional cobbles Compact to Very Dense Grey Moist								
		11	SS	14					
		12	SS	40					
		13	SS	53		0	32	64	4

ELEVATION SCALE: 120, 119, 118, 117, 116, 115, 114, 113, 112, 111

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 20 40 60 80 100

WATER CONTENT (%): 20 40 60

Plotted values: 3.0, 2.7, 3.0

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-03

METRIC

W.P. 2061-13-00 | LOCATION Don River Bridge N 4 847 311.3 E 316 033.9 | ORIGINATED BY ES

HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2015.04.27 - 2015.04.29 | CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Sandy **SILT** to Silty **SAND**, trace clay, trace gravel, occasional clay lense, occasional cobbles Very Dense Grey Moist		14	SS	63		110				
							109				
							108				
			15	SS	61		107				
105.8							106				
24.4	**SAND** to Silty **SAND**, some gravel, some silt, trace to some clay, occasional limestone fragments Compact Grey Moist (TILL)						105				
			16	SS	23		104				13 68 19 (SI+CL)
							103				
							102				
101.3											
28.9	**SHALE**, highly to moderately weathered, thinly bedded, weak, grey: (Georgian Bay Formation) Broken core (150mm) at 29.1m		17	SS	100/					FI	
			1	RUN	0.050		101			>25 >25 5	RUN #1 TCR=100% SCR=27% RQD=0%

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-03

METRIC

W.P. 2061-13-00 | LOCATION Don River Bridge N 4 847 311.3 E 316 033.9 | ORIGINATED BY ES

HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2015.04.27 - 2015.04.29 | CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
	Continued From Previous Page										
	Sub-vertical fracrture (25mm to 50mm) at 29.3m, 29.4m, 29.5m, 29.6m and 29.7m		2	RUN			100			5	RUN #2 TCR=100% SCR=93% RQD=83%
	Broken core (50mm) at 29.4m and 29.6m									3	
	Sub-vertical fracture (25mm to 50mm) at 31.0m, 31.2m and 31.7m									1	
										6	
	Sub-horizontal fracture (25mm) at 31.2m and 31.4m		3	RUN			99			5	RUN #3 TCR=100% SCR=94% RQD=94%
98.4										3	
31.8	END OF BOREHOLE AT 31.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.										

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Jun 03/2015	0.5*	130.7
Jun 17/2015	3.5	126.7
Mar 20/2017	Found destoyed	

* Above ground surface

+ 3, × 3 : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-04

METRIC

W.P.	2061-13-00	LOCATION	Don River Bridge N 4 847 327.5 E 316 072.7	ORIGINATED BY	ES
HWY	401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.04.30 - 2015.05.04	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
129.2	GROUND SURFACE							
0.0	**TOPSOIL:** (125mm)							
0.1	**SAND** and **SILT**, trace to some clay, trace to some gravel, occasional cobbles Compact to Dense Brown Moist	1	SS	32				
		2	SS	28				
		3	SS	40				
		4	SS	21				
125.0								
4.2	Silty **SAND** Very Loose	5	SS	2	14	41	32	13
124.0								
5.2		6	SS	13				
	Possible cobbles at 7.6m depth	7	SS	50/ 0.100				
120.7								
8.5	Silty **CLAY**, with sand, trace gravel Firm to Stiff Grey Moist to Wet	8	SS	4				

GROUND WATER CONDITIONS

ELEVATION SCALE: 129, 128, 127, 126, 125, 124, 123, 122, 121, 120

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 3/20/17

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-04

METRIC

W.P. 2061-13-00 | LOCATION Don River Bridge N 4 847 327.5 E 316 072.7 | ORIGINATED BY ES

HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2015.04.30 - 2015.05.04 | CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	Continued From Previous Page													
	Silty **CLAY**, with sand, trace gravel Stiff Grey Wet						119	2.8						
			9	SS	9		118				1	34	45	20
								3.3						
116.4			1	TW	PH		117							
12.8	**SAND**, some silt, trace to some gravel, trace clay, occasional sand seams Compact to Very Dense Grey Wet						116							
			10	SS	20		115							
			11	SS	77		114				0	81	15	4
			12	SS	27		113							
			13	SS	79		112							
							111							
							110							

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-04

METRIC

W.P. 2061-13-00 | LOCATION Don River Bridge N 4 847 327.5 E 316 072.7 | ORIGINATED BY ES

HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2015.04.30 - 2015.05.04 | CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
	Continued From Previous Page										
	SAND, some silt, trace to some gravel, trace clay, occasional sand seams Compact to Very Dense Grey Wet		14	SS	22		109		○		
							108				
			15	SS	46				○		18 72 10 (SI+CL)
							107				
	Occasional cobbles below 22m depth		16	SS	53		106		○		
							105				
103.9							104				
25.3	Silty **SAND**, trace to some gravel, trace clay, occasional cobbles Very Dense Grey Moist (TILL)		17	SS	103/ 0.200		103		○		10 50 31 9
	Coring below 26.7m depth Occasional limestone fragments		1	RUN			102				
	Shale layer (100mm) at 27.2m		2	RUN			101				
100.4	Shale layer (75mm) at 28.7m									FI	
28.8	**SHALE**, moderately weathered to fresh, thinly bedded, horizontally laminated, weak, grey: (Georgain Bay Formation) Sub-vertical fracure at 29.2m, 29.3m, 29.8m and 29.9m		3	RUN			100			5 7 5 5	RUN #3 TCR=100% SCR=95% RQD=92% UCS=16MPa (Average)

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No DRB-04

METRIC

W.P.	2061-13-00	LOCATION	Don River Bridge N 4 847 327.5 E 316 072.7	ORIGINATED BY	ES
HWY	401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.04.30 - 2015.05.04	CHECKED BY	RPR

SOIL PROFILE ELEV/DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Limestone interbeds (25mm to 50mm) at 29.2m, 29.5m, 29.8m, 29.9m and 30.1m						99			2	
	Becoming slightly weathered at 30.3m									2	
	Limestone interbeds (25mm to 75mm) at 30.3m, 30.9m, 31.2m, 31.4m, 31.5m and 31.7m		4	RUN						3	RUN #4 TCR=100% SCR=100% RQD=97% UCS=17MPa (Average)
										6	
	Sub-vertical fracture at 30.7m and 30.9m						98			3	
97.4	Highly weathered zone (25mm) at 31.3m									3	
31.8	END OF BOREHOLE AT 31.8m. BOREHOLE BACKFILLED WITH CEMENT GROUT TO SURFACE.										

+ 3, × 3 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-05

METRIC

W.P.	2061-13-00	LOCATION	Don River Bridge N 4 847 331.7 E 316 157.9	ORIGINATED BY	
HWY	401	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.02.25 - 2015.02.26	CHECKED BY	MEF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
129.4	GROUND SURFACE								
0.0	**TOPSOIL**, with roots and rootlets: (50mm)								
0.0	Silty **CLAY**, trace to some gravel, trace organics Very Stiff to Hard Brown Moist (FILL)	1	SS	22	129				
		2	SS	33	128				
		3	SS	27					
127.2									
2.2	**SAND** and **SILT**, some clay, trace to some gravel, occasional sand and gravelly sand lenses Compact Brown Moist	4	SS	16	127	0	41	47	12
		5	SS	16	126				
		6	SS	14	125				
	Loose	7	SS	4	124				
122.4									
7.0	Silty **CLAY**, with sand, trace gravel Firm Grey Moist to Wet	8	SS	4	123	0	31	38	31
		9	SS	2	122				
					121				
					120				

STRAT PLOT

GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 3/20/17

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-05

METRIC

W.P. 2061-13-00 | LOCATION Don River Bridge N 4 847 331.7 E 316 157.9 | ORIGINATED BY

HWY 401 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2015.02.25 - 2015.02.26 | CHECKED BY MEF

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page									
119.0										
10.4	Sandy **SILT**, trace clay, trace gravel Very Loose Grey Moist	10	SS	3		119				
117.7						118				
11.7	Silty **CLAY**, trace sand, trace gravel Very Stiff to Hard Grey Moist	11	SS	36		117				
						116				
		12	SS	26						0 5 37 58
114.8						115				
14.6	**SAND** and **SILT**, trace clay, trace gravel Very Loose to Compact Grey Moist	13	SS	3		114				
						113				
		14	SS	4						0 60 38 2
						112				
		15	SS	7		111				
						110				

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No DRB-05

METRIC

W.P.	2061-13-00	LOCATION	Don River Bridge N 4 847 331.7 E 316 157.9	ORIGINATED BY	
HWY	401	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.02.25 - 2015.02.26	CHECKED BY	MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	Continued From Previous Page													
	SAND and **SILT**, trace clay, trace gravel Compact to Very Dense Grey Moist		16	SS	18		109 108 107		○					
	Becoming very dense below 22.5m depth		17	SS	78		106		○					
105.0 24.4	Silty **CLAY**, trace sand, trace gravel Very Stiff to Hard Grey Moist (TILL)		18	SS	19		105 104 103		○					
			19	SS	100/ 0.200		102 101		○ ○		0	0	66	34
			20	SS	100/ 0.150		100		○					
99.8 29.6	**SHALE**, weathered, grey													

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No DRB-05

METRIC

W.P. 2061-13-00 | LOCATION Don River Bridge N 4 847 331.7 E 316 157.9 | ORIGINATED BY

HWY 401 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2015.02.25 - 2015.02.26 | CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
	Continued From Previous Page										
	SHALE, weathered, grey										
98.8			21	SS	100/		99		○		
30.6	END OF BOREHOLE AT 30.6m. BOREHOLE OPEN TO 22.4m AND WATER LEVEL AT 8.8m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG MIXED WITH AUGER CUTTINGS TO SUFACE.				0.075						

+ 3, × 3: Numbers refer to Sensitivity

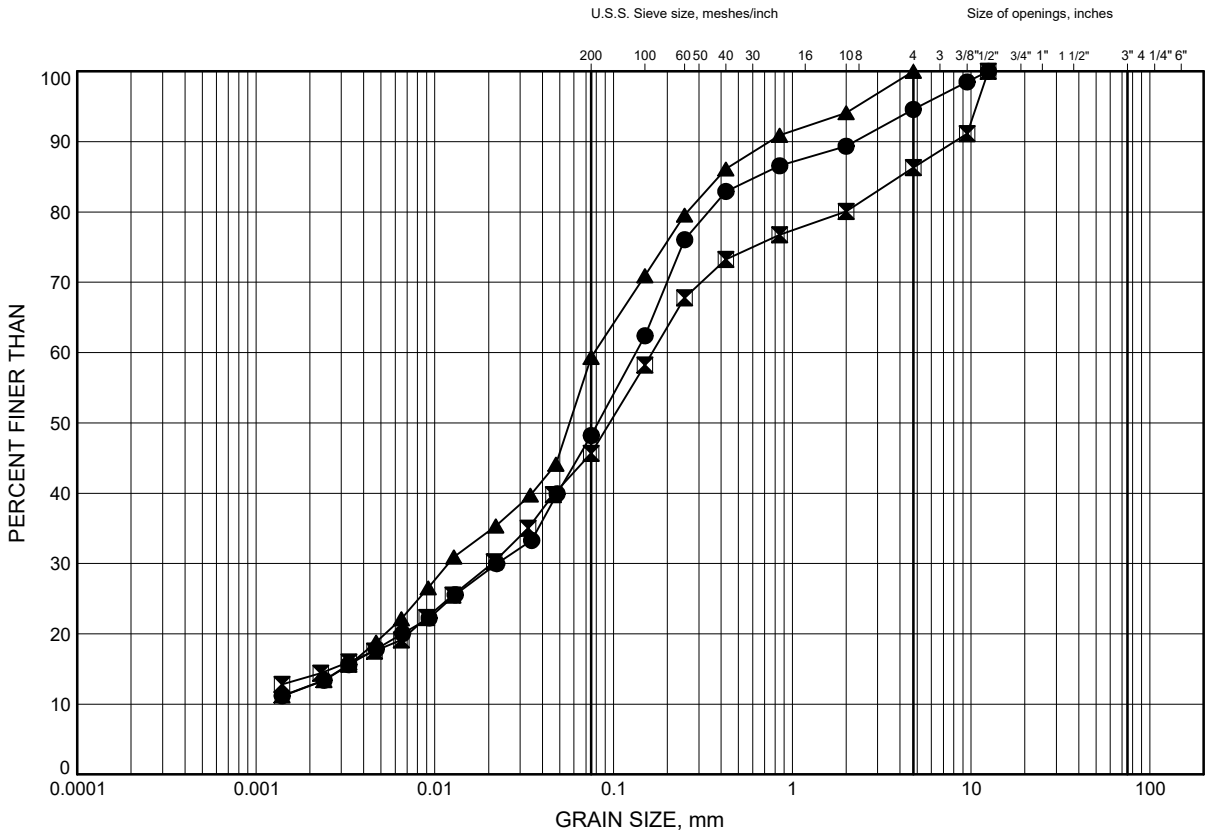
15 20 5 10 (%) STRAIN AT FAILURE

Don River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE A1

SAND and SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-03	3.35	126.85
⊠	DRB-04	4.88	124.32
▲	DRB-05	2.59	126.81

Date March 2017

W.P. 2061-13-00

THURBER

Prep'd AN

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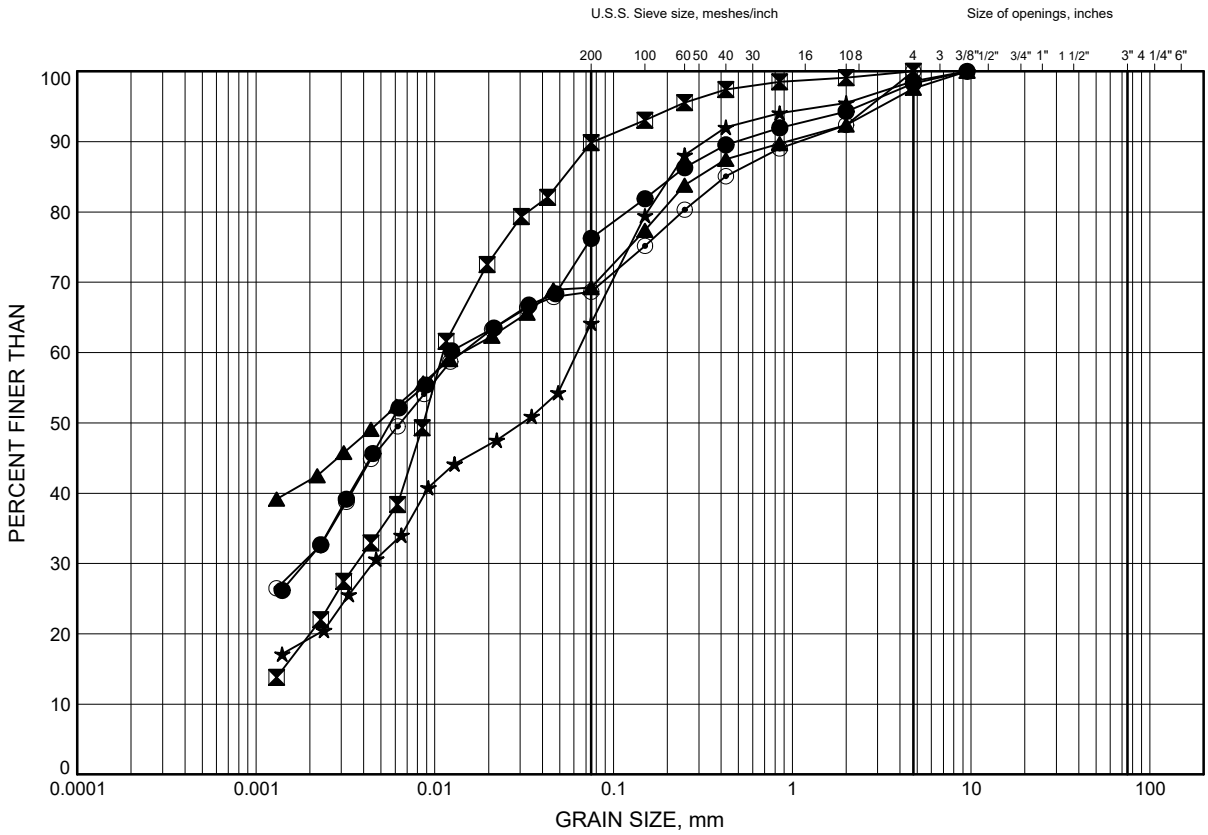
GRAIN SIZE DISTRIBUTION - THURBER 1205.GPJ 3/16/17

Don River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE A2

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-01	9.45	120.85
⊠	DRB-02	12.50	117.50
▲	DRB-03	9.45	120.75
★	DRB-04	10.97	118.23
⊙	DRB-05	7.92	121.48

Date March 2017

W.P. 2061-13-00

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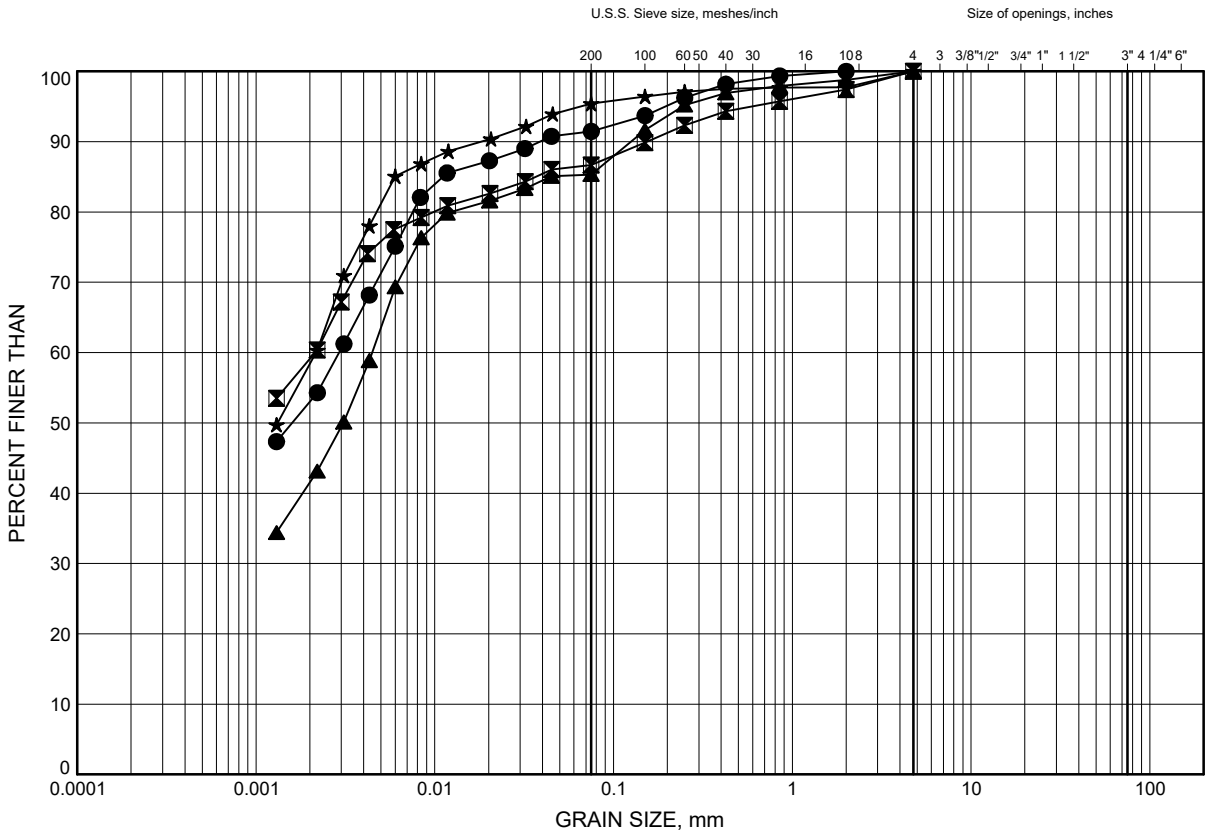
GRAIN SIZE DISTRIBUTION

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)
●	DRB-02	1.07	128.93
⊠	DRB-02	4.88	125.12
▲	DRB-03	12.50	117.70
★	DRB-05	14.02	115.38

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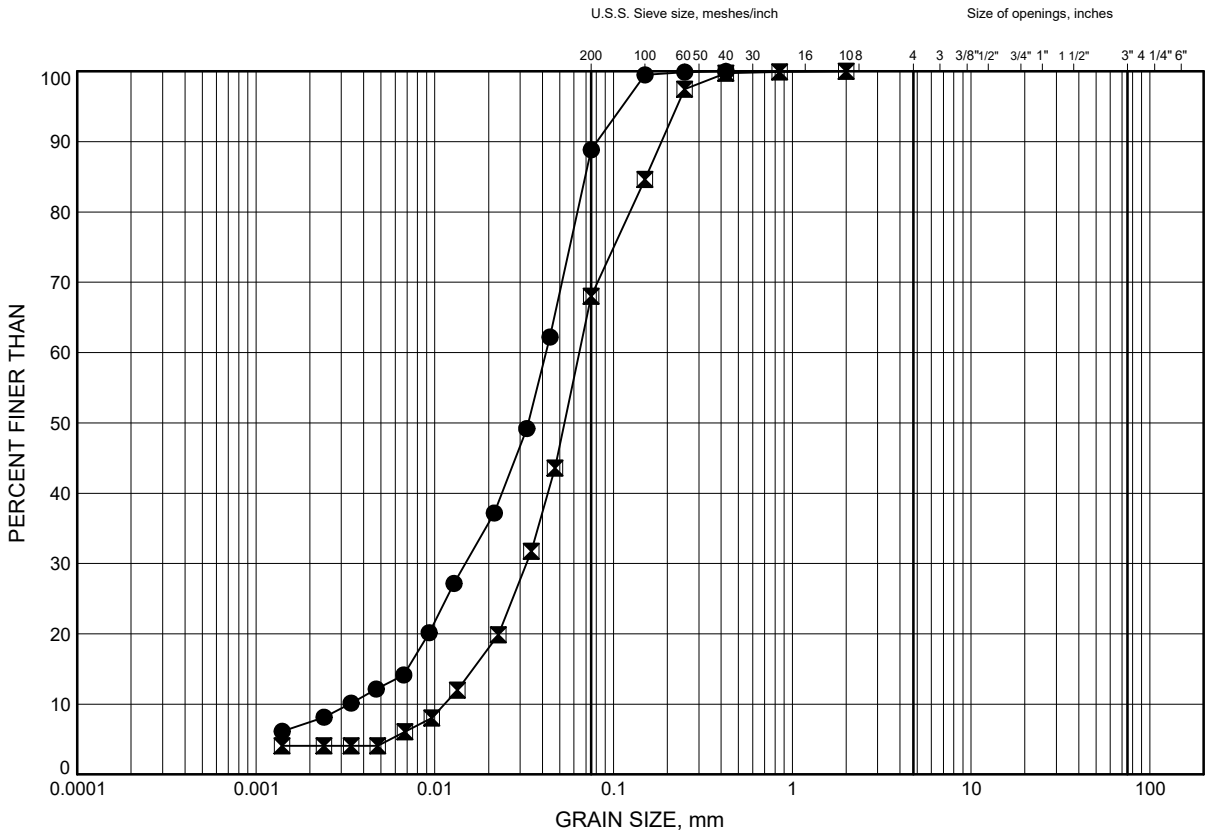
GRAIN SIZE DISTRIBUTION

FIGURE A4

SILT to Sandy SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-02	17.07	112.93
⊠	DRB-03	18.59	111.61

Date March 2017

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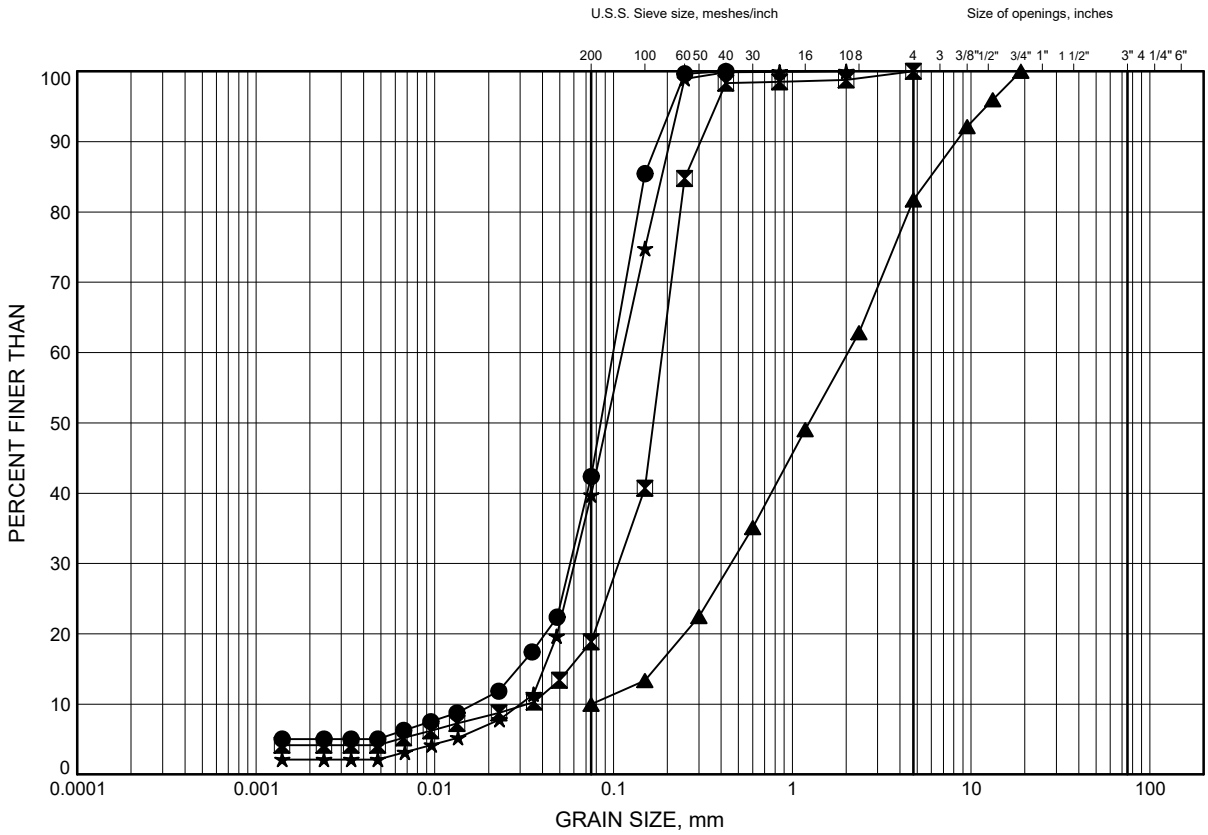
GRAIN SIZE DISTRIBUTION

FIGURE A5

SAND to SAND and SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-01	18.59	111.71
⊠	DRB-04	15.54	113.66
▲	DRB-04	21.64	107.56
★	DRB-05	17.07	112.33

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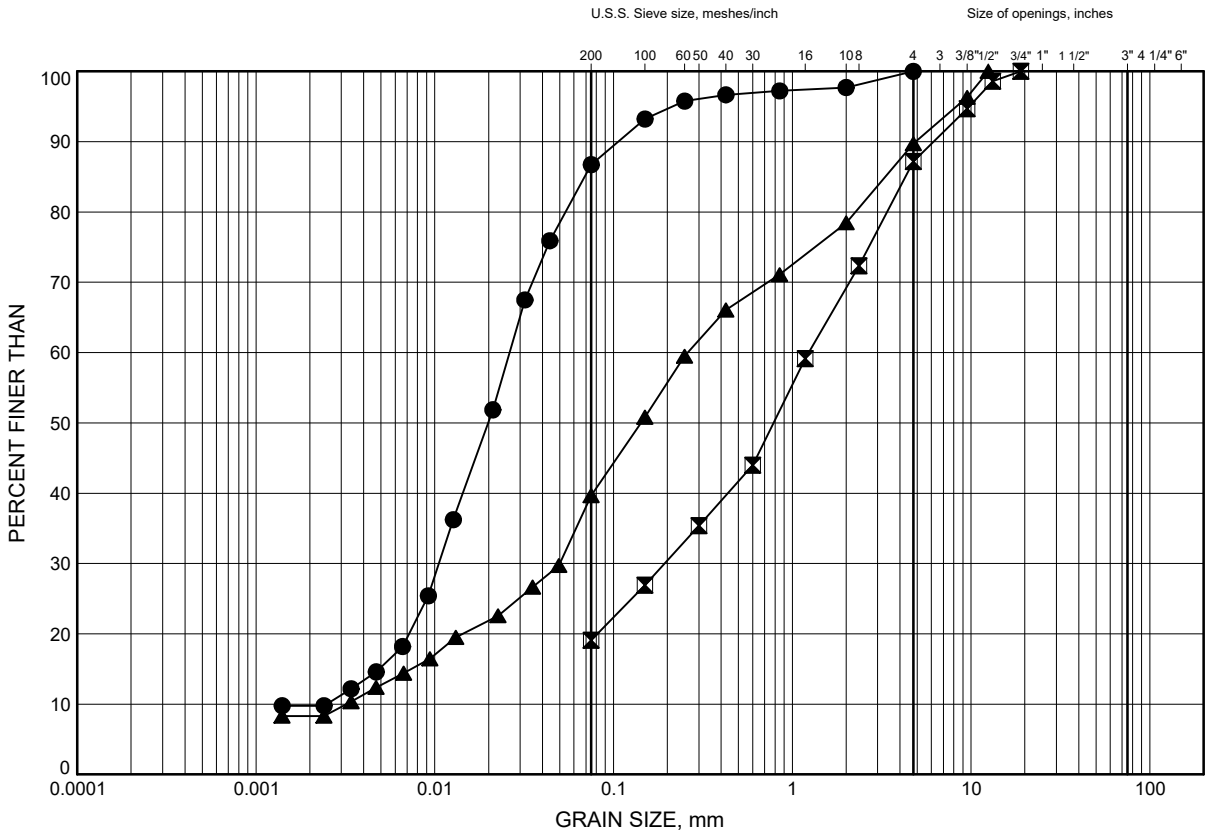
GRAIN SIZE DISTRIBUTION

FIGURE A6

SILT to SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-01	27.74	102.56
⊠	DRB-03	26.21	103.99
▲	DRB-04	26.21	102.99

Date March 2017

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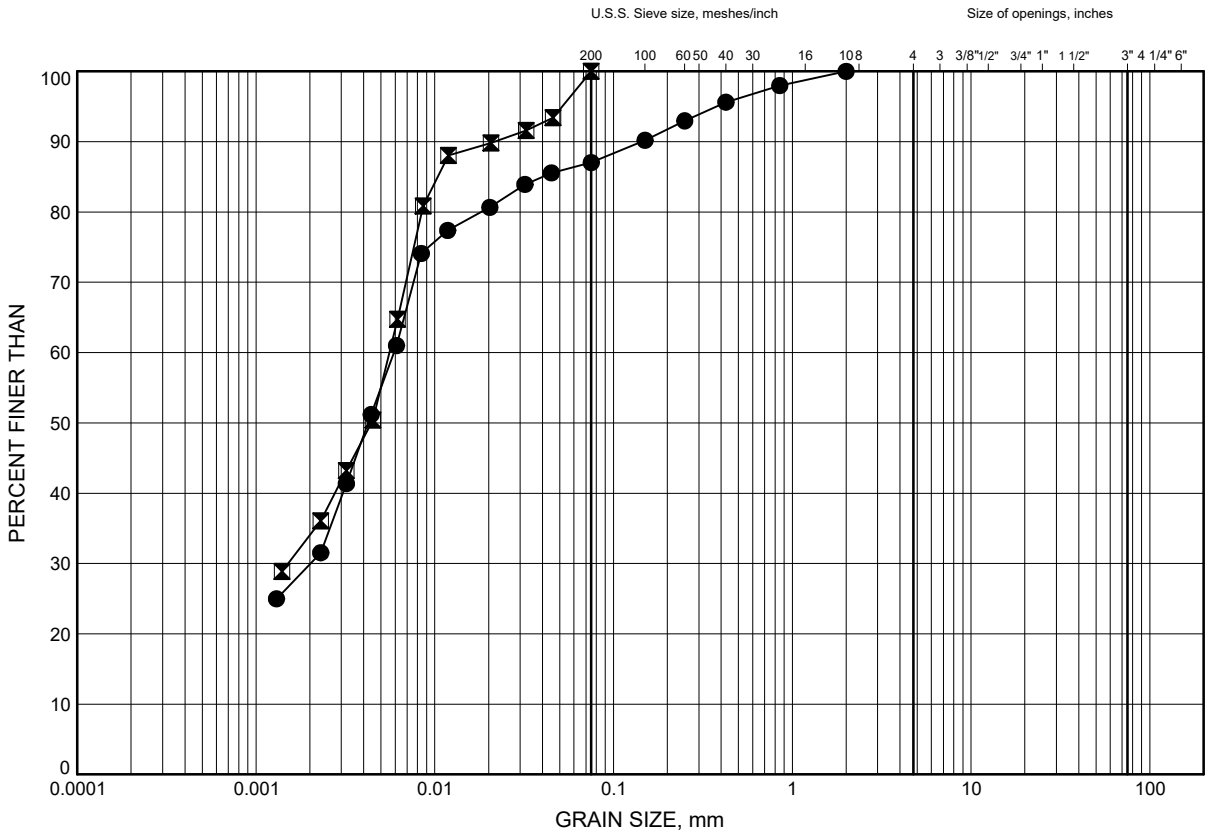
GRAIN SIZE DISTRIBUTION

FIGURE A7

Silty CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-02	21.56	108.44
⧗	DRB-05	27.74	101.66

Date March 2017

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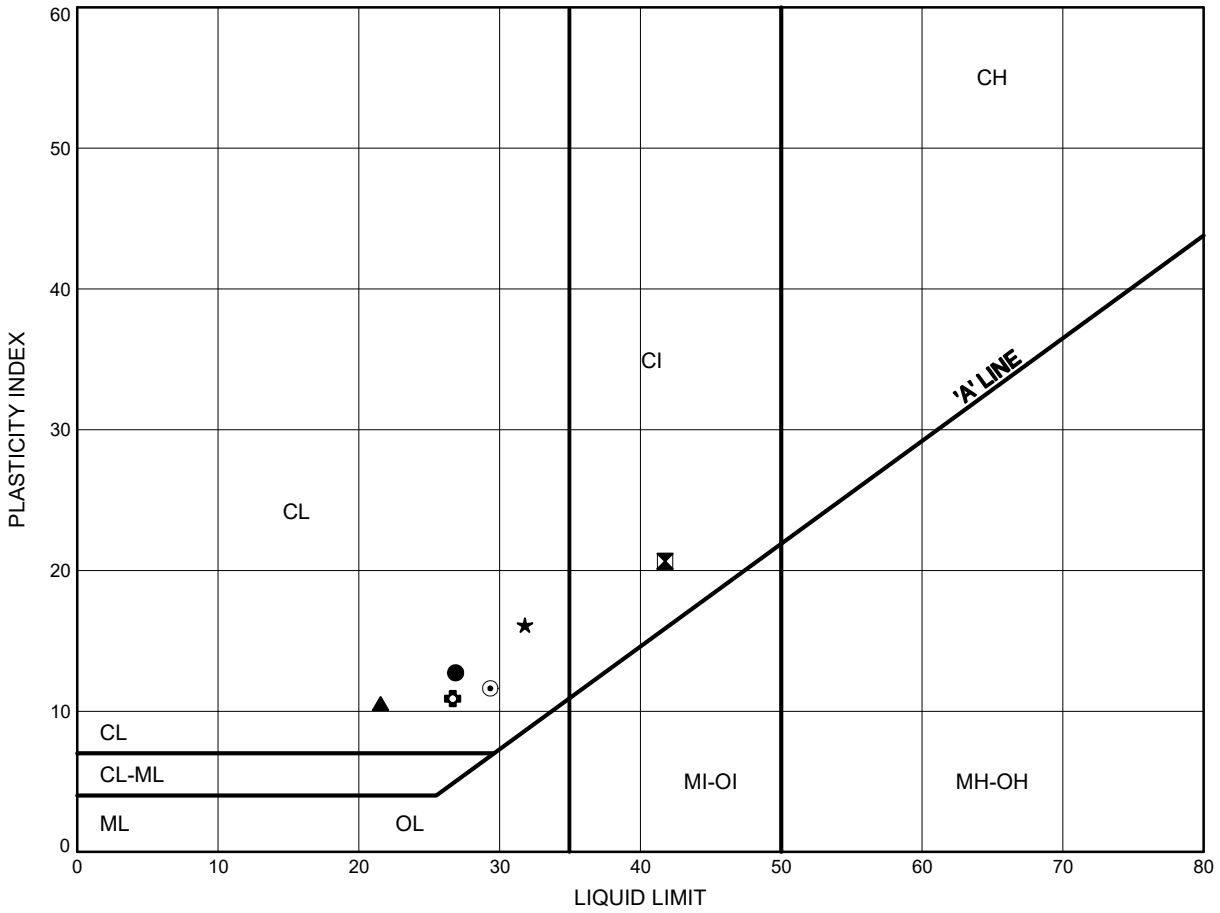
GRAIN SIZE DISTRIBUTION - THURBER 1205.GPJ 3/16/17

Don River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE A8

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-01	9.45	120.85
⊠	DRB-02	1.07	128.93
▲	DRB-02	4.88	125.12
★	DRB-03	9.45	120.75
⊙	DRB-03	12.50	117.70
✿	DRB-04	10.97	118.23

Date March 2017

W.P. 2061-13-00

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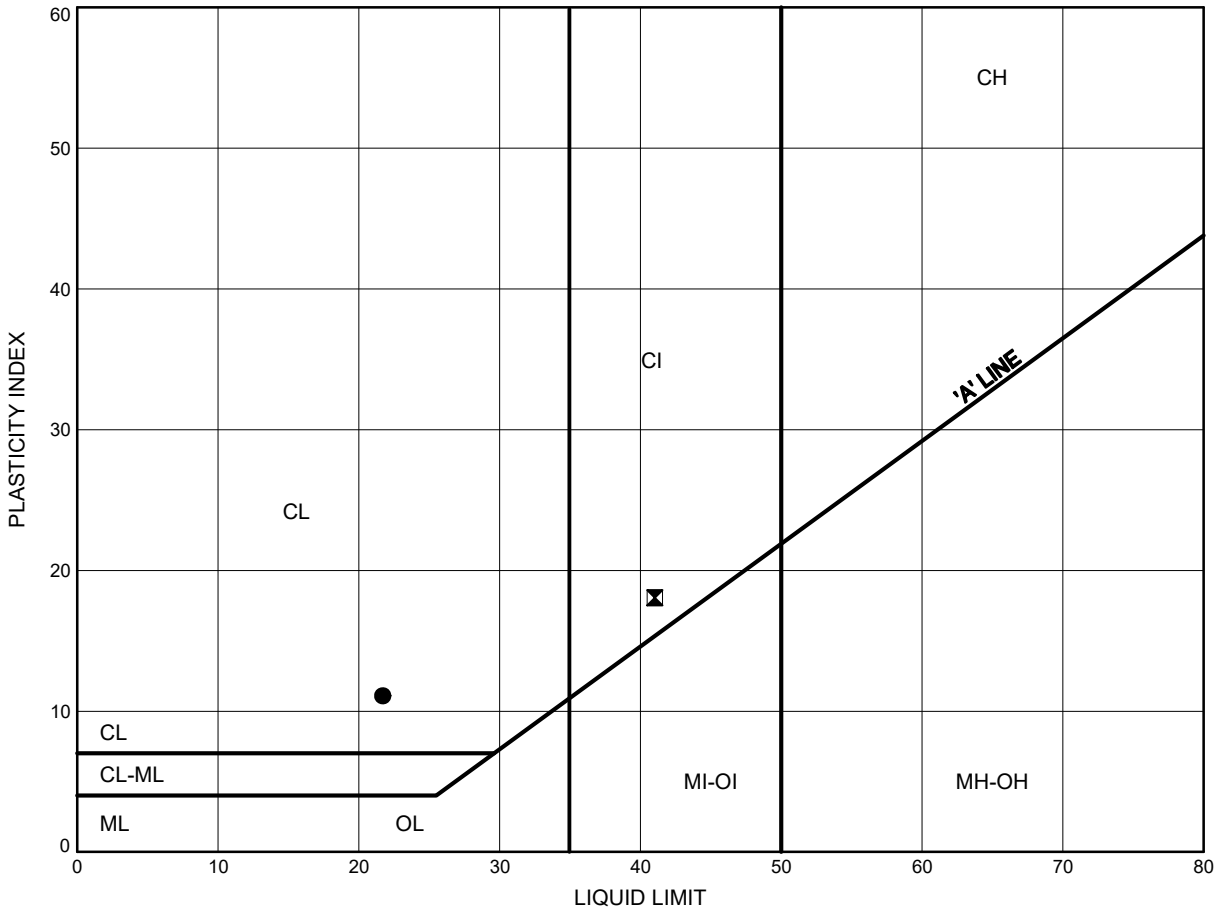
THURBALT 1205.GPJ 3/16/17

Don River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE A9

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DRB-05	7.92	121.48
⧓	DRB-05	14.02	115.38

Date March 2017

W.P. 2061-13-00

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THURBALT 1205.GPJ 3/16/17

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No :	19-5161-205	**Client :**	MMM
		Date Drilled :	2015-05-04
Project Name :	Highway 401 & Leslie Street	**Date Tested :**	2015-05-07
Core Size :	NQ **BH No :** DRB=-4	**Tester :**	ISP

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	27.3	A	3.1	46.6	60.5	22.2	-	Sand/Silt Till
2	2	28.7	A	11.9	47.7	78.2	69.5	-	Sand/Silt Till
3	3	29.0	A	2.4	147.0	64.1	6.8	Shale	Weak
4	3	29.4	A	3.2	46.9	58.2	24.0	Shale	Weak
5	4	30.3	A	2.4	46.6	56.9	18.1	Shale	Weak
6	4	31.0	A	2.4	47.3	61.6	17.3	Shale	Weak
7	4	31.6	A	2.2	47.0	62.7	15.6	Shale	Weak
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.

Photo 1 Photograph of Borehole DRB-04 Core Runs #1 to #4

APPENDIX B

Current Investigation – Record of Boreholes/Drillholes, Cone Penetration Tests, 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.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* trace fines)

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

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

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 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, Gs)
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)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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 grainsize. 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.

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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 = σ'_p / σ'_{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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

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 90º 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		

GOLDER MEMBER OF WSP

RECORD OF BOREHOLE No DR-1

SHEET 1 OF 1

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847342.6; E 316023.0 MTM NAD 83 ZONE 10 (LAT. 43.765966; LONG. -79.360609)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 130 mm O.D., 105 mm I.D. Hollow Stem Augers

COMPILED BY RM

DATUM Geodetic

DATE June 3 and 4, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
138.5	GROUND SURFACE									
0.0	ASPHALT (150 mm)									
0.2	SAND (SW), some fines, trace gravel (FILL) Compact Brown Moist	1	SS	23	138					
		2	SS	20						
		3	SS	26	137					
136.3										
2.2	Sandy CLAYEY SILT (CL), trace to some gravel (FILL) Firm to very stiff Brown Moist	4	SS	9	136					
		5	SS	30	135					
		6	SS	6						
134.0										
4.5	SILTY SAND (SM), some gravel Compact to very dense Brown Moist	7	SS	24	134					
					133					
		8	SS	49	132		18	32	45	5
					131					
		9	SS	52	130					
128.9		10A 10B	SS	23	129					
9.7	Sandy CLAYEY SILT (CL), trace gravel Stiff to very stiff Brown to dark grey Moist									
		11A 11B	SS	21	128		7	27	48	18
	Trace rootlets from a depth of 11.7 m to 12.8 m. (between Elev. 126.8 m and 125.7 m).				127					
125.7		12	SS	11	126		0	29	43	29
12.8	END OF BOREHOLE									

NOTES:

1. Borehole open and dry upon completion of drilling.

2. Borehole caved to a depth of 11.7 m below ground surface (Elev. 126.8 m) upon removal of augers.

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH kPa (20 40 60 80 100)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) (10 20 30)

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-3

SHEET 1 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847343.0; E 316026.2 MTM NAD 83 ZONE 10 (LAT. 43.765970; LONG. -79.360569)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY RM

DATUM Geodetic

DATE June 9 to 13, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
138.5	GROUND SURFACE							
0.0	ASPHALT (100 mm)							
0.1	Gravelly SILTY SAND (SP) (FILL) Compact Brown Dry	1	SS	24				
137.8 / 0.7	SILTY SAND (SM), trace to some gravel (FILL) Compact Brown to grey Moist to wet	2	SS	18				
		3	SS	28				
		4	SS	11				
135.5 / 3.0	CLAYEY SILT (CL), some sand (FILL) Soft Brown Moist	5	SS	3				
134.7 / 3.8	Sandy SILT (ML) of slight plasticity, trace gravel, containing silty sand pockets Compact to very dense Brown to grey Moist	6	SS	24				
		7	SS	63	11	35	49	5
		8	SS	40				
		9A 9B	SS	38				
128.3 / 10.2	Sandy CLAYEY SILT (CL), trace gravel Firm to very stiff Brown to dark grey Moist	10	SS	17				
		11	SS	9				
	Trace rootlets from a depth of 13.0 m to 14.0 m. (between Elev. 125.5 m and 124.5 m).	12	SS	10	0	26	56	18
124.5 / 14.0	SILTY SAND (SM), trace to some gravel Loose to compact Grey to dark grey Moist	13A 13B	SS	8				

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 138, 137, 136, 135, 134, 133, 132, 131, 130, 129, 128, 127, 126, 125, 124

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-3

SHEET 2 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847343.0; E 316026.2 MTM NAD 83 ZONE 10 (LAT. 43.765970; LONG. -79.360569)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY RM

DATUM Geodetic

DATE June 9 to 13, 2019

CHECKED BY DH

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY SAND (SM), trace to some gravel Loose to compact Grey to dark grey Moist		14	SS	19		123				20 46 30 4
121.4							122				
17.1	CLAYEY SILT (CL) , some sand, contains sand seams Firm to stiff Grey Moist						121				
			15	SS	10		120				
							119				
			16	SS	7						0 14 49 37
							118				
			17	TO	-						
117.3											
21.2	Sandy CLAYEY SILT (CL), some gravel Hard Grey Moist		18	SS	32		117				
115.6							116				
22.9	SILTY SAND (SM) Very dense Grey Wet						115				
			19	SS	62		114				
							113				
							112				
			20	SS	69		111				0 52 46 2
							110				
109.2											
29.3	CLAYEY SILT (CL), some sand Hard Grey Moist						109				

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-3

SHEET 3 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847343.0; E 316026.2 MTM NAD 83 ZONE 10 (LAT. 43.765970; LONG. -79.360569)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY RM

DATUM Geodetic

DATE June 9 to 13, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	CLAYEY SILT (CL), some sand Hard Grey Moist		21	SS	45		108 107				
106.2 32.3	SILTY SAND (SM), some gravel (TILL) Very dense Grey Wet		22	SS	67		106 105 104 103				
101.6 37.0	Inferred moderately weathered, grey SHALE (BEDROCK)		23	SS	107/0.23		102				
	SHALE (BEDROCK) Grey Bedrock cored from a depth of 37.0 m to 41.0 m. (between Elev. 101.5 m and 97.5 m). For bedrock coring details refer to Record of Drillhole DR-3.		1	RC	REC 91%		101				RQD = 61%
			2	RC	REC 100%		100				RQD = 89%
			3	RC	REC 100%		99 98				RQD = 89%
97.5 41.0	END OF BOREHOLE NOTE: 1. Borehole open and dry before switching to mud rotary method at 5.2 m below ground surface (Elev. 133.3 m).										

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786302

RECORD OF DRILLHOLE: DR-3

LOCATION: N 4847343.03 ;E 316026.21

DRILLING DATE: June 13, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Truck-Mounted Drillimg

DRILLING CONTRACTOR: Geo-environmental Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole DR-3		101.46															
	Rotary Diamond Drill; HQ Size Core Barrel	Moderately weathered to fresh, thinly bedded, grey, fine grained, faintly porous, weak to medium strong, SHALE (Georgian Bay Formation) interbedded with LIMESTONE		37.01	1							BD,PL,RO,PC,CL BD,PL,RO,PC,CL BD,PL,RO,PC,CL							LC BC
38		LIMESTONE/SILTSTONE Layers: 38.65 m - 38.68 m, 39.12 m - 39.15 m			2							BD,PL,SM,SA BD,PL,SM,SA BD,PL,SM,SA BD,PL,RO,PC,CL BD,PL,RO,PC,CL							
39												BD,PL,RO,PC,CL BD,CU,SM,SA							
		LIMESTONE/SILTSTONE Layers: 39.57 m - 39.60 m			3							BD,PL,RO,SA BD,PL,SM,SA BD,PL,SM,SO BD,UN,SM,SO BD,UN,RO,SA BD,PL,SM,SO							
40																			
41		UCS = 34.6 MPa		97.45								BD,PL,RO,SO							
		END OF DRILLHOLE		41.02															
42																			
43																			
44																			
45																			
46																			
47																			

DEPTH SCALE
1 : 50

GOLDER
MEMBER OF WSP

LOGGED: DH
CHECKED: DH

GTA-RCK 046 S:\CLIENTS\MTO\HWY_401_LESLIE_STREET\02_DATA\GINT\HWY_401_LESLIE_STREET.GPJ GAL-MISS.GDT 9/21/21

RECORD OF BOREHOLE No DR-5

SHEET 1 OF 3

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847350.9; E 316071.6 MTM NAD 83 ZONE 10 (LAT. 43.766040; LONG. -79.360005)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY KN

DATUM Geodetic

DATE August 28 to 30, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
129.1	GROUND SURFACE									
0.0	SAND (SW), some fines, trace gravel, contains asphalt fragments (FILL) Compact Brown Moist	1	SS	23						
128.4 / 0.7	Sandy CLAYEY SILT (CL), some gravel, contains fibrous organics (FILL) Firm to very stiff Brown to grey below Moist	2	SS	19						
		3	SS	17						
	Contains rootlets, wood chips and an organic odour from a depth of 2.3 m to 5.2 m. (between Elev. 126.8 m and 123.9 m).	4	SS	11						
		5	SS	12			13	33	38	16
		6	SS	5						
		7	SS	5						
123.5 / 5.6	Sandy SILT (ML), some gravel, contains rootlets and wood fragments Loose Brown Moist	8	SS	5						
122.1 / 7.0	Sandy CLAYEY SILT (CL), trace to some gravel, contains sand seams Soft to very stiff Grey Moist to wet	9	SS	7						
		10	SS	9			13	29	38	20
		11	SS	3						
		12	TO	-						
116.6 / 12.5	SAND (SW), trace to some fines, trace gravel Compact to very dense Grey Wet	13A 13B	SS	26			6	25	57	12
		14	SS	11						

Dynamic cone penetration resistance plot: 20, 40, 60, 80, 100

Shear strength kPa: ○ Unconfined ● Quick triaxial + Field vane × Remoulded

Plastic limit w_P — Natural moisture content w — Liquid limit w_L

Water content (%): 10, 20, 30

Ground water conditions: ▼ 123.5

Elevation scale: 129, 128, 127, 126, 125, 124, 123, 122, 121, 120, 119, 118, 117, 116, 115

>96 +

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-5

SHEET 2 OF 3

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847350.9; E 316071.6 MTM NAD 83 ZONE 10 (LAT. 43.766040; LONG. -79.360005)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY KN

DATUM Geodetic

DATE August 28 to 30, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND (SW), trace to some fines, trace gravel Compact to very dense Grey Wet		15	SS	26		114				
							113				
							112				
							111				
			16	SS	15				○	Non-Plastic	
							110				
							109				
							108				
			17	SS	57				○	Non-Plastic	
							107				
105.9											
23.2	Gravelly SAND (SW) to Sandy GRAVEL (GW), containing clayey silt-silt pockets (TILL) Very dense Grey Wet						106				
							105				
			18	SS	100/0.13						
							104				
			19	SS	100/0.18		103		⊢⊣		
101.8							102				
27.3	CLAYEY SILT (CL), some gravel, some sand, containing shale fragments (RESIDUAL SOIL) Hard Grey Moist		20	SS	100/0.28						
101.2 27.9	END OF BOREHOLE										

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-5

SHEET 3 OF 3

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847350.9; E 316071.6 MTM NAD 83 ZONE 10 (LAT. 43.766040; LONG. -79.360005)

ORIGINATED BY DH

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY KN

DATUM Geodetic

DATE August 28 to 30, 2019

CHECKED BY DH

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, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED)	w_P	w	w_L	γ kN/m³	GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---												

NOTES:

1. An additional borehole was advanced for well installation about 2 m south of sampled borehole on August 30, 2019.

2. Groundwater level measurements in piezometer:

Date:	Depth (m)	Elev. (m)
19/10/25	5.6	123.5
20/09/14	5.8	123.3

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-6

SHEET 1 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847329.4; E 316137.5 MTM NAD 83 ZONE 10 (LAT. 43.765846; LONG. -79.359188)

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 KN

DATUM Geodetic

DATE July 23 and 24, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED)	WATER CONTENT (%) (PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
128.0	GROUND SURFACE								
0.0	ASPHALT (38 mm)								
0.1	SAND (SP) and gravel (FILL) Loose Brown Moist	1A 1B	SS	4					
	Sandy CLAYEY SILT (CL), contains rootlets, some gravel (FILL) Soft to very stiff Brown Moist	2	SS	9	127				
		3	SS	17	126				
125.8									
2.2	CLAYEY SILT (CL), trace sand to sandy, some gravel Firm to very stiff Grey Moist	4	SS	7					
		5	SS	8	125				
					124	>96 +			
		6	SS	6	123				
122.7									
5.3	SILTY SAND (SM) Compact Grey to black Wet	7	TO	-			43.1		0 63 35 2
	Contains organics at a depth of 5.5 m (Elev. 122.5 m)	8	SS	13	122				
120.8					121				
7.2	Sandy CLAYEY SILT (CL), trace to some gravel, contains sand seams Firm to hard Grey Moist	9	SS	5	120				12 29 42 17
		10	TO	-					
					119	3.0 +			
		11	SS	32	118				
					117				
		12	SS	43	116				2 30 48 20
					115				
	Trace sand from a depth of 13.7 m to 14.8 m. (between 114.3 m and 113.2 m)	13	SS	10	114				
113.2									
14.8									

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-6

SHEET 2 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847329.4; E 316137.5 MTM NAD 83 ZONE 10 (LAT. 43.765846; LONG. -79.359188)

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 KN

DATUM Geodetic

DATE July 23 and 24, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Sandy SILT (ML) Dense Grey Wet		14	SS	32						
							112				
							111				
							110				
			15	SS	38				○		0 33 65 2
							109				
							108				
							107				
			16A 16B	SS	37						
							106				
105.1											
22.9	Sandy Gravelly CLAYEY SILT (CL) (TILL) Hard Grey Moist Casing grinding from a depth of 24.4 m to 27.4 m. (between Elev. 103.6 m and 102.1 m)						105				
							104				
			17	SS	104/0.20						
							103				
102.1											
25.9	Sandy Gravelly CLAYEY SILT (CL) (RESIDUAL SOIL) Hard Grey Moist Casing grinding from a depth of 25.9 m to 27.4 m. (between Elev. 102.1 m and 100.6 m).						102				
							101				
100.3			18A 18B	SS	102/0.20				○		
27.8	Inferred highly weathered, grey, SHALE (BEDROCK)										
	SHALE (BEDROCK) grey Bedrock cored from a depth of 27.8 m to 30.8 m. (between Elev. 100.2 m and 97.2 m). For rock coring details refer to Record of Drillhole DR-6.		1	RC	REC 93%		100				RQD = 51%
							99				
			2	RC	REC 100%						RQD = 85%

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-6

SHEET 3 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847329.4; E 316137.5 MTM NAD 83 ZONE 10 (LAT. 43.765846; LONG. -79.359188)

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 KN

DATUM Geodetic

DATE July 23 and 24, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SHALE (BEDROCK) Grey		2	RC	REC 100%						RQD = 85%
97.3	Bedrock cored from a depth of 27.8 m to 30.8 m. (between Elev. 100.2 m and 97.2 m).										
30.8	For rock coring details refer to Record of Drillhole DR-6.										
	END OF BOREHOLE										
	NOTE: 1. Borehole open and dry prior to mud rotary drilling at a depth of 3.7 m below ground surface (Elev. 124.3 m).										

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786302

RECORD OF DRILLHOLE: DR-6

LOCATION: N 4847329.39 ;E 316137.47

DRILLING DATE: July 24, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole DR-6		100.20															
28	Rotary Diamond Drill, HQ Size Core Barrel	Moderately weathered, thinly bedded, grey, slightly porous, weak to medium strong, SHALE (Georgian Bay Formation) with interbeds of strong LIMESTONE/SILTSTONE		27.79	1							BD,PL,SM,IN, CL, 10 mm	0.5	15	0			LC, BC	
												BD,PL,RO,SA	1.5	1	20				
												BD,PL,SM,CC, CL	1	4	12			BC	
												BD,PL,SM,PC, CL	1	4	12				
29												BD,PL,RO,CC, CL	1.5	4	16			BC	
																		BC	
					2							BD,PL,SM,PC, CL	1	4	12			BC	
30												BD,PL,RO,PC, CL	1.5	4	16			BC	
		UCS = 37.2 MPa		97.24														BC	
		END OF DRILLHOLE		30.75															
31																			
32																			
33																			
34																			
35																			
36																			
37																			

DEPTH SCALE
1 : 50

GOLDER
MEMBER OF WSP

LOGGED: DH
CHECKED:

GTA-RCK 046 S:\CLIENTS\MTO\HWY_401_LESLIE_STREET\02_DATA\GINT\HWY_401_LESLIE_STREET.GPJ GAL-MISS.GDT 9/21/21

RECORD OF BOREHOLE No DR-7

SHEET 1 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847347.1; E 316136.0 MTM NAD 83 ZONE 10 (LAT. 43.766005; LONG. -79.359205)

ORIGINATED BY DH/RM

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY KN

DATUM Geodetic

DATE July 18, 19, and 22, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT (%) 10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
129.3	GROUND SURFACE							
0.0	TOPSOIL (100 mm)							
0.1	Gravelly SILTY SAND (SM), contains rootlets (FILL) Compact Brown Dry to moist	1A 1B	SS	25				
128.7 / 0.6	CLAYEY SILT (CL), some sand to CLAYEY SILT-SILT (CL-ML) and sand, trace to some gravel (FILL) Stiff to very stiff Brown Moist	2	SS	29				
		3	SS	12				
		4	SS	15				1 40 47 12
		5	SS	15				
		6	SS	11				
122.8 / 6.6	CLAY (CH) to CLAYEY SILT (CL), trace sand, trace gravel, contains sand seams Stiff to very stiff Brown to grey Moist	7A 7B	SS	6	>96	56	Unit weight = 20.95 kN/m3	
		8	TO	-				
		9	SS	2	3.5			0 3 50 47
119.2 / 10.1	SILT (ML) and sand, trace gravel Compact Grey Moist	10	SS	19				1 44 51 4
117.7 / 11.6	CLAYEY SILT (CL), trace to some sand, trace to some gravel Hard Grey Moist	11	SS	90				
		12	SS	32				
114.5 / 14.8								

GROUND WATER CONDITIONS: ▼ 1

ELEVATION SCALE: 129, 128, 127, 126, 125, 124, 123, 122, 121, 120, 119, 118, 117, 116, 115

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-7

SHEET 2 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847347.1; E 316136.0 MTM NAD 83 ZONE 10 (LAT. 43.766005; LONG. -79.359205)

ORIGINATED BY DH/RM

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY KN

DATUM Geodetic

DATE July 18, 19, and 22, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---									
	SILTY SAND (SM) to SILT (ML), trace gravel, Compact to dense, Grey, Wet	13	SS	26		114				
						113				
		14	SS	32		112				
						111				
						110				
		15	SS	32		109		○		1 15 82 2
107.7						108				
21.6	SILTY CLAY (CI), trace sand, contains sand seams, varved, Hard, Grey, Moist					107				
		16	SS	31		106		○ 41		0 4 43 53
104.9						105				
24.4	Sandy Gravelly CLAYEY SILT (CL) (TILL), Hard, Grey, Moist					104				
		17	SS	165/0.23		103		○		
101.9						102				
27.4	Gravelly SILTY SAND (SM) (TILL), Very dense, Grey, Moist					101				
100.2		18	SS	103/0.10		100				
29.3		1	RC	REC 87%						RQD = 65%

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-7

SHEET 3 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847347.1; E 316136.0 MTM NAD 83 ZONE 10 (LAT. 43.766005; LONG. -79.359205)

ORIGINATED BY DH/RM

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 150 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY KN

DATUM Geodetic

DATE July 18, 19, and 22, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY (CI), containing shale fragments (RESIDUAL SOIL) Hard Grey Moist		1	RC	REC 87%		99				RQD = 65%
98.5 / 30.8	SHALE (BEDROCK) Grey										
	Bedrock cored from a depth of 29.3 m to 30.8 m. (between Elev. 100.0 m and 98.5 m).										
	For rock coring details refer to Record of Drillhole DR-7.										
	END OF BOREHOLE										

NOTES:

1. Borehole open and dry before switching to mud rotary method at a depth of 3.7 m below ground surface (Elev. 125.6 m).

2. Piezometer installed approximately 5 m East and 2 m North of Borehole DR-7 location.

3. Groundwater level measurements in piezometer:

Date:	Depth (m)	Elev. (m)
20/05/01	4.5	124.8
20/09/14	4.8	124.5

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786302

RECORD OF DRILLHOLE: DR-7

LOCATION: N 4847347.14 ;E 316136.05

DRILLING DATE: July 22, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole DR-7		99.97															
30	Rotary Diamond Drill, HQ Size Core Barrel	Moderately to completely weathered, grey, weak to very weak, SHALE (Georgian Bay Formation) with clay seams and strong to very strong LIMESTONE/SILTSTONE layers		29.31	1							BD,PL,SM,IN, CL, 20 mm	1	15	0				LC
												BD,PL,SM,PC, CL	1	4	12				
												BD,PL,SM,CC, CL	1	4	12				
												BD,PL,SM,CC, CL, 5 mm	1	15	0				
												BD,PL,SM,IN, CL, 15 mm	1	15	0				
												BD,PL,SM,CC, CL	1	4	12				
												BD,PL,SM,CC, CL	1	4	12				
				98.44								BD,PL,SM,CC, CL	1	4	12				
31		END OF DRILLHOLE		30.84															
32																			
33																			
34																			
35																			
36																			
37																			
38																			
39																			

DEPTH SCALE
1 : 50

GOLDER
MEMBER OF WSP

LOGGED: DH
CHECKED: DH

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RECORD OF BOREHOLE No DR-9

SHEET 1 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847356.9; E 316164.9 MTM NAD 83 ZONE 10 (LAT. 43.766093; LONG. -79.358847)

ORIGINATED BY AM

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY RM

DATUM Geodetic

DATE June 4 to 11, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
134.5	GROUND SURFACE										
0.0 / 134.2	ASPHALT (280 mm)										
0.3	SAND (SP/GP) and gravel, trace fines (FILL) Compact Brown Moist	1	SS	12	134						
		2	SS	30							
133.1 / 1.4	Sandy CLAYEY SILT (CL), trace to some gravel, containing asphalt fragments (FILL) Firm to very stiff Brown to grey Moist	3	SS	19	133						
		4	SS	5	132						
		5	SS	14	131						
		6	SS	16	130			4	34	50	12
128.3 / 6.3	SAND (SP) and gravel, trace fines (FILL) Compact Brown Moist	7A 7B 7C	SS	20	129						
	Sandy CLAYEY SILT (CL), trace to some gravel (FILL) Firm to stiff Brown to grey Moist	8	SS	17	128						
	Auger grinding at a depth of 8.5 m. (Elev. 126.0 m)	9A 9B 9C 9D	SS	2	127						
125.1	ASPHALT (130 mm)				126						
9.7	SAND (SP) and gravel (FILL) Very loose Brown Moist				125						
124.3 / 10.2	Sandy CLAYEY SILT (CL), trace gravel, containing rootlets (FILL) Hard Brown-grey Moist	10	SS	4	124			6	46	31	17
122.8 / 11.7	CLAYEY SAND (SC), trace gravel, trace rootlets, contains wood fragments (FILL) Soft to firm Grey Moist				123						
122.0 / 12.5	ORGANIC SILT (OL), trace sand, contains wood chips Firm Black Moist	11A 11B	SS	7	122	125.5					
	Sandy CLAYEY SILT (CL), trace gravel Firm to hard Grey Moist	12	TO	-	121	2.0					
					120	>96					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

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RECORD OF BOREHOLE No DR-9

SHEET 2 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847356.9; E 316164.9 MTM NAD 83 ZONE 10 (LAT. 43.766093; LONG. -79.358847)

ORIGINATED BY AM

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY RM

DATUM Geodetic

DATE June 4 to 11, 2019

CHECKED BY DH

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
119.1 / 15.4	Sandy CLAYEY SILT (CL), trace gravel Firm to hard Grey Moist - Auger grinding from depths of 14.9 m to 15.2 m. (between Elev. 119.6 m and 119.3 m)	13A 13B	SS	15	▽	119		1	25	53	21
	SILTY SAND (SM) Compact to dense Grey Wet Auger grinding at a depth of 17.5 m. (Elev. 117.0 m)					118 117					
116.1 / 18.4	Sandy CLAYEY SILT-SILT (CL-ML), trace gravel Hard Grey Moist	14A 14B	SS	42		116 115		2	23	58	17
114.4 / 20.1	SILTY SAND (SM) Compact Grey Wet	15	SS	11		114 113		0	51	49	0
109.8 / 24.7	CLAYEY SILT (CL) to SILTY CLAY (CI), trace sand, trace gravel Stiff to hard Grey Moist Increased drilling resistance from depths of 25.3 m to 27.1 m. (between Elev. 109.2 m and 107.4 m)	16A 16B	SS	12		112 111 110 109 108					
	Auger grinding at a depth of 28.0 m. (Elev. 106.5 m)	17	SS	40		107 106	40	0	4	36	60
105.5 / 29.0	SAND (SW), some gravel, trace fines Loose Grey Wet					105					

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100)

SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED (20, 40, 60, 80, 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) (10, 20, 30)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-9

SHEET 3 OF 4

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847356.9; E 316164.9 MTM NAD 83 ZONE 10 (LAT. 43.766093; LONG. -79.358847)

ORIGINATED BY AM

DIST Central HWY 401

BOREHOLE TYPE Power Auger; 200 mm O.D. Hollow Stem Augers; Casing Advance (110 mm I.D.)

COMPILED BY RM

DATUM Geodetic

DATE June 4 to 11, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND (SW), some gravel, trace fines Loose Grey Wet		18	SS	6		104		○		
							103				
102.5 32.0	Inferred completely weathered, grey, SHALE (BEDROCK)						102				
			19	SS	100/0.08		101				
99.8 34.8	SHALE (BEDROCK) Grey		20	SS	100/0.08		100				
	Bedrock cored from 35.2 m to 39.3 m. (between Elev. 99.3 m and 95.2 m) For rock coring details refer to Record of Drillhole DR-9.		1	RC	REC 100%		99				RQD = 50%
			2	RC	REC 100%		98				RQD = 88%
							97				
95.2 39.3	END OF BOREHOLE NOTE: 1. Groundwater level measured at a depth of 15.8 m below ground surface (Elev. 118.7 m) during augering before water was added to the borehole.		3	RC	REC 100%		96				RQD = 93%

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786302

RECORD OF DRILLHOLE: DR-9

LOCATION: N 4847356.94 ;E 316164.91

DRILLING DATE: June 11, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75 Truck-Mounted Drill Rig

DRILLING CONTRACTOR: Geo-environmental Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jconn	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole DR-9		99.29															
	Rotary Diamond Drill, HQ Size Core Barrel	Completely to slightly weathered, thinly bedded, grey, fine grained, slightly porous, very weak to medium stong SHALE (Georgian Bay Formation) with medium strong to strong LIMESTONE interbeds		35.21	1		80 60 40 20	80 60 40 20	80 60 40 20	5 10 15 20	0 30 60 90	BD,PL,SM,PC, CL BD,CU,RO,SO	1 2	4 1	12 25				
36												BD,CU,RO,SO	2	1	25			BC BC	
					2							BD,ST,RO,PC, CL BD,ST,RO,SO	2 2	4 1	12 25			BC	
37												BD,PL,SM,SO	1	1	16				
		LIMESTONE/SILTSTONE Layers: 37.88 m - 37.90 m, 38.33 m - 38.35 m, 38.45 m - 38.47 m, 38.56 m - 38.58 m, 38.80 m - 38.82 m, 39.09 m - 39.10 m UCS = 34.6 MPa																	BC	
38					3							BD,PL,SM,PC, CL	1	4	12				
												BD,CU,SM,SA	2	2	26				
39				95.16								BD,PL,SM,PC, CL BD,ST,RO,SO	1 2	4 1	12 25				
		END OF DRILLHOLE		39.34															
40																			
41																			
42																			
43																			
44																			
45																			

DEPTH SCALE
1 : 50

GOLDER
MEMBER OF WSP

LOGGED: AM
CHECKED: DH

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RECORD OF BOREHOLE No DR-10

SHEET 1 OF 2

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847357.5; E 316171.2 MTM NAD 83 ZONE 10 (LAT. 43.766098; LONG. -79.358769)

ORIGINATED BY ML

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 3, 2019

CHECKED BY DH

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
134.4	GROUND SURFACE
0.0	ASPHALT (150 mm)
	GRANULAR (80 mm)
133.8	CONCRETE (70 mm)
0.6	SAND (SP) and gravel (FILL) Compact Brown Moist
132.7	Sandy CLAYEY SILT-SILT (CL-ML), trace to some gravel (FILL) Firm to very stiff Brown to grey at 8.7 m Moist
1.8	SAND (SP) (FILL) Compact Brown Moist
	Sandy CLAYEY SILT-SILT (CL-ML), trace to some gravel (FILL) Firm to very stiff Brown to grey at 8.7 m Moist
124.2	
10.2	Sandy SILTY CLAY (CI), containing rootlets Stiff Greyish black Moist
123.2	
11.2	SILTY SAND (SM) Loose Grey Wet
122.7	
11.7	SILTY CLAY (CI), trace sand, trace gravel, contains sand seams Flrm to very stiff Brown to grey Wet

STRAT PLOT

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	14
2	SS	10
3A 3B 3C	SS	15
4	SS	15
5	SS	10
6	SS	11
7A 7B	SS	15
8	SS	17
9	SS	5
10A 10B	SS	9
11	SS	7
12	SS	5

GROUND WATER CONDITIONS

ELEVATION SCALE: 134, 133, 132, 131, 130, 129, 128, 127, 126, 125, 124, 123, 122, 121, 120

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

SAMPLE	GR	SA	SI	CL
4	3	35	51	11
10A 10B	0	24	70	6
11	1	8	52	39

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DR-10

SHEET 2 OF 2

METRIC

PROJECT 1786302

G.W.P. 2130-01-00

LOCATION N 4847357.5; E 316171.2 MTM NAD 83 ZONE 10 (LAT. 43.766098; LONG. -79.358769)

ORIGINATED BY ML

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 3, 2019

CHECKED BY DH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
119.0			13A				119				
118.6	Sandy SILT (ML), trace gravel Compact Grey Wet		13B 13C	SS	27				○		1 34 65 0
15.9	SILTY CLAY (CI), trace sand, trace gravel Very stiff Grey Wet										
	END OF BOREHOLE										

NOTES:

1. Groundwater level measured inside augers at 11.4 m below ground surface (Elev. 123.0 m) upon completion of drilling.

2. Borehole open to 14.0 m and groundwater level measured at 10.1 m below ground surface (Elev. 120.4 m) upon removal of augers.

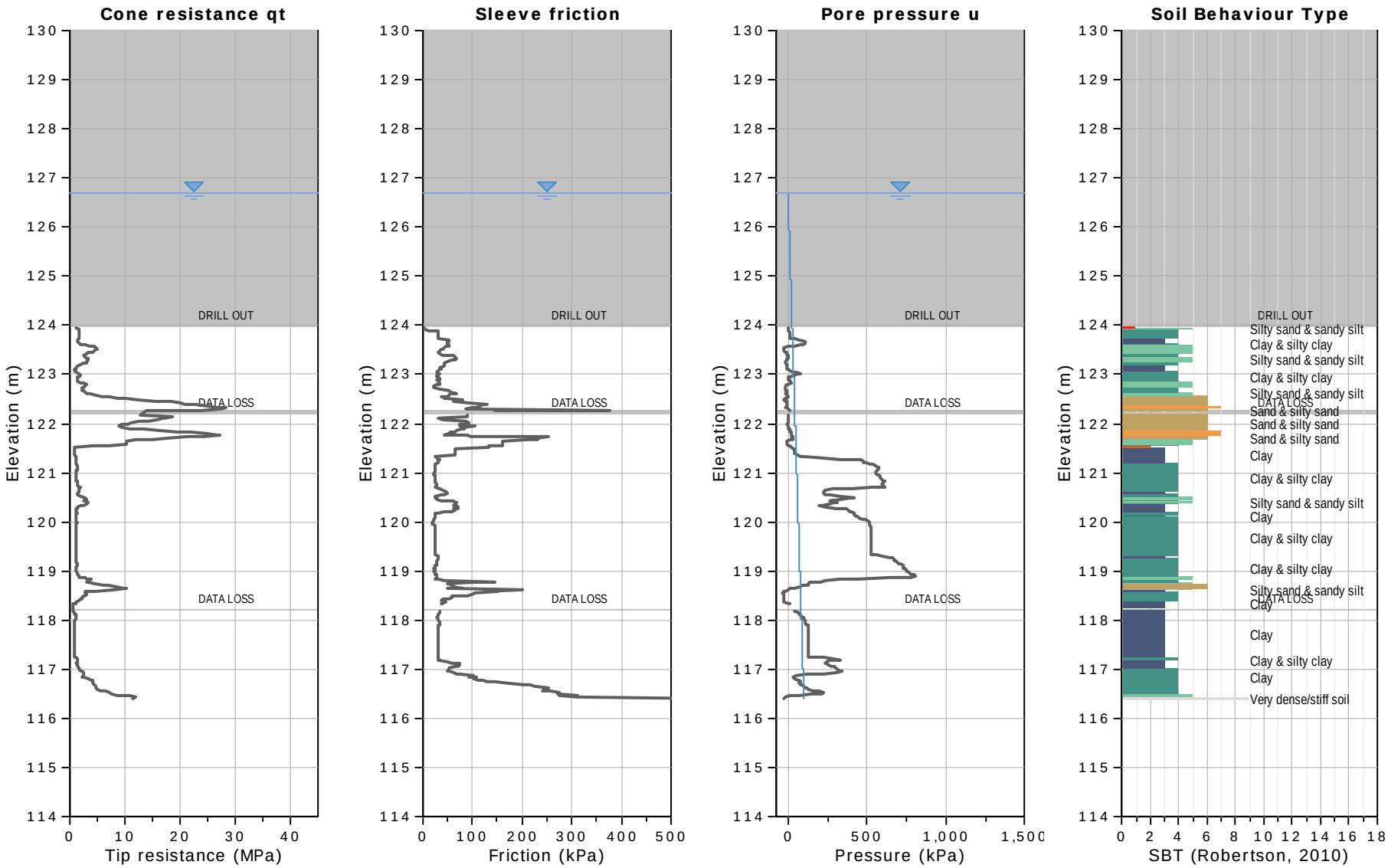
+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Golder Associates Ltd.
Geotechnical Department
Mississauga, Ontario
Project No. 1786302

CPT: DR-2a
Total depth: 13.65 m, Date: 2019-08-26
Surface Elevation: 130.06 m
Coords: X:4847305.47, Y:316039.15
Cone Type: Geoprobe #3742
Cone Operator: Darcy Hansen

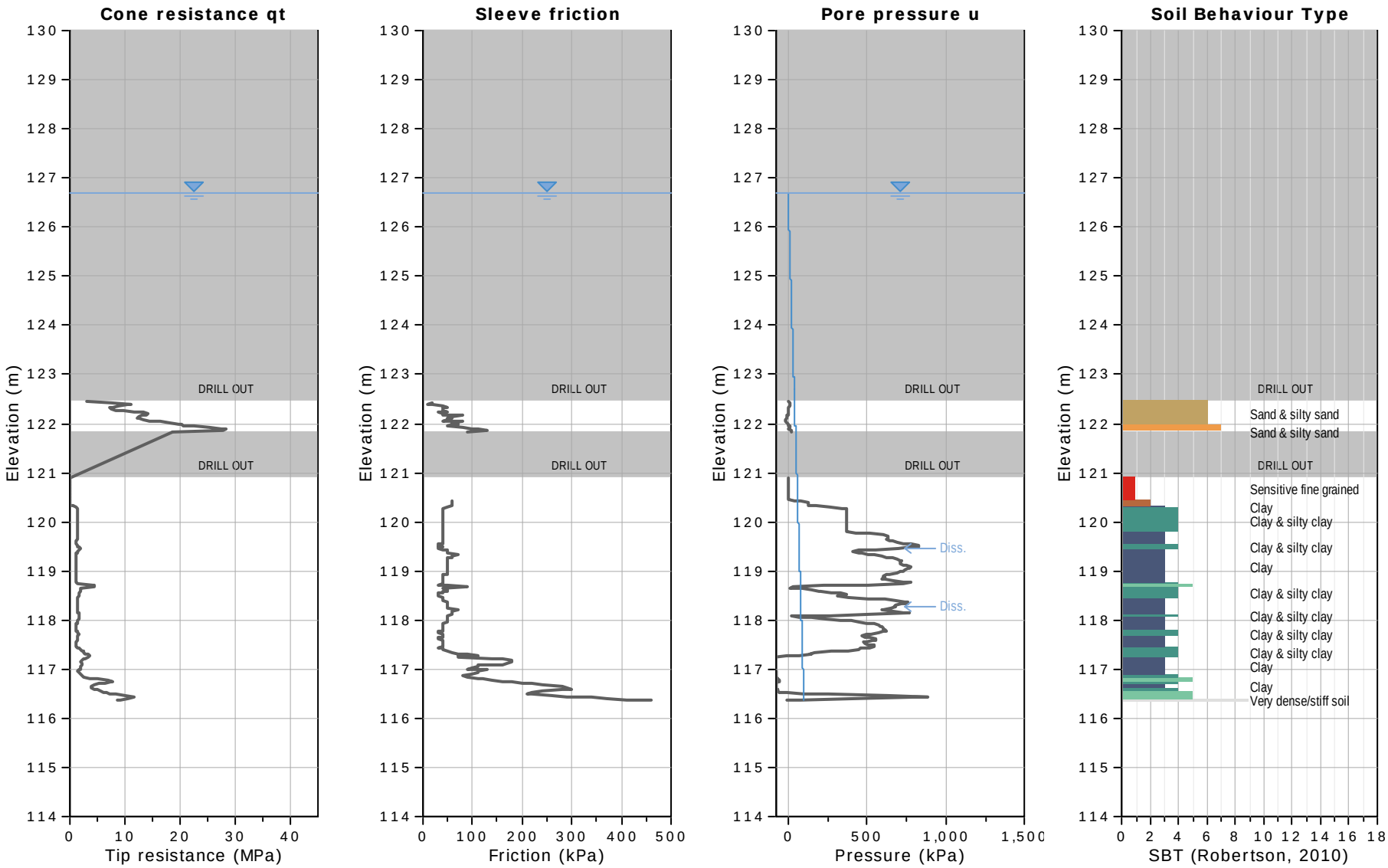
Project: **Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue**
Location: **North York, Ontario (MTO Assignment No. 2016-E-0089)**



Golder Associates Ltd.
Geotechnical Department
Mississauga, Ontario
Project No. 1786302

CPT: DR-2b
Total depth: 13.70 m, Date: 2019-08-26
Surface Elevation: 130.06 m
Coords: X:4847305.47, Y:316039.15
Cone Type: Geoprobe #3742
Cone Operator: Darcy Hansen

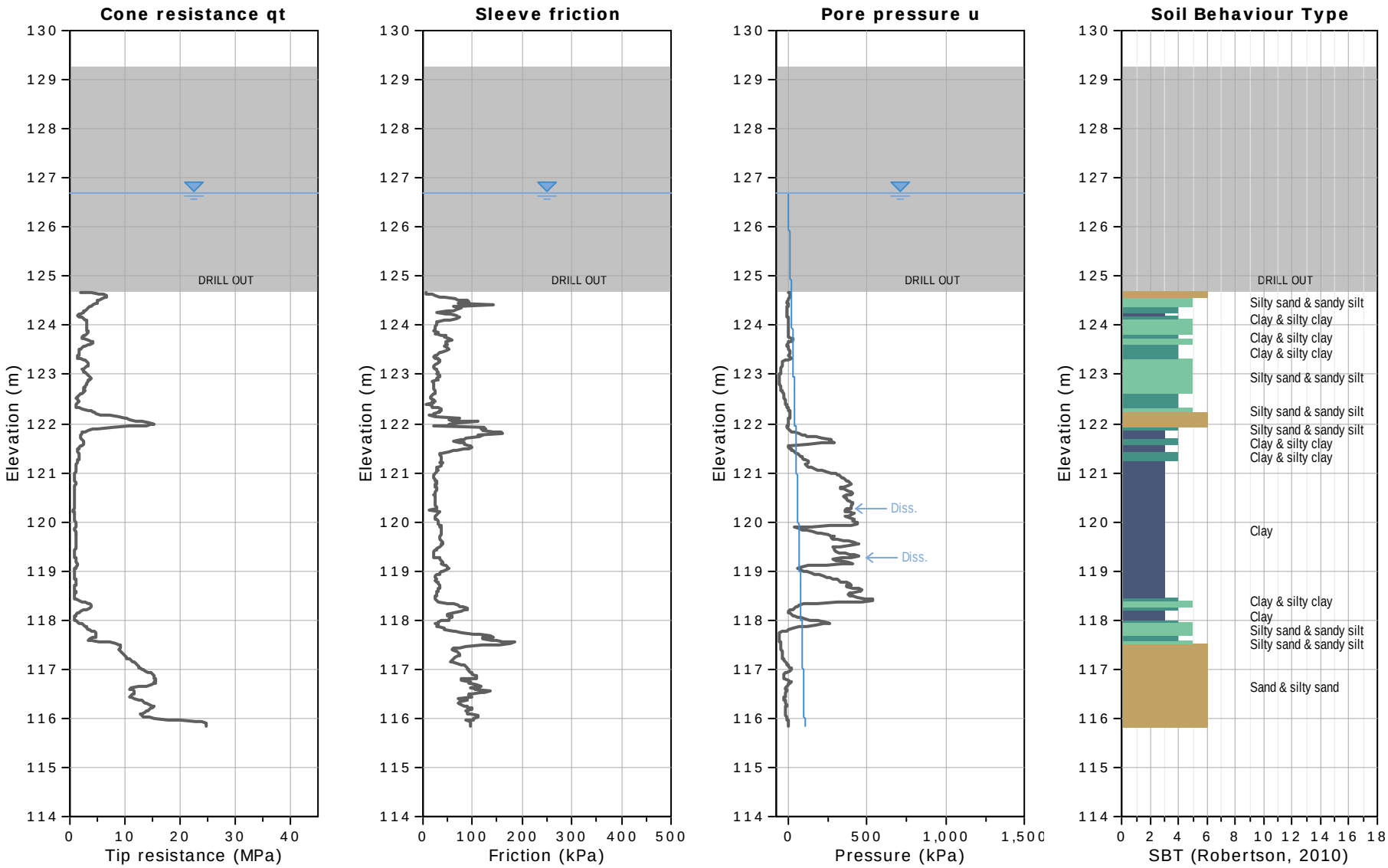
Project: **Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue**
Location: **North York, Ontario (MTO Assignment No. 2016-E-0089)**



Golder Associates Ltd.
Geotechnical Department
Mississauga, Ontario
Project No. 1786302

CPT: DR-4
Total depth: 13.44 m, Date: 2019-08-27
Surface Elevation: 129.28 m
Coords: X:4847328.03, Y:316067.62
Cone Type: Geoprobe #3742
Cone Operator: Darcy Hansen

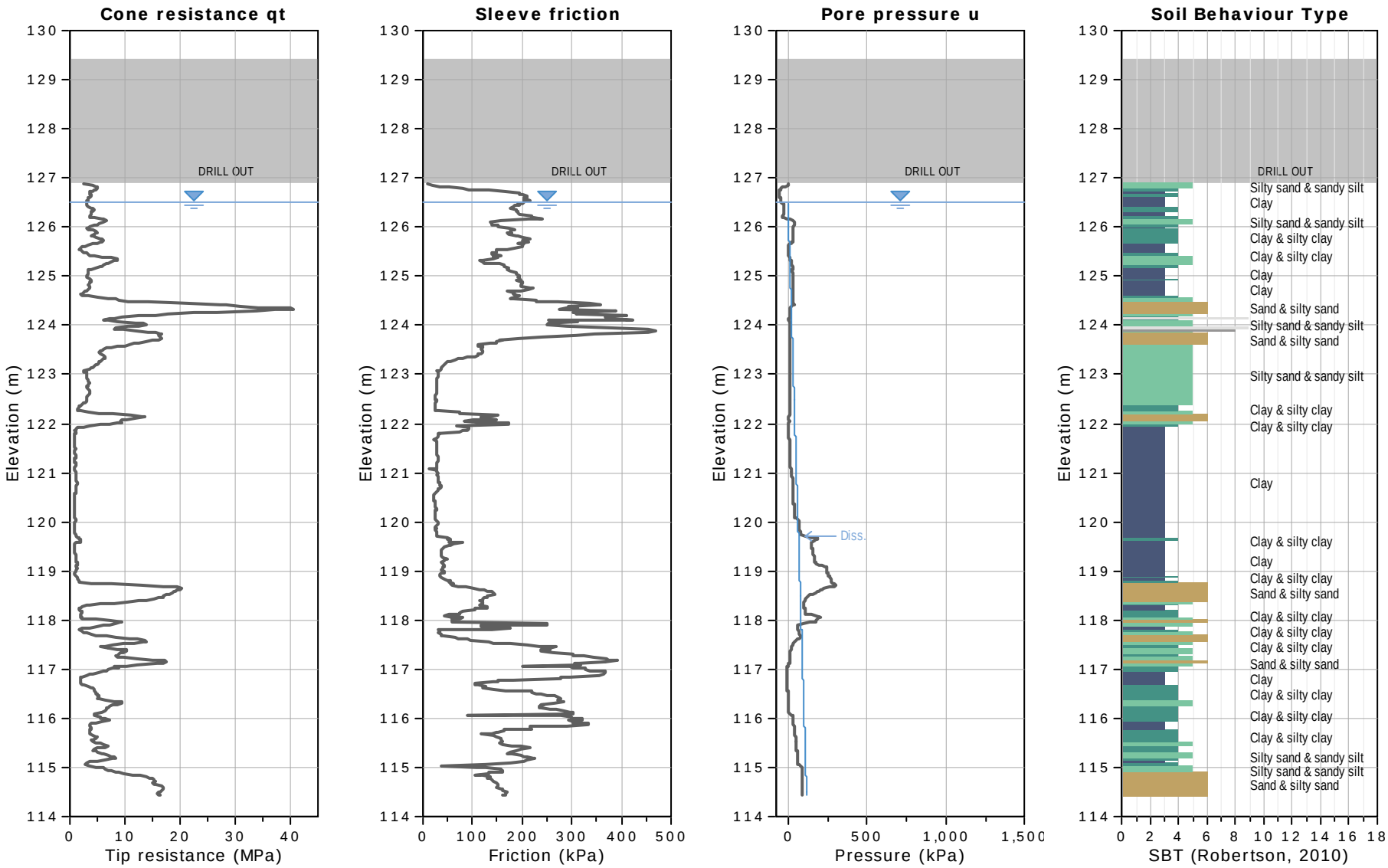
Project: **Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue**
Location: **North York, Ontario (MTO Assignment No. 2016-E-0089)**



Golder Associates Ltd.
Geotechnical Department
Mississauga, Ontario
Project No. 1786302

CPT: DR-8a
Total depth: 15.00 m, Date: 2019-08-19
Surface Elevation: 129.43 m
Coords: X:4847337.40, Y:316158.62
Cone Type: Geoprobe #3936
Cone Operator: Darcy Hansen / Alex Szot

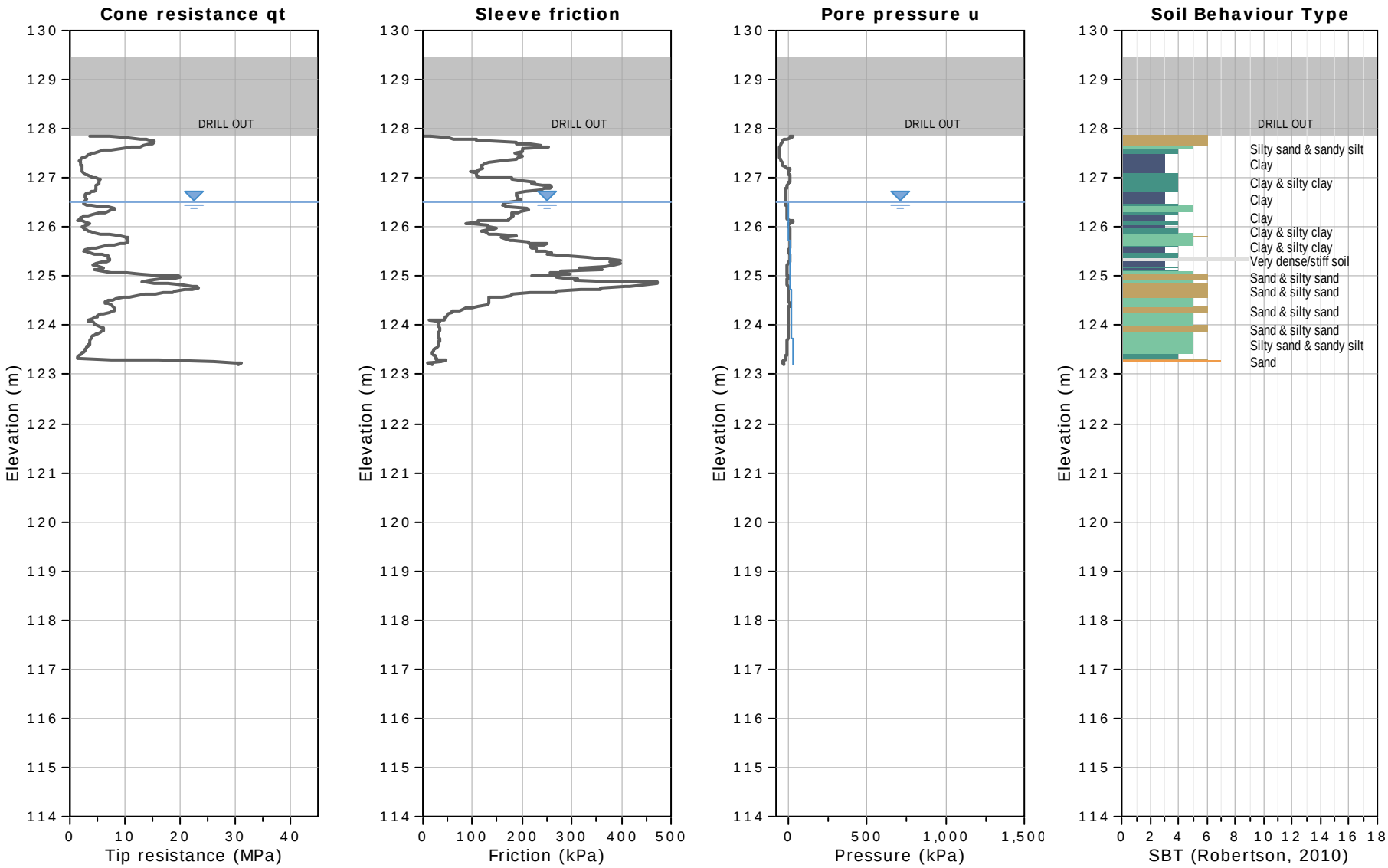
Project: **Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue**
Location: **North York, Ontario (MTO Assignment No. 2016-E-0089)**

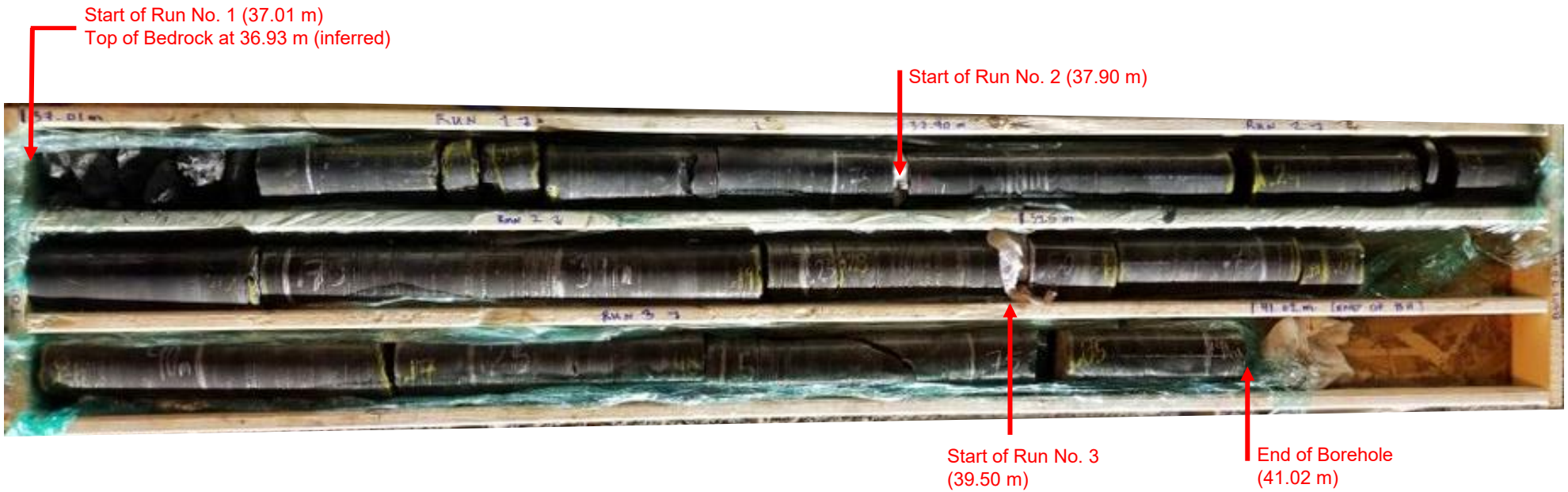


Golder Associates Ltd.
Geotechnical Department
Mississauga, Ontario
Project No. 1786302

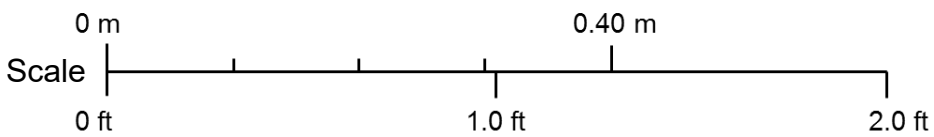
CPT: DR-8b
Total depth: 6.23 m, Date: 2019-08-19
Surface Elevation: 129.43 m
Coords: X:4847337.40, Y:316158.62
Cone Type: Geoprobe #3936
Cone Operator: Darcy Hansen / Alex Szot

Project: **Highway 401 Eastbound Collector Lanes, Avenue Road to Warden Avenue**
Location: **North York, Ontario (MTO Assignment No. 2016-E-0089)**





Borehole DR-3: Bedrock cored between depths of about 37.01 m to 41.02 m



PROJECT	**Highway 401 EB Collectors Avenue to Warden, (Site No. 37-207/1) North York, Toronto, Ontario**				
TITLE	**BEDROCK CORE PHOTOGRAPH BOREHOLE DR-3**				
GOLDER MEMBER OF WSP	PROJECT No. 1786302			FILE No. ----	
	DESIGN	RM	20190724	SCALE NTS	VER. 1.
	CADD	--		**FIGURE B1**	
	CHECK	KN	20191224		
	REVIEW	CNN	20200716		

REVISION DATE: 20191224 BY: KN Project: 1786302

Start of Run No. 1 (27.79 m)

Start of Run No. 2
(29.26 m)

End of Borehole
(30.75 m)

Borehole DR-6: Bedrock cored between depths of about 27.79 m to 30.75 m

Scale

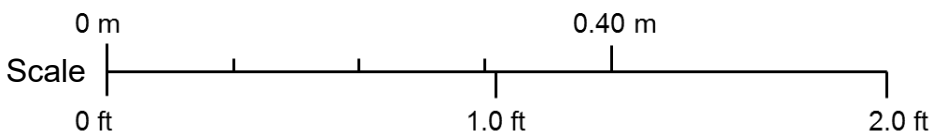
0 m — 0.40 m

0 ft — 1.0 ft — 2.0 ft

PROJECT	**Highway 401 EB Collectors Avenue to Warden, (Site No. 37-207/1) North York, Toronto, Ontario**				
TITLE	**BEDROCK CORE PHOTOGRAPH BOREHOLE DR-6**				
GOLDER MEMBER OF WSP	PROJECT No. 1786302			FILE No. ----	
	DESIGN	RM	20190724	SCALE NTS	VER. 1.
	CADD	--		**FIGURE B2**	
	CHECK	KN	20191224		
	REVIEW	CNN	20200716		

REVISION DATE: 20191224 BY: KN Project: 1786302

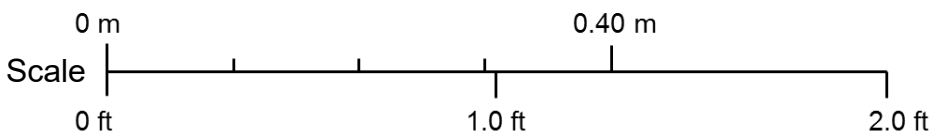
Borehole DR-7: Bedrock cored between depths of about 29.31 m to 30.84 m



PROJECT	**Highway 401 EB Collectors Avenue to Warden, (Site No. 37-207/1) North York, Toronto, Ontario**				
TITLE	**BEDROCK CORE PHOTOGRAPH BOREHOLE DR-7**				
GOLDER MEMBER OF WSP	PROJECT No. 1786302			FILE No. ----	
	DESIGN	RM	20190724	SCALE NTS	VER. 1.
	CADD	--		**FIGURE B3**	
	CHECK	KN	20191224		
	REVIEW	CNN	20200716		

REVISION DATE: 20191224 BY: KN Project: 1786302

Borehole DR-9: Bedrock cored between depths of about 35.20 m to 39.34 m



PROJECT	**Highway 401 EB Collectors Avenue to Warden, (Site No. 37-207/1) North York, Toronto, Ontario**				
TITLE	**BEDROCK CORE PHOTOGRAPH BOREHOLE DR-9**				
GOLDER MEMBER OF WSP	PROJECT No. 1786302			FILE No. ----	
	DESIGN	RM	20190724	SCALE NTS	VER. 1.
	CADD	--		**FIGURE B4**	
	CHECK	KN	20191224		
	REVIEW	CNN	20200716		

REVISION DATE: 20191224 BY: KN Project: 1786302

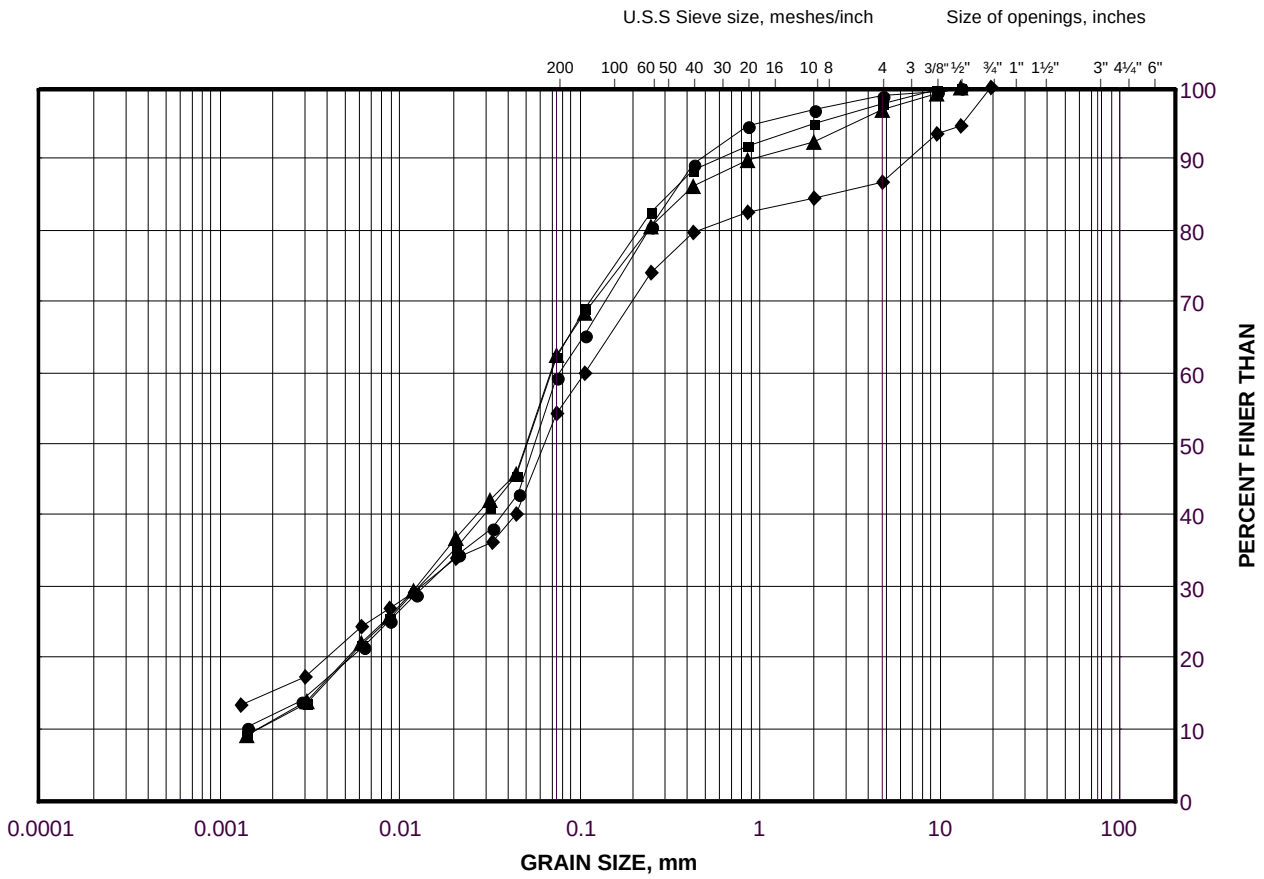
APPENDIX C

Geotechnical and Analytical Test Results

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML/SC) and Sand (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)
●	DR-7	4	126.7
■	DR-10	4	131.8
◆	DR-5	5	125.8
▲	DR-9	6	129.6

Project Number: 1786302

Checked By: KN

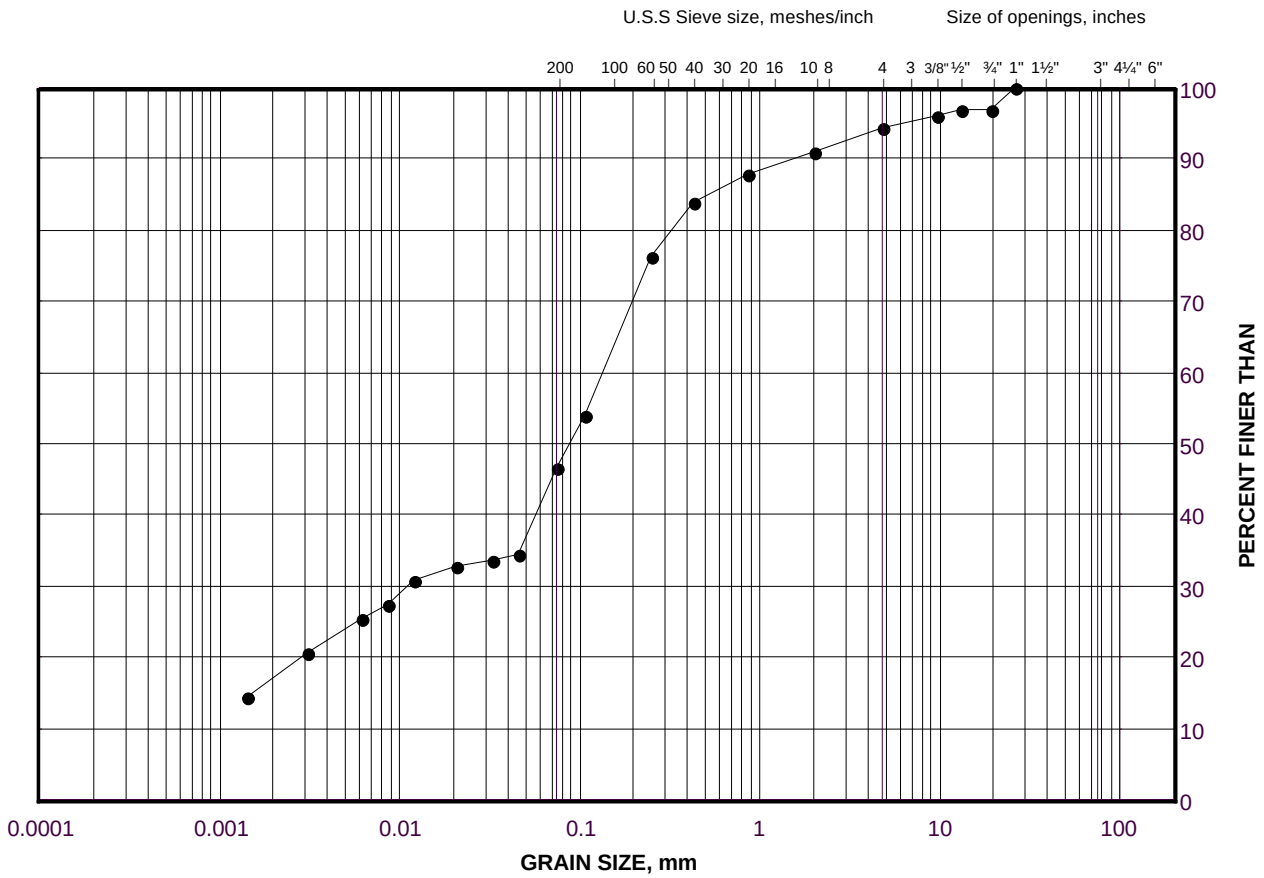
Golder Associates

Date: 06-Aug-20

GRAIN SIZE DISTRIBUTION

CLAYEY SAND (SC) (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)
●	DR-9	10	123.5

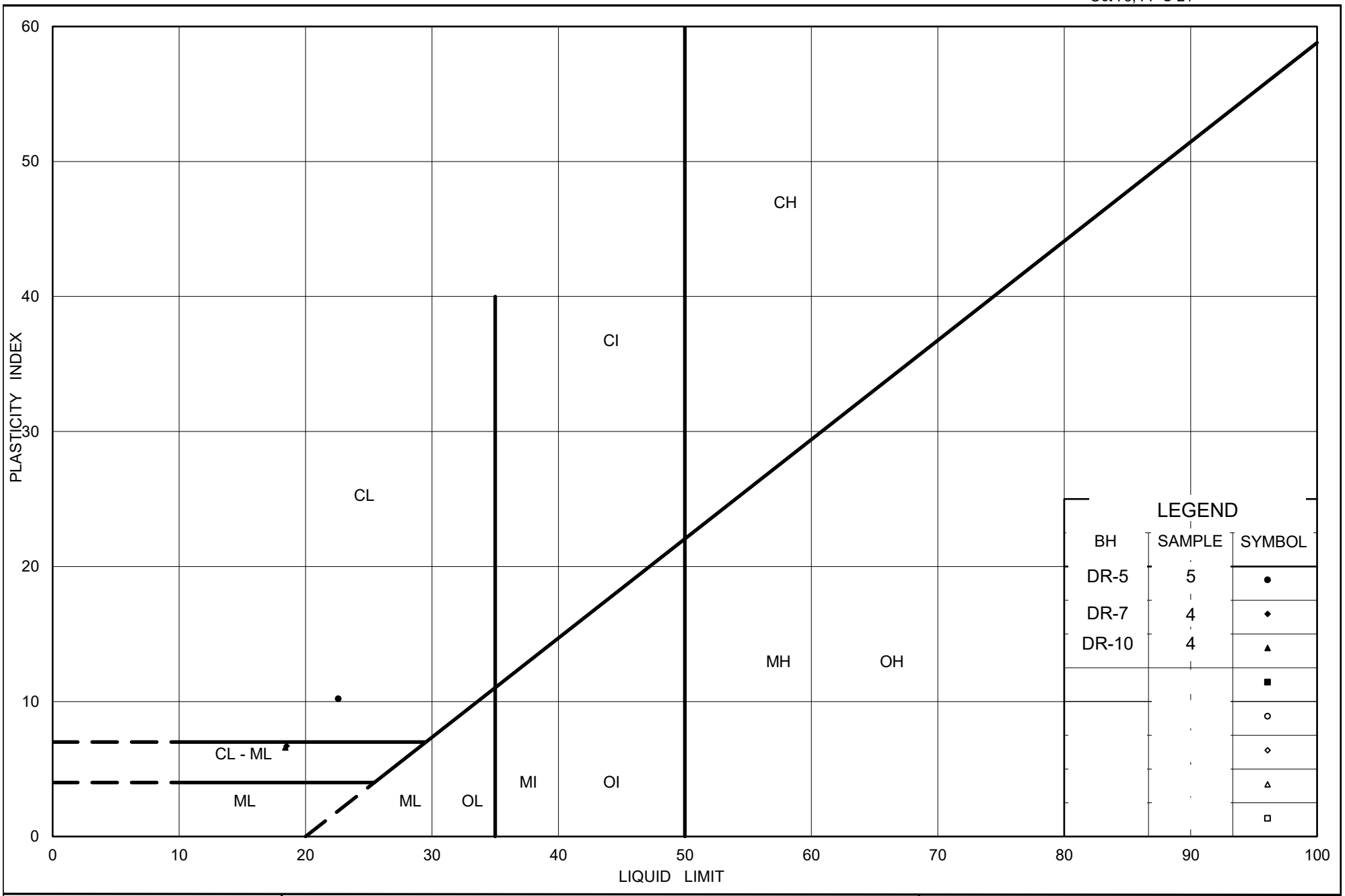
Project Number: 1786302

Checked By: KN

Golder Associates

Date: 06-Aug-20

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

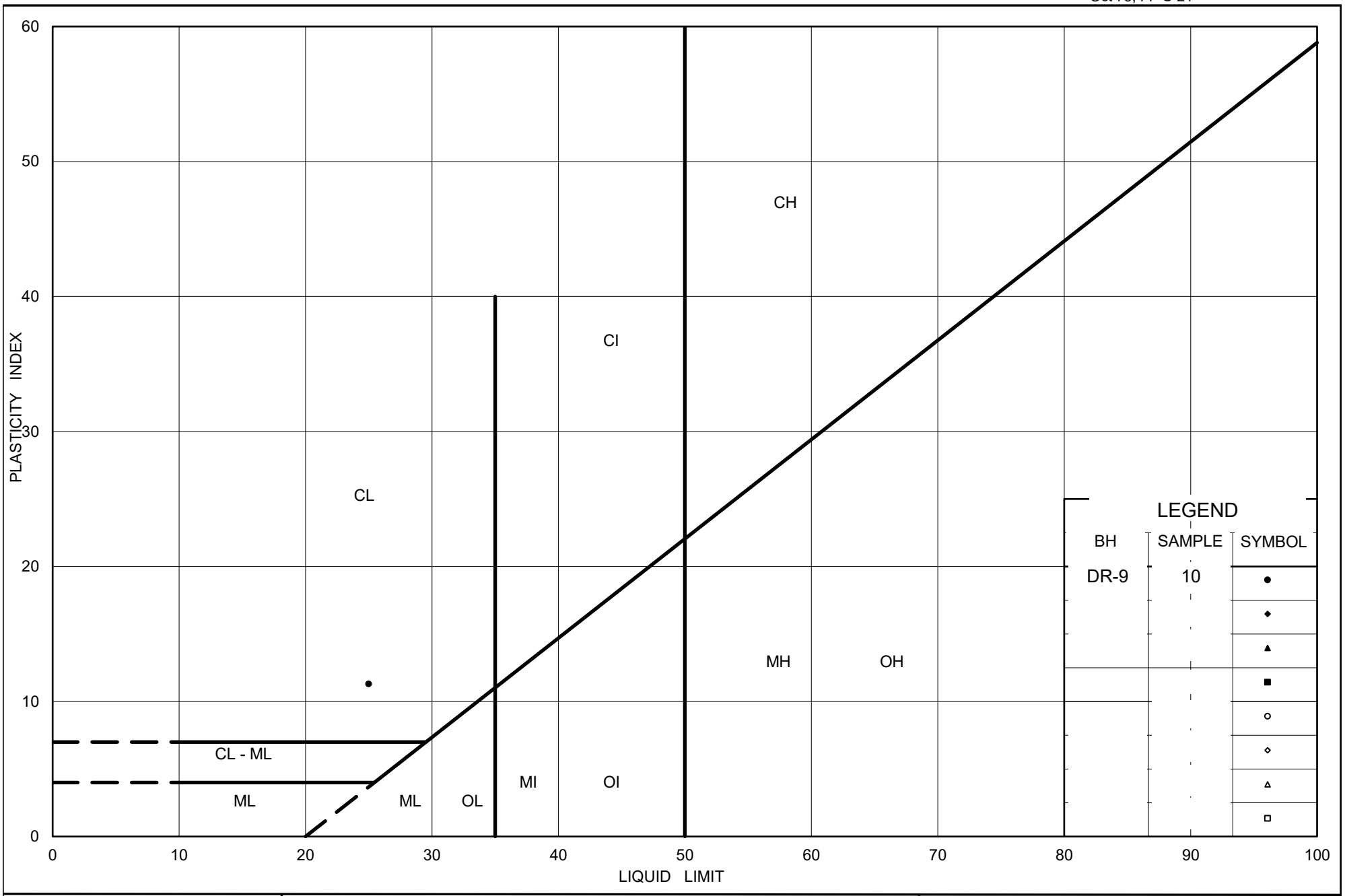
Sandy CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML/SC) and Sand (FILL)

Figure No. C3

Project No. 1786302

Checked By: KN

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

CLAYEY SAND (SC) (FILL)

Figure No. C4

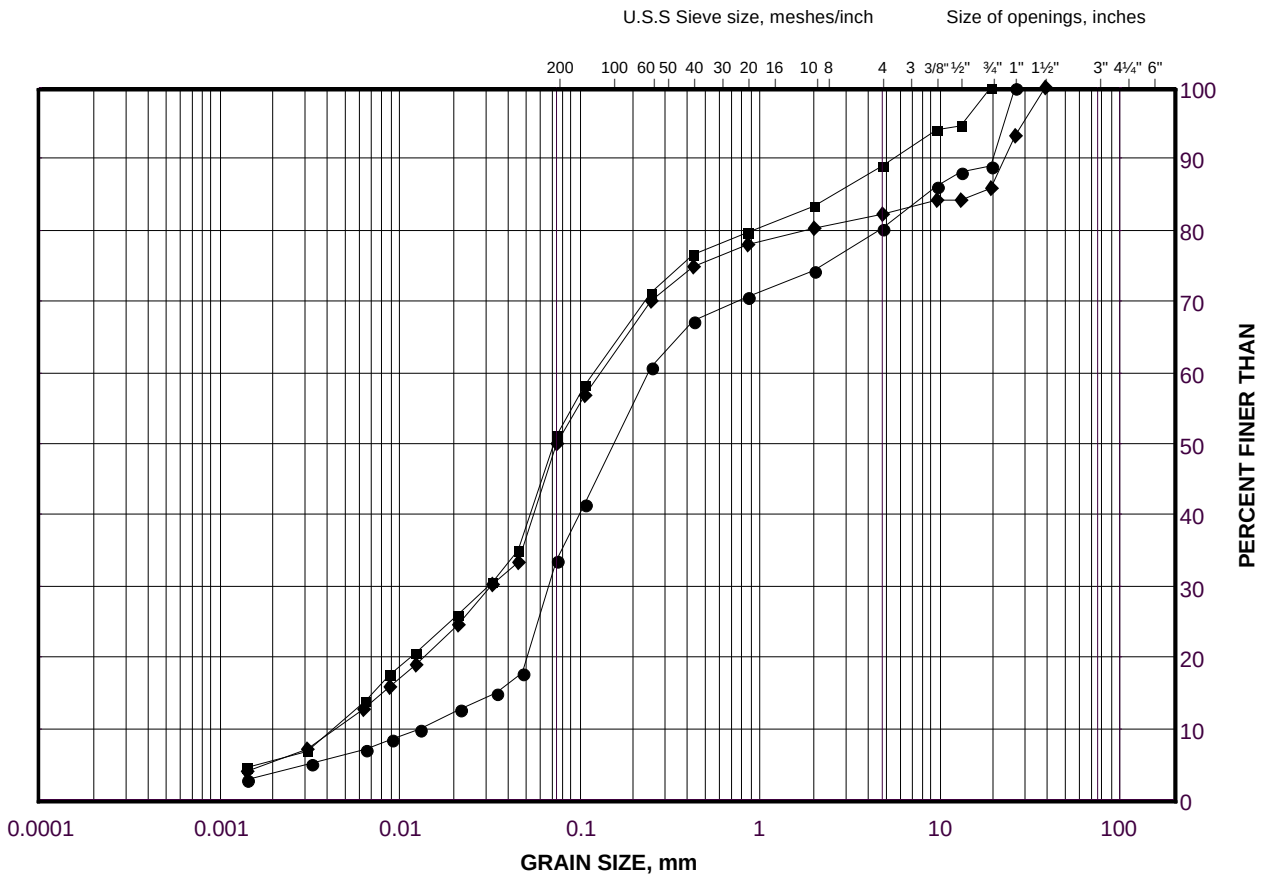
Project No. 1786302

Checked By: KN

GRAIN SIZE DISTRIBUTION

Sandy SILT of Slight Plasticity (ML) to SILTY SAND (SM)

FIGURE C5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-3	14	122.9
■	DR-3	7	132.1
◆	DR-1	8	132.1

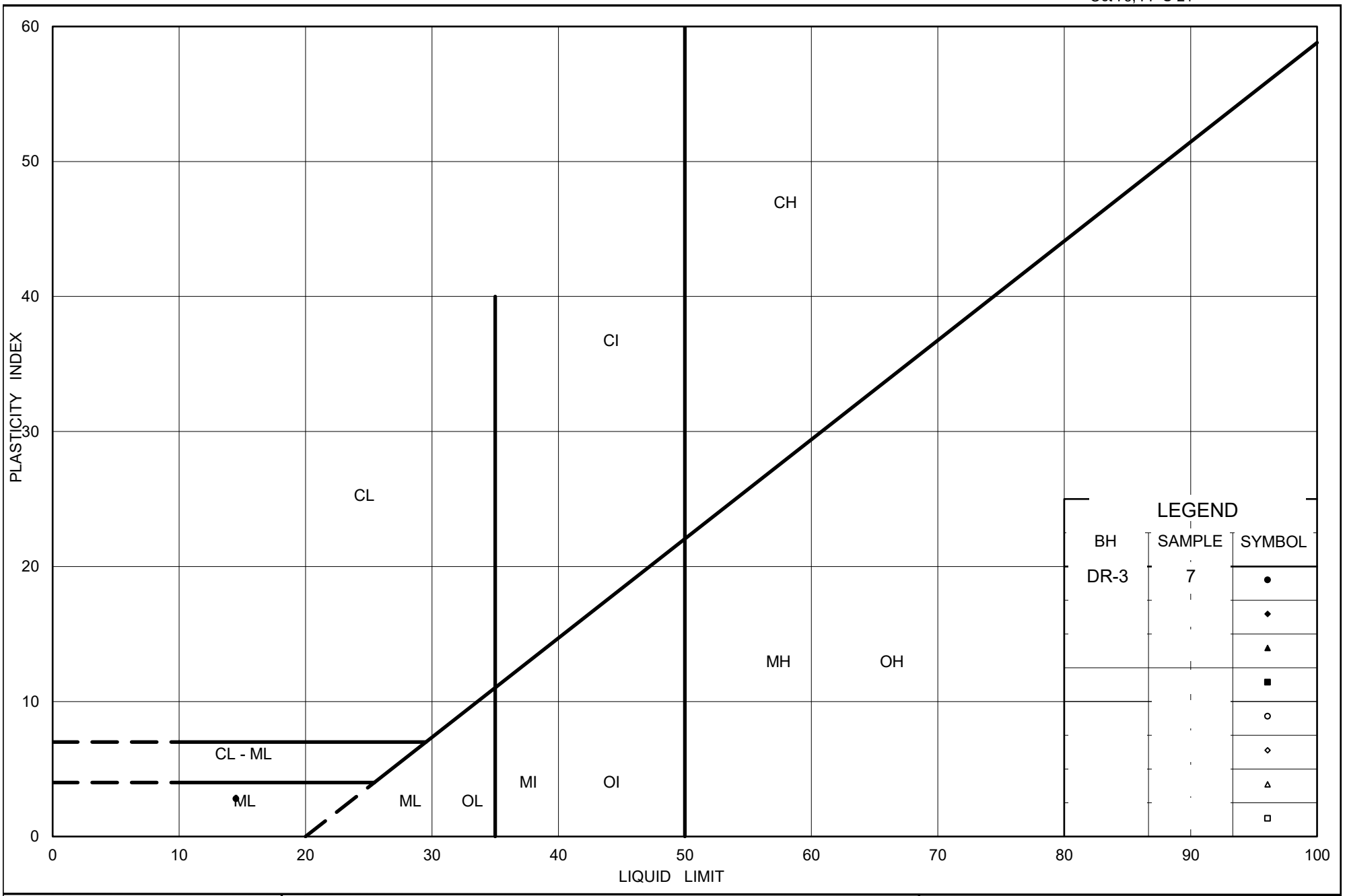
Project Number: 1786302

Checked By: KN

Golder Associates

Date: 06-Aug-20

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy SILT of Slight Plasticity (ML)

Figure No. C6

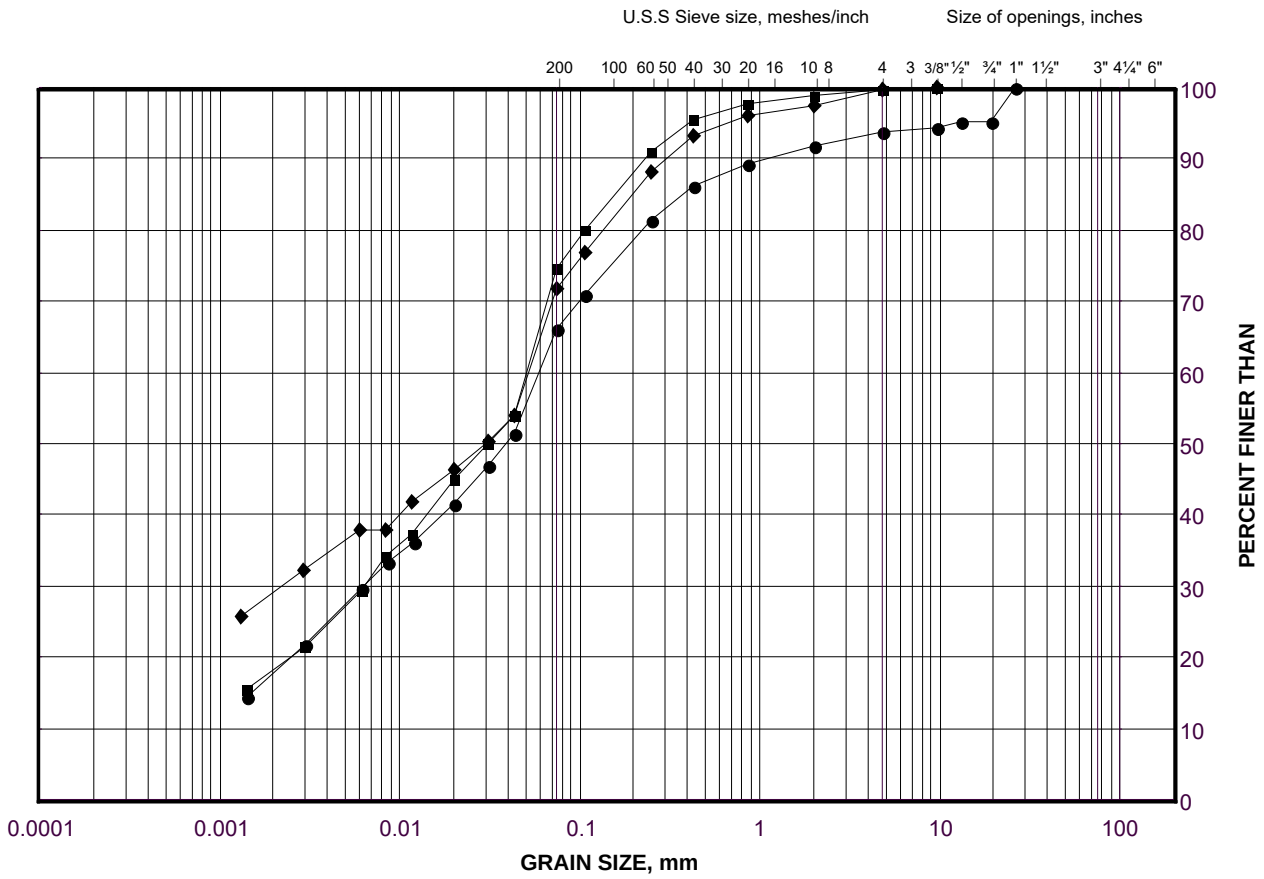
Project No. 1786302

Checked By: KN

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT (CL) - Interlayer

FIGURE C7



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-1	11A	127.7
■	DR-3	12	125.2
◆	DR-1	12	126.0

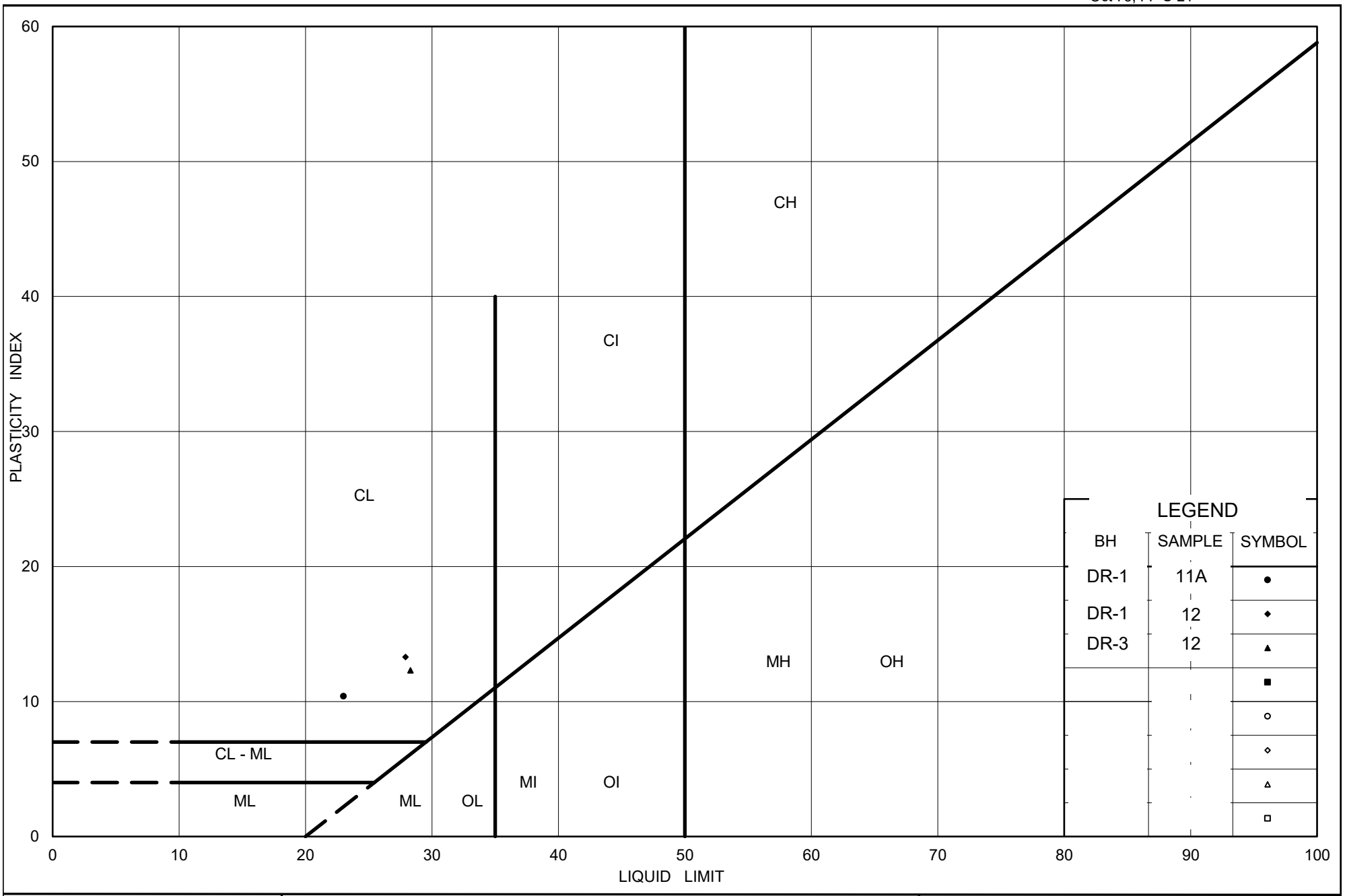
Project Number: 1786302

Checked By: KN

Golder Associates

Date: 06-Aug-20

Oct 75, FF-S-21



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Ontario

PLASTICITY CHART

Sandy CLAYEY SILT (CL) - Interlayer

Figure No. C8

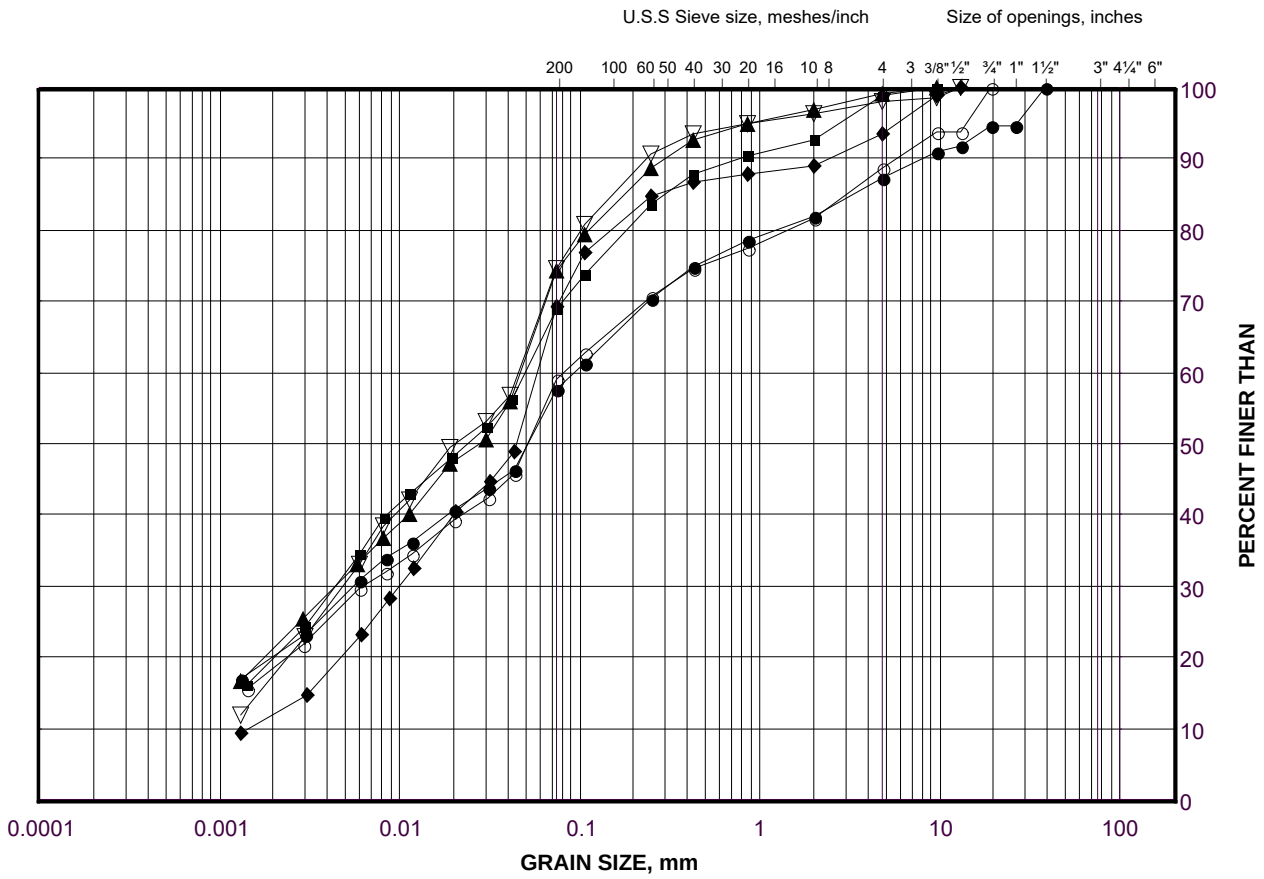
Project No. 1786302

Checked By: KN

GRAIN SIZE DISTRIBUTION

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

FIGURE C9A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-5	10	119.8
■	DR-6	12	115.5
◆	DR-5	13A	116.8
▲	DR-9	13A	119.2
▽	DR-9	14B	115.8
○	DR-6	9	120.1

Project Number: 1786302

Checked By: KN

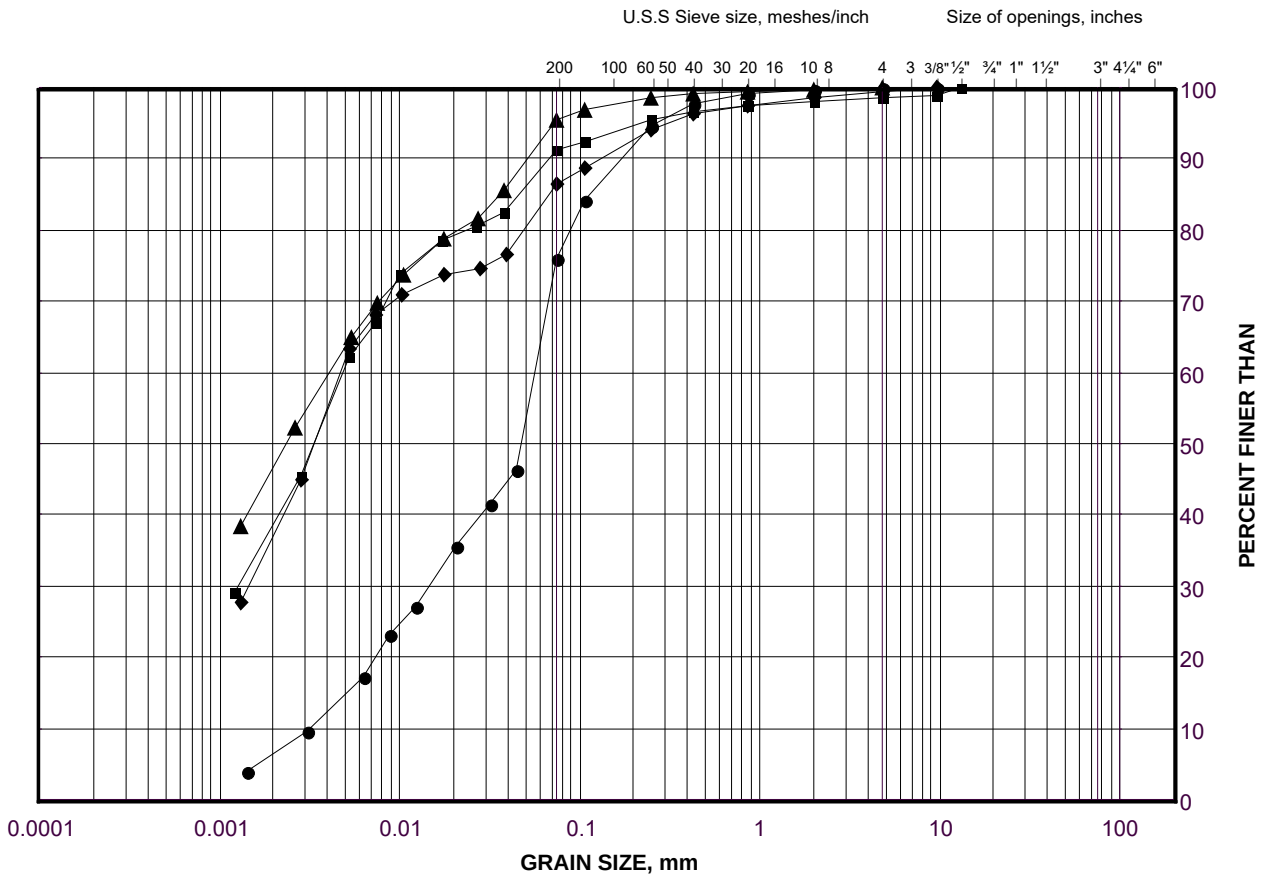
Golder Associates

Date: 06-Aug-20

GRAIN SIZE DISTRIBUTION

SILTY CLAY (CI) to Sandy SILTY CLAY(CI) to CLAYEY SILT (CL)

FIGURE C9B



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

LEGEND

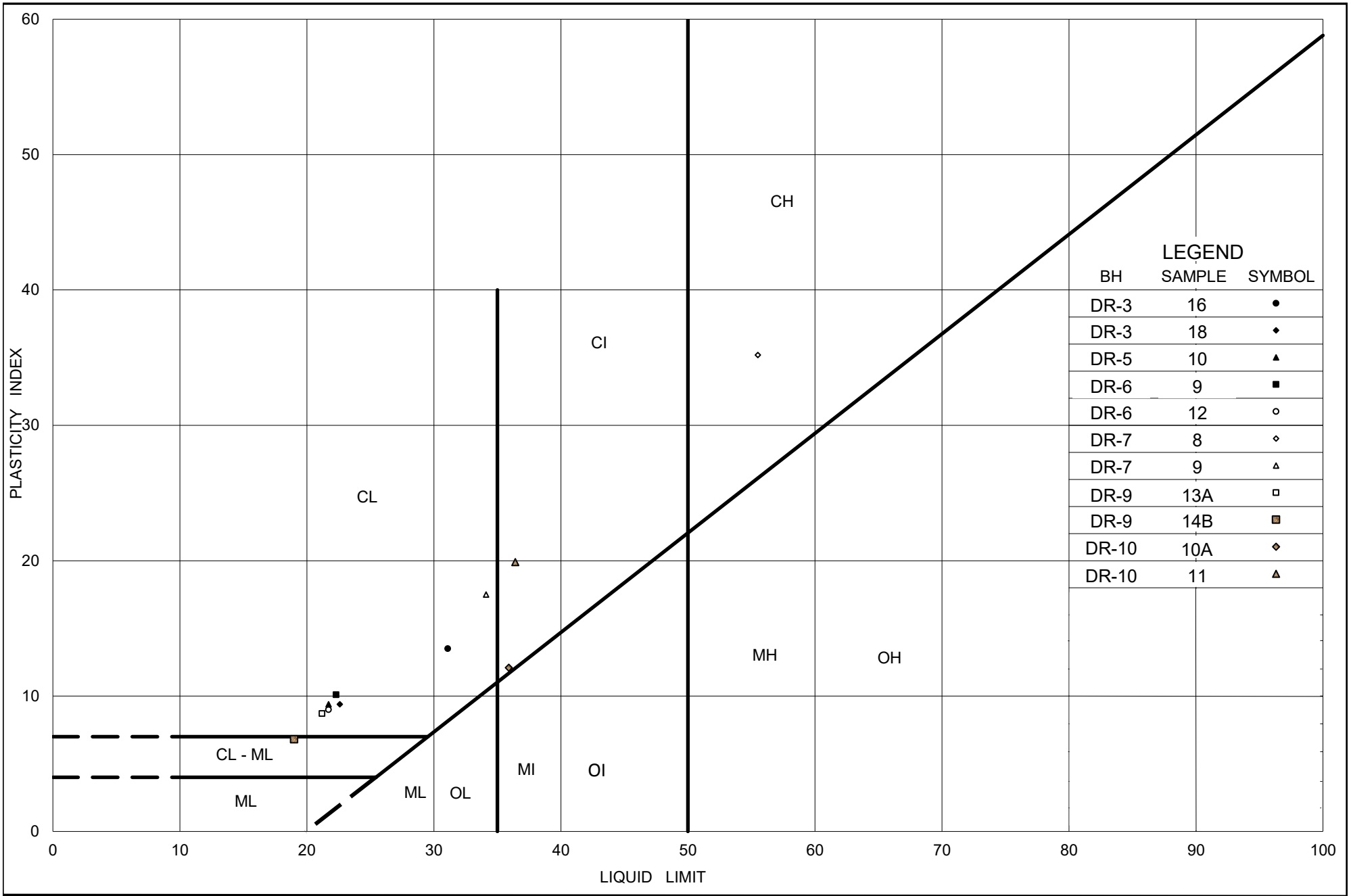
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-10	10A	123.4
■	DR-10	11	121.9
◆	DR-3	16	118.4
▲	DR-7	9	120.6

Project Number: 1786302

Checked By: KN

Golder Associates

Date: 06-Aug-20



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PLASTICITY CHART

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

Figure No. C10

Project No. 1786302

Checked By: KN

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE 11A

SAMPLE IDENTIFICATION

Project Number	1786302 (1000)	Sample Number	8
Borehole Number	DR-7	Sample Depth, m	7.62-8.15

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	10/30/2019		
Date Completed	11/15/2019		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	20.95
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.75
Area, cm^2	31.65	Specific Gravity, measured	2.74
Volume, cm^3	80.29	Solids Height, cm	1.676
Water Content, %	18.02	Volume of Solids, cm^3	53.05
Wet Mass, g	171.57	Volume of Voids, cm^3	27.24
Dry Mass, g	145.37	Degree of Saturation, %	96.2

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	2.537	0.513	2.537				
6.01	2.539	0.514	2.538				
10.64	2.537	0.513	2.538	101	1.35E-02	1.22E-04	1.62E-07
20.68	2.535	0.512	2.536	217	6.28E-03	8.51E-05	5.24E-08
40.08	2.526	0.507	2.531	154	8.82E-03	1.75E-04	1.51E-07
78.62	2.507	0.496	2.517	317	4.24E-03	1.94E-04	8.07E-08
151.00	2.484	0.482	2.495	135	9.78E-03	1.29E-04	1.24E-07
40.09	2.489	0.485	2.486				
10.69	2.496	0.489	2.493				
39.89	2.492	0.487	2.494	101	1.31E-02	5.67E-05	7.26E-08
151.00	2.480	0.479	2.486	60	2.18E-02	4.51E-05	9.64E-08
310.78	2.447	0.460	2.463	217	5.93E-03	7.92E-05	4.60E-08
620.36	2.405	0.435	2.426	154	8.10E-03	5.42E-05	4.31E-08
1239.31	2.349	0.401	2.377	135	8.87E-03	3.59E-05	3.12E-08
2476.57	2.286	0.364	2.317	101	1.13E-02	2.00E-05	2.21E-08
620.36	2.295	0.369	2.290				
151.00	2.314	0.380	2.304				
39.83	2.333	0.392	2.323				
10.76	2.353	0.404	2.343				

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 36-42cm from top of the tube.

Specimen swelled under 5 kPa

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.35	Unit Weight, kN/m^3	21.99
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.14
Area, cm^2	31.65	Specific Gravity, measured	2.74
Volume, cm^3	74.48	Solids Height, cm	1.676
Water Content, %	14.86	Volume of Solids, cm^3	53.05
Wet Mass, g	166.97	Volume of Voids, cm^3	21.42
Dry Mass, g	145.37		

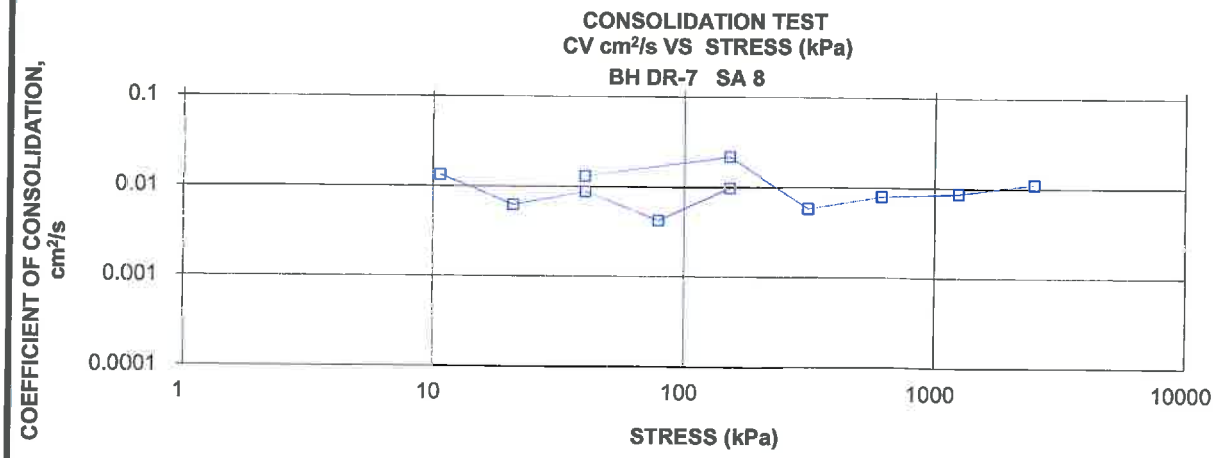
Prepared By: SJ

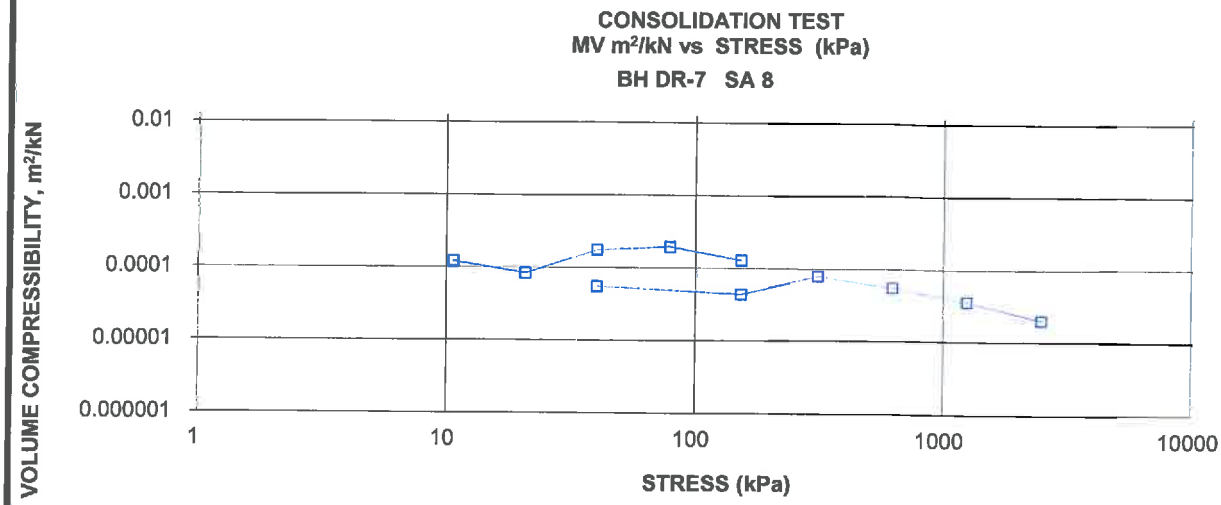
Golder Associates

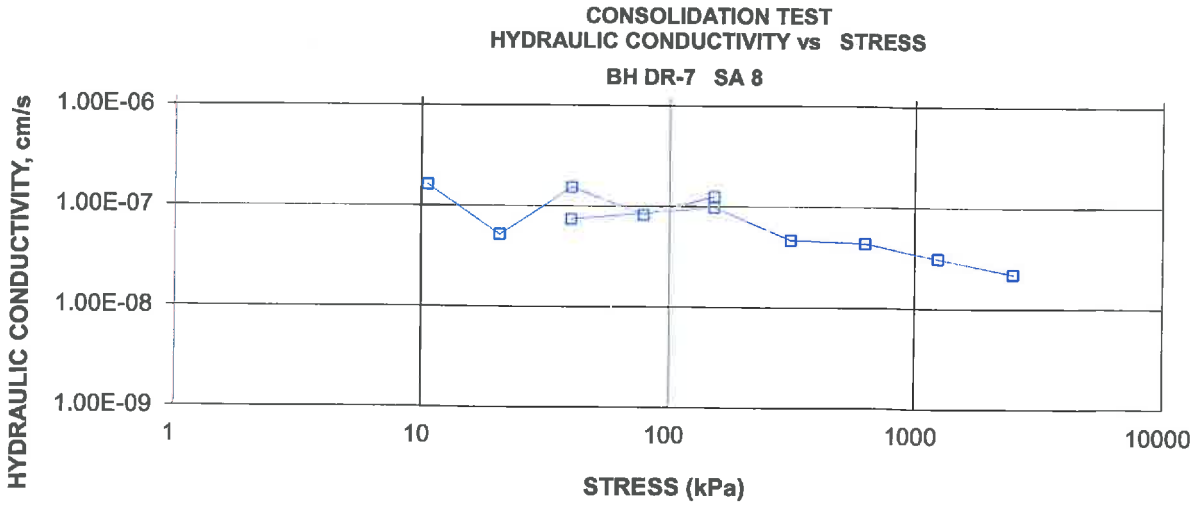
Checked By: hlp

CONSOLIDATION TEST SUMMARY

FIGURE 11B







Project No. 1786302 (1000)

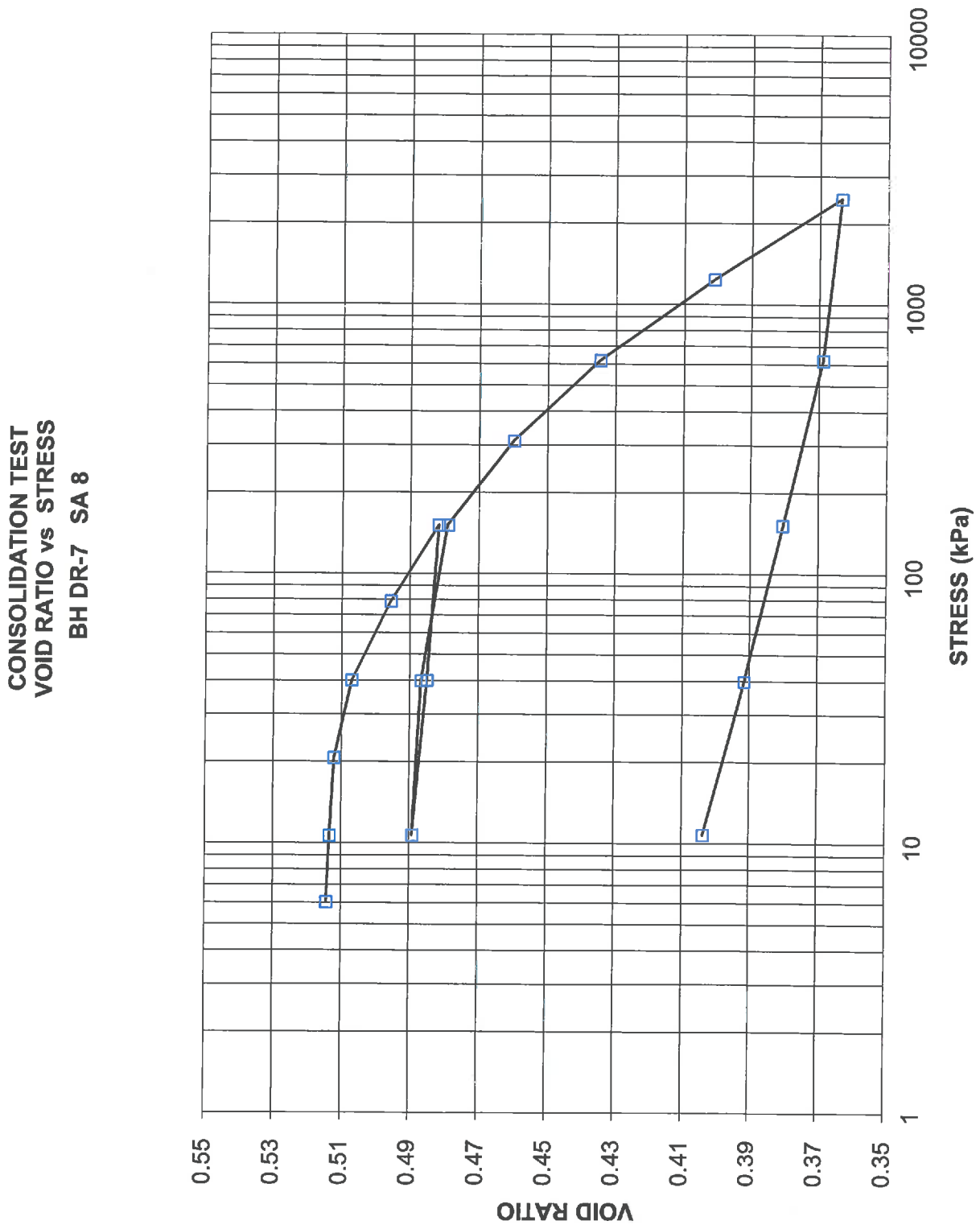
Prepared By: SJ

Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE 11C



Project No. 1786302 (1000)

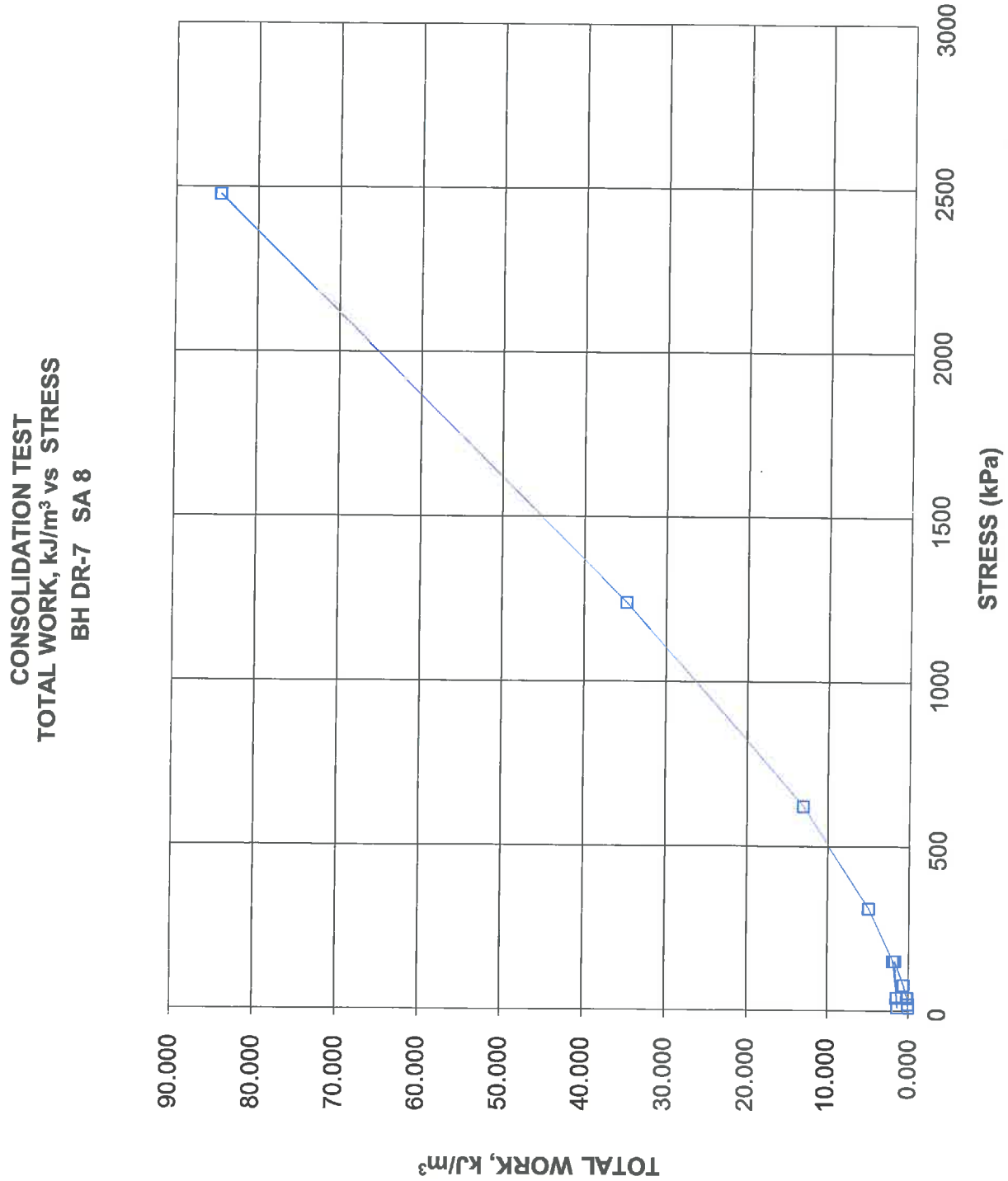
Prepared By: SJ

Golder Associates

Checked By: MG

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE 11D



Project No. 1786302 (1000)

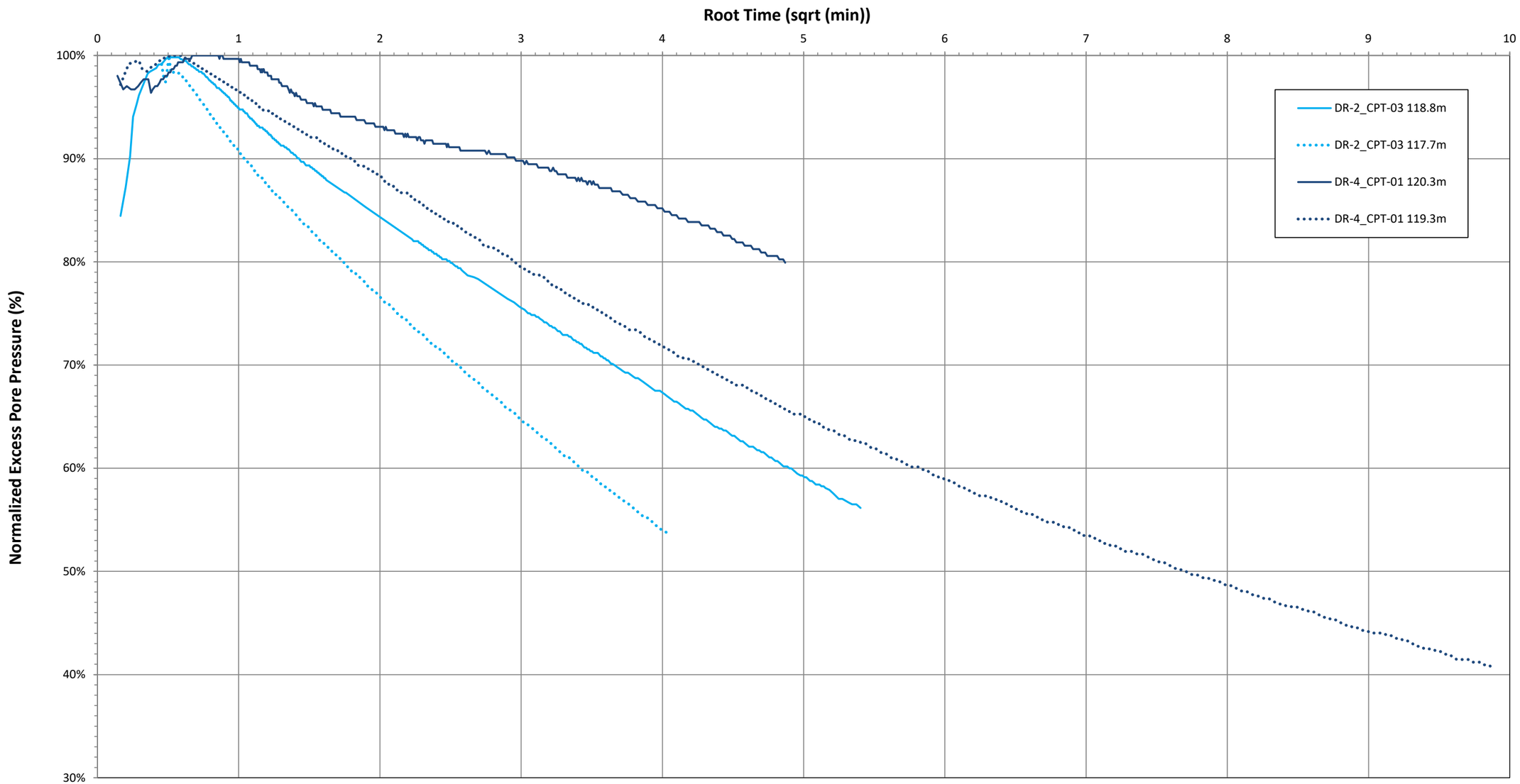
Prepared By: SJ

Golder Associates

Checked By: bb

SUMMARY PLOT OF PORE WATER DISSIPATION TESTING
Highway 401 Widening at Leslie Street
Don River East Branch Overpass

FIGURE C12



Date: 2020-06-04
Project No: 1786302 - 2000

Prepared By: DH
Checked By: ACK

Golder Associates Ltd.

GOLDER
MEMBER OF WSP

https://golderassociates.sharepoint.com/sites/20393g/Technical Work/Foundations/06 - Analysis/0.0 Material Parameters/[1786302 CPT Dissipation Tests 2020Feb26.xlsx]Dissipation Plot 1

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
ASTM D4767
SHEET 1 OF 4

FIGURE 13A

TEST STAGE	A	B	C
BOREHOLE NUMBER		DR-9	
SAMPLE		SA1	
DEPTH, m		13.72-14.18	
SPECIMEN DIAMETER, cm	5.08	5.07	4.94
SPECIMEN HEIGHT, cm	10.06	10.03	9.75
NATURAL WATER CONTENT, %	20.7	18.3	-
DRY DENSITY, Mg/m^3	1.77	1.85	-
WATER CONTENT AFTER SATURATION, %	22.1	20.2	-
CELL PRESSURE, σ_3, kPa	300.0	400.0	500.0
BACK PRESSURE, kPa	200.0	200.0	200.0
PORE PRESSURE PARAMETER "B"	0.96	0.95	-
EFFECTIVE CONSOLIDATION STRESS, σc, kPa	100.0	200.0	300.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	4.0	7.9	1.6
WATER CONTENT AFTER CONSOLIDATION, %	19.8	15.9	15.1
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	9.3	5.6	14.2
WATER CONTENT AFTER TEST, %	19.0	-	15.0
MAX. DEVIATOR STRESS, $(\sigma_1-\sigma_3)$, kPa	102.2	198.7	249.6
AXIAL STRAIN AT $(\sigma_1-\sigma_3)$ maximum, %	4.7	2.8	7.1
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1/σ'_3) maximum	3.7	3.3	3.4
DEVIATOR STRESS AT (σ'_1/σ'_3) maximum, kPa	101.9	198.7	248.4
AXIAL STRAIN AT (σ'_1/σ'_3) maximum, %	4.2	2.8	9.0
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1-\sigma_3)$ maximum	0.60	0.56	0.77
PORE PRESSURE PARAMETER, Af, AT (σ'_1/σ'_3) maximum	0.61	0.56	0.78
FILTER DRAINS USED, y/n	y	y	y

TEST NOTES: Multistage Test.
Effective consolidation stresses are assigned by the client.

FAILURE PLANE NUMBER	1.0	-	1.0
ANGLE OF FAILURE PLANE, DEGREES	60.0	-	60.0

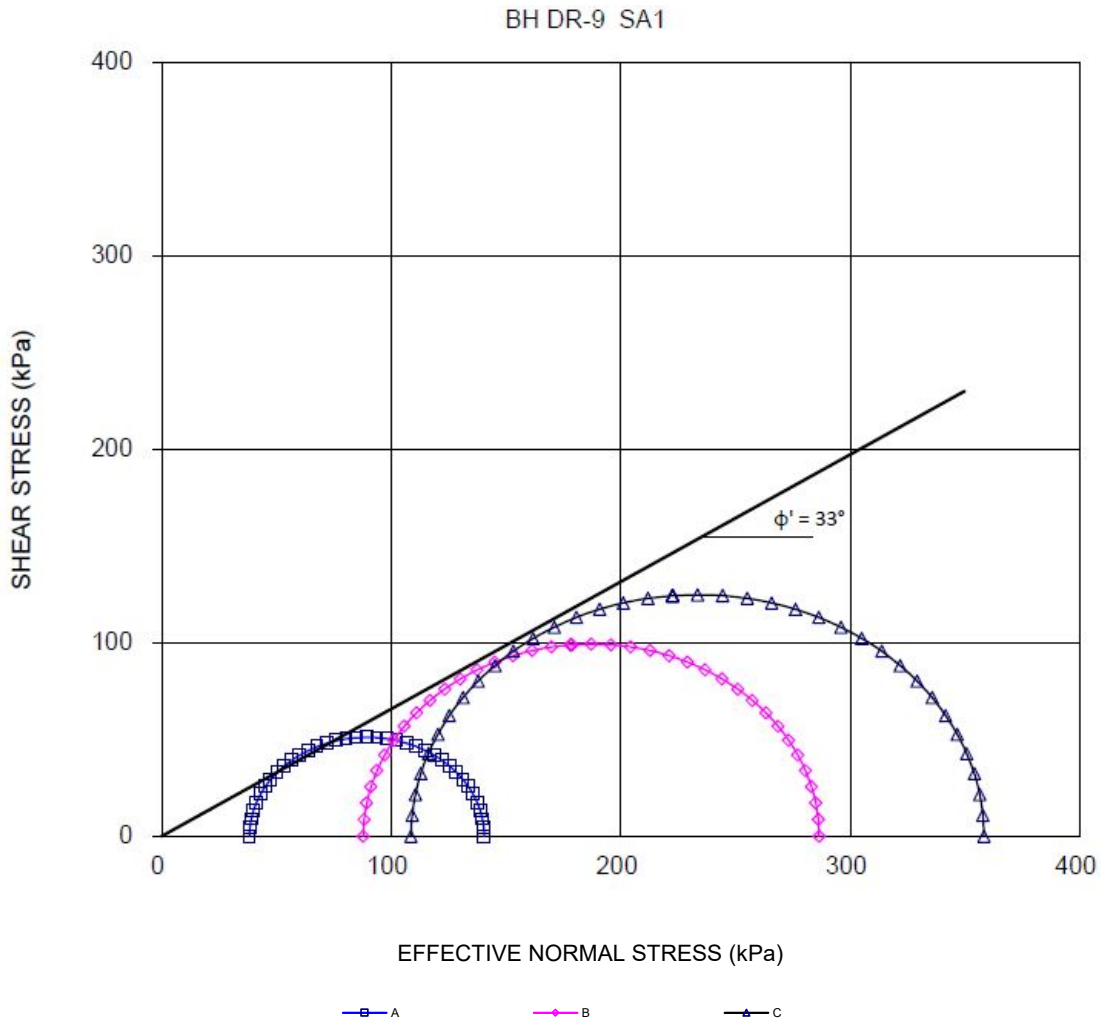
Date: 03/25/2020
Project No. 1786302

Golder Associates

Prepared By LH
Checked By: MH

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
ASTM D4767
SHEET 2 OF 4

FIGURE 13B



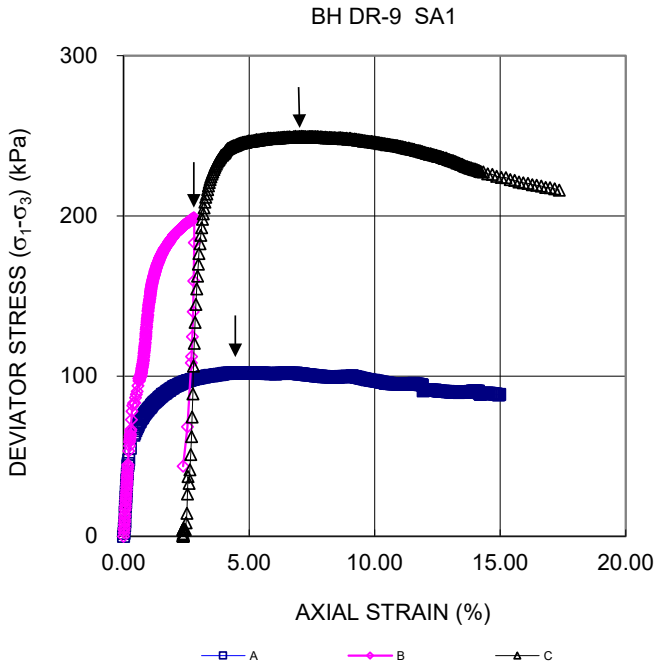
Date: 03/25/2020
Project No. 1786302

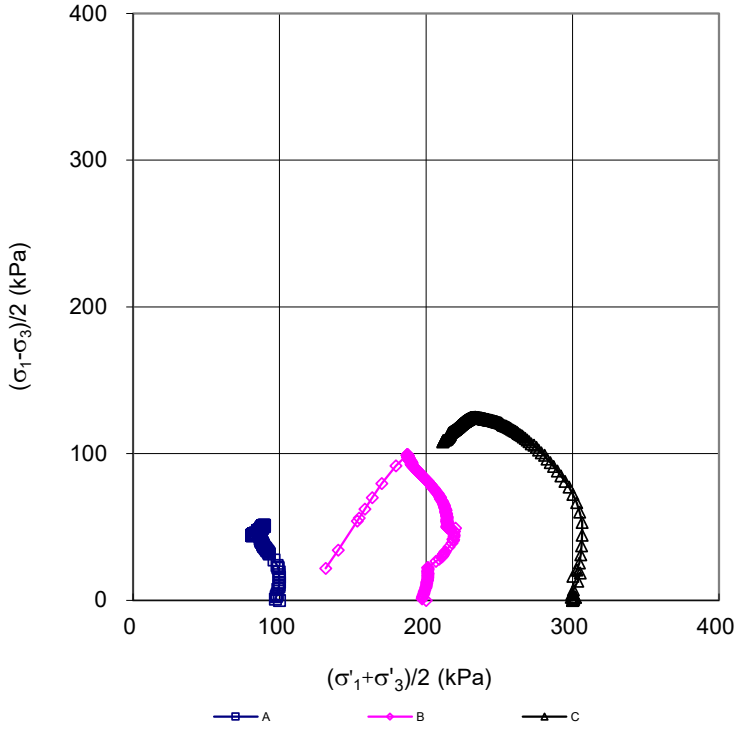
Golder Associates

Prepared By LH
Checked By:

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
ASTM D4767
SHEET 3 OF 4

FIGURE 13C





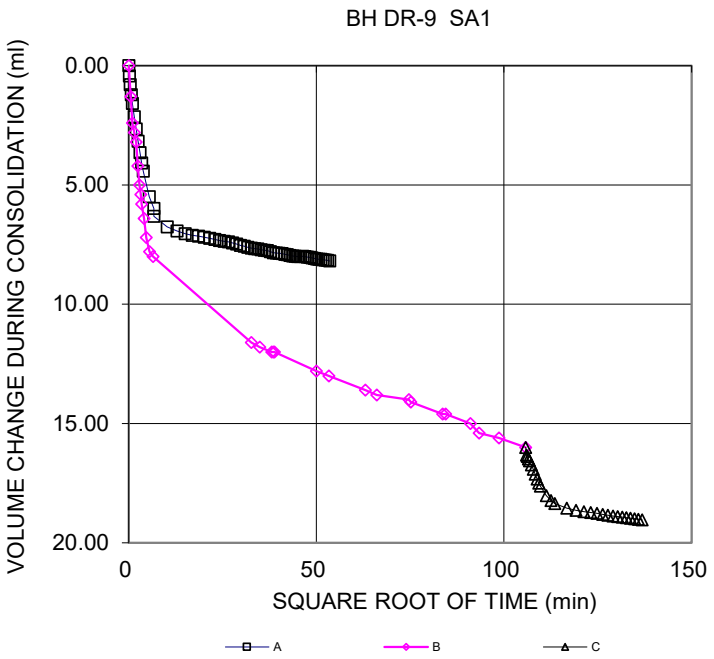
Date: 03/25/2020
Project No. 1786302

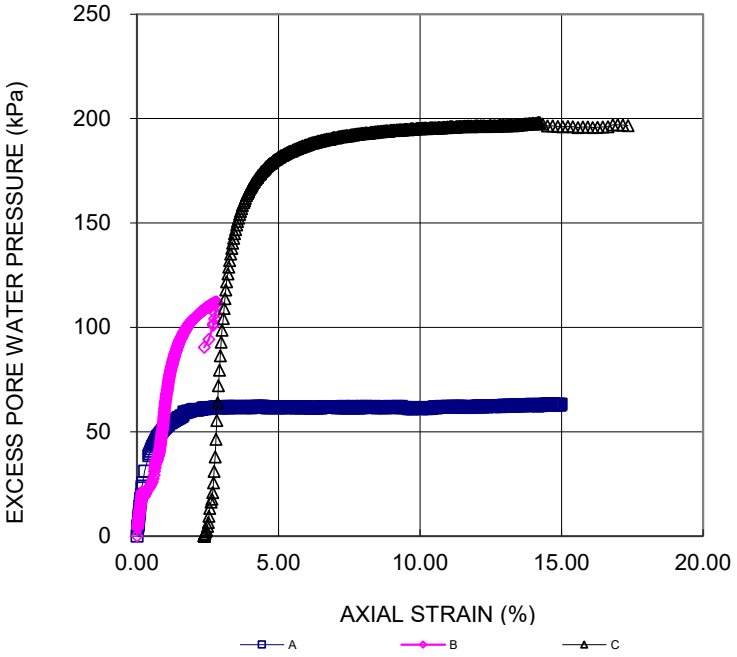
Golder Associates

Prepared By LH
Checked By:

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
ASTM D4767
SHEET 4 OF 4

FIGURE 13D





Date: 03/25/2020
Project No. 1786302

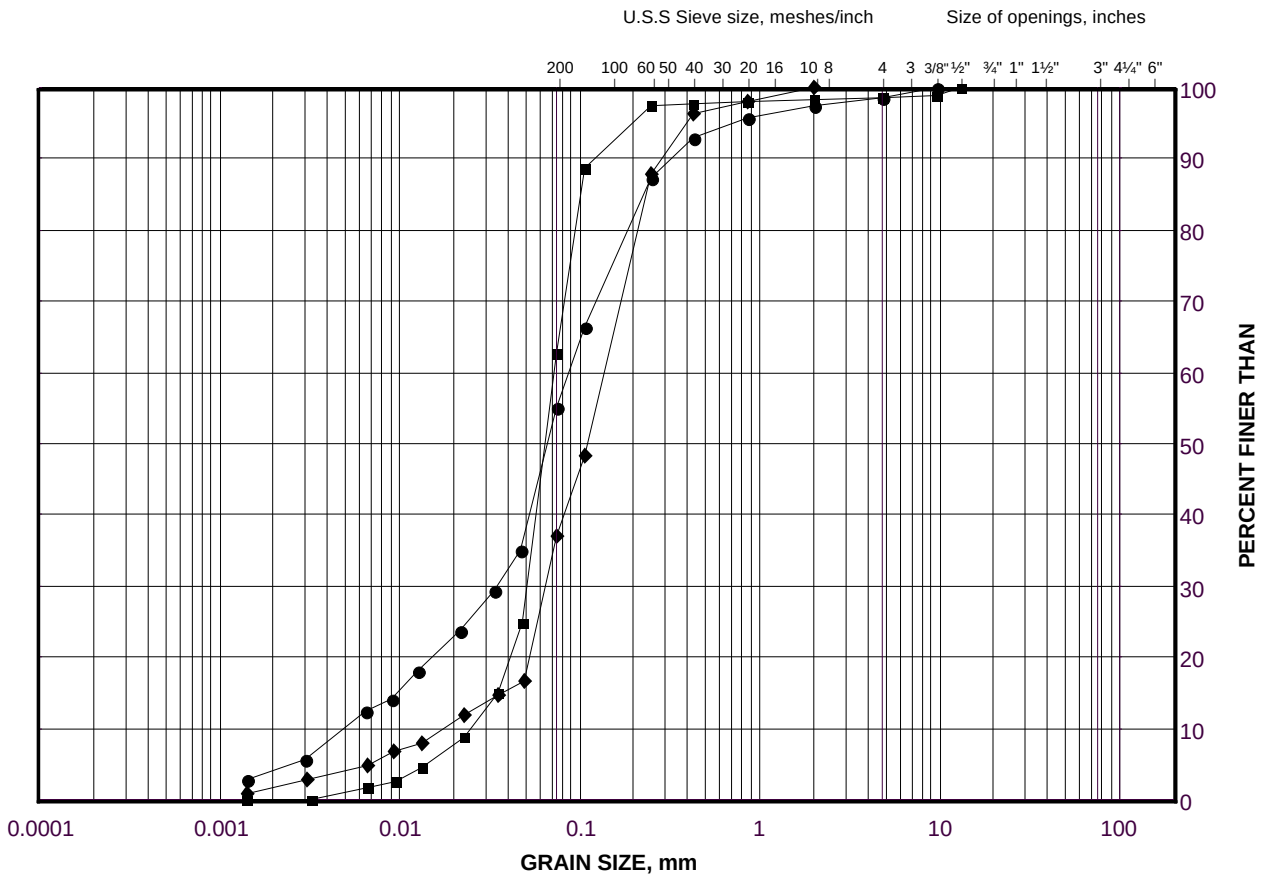
Golder Associates

Prepared By LH
Checked By: MM

GRAIN SIZE DISTRIBUTION

Sandy SILT (ML) to SILT (ML/SM) and Sand to SILTY SAND (SM) - Interlayer

FIGURE C14



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-7	10	118.3
■	DR-10	13B	118.8
◆	DR-6	7	122.4

Project Number: 1786302

Checked By: KN

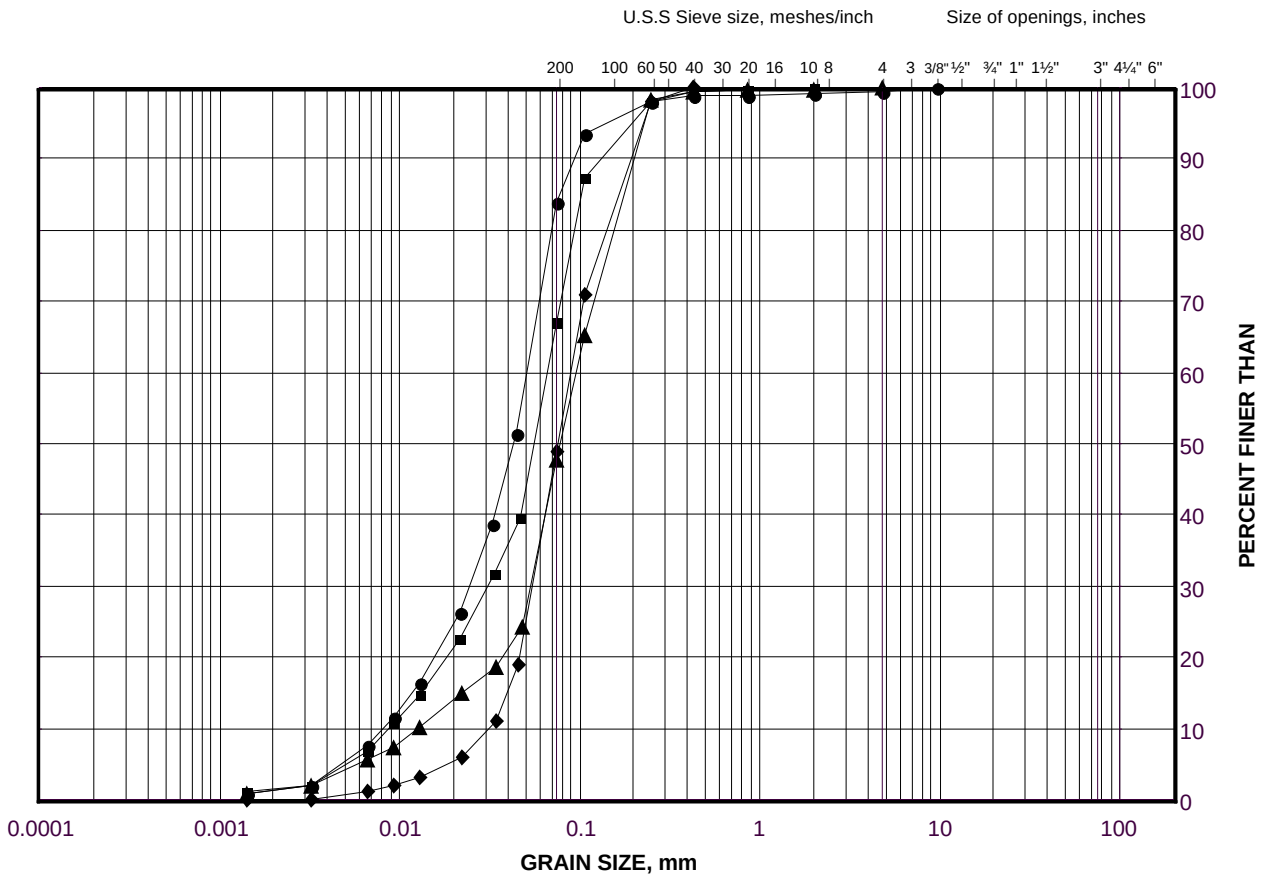
Golder Associates

Date: 06-Aug-20

GRAIN SIZE DISTRIBUTION

Sandy SILT (ML) to SILTY SAND (SM)

FIGURE C15



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-7	15	109.2
■	DR-6	15	109.4
◆	DR-9	15	112.9
▲	DR-3	20	110.7

Project Number: 1786302

Checked By: KN

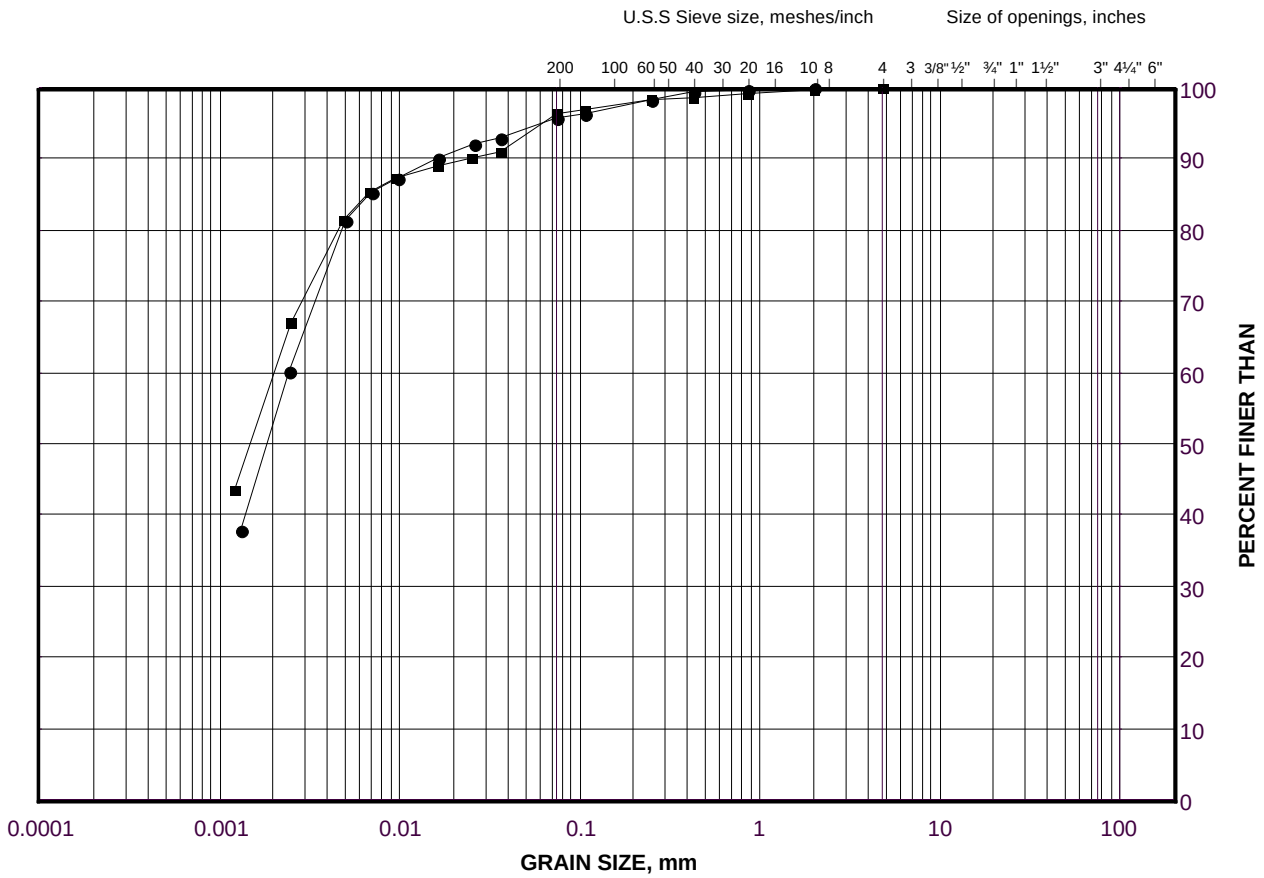
Golder Associates

Date: 06-Aug-20

GRAIN SIZE DISTRIBUTION

SILTY CLAY (CI) - Interlayer

FIGURE C16



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	DR-7	16	106.1
■	DR-9	17	106.8

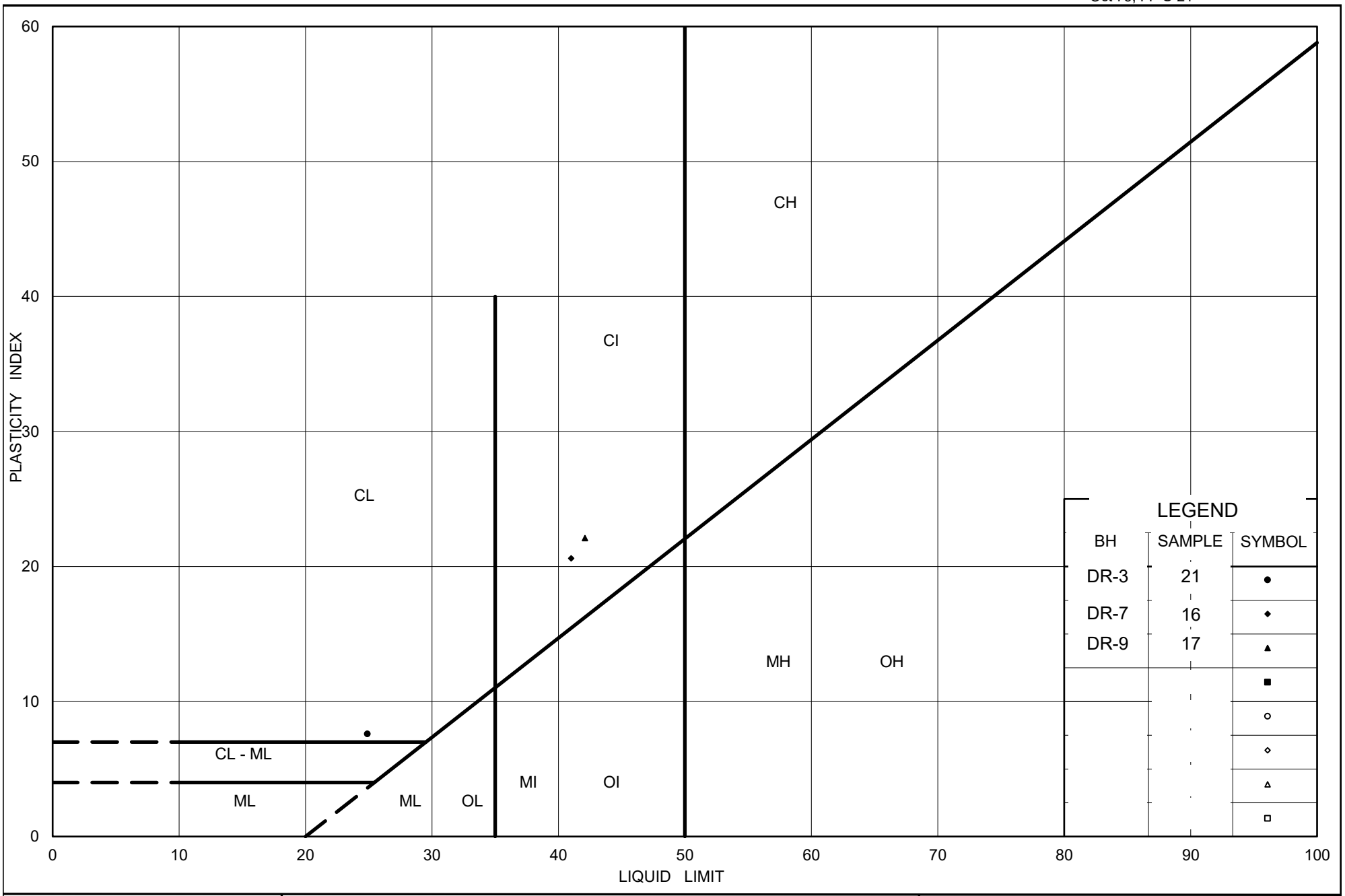
Project Number: 1786302

Checked By: KN

Golder Associates

Date: 06-Aug-20

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

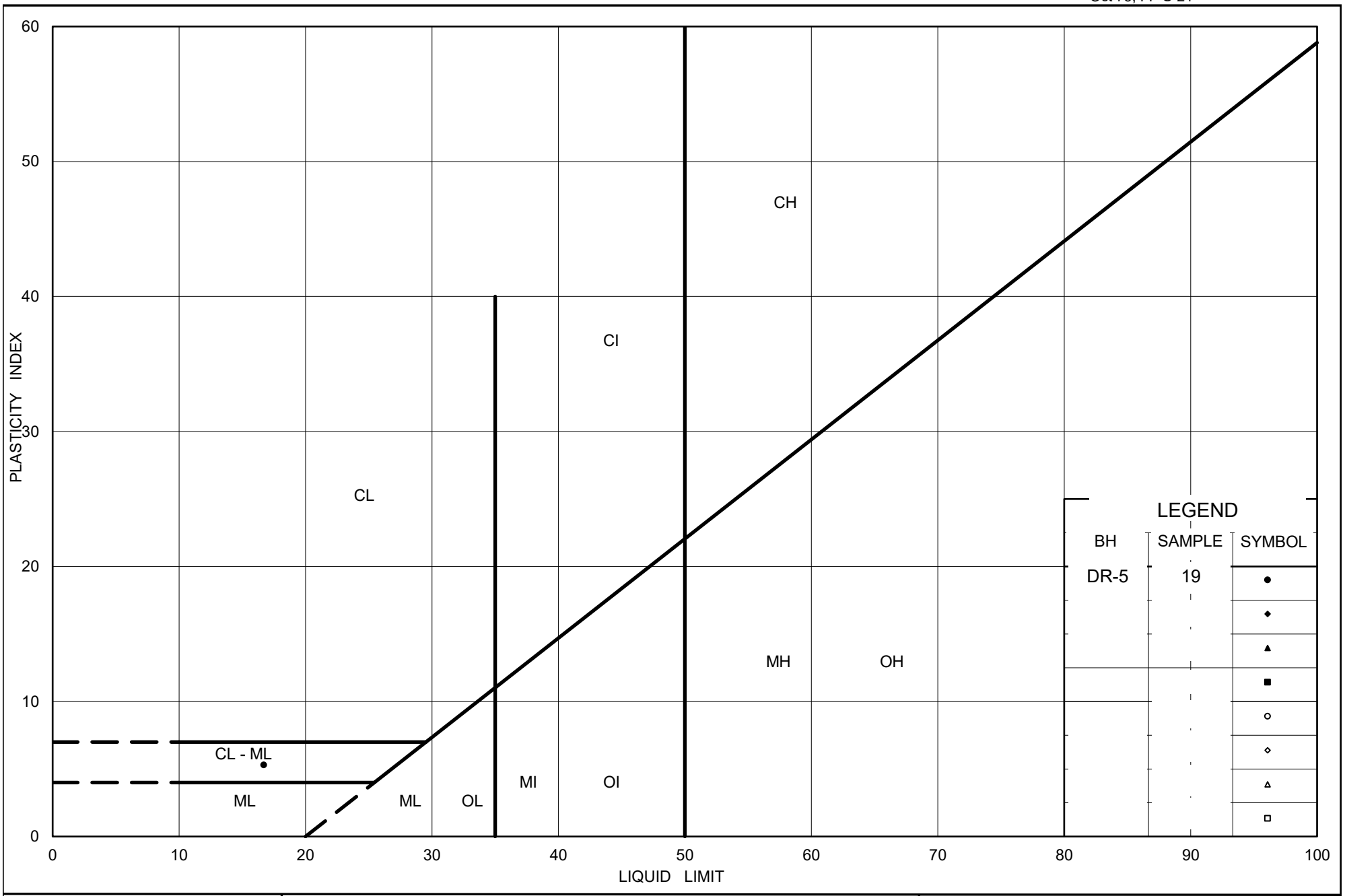
SILTY CLAY (CI) to CLAYEY SILT (CL) - Interlayer

Figure No. C17

Project No. 1786302

Checked By: KN

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Gravelly SAND (SW) (TILL)

Figure No. C18

Project No. 1786302

Checked By: KN

Your Project #: 1786302
Site#: HWY 401/LESLIE ST.
Site Location: HWY 401/LESLIE ST.
Your C.O.C. #: 724900-01-01

Attention: Katelyn Nero

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

Report Date: 2019/07/09
Report #: R5790260
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9I3294
Received: 2019/07/04, 14:47

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	2019/07/08	2019/07/08	CAM SOP-00463	SM 4500-Cl E m
Conductivity	3	2019/07/09	2019/07/09	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	3	2019/07/05	2019/07/05	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2019/07/04	2019/07/09	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	2019/07/08	2019/07/09	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, MDDELCC, 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.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

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

Your Project #: 1786302
Site#: HWY 401/LESLIE ST.
Site Location: HWY 401/LESLIE ST.
Your C.O.C. #: 724900-01-01

Attention: Katelyn Nero

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

Report Date: 2019/07/09
Report #: R5790260
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9I3294
Received: 2019/07/04, 14:47

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
==========
BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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BV Labs Job #: B9I3294
Report Date: 2019/07/09

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

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		KEI511	KEI512			KEI512			KEI513		
Sampling Date		2019/06/10	2019/06/09			2019/06/09			2019/06/19		
COC Number		724900-01-01	724900-01-01			724900-01-01			724900-01-01		
	UNITS	**DR3_SA11**	**DR9_SA17**	**RDL**	**QC Batch**	**DR9_SA17 Lab-Dup**	**RDL**	**QC Batch**	**NERS2_SA4B**	**RDL**	**QC Batch**
Calculated Parameters											
Resistivity	ohm-cm	2100	3200		6211426				1900		6211426
Inorganics											
Soluble (20:1) Chloride (Cl-)	ug/g	210	25	20	6216266				220	20	6216266
Conductivity	umho/cm	480	313	2	6218306				528	2	6218306
Available (CaCl2) pH	pH	7.61	7.84		6213015				7.75		6213015
Soluble (20:1) Sulphate (SO4)	ug/g	<20	140	20	6216267	130	20	6216267	<20	20	6216267

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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BV Labs Job #: B9I3294
Report Date: 2019/07/09

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

TEST SUMMARY

BV Labs ID: KEI511
Sample ID: DR3_SA11
Matrix: Soil

Collected: 2019/06/10
Shipped:
Received: 2019/07/04

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6216266	2019/07/08	2019/07/08	Deonarine Ramnarine
Conductivity	AT	6218306	2019/07/09	2019/07/09	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6213015	2019/07/05	2019/07/05	Surinder Rai
Resistivity of Soil		6211426	2019/07/09	2019/07/09	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	6216267	2019/07/08	2019/07/09	Deonarine Ramnarine

BV Labs ID: KEI512
Sample ID: DR9_SA17
Matrix: Soil

Collected: 2019/06/09
Shipped:
Received: 2019/07/04

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6216266	2019/07/08	2019/07/08	Deonarine Ramnarine
Conductivity	AT	6218306	2019/07/09	2019/07/09	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6213015	2019/07/05	2019/07/05	Surinder Rai
Resistivity of Soil		6211426	2019/07/09	2019/07/09	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	6216267	2019/07/08	2019/07/09	Deonarine Ramnarine

BV Labs ID: KEI512 Dup
Sample ID: DR9_SA17
Matrix: Soil

Collected: 2019/06/09
Shipped:
Received: 2019/07/04

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	6216267	2019/07/08	2019/07/09	Deonarine Ramnarine

BV Labs ID: KEI513
Sample ID: NERS2_SA4B
Matrix: Soil

Collected: 2019/06/19
Shipped:
Received: 2019/07/04

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6216266	2019/07/08	2019/07/08	Deonarine Ramnarine
Conductivity	AT	6218306	2019/07/09	2019/07/09	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6213015	2019/07/05	2019/07/05	Surinder Rai
Resistivity of Soil		6211426	2019/07/09	2019/07/09	Ewa Pranjic
Sulphate (20:1 Extract)	KONE/EC	6216267	2019/07/08	2019/07/09	Deonarine Ramnarine

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BV Labs Job #: B9I3294
Report Date: 2019/07/09

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

GENERAL COMMENTS

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

Package 1	8.0°C

Results relate only to the items tested.

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BV Labs Job #: B9I3294
Report Date: 2019/07/09

QUALITY ASSURANCE REPORT

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

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6213015	Available (CaCl2) pH	2019/07/05			100	97 - 103			0.40	N/A
6216266	Soluble (20:1) Chloride (Cl-)	2019/07/08	NC	70 - 130	101	70 - 130	<20	ug/g	3.4	35
6216267	Soluble (20:1) Sulphate (SO4)	2019/07/09	NC	70 - 130	98	70 - 130	<20	ug/g	8.4	35
6218306	Conductivity	2019/07/09			104	90 - 110	<2	umho/cm	3.8	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)

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BV Labs Job #: B9I3294
Report Date: 2019/07/09

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

VALIDATION SIGNATURE PAGE

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

Ewa Pranjic, M.Sc., C.Chem, Scientific Specialist

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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Page of

CHAIN OF CUSTODY RECORD

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name:	#1326 Golder Associates Ltd	Company Name:	Golder Associates	Quotation #:	B80683	BV Labs Job #:	Bottle Order #: 724900
Attention:	Accounts Payable	Attention:	Katelyn Nero	P.O. #:			
Address:	6925 Century Ave Suite 100 Mississauga ON L5N 7K2	Address:		Project:	1786302	COC #:	Project Manager:
Tel:	(905) 567-4444 Fax: (905) 567-6561	Tel:	Fax:	Project Name:	HWY401/Leslie St.	C#724900-01-01	Ema Gitej
Email:	AP_CustomerService@golder.com	Email:	katelyn_nero@golder.com & Alysha_Kobylinski@golder.com	Site #:	HWY401/Leslie St.		
				Sampled By:	DH / AM		

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
☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___	☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality _____ ☐ PWQO ☐ Other _____	

Include Criteria on Certificate of Analysis (Y/N)? _____

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Soil Corrosivity Package									# of Bottles	Comments
1		DR3_SA11	2019/6/10	AM	SOIL		X									1	
2		DR9_SA17	2019/6/9	AM	SOIL		X									1	
3		NERS2_SA4B	2019/6/19	PM	SOIL		X									1	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

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.

Job Specific Rush TAT (if applies to entire submission) ☐
Date Required: _____ Time Required: _____
Rush Confirmation Number: _____ *(call lab for #)*

04-Jul-19 14:47
Ema Gitej
B9I3294
KFL ENV-790

* 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: Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
Katie Nero / Katie Nero	19/07/04	2:45	Dipika Singh / DIPIKA SINGH	2019/07/04	14:47			10/8/6	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

Bureau Veritas Canada (2019) Inc.

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 Extracted	Date Analyzed	Laboratory Method	Analytical Method
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.

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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

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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

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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

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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.

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

QUALITY ASSURANCE REPORT

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

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6460218	Available (CaCl2) 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 (SO4)	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)

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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

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.

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Page of

CHAIN OF CUSTODY RECORD

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:		Laboratory Use Only:	
Company Name:	#2292 Golder Associates Ltd	Company Name:	Golder	Quotation #:	B80683	BV Labs Job #:	Bottle Order #:
Attention:	Accounts Payable	Attention:	~~Chris Fons~~ Katelyn Nero	P.O. #:			729512
Address:	100 Scotia Crt	Address:		Project:	1786302		
	Whitby ON L1N 8Y6			Project Name:	HWY401 / Leslie	COC #:	Project Manager:
Tel:	(905) 723-2727 Fax: (905) 723-2182	Tel:	~~(905) 845-5100 Ext 280~~ Fax:	Site #:		C#729512-02-01	Ema Gitej
Email:	AP_CustomerService@golder.com	Email:	~~Chris_Fons@golder.com~~ knero@golder.com	Sampled By:	RM / DH		

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
☐ Table 1 ☐ Res/Park ☐ Medium/Fine	☐ CCME ☐ Sanitary Sewer Bylaw	
☐ Table 2 ☐ Ind/Comm ☐ Coarse	☐ Reg 558 ☐ Storm Sewer Bylaw	
☐ Table 3 ☐ Agri/Other ☐ For RSC	☐ MISA Municipality _____	
☐ Table ___	☐ PWQO	
	☐ Other _____	

Include Criteria on Certificate of Analysis (Y/N)? _____

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

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.

Job Specific Rush TAT (if applies to entire submission) ☐
Date Required: _____ Time Required: _____
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	SOIL CORROSIVITY PACKAGE								# of Bottles	Comments
1		DR6_SA5	July 23/19	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																	

20-Nov-19 16:33
Ema Gitej
B9W7428
JCC ENV-571

* 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				
Kate Nero / Katelyn	19/11/20		Jun - COLENE CURITAO	2019/11/20	16:33		Time Sensitive	Temperature (°C) on Receipt: 9/9/10	Custody Seal Present	Yes	No ✓
									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

Bureau Veritas Canada (2019) Inc.

Geomechanica Inc.
Suite 900 – 390 Bay St.
Toronto Ontario
Canada M5H 2Y2

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

Disclaimer: This report was prepared by Geomechanica Inc. for Golder Associates Ltd.. The material herein reflects Geomechanica Inc.'s best judgment given the information available at the time of preparation. Any use which a third party makes of this report, any reliance on or decision to be made based on it, are the responsibility of such third parties. Geomechanica Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

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.

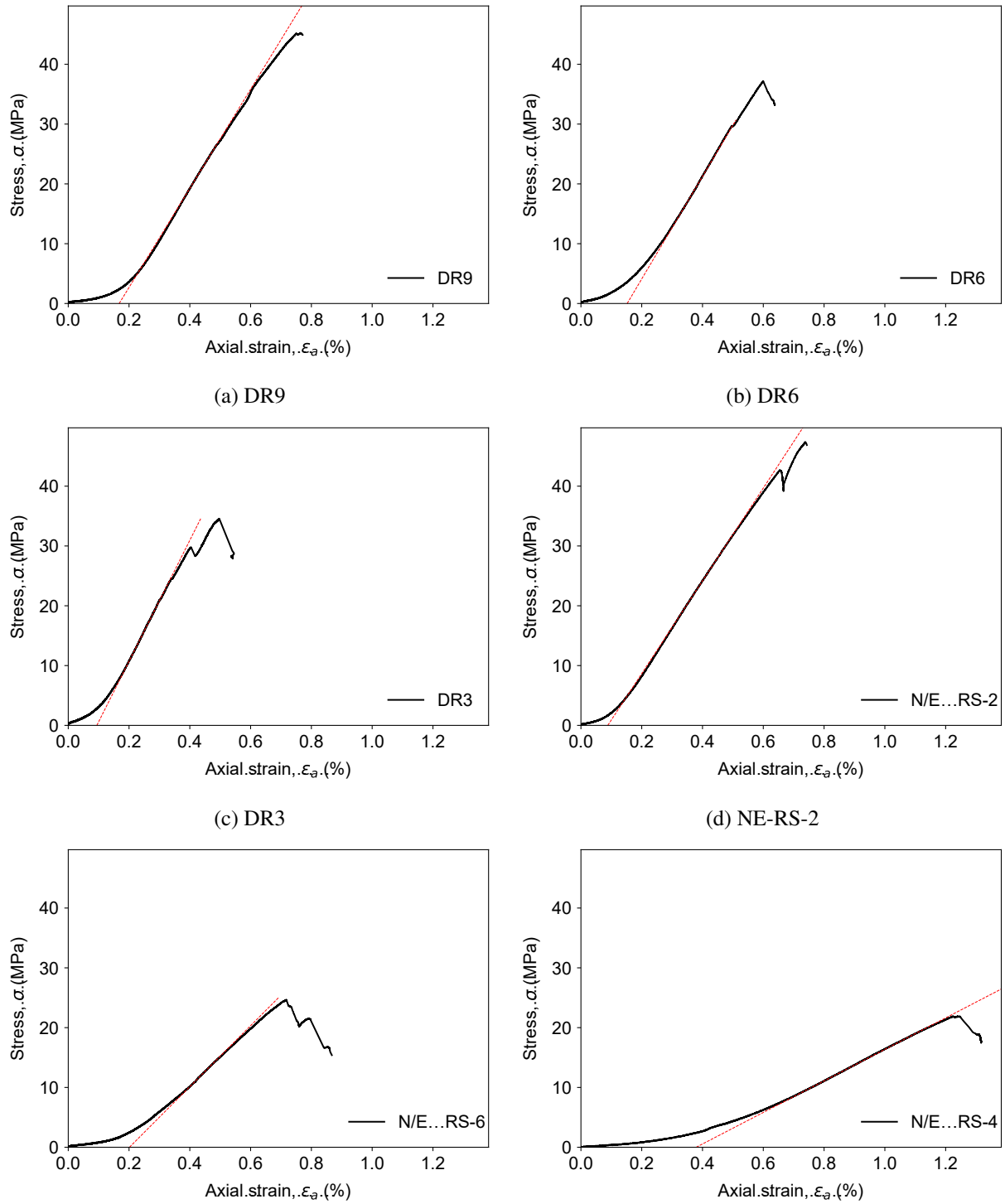
Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
DR9	37.99 - 38.21	2.616	45.2	8.3	Georgian Bay Formation - Shale	1
DR6	30.23 - 30.42	2.627	37.2	8.6	Georgian Bay Formation - Shale	1
DR3	40.84 - 41.02	2.619	34.6	10.1	Georgian Bay Formation - Shale 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.



(a) DR9

(b) DR6

(c) DR3

(d) NE-RS-2

(e) NE-RS-6

(f) NE-RS-4

Figure 2: Measured stress-strain curves.

Appendices

Specimen sheets

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

GEOMECHANICA

Uniaxial Compression Test

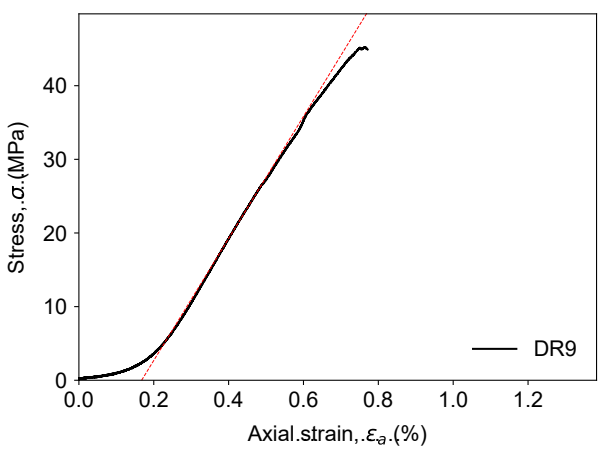
Client	Golder Associates Ltd.	**Project**	1786302
Sample	DR9	**Depth**	37.99 - 38.21

Specimen parameters	
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

[a] Additional specimen measurement/details provides 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: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2019-11-25

GEOMECHANICA

Uniaxial Compression Test

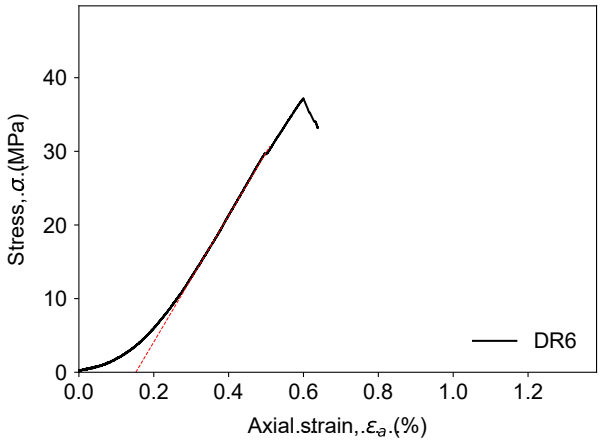
Client	Golder Associates Ltd.	**Project**	1786302
Sample	DR6	**Depth**	30.23 - 30.42

Specimen parameters	
Diameter (mm) [a]	62.67
Length (mm) [a]	126.01
Bulk density ρ (g/cm^3)	2.627
UCS (MPa)	37.2
Young's modulus E (GPa) [b]	8.6
Lithology	Georgian Bay Formation Shale
Failure description [c]	1

[a] Additional specimen measurement/details provides 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: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2019-11-25

Uniaxial Compression Test

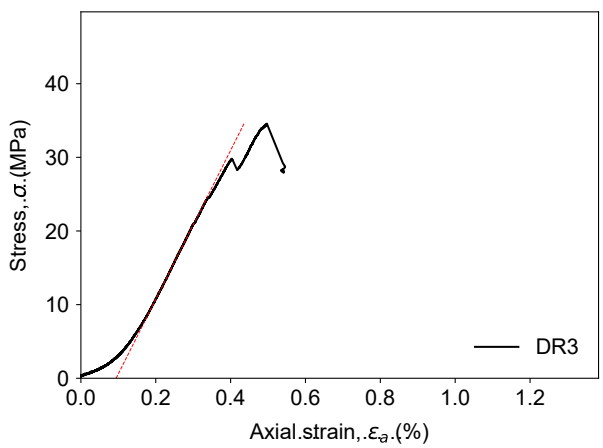
Client	Golder Associates Ltd.	**Project**	1786302
Sample	DR3	**Depth**	40.84 - 41.02

Specimen parameters	
Diameter (mm) [a]	62.69
Length (mm) [a]	126.28
Bulk density ρ (g/cm^3)	2.619
UCS (MPa)	34.6
Young's modulus E (GPa) [b]	10.1
Lithology	Georgian Bay Formation Shale and Siltstone
Failure description [c]	1

[a] Additional specimen measurement/details provides 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: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2019-11-25

Uniaxial Compression Test

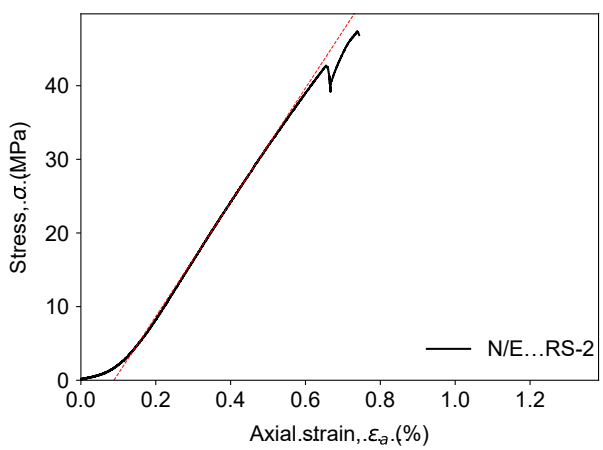
Client	Golder Associates Ltd.	**Project**	1786302
Sample	N/E RS-2	**Depth**	40.62 - 40.79

Specimen parameters	
Diameter (mm) [a]	62.67
Length (mm) [a]	125.59
Bulk density ρ (g/cm^3)	2.615
UCS (MPa)	47.4
Young's modulus E (GPa) [b]	7.8
Lithology	Georgian Bay Formation Shale and Siltstone
Failure description [c]	1

[a] Additional specimen measurement/details provides 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: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2019-11-25

Uniaxial Compression Test

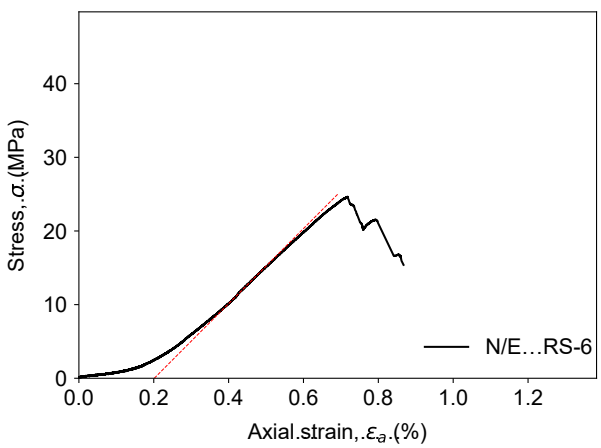
Client	Golder Associates Ltd.	**Project**	1786302
Sample	N/E RS-6	**Depth**	39.35 - 39.51

Specimen parameters	
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

[a] Additional specimen measurement/details provides 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: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2019-11-25

GEOMECHANICA

Uniaxial Compression Test

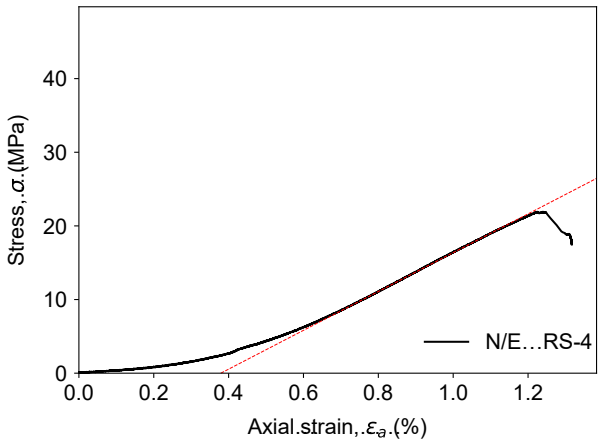
Client	Golder Associates Ltd.	**Project**	1786302
Sample	N/E RS-4	**Depth**	37.59 - 37.69

Specimen parameters	
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

[a] Additional specimen measurement/details provides 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: [1] Axial splitting failure; [2] Length:Diameter ratio less than 2;

Prior to testing

After testing



Remarks:

Performed by	BSAT	**Date**	2019-11-25

APPENDIX D

Non-Standard Special Provisions

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

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

Steel Structures Painting Council (SSPC)

SP10/NACE No.2-Jan. 1, 2001	Near-White Blast Cleaning

International Organization for Standardization/International Electrotechnical Commission (ISO/IEC)

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

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 threated steel of 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 EFFC 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 (^{o}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 (^{o}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 (ºC) 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 (±13mm) 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 ± 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 cm^2, 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 cm^2 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.
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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 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.

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

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	$m^2.^oC/W$ (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 90º 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 DRB-03, DRB-04, DR-6 and DR-9. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, pile installation, and installation of drilled shaft piles (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.

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.

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 Contact 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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